3.4. Water Sensitive Urban Design and Infiltration

The water sensitive urban design (WSUD) and infiltration option investigates a number of environmentally focused technologies that are used to capture runoff, absorb the energy of high runoff volumes and significantly improve water quality characteristics.

3.4.1. Bio-retention Basins

A bio-retention basin consists of a basin or trench area that is filled with porous materials and vegetation to collect and filter stormwater. These treatments are often paired with an overflow basin, making them an appropriate treatment for larger water volumes. These are a popular WSUD treatment due to their aesthetic appeal and promotion of biodiversity (Queensland Government, 2007).

3.4.1.1. Bio-retention Basin Components

Bio-retention basins usually consist of a vegetated layer, filtration layer, transition layer, drainage layer, perforated pipe system and bypass system. A typical cross section is shown in Figure 30, which identifies the layers within the bio-retention system and their effective depths.

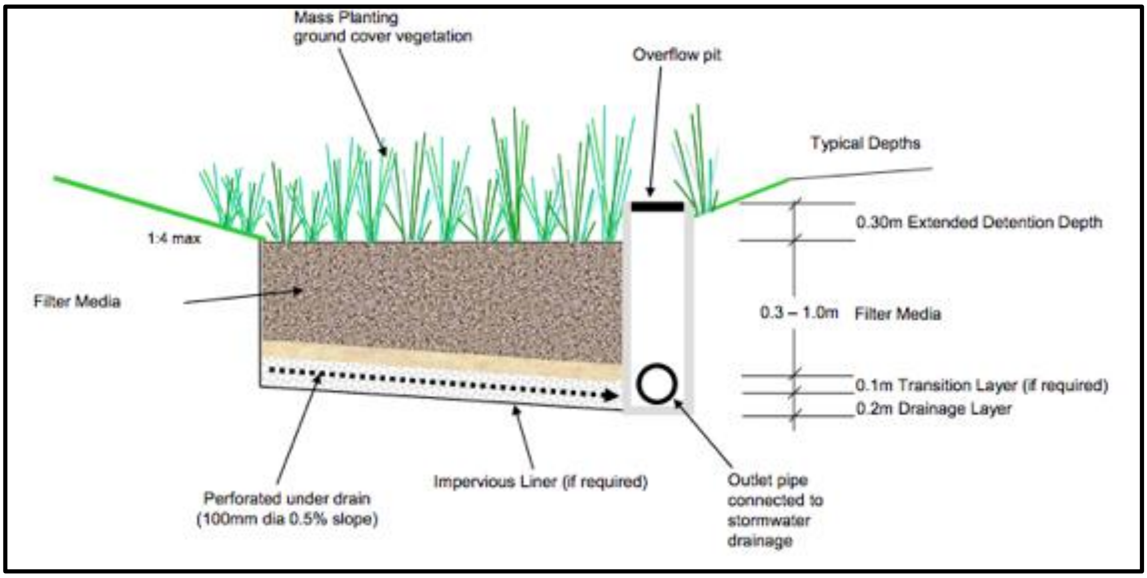


Figure 61: Bio-Retention System Layers (Brisbane City Council, 2005)

3.4.1.1.1. Vegetation Layer

A typical bio-retention basins top layer consists of a vegetated surface, planted into the lower layers, which acts as the detention layer. It is within this layer that water is captured, and ponds, during a rainfall event. As well as providing an appropriate area for the water to pond, the top layer also provides the first stage of filtration for the stormwater. The biofilm, which is present on the roots of the vegetation, is known to decrease the concentration of some pollutants.

It is important to carefully select vegetation that will thrive in following criteria:

- Soil type and climate
- Water conditions and filter depth
- Wet/dry periods
- Dense planting with a range of different species

The vegetation must also display the following characteristics:

- Be aesthetically pleasing

- Efficient at pollutant removal
- Low maintenance
- Large diameter roots

As well as providing the benefits listed above, the planted vegetation also reduces the effects of erosion and the roots prevent any clogging and transport oxygen to the lower layers, sustaining the basin. (Brisbane City Council, 2005 & Boskovic, 2008)

3.4.1.1.2. Filtration Layer

After passing though the vegetated layer, the water passes through a filtration layer. This layer is constructed from a sandy loam. The sandy loam is appropriate due to its hydraulic conductivity properties (100-300mm/hr for temperate climates) and its capacity to filter stormwater pollutants through physical and chemical filtration. The hydraulic conductivity characteristics also mean that a sufficient water supply is retained to promote vegetation growth. (Monash University, 2010, Brisbane City Council, 2005 & Boskovic 2008)

3.4.1.1.3. Transition Layer

Following this, the water flows to a transition layer, usually consisting of a permeable geotextile fabric. The purpose of this layer is to prevent filter media from the above layer flowing into the drainage layer or perforated pipes.

This layer is not required in every design and is dependent on the size of the material used in the drainage layer. If the drainage layer is constructed of fine gravel then a transition layer is recommended due to the difference in particle size. It should be noted that coarse sand is also considered as an acceptable material for this layer. (Brisbane City Council, 2005 & Boskovic, 2008)

3.4.1.1.4. Drainage Layer

The drainage layer is the bottom layer of the system and usually consists of 1mm sand or 2-5mm gravel. The design of the drainage layer is utilised in a way that the draining material surrounds the perforated pipes. The main functions of this layer are to transport the filtered water into the perforated underdrains and remove any suspended solids that may still exist in the water, preventing the perforations in the pipe from blocking.

It is important that this material does not contain any silt or clay, to ensure hydraulic conductivity requirements are maintained. The material within this layer should also be washed before use to remove fines. (Brisbane City Council, 2005 & Boskovic, 2008)

3.4.1.1.5. Perforated Underdrains

The perforated underdrains are usually placed under, or within, the drainage layer. These pipes collect the water, which has been filtered, and transport the water to an area to be discharged. The discharge locations include existing waterways or into water storage systems, if the water is considered for reuse. In some cases, this layer is not included and the water is left to infiltrate into the surrounding soils and eventually recharge groundwater. (Brisbane City Council, 2005 & Boskovic, 2008)

3.4.1.1.6. Bypass System

Most bio-retention systems also utilise a bypass, or overflow, system to effectively manage rainfall in a major rainfall event, minimise flooding and assure that scouring of the filter media or erosion of soils does not occur.

The bypass system is typically constructed using a grated, or outlet, pit or side entry pit that connects to the existing conventional stormwater system. (Boskovic 2008)

3.4.1.2. Advantages

Employing a bio-retention basin as a WSUD treatment provides a range of benefits, including:

- Promotion of biodiversity
- Improved quality of stormwater
- Aesthetically pleasing
- Does not require a large amount of space to be implemented

(Queensland Government, 2007 & Susdrain, 2012)

3.4.1.3. Disadvantages/Limitations

Though these are widely used and present many benefits, disadvantages and limitations do exist. These are:

- Require maintenance
- Clogging or blockages may occur
- Cannot be implemented on land which has a steep slope

(Susdrain 2012)

3.4.1.4. Impact to Water Quality

A main feature of this treatment is its efficiency in removing pollutants, which is summarised in Section 3.4.1.2.

3.4.1.5. Design Calculations/Considerations

The following calculations were performed to check what size bio-retention basin would be required to efficiently process the estimated water volume, as previously calculated.

3.4.1.5.1. Size of Bio-Retention Basin

The water flow run off for the bio-retention basin was calculated assuming the basin would utilise runoff from the road and the pavement (a combined area of 16220m^2). The total runoff for these areas was calculated using a 5 year ARI and a corresponding flow rate of 0.322m^3/s.

The total size of the required bio-retention basin was calculated based on the expected pollutant reductions shown in Figure 62, Figure 63 and Figure 64 (Melbourne Water, 2005).

The treatment was designed to meet water quality targets of 80%, 45% and 45% reductions of TSS, TP and TN respectively, compared to development without any water sensitive urban design applied (Department of Water: Government of Western Australia 2007, Melbourne Water 2007, Local Government Association of South Australia 2009). Graphs in Figure 62 to Figure 64 were used to achieve these pollutant targets. Bio-retention basins with a total size of 0.9% of the total impervious area should be designed. This equated to an approximate area of 150m^2.

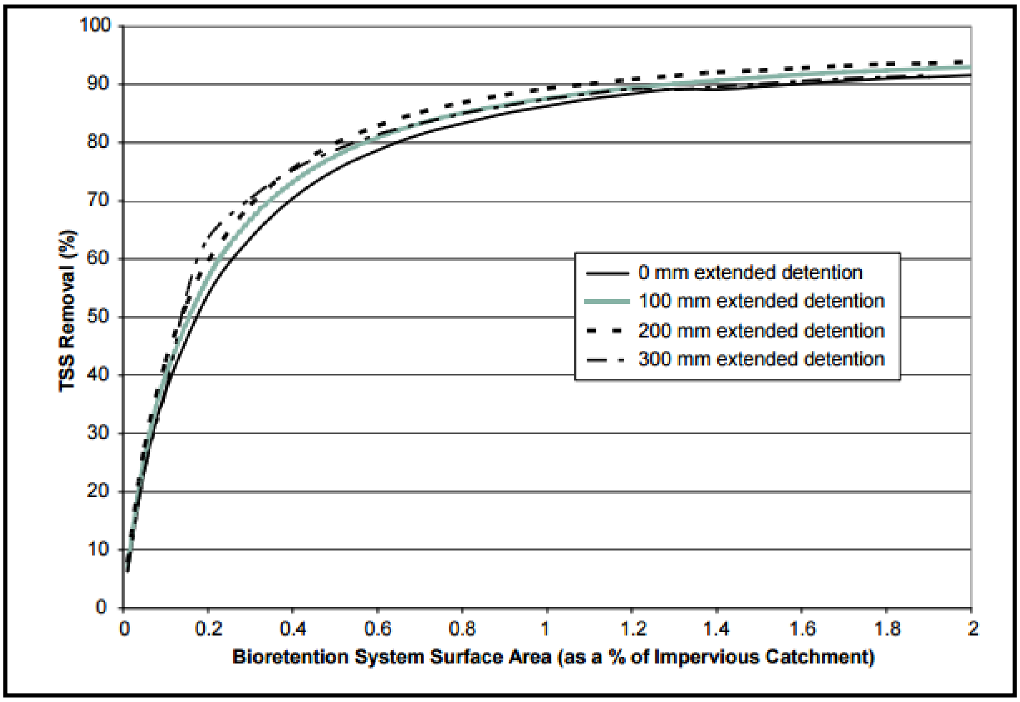


Figure 62 Bio-retention system TSS removal performance (Department of water, Government of Western Australia 2007).

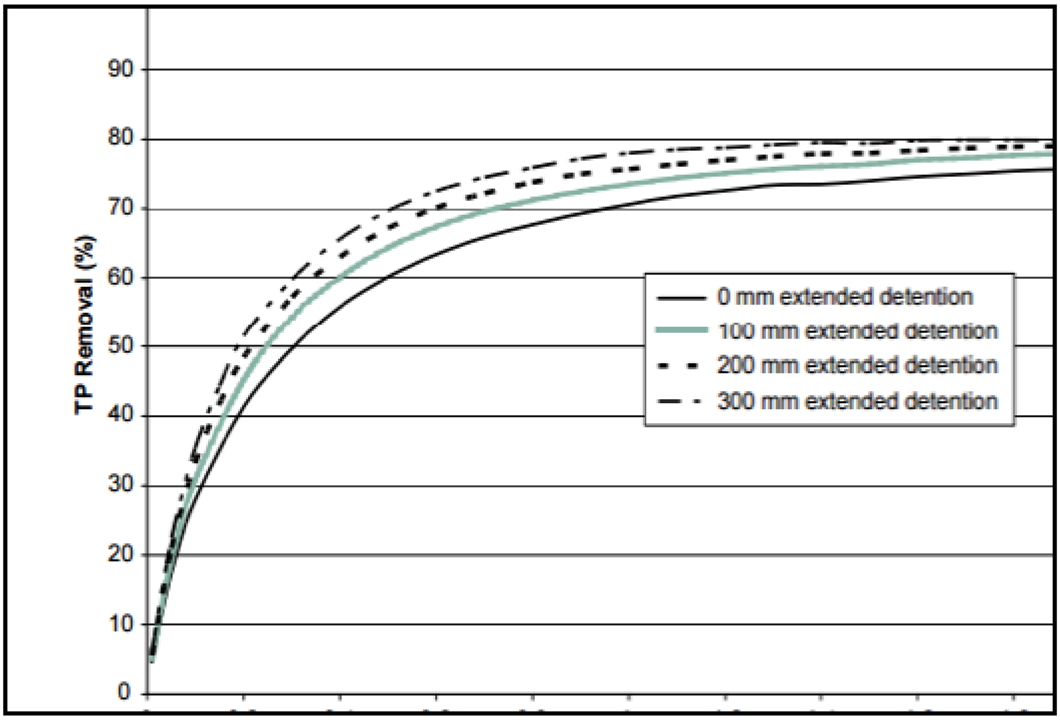


Figure 63 Bio-retention system TP removal performance (Department of water, Government of Western Australia 2007).

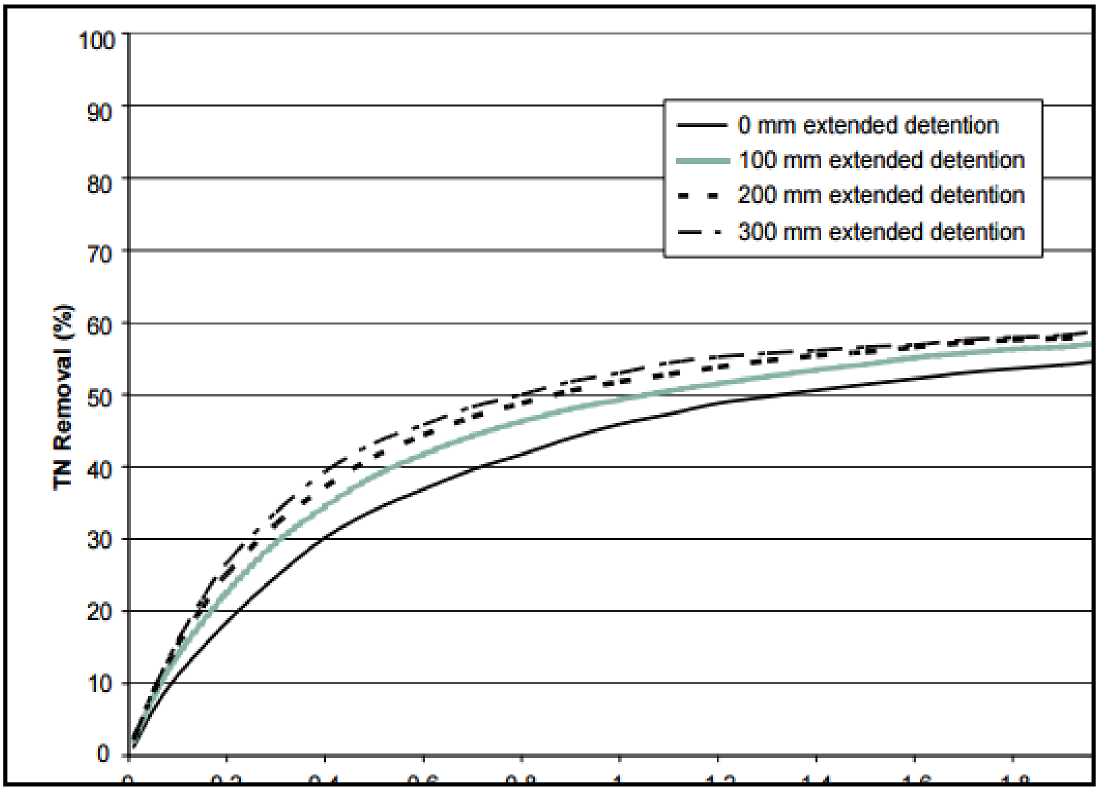


Figure 64: Bio-retention system TN removal performance (Department of Water, Government of Western Australia 2007).

In the catchment area the only available area to place a bio-retention basins are shown in Figure 65 and Figure 66. The area available in Figure 65 is approximately 100m^2; whereas the area in Figure 66 is much bigger, approximately 400m^2.

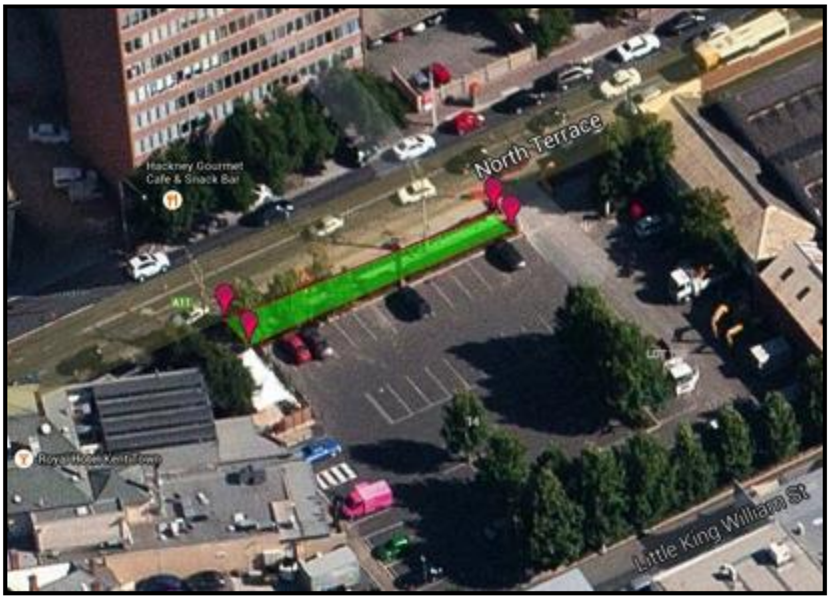


Figure 65: Area that could be used for bio-retention basin (lot 103)

Figure 66: Area that could be used for bio-retention basin (in front of St Peter's College)

There are no other places that can be used for a bio retention basin, due to large space requirements. The area highlighted in Figure 66 is less preferable as that shown in Figure 65, as it is on a high point within the catchment area. Yet, the location shown in Figure 65 will not meet design requirements.

A combination of the 2 locations could be used to achieve the water treatment requirements. The design of a cross section of the bio-retention basin will be as shown in Figure 67.

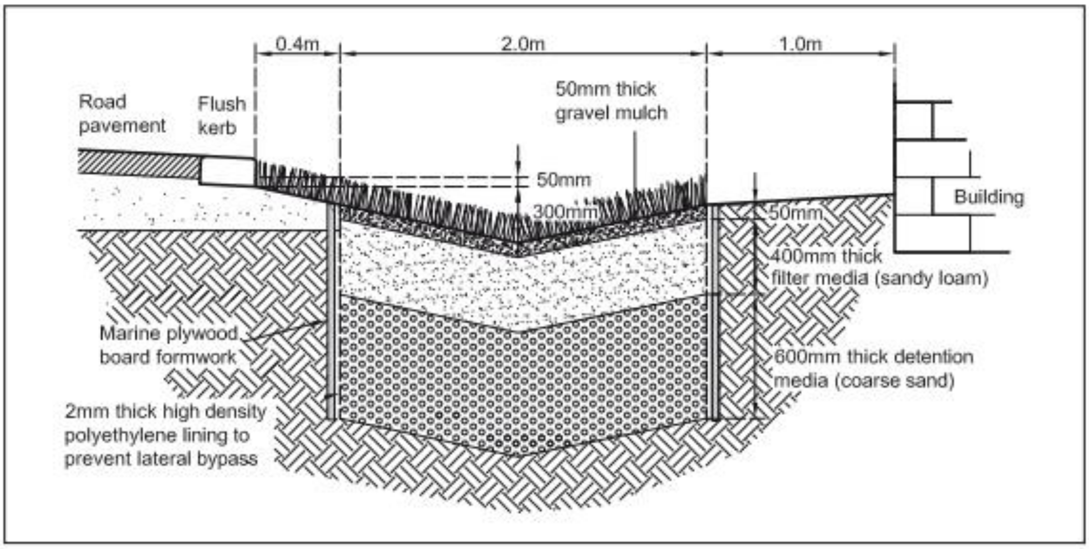


Figure 67: Typical liner arrangement for bio-retention system (Department of Water, Government of Western Australia 2007).

Further details about the bio-retention properties like maximum infiltration rate (shown in the equation below), inlet details, vegetation scour velocity check, size of slotted collection pipe, soil media specification, etc will be defined in the detailed design.

$$Q_{max} = k \times L \times W_{base} \times \frac{h_{max} + d}{d}$$

Where:

k = the conductivity of the soil filter (m/s)
W=the average width of the ponded cross section above the sand filter (m)
L= the length of the bioretention zone (m)
h_{max}= depth of podding above the soil filter (m)
d = depth of filter media (m)

3.4.1.5.2. Considerations to be Made Regarding Bio-Retention Basin Location

In terms of location and implementation of a bio-retention basin within the specified catchment, there are a number of design options that could be utilised. These are summarised in Table 23 below.

Options	Bio retention Basin taking part of the road	Bio-retention basin using part of the foot path	Bio-retention basin where current vegetation is present	Carefully chosen locations where the footpath is wide enough
Illustrations	 (Melbourne Water,2013)	 (Southwest Urban Hydrology, 2015)	 (Strand Associates, 2011)	
Discussions	• Taking part of the road id not affordable as North Terrace is a very road with high traffic volumes. • Therefore, this option is not feasible.	• The foot path in the area is 2.5m average. • Minimum wide of a detention basin is 2 m (Melbourne Water,2005) • This mean only 0.5 m of foot path will be left which is not as it will not accessible to disabled pedestrians	• All vegetation areas in the project area have trees. • Replacing these areas with bio-retention basin is feasible • However, it will not cause a big change in runoff rates as these are placed in already permeable areas.	Area that can be used for bio-retention basin in the design project are limited and small • These areas are not sufficient to accommodate the WSUD design flows calculated for the North Terrace Drainage Design.

Table 23 - Bio-retention basin design options

3.4.1.6. *Cost Estimate*

The costing of the best bio-retention basin design can be estimated using the following (Figure 68) extracted from Department of Water, Government of Western Australia 2007 and the costing provided by Local Government Association of South Australia (2009) in *Water Sensitive Urban Design – Greater Adelaide Region Technical Manual.*

Table 2. Cost estimates for bioretention systems

Publication/ Data source	Construction ($)	Maintenance ($/m²/yr)	Location	Description
Basins				
Leinster (2004)	$125–$150/m² $225–$275/m²	-	South East Queensland	> 100 m² area < 100 m² area
Swales				
Leinster (2004)	$100-$120/m	-	South East Queensland	3-4 m top swale width
Fletcher et al. (2003)	$135/m	$2.50 $1.50	South East Melbourne	Grassed system Vegetated system (Natives)
Lane (2004)	$350/m	-	NSW	-
URS (2003)	$410/m	-	Western Sydney	3 m wide

Figure 68 - Cost estimate for bio retention systems - extracted from Bioretention Systems design Department of Water, Government of Western Australia 2007

Costing of the estimated required area of detention basin (using Figure 68)

$$=150\ \mathrm{m}^2 \times \$\ 140/\mathrm{m}^2 \approx \$21{,}000$$

Costing of the estimated required area of detention basin (Local Government Association of South Australia)

$$=150\ \mathrm{m}^2 \times \$\ 137/\mathrm{m}^2 \approx \$20{,}550$$

The greater cost is taken to be conservative for the bio-retention basin and ca be seen Table 24.

Table 24: Cost Analysis of Bio-Retention Basin

Item	Description	Unit	Size	Qty	Rate/m²	Cost ($)
1.1	Bio-Retention Basin Construction	m²	150	-	$140	21,000.00
					Total	21,000.00

3.4.2. Porous Pavements

Porous, permeable and pervious are all words used to describe paving which allows water to infiltrate through it. Though these words describe the same technology, all pervious paving can be categorised into either a porous or permeable surface. Porous pavement contains a layer of pavement with small pores. The porous nature of this material means that water can infiltrate through, what would usually be an impervious surface, quite efficiently.

Permeable pavements are constructed of normal paving material, with no pores, but water infiltrates into the ground due to the shape of the pavers. Instead of including one large, impermeable, slab of pavement or concrete, the pavers are shaped in a way so voids between the pavers create a pattern and allow infiltration. If these voids are large, they are often filled with gravel to make the design more aesthetically pleasing and prevent health (trip) hazards. (Griffith University, Government of South Australia, 2010, NC State University, A&T State University 2008, Paving Expert, 2013)

3.4.2.1. Porous Pavement Design

Both types of pervious pavements are constructed in the same way. The top layer is the only layer that differs between the two, as can be seen in Figure 69.

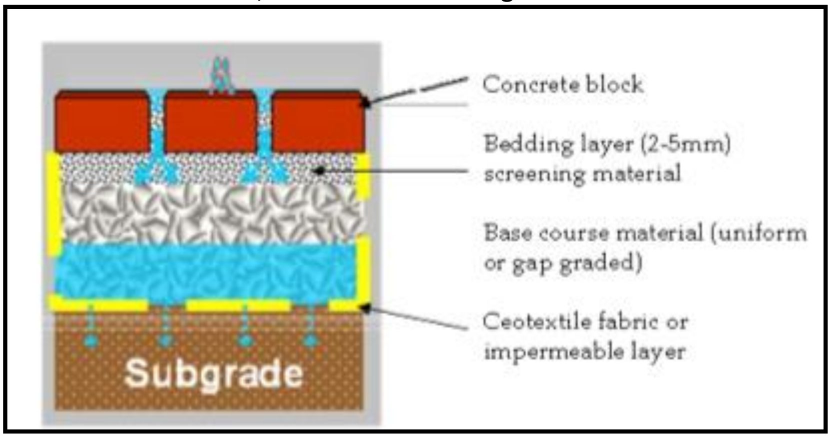


Figure 69 - Cross Section of Pervious Pavement Design (Government of South Australia, 2010)

The water first infiltrates through the pervious surface layer (concrete block).This layer contains the permeable or porous pavement that allows the water to infiltrate through the hard surface. The permeable pavements are often referred to as having modular design as the surface is constructed of individual pavers (eg. bricks). Porous pavers have monolithic properties as individual pavers cannot be seen and a continuous surface is produced. Examples of these are asphalt roads and concrete slabs.

The next layer is the aggregate storage layer which is typically constructed from coarse gravel. The main purpose of this layer is to provide structural support to the surface layer (ie. prevent the slipping and moving of tiles and support weights of loads such as vehicles and pedestrians), but can be also used to store water if the layer below this is full. This would occur after a heavy downpour of rain, as the water would slowly infiltrate into the ground and prior to this be stored in the reservoir storage section. If a large amount of water infiltrates through the pavement at once then the water may not infiltrate into the ground quickly enough, cause the water to rise and move about the reservoir layer. This layer is usually lined with a permeable geotextile fabric to prevent soil or gravel entering the reservoir section. (Griffith University, Government of South Australia, 2010, NC State University, A&T State University 2008, Paving Expert, 2013)

The reservoir storage layer is used to store water prior to moving to the sub-base. This part consists of gravel and stones.

After this, the water moves into the sub-base which is formed by compacted soil. A large amount of water can be stored in this area as it slowly infiltrates through to the bottom layer. The bottom layer of the design can either consist of an underdrain, soil layer or submerged tank. After filtering through the above layers the water can be discharged into the area directly below the pervious surface through a soil layer. Though if more benefits are provided by using water in a different area, then an underdrain is constructed. These are small drain and pipe networks which are used to transport the water to the preferable area. The third option is to construct a tank as the bottom layer of the design and have the water pumped directly from the tank back to the surface through a tap at this location. (Government of South Australia, 2010, NC State University, A&T State University 2008, Paving Expert, 2013)

In the North Terrace Drainage Design, the water would be fed into a pipe system and pumped elsewhere, due to the high clay content in the surrounding soil.

3.4.2.1.1. Design Considerations

Other design considerations include implementing the correct design for the correct environment in terms of stability and infiltration (some soil types can support structural loads better than others and some have higher infiltration rates than others), retention time, maintenance schedule to reduce clogging, safety, slope, estimated traffic volume and type, incorporating the use of vegetation, evaporation and construction costs. (Griffith University, Government of South Australia, 2010)

3.4.2.1.2. How Pollutants are Removed

Pollutants are removed within each layer of the pervious pavement through physical, chemical and microbial factors.

Physically, the water undergoes a straining process. During this process water infiltrates through surfaces which contain progressively smaller pores. The pore space strains the particle from the water, leaving the impurity within the pore. This can cause clogging to occur as pore spaces become smaller and are reduced. The process of clogging improves particle removal as smaller pore sizes lead to finer particles becoming removed from the fluid but can reduce infiltration, and hence increase runoff, rates (National Programme on Technology Advanced Learning, 2013).

There are three processes which occur to remove contaminants, chemically, from the water. These are sedimentation, diffusion and interception. Sedimentation occurs when slow moving water causes large impurities to be removed from the flow and sink. Brownian Diffusion is the process in which smaller particles are removed from the water due to the unsequenced movement of impurities caused by thermal gradients and interception, a chemical filtration technique which is efficient in removing larger particles. This occurs when the larger impurities move within the flow streamline and collide with the filter media, removing and storing the contaminant. (National Programme on Technology Advanced Learning, 2013, Melbourne Retail Water Agencies)

Within the structure, microorganism cultures also form to remove pollutants. They do this by converting some pollutants into a less harmful form.

Downstream, at a local catchment, pollutant loadings are also decreased due to the high infiltration rates of this design, causing surface runoff rates to be decreased. The following (Table 25) illustrates the capabilities of different types of pervious pavements to reduce surface runoff.

Table 25 - Percent Reduction in Surface Runoff Depending on Pavement Type

Pavement Type	% Reduction in Surface Runoff
Asphalt (Impervious)	34.6
Porous Concrete	99.9
Permeable Interlocking Concrete Pavers	99.3
Concrete Grid Pavement	98.2

3.4.2.1.3. Efficiency of Pollutant Removal

Not only is this WSUD feature capable of drastically reducing runoff, it also improves the quality of stormwater. Table 25 of this report summarises the efficiency of the pollutant removal for pervious pavements. It should be noted when viewing Table 25 that the majority of TSS are removed due to the filtration techniques which are utilised in the design. Though removal of most pollutants is quite high, technology is currently being developed to increase these further.

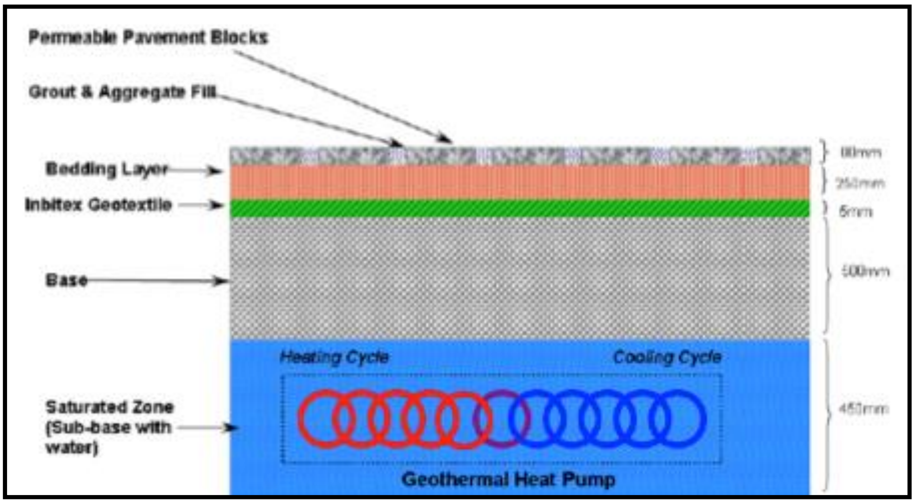


Figure 70 Incorporating Geothermal Heat Pump Technology with Permeable Pavements (Grabowiecki et al 2010)

An example of this technology includes incorporating the use of permeable pavements with geothermal heat pumps, as shown in Figure 76. By implementing this, an even heat distribution will be provided to the infiltrated water at the bottom layer. This reduces pollutant loadings as the amount of pathogens decreases and promotes life of beneficial microorganisms. (Grabowiecki et al 2010)

3.4.2.1.4. Advantages

Many advantages are associated with the use of this WSUD technology, and include:

- An increase in the amount of permeable surfaces and a decrease in surface runoff
- Recharging groundwater aquifers
- Easily implemented – ie. does not require land acquisition or require removal of existing pavement structure
- Can be applied to a wide range of sites
- Improved stormwater quality
- Cost effective
  (Griffith University, Government of South Australia, 2010, NC State University, A&T State University 2008 & Thorpe, Zhuge, 2010)

3.4.2.1.5. Disadvantages/Limitations

Some constrains associated with this WSUD are:

- Clogging – If not maintained correctly it has been known for the pavement to become clogged.
- Composition of Concrete – This is an important factor as incorrect constituents can lead to failure and decrease technology life.
- Recycled aggregate – It has been found through testing that the use of recycled aggregate further decreases technology life. Therefore, some may argue the design is not sustainable.
- (Government of South Australia, 2010, NC State University, A&T State University 2008, Thorpe, Zhuge, 2010

3.4.2.2. Design Calculations/Considerations

The porous pavement was designed assuming that this treatment would be implemented in the car park areas. All preliminary design calculations and results for the porous pavements are included in Appendix A3.1. Porous Pavements.

3.4.2.2.1. Design Calculations

In this part of the feasibility study, the required area of pervious pavements were calculated using car parks only. This is done to gain an estimate of the total required area if pervious pavement was used on its own for WSUD flow.

According to the Department of water, Government of Western Australia (2007), the design should be considered as shown in Figure 80. This is because the car parking is considered to be outside the catchment area, which in this case is considered to be only the road and the pavement of the foot path. Therefore, the water runoff on the car parks itself (A_{inf}) need to be added to the required design rainwater runoff.

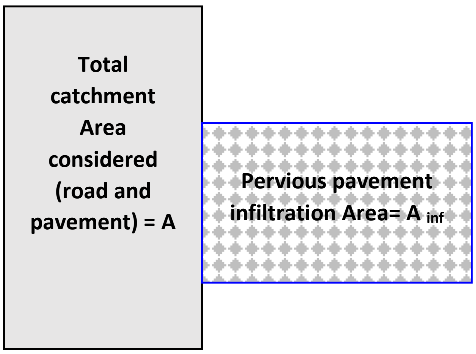


Figure 71 Area where the infiltration surface is located outside the defined site area (Hydro-Future Consulting, 2015)

As can be seen in A3. Water Sensitive Urban Design Calculations, the minimum area required for draining the water coming from the road and the pavement has to be at least 36,689m^2. Since this area cannot be achieved by all the footpath area and the car parks present in the design area.

Therefore, using pervious pavement on its own is not a feasible option.

Pervious pavement could be used in conjunction with other options. For example, there are 2 car parks that are would be most appropriate as seen in Figure 72. Car park in lot 103 (Car park

1) and the car park near Clark Rubber (Car park 2). Even though the car park in front of St Peter's College has the biggest area, this car park will not be effective as it is not in the sag area and therefore would not receive a significant design flow.

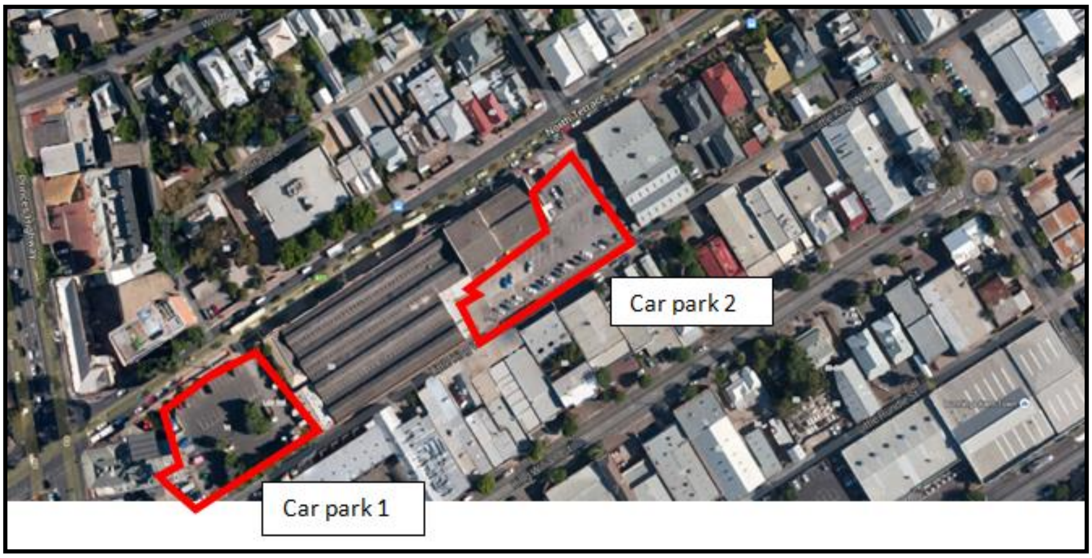


Figure 72 - Car Parks Locations (Google Maps, 2015)

The area of car park in Lot 103 (car park 1) is 3072.5m^2

The area of car park next to Clark Rubber (car park 2) is 2391.4m^2

If car park 1 and car park 2 are to be paved with pervious pavement, the water flow that can captured by these to area can be calculated to approximately as 0.0476m^3/s. This is approximately equal to 14.5% of the required WSUD design flow.

$$Design\ water\ infiltration\ from\ car\ parks\ 1\ \&\ 2\ pervious\ paving$$

$$= 5463 \times \left[(1 - 0.2) \times 2 \times 2 \times 10^{-5} - \frac{83.6}{60 \times 60 \times 1000}\right.$$

$$= \mathbf{0.0476 m^3/s}$$

3.4.2.2.2. Design Considerations

For the design area there are a few options where the pervious pavement can be implemented. Table 26 summarises the possible options for the use of pervious pavement.

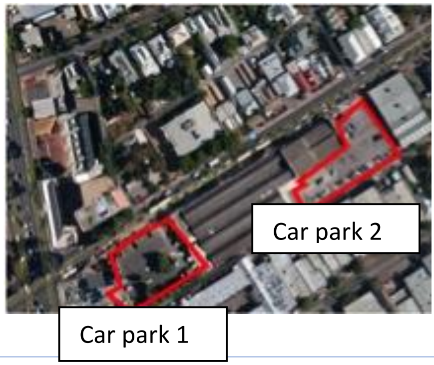
Options	Using pervious pavement in car parks area only	Using pervious pavement in all footpath	Using strips of pervious pavement in the footpath	Using strips of pervious pavement in the foot path, in addition to, using pervious pavement in car parks
Illustrations				
Discussions	• The car park areas are very small to contain the WSUD water flow coming from all the catchment. • Costing is not as high reconstructing all the footpath • Preferred option is to use the car parks to drain water from immediate surrounding areas; this will save cost of reconstructing all slops to drain in the car parks.	• This option will be very expensive as it would need reconstruction of the whole area of the footpath. • However, this option would have the biggest area	• Costing is less than using it for all the foot path • The area is less changing the full footpath length • Area might not be sufficient	• More area that strips in the footpath on its own. • Less expensive than reconstructing the entire footpath • Dimensions of the design will be specified in more details in the detailed design stage if it was chosen as the preferred option

Table 26 - Design options for pervious pavements

3.4.2.3. Cost Estimate

The costing of installation of pervious pavement in the 2 available car parks shown in Figure 72, can be estimated using estimated using values from Figure 73 (Department of Water, Government of Western Australia, 2007).

Pervious Paving Method	**Construction Cost**
Porous paving allowing infiltration	$111/m²
Porous paving over sealed sub-grade allowing water collection	$119/m²
Augmentation with porous paving (i.e. mixing porous with normal pavers)	$98/m²
Porous paving with asphalt	$67/m²
Porous paving with concrete slab	$90/m²

Figure 73 - Pervious Paving Installation Costs (Boral, 2003 cited in Taylor, 2005)

The preferred option for this design is Porous pavement over sealed sub-grade allowing water collection, as this will mean that the treated water are used for useful application rather than being sent to the normal drainage system and mixed with untreated water.

The total cost for porous pavements options is included in Table 27 and Table 28 below.

Table 27 - Costing of Pervious Pavement in Car Park Lot 103

Option A – Lot 103 Car Park						
Item	Description	Unit	Size	Qty	Rate/m²	Cost ($)
1.0	**Paving**					
1.1	Porous paving over sealed sub-grade allowing water collection	m²	3072.5	-	$119	365,628.00
					Total	**365,628.00**

Table 28 Costing of Pervious Pavement in Car Park Adjacent to Clark Rubber

Option B – Clark Rubber						
Item	Description	Unit	Size	Qty	Rate/m²	Cost ($)
1.0	Paving					
1.1	Porous paving over sealed sub-grade allowing water collection	m²	2391.4	-	$119	284,529.00
					Total	284,529.00

3.4.3. Sedimentation Basins

A sedimentation basin consists of an inlet, settling pond (permanent settling zone and sediment storage zone), overflow and outlet, as shown in Figure 74 below. This system works to reduce the amount of coarse and medium sized particles within the stormwater runoff by decreasing flow velocities and promoting detention. These are typically used upstream of other WSUD treatments such as bio-retention basins, constructed wetlands or macrophyte zones. Implementing a sedimentation basin ensures that when the water reaches the downstream treatment option, the vegetation does not experience scouring from the particles and can filter other pollutants. (Brisbane City Council, 2005, Department of Planning and Local Government, 2010, United States Environmental Protection Agency, 2014, State of Michigan, 2014).

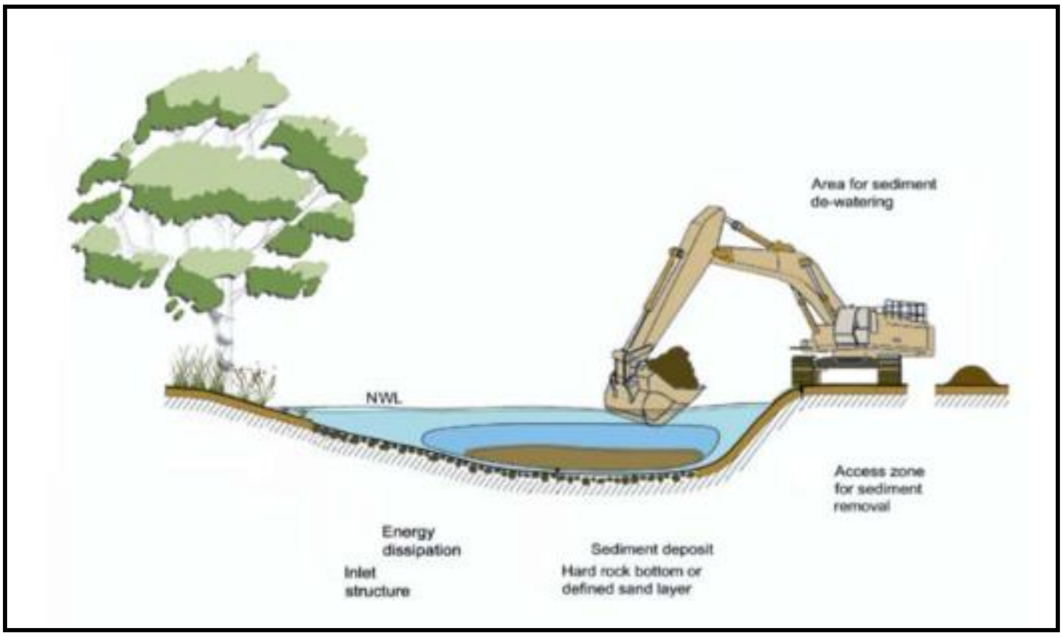


Figure 74 Components of a Sedimentation Basin (Department of Planning and Local Government, 2010)

3.4.3.1. Advantages

Advantages of installing a sedimentation basin include:

- Promote biodiversity
- Aesthetically pleasing
- Reduced pollutant volume in stormwater runoff

(USEPA, 2014)

3.4.3.2. Disadvantages/Limitations

Limitations of this system include:

- Maintenance required every 2-5 years
- Usually require large amount of space

(United States Environmental Protection Agency, 2014)

3.4.3.3. Impact to Water Quality

The pollutant removal efficiency of a sedimentation basin is outlined in Section 3.4.6.

It should be noted that many of these mean removal values are presented as a range, not just a single value. The reason for this is that many of the removal efficiencies are dependent on the speciation of the chemical, particle size distribution, charge of the elements and detention time.

3.4.3.4. Design Calculations/Considerations

The preliminary calculations for a sedimentation basin were then completed. It was assumed that water entering the basin was the runoff from the road and footpath area. This WSUD option was also designed for an ARI of 5 years. All preliminary design calculations and results are included in Appendix A3.2. Sedimentation Basins.

3.4.3.4.1. Flow Rate

A sedimentation basin is designed to utilize runoff from the road and pavement areas within the catchment area. Therefore, as shown in previous calculations, the corresponding flow rate for this area size and type is 0.332m^3/second.

3.4.3.4.2. Hydraulic Efficiency

The hydraulic efficiency of the system (λ) can be assumed to be 0.26, as the shape of the basin is likely to be rectangular, where the water enters and leaves the system along the same path. This value ranges from 0 to 1 and should be confirmed as part of the detailed design. For the feasibility study, assuming a rectangular arrangement is a conservative assumption made for high level design purposes.

3.4.3.4.3. Turbulence Factor

The turbulence factor, n, was found through calculation to be 1.35.

3.4.3.4.4. Sediment Removal Efficiency

For a sedimentation basin to be considered feasible for implementation, the removal efficiency of sediments must be equal to, or greater than, 80%. By trial and error, if it is assumed that the area is equal to 50m^2, then an efficiency of approximately 80% is found through calculation.

Therefore, designing a basin, which has a total area of 50m^2, will meet sediment removal efficiency requirements.

3.4.3.4.5. Required Storage

The required storage, is the area which is required to store the expected runoff in the basin. This is dependent on the size of the catchment which will contribute runoff, the expected removal efficiency of the basin, sediment loading rate and how often it is expected that the basin will be cleaned. For this option to be considered feasible the available storage, the area used in the above equation as A (50 square meters) must be greater than or equal to the required storage.

The required storage area through calculation is:

$$S_t = 10.62\text{m}^2$$

As this value is less than the area of the basin (50m^2), then the system will work as the available storage is less than the required storage. Though the available storage greatly exceeds the required storage, the basin must be a minimum of 50m^2, otherwise the pollutants and sediments will not settle and be removed from the water.

3.4.3.4.6. Design of Outlet Pit

The outlet pit for the sedimentation basin was then designed. Through calculation it is found to be 3.12 metres.

The area of the outlet pit was also found through calculation to be 0.5m^2.

3.4.3.5. Cost Estimate

It is estimated that in Australia a sedimentation basin costs between \$150 and \$250 per m^2 to install (Lake Macquarie City Council, 2012) Therefore, if the worst case is considered for a $50m^2$ basin, the initial cost will be approximately \$12,500. There are also maintenance costs of approximately \$14/m^2 per year (ie. \$700/year).

The cost of land acquisition has been estimated to be approximately \$2,700 per square metre. Therefore, the cost to acquire $50m^2$ for the basin and an additional $0.5m^2$ for the overflow pit would be \$136,350, bringing the total to \$148,850 as seen in Table 29. It should be noted that this cost does not take into account the cost of labour or ongoing maintenance costs.

Table 29 - Sedimentation Basin Material Costs

Item	Description	Unit	Size	Qty	Rate/m^2	Cost (\$)
1.0	**Sedimentation Basin**					
1.1	Basin Implementation	m^2	50	-	\$250	12,500.00
1.2	Land Acquisition	M^2	50.5	-	\$2,700	136,350
					Sub-total	148,850
					Total	148,850

3.4.3.5.1. Possible Location of Sedimentation Basin

Due to the requirement of 80% pollutant removal efficiency the sedimentation basin must have an area of $50m^2$, which approximately equates to a basin 7.5mx7.5m in size. There is a possibility that this could be implemented in the carpark next to Clark Rubber (as shown in Figure 75) or the carpark located next to the hotel, but the business would lose a large portion of their parking space. If any flooding event did occur, this treatment would be very close to businesses and residential areas, though as the car parks are considerable large, the basin could be strategically located to minimise this risk.

Figure 75: Possible Locations of Sedimentation Basins (Google Maps, 2015).

3.4.4. Infiltration Trenches

An infiltration trench consists of a linear, shallow, excavated trench area that is lined in a geotextile fabric and filled with gravel or rocks. The trench is filled to the top, so that the top of the trench is flushed with the surrounding surface level. An overflow berm is also incorporated into these designs to mitigate flooding risk. (Dublin Drainage, Minnesota Urban Small Sites BMP Manual, Newcastle City Council)

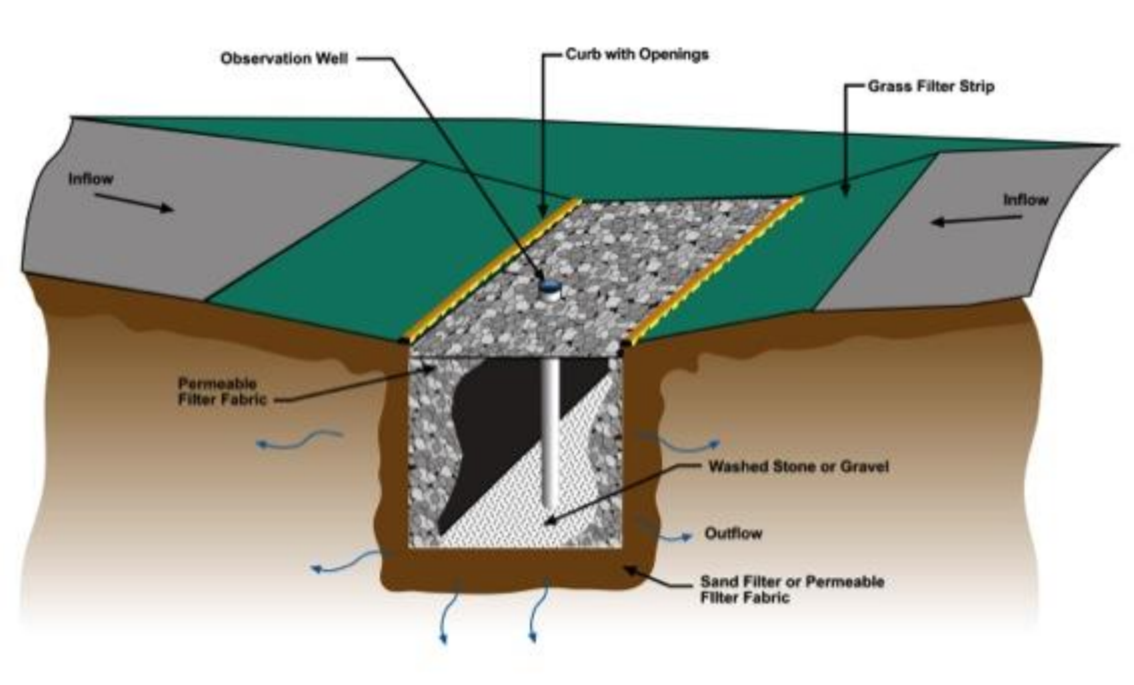


Figure 76 Infiltration Trench Components (Ken Eulie Graphics, 2010)

Figure 76, above, shows a typical cross section of an infiltration trench.

After a rainfall event, the water slowly filters through the rocks and gravel and through the geotextile lining on the vertical walls or base. Following this, the water flows through the surrounding soil to recharge groundwater. For this reason, it is also recommended that these treatments be placed in a location that contains permeable, sandy, soil. As a guideline these soils should have a clay content of less than 20% and a silt/clay content of less than 40%. (Dublin Drainage, Minnesota Urban Small Sites BMP Manual, Newcastle City Council).

3.4.4.1. Advantages

Advantages of this design include:

- Groundwater is replenished
- Improved water quality
- Decreases speed of water entering waterways during peak flow and limiting erosion effects

(Dublin Drainage, SusDrain, 2012, Melbourne Water)

3.4.4.2. Disadvantages

The disadvantages of this are:

- High maintenance is required to prevent clogging
- Difficult to implement effectively in a clay environment
- Can pollute groundwater if water flows through trench too quickly

- Cannot be installed on a steep slope
- Not ideal for large catchment areas

(SusDrain, 2012, Melbourne Water)

3.4.4.3. Impact to Water Quality

Sedimentation basins can have a positive impact on runoff quality, as the decreased water velocities give large particles time to settle. The pollutant removal efficiency of this treatment is described in the Water Quality Section of this report. The relative impacts on water quality are shown in Table 35.

3.4.4.4. Design Calculations/Considerations

Due to the clay content of the surrounding soil at North Terrace, an infiltration trench was considered but was designed slightly different to a normal infiltration trench. As with the sedimentation basin, it was assumed that the water running through this trench would be that from the pavement and road runoff, and was designed for an ARI of 5 years, as defined in Section 2.1.1. All infiltration trench preliminary calculations and results are included in Appendix A3. Water Sensitive Urban Design.

3.4.4.4.1. Flow Rate

An infiltration trench is designed to utilize runoff from the road and pavement areas within the catchment area. Therefore, as shown in previous calculations, the flow rate is $0.332m^3$/second.

3.4.4.4.2. Critical Stormwater Runoff Volume

The critical stormwater runoff volume (V) is equal to $101.7m^3$

3.4.4.4.3. Length of Infiltration Trench

The length of the infiltration trench is required to be a minimum of 220.5 metres.

It should be noted that the length of this trench was determined using the standard dimensions of an infiltration trench, 1m wide and 0.5m high, as it is the smallest size that will provide an acceptable emptying time.

3.4.4.4.4. Emptying Time

The emptying time for the infiltration trench designed above was then checked and resulted in 34.5 hours for an ARI of 5 years, the recommended emptying time is 1.5 days (ie.36 hours), therefore this emptying time is acceptable.

3.4.4.4.5. Designing in Clayey Soil (Slow Drainage System)

As this design is to be implemented in clayey soil, a slow drainage system is required. The addition of a slow drainage system involves installing a small diameter pipe to the corner of the downstream end of the treatment option (in this case the infiltration trench), as seen in Figure 77.

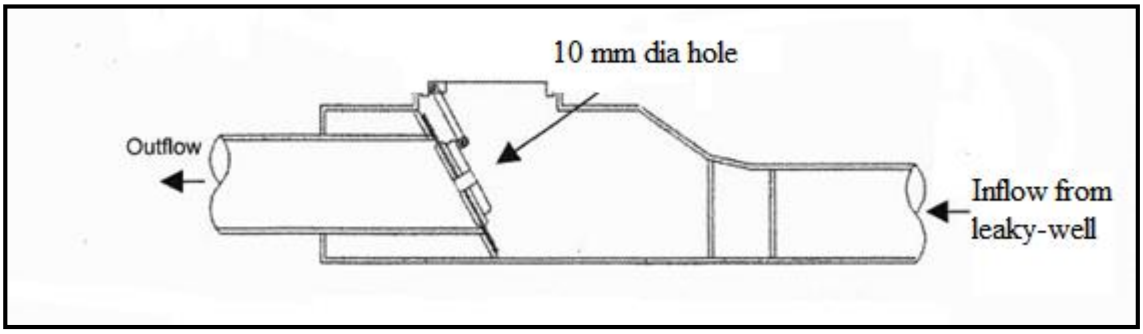


Figure 77: Components of the Slow Drainage System (Argue, J.R, 2005)

When a major storm event occurs, and the soil has low permeability, the soil cannot receive the water at the same rate that it is flowing into the WSUD treatment.

The smaller pipe is placed on an angle and is connected to a large outflow pipe. When the water in the inflow reaches a level, level with the entrance of the small pipe, the small pipe is filled and slowly flows into the larger outflow pipe. (Argue 2005)

The speed of the flow into the outflow pipe is regulated as only a very small amount of water can fit through the 10mm hole and the outflow pipe is considerable larger than this. Therefore, the outflow pipe does not fill (Argue 2005).

The advantage of this system is that infiltration options can still be implemented into clayey soil and though some water is likely to flow into the conventional stormwater system through the outflow pipe, it is not all received at once, as opposed to a conventional stormwater system.

3.4.4.5. Possible Location of Infiltration Trench

As seen in the above calculations the infiltration trench is required to be 220.5m long, 1m wide and 0.5m deep.

This could be implemented along the footpath as shown below. However structurally, these would need to be constructed from lots of small infiltration trenches to ensure driveway access is not blocked. Regarding pedestrian movement, this would still be an acceptable option as a pavement exists on the other side of the road. Also, the trench would not require the use of the entire length of pavement, meaning a narrow pavement could be constructed parallel to the trench. This has been placed on the opposite side of the school, and is quite far upstream of the creek. This should prevent flooding downstream and also have a lesser impact on pedestrian movement as this tends to increase closer to the CBD. Bus stop relocation is also not required in this design. In Figure 78 the blue line represents the location of the proposed trench. This may not meet requirements for footpaths in the CBD area, and hence, the location described below, and shown in Figure 79, is considered to be more feasible.

Figure 78 Possible Location of Infiltration Trench (Google Maps, 2015)

The school also has a green area which is just over 220m long. The council could purchase this land from the school, then part of the existing garden could be utilised as an infiltration trench (see Figure 79). This option is preferred as there is still sufficient room for a path and the schools grounds will still remain aesthetically pleasing as the infiltration trench is planted.

Figure 79 Possible Location of Infiltration Trench (2) (Google Maps, 2015)

3.4.4.6. Cost Estimate

Cost estimates for the installation of an infiltration trench are typically $138 per linear metre. (WA Government, 2004) Though this estimate is based on the depth of the trench being 1m deep. This cost estimate includes the cost of excavation, soil removal, installation of geotextile, perforated pipe, gravel layer and filter layer. This also included the cost of applying top soil, grass seeds and fertiliser and watering the planted vegetation. The total volume of a trench which is 220m long, 0.5m high and 1m wide is 110m^3. As this cost estimate is based on the depth of the trench being 1m, the cost estimate can be applied to this problem, assuming the trench is only 110m long (110mx1mx1m = 110m^3). Therefore, this will cost approximately $15,180.

Maintenance costs should also be considered and are estimated to be approximately 20% of the construction cost (ie. $3,035 per year).

As with the cost estimate of the sedimentation basin, this cost could alter depending on the location of the trench (ie. whether land acquisition is required) and number of services, which would require relocation. These should be explored further in the detailed design, but as an initial estimate, if the cost of land acquisition is $2,700 per square metre, and the installation of this treatment requires 220m^2 of land (trench which is 220m long and 1m wide) to be acquired, the cost of land acquisition alone would be equal to $594,000.

This would increase the cost of implementation to $609,180 plus ongoing maintenance costs as seen in Table 30.

Table 30 - Infiltration Trench Material Costs

Item	Description	Unit	Size	Qty	Rate/m^2	Cost ($)
1.0	**Infiltration Trench**					
1.1	Trench Construction	m^2	110	-	$138	15,180
1.2	Land Acquisition	m^2	220	-	$2,700	594,000
					Total	**609,180**

3.4.5. Leaky Wells and Soak-away Crates

For the purpose of this feasibility study, leaky well and soak-away treatments have been grouped due to their similarity in design and efficiency.

3.4.5.1. Soak-away Crates

A soak-away is a plastic crate that has a high void ratio. This crate is buried in the ground and acts as an attenuation cell for surface runoff. As the water moves into the soak-away crate, through the inlet pipe or through natural infiltration, the crate provides a storage area for the water. The water is then slowly released into the surrounding soil at a rate at which the water can be absorbed by the soil (Sustainable Drainage Centre, 2014).

It is a requirement that the crate be wrapped in a geo-membrane or geotextile fabric to prevent surrounding soils entering the crate and clogging the void space.

Once entering the surrounding soil, the water is required to drain quickly away from the soak-away crate to prevent clogging and should therefore, not be implemented in an area which has a high clay content (Sustainable Drainage Centre, 2014).

Figure 80 shows a typical layout of a soak-away crate.

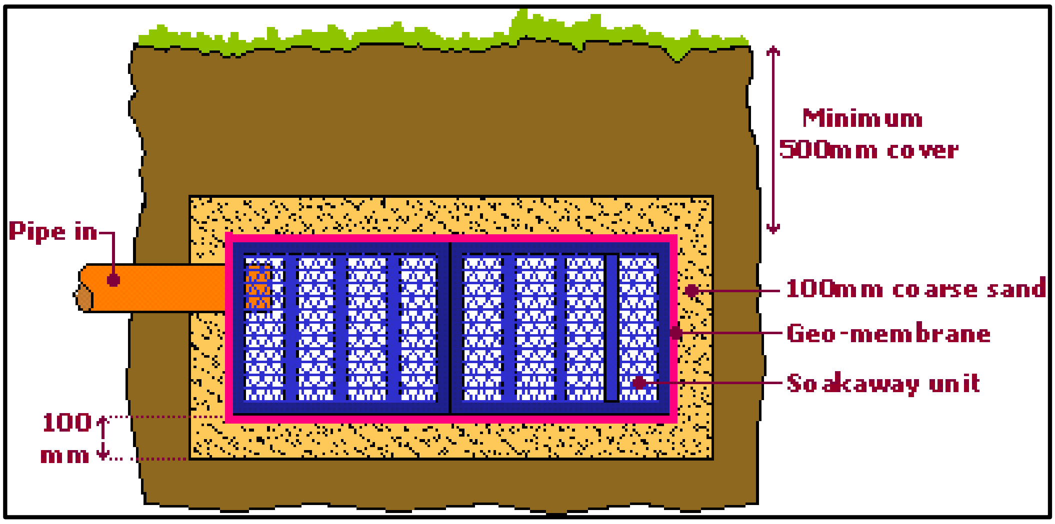


Figure 80 Soakaway Components (Paving expert)

3.4.5.2. Leaky Wells

A leaky well is very similar to a soak-away. This treatment consists of a large vertical pipe that has perforations along its vertical walls. These perforations are covered with a geotextile fabric, as is the base of the well. The stormwater enters the large pipe through an inlet then flows down through the pipe and exits the well through the side perforations or base. A layout of this is shown below in Figure 81 (Townsville City Council, 2013)

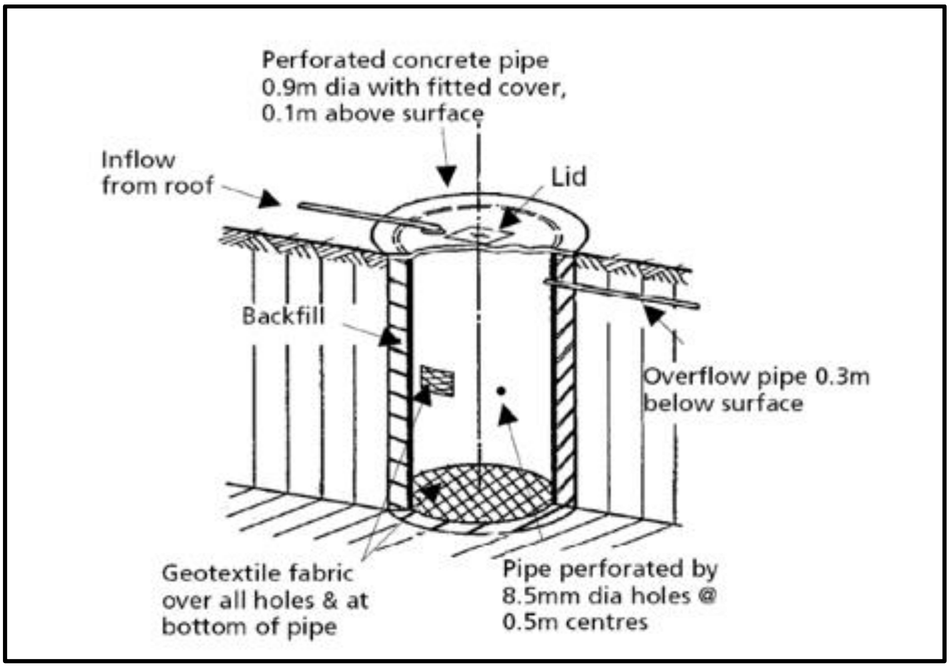


Figure 81 Leaky Well Components (Townsville City Council, 2013)

In terms of water quality, pollutants are only removed by the geotextile fabric, as there are no other filter media incorporated into this design.

These are typically paired with an overflow pipe, which can transport excess water downstream.

Ideally, these are also placed in an area that contains sandy soils. The reason for this is that when the water drains from the vertical pipe wall or through the base, the surrounding soil needs to be permeable enough for the water to move away from the leaky well. If the soils permeability is too low then clogging may occur. (Townsville City Council, 2013, Brisbane City Council, 2005).

3.4.5.3. Advantages

The advantages of installing a leaky well are:

- Does not require large space for installation
- Cheap to install
- Water infiltrates into surrounding soil, providing groundwater recharge

(Brisbane City Council, 2005)

3.4.5.4. Disadvantages

The disadvantages of leaky wells include:

- Pollutants only removed through geofabric
- Does not promote biodiversity
- Most effective in soils with high permeability

(Brisbane City Council, 2005)

3.4.5.5. Impact to Water Quality

The pollutant removal efficiency of the leaky well and soak-away treatments is relatively low as the geotextile liner is the only filtration media employed in these treatments. The efficiency of pollutant removal for these are summarised in the Water Quality Section of this report.

When viewing this table it should be noted that many of the pollutants have an NA rating. The reason for this is because they were not tested in soakaways or leaky wells. Testing did not occur for removal of these contaminants for these designs because a complex filtration method or filtration layer does not exist, and hence pollutant removal efficiency is expected to be very low.

3.4.5.6. Design Calculations/Considerations (Soak-Away Crate)

First, a soak-away system was designed for the catchment. As seen in Figure (45: Soak-Away Crate Design) the crate has an inlet pipe, meaning that runoff from different sources can be directed into this system. An example calculation method is shown for the soakaway, which assumes the only runoff utilised by the system, is roof runoff. The other results and values used are summarised in Table 31. All design calculations for the soakaway system can be seen in Appendix A3.4. Soak-Away .

3.4.5.6.1. Critical Stormwater Runoff Volume

The critical stormwater runoff volume is calculated to be 542.1m^3.

The rainfall intensity (required for the calculations) was found using an Intensity-Frequency-Duration Table provided by the Bureau of Meteorology for an ARI of 5 years and a time of concentration of 10 minutes (see Section 0, Figure 4).

As can be seen, this gives an intensity of 62.2mm/hr.

3.4.5.6.2. Area of Soak-Away

The required area of soak-away is calculated to be a minimum of 1173.4m^2

3.4.5.6.3. Emptying Time

The calculated emptying time is 50 hours and as the recommended emptying time for a 5-year ARI is 1.5 days, this option is therefore unsuitable.

3.4.5.6.4. Calculations for Different Runoff Scenarios

If the soakaway was utilised for other runoff types, then different required areas and emptying times must be included. These are summarised in Table 31. It should be noted that these calculations are based on the soakaway being 0.5m high and designed for a 5 year ARI.

Table 31 Calculations for Different Runoff Types for Soak-Away Treatments

Runoff Type	Runoff Coefficient (c)	Rainfall Intensity (I)	Contributing Catchment Area (A)	Time of Concentration (t_c)	Runoff Volume (V)	Area of Soak-away (A)	Emptying Time (T_e)
Roof Runoff	0.9	62.2mm/hr	58,100m^2	10 mins	542.1m^3	1173.4m^2	50 hours
Road and Pavement	0.9	83.6mm/hr	16,220m^2	5 mins	101.7m^3	220.1m^2	50 hours
Total Runoff	0.799	43.7mm/hr	79,400m^2	20 mins	924.1m^3	2000.2m^2	50 hours

The slow drainage system described in Section 3.4.4.4.5 can potentially decrease this emptying time and may make the above options feasible. The extent of which the emptying time is reduced by the drainage pipe will require investigation in the detailed design stage.

3.4.5.7. Possible Location of Soak-away System

Soakaway crates are buried beneath the surface of the natural ground so can be implemented under any footpath or green area. Drainage Online provides builders in the United Kingdom with soak-away crates that are 0.5m high, 0.4m wide and 1m long (ie. each crate has a volume of $0.2m^3$). Therefore, 5 crates would be required to produce $1m^3$ (0.2x5) of soak-away crate volume.

If the area of soak-away crates required is $1173.4m^2$ then this would require a square trench of 35x35m. Within this trench, 3,080 crates would be placed as each crate is 1m wide and 0.4m long. This means the trench would be filled with 35 crates placed along one side of the trench and 88 crates (35m/0.4m) placed along the other, giving a total of 3,080 crates (35x88 = 3080).

Ideally, these would also be buried under school gardens or within the car parks identified in Section 3.5.

3.4.5.8. Cost Estimate

These crates cost approximately \$50 each and excavation to a depth less than 6m in Adelaide in a clayey soil is approximately $\$16/m^3$. The cost of materials is shown in Table 23 below, assuming the depth of excavation was 1m (0.5m depth for soak-away and 0.5m depth coverage). When reviewing this table it should be noted that the cost of labour and installation of the slow drainage system has not been included in this cost and would be required for review in the detailed design.

Table 32 Cost Estimate of Soakaway Treatment Options (Rawlinsons, 2014)

Runoff Type	Number of Crates Required	Cost of Crates	Size of Excavation Required	Cost of Excavation	Total Cost
Roof Runoff	3080	\$\$154,000	$1225m^3$	\$19,600	\$173,600
Road and Pavement	570	\$28,500	$225m^3$	\$3,600	\$32,100
Total Runoff	5085	\$254,250	$2025m^3$	\$32,400	\$286,650

3.4.5.9. Design Calculations/Considerations (Leaky Well)

Similar calculations were then performed for the implementation of a leaky well. The same runoff types were considered as the inlet pipe could be connected to roof runoff pipes or conventional stormwater system pipes. Again, the example calculation performed is just considering the leaky well will utilise runoff from the rooves of local residents and businesses.

All preliminary calculations and results for the leaky well are included in Appendix A3.4.

3.4.5.9.1. Critical Stormwater Runoff Volume

The calculated critical stormwater runoff volume is found to be $542.1m^3$

3.4.5.9.2. Diameter of Leaky Well

The minimum diameter required for a leaky well is 18.5 metres.

This would not be considered acceptable because the diameter greatly exceeds the nominated height of 2m, and they should be approximately equal. To overcome this, the height could be altered until the height is approximately equal to the diameter or multiple wells could be used.

3.4.5.9.3. Diameter of Single Leaky Well

For the diameter of the well to be approximately equal to the height, a height of 9m was trialled and after calculation a diameter of 8.7 metres was selected.

Though this requires an incredibly high well, the ratio of the height to diameter would be considered acceptable.

3.4.5.9.4. Multiple Leaky Wells

If it assumed that standard sized leaky wells were installed, with a diameter of approximately 2m and height of 2m, then each tank could hold almost 7m^3 of water. Therefore, if multiple wells were used, a total of 78 tanks would be required. This would mean each tank would hold approximately 6.95m^3 of water and that the diameter and height are equal.

As a result, the new tank volume required would need to be a minimum of 6.95m^3. Through further calculation this yielded a diameter of 2.1 metres, which is approximately the height.

3.4.5.9.5. Emptying Time

The emptying time of both the single leaky well and multiple leaky wells were found through calculation and are as follows.

3.4.5.9.6. Emptying Time – Single Leaky Well

The emptying time for a large, single, leaky well is calculated to be 395 hours.

This greatly exceeds the recommended emptying time for a 5 year ARI (1.5 days), which may make this option unsuitable.

3.4.5.9.7. Emptying Time – Multiple Leaky Wells

The emptying time for a large, single, leaky well is calculated to be 91 hours.

Though this still exceeds the recommended emptying time of 1.5 days, this may be something which is altered by the addition of the slow drainage system, described in Section 3.4.4.4.5 and would need to be investigated further during the detailed design.

3.4.5.9.8. Calculations for Different Runoff Scenarios

If the leaky was utilised for other runoff types, then different required areas and emptying times are derived. These are summarised in Table 33 These calculations were performed for multiple wells and a single well for each runoff type.

Table 33 Calculations for Different Runoff Types for Leaky Well Designs

Runoff Type	Runoff Coefficient (c)	Rainfall Intensity (I)	Contributing Catchment Area (A)	Time of Concentration (t_c)	Runoff Volume (V)	Diameter and Height of Single Leaky Well (m)	Number of Wells Required if Multiple Wells are Installed	Diameter and Height of Each Well for Multiple Wells	Emptying Time for Single Well	Emptying Time for Multiple Wells
Roof Runoff	0.9	62.2mm/hr	58,100m^2	10 mins	542.1m^3	D = 8.7m H = 9m	78	D = 2.1m H = 2m	395 hours	91 hours
Road and Pavement	0.9	83.6mm/hr	16,220m^2	5 mins	101.7m^3	D = 5.1m H = 5m	15	D = 2.1m H = 2m	226 hours	91 hours
Total Runoff	0.799	43.7mm/hr	79,400m^2	20 mins	924.1m^3	D = 10.3m H = 11m	133	D = 2.1m H = 2m	475 hours	91 hours

The feasibility of each of these options depends on space availability and how much of an impact the slow drainage system implemented in clayey soils has on the emptying time.

3.4.5.10. Possible Location of Leaky Well System

Similar to the soak-away system, a leaky well is also buried so would be practical to implement under the school field, under either of the car parks identified in Section.

3.4.5.11. Cost Estimate

The costs estimated below in Table 34 are for a number of leaky wells, as opposed to one large well. The cost for a standard leaky well could not be obtained; therefore the cost of the same sized tank (2m high and 2m diameter, ie. 7000L capacity) and geotextile liner was used for the estimate.

Table 34 Cost Estimates for Different Leaky Well Designs (Rawlinsons, 2014)

Runoff Type	Number of Tanks Required	Cost of Tanks	Size of Excavation Required	Cost of Excavation	Required Area of Geotextile Fabric	Cost of Geotextile Fabric	Total Cost
Roof Runoff	78	$510,900	$7m^3$x78 tanks = $578m^3$	$8,736	$19m^2$	$4,725	$524,378
Road and Pavement	15	$98,250	$7m^3$x15tanks = $105m^3$	$1,680	$285m^2$	$912	$100,842
Total Runoff	133	$871,150	$7m^3$x133tanks = $931m^3$	$14,896	$2,527m^2$	$8,086	$894,132

The cost of trenching would also be added to this final cost but will be dependent on the location of the leaky wells and whether trenching would be required through residential areas or if it would run parallel to the road way.

3.4.6. Water Quality

It was briefly described in each section that each treatment has a different impact to the quality of runoff and pollutant removal efficiency. This is summarised in Table 35 for each of the WSUD infiltration options considered.

Table 35 Pollutant Removal Efficiency of Each Treatment (WA Government, 2004, Austin.G, 2012), Nillson. E, Stigsson. A, 2012, Dublin Drainage, Environmental Protection Agency, 1999, Sustainable Technologies, 2013 and National Asphalt Pavement Association, 2013

	Litter	Coarse Sediment	Total Suspended Solids (TSS)	Total Nitrogen (TN)	Total Phosphorus (TP)	Heavy Metals	Total Lead (TPB)	Total Copper (TCU)	Total Zinc (TZN)	Fecal Coliform Bacteria	E.Coli
Bio-retention basin	NA	90%	80%	50%	60%	80%	31%	54%	77%	69%	71%
Porous Pavements	NA	NA	92%	83%	68%	NA	74%	42%	80.5%	NA	95.8%
Sedimentation Basin	>95%	>95%	50-80%	20-60%	50-75%	40-70%	NA	NA	NA	NA	NA
Infiltration Trench	NA	NA	80%	60%	60%	90%	80%	80%	80%	NA	90%
Leaky well and Soakaway	NA	NA	60%	NA	NA	NA	98%	NA	54-88%	NA	NA

As seen, the leaky well and soak-away treatments are the most poorly performing of all the options and this is due to the geotextile fabric providing the only means of filtration, and hence due to their large costs , space requirements and poor pollutant removal efficiency should not be deemed feasible.

3.5. Water Harvesting

Stormwater Harvesting is another potential option to promote drainage along North Terrace as it is a popular concept used to enable water reuse through storage, which is essential to water conservation. As such, it is often used alongside other sustainable urban water management systems. Its main characteristics include collection, storage, treatment and distribution of stormwater runoff.

3.5.1. Limitations

On the other hand, it is also important to recognize the limitations of stormwater harvesting systems. Firstly, they are significantly influenced by variable rainfall patterns. This is because storage volume is directly related to the volume of rainfall. In the case of a storm event and/or consecutive number of days with rainfall, designed storage might be inadequate. When rainfall volume is low, harvested stormwater will be reduced and might not meet the usual demands. In an area where rainfall volume is too high, much of the water would go into overflow. Secondly, there is also a risk of infections due to pathogens found in untreated stormwater (Ahmed, Goonetilleke, Gardner, 2009). Thirdly, roof size is a major variable in determining suitable tank size. Regardless of rainfall intensity, roof size is directly proportional to the amount of runoff it can capture.

3.5.2. Rainwater Tanks

Stormwater storage or the use of rainwater tanks as seen in Figure 82 is one of the major techniques implemented to store stormwater. Roof runoff will be directed by a roof drainage system to a rainwater tank. As calculated in flow rate section, roof surface greatly contributes to the paved catchment area at the project site. As a result, by storing roof surface runoff, the total stormwater flow rate can be reduced. When properly integrated with other infiltration techniques such as soak away, it can greatly improve flood mitigation.

In this study, a certain amount of household and mitigation usage will be assumed when calculating the size of rainwater tank.

According to the Department of Environment and Conservation NSW (2006), the stored water can be designed to use in current urban areas and new developments such as:

- Residential uses (toilet flush, garden watering)
- Irrigating public areas (nearby vegetation, school ovals)
- Industrial uses (dust suppression)
- Public or private ornamental ponds and water features
- Aquifer storage and recovery.

In the occurrence of a storm event, runoff of a significant volume from impervious surfaces is rapidly directed towards storm sewers and waterways (Sustainable Stormwater Management 2009).

Consequently, there is a need to reduce this flow rate to prevent potential flooding. Rainwater tanks can be installed as a standalone tool to capture the excessive stormwater volume at a designed rate in order to reduce peak flow. More recently, they have been incorporated with infiltration techniques, for example a soakaway. The system can be designed so that rainwater would be captured in a soakaway and in the event of a major storm event, the excess water will be directed through an overflow pipe into a storage tank. As such, the structure can both reduce runoff volume while the overflow can be stored for reuse purposes as mentioned above.

Figure 82 - Example of underground rainwater tank. Retrieved from: (CapitolGreenroofs, 2015)

3.5.2.1. Advantages of Water Harvesting

The implementation of rainwater tanks comes with many benefits:

- Rainwater harvesting is very cost efficient. The collected water is relatively clean and can be used to replace consumption in home (Department of Environment and Conservation NSW 2006). Consequently, it will contribute to water conservation.
- Stormwater harvesting techniques are flexible and highly customisable. It can be used as a standalone option or in a combined system
- Preventing a considerable amount of rainfall to enter stormwater system thus less pollutants would be able to reach the receiving waters. The quality of stormwater entering First Creek would then be improved.

3.5.2.2. Disadvantages of Water Harvesting

Rainwater tanks however do come with some shortcomings:

- There is a public health risk as the collected water is untreated. The level of metal components in the water is higher than the recommended limits as specified by the Environmental Protection Agency (EPA) thus unsafe for human consumption (refer to C1. Pollutant Concentration Table (France)).
- Depending on the roof size and rainfall intensity, a geotechnical analysis must be conducted to determine whether the existing soil is suitable to accommodate the required tank.
- The efficiency of rainwater harvesting depends heavily on the rainfall volume. In the dry season, the stored water might be inadequate for in-house usage and irrigation.

3.5.2.3. Potential sites and solutions

This study will explore two main options of rainwater harvesting. In the first option, the design will retain all rainfall in the catchment area using one or more customized tanks. In the second approach, the total captured rainwater volume of all properties, assuming each property owns separate sufficient water tank would be calculated as a percentage against the total design flow.

Tank size will be determined using Rain Tank Analyser spreadsheet, namely RAIN TANK.

3.5.3. Option A – One Tank for entire Catchment Area

This water harvesting option explores the possibility of implementing a large rainwater tank in an area in relation to North Terrace. Figure 2 and Figure 83 below shows the identified catchment area. By observing the general land use, it would be clear that the workable sites are very limited. The only viable lot that might fit the criteria is St Peters school oval.

Figure 83 - Catchment area (terrain) (Google©, 2015)

The variables to calculate the corresponding tank for the entire project site are determined based on these assumptions:

- Road and pavement runoff would be contaminated especially during first pour to be reused.
- At this stage, the total irrigation area would be assumed to be area of St Peters' ovals.
- Roof surface would be assumed to be concrete for the purpose of this feasibility study as it is the most critical or the worst case scenario in determining initial loss.
- First flush loss, or the initial surface water runoff in the event of a storm, would be assumed to be 500L due to large tanks (20 times normal tank first flush loss to accommodate unique tank size)
- Soil type is assumed to be sandy loam even though the actual soil in the area is clay.
- Water collected would be used in irrigation only.

Table 36 below shows the capacity of St. Peters oval to store water in terms of available land space or irrigation area. Figure 84 below this table displays the respective lots.

Table 36 - St. Peters oval lot and area (DaftLogic, 2015)

Lot	Area (m^2)
1	21,425
2	14,490
3	5,000
4	18,800
5	32,200
6	16,300
Total	108,215

Figure 84 - St Peter's College Areas (Google©, 2015)

3.5.3.1. Preliminary Design for Option A Tank Sizes

In order to check the functionality of using rainwater tanks, a software package named Rain Tank Analyser (UniSA 2015) is used.

Table 37 below shows the number of variables required to calculate the tank size for the entire roof area. These variables are input into the software to determine a required tank size.

Table 37 - Variables for calculating tank size for entire roof area

Values input	
Total roof area (previous section, m^2)	58100
In house daily demand (L)	0
Irrigation area (m^2)	108,215
Initial loss, critical (concrete, mm)	1.5
First flush loss (L)	500
Plant	Turf
City	Adelaide
Irrigation	Sprinklers
Plant available water(PAW, mm)	16.5
% allowable depletion	50%
Application efficiency	75%
Irrigation depth (mm)	11
Soil Type	Sandy
Root zone depth (mm)	150
Available holding capacity (mm)	110

A suggested tank size for this option can be seen in Figure 85. This shows the tank requirements including size, yield per annum, and number of days where the tank will hold zero water as well as the efficiency of the tank in terms of percentage of total demand.

OUTPUT DATA		
Rainfall input data years	25.2	yr
Average annual rainfall	548	mm/yr
Average annual inhouse demand	0	kL/yr
Average annual irrigation demand	67976	kL/yr
Tank selection		
Suggested tank size is approx.	955333	L
Average annual yield	7583	kL/yr
Aver. number of days with zero supply	193	days/yr
% of total demand supplied by tank	11%	

Figure 85 - Suggested tank size for option 1 (RAIN TANK, UniSA, 2015)

From Figure 85 the suggested tank size is approximately 956 m^3.

This translates into a tank with approximate dimensions of 3 x 18 x 18 m. A representation of this can be seen in Figure 86 below.

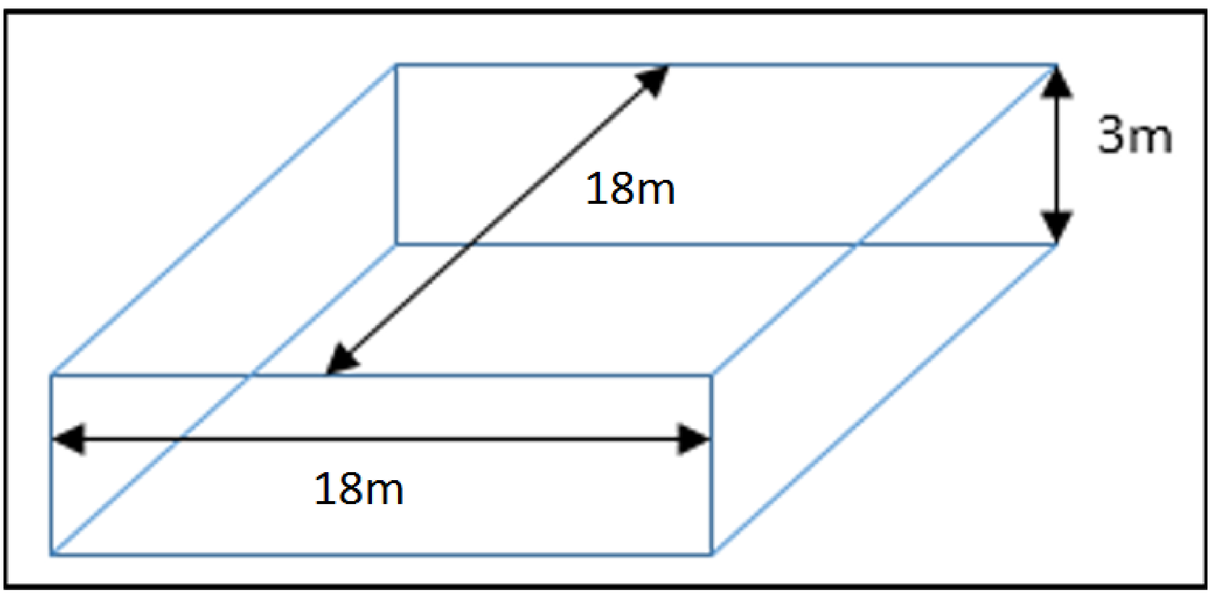


Figure 86 - Required dimensions of large rain water tank. (Hydro Future, 2015)

Required area: $18 \times 18 = 324\ m^2$

Figure 87 below displays the approximate size of the tank in proportion to the first grassed lot of St Peter's College.

Figure 87 - Size of RWT in proportion to St Peter's grassed area (Google©, 2015)

Alternatively, the tank could be buried beneath the Clark Rubber carpark, as shown in Figure 88 . This may eventuate to higher costs than using the school oval due to the cost to backfill, plump system into existing networking along North Terrace and laying down new asphalt. On the other hand, as this is located at lower elevation, it is possible to save cost from excavation and pumping.

These options however present stakeholder concerns due to the requirement to lease the land from Clark Rubber, thereby preventing the landowners from being available to develop the land in the future due to structural concerns. A preliminary figure for the land cost is $2,700, though this will be accurately defined in the detailed design stage.

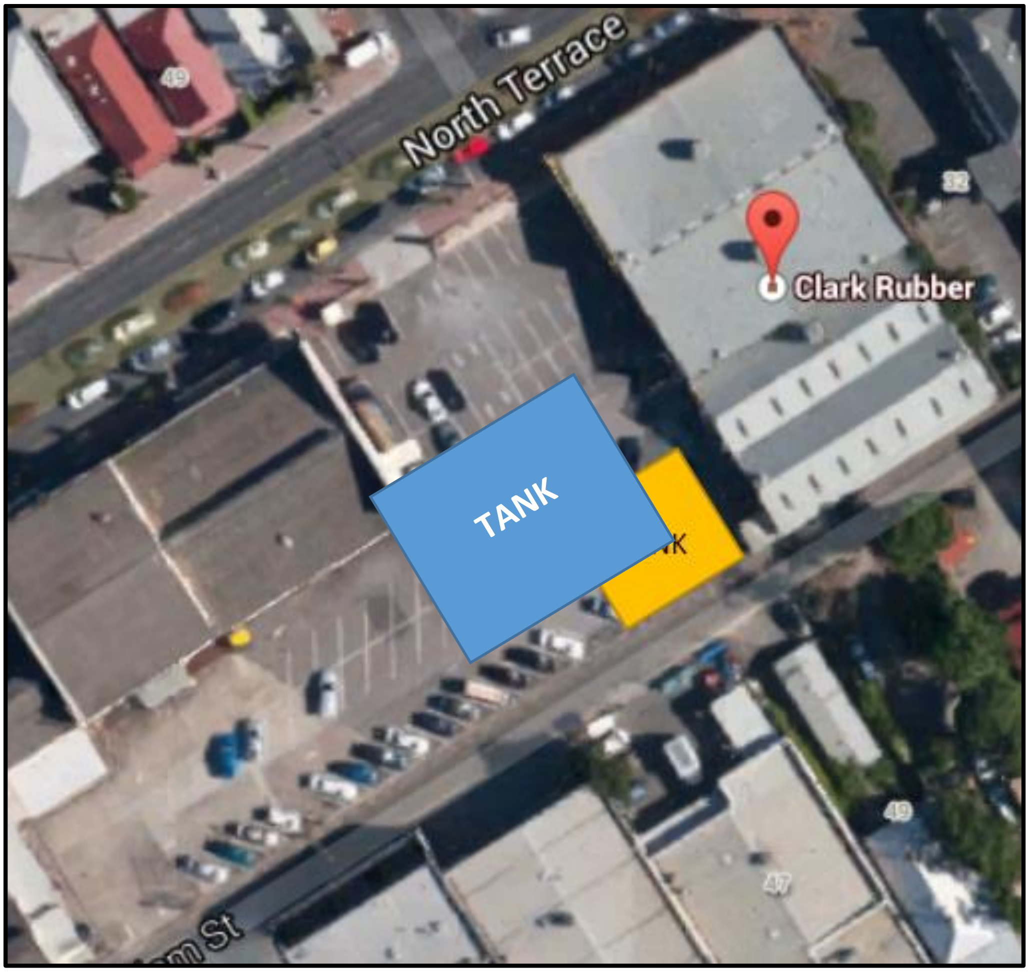


Figure 88 - Potential location in the car park of Clark Rubber for a 13x12x3 tank (Google©, 2015)

As the tank will need to be buried to receive stormwater flows, it will need to be specially constructed to be able to withstand structural loads in either the St Peters College, or Clark Rubber location. Though these loadings will be different and will be accurately calculated in the detailed design section.

In terms of creating a large tank to take the full 956 m^3, a custom unit would need to be constructed on site. Due to sheer size, its structure would have to be similar to that of a large underground basement. The technical expertise and cost that must be invested in this option would be extremely high.

In terms of availability for an off the shelf rainwater tank to meet these needs, the current largest available tank is 50 kL (poly) provided by Team Poly or 11.5 kL (concrete) provided by RainwaterTanksDirect. These 2 are displayed in Figure 89 and Figure 90 below.

Figure 89: Team Poly 50 kL poly tank (Team Poly, 2012)

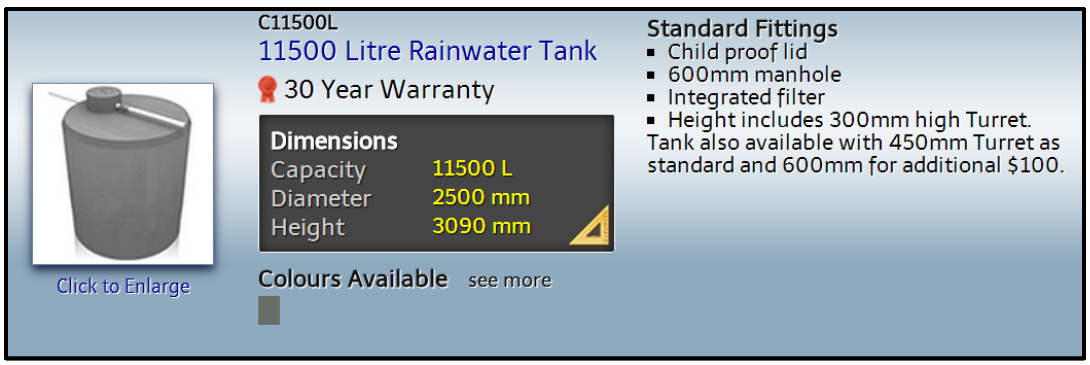


Figure 90: Rainwater Tank Direct 11.5 kL concrete tank (RainwaterTanksDirect, 2013)

As mentioned above, the workable site in the area is very limited and flow grade is also of concerns, these tanks would have to be underground. That makes the precast concrete tank the most appropriate choice.

956,000 = 956 m^3

No. of tanks required:

$$\frac{956000L}{11500L} = 83.13 \approx 84\ tanks$$

84 concrete tanks – 11.5k L per unit

Concrete tank (cylindrical):

D = 2500mm => r = 1.25m => A = 4.9 m^2 -> Total area required is at least

$$4.9 \times 84\ =\ 411.6\ m^2$$

Total cost:

$$\$4{,}150\ each \times 84\ =\ \$348{,}600\ total$$

3.5.4. Option B – Implementation of rainwater tanks in each properties

In this option, the study explores the capacity of rainwater tanks in reducing total surface runoff if every property owner located within the project area is willing to install a rainwater tank on their property, corresponding to their roof size and irrigation area. The collected water can be used for non-potable purposes such as toilet flushing, gardening irrigation or even laundry. The variables to determine the appropriate tank for each properties are made up from these assumptions:

- Total number of properties in project area: 78 or equivalent
- Average irrigation area would be assumed to be: 100 m^2
- Assumed household daily demand (assuming 3 people): 470 L
- Roof size: 100 – 500 m^2
- First flush loss for each tank: 50 L (twice normal to account for critical situations)

3.5.4.1. Preliminary Design for Option B

Table 38 below displays these variables along with other requires parameters to be in put into Rain Tank Analyser (UniSA 2015) to determine the required tank capacity for option B, depending on roof size

Table 38 - Variables for calculating corresponding tank size for each roof size

Values input	
Total roof area (previous section, m^2)	100-500
In house daily demand (L)	470
Irrigation area (m^2)	100
Initial loss, critical (concrete, mm)	1.5
First flush loss (L)	50
Plant	Turf
City	Adelaide
Irrigation	Sprinklers
Plant available water(PAW, mm)	16.5
% allowable depletion	50%
Application efficiency	75%
Irrigation depth (mm)	11
Soil Type	Sandy
Root zone depth (mm)	150
Available holding capacity (mm)	110

Figure 91 below shows the linear relationship between tanks and roof size.

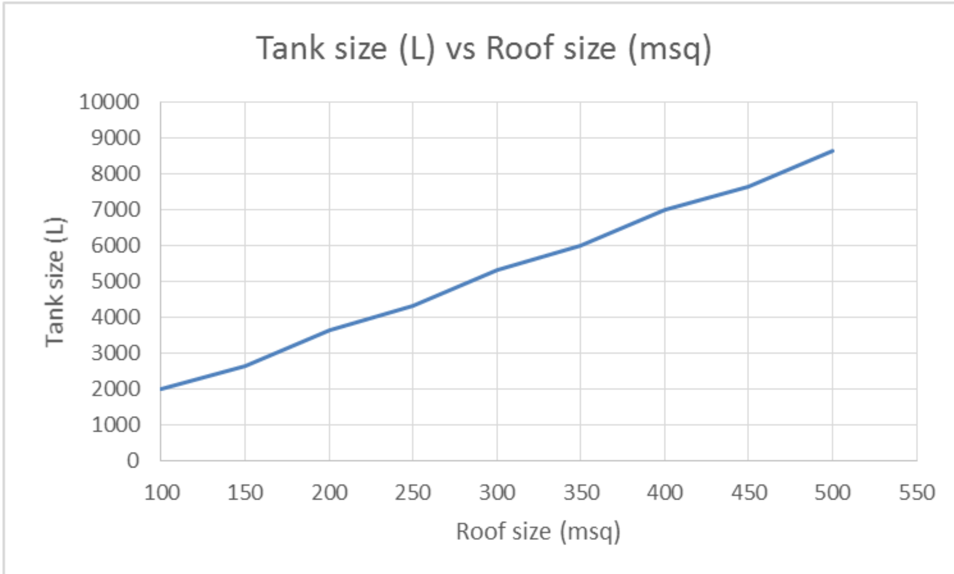


Figure 91: Graph of tank size (L) against corresponding roof size (m²). (UniSA 2015)

At this conceptual stage, a tank would be implemented per 200 m^2 roof to simplify working procedure. According to the graph, the minimum tank size for 200 m^2 is 3667 L. This translates into an on-the-shelf size of 3800 L concrete tank as they are cheaper than poly tank (RainwaterTanksDirect, 2013).

Figure 92: Rainwater Tanks Direct's 3800 L concrete rainwater tank (RainwaterTanksDirect, 2013)

$$Total\ cost: 78 \times \$2750 \ = \ \$214{,}500$$

$$Total\ Volume: 78 \times 3800\ L = \ 296{,}400\ L$$

Total flow rate for roof only:

$$Q_{roof} = Q_{total} - Q_{road\ and\ pavement} = 1.09 - 0.32 = 0.77 m^3\ /s$$

Total volume generated by the storm, 20 years ARI

$$Vol = Q \times t_c = 0.858 \times 10 \times 60 = 462 m^3$$

Total capacity of rainwater tanks:

$$\frac{296{,}400\ L}{462000\ L} \times 100 = 64.07\%$$

Estimated number of small tanks needed to capture all roof runoff: $122\ tanks \approx \$335{,}500$

Conclusively, for the scope of our project, rainwater tanks would not be used as a standalone solution. A better solution would be integrating the water harvesting system along with other stormwater management systems such as water sensitive urban design and infiltration options. This idea is conveyed in Figure 93 below. The next option would explore the feasibility of this approach. This system however, is very complex and has to be designed sub-catchment by sub-catchment. If deemed feasible, a detailed design would be done in the later stage.

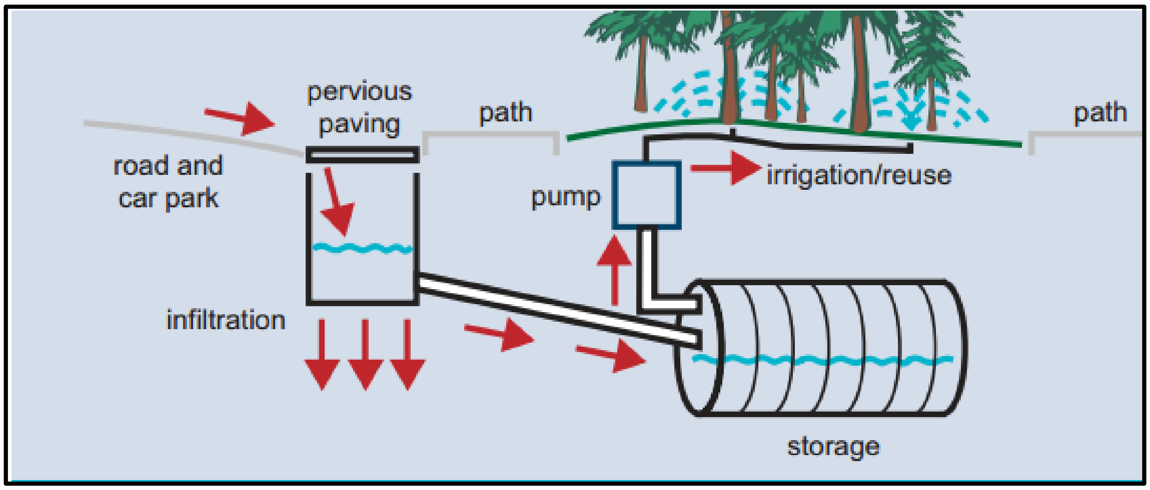


Figure 93: Example of integrated rainwater tank and infiltration system. (Department of Environment and Conservation NSW, 2006)

3.6. Combined Drainage Design

The combined option discusses the design of combining a number of different WSUD/Infiltration and water harvesting options to create a system that together they meet the required demands. The advantage of implementing a system like this, is we can chop and change options based on their individual feasibility performances. This option will allows more flexibility in cost and provides more options to limit the amount of space the infrastructure consumes.

The WSUD and infiltration options listed in Section 3.4 have all been designed according to a 1 in 5 year average recurrence interval. Yet, to avoid any risk of flooding during major storm events (1 in 20 years ARI); it is advised that a conventional drainage system should be used in combination with the WSUD/infiltration and water harvesting systems. In a minor storm event the WSUD and water harvesting system(s) will target improving the quality of the drained water and/or provide a beneficial use for the rain water rather than treating it like waste. However, in a major storm event reducing the flooding risk will be a higher priority than improving the general water quality. Therefore, the presence of a conventional stormwater drainage system could ensure that this risk is more manageable compared to WSUD system alone.

In this design project the calculated total runoff flow in a 1 in 20 years ARI scenario, used in designing the conventional stormwater system is not extremely different from the total flow of 1 in 5 years ARI used in WSUD systems. This is due to differences in times of concentration and rainfall intensity for both scenarios. The total runoff flow used for conventional system is 1.36 m^3 /s versus 1.09 m^3 /s for the WSUD Flow. This indicates that if a WSUD system is implemented to accommodate the calculated WSUD flow, in a major storm event only a very small amount of overflow will occur.

Table 39, summarises most of the WSUD, infiltration and water harvesting feasible options analysed in the report. It also identifies where the stormwater will be collected from, whether that is from the roof only or pavement and road or both. This is to ensure that all the water present in the design area is collected and/or treated. It also summarises the suggested and most feasible locations or sizes to implement each of the options as discussed in their relevant sections (Sections 3.2, 3.4 & 3.5). For these available or suggested areas, the capacity (in volume or runoff flow) that each system can carry is stated as calculated in the respective sections. Their capacities were then compared to the calculated required runoff flow for the WSUD. The associate cost for each of the options is also stated.

Table 39: Summary of feasible options

Feasible WSUD Option	Runoff collected from	Exact Location/ Dimensions and number	Discussion	Capacity (Run off Value m^3/s Or Volume)	Contribution to Total WSUD Run off (%)	Total Cost
Pervious Pavement	Road and pavement only	 (Google Maps, 2015)	• The best 2 suggested feasible locations for the implementation of pervious pavement as discussed in Section XX	$0.0476m^3/s$	• 14.7% of Road and pavement runoff • 4% of total runoff	**$365,628**

Feasible WSUD Option	Runoff collected from	Exact Location/ Dimensions and number	Discussion	Capacity (Run off Value m^3/s Or Volume)	Contribution to Total WSUD Run off (%)	Total Cost
Bio-Retention Basin	Road and pavement only	 Area that could be used for bio retention basin (lot 103) (Google Maps, 2015)	• These 2 locations combined fulfil the water quality treatment requirements for road and pavement run off.	0.332m^3/s	• 100% of Road and pavement runoff • 30% of total runoff	**$21,000**

Feasible WSUD Option	Runoff collected from	Exact Location/ Dimensions and number	Discussion	Capacity (Run off Value m^3/s Or Volume)	Contribution to Total WSUD Run off (%)	Total Cost
		 Area in front of St Peter's College (Google Maps, 2015)				

Feasible WSUD Option	Runoff collected from	Exact Location/ Dimensions and number	Discussion	Capacity (Run off Value m^3/s Or Volume)	Contribution to Total WSUD Run off (%)	Total Cost
Rain Water Tank	Roof only	 (Google Maps, 2015)	• Calculations using software estimation showed that a 3800 L concrete tank / 200 m^2 area of roof could be used • To collect all the water coming from the roof approximately 122 tanks are needed. • Cost of 122 tank is extremely high and there is no available location to place them • 20 tanks of 3800 L (or equivalent) capacity is the suggested as the most feasible choice and is used in following calculations *This could be altered in detailed design stage*	76,000L	• 16.4% of roof runoff • 11 % of total runoff	**$55,000**

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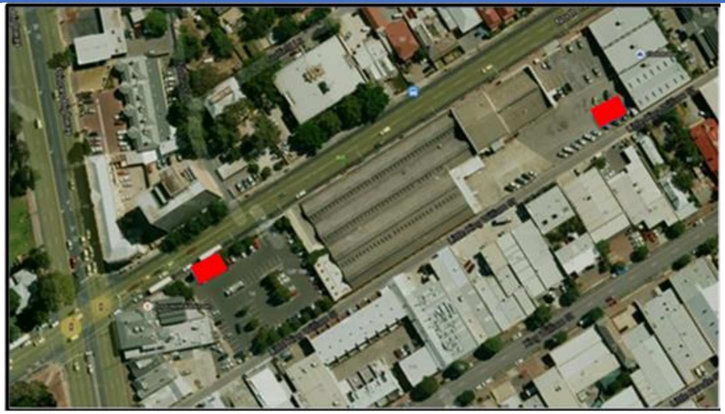
Feasible WSUD Option	Runoff collected from	Exact Location/ Dimensions and number	Discussion	Capacity (Run off Value m^3/s Or Volume)	Contribution to Total WSUD Run off (%)	Total Cost
Sedimentation Basin		 Sedimentation Basin could be placed in either of the 2 car parks, preferably in the car park situated further downstream (Google Maps, 2015)	• If placed in either of the car park areas, a $50m^2$ basin can handle the total runoff from the road and pavement.	$0.332m^3/s$	• 100% of Road and pavement runoff • 30% of total runoff	**$148,850** (not including maintenance cost)
Infiltration trench using **Slow Drainage System**	Road and pavement	 (Google Maps, 2015)	• Along edge of school area is the only place where there enough room for the width of the trench • If 220m long can utilise all runoff from road and pavement which is achieved in the highlighted area	$101.7m^3$ runoff volume	• 100% of Road and pavement runoff • 30% of total runoff	**$609,180** (assuming land acquisition is $2,700 per square metre)

Feasible WSUD Option	Runoff collected from	Exact Location/ Dimensions and number	Discussion	Capacity (Run off Value m^3/s Or Volume)	Contribution to Total WSUD Run off (%)	Total Cost
Leaky Well using **Slow Drainage System**	Roof, road pavement	• Not feasible – high cost and exceeds emptying time and space requirements. • Also pollutant removal efficiency is not as high as other treatment options.				
Soak Away using **Slow Drainage System**	Roof runoff	• Not feasible – high cost and exceeds emptying time and space requirements. • Also pollutant removal efficiency is not as high as other treatment options. • Would also require 35m widex35m long x1m depth excavation and possible trenching through residents homes which is not feasible.				

As summarised in Table 39, it can be seen that there is more than one option that can accommodate all the run of from pavement and road. However, there are no feasible options that can accommodate all the runoff from roofs. The only feasible option that can accommodate part of the roof runoff is Water Harvesting Tanks. Through negotiations with relevant stakeholders via Urban Design Team, the number of tanks that could be installed can possibly increase.

Comparing the options that can carry full capacity of road and pavement runoff; it can be seen that Bio-retention basin or sedimentation basin are the best option for road and pavement run off. Considering the construction price and maintenance price; Bio-retention basin will be the cheapest option.

This option represents the cheapest and most feasible combination of all analysed technologies. As discussed in Section 3.6, Bio-retention basin and water harvesting tanks have a total capacity to carry 41% of the total WSUD flow. This could increase in the detailed design stage after negotiations with relevant stakeholders to increase number of water harvesting tanks (Section 3.5). To accommodate the rest of the water flow and to minimise the risk of flooding in the case of a major storm even , a conventional stormwater drainage system is to also be combined with this option (as shown in Figure 94). These dimensions will be defined during detailed design.

Figure 94- Combined Option Locations of Infrastructure

Table 40 - Combined Options Price

Element	Conceptual design Size	Cost
Bio-retention Basin	$150m^2$	$21,000
Rain Water Tanks	20*3800L tank (or equivalent)	$55,000
Conventional Stormwater drainage system size.	as calculated in Table 21 (900mm diameter pipe).	$155,396
	Total	≅ **$232,000**

3.7. Recommended Design

The final design option has been decided by the Hydro-Future Consulting Management Team through our decision making matrix as introduced in Section ? and seen in Appendix B – Water Decision Making Matrix

Table 41 below shows the results for the five explored options.

Table 41 - Final Decision Making Matrix Results

Final Decision Making Matrix Results					
	Cost (35%)	Flood Mitigation (35%)	Quality (20%)	Amenity (10%)	
Options	**Rank (%)**	**Rank (%)**	**Rank (%)**	**Rank (%)**	**Total**
Upgrade	5	5	1.33	2	13.33
Swale	3	1	2.66	2.33	8.99
WSUD	1.33	3.66	4.66	4	13.65
Water Harvesting	2.66	4.33	1	2.33	10.32
Combined	**4**	**4.66**	**4**	**3.66**	**16.32**

As can be noted from this table, the combined option received the highest votes with a total score of 16.32. The combined design option has been chosen based on its overall advantages in terms of cost, high drainage performance, visual amenity improvement and high water quality control measures.

4. Structural Engineering Design Options

Hydro-Future aims to effectively incorporate First Creek, by means of utilising either the stone arch or box culvert, to create a modern stormwater drainage design whilst placing a strong prominence on heritage, culture, cost, constructability and water efficiency. A strong emphasis has been placed on the sandstone arch culvert, if used, which will require detailed structural analysis using strength parameters from old construction materials which have undergone significant decay over the past 150 years. This section of the feasibility study will address issues on how to effectively connect a new stormwater pipe into the sandstone arch culvert without compromising its structural integrity.

4.1. Arch Culvert

4.1.1. Concerns

Hydro-Future has document expected damage which is likely to occur within a structure which is 150 years old. However Hydro-Future identifies that the current damage of the culvert is not structurally related, but appearance related. The age of the structure will cause significant signs of decay along with the vandalism that has occurred but it is unlikely the culvert is not structurally sound in its current condition.

However Hydro-Future has identified several viable options to improve the appearance and damage to the structure which will ultimately increase the culverts structural capabilities throughout the projects design and construction, as seen in section 4.1.2.

4.1.2. Repair Methods

The current structural integrity of the culvert is of some concern. Ultimately, it is quite difficult to analyse the structural strength of the sandstone culvert due to age and outdated construction, therefore it is assumed that the structure should be supported or repaired via one of the following methods:

4.1.2.1. Temporary construction frame:

The temporary construction frame will remain inside the arch culvert until the box culvert has been installed and associated works completed. At this point the temporary construction frame will provide the support necessary to keep the culvert structure from collapsing. After construction is complete, the temporary support system will be removed (Noll, J. & Westrich, M., Spet 2008).

4.1.2.2. Rehabilitation and relining method:

Rehabilitation and relining of the sandstone culvert includes the installation of a slipline, as seen in *Figure 95 - Sliplined culvert* , by installing a new internal pipe/section inside the existing culvert. The sliplined carrier culvert conforms to the shape of the existing section and in addition smoothens out the profile of the culvert to maximize the flood flow capacity as much as possible (Noll, J. & Westrich, M., Spet 2008).

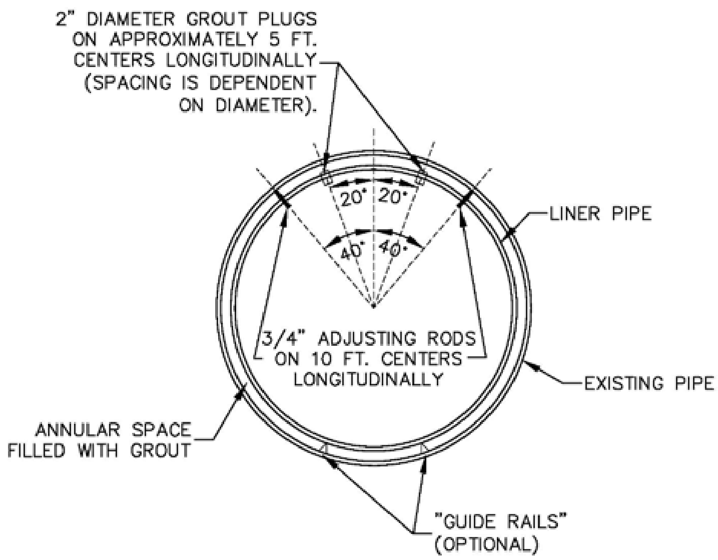


Figure 95 - Sliplined culvert (Noll, J. & Westrich, M., Spet 2008).

4.1.2.3. Spot patch and repair:

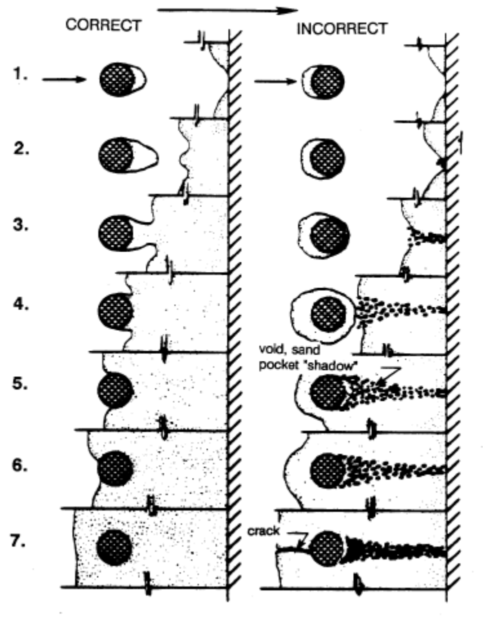
A common rehabilitation method utilized to improve the structural integrity of culverts involves the localized repair of culvert walls using coatings of concrete to address issues with spalling concrete mortar (Noll, J. & Westrich, M., Spet 2008).

4.1.2.4. Applying Shotcrete lining

This rehabilitation option utilizes shotcrete i.e. compressed air applied concrete. These linings are typically 100 to 200 mm in thickness and provide a dense and resistant; structural strengthening material for the culvert section. In addition to shotcreting the interior of the culvert, sections of reinforcement may be added before shotcreting to improve the structural strength even further (Noll, J. & Westrich, M., Spet 2008).

U.S. Department of Transportation (1995) details information in regards to culvert repair via shotcrete method. Steel reinforcement that consists of mesh or reinforcing bars must utilize the procedure illustrated in *Figure 96*, which assures that proper interaction of reinforcement, shotcrete and the existing structure will occur.

NOZZLE DIRECTION AND FLIGHT OF MATERIAL



11.6. Correct and incorrect method of encasing reinforcing bars.[1]

Figure 96 – Shotcrete Application (U.S. Department of Transportation, May 1995)

4.1.3. Design Options

This section covers the details and feasibility investigation of the 3 designs that aim to reduce the impact to the structural integrity of the sandstone arch culvert through utilisation of stormwater drainage connections.

- Option 1 – No alterations (use existing system)
- Option 2 – Direct Connection of the proposed drainage pipe into the Sandstone Arch Culvert underneath North Terrace.
- Option 3 – Direct Connection of the proposed drainage pipe into existing box culvert through an easement on underneath the Royal Hotel Carpark.

Each of the design options will be predominantly evaluated on, but not limited to the following points:

- Impact on the existing box or arch culverts structural integrity.
- Impact on the existing box or arch culverts aesthetics e.g. historical and cultural appearance of the current design.
- Cost effectiveness.
- Maintenance of the design.
- Simplicity of the design and construction.
- Efficient flood mitigation along North Terrace

Figure 97 shows the approximate location of the culvert based off the information that has been provided to the Hydro-Future. The drawing utilises council documents along with the '*Survey of North Terrace'* (Allsurv Engineering Surveys, Dec 2010) drawing, both these documents were provided by Tonkin Consulting. Figure 97 is utilised throughout each design option in order to assist with the feasibility study.

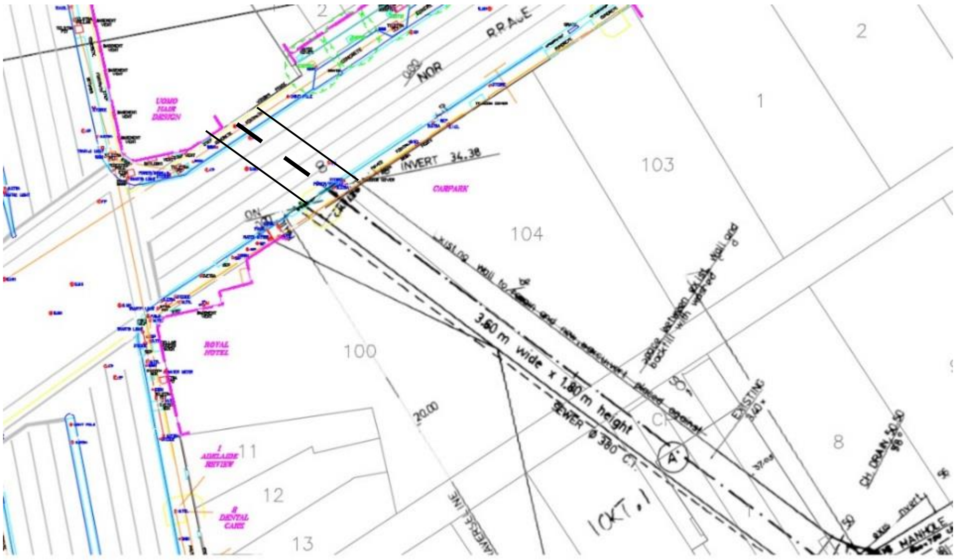


Figure 97 – Existing Culvert Location (Hydro-Future, 2015), (City of Kensington and Norwood, Oct 1993) & (Allsurv Engineering Surveys, Dec 2010)

4.1.3.1. Option 1

The first design option which Hydro-Future has investigated is to leave the existing stormwater condition untouched, this will prevent any design and construction associated with the project. Although this option is the most convenient and cost effective it poses several design risks including:

- The ongoing flooding along North terrace will continuously pose a risk to the transport and infrastructure systems associated within the projects region, indicating the current design stormwater components are inadequate.
- The drainage within this area has been simplified due to the limited access to a natural waterway which would not be utilised to its full capacity if this option was undertaken.

Hydro-Future has also recognised several design advantages associated with this design option including:

- This option will not require any further design which will reduce the cost substantially.
- Construction will not occur which will allow the current transport, commercial business and pedestrian aspects associated with North terrace to remain undisrupted.
- No design or construction budget is required for this option.

4.1.3.2. Option 2

This design option involves connecting the proposed stormwater pipe directly into the sandstone arch culvert which will allow the design flow to be transferred to First Creek directly beneath North Terrace.

Hydro-Future has identified two locations along the arch culvert where the storm water pipe may be connected:

- Connecting the stormwater pipe directly into the side of the culvert (vertical region).
- Connecting the stormwater pipe into the roof of the culvert (arched region).

Several methods have been further listed which may reduce the stress on the arch culvert:

- Connection to the arch using two pipes to reduce the pipe diameter.

- Using a structural, stainless steel grate for the water to pass through at the connection point to increase the rigidness of the connection.

4.1.3.2.1. Location

As seen below in *Figure 98*, is the proposed stormwater pipe (highlighted in red) that will connect into the sandstone arch culvert which passes underneath North Terrace.

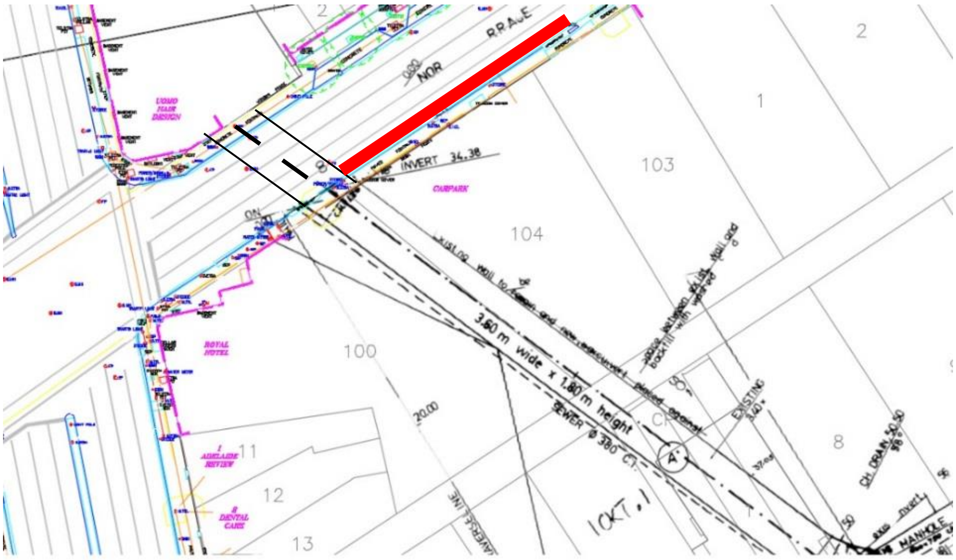


Figure 98: Stormwater Pipe - Arch Culvert Connection (Hydro-Future, 2015), (City of Kensington and Norwood, Oct 1993) & (Allsurv Engineering Surveys, Dec 2010)

4.1.3.2.2. Advantages & Disadvantages

Advantages:

- Stormwater pipe is ran only underneath North Terrace which increases the simplicity of the design.
- The stormwater pipe route is completely straight which increases the design and efficiency of this project.
- There is no requirement to excavate under commercial or residential property which reduces the scope of the project significantly in terms of cost and time.
- Construction time may be reduced depending on the ease of construction
- The cost may be lowered depending on the relative ease of construction.

Disadvantages:

- The sandstone arch culvert may pose design and construction delays due to extensive safeguards which will be required to prevent damage to the heritage infrastructure.
- The sandstone arch culvert was not designed to allow a connection from stormwater pipes, unlike the modern box culvert.

- Due to the associated cultural and heritage concerns this option may raise issues in the community.
- The sandstone arch culvert has a complex shape and design, as opposed to the modern box culverts.

4.1.3.2.3. Preliminary Design Drawings

Figure 99 below shows the stormwater pipe connecting into the side wall of the arch culvert and *Figure 100* shows the stormwater pipe connecting into the arch culverts roof. Both figures include the preliminary dimensions which may change through further inspection.

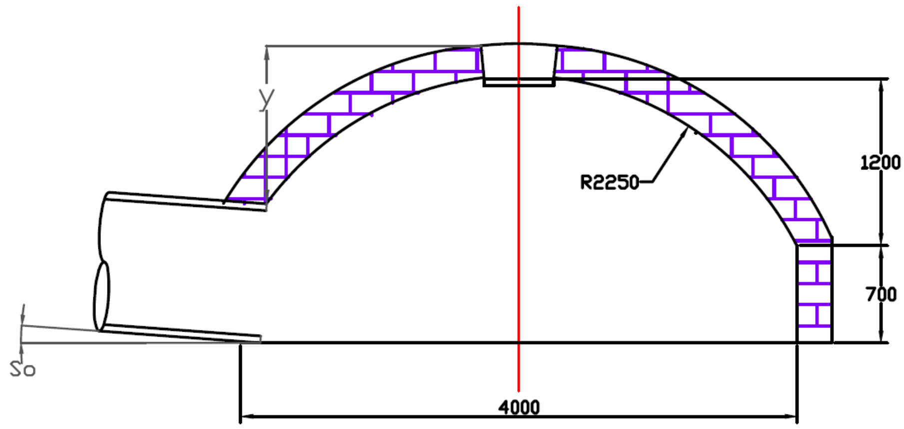


Figure 99: Side Wall Connection (Hydro-Future, 2015)

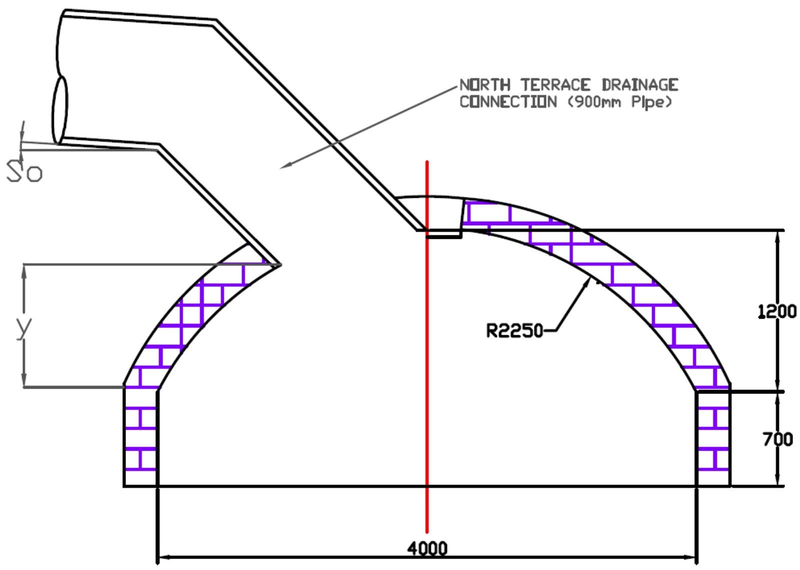


Figure 100: Roof Connection (Hydro-Future, 2015)

4.1.3.2.4. Advantages and Disadvantages of Side Wall Connection

Disadvantages:

- This connection configuration will require extra excavations to be completed in comparison to all other proposed connection configurations as this pipe connection requires deeper excavations.

- Connection pipe may degrade the structural integrity of the old sandstone culvert structure.

- Even though the location of the stormwater pipe facilitates load transfer and reduction of stress throughout the structure, a 900mm void in the culverts side wall will still have a substantial effects on weakening the localised area and increasing the stress concentration around the pipe.

- The structural integrity of the arch culvert must be analysed at the point of insertion with the drainage pipe. This analysis may require the utilisation of a computer assisted package, such as SpaceGass or STRAND7 to correctly calculate the stress and loading distributions that exist at this entry point.

Advantages:

- An advantage to this option is that the connection is not destabilising the actual arch component of the culvert section

- This option improves the culverts structural integrity as the reinforced concrete, which is required to reduce the load on the pipe, will transfer vertical loads from the sandstone arch culvert and into the ground as seen below in *Figure 101*.

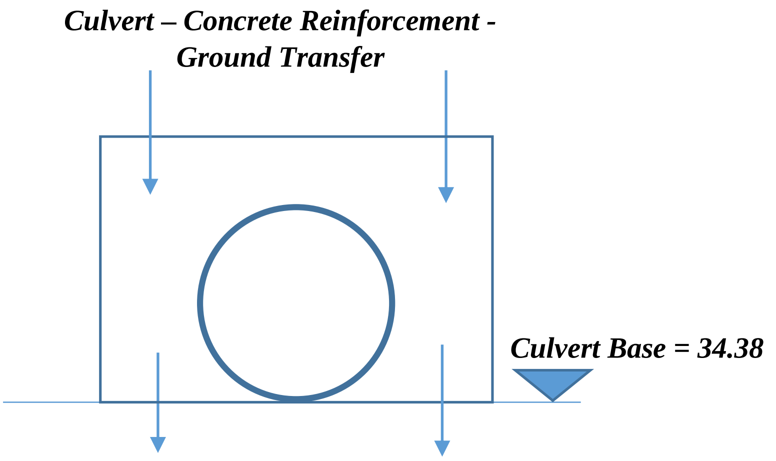


Figure 101: Load Transfer (Hydro-Future, 2015)

4.1.3.2.5. Advantages & Disadvantages of Roof Connection

Disadvantages:

- Connection of the pipe may degrade the structural integrity of the old sandstone culvert structure.

- The 900mm void in the culverts top arching wall will have a substantial effect on weakening the overall integrity of the arch culvert, there is a high chance due to the construction of arch culverts that this option will cause collapse.

- Due to this design the excavation, drilling and construction with respect to the configuration will occur directly on top of the arch culvert. Undertaking these activities on top of the sandstone culvert may easily reduce its strength which may lead to further damage and structural weakness of the culvert.

- The structural integrity of the arch culvert must be analysed at the point of insertion with the drainage pipe. This analysis may require the utilisation of a computer assisted package, such as SpaceGass or STRAND7 to correctly calculate the stress and loading distributions that exist at this entry point.

- Due to the location of this connection the stormwater pipe will not support the culvert nor will it transfer loadings into the ground as opposed to configuration 1

- A diagonally cut concrete stormwater pipe may also expose the concrete inner materials if left unsealed properly

Advantages:

- The advantage of this design is that the work to excavate is reduced as the stormwater pipe no longer needs to be lowered as it will be connected to the roof of the sandstone culvert instead of the side.

4.1.3.2.6. Evaluation

Upon a site investigation and preliminary study of the arch's construction it is recommended that the stormwater pipe is connected into the sandstone arch culverts side wall to prevent collapse of the structure.

4.1.3.2.7. Structural Impact

If the proposed stormwater pipe is to be connected into the sandstone arch culvert the connection will act as a void in the culvert with a diameter equivalent to the pipes diameter. This situation will create abnormal stresses similar to those seen below in *Figure 102*, Hydro-Future will use finite element analysis software to determine the stresses throughout the culvert if the Option 2 is considered for a detailed design.

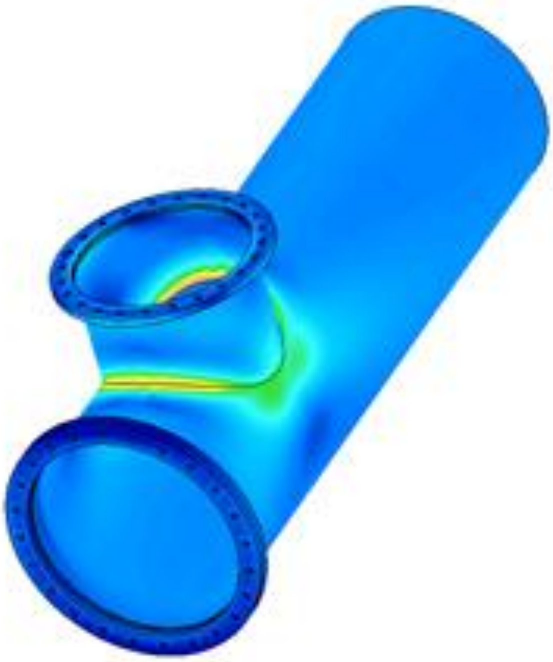


Figure 102: Stress Concentrations (Applied Technical Services, n.d.)

4.1.3.2.8. Stress Reduction

To effectively reduce the stress in the sandstone culvert the connecting point needs to be strengthened or the pipe diameter needs to be reduced, both options have been considered for the project.

4.1.3.2.9. Original Design

This option involves connecting the designed drainage pipe directly into the sandstone arch culvert with no modification. This option is cost efficient and has greater design and construction simplicity. However using the designed pipe diameter will increase the stress throughout the arch as the void at the point of connection will be larger.

4.1.3.2.10. Reduced Pipe Diameter

As seen below in Figure 103 and Figure 104, there are 2 arrangements which can be used to reduce the pipe diameter and keep the same design flow capacity of the pipe. Using 2 pipes at the point of connection will spread the void and effectively reduce the stress concentrations associated with the connecting area. Each arrangement effectively creates the same effect but requires different materials to construct.

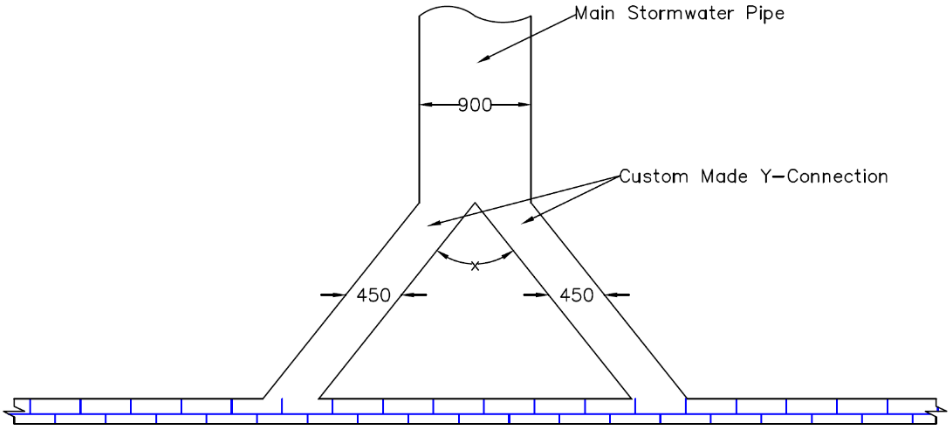


Figure 103: Arrangement 1 (Hydro-Future, 2015)

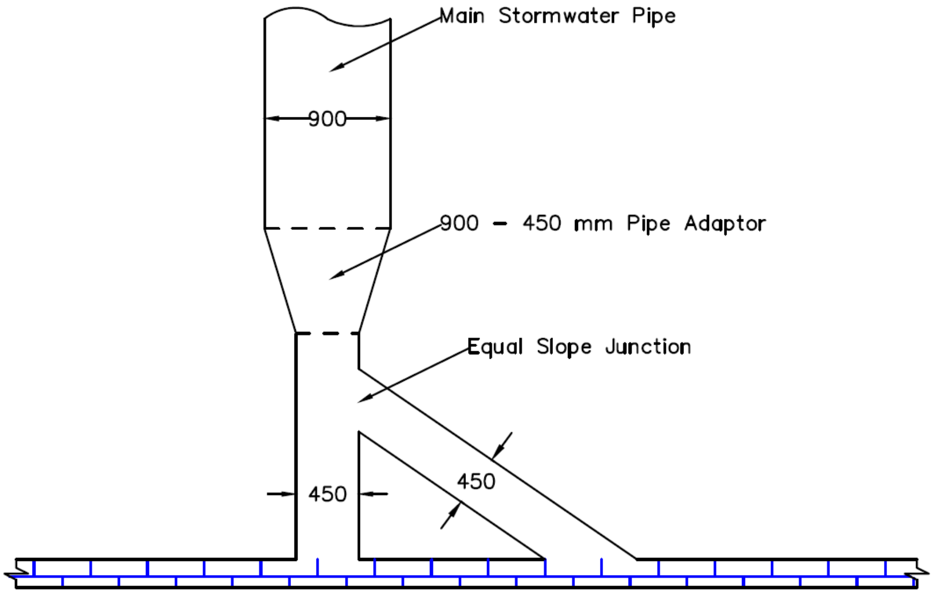


Figure 104: Arrangement 2 (Hydro-Future, 2015)

4.1.3.2.11. Advantages and Disadvantages of Multiple Pipes

Advantages:

- This design modification will aim to reduce the structural impact on the culvert by reducing the void size and to separate or spread the stress concentrations over two smaller locations.

- Having two smaller voids in the sandstone culvert will reduce the amount of stress concentration but will distribute the stress concentration over 2 regions ultimately increasing the culverts structural stability.

- The structural stability should be effectively reduced in this configuration as the stress and voids of the connecting point have been spread over a greater distance.

Disadvantages:

- This option is likely to produce high excavation costs as the excavation trench which will be constructed to install the stormwater piping will need to be widened significantly to install both 450mm stormwater pipes on the required angle at the culvert connection. This may require digging up the road laterally (to traffic flows) as well as longitudinally which may conflict with the following:

- The number of lanes which can carry traffic may be reduced as the widening of excavation is occurring.

- May render the pedestrian walkway or footpath unusable due to the widening of the excavation.

- Analysis using STRAND7 will still need to be required to ensure the stress concentrations at each pipe have been minimised to an acceptable level

- Is likely to encounter utilities conflict if any other services are running parallel to the stormwater pipe.

4.1.3.2.12. Steel Grate

This option aims to increase the rigidity of the connection between the stormwater pipe and the sandstone culvert with the utilisation of a stainless steel grate as shown in Figure 105. The steel grate will be concreted into the sandstone culvert which will increase the rigidity of the void and allow to load to be transferred into the grate and into the ground as shown in *Figure 106*.

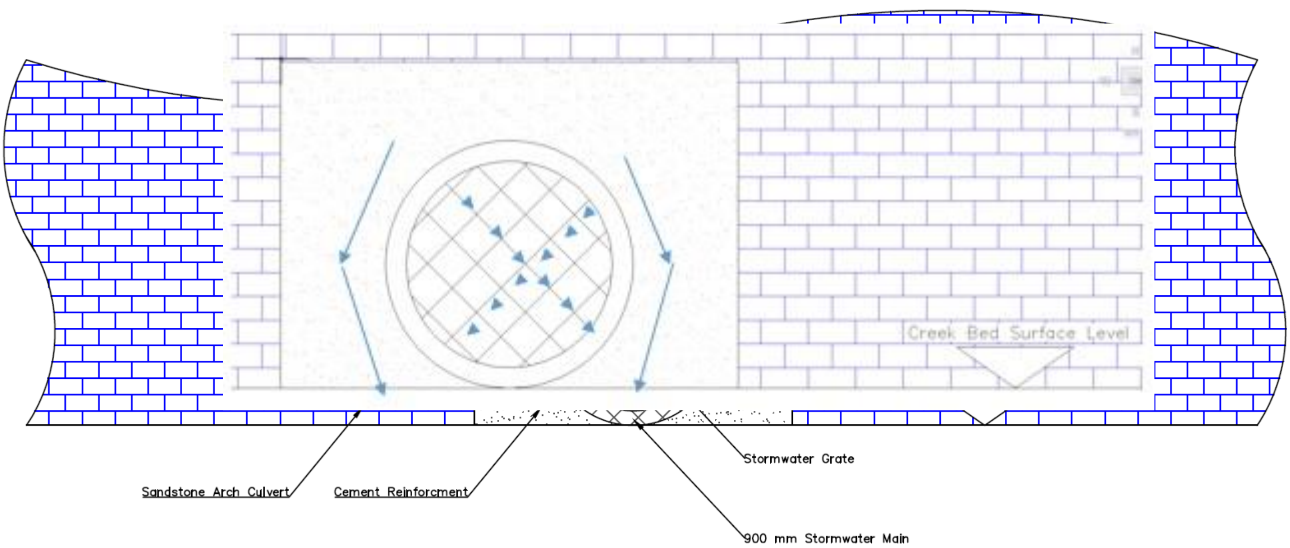


Figure 105: Steel Grate (Hydro-Future, 2015)

Figure 106: Load Transfer (Hydro-Future, 2015)

4.1.3.2.13. Advantages and Disadvantages of a Steel Grate

Advantages:

- The use of this design will increase the structural stability of the localised area. The sandstone around the culvert will be replaced with cement which increases the strength of the region and its capacity for stress concentrations

- The 900 mm void which is created by the stormwater pipe is reduced significantly due to the steel grate which will allow loads and stress to be transferred through the grate as well as around it which will ultimately increase the effectiveness of load transfer and reduce the stress concentrations in the localised area.

Disadvantages:

- Construction and design costs will likely increase as the design will require structural analysis.

- The cost will increase due to the material requirements of using stainless steel to prevent corrosion.

4.1.3.2.14. Evaluation

Each of the above listed options which includes reducing the pipe diameter by using two drainage pipes or cementing a steel grate into the culvert will likely decrease the stress on the sandstone culvert. However they are likely to increase construction and design costs. The sandstone arch culvert has shown no signs of severe structural decay and connecting a single drainage pipe directly into it will decrease costs associated with design and construction. Hydro-Future recommends using a single drainage pipe to connect directly into the sandstone arch culvert.

4.1.3.2.15. Work to be completed in the Detailed Design

This design will require several of the following stages to be completed during the design phase of the project.

1. **Excavation Design** – Location and costing of the required excavation along the full length of the proposed stormwater drainage pipe including trench stability.
2. **Existing Arch Culvert Analysis** – Detailed design showing the stresses which are likely to occur around the connection point of the arch culvert and how the structure will react.
3. **Arch Culvert protection** – Design to protect the sandstone arch culvert from undergoing damage during excavation and increasing the design life post construction.
4. **Connection Design** – Design of a suitable connection into the sandstone arch culvert to allow for flexibility, rigidity and waterproofing.
5. **Detailed Design Loading** – Traffic, dead, live and earth loads on the associated components of the proposed deign option.

4.1.3.2.16. Design Loads

Preliminary design loads which include earth pressures on top of the sandstone arch culvert have been investigated and determined using the appropriate values from geotechnical analysis. The sandstone arch culvert is located near Hackney Road which places the underground infrastructure within the alluvial soil deposit which will be calculated for as follows:

$Culvert\ Base\ IL \approx 34.38\ m$, (City of Kensington and Norwood, Oct 1993)

$Surface\ IL \approx = 37.22\ m$, (City of Kensington and Norwood, Oct 1993)

$Culvert\ Height\ \approx 2.13\ m$

$Depth\ to\ Culvert\ Roof \approx Surface\ IL - Culvert\ Base\ IL - Culvert\ Height$

$Depth\ to\ Culvert\ Roof \approx 37.22 - 34.38 - 2.13 = \ 0.71\ m$

As these measurements are considered estimates by Hydro-Future. The culvert depth will be approximated as 1 meter for a conservative loading values. As there is only 1 soil layer between the depths of 0.0 – 1.0 meters the vertical earth pressure on the retaining wall is equal to.

$\gamma = 21.5\ kN/m^3$

$z = 1.0\ m$

$\sigma_v = \gamma z = 21.5 * 1.0 = 21.5\ kPa$

The pressure which is acting on the roof of the sandstone arch culvert has been determined to be *21.5 kPa.*

4.1.3.2.17. Costing

Costing for this design has been based on the approximate materials required which will compromise of concrete, and steel to reinforce the connection between the sandstone culvert and the drainage pipe as shown in Table 42.

Table 42: Option 2 Costing (Rawlinson, 2014)

Material Costing				
Description	*Units*	*Quantity*	*Rate*	*Cost*
Construction Site Preparation				
Precast Slab - Bridge Deck (Assumed) – 40MPa Concrete	(m^3)	2.4mx0.46mx2.26m = 2.495 ≈ 2.5 m^3	\$243.00	\$607.5
Reinforcement	t	0.3	\$1850.00	\$555.00
TOTAL				\$1157.5

4.1.3.3. Option 3

Option 3 involves connecting the proposed stormwater pipe to a concrete box culvert which runs underneath the commercial areas of North Terrace, as seen below in Figure 107. The concrete box culvert was constructed to deal with loads due to the commercial infrastructure and was developed after the sandstone culvert. The box culvert is of a modern design and has greater structural capacity for the connection of a stormwater pipe rather than the older sandstone culvert.

4.1.3.3.1. Location

Figure 107 below shows the region in which an easement needs to be purchased in order to connect a stormwater pipe into the existing concrete box culvert which is highlighted in blue.

Figure 107 - Plan view of construction region for small box culvert (Google Earth, 2013)

4.1.3.3.2. Property Ownership Concern

As per Figure 107, the proposed construction site for the connection of the box culvert into the concrete box culvert is located on private property. This will required an easement that must be purchased by the City of Norwood Payneham and St Peters; this therefore allows the council to install manholes in the construction region that can be utilised at a later date for routine maintenance checks of both the small box and arch culvert.

4.1.3.3.3. Economic Advantages and Disadvantages

Disadvantages:

- The scope of this option is larger than the previously considered options. This option requires a more comprehensive detailed design i.e. more required calculations.

The Urban Planning department has given the details regarding the costs for purchasing this easement, these details are illustrated in Table 43.

Table 43- Costs of Land, Kent Town (Hydro-Future, 2015)

Year	Land Cost ($AUD)	Total Area of Land Required (m^2)
2001	121,000	132.9
2015	250,000 to 300,000	132.9

- The cost of easement shown in

Year	Land Cost ($AUD)	Total Area of Land Required (m2)
2001	121,000	132.9
2015	250,000 to 300,000	132.9

- for the property located on 14 North Terrace, Kent Town, S.A. 5067 will exceed the budget of the proposed project option. The costing for purchasing this easement is approximately $300,000 dollars, which is an unreasonable cost in comparison to other options.

- The purchase of this easement may be rejected by the registered property title owner, or further negotiations with the owner may be necessary to allow for its purchase.

Advantages:

- An advantage of this option is that no stormwater pipe will be connected to the sandstone arch culvert hence mitigating the risk of damaging or collapsing the old sandstone structure.

- Easement provides lots of room for installation of maintenance structures which allow for easy access to the drainage pipe and box culvert in the near future

4.1.3.3.4. Preliminary Design Drawings

The preliminary dimensions of the small box culvert drawn using AutoCAD and has been illustrated in Figure 108. It should be noted, these dimensions are preliminary, and only define the scope of work necessary for the detailed design and construction phase of the project. All preliminary drawings are subject to change during the detailed design phase of the project.

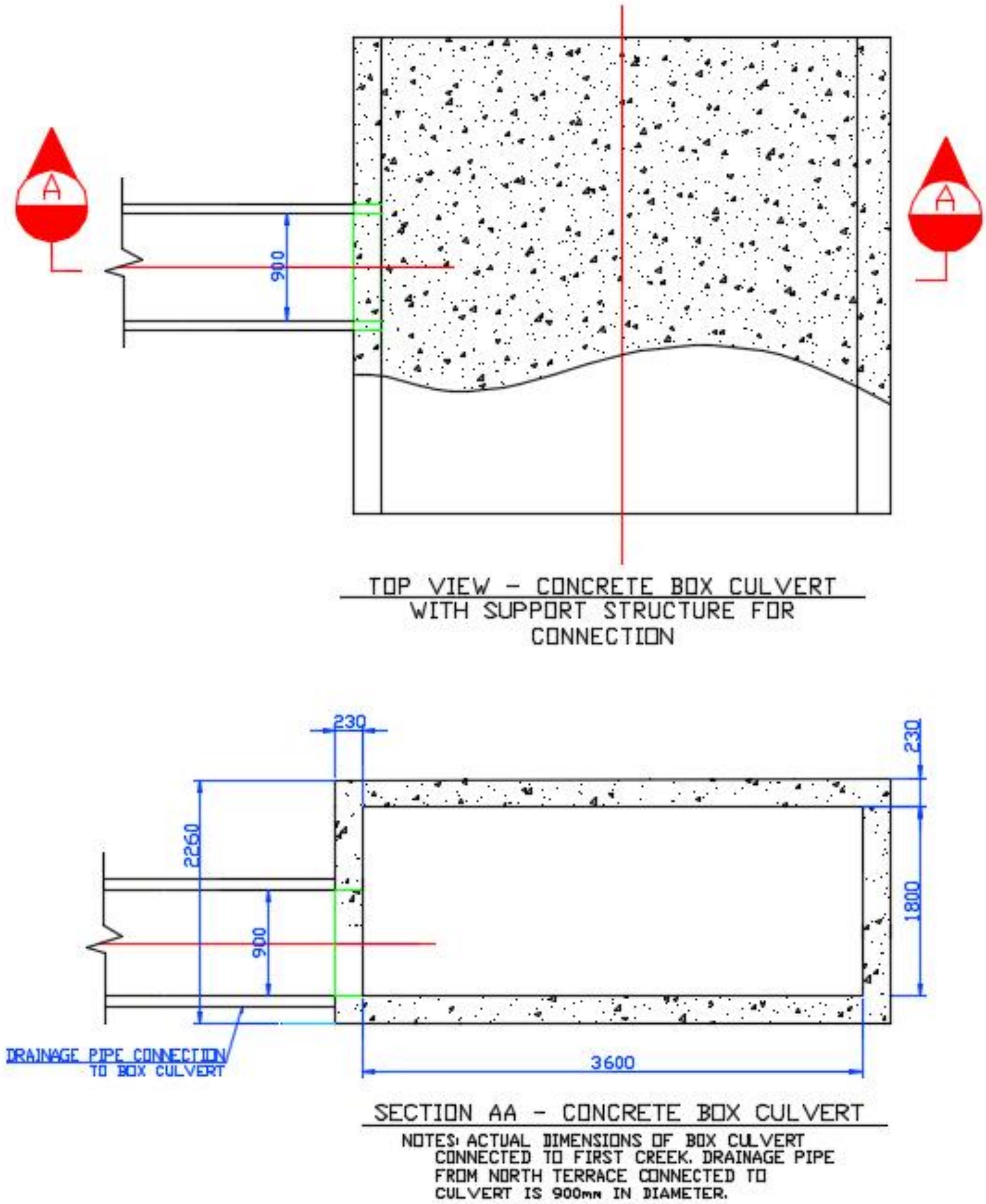


Figure 108 – Stormwater Pipe - Box Culvert Connection (Hydro-Future, 2015)

4.1.3.3.5. Design Advantages and Disadvantages

Disadvantages:

- The structural integrity of the concrete box culvert must be analysed at the point of insertion with the drainage pipe. This analysis may require the utilisation of a computer assisted analysis, such as SpaceGass or STRAND7 to correctly calculate the stress and loading distributions that exist at this entry point.

- Reinforcement removal in the existing concrete box culvert may decrease structural integrity; further calculations may be required to determine whether additional reinforcement or support structures must be installed

- The use of relatively complex hand calculations might need to be worked out in conjunction with any computer analysis to ensure that accuracy of the computer results is maintained throughout the analysis. These hand calculations would increase the difficulty of the detailed design phase of the project.

Advantages:

- Connecting the drainage pipe into a box culvert section is easier to analyse in comparison to an arch culvert section. There would exist many complexities with modelling sandstone arch sections in comparison to concrete wall sections, and ultimately this analysis would require less work to be completed than option 2 analysis.

- Reinforced concrete pipes are generally considered to be much stronger than most other pipes available in the civil engineering industry

- Reinforced concrete pipes are more resilient to forces exerted by uneven bedding due to their high beam strength i.e. resilience to deflection.

- Decreased maintenance cost over the design life of the pipe due to the reduced likelihood of structural failure.

4.1.3.3.6. Preserving Concrete Box Culvert Structural Integrity

The connection from the drainage pipe could cause structural deficiencies in the existing box culvert.

Section 4.1.2 addresses the structural concern through the utilisation of the following rehabilitation and relining methods:

1. **Temporary construction frame**
2. **Rehabilitation and relining method**
3. **Spot patch and repair**
4. **Applying shotcrete lining**

The rehabilitation methods listed above only provide limited support to the structure and may not provide enough support to withstand any construction alterations to the box culvert.

4.1.3.3.7. Advantages and disadvantages of rehabilitation methods on box culvert

Advantages:

- Rather than considering major structural repairs or improvements i.e. installation of support structures, the rehabilitation options offer a quicker, easier and cheaper solution to solving culvert integrity issues.

Disadvantages:

- Rehabilitation methods only provide a limited solution to the problem, and may not address all concerns with preserving the structural integrity

- There is no ideal way of determining the life expectancy of some rehabilitation options. Spot and patch repair may only temporarily fix spalling issues, however other sections of the culvert may eventually spall (breaking away of sections or chipping of culvert).

4.1.3.3.8. Work to be completed in the Detailed Design Stage

This design will require several of the following stages to be completed during the design phase of the project.

1. **Excavation Design** – Location and costing of the required excavation along the full length of the proposed stormwater drainage pipe including trench stability.
2. **Existing Arch Box Culvert Analysis** – Detailed design showing the stresses which are likely to occur around the connection point of the box culvert and how the structure will react along with the location of reinforcement within the box culvert.
3. **Box Culvert protection** – Design to protect the box culvert from undergoing damage during excavation and increasing the design life post construction.
4. **Connection Design** – Design of a suitable connection into the box culvert to allow for flexibility, rigidity and waterproofing.
5. **Detailed Design Loading** – Traffic, dead, live and earth loads on the associated components of the proposed deign option.

4.1.3.3.9. Design Loads

This section of the report covers the introduction to the calculation of the Dead and Live Loads, their corresponding loading combinations.

Loadings acting on Box Culvert

- These calculations are loadings estimation for the culvert section

Due to inconsistencies with data, the depth from surface level to the base of the Arch Culvert and Box Culvert are assumed to be 3m to ensure conservative preliminary results.
The dimensions of the box culvert are shown in Figure 109, these dimensions are utilised throughout the calculation procedure. The strength characteristics for the soil three soil layers presented in Figure 109.

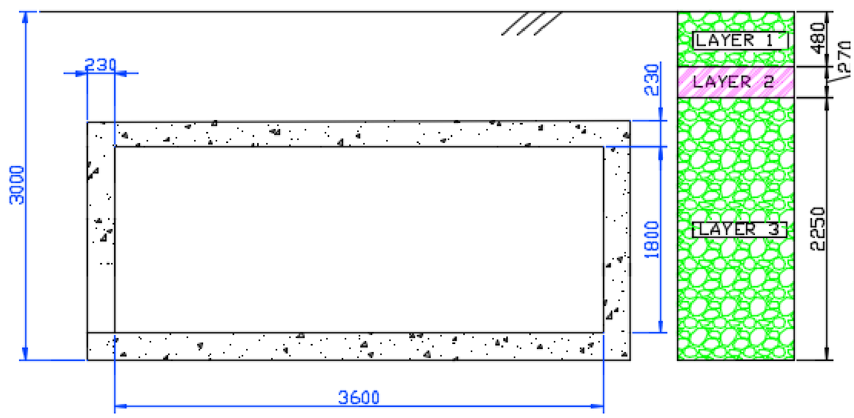


Figure 109 - Dimensions of box culvert (Hydro-Future, 2015)

The geotechnical department has determined the characteristics and parameters for the soil available at North Terrace, these soil characteristics are illustrated in *Table 44*.

Table 44 - Geotechnical soil parameters for North Terrace (Hydro-Future, 2015)

Aspect	Depth	Unit Weight *(y)*	Angle of internal friction	Cohesion
Layer 1	0 to 0.48m	21.5 kN/m^3	22°	3
Layer 2	0.48m to 0.75m	18.5 kN/m^3	28°	8.5
Layer 3	0.75m to 4.5m	21.5 kN/m^3	22°	3

Dead Load Calculations

For the box culvert section illustrated in Figure 109, the following loading conditions are considered:

- Uniform Distribution Loads (i.e. UDL on top and side walls of culvert section)
- Weight of side walls
- Earth pressures on side and top walls
- Uniform lateral load on side and top walls

Uniform Distributed Loads

The weight of soil above the culver is uniformly distributed to the top section of the culvert, as illustrated in *Figure 110*.

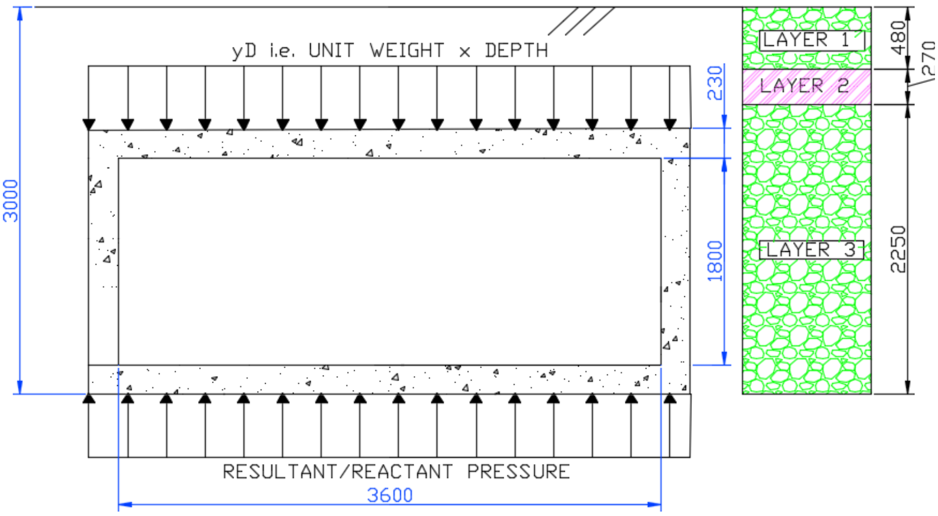


Figure 110 – Earth Pressures (Hydro-Future, 2015)

Vertical Pressures:

Vertical Earth Pressure (On top of culvert):

$$W_{FV} = y\ H$$

$$W_{FV} = 21.5\text{kN/m}^3 \times 0.48\text{m} + 18.5\text{kN/m}^3 \times (0.75 - 0.48)\text{m}$$

$$\mathbf{W_{FV} = 15.315\ kPa}$$

Vertical Earth Pressure (Side of culvert):

$$W_{FV} = 15.315 \text{ kPa} + (1.8 + 2x0.23)\text{m x } 21.5\text{kN/m}^3$$

$$\mathbf{W_{FV} = 63.91 \text{ kPa}}$$

Horizontal Pressures:

The calculated vertical pressures are utilised in conjunction with the angles of internal friction given in *Table 44*. The vertical pressures for the entire depth of the box culvert are in the vertical pressure diagram in *Figure 111*.

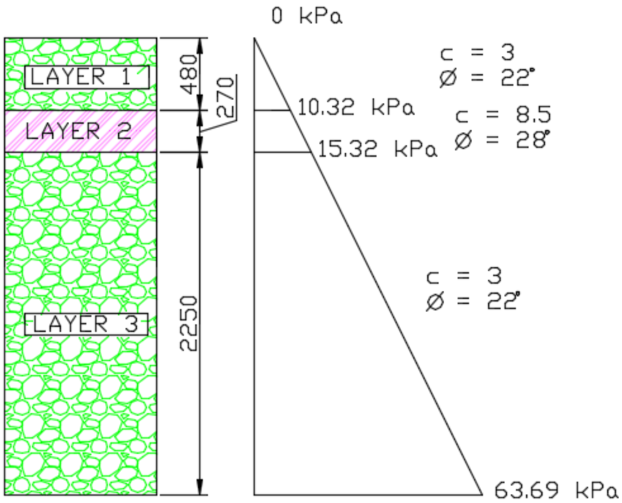


Figure 111 - Vertical Pressure Distribution (Hydro-Future, 2015)

Coefficients of Earth Pressure:

$$k = \frac{(1 - Sin(\emptyset))}{(1 - Sin(\emptyset))}$$

For: Layer 1 0 to 0.48m

$$k = \frac{(1 - Sin(22))}{(1 - Sin(22))}$$

$$k = 0.456$$

For: Layer 2 0.48m to 0.75m

$$k = \frac{(1 - Sin(28))}{(1 - Sin(28))}$$

$$k = 0.361$$

For: Layer 3 0.48m to 0.75m

Similar to Layer 1, i.e. k = 0.456

Horizontal Earth Pressure (Side of culvert wall):

$$W_{FH} = k \text{ y } H$$

Conservatively and for simplicity, assume wall is 0.75m deep.

Horizontal Pressure at top of wall

$$W_{FH} = 0.456 \times 15.32 \text{ kPa}$$

$$\mathbf{W_{FH} = 6.986 \text{ kPa}}$$

Horizontal Pressure at bottom of wall

$$W_{FH} = 0.456 \times 47.375 \text{ kPa}$$

$$\mathbf{W_{FH} = 22.06 \text{ kPa}}$$

Self-weight calculation of Box Culvert

Assume culvert length is equal to 1m (i.e. analysing 1m sections of culvert) (LRDF Bridge Design, 2013, pg 29)

Self-weight of top slab section:

$$G_{TOP} = \text{Density of Concrete} \times \text{Concrete Thickness} \times 1\text{m Culvert Section}$$

$$G_{TOP} = 24\text{kN/m}^3 \times 0.23\text{m} \times 1\text{m}$$

$$\mathbf{G_{TOP} = 5.52 \text{ kN/m}}$$

Self-weight of top slab section (As concentrated loading):

$$G_{TOP(CL)} = \text{Density of Concrete} \times \text{Concrete Thickness} \times 1\text{m Culvert Section} \times \text{Span}(3.6\text{m}+2\text{x}0.23\text{m})$$

$$G_{TOP(CL)} = 5.52 \text{ kN/m} \times (3.6\text{m} + 2\text{x}0.23\text{m})$$

$$\mathbf{G_{TOP(CL)} = 22.41 \text{ kN}}$$

Self-weight of the side walls:

The self-weight of the side walls produce concentrated loads at the bottom of the slab, it is assumed that these concentrated loads produce a uniform slab reaction on the bottom of the slab.

$$G_{SIDEWALLS(CL)} = 2 \text{ Walls} \times \text{Density of Concrete} \times \text{Wall Height} \times 1\text{m Culvert Section} \times \text{Wall Thickness}$$

$$G_{SIDEWALLS} = 2(24\text{kN/m}^3 \times 1.8\text{m} \times 1\text{m} \times 2\text{x}0.23\text{m})$$

$$\mathbf{G_{SIDEWALLS} = 39.74 \text{ kN}}$$

Self-Weight Uniformly Distributed Loads (UDL)

Reaction force to weight of side walls and top slab:

It is assumed the top slab weight and wall heights are applied to the bottom slab as an upward resultant force assuming an equivalent uniform pressure. (LRDF Bridge Design, 2013)

$$G_{BOTTOM\ \{RESULTANT)} = G_{SIDEWALLS} + G_{TOP}$$

$$G_{BOTTOM\ (RESULTANT)} = 22.41 \text{ kN} + 39.74 \text{ kN}$$

$$\mathbf{G_{BOTTOM\ (RESULTANT)} = 62.15 \text{ kN}}$$

Water pressures calculation inside Box Culvert

As stated on page 31 of LRDF Bridge Design (LRDF Bridge Design, 2013), all designers must consider load cases where the culvert is full of water and without water.

The calculation for the water pressure distribution inside the culvert utilises the following formula:

$$WA_{TOP} = 0 \text{ kPa}$$

$$WA_{BOTTOM} = \gamma_w \; x \; Height \; x \; 1m \; Section$$

$$WA_{BOTTOM} = 9.81 kN/m^3 \text{ x } 1.8m \text{ x } 1m$$

$$\mathbf{WA_{BOTTOM} = 17.66 \; kN/m}$$

This result is the water pressure acting on the base of the culvert, as well as the maximum pressure due to contained water, acting on the sidewalls.

Live load calculation on Box Culvert

The live load is calculated in accordance to Cl 3.3.5.5.2 of AS1597.2.

It is assumed that the road surface above the culvert will be M1600 traffic wheel contact load areas, where a = 200mm for serviceability and 300mm for ultimate contact lengths

The design will be calculated in accordance with ultimate strength limit states through equation 3.3.5.5.2(3), i.e. multiple wheel loads

$$A = L_1L_2 = (b + 1.15H)(a + 1.15H)$$

Where, a = 300mm, b = 0.5m, H = Height of fill above culvert i.e. 0.74m

$$A = (0.3m + 1.15(0.74m)(0.3m + 1.15(0.74m))$$

$$\boldsymbol{A = 1.324}$$

The DLA (*Dynamic Load Allowance)* factor is conservatively assumed to be 0.1, as per Cl 3.3.5.5.3

Vertical live load due to traffic loadings:

$$W_{LV} = (1 + DLA)(\sum P)/A \; (kPA)$$

More details regarding these calculations is listed on pg 32 of AS1597.2. The road vehicle loads in terms of the moving A160 vehicles is given in AS 5100.2 (i.e. kPa and P), pg 15.

$$W_{LV} = (1 + 0.1)(2x160kN)/(0.4mx0.2m) \; (kPA)$$

$$W_{LV} = (1 + 0.1)(2x160kN)/(3.2m \; x \; 3.2m)$$

$$\mathbf{W_{LV} = 34.37 \; kN/m^2}$$

Horizontal live load due to traffic loadings:

$$W_{LH} = k_0 \times W_{LV}$$

Where, k_o= Coefficient of earth pressure at rest, i.e. 0.456

$$W_{LH} = 0.456 \times 34.37 \ kN/m^2$$

$$\mathbf{W_{LH} = 15.68 \ kN/m^2}$$

Loading Combinations to consider for detailed design phase:

The stability and limit state load factors for the horizontal and vertical loads are illustrated in Table 3.3 and 3.4 on pg 33 of AS1597.2. These limit state factors are used to determine the required ultimate loadings acting on the culvert structure.

4.1.4. Standards & Requirements

4.1.4.1. Construction Details - Excavation

The following standards shown below will be used during the detailed design phase of the project. These standards should provide information for the chosen design option and the proposed structural support.

- *AS/NZS 1170.0:2002 Structural design actions - General principles*
- *AS/NZS 1170.1:2002 Structural design actions – Permanent, imposed and other actions*
- *AS3500.3(2003) Plumbing and Drainage – Stormwater Drainage*
- *AS3600 – Concrete Structures*
- *AS1597.2 – Precast Reinforced Concrete Box Culverts*
- *AS1289.0 – Methods of Testing soils for engineering purposes*
- *AS1289.1 – Methods of Testing soils for engineering purposes*
- *AS/NZS 4058 – Precast concrete pipes (pressure and non-pressure)*
- *AS/NZS 3725 – Design for installation of buried concrete pipes*

4.1.4.2. Design Loading Combinations

AS1597.2 describes the process required to calculate the geotechnical earth pressures acting on the surface of a culvert structure. This Australian Standard is designed to be used for square culvert units, however the shape of the proposed structural support system in Section 4.1.3.3.6 is similar to a culvert, and it is assumed that certain sections of this code are applicable to the design of this structural support reinforced concrete section

The design must withstand loadings that can affect the stability, strength, serviceability and durability limit states.

The design for strength and serviceability utilise the specified Design Loads and Loading Combinations from Clause 3.3 and 3.4 of AS1587.2

- ***Clause 3.3 – Design Loads:***

Dead Load

As per Cl3.3.1 of AS1597.2, the dead load for the culvert unit self-weight is equal to W_{DC}, and the vertical earth pressure due to the fill W_{FV} is calculated in accordance to one of the two formulas provided under Cl3.3.1

Horizontal Pressure (WFH and WAH)

As stated in AS1597.2, Clause 3.3.3, in the absence of site derived horizontal earth pressures, the pressure due to the fill can be calculated using formula 3.3.3 under the specified clause.

If the fill above culvert is compacted, Clause 3.3.4 becomes applicable to the calculation of horizontal pressures.

Traffic Loadings

The box and arch culvert are currently located beneath a high traffic area including a carpark and main road, it is assumed that the culverts will sustain loadings from *Standard road traffic loads* (Clause 3.3.5.2).

All construction loadings are calculated with respect to Clause 3.3.5.3 and 3.3.5.4. These clauses related to the effects of construction vehicles and heavy load platform equipment (i.e. cranes) that are located within the vicinity of the excavation region. If fill is completed after heavy vehicles have left the construction region, it is therefore assumed that the loadings from heavy load platforms can be ignored.

The distribution of loadings, as specified under Clause 3.3.5.5.2, states that the contact area of the wheel with the road shall mimic a uniform vertical pressure distribution over the area on top of the culvert over a rectangular distribution area.

This rectangular area (A) is calculated under clause 3.3.5.5.2 for (a) Serviceability and (b) Strength

The vertical (W_{LV}) and horizontal (W_{LH}) traffic loadings are calculated with respect to Clause 3.3.5.5.4 and 3.3.5.5.5, in AS1597.2 These calculations require the DLA (*Dynamic Load Allowance)* factor listed under Clause 3.3.5.5.3.

Design Loadings not applicable to the design procedure

The following live and dead loadings have been disregarded from the design procedure, as they do not factor into the culverts design environment.

- W_{PV} and W_{PH} – Vertical and horizontal heavy load from platforms. This loading is assumed to be ignored (*Subject to further discussion*)
- W_{RV} and W_{RH} – Horizontal and vertical pressures from railways. No railways present

Final load effect analysis and design

The final calculations of the structural design and load effect analysis of all components of the box culvert shall be designed in accordance to AS3600 and AS1597.2.

Plumbing and Drainage – Water Services

The Australian Standard (AS) 3500.3, is used to determine the materials, design, installation and testing of drainage systems to a point of connection.

The materials for stormwater pipes are listed under Clause 2.4 of AS3500.3.

The Hydro-Future Structural Engineering team will assume that precast concrete pipes (Steel Reinforced) can be used for this design option, as under the guidelines illustrated in AS/NZS 4058.

The Concrete Materials, as per Clause 2.2 of AS4058, specifies the type of cement, aggregate, slag, water mixtures, admixtures and restrictions of chemical content that may be used in concrete pipe materials. These requirements will be met by the manufacturer.

The reinforcement in the precast sections shall be in accordance to AS/NZS4761.

The excavation requirements for pipes is available in Clause 6.2.3 of AS3500.3. This clause specifies the requirements for trench widths with respect to the type of material used for the drainage pipe. For this proposed option (Concrete pipes) the AS/NZS 3725 is required.

Clause 8 of AS/NZS 3725 specifies the type of compaction that must be achieved during the construction period and installation of pipes. Compaction is achieved via one of the specified options under subsections (a) and (b) of AS/NZS 3725.

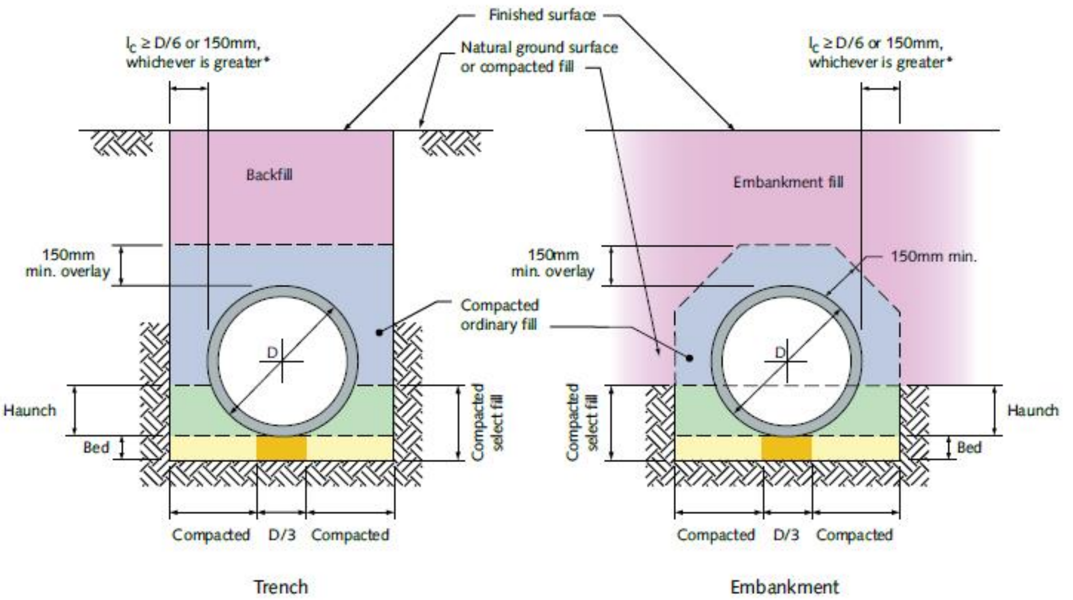
4.1.4.3. Construction Details – Concrete Pipe

AS/NZS 3725 *'Design for installation of buried concrete pipes'* explains the methodologies related to the required calculations for working out the loadings of concrete pipes buried underground. It also specifies details of the installation.

Clause 9.1.1 explains the type of pipe support systems and bedding factors that must be evaluated in order for correct placement of the pipe. This clause states that the pipe shall be placed in a suitable area with various fill material acting as 'support zones' for the pipe.

Clause 9.2 details the type of Support systems that are specified for fill material used during placement of the pipe. There exist 3 major support systems, Type H, Type HS and Type U.

It is assumed that Type H2 is used for the bedding support system, either choice works in conjunction with the specified information provided by *Humes* in their Precast Concrete Pipe Brochure (Humes, 2009). An extract from Humes brochure is shown in *Figure 112*, this figure provides a detailed layout of the pipe in a H2 and HS2 (*Haunch)* support configuration.



* Refer AS/NZS 3725: 2007 for cement stabilised soil

Figure 112 - Haunch support configuration for pipes (Humes, 2009)

Figure 112 provides useful insight on the type of Haunch configurations that can be utilised in the design phase of this project. For this construction option, a H2 configuration seems more suitable for the scope of the project.

The *Humes* brochure provides two tables (Tables 1.1 to 1.4) that are used to evaluate the required fill heights and minimum trench/embankment widths during trench or embankment installation of the pipe (Humes, 2009).

The brochure specifies the maximum applicable Cracking and Ultimate loadings that can be applied to the pipe before failure occurs, an extract of this information from the Humes Brochure is shown in Table 45 and Table 46. This information will be utilised during the detailed design phase of the project to determine a reasonable pipe size that can withstand the forces from the soil and or other concrete/support structures above or embedded with the pipe.

Table 45 – Concrete Pipe Loadings (Humes, 2009)

Load Class	**Standard Strength**					
	Class 2		Class 3		Class 4	
Size Class (DN)	Crack	Ultimate	Crack	Ultimate	Crack	Ultimate
675	29	44	44	66	58	87
750	32	48	48	72	64	96
825	35	52	52	78	69	104
900	37	56	56	84	74	111

Table 46 – Concrete pipe Loadings (Humes, 2009)

Load Class	**Super Strength**					
	Class 2		Class 3		Class 4	
Size Class (DN)	Crack	Ultimate	Crack	Ultimate	Crack	Ultimate
675	29	44	44	66	58	87
750	32	48	48	72	64	96
825	35	52	52	78	69	104
900	37	56	56	84	74	111

4.1.5. Evaluation

As seen below in Table 47, Hydro-Future had devised an evaluation matrix to determine the most suitable option for this projects detailed design win terms of:

- Flood Mitigation
- Cost
- Impact on the sandstone culvert
- Design Simplicity

Table 47 - Evaluation Matrix (Hydro-Future, 2015)

Option	Flood Mitigation (%50)	Cost (%20)	Impact on Sandstone Culvert (%20)	Design Simplicity (%10)	Total
1	0	95	95	95	47.5
2	**95**	**80**	**20**	**75**	**75**
3	95	0	95	50	71.5

Based on the above analysis Hydro-Future recommends option 2 as a viable design option with respect to flood mitigation, cost, impact on the sandstone culvert and design simplicity.

Due to scarce information provided in the costing documents for precast sections i.e. Precast Reinforced Concrete Blocks, it is assumed that a conservative price for the material cost is appropriate.

4.1.6. Cost

Table 48 – Costing for Support System 1 (Rawlinsons, 2014)

Material Costing for Structural Support System 1 (Option 1)				
Description	***Units***	***Quantity***	***Rate***	***Cost***
Precast Slab - Bridge Deck (Assumed) – 40MPa Concrete	(m^3)	2.4mx0.46mx2.26m = 2.495 ≈ 2.5 m^3	$243.00	$607.5
Reinforcement	t	0.3	$1850.00	$555.00
Mortar - Bridge Deck (Assumed) – 40MPa Concrete	(m^3)	2xπx0.9mx0.1m = 0.565 ≈ 0.6 m^3	$243.00	$145.80
Reinforcement (Anchor - Tendon)	t	0.2	$1850	$370.00
Prestressing - Continuity	t	0.2	$6300	$1260.00
			TOTAL	**$2,940**

5. Geotechnical Design

5.1. Trench Stability

Excavation may be required to allow the stormwater pipes and its associated components, to be placed at the correct depth below North Terrace. Hydro-Future recognises that a suitable trench design will be required for the project to maximise worker safety and to minimise construction damage.

5.1.1. Trench Height

Referring to the nature of the above soil profiles the following design criterions will need to be applied to ensure a safe trench height is selected for excavation:

- Drainage within the trench may be difficult due to the low permeability of the clay dominant RB5 layer.
- The permeability of the alluvial soil layers can be quite high due to the sand and gravel content and should be considered as a potential location for trench drainage within the design.
- The high sand content in alluvial soils will reduce the structural stability of the trench walls due to the limited cohesion.
- The structural integrity of the trench walls will be strengthened by the high cohesion which is associated with the clay layers within RB5.

(Nemati 2007)

The safe height of the trench can be determined using the soil characteristics and limitations as highlighted above. Reviewing the soil profiles the maximum trench height allowed during excavation without a stabilising system will be 1.5 meters or less.

Table 49: Safe Trench Heights (Nemati, 2007)

Soil Consistency	*Cohesion (c')*	*Safe Height (m)*
Very soft	< 250	< 1.5
Soft	250 – 500	1.5 –3
Medium	500 – 1000	3 – 6
Stiff	1000 – 2000	6 – 12
Very stiff	2000 – 4000	12 – 24
Hard	> 4000	> 24

5.1.2. Stabilising Methods

Several methods to stabilise trenches during excavation are available for use. Several reinforcement options can be seen below in Figure 113, Figure 114 and Figure 115 which will be suitable for the shallow excavation which may be required for this project.

Figure 113 is an intermittent reinforcing construction which utilises extendable bars to hold up vertical sheets along the trench (Nemati 2007)

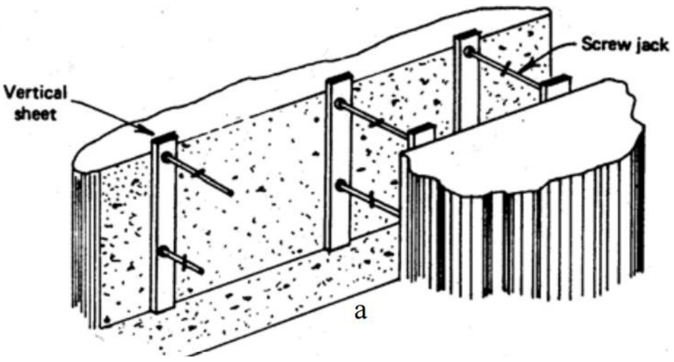


Figure 113: Intermittent Sheeting & Bracing (Nemati, 2007)

Advantages:

- Materials used to construct the trench are light and easily transferrable.
- The construction and removal is generally quick and efficient due to the simplistic construction.

Disadvantages:

- The horizontal struts or bars will hinder movement throughout the trench, especially if a stormwater pipe is to be laid.
- Reinforcement spacing's are limited, if the spacing is to far the earth pressures on the vertical sheets will become too intense however if they are to close they will hinder movement.

Figure 114 shows a method which allows for a more rigid and continuous construction. The sheeting and top struts are generally placed for a substantial amount of time to allow work in the trench.

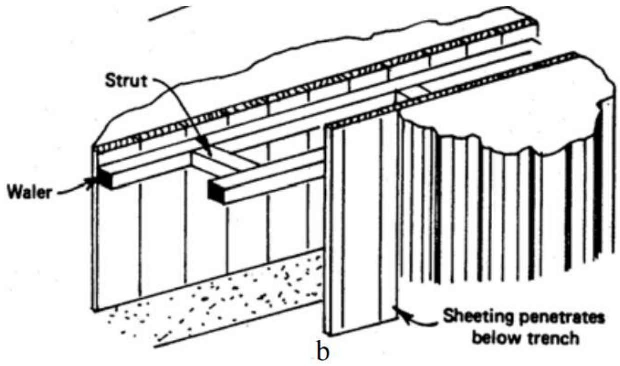


Figure 114: Continuous Sheeting & Bracing (Nemati, 2007)

Advantages:

- The rigid and continuous construction is allowed over long distances.

Disadvantages:

- The horizontal struts will hinder movement throughout the trench, especially if a stormwater pipe is to be laid.
- The length of time for construction is substantially longer.
- The materials are more rigid and heavy, reducing the efficiency of the construction and removal processes.

Shown below in Figure 115 is a configuration which allows for a metal box to slide along the trench providing lateral support as the excavation and construction process moves. These boxes are generally light and include handles for workers to relocate the construction easily.

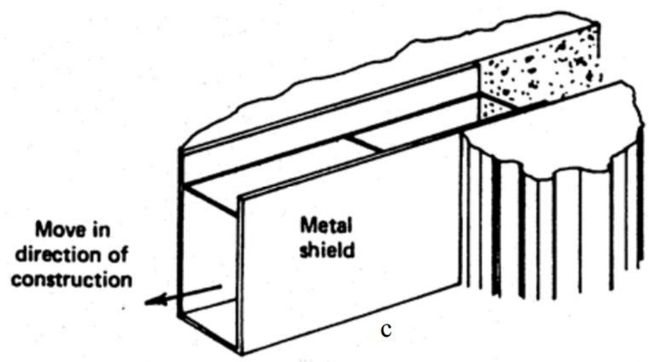


Figure 115: Trench Shielding (Nemati, m.k., 2007)

Advantages:

- The set-up is rigid and resistant to high earth pressures.
- Easily transferrable and movable along the trench.
- Setup and removal is quick and efficient due to the limiting parts.
- Will allow for excavation in front of the box whilst installation of the stormwater pipe and backfilling behind it.

Disadvantages:

- Limited length along the trench.

5.1.3. Australian Standards

The following Australian standards will be used during the design and construction of the chosen trench reinforcement method:

- AS 5047 – 2005 Hydraulic Shoring and Trench Lining Equipment
- AS4744.1– 2000 Steel Shoring and Trench Lining Equipment

5.1.4. Evaluation

Through evaluation of the Australian Standards and the above options, Hydro-Future recommends trench shielding (option 3) as the most effective and convenient method for this project due to its ease of mobility and construction simplicity.

5.1.5. Cost

Due to the limited costing information provided for trench supports, Hydro-Future has assumed a conservative price for the recommended design option which can be seen below in. It has been approximated that a minimum of 1 x 4 x 3 m to 1 x 5 x 3 m (length x width x height) of excavation is necessary for the projects construction. The approximation is conservative and may change during the detailed design, the costing is based on 1 meter lengths of trench excavation (Table 50).

Table 50: Trench Costing (Rawlinson's 2014)

Description	***Units***	***Quantity***	***Rate***	***Cost***
(Clay) Planking and strutting, shoring (Sides of trench excavation) i.e. Trench support	(m^2)	(1m x 3m) x 2 (Both Sides) = 6 m^2	$7.00	$42
TOTAL COST PER METER LENGTH				$42

5.2. Retaining Wall design

Due to the age of first creek, significant sediment transportation and erosion of the creek beds and side slopes have occurred over the past years. Hydro-Future aims to mitigate any collapse of the eroded creek bed which may render the creeks function to transport water to the river Torrens. A retaining wall design is required to be undertaking to reduce the erosion of the creeks slopes and to increase the creeks design life.

5.2.1. Location

Hydro-Future has analysed the full length of first creek and has identified an area which has undergone significant erosion and requires the implementation of a retaining wall to maintain the creeks functionality. As seen below in Figure 116 and Figure 117 are photos of the eroded creek bed, which occurs on a bend where the water velocity is concentrated on the sides of the creek. Hydro-Future aims to reduce any further erosion throughout this portion of First Creek with the application of a retaining wall.

Figure 116: Eroded Creek Bed (Hydro-Future 2015)

Figure 117: Eroded Creek Bed (Hydro Future 2015)

The location of the required retaining wall can be seen below in Figure 118 and Figure 119. The below images show the retaining wall will be located within botanical parklands which have an adverse effect on the aesthetics of the design.

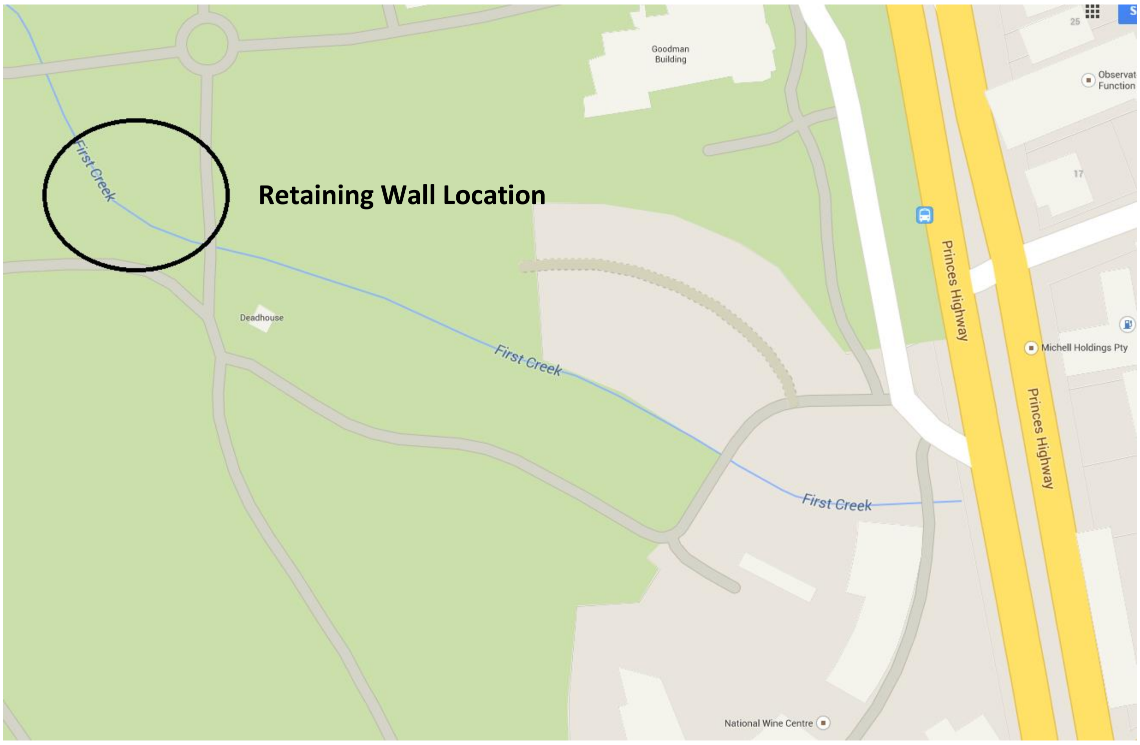


Figure 118: Retaining Wall Location (Hydro-Future, 2015)

Figure 119: retaining Wall Location (Hydro-Future, 2015)

5.2.2. Geotechnical Model

During the site investigation undertaken by Hydro-Future, several key design characteristics were documented to effectively create a preliminary design drawing and calculate the required earth pressures.

Hydro-Future inspected the embankment side slopes, heights and widths to determine the retaining walls initial design dimensions, which included a height of 1 metre above the creek bed (exposure height) and a 1 metre deep foundation (foundation hieght) which can be observed below in Figure 120.

Figure 120 also shows the associated geotechnical model for the site area which includes the soils layers which will exert earth pressures on the retaining wall. Hydro-Future carried out a visual-tactile inspection of each soil layer during the site investigation which were compliant with AS 1726, Geotechnical Site Investigations. Refering to Figure 116, it can be observed that 2 soil different soil layers occur along the creek embankment which include a Silty Sandy CLAY ontop of a silty CLAY layer. Upon further inspection of the creek bed, it was observed that the silty CLAY layer continued below the creek and is assumed to be the dominant soil below the creek bed level.

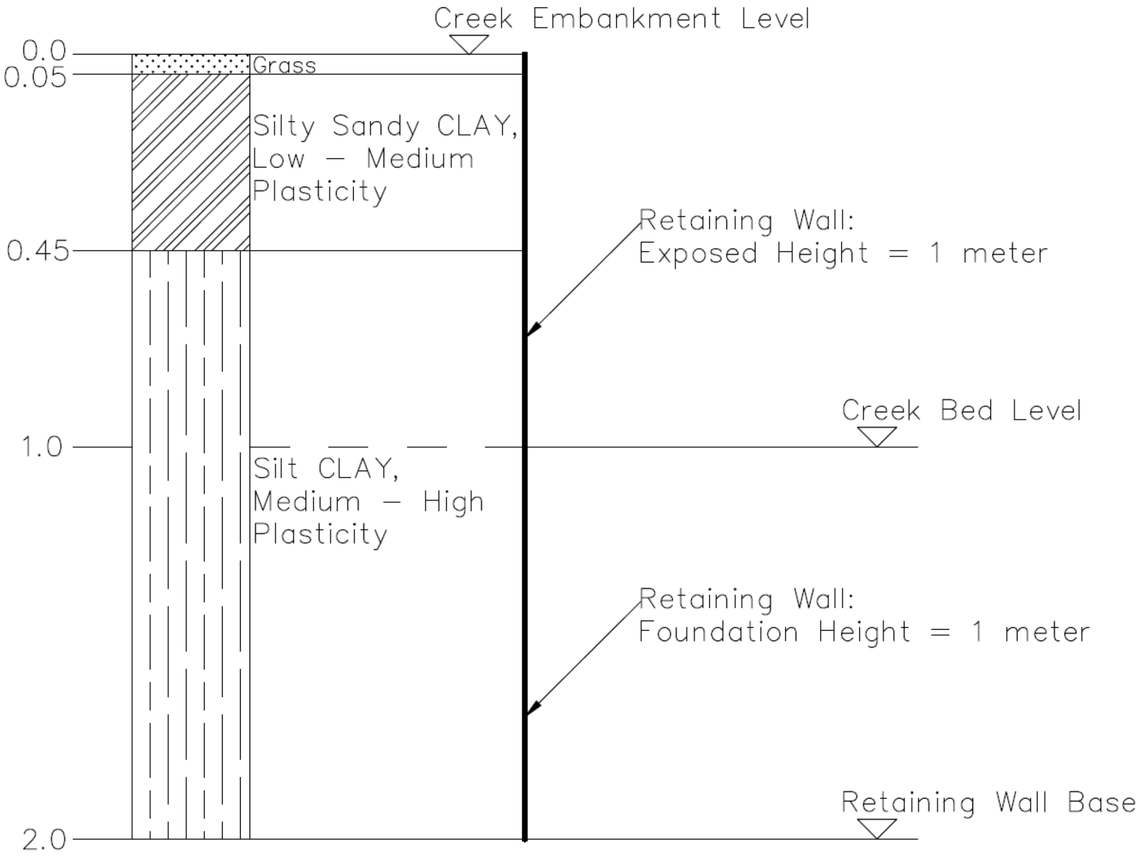


Figure 120: Geotechnical Model and Retaining Wall Dimensions (Hydro-Future 2015)

Table 51 are the required geotechnical strength and stiffness parameters which are associated with each soil layer within Figure 120

Depth	*Soil*	*Unit Weight* γ (kN/m^3)$_3$	*Undrained Shear Strength* C_u (kPa)$_{1,3,,4}$	*Shear Strength* C' (kPa)$_2$	*Internal Frictional Angle* ϕ_2
0.05 – 0.45	Silty Sandy CLAY	18.5	85	0 – 10	30
0.45 – 2.0	Silty CLAY	20.5	150	0 – 5	20

1) **AS 1726**
2) **AS 4678**
3) **AUSTROADS (1992)**
4) **p48, 337 & 460, Smith 2006**

Table 51: Retaining Wall Soil Properties

5.2.3. Earth Pressures

Hydro-Future has identified the critical condition for which the permanent earth pressures will govern the design, this conditions states:

- The creek will have little or no water level, this will reduce the amount of resistance, overturning and sliding, which the pore water pressure creates against the permanent earth pressures which are formed by the embankment.

This condition will be critical for any retaining wall design including those which are not required to have a foundation. As the soil beneath the creek bed is of the same composition, the earth pressures will remain the same on either side of the founded section, this will reduce any sliding or moment forces to zero, ensuring that no design loads are required underneath the level of the creek bed. The design load calculations can be seen below:

Vertical Pressure:

Assuming the grass will not applying any pressure to the retaining wall.

- *Depth 0.05 – 0.45:*

$$\sigma_{y1} = \gamma z = 18.5 * 0.4 = 7.4\ kPa$$

- *Depth 0.45 – 1.0:*

$$\sigma_{y2} = \gamma z\ +\ \sigma_{y1} = 20.5 * 0.55\ +\ 7.4 = 18.675\ kPa$$

As seen below in Figure 121 is a summary of the vertical earth pressures.

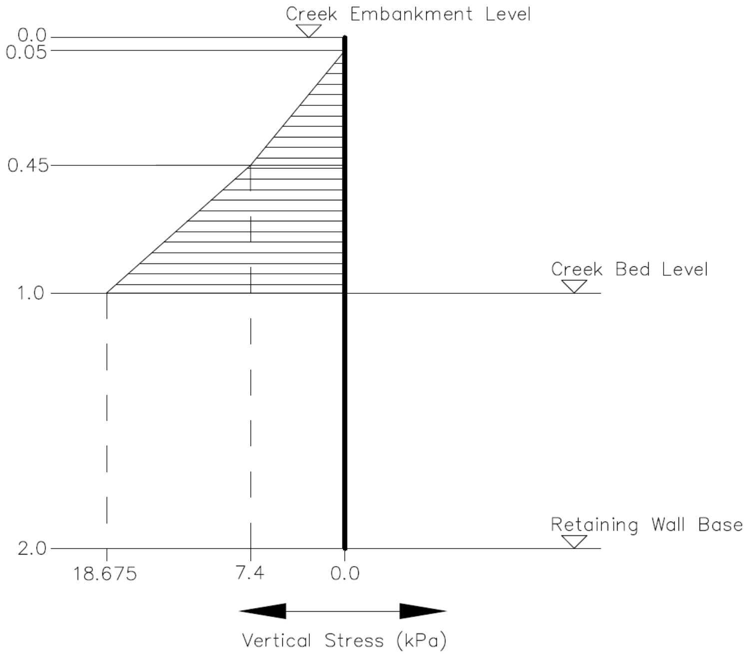


Figure 121: Vertical Earth Pressures (Hydro-Future, 2015)

Horizontal Pressure:

Assuming that grass will not applying any pressure to the retaining wall and c' = 0, as the soil is dry and has no cohesion.

- *Horizontal Pressure Coefficient (0.05 – 0.45):*

$$K_{a_1} = \frac{1-\sin(\phi)}{1+\sin(\phi)} = \frac{1-\sin(30)}{1+\sin(30)} = 0.33$$

- *Horizontal Pressure Coefficient (0.45 – 1.0):*

$$K_{a_2} = \frac{1-\sin(\phi)}{1+\sin(\phi)} = \frac{1-\sin(20)}{1+\sin(20)} = 0.49$$

- *Horizontal Pressure at 0.45m:*

$$\sigma_H\ (Silty\ Sandy\ CLAY) = K_{a_1}\sigma_{y1} - 2c'\sqrt{K_{a_1}} = 0.33 * 7.4 - 2 * 0 * \sqrt{0.33} = 2.4\ kPa$$

$$\sigma_H\ (Silty\ CLAY) = K_{a_2}\sigma_{y1} - 2c'\sqrt{K_{a_2}} = 0.49 * 7.4 - 2 * 0 * \sqrt{0.49} = 3.6\ kPa$$

- *Horizontal Pressure at 1.0m:*

$$\sigma_H\ (Silty\ CLAY) = K_{a_2}\sigma_{y2} - 2c'\sqrt{K_{a_2}} = 0.49 * 18.675 - 2 * 0 * \sqrt{0.49} = 9.2\ kPa$$

Evaluating the horizontal pressure calculations, the total distributed thrust which may contribute to sliding, bending or overturning stresses on the retaining wall can be seen below in Figure 122. These stresses will dictate the configurations and final dimensions which will be used in the detailed design.

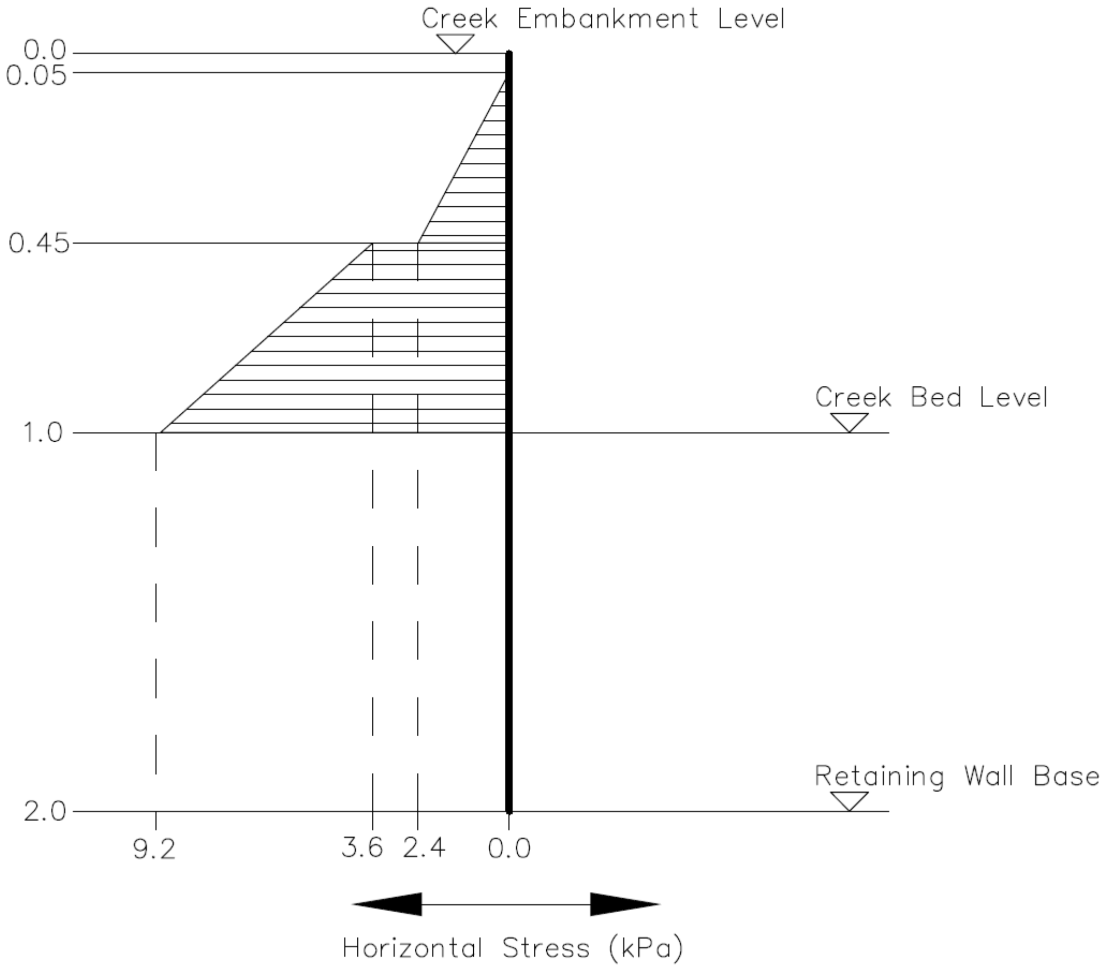


Figure 122: Horizontal Pressure (Hydro-Future 2015)

5.2.4. Configurations

Hydro-Future has looked into several retaining wall configurations which may be suitable for construction along first creek. The final design must meet requirements must be suitable with respect to the aesthetics of the natural surrounds, the high amount of erosion and bacteria processes which occur throughout creeks and the continuous soil composition change which will occur in this area.

5.2.4.1. Concrete Sleepers

The most common design for a retaining wall system used with respect to durability, rigidity and design life are concrete sleepers and steel I beams. The arrangement of the retaining wall can be seen below in Figure 123 which shows the concrete sleepers which interlock themselves within the flanges of a steel I beam due to the self-weight of the structure.

Figure 123: Concrete Retaining Wall (Brislandscaping, 2015)

Advantages:

- Designed for landscaping which is the main purpose of this area of study, to prevent erosion of the creek bed.
- Are rigid, durable and can be used throughout a wide variety of circumstances.
- The design is convenient and effective for quick construction and installation. This is highly regarded as earthwork machinery should not be left within Botanical Park which has a high volume of pedestrian traffic and has limited places for storage.
- The durability of the concrete ensures it may be resilient to algae, rust, fungus and other associate natural bacteria and oxidization processes which are associated with waterways.
- The rigidity of the design prevents long term serviceability failures including movement (deflection or creep).
- The machinery and materials may be temporarily stored in the construction sheds which are currently being utilized by the construction of the future festival center.

Disadvantages:

- The concrete and steel I beam construction does not suit the aesthetics or natural surroundings that is Botanical Park, first creek and the River Torrens.
- Rust may occur throughout the steel I beams over time as these members are likely to be submerged and dried constantly.
- The weight of the retaining wall may not be feasible with respect to the softness of the soil surrounding the creek bed i.e. consolidation may occur over time.
- This construction uses materials which rely on heavy machinery for their transporting in and around the site. This may not be allowed within botanic park as the ground is a combination of delicate grass and modern paved walkways. Referring to Figure 119, the site area has limited space as natural foliage including tress make the access for heavy machinery difficult.
- Concrete is a rigid material and due to the high amount of movement associated with a waterway, cracking of the concrete may occur over time.
- The rigidity of the structure will prevent it from following the curves along first creek.

Cost

Due to the limited costing information provided for some of the material costs, Hydro-Future has assumed a conservative price for unknown materials associated with this option. The pricing for universal beams is conservatively assumed as $450 each based off average pricing online (Midaliasteel, 2015). The cost has been calculated per 1 meter length of the retaining wall with respect to materials only.

Table 52: Concrete Sleepers Cost (Rawlinson's 2014)

Description	*Units*	*Quantity*	*Rate*	*Cost*
Precast Concrete Wall Panels	(m^2)	2m x 1m = 2 m^2	$335.00	$670.00
Universal Beams	Each	Approx. 2	(Assumed)$450.00	$900
		TOTAL PER METER LENGTH		$1,570

5.2.4.2. Wooden Sleepers

Wood and timber sleepers are the ideal type of retaining wall, and they are universally used. They use a similar system to that of concrete sleepers, wooden sleepers which interlock at rigid columns as seen below in Figure 124.

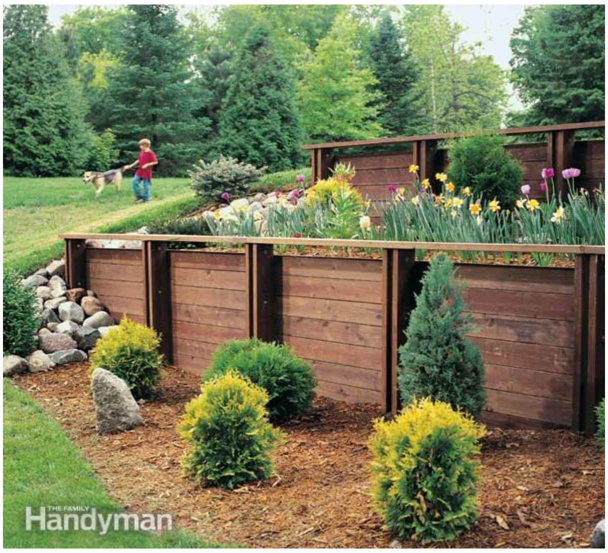


Figure 124: Wooden Sleepers ***(FHM, 2015)***

Advantages:

- Cost Effective and readily available materials.
- Are highly suitable for the soft soil conditions which are located along first creek.
- Are able to withstanding heavy earth pressures.
- They absorb shock and vibrations better than other type of sleeper. This is a highly regarded attribute as the movement and vibration which will occur along water transport systems are far extreme then other conditions.
- The natural appeal and versatility of the timber will highly suit the aesthetics within the botanical garden.
- The flexibility of the timber will allow the design to bend along the corners of first creek.

Disadvantages:

- Wood sleepers are a natural material and will undergo significant decay due to the associated bacteria which will occur within first creek.
- Wooden sleepers are also susceptible to moisture created by first creek or the exposure to sun and rain.
- Deterioration of materials within the associated region will requiring ongoing permanent maintenance

Cost:

Due to the limited costing information provided for some of the material costs, Hydro-Future has assumed a conservative price for unknown materials associated with this option. It has been assumed that domestic construction and carpentry material is suitable for estimating the price for the materials involved. The cost has been calculated per 1 meter length of the retaining wall with respect to materials only.

Table 53: Wooden Sleepers Cost (Rawlinson's 2014)

Description	*Units*	*Quantity*	*Rate*	*Cost*
150x50 (h x w) mm ditto Timber Cross section	(m)	(2/0.15) = l4 planks per meter 14 x 1 ≈ 14 m	$11.90	$166.6
TOTAL PER METER LENGTH				$166.6

5.2.4.3. Gravity Walls

Gravity walls (Figure 125) are constructed using soild concrete, stone or a combination of rock and rubble to form a rigid block. The large segments of gravity retaining walls are highly effective when resisting overturning or sliding earth pressures due to the size and weight of the materials. This structural system provides effective embankments and a cost effective solution, but it is limited in height .i.e they are economical for height up to 3 m (BORAL 2007).

Figure 125: Gravity Retaining Wall (ROADWAY, n.d.)

Advantages:

- Constructed of highly durable material which will resist movement and ersoion from first creek.
- Cheap and simple construction.
- Due to the weight they require a smaller quanitiy of select backfill.

Disadvantages:

- The retaing walls stability will be reduced as the height increases. As the retaining wall along first creek will need to be a significent distance below the ground level this may hinder the effectivness of this design.
- Requires heavy machinery to construct the design, this will prioduce the same issues developed under concrete sleepers.
- A high amount of excavation is required behind the retaining wall to allow for backfilling.
- The retaining wall requires the foundation soil to retaing a high bearing capacity which may not be available due to the soil softness associated with first creek.
- Consoidation will occur over tuime due to the weight of the structure.

Cost:

Due to the limited costing information provided for some of the material costs, Hydro-Future has assumed a conservative price for unknown materials associated with this option. The cost has been calculated per 1 meter length of the retaining wall with respect to materials only (Table 54).

Table 54: Gravity Wall Cost (Rawlinson's 2014)

Description	Units	Quantity	Rate	Cost
Precast Concrete Wall Panels	(m^2)	2m x 1m + 1/2(0.5mx0.5m) x 1m = 2.125 m^2	\$335.00	\$711.9
TOTAL PER METER WIDTH				\$711.9

5.2.4.4. Cantilever Walls

Cantilever walls (Figure 126) are built of reinforced concrete and are supported using a horizontal footing and a vertical stem wall. The weight of the soil mass above the heel helps keep wall stable which resists moment and sliding forces. Cantilever walls are economical for heights up to 6 m.

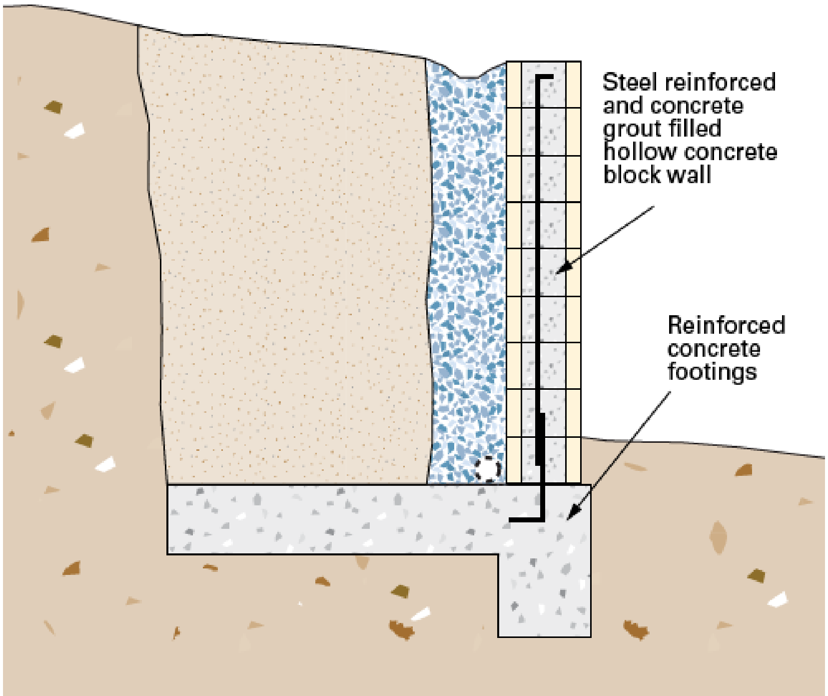


Figure 126: Cantilever Retaining wall (BORAL 2007).

Advantages:

- Will be highly durable against erosion and decay along the creek.
- Economical and simplistic with construction. Which will be highly regarded within Botanical Park.
- The reinforced concrete wall takes up little space which reduces excavation and a change in the creeks dimensions.
- Is suitable up until a depth of 6 meters which is far more than required for this situation and location.
- Readily available materials (concrete and steel)

Disadvantages:

- Limited height, maximum excavation for cantilever walls is rather limited, typically up to about 6 m.
- Deep foudation support may be necessary which will increase the excavation.
- Realatively long construction time which will hinder the ability to store materials and machinery within botanical park.
- In general it is not recommonded to used of this type of retaining walls next to adjacent buildings (BORAL 2007 & Earth Retaing structures 2015).

Cost:

Due to the limited costing information provided for some of the material costs, Hydro-Future has assumed a conservative price for unknown materials associated with this option. The cost has been calculated per 1 meter length of the retaining wall with respect to materials only.

Table 55: Cantilever Wall Cost (Rawlinson's 2014)

Description	Units	Quantity	Rate	Cost
Concrete Cantilever Walls (Assumed Concrete 40MPa	(m^3)	3m x 2m x 1m = 6 m^2	$243.00	$1458
Reinforcement	(t) i.e. tonne	Assuming 5 kg per m length, = 5kg/m x 1m = 5 kg	$1,850	(1850/1000)*5 $9.25
		TOTAL PER METER LENGTH		$1467.25

5.2.4.5. Crib Walls

Crib walls are one the oldest gravity walls systems. They are made up of interlocking individual boxes made from timber or pre-cast concrete. These boxes are then filled with crushed rock, stone or other coarse granular materials to generate a free draining structure which can be seen below in Figure 127.

Figure 127: Crib Wall (Phi Group 2015)

Advantages:

- Ease of construction will reduce the time required within Botanical Park.
- Crib walls section can be pre-cast and transported to the site and held in stock for emergency works, this is ideal as no construction is required on site.
- Wall has substantial flexibility which will suit the surroundings of a creek.
- Can be made to look aesthetically pleasing with further plantation in and around the crib wall.

- Height can be interchanged easily and is the most economical solution for varying wall heights.

Disadvantages

- Usually needs a concrete base which will be difficult within a creek.
- Wood / Timber is not highly durable in the required location.
- Pre-casting will require large amounts of transport in and around Botanical Park.
- They are not very economical for short length of wall which is generally required within this project.
- Permeability of the wall may still cause sediment transfer while the creek is full.

Cost:

Due to the limited costing information provided for some of the material costs, Hydro-Future has assumed a conservative price for unknown materials associated with this option. It has been assumed that domestic construction and carpentry material is suitable for estimating the price for the materials involved. The cost has been calculated per 1 meter length of the retaining wall with respect to materials only.

Table 56: Crib Wall Cost (Rawlinson's, 2014)

Description	Units	Quantity	Rate	Cost
150x50 (h x w) mm ditto Timber	(m)	2/0.15 = 14 planks per meter 14 x 1 x 4 ≈ 56 m	$11.90	$666.4
	TOTAL PER METER LENGTH			$666.4

5.2.4.6. Gabion Retaining Wall

Gabions retaining structures as seen in Figure 128 are cages which are made of welded wire or rectangular wire mesh boxes filled with rocks, stones or sometimes sand and soil. They are used for construction of erosion control structures and the stabilisation of steep slopes. Gabions can fixed to ground movement, dissipate energy from flowing water, and drains free. Their strength and effectiveness may increase with time, as vegetation and silt fill the voids and reinforce the structure (Sahero Gabion Barrier n.d.).

Figure 128: Gabion Wall (Sahero Gabion Barrier n.d.).

Advantages:

- The construction is very simple, reducing the amount of skilled labor and specialized equipment which may be required for other designs.
- The materials used can be sourced locally due to the high organic compounds which are utilized.
- Highly flexible and dissipate energy from flowing water which will reduce the erosion and sediment transport within first creek.
- Does not require a foundation as the friction between the base and the underlying gravel is significantly high. This is highly regarded as a foundation within the creek will produce design difficulties.
- Pre-assembled will allow for fast construction within botanical gardens.
- Can be aesthetically pleasing with the plantation of native flora.

Disadvantages:

- Potential for loss of material through the cage voids. This is likely to occur due to the movement of water along first creek.
- The design is heavy in construction and will require heavy vehicle transport within first creek.

Cost:

Table 57: Gabion Wall Cost (Rawlinson's 2014)

Due to the limited costing information provided for some of the material costs, Hydro-Future has assumed a conservative price for unknown materials associated with this option. The cost has been calculated per 1 meter length of the retaining wall with respect to materials only.

5.2.4.7. Reinforced soil wall

Description	Units	Quantity	Rate	Cost
Crushed rock filling laid and consolidated in 150mm layers	(m^3)	2m x 2m x 1m = 4 m^3	$61.00	$244.0
Mesh Sheeting (Assumed reinforcement mesh SL72 fabric steel wire galvanised finish)	(m^2)	2m x 1m x 4 sides = 8m^2	$9.65 + $5 Assumed Galvanised finish	$117.2
TOTAL PER METER LENGTH				$361.2

Reinforced soil walls as seen below in use layers of geogrids or strips to combine the soil and block together to form a reinforced soil mass. These walls are composed of vertical (commonly concrete) facing panels attached to metal or plastic reinforcement (in the form of strips) in the soil behind the wall. They are often used in highway embankments where a vertical slope is needed, for example at a grade-separated junction (Bruce & Jewell n.d.).

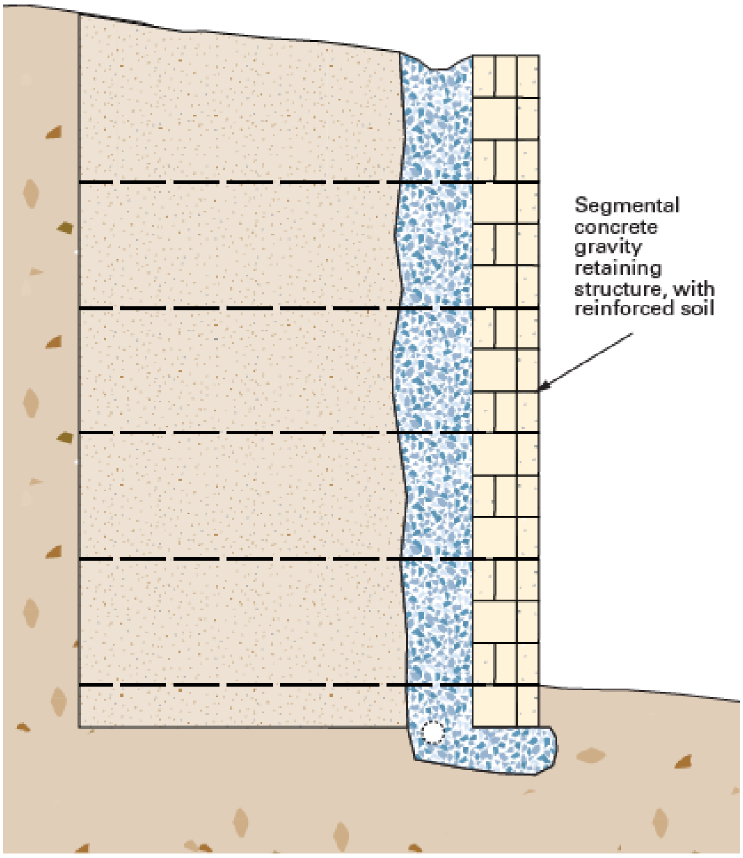


Figure 129: Reinforced Soil Wall (BORAL, 2007)

Advantages:

- This retaining wall design can be constructed by hand which limits the amount of heavy vehicle machinery required.
- Highly flexible design and its ability to resist large deformations without stress is ideal for first creek.
- No foundation within first creek will be required.

Disadvantages:

- Constructed into the ground, so probably only cost effective if constructing wall in excavated or slipped material.
- Requires use of select backfill which will require transport to and from the Botanical Gardens.
- Subject to corrosion in aggressive environment (metallic reinforcement) which will be likely to occur along first creek.

Cost:

Due to the limited costing information provided for some of the material costs, Hydro-Future has assumed a conservative price for unknown materials associated with this option. The cost has been calculated per 1 meter length of the retaining wall with respect to materials only.

Table 58: Reinforced Soil Wall Cost (Rawlinson's 2014)

Description	*Units*	*Quantity*	*Rate*	*Cost*
Precast Concrete Wall Panels	(m^2)	2m x 1m = 2 m^2	\$335.00	\$670
Reinforcement (Anchor - Tendon)	t	Assumed 0.2	\$1850	\$370.00
Prestressing - Continuity	t	Assumed 0.2	\$6300	\$1260.00
TOTAL PER METER LENGTH				\$2300

5.2.4.8. Soil Nail Wall

Soil nailing is a method to stabilising existing embankments using grouted steel bars. A soil nailing wall usually consists of the soil nails themselves, a hard, flexible or soft facing to the wall surfaces and surface water and sub-surface drainage systems. It is an effective and economical technique of constructing retaining wall for excavation support, support of hill cuts, roads and highway. The process is effective in cohesive soil, broken rocks, shales and fixed face conditions (Bruce & Jewell n.d.).

Figure 130: Soil Nail Wall (Kutschke, Tarquinio & Petersen 2007)

Advantages:

- Requires cohesive soils which are largely available along first creek.

Disadvantages:

- *This design is only suitable above groundwater, hence it is likely to fail if it is implemented along first creek.*
- *Specialist equipment and labor is required for the construction.*

Cost:

Due to the limited costing information provided for some of the material costs, Hydro-Future has assumed a conservative price for unknown materials associated with this option. The cost has been calculated per 1 meter length of the retaining wall with respect to materials only.

Table 59: Soil Nail Wall Cost (Rawlinson's 2014)

Description	Units	Quantity	Rate	Cost
Mesh Retaining Wall – Prefabricated Steel Wire facing panels, hot dipped galvanised finish	(m^2)	2m x 1m = $2m^2$	$280.00	$560.0
TOTAL PER METER LENGTH				$560

5.2.5. Evaluation

As can be seen below in Hydro – Future has evaluated each configuration in terms of cost, capability to retain first creek, resistance to corrosion / Erosion and Visual Amenity.

Table 60: Retaining Wall Evaluation Matrix

Configuration	Criteria				
	Cost (30%)	Capacity to Contain First Creek (30%)	Resistance to Corrosion / Erosion (30%)	Visual Amenity (10%)	Total
Concrete Sleepers	30	70	70	50	52
Wooden Sleepers	20	60	10	95	36.5
Gravity Walls	70	90	90	60	81
Cantilever Walls	40	90	90	60	72
Crib Walls	75	50	50	85	61
Gabion Retaining Walls	**95**	**95**	**95**	**90**	**94.5**
Reinforced Soil Wall	10	90	85	85	64
Soil Nail Wall	85	0	10	0	28.5

Hydro-Future recommends the implementation of a Gabion Retaining Wall design at a cost of ***$360.2*** per meter length.

5.2.6. Alternatives

As an alternate design option a stable slope can be considered as opposed to a retaining wall design. This design option will be environmentally friendly, cost effectively and simple throughout the construction and design process. However a stable slope is more susceptible to erosion hence a dense gravel or rock layer should be placed above the slope to reduce the amount of sediment transfer.

Based on the geotechnical information such as borehole data, and our site observations, a slope stability study can be carried using the software package 'Galena'. A site visit was made by a geotechnical engineer to carry out visual tactile testing according to AS 1726 Geotechnical Site Investigations. An examination of the general slope conditions, including vegetation conditions on the slope and creek bank erosion conditions were observed and documented.

Based on our site visit and observations, the slope conditions are described as follows:

- The height of the slope from the edge of water to the top of bank generally ranges from 2 to 3 m. The steepness of the existing slope varied from 1.1H: 1V to 1.6H: 1V.
- The slope surfaces are generally covered with grass, as shown in the photographs in Figure 116.
- No seepage was observed on the slope surfaces during our site visit.
- Creek bank erosion and surface was observed at some locations.

In conjunction with a stable slope design, Articulate Concrete Revetment Mats can be used to maintain the slopes stability and increase the amount of vegetation which is allowed to be planted along the sides of the slope.