5. Environmental – Gross Pollutant Trap

As discussed previously, a gross pollutant trap is to be implemented at the downstream end of North Terrace, at the end of and in line with the main 750mm stormwater collector pipe and before the sandstone arch culvert at First Creek. A secondary 750mm pipe will carry the filtered stormwater from the outlet side of the gross pollutant trap into the sandstone arch culvert, through a newly implemented opening in the side.

5.1. Purpose and Location

Gross pollutant traps (GPTs) are implemented in existing developments, new developments and infrastructure upgrade projects to remove debris, litter and sediments from stormwater, normally as a pre-treatment before the stormwater travels into a pond, creek or wetland.

For the North Terrace drainage system upgrade project, a GPT is required to be installed at the outlet of the main stormwater pipe at the downstream end, and before the heritage listed sandstone arch culvert.

5.2. Design Requirements

A number of design parameters are required for the GPT to ensure it functions with the complete drainage network along North Terrace.

A number of these include:

- Sufficient width for the 750mm inflow pipe
- The capacity to process the 1.57 m^3/s total flow rate from North Terrace (20 year ARI)
- The capacity to process a 1 in 3 month treatment flow/water quality storm to retain all litter and debris (as standard for all GPTs)
- The capacity to hold the 7.21 hectare sub-catchment area
- Easily cleaned by the use of eductor trucks (vacuum trucks)

5.3. Specific GPT Type

A number of different GPT types exist, each with varying benefits and limitations and ultimately design purposes. To meet the aforementioned design requirements, the GPT suited to the North Terrace drainage design application is the SBTR trap, as can be seen in Figure 54 below.

This GPT combines the functions of sedimentation basins and trash racks, hence the name. Some of the specifications and benefits of the SBTR trap include:

- An enclosed system, installed under ground

- Suited to urban/city areas as the system is virtually hidden
- Best suited to pipe drainage systems
- Ability to treat catchment areas up to 2000 hectares
- Cost effective (for both implementation and maintenance)

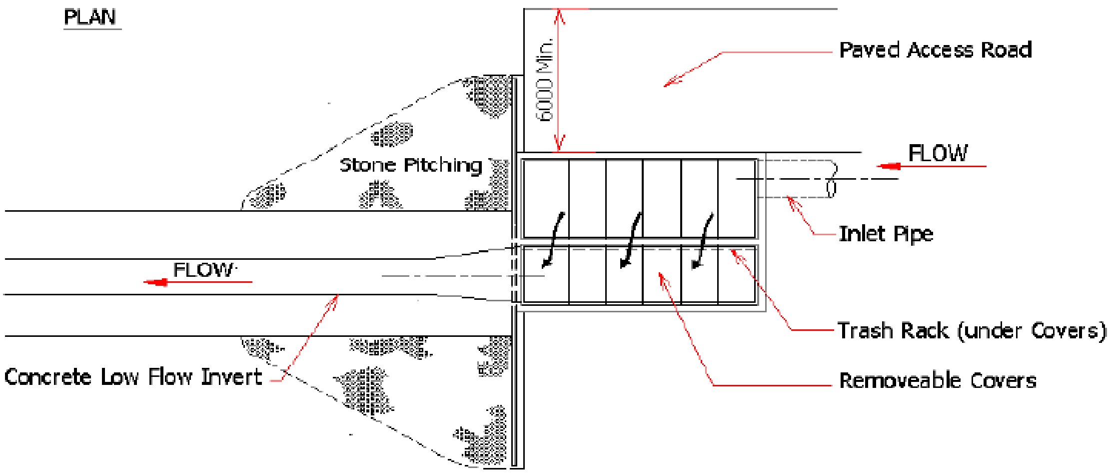


Figure 54 - SBTR Type 2 GPT Layout Diagram (Department of Irrigation and Drainage Malaysia, 2012)

A ranking chart of the SBTR trap in relation to other GPT types is shown in Figure 55 below and shows that the relative cost of the trap is medium and the relative cost effectiveness is high. The pollutant removal scores are high in comparison to some of the other GPT types.

	Pollutants						Combination of Pollutants													Cost-effectiveness	
	Litter	Organic Matter	Sediment	Oils and Grease	Metals	Nutrients	Litter, Sediment	Litter, Oils	Litter, Organics	Litter, Nutrients	Organics, Sediment	Organics, Nutrients	Litter, Organic, Oils	Litter, Sediment, Oils	Litter, Organic, Sediment	Litter, Sediment, Nutrients	Litter, Oils, Organic, Sediment	Litter, Oils, Organics, Metals	Litter, Organics, Sediment, Nutrients	Relative Cost	Relative Effectiveness
GPT STRUCTURES																					
Floating Debris Trap: boom, Bandalong	5	3	1	7	1	1	3	6	4	3	2	2	5	4	3	2	4	4	3	low	low
In-pit devices	5	5	3	1	1	1	4	3	5	3	4	3	4	3	4	3	3	3	4	low	low
Litter Control Device: Net-tech	7	7	1	1	1	1	4	4	7	4	4	4	5	3	5	3	4	4	4	medium	medium
Trash rack	7	7	6	1	6	1	7	4	7	4	7	4	5	5	7	5	5	5	5	medium	medium
SBTR trap	7	7	7	4	7	5	7	6	7	6	7	6	6	6	7	6	6	6	7	medium	high
Proprietary devices: Baramy	8	8	4	1	4	2	6	5	8	5	6	5	6	4	7	5	5	5	6	medium	medium

Figure 55 - Ranking of the SBTR trap in relation to other GPTs (Department of Irrigation and Drainage Malaysia, 2012)

As stormwater enters the SBTR trap, the coarse sediments settle, due to the decrease in flow velocity. As the channel depth and/or width is adjusted, the flow velocity is reduced and ultimately leads to a longer retention time, allowing more sediments to settle to the channel base.

Due to the design of sedimentation basin trash rack GPTs, they are generally not designed to withstand traffic loads, unless the interior is reinforced to meet the necessary structural requirements. For the purpose of this project, since the GPT is located under an arterial road, with high AADT and VPD statistics, the GPT would require to be heavily reinforced to withstand traffic loads, which would increase the overall dimensions of the GPT. Normally, the top of the GPT would be constructed 150mm above the existing ground level or a barrier would be constructed around the GPT, both to avoid the GPT being driven over by vehicles (Department of Irrigation and Drainage Malaysia, 2012), however this would require a lane on North Terrace to be closed off. For the scope of this project, a bypass system will be constructed to allow vehicles to drive over the GPT, which will distribute loads to the existing surface of North Terrace at a distance of at least 2.5 metres either side of the GPT, which will allow all lanes to remain open on North Terrace, as can be seen in Figure 56 below.

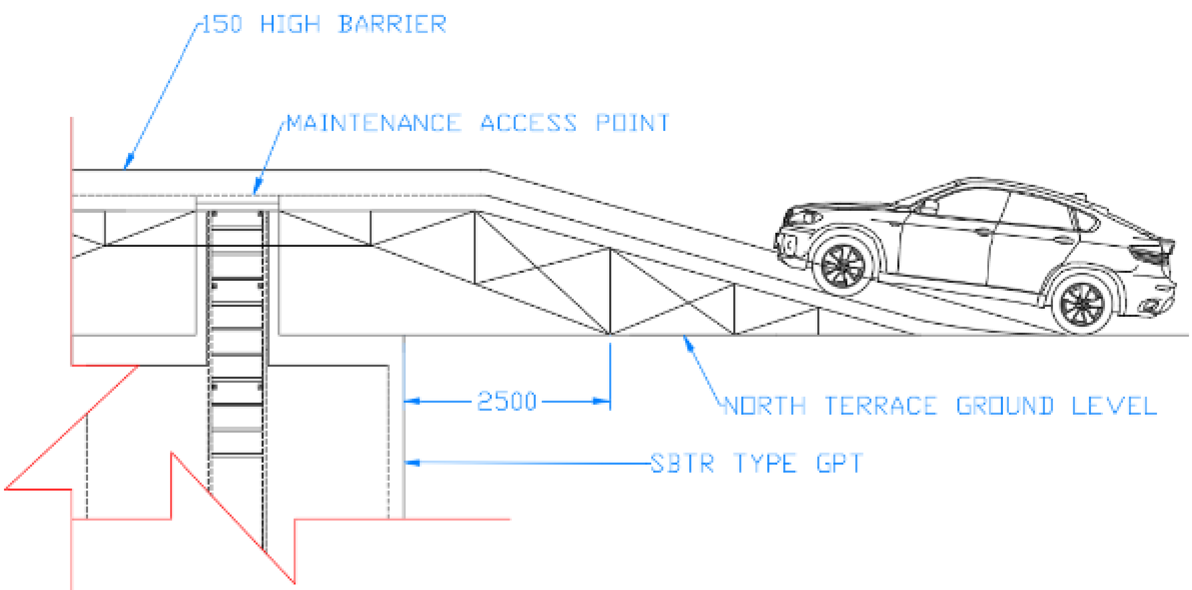


Figure 56 - GPT Bypass Design Drawing (Hydro-Future Consulting, 2015)

5.4. Trash Rack

Trash racks assists to collect the larger pollutants and debris such as litter in a GPT system. The location of the trash rack will be at the downstream end/side of the GPT and is placed perpendicular to the inflow pipe.

5.5. Design Requirements for SBTR Traps

- The width of sediment trap is required to have a ratio length between 2 and 3
- Velocity of stormwater flowing through the trap has to be less than or equal to 1m/s

- The trash rack is required to pass the required design flow efficiently without overtopping and with a 50% blockage rate.
- The trash rack is required to withstand log impacts and drag loads. In the event that the rack is completely blocked (100%), water should be diverted past rack (through the overflow clearance above trash rack).

5.6. Gross Pollutant Trap Calculations

The outlet side, which is to the opposite side to the inlet side does not require design calculations, but is rather determined based on the outlet pipe diameter and the associated clearance for proper functionality.

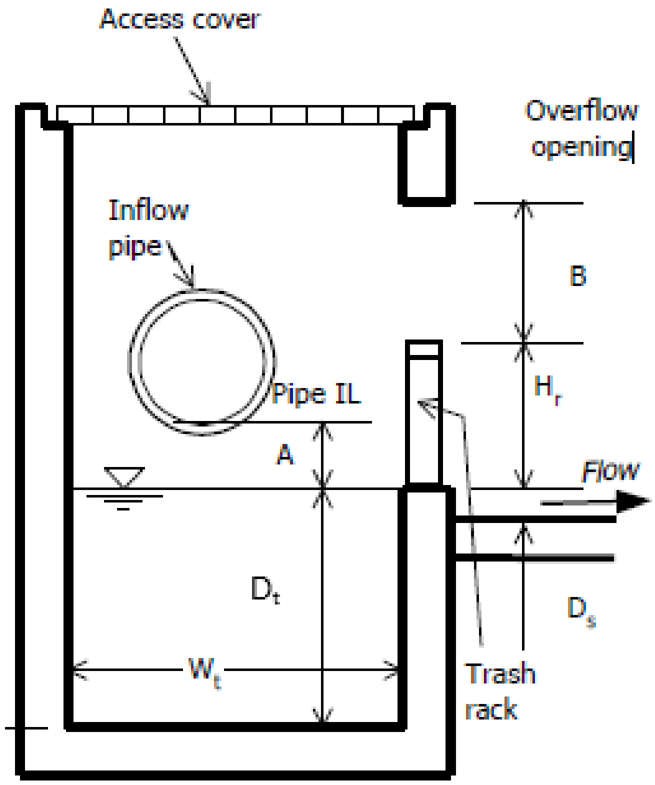


Figure 57 - SBTR Type 2 Gross Pollutant Trap Diagram (Department of Irrigation and Drainage Malaysia, 2012)

A number of the following calculations for dimension determination will relate to Figure 57 and are expressed as symbols, as seen above.

5.6.1. Required Removal Efficiency

As depicted in Figure 58 below, according to Chapter 4 of the Urban Stormwater Management Manual 2012, for a drainage system upgrade, the target pollutant removal rate of sediments is 20% (Department of Irrigation and Drainage Malaysia, 2012). Chapter 4 of the Urban Stormwater Management Manual also states that "*Gross pollutant traps (GPTs) shall be sized to* *retain 100% of all litter and debris greater than 1mm in size and a minimum of 70% of coarse sediments greater than or equal to 70%"* (Department of Irrigation and Drainage Malaysia, 2012). Based on this a 70% target rate is used for the GPT design. The 20% target for upgrading of drainage systems is used as a starting point for trap area ratio determination and is adjusted according to the outcome, but will ultimately need to yield a 70% target pollutant removal rate for coarse sediments.

Table 4.5 Pollutant Retention or Load Reduction Targets

Pollutant	New Development	Land Redevelopment (see note)	Drainage System Upgrading
	Annual Average Pollutant Removal Efficiency (%)	Reduction in Annual Average Pollutant Load from Existing Conditions (%)	Reduction in Annual Average Pollutant Load from Existing Conditions (%)
Floatables	90	90	30
Sediment	70	50	20
Suspended Solids	60	40	20
Nitrogen	50	30	20
Phosphorus	50	30	20

Note: Local Authorities may set lower targets for redevelopment to take account of land constraints.

Figure 58 - Pollutant Retention or Load Reduction Targets for GPT Design (Department of Irrigation and Drainage Malaysia, 2012)

5.6.2. Catchment Area

The catchment area (A_c) as calculated in the Feasibility Study is:

$$A_c = 7.21ha$$

The percentage of impervious or urbanised area (U) for the catchment as determined in the Feasibility Study is 90%

$$U = 90\%$$

5.6.3. Trial Trap Area Ratio (R)

As previously mentioned, from Figure 58 above, the GPT in design for this project is required to have a 20% reduction target for sediments. This is based on the 'drainage system upgrading' category.

Since our catchment area is 90% impervious (U=90%), for the upper curve (curve A) for soil grain sizes above 0.04mm, the Area Ratio, R can be found. The annual sediment retention is taken as 20% as mentioned previously.

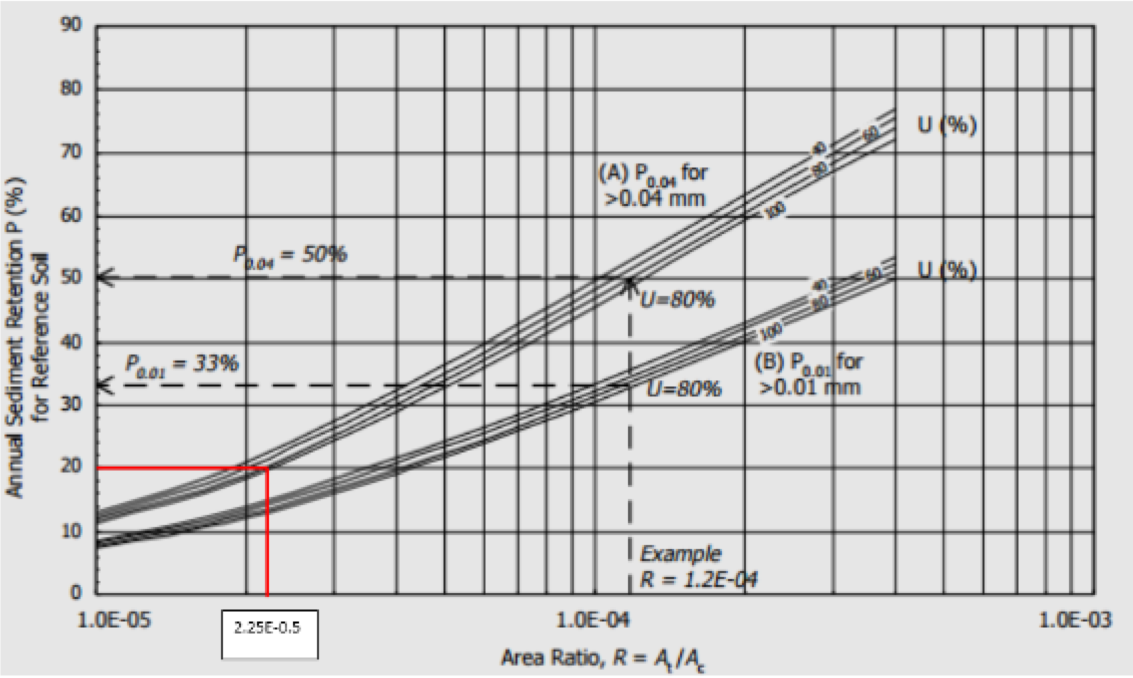


Figure 59 - Annual Sediment Retention vs Area Ratio Graph (Department of Irrigation and Drainage Malaysia, 2012)

The area ratio can be found from using the graph in Figure 59 above from chapter 34 of the Urban Stormwater Management Manual, based on a 20% sediment retention target and an urbanised area percentage of 90%.

$$Area\ Ratio, R = 2.25 \times 10^{-5}$$

5.6.4. Average Annual Retention of Sediment

Using the following graph in Figure 60, the F1 factor is found (which is used for soil grain size >0.04mm).

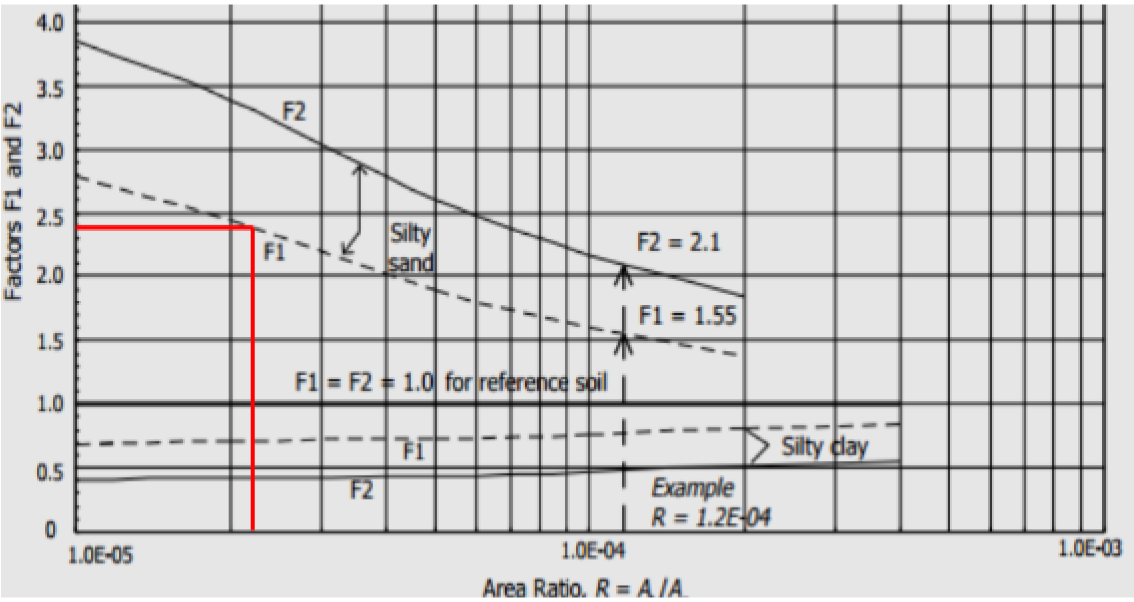


Figure 60 - F1 and F2 factors vs Area Ratio Graph (Department of Irrigation and Drainage Malaysia, 2012)

So based on the 20% target removal ($P_{0.04}$=20%) and the found Area Ratio, R,

F1 = 2.4 is found (F1 curve relates to 0.04mm soil grain size).

5.6.4.1. Trap Removal efficiency

$$P_{0.04} *= 20\% \times 2.4 = 48\%$$

Since the minimum target requirement for sediments over 0.04mm is 70%, as previously stated, the trap removal efficiency is no good and requires an increased initial target range.

Increase P target range to 45%:

$$R \ = \ 9.5 \times 10^{-5}$$

$$F1 = 1.62$$

$$P_{0.04} *= 45\% \times 1.62 = 73\%$$

Since this is greater than 70%, it is satisfactory.

5.6.4.2. Trap Size Area

As stated previously, the catchment area is 7.21 hectares. The trap size area is therefore:

$$A_t \ = \ R \times A_c \ = \ 9.5 \times 10^{-5} \times \ 72100m^2 = 6.85m^2$$

5.6.4.3. *Length and Width of Trap*

$\frac{L_t}{W_t}$ is required to between 2 and 3.

Try dimensions:

Lt= 4m and Wt = 1.8m

$$\frac{L_t}{W_t} = 2.22$$

Actual Trap Area:

$$A_t = 4m \times = 1.8m = 7.2m^2 > 6.85m^2$$

Trap is slightly bigger than required to allow room for placement of fixed trash rack.

5.6.5. Average Annual Sediment Export

In order to calculate the average annual sediment export (M), a basic formula is required. No South Australian data could be located for this application, so the data available for ACT and Brisbane has been used (Ahammed, 2015) as seen in Figure 61 below.

Table 15.A1 Storm Event Pollutant Exports (kg/km^2) for ACT and Brisbane, Australia

Pollutant	Landuse/vegetation categories		
	Native vegetation/ forest	Rural grazing	Established Urban
Sediment – ACT (no Brisbane data)	$200R^{1.1}$	$400R^{1.1}$	$1000R^{1.4}$
Suspended solids-ACT Brisbane	8R $130R^{0.75}$	20R 6.1R	200R $166R^{0.75}$
Total phosphorus-ACT Brisbane	$0.05R^{0.57}$ $0.17R^{0.9}$	$0.12R^{0.57}$ 0.022R	$0.4R^{0.8}$ $0.15R^{0.9}$
Total nitrogen – ACT Brisbane	$0.15R^{1.6}$ $1.5R^{0.86}$	$0.3R^{1.6}$ 0.16R	$3R^{0.84}$ $1.45R^{0.86}$
Faecal coliforms ACT (cfu/km^2) Brisbane	$30\text{-}100\times10^9 R^{0.9}$ $6.4\times10^9 R^{1.1}$	$300\text{-}1500\times10^9 R^{0.9}$ $1.0\times10^9 R^{0.95}$	$400\text{-}1000\times10^9 R^{0.9}$ $10.3\times10^9 R^{1.1}$

R = event runoff, in mm

Source: Willing & Partners (1999)

Reference areas: Brisbane and Canberra, Australia

Figure 61 - Storm Event Pollutant Exports for ACT/Brisbane (Department of Irrigation and Drainage Malaysia, 2012)

The formula required for this application is for the established urban category as follows:

$$M = 1000R^{1.4}$$

Where R = event runoff, in mm.

The rainfall depth (event runoff) is obtained through the Bureau of Meteorology Website for the project area. The IFD relationship requires input of the location in terms of geographical coordinates (Figure 62).

Location

Label: Kent Town

Latitude: 34.9203 [Nearest grid cell: 34.9125 (S)]

Longitude: 138.6192 [Nearest grid cell: 138.6125 (E)]

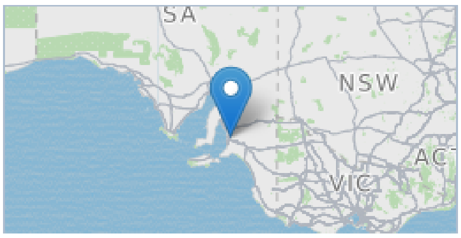


Figure 62 - Project Area Results for IFD Data System (Bureau of Meteorology, 2015)

The IFD design rainfall depth is then displayed as well as exceedance per year (EY) and respective annual exceedance probabilities (AEP), as seen in Figure 63 below.

IFD Design Rainfall Depth (mm)

Issued: 20 May 2015

Rainfall depth for Durations, Exceedance per Year (EY), and Annual Exceedance Probabilities (AEP).

	EY	Annual Exceedance Probability (AEP)					
Duration	**1EY**	**50%**	**20%**	**10%**	**5%**	**2%**	**1%**
1 min	1.3	1.5	2.1	2.6	3.1	3.9	4.5
2 min	2.3	2.6	3.6	4.5	5.4	6.8	7.9
3 min	3.0	3.4	4.9	6.0	7.2	9.0	10.6
4 min	3.6	4.1	5.9	7.3	8.7	10.9	12.7
5 min	4.2	4.8	6.8	8.3	10.0	12.5	14.6
10 min	6.0	6.9	9.9	12.1	14.6	18.2	21.2
15 min	7.3	8.3	11.9	14.7	17.6	22.0	25.6
30 min	9.7	11.1	15.9	19.5	23.4	29.2	34.1
1 hour	12.6	14.3	20.4	25.0	30.0	37.4	43.8

Figure 63 - IFD Design Rainfall Depth Results for Project Area (Bureau of Meteorology, 2015)

From the 7.21 hectare catchment with 90% directly-connected impervious area (DCIA), the rainfall for 3 months ARI is as follows:

In reference to Figure 63, 1EY = 1 year ARI. For 10 minute duration (T_c) = 6.0mm.

Since there is no probability for 3 months ARI, take the event runoff, $I_{0.25}$ as 4mm (Ahammed, 2015).

$$I_{0.25} = 4mm$$

Since 90% of the catchment area is urbanised or impervious:

$$R = U \times I_{0.25} = 0.9 \times 4mm = 3.6mm$$

Therefore, the average annual sediment export (M) for Kent Town is found:

$$M = 1000 \times R^{1.4} = 1000 \times 3.6^{1.4} = 6\ tonnes$$

5.6.6. Average Annual Pollutant Retention

Again, referring to Figure 6, the pollutant retention for reference soil $P_{0.01}$ = 31% and F2 = 2.2.

Equation 34.4 of the Urban Stormwater Management Manual displays the pollutant retention for site soil (Department of Irrigation and Drainage Malaysia, 2012) as follows:

$$P_{0.01} * = 31\% \times 2.2 = 68.2\%$$

5.6.7. Minimum Sediment Trap Depth

Equation 34.5 of the Urban Stormwater Management Manual displays the pollutant retention for site soil (Department of Irrigation and Drainage Malaysia, 2012) as follows:

$$D_t = 0.0065 \times P_{0.01} * \times \frac{M}{A_t} = 0.0065 \times 68.2 \times \frac{6}{7.2} = 0.37m$$

5.6.8. Peak Flow for Water Quality Design Storm

In order to calculate the peak flow for a 3 months ARI water quality storm, Argue's design principles of source control are used from the 1986 book 'Storm drainage design in small urban catchments: a handbook for Australian practice'.

From Section 13.2.8 and using equation 13.5b of the Urban Stormwater Management Manual the IFD value for frequent storm events can be calculated (Department of Irrigation and Drainage Malaysia, 2012) as follows:

$$0.25_{I_D} = 0.5 \times 2_{I_D}$$

Where 2_{ID} is the rainfall intensity for a 2 year ARI, for the project area. From Figure X below, the 2 year ARI is displayed for 10 minutes duration.

Intensity-Frequency-Duration Table

Location: 34.925S 138.625E NEAR.. Kent Town Issued: 24/5/2015

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

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

(Raw data: 17.77, 3.79, 0.91, 35.01, 6.8, 1.72, skew=0.56, F2=4.47, F50=14.98)

Figure 64 - Intensity Frequency Duration Results Table (Bureau of Meteorology, 2015)

Based on 10 minutes storm duration (Ahammed, 2015) and 2 year ARI:

$$2_{I_D} = 46mm/hr$$

So as introduced before:

$$0.25_{I_D} = 0.5 \times 46mm/hr = 23mm/hr$$

Now, by using Argue's principles, presented in 'Storm drainage design in small urban catchments: a handbook for Australian practice', 1986, the flow rate for a 3 month ARI can be calculated.

$$\mathrm{Q} = \frac{\mathrm{CIA}}{360} \text{ where:}$$

$$\mathrm{Q} = \text{design flow rate } \left(\frac{\mathrm{m}^3}{\mathrm{sec}}\right); \mathrm{C} = \text{runoff coefficient;}$$

$$\mathrm{I} = \text{rainfall intensity } \left(\frac{\mathrm{mm}}{\mathrm{hr}}\right); \mathrm{A} = \text{catchment area (ha);}$$

The basic runoff coefficient for our catchment area (U=90% from before) can be found using Table 5.3 or Argue's 1986 aforementioned book, as see in Figure X below and is taken as C_{10}=0.9 for paved areas in South Australia Zone.

Surface	Basic runoff coefficient (C_{10})	
	Northern Australian Zone	Southern Australian Zone
Paved areas - roads - roofs	0.90	0.90
Pervious areas	0.70	0.10

Figure 65 – Basic Runoff Coefficient Values (Argue, 1986)

The derived runoff coefficient is also required, however, in Table 5.5 of Argue's 1986 aforementioned book, there is no value provided for 0.25 years ARI (Figure 66). Therefore C_{10} as 1 year ARI (Ahammed, 2015) is taken. So C_{10} = 0.8.

ARI (yrs)	1	2	5	10	20	40	60	80	100
C_{10}	0.8	0.85	0.95	1.00	1.05	1.13	1.17	1.19	1.20

Figure 66 - Derived Runoff Coefficient Values (Argue, 1986)

To find the 3 month ARI:

From Argue's 1986 aforementioned book:

$$C_y = F_y \times C_{10}$$

Paved Areas:

$$C_{0.25_{paved}} = 0.9 \times 0.8 = 0.72$$

Pervious Areas:

$$C_{0.25_{pervious}} = 0.1 \times 0.8 = 0.08$$

Now, the effective runoff coefficient can be derived:

$$C_{10} = \frac{a1.C1 + a2.C2}{A}$$

$$C_{20} = \frac{(90 \times 0.72) + (10 \times 0.08)}{90 + 10} = 0.656$$

Finally the flow rate can be calculated, using the formula introduced earlier:

$$Q_{0.25} = \frac{0.656 \times 23 \times 7.21}{360} = 0.302m^3/s$$

5.6.9. Trash Rack Height

Equation 34.7 of the Urban Stormwater Management Manual (Department of Irrigation and Drainage Malaysia, 2012) displays the formula for trash rack height (H_r) as follows:

$$H_r = 1.22\left(\frac{Q_{0.25}}{L_r}\right)$$

Try length of trash rack as 1.8m to be same as width of trap.

$$H_r = 1.22 \times \left(\frac{0.302}{1.8}\right) = 0.20m$$

Take height of rack to be 0.2m for ease of construction.

5.6.10. Nominal Flow Velocity

Equation 34.8 of the Urban Stormwater Management Manual (Department of Irrigation and Drainage Malaysia, 2012) displays the formula calculating flow velocity for a 3 month ARI and stipulates a maximum velocity of 1m/s:

$$V_{0.25} = \frac{Q_{0.25}}{(D_t + H_r)W_t}$$

$$V_{0.25} = \left(\frac{0.302}{(0.4 + 0.2) \times 1.8}\right) = 0.30m/s < 1m/s$$

5.6.11. Minimum Overflow Clearance

Equation 34.9 of the Urban Stormwater Management Manual (Department of Irrigation and Drainage Malaysia, 2012) displays the minimum overflow clearance (B) formula as follows:

$$B = \left(\frac{Q_p}{1.7L_r}\right)^{\frac{2}{3}}$$

Using the total flow rate from the inlet pipe (1.57m^3/s) based on 20 year ARI:

$$B = \left(\frac{1.57}{1.7 \times 1.8}\right)^{\frac{2}{3}} = 0.64m > 0.35m$$

Since the catchment area is relatively small, the overflow clearance does not need to be this large as it would be excessive. Take as 0.40-0.45m (Ahammed, 2015). The minimum overflow clearance for the design is 0.43m.

5.6.12. Wall Thickness

The GPT structure is constructed of reinforced concrete at a thickness of 180mm for all walls to withstand associated soil pressure as well as any contributed loads from trapped debris and hydrodynamic pressures and loads.

5.7. Final Design

Table 23 below displays the final design specifications in terms of dimensions and internal areas. The detailed design drawing entailing these dimensions along with the location and associated items is included in CAD drawings HF-201A and HF-201B).

Table 23 - Gross Pollutant Trap Final Specifications

Gross Pollutant Trap Final Specifications	
Trap Size Area	$7.2m^2$
Length of Trap	4m
Width of Trap	1.8m
Sediment Trap Depth	0.4m
Trash Rack Height	0.2m
Wall Thickness	180mm

Table 24 below displays the GPTs final performance results, displaying that it meets the minimum or maximum requirements as specified by the Urban Stormwater Management Manual 2012, Department of Irrigation and Drainage Malaysia.

Table 24 - Gross Pollutant Trap Performance Outcomes

Gross Pollutant Trap Performance Outcomes		
Type	**Required**	**Final Design**
Trap Removal Efficiency	70% minimum	73%
Annual Average Sediment Export	-	6 tonnes
Average Annual Pollutant Retention	~70%	68.2%
Flow Velocity	1m/s maximum	0.30m/s
Min overflow clearance	0.35m minimum	0.40m

5.8. Maintenance

As discussed previously, the chosen GPT design is generally low maintenance when compared to some of the off-the-shelf products. Hydro-Future Consulting has created three maintenance checklists for managing the gross pollutant trap throughout its life.

The first checklist to be used during the construction phase will ensure the works reflect the design drawings and specifications and can be seen in Appendix 2.1. The second checklist is to be used post construction to again ensure the GPT has been constructed to specifications and to ensure construction related equipment and devices are removed when required and no debris or rubbish is left behind. This checklist can be seen in Appendix 2.1. The final checklist is a maintenance checklist, which will be submitted to the Council for their use. When each inspection is due, the Council will select a number of contractors (depending on maintenance period) to undertake maintenance works as outlined on the form, as can be seen in Appendix 2.3.

5.9. Bill of Quantities

5. Bill of Quantity
Client: Tokin Consulting
Project: North Terrace Drainage System Upgrade
Department: Environmental

#	Item name	Catalogue reference or special specification (if needed)	Unit	Quantity	Rate	Cost ($)
			Subject: GPT			
1	35 degree bend DN750 pipe	Hynds Catalogue Item #: 401628	m	2	1500	3000
2	GPT- 32MPa concrete	Hason Heidelburg Cement Group	m^3	14	239.5	3339
3	Rubber ring	Hynds Catalogue Item #: RO750	No.	1	35	35
4	Labour	Includes truck/crane drivers off site concrete workers & on site workers for placement at average wage	Hourly	22	50	1100
5	50 tone truck mounted crane	A1 Lift Crane Hire	Hourly	2	250	500
6	Flat top truck	Rentco Transport Equipment Rentals	Daily	1	400	400
7	25mm no slip steel plate	Shore Sales	m^2	2	1000	2000
8	Wall mounted galvanized ladder	Hydnes Catalogue Item #: RO750	m	5.3	200	1060
9	Manhole liner and lid (600mm diameter)	Mr Manhole Australia	No.	1	500	500
			Sub-total (1)			10734
			Subject: Trash Rack			
1	Steel-flats 20mm*180mm	Southern First For Steel	m	3.64	150	546
2	Steel-RHS (100*50*0.3) mm	Southern First For Steel	m	3.64	35	127.4
3	Steel-flats 10mm*75mm	Southern First For Steel	m	12	32	384
4	Welding labour	Includes welding time, welding wire and gas	-	-	-	650
5	M16 Bolts	James Glen Steel Fasteners Item	No.	16	1	16
6	Installation	Placement of trash rack within GPT	Hourly	1	50	50
			Sub-total (2)			1773.4
			Subject: Other			
1	240L Spill Kits	Stratex Australia	ea	3	408	1224
2	Bunding Trays	Containit C4DBP-P 4x250L Drums	ea	2	515	1030
3	PVC Spill Tray	Containit 3m x 3m PVC Tray	ea	3	1600	4800
			Sub-total (3)			7054
			Total (Optional)			**19,561.40**

5.10. Safety

Task Specific Safety Assessment Form	
Department/Section:	Environmental
Task/ stage Name:	Gross Pollutant Trap Construction

Brief Description of works to be undertaken

Off-site

- Have gross pollutant trap constructed to design specifications (reinforced concrete shell only), fixtures to be added on site.
- Allow for adequate curing time before being transported to site and implemented for use
- Have overpass frame constructed to design specifications

On-site

- Removal of existing road surface
- Excavation of sub-surface soils to the designated depth of the GPT
- Apply trench stabilisation measures
- Create void in sandstone arch wall suitable for new 750mm RCP connection from GTP
- Apply reinforcement to void based on design specifications
- Compact and level new base soil for placement of the GPT
- Transport GPT to site
- Install fixtures to GPT (trash rack panel and ladder)
- Lower GPT into location using appropriate machinery
- Connect existing 750mm RCP into inlet side of GPT along with new 750mm RCP elbow
- Connect outlet RCP from side of GPT into newly created void in sandstone arch culvert wall
- Backfill and compact to geotechnical design specifications
- Resurface with appropriate soil and wearing course layer thicknesses to design specifications
- Transport overpass frame to site
- Drill post holes for overpass
- Concrete overpass frame posts into correct position using appropriate machinery
- Create road surface for overpass level with the maintenance cover of the GPT

Summary of major risks or hazards

- Moving of heavy loads
- Working in confined spaces
- Injuries resulting from machinery and/or tools
- Dust inhalation
- Exposure to chemicals and dangerous substances
- Loud noises
- Vibration from machinery and/or tools
- Tripping
- Slipping

Mitigation strategies

- *Moving of heavy loads* - use machinery as much as possible to reduce the risk of injury as a result of moving heavy loads. Where possible exercise team lifts and always follow manufacturer's recommendations on the number of individuals required to lift a specified object (if available)
- *Working in confined spaces* – all employees working in confined spaces on site (such as the trench for the GPT) are to have obtained a confined space permit and are to only enter confined spaces if the space is deemed structurally safe and is of a safe large enough to work appropriately and safely in.
- *Injuries resulting from machinery and/or tools* – all employees on-site are to follow occupational health and safety (OH&S) guidelines outlined by their key employer and

Hydro-Future Consulting. Personal protective equipment (PPE) is to be worn at all times on-site and will be strictly monitored by an OH&S representative who will perform random visits to the site. Proper use of PPE will ensure injuries resulting from machinery and/or tools is minimised and avoided as much as possible.

- *Dust Inhalation* – All employees on-site are to wear safety masks whenever they are in the presence of air-borne substances such as dust.
- *Exposure to chemicals and dangerous substances* – the use of PPE should minimise the chance of exposure to chemicals and dangerous substances. In the event an employee is exposed to chemicals and/or dangerous substances the appropriate procedures are to be actioned by those trained in first-aid such as using first-aid kit for small injuries, or other relevant items such as an eye cleaning kit or calling the ambulance and following their instructions while waiting for the vehicle to arrive for more serious cases.
- *Loud noises* – all employees using or within close proximity of machinery and power tools are to wear ear muffs (inclusive of standard PPE).
- *Vibration from machinery and/or tools* – employees are advised to take small breaks when using machinery and tools such as compactors or jack-hammers, which cause vibration to minimise any adverse effects to their health.
- *Tripping* – appropriate signage is to be used on-site where a sudden change in elevation exists or where any potential tripping hazards exist. This could include brightly-coloured flagging on heavy items placed on the ground. Constant cleaning and tidying up around the site is to be actioned by all employees.
- *Slipping* - appropriate signage is to be used on-site where any potential slipping hazards exist. In the case a substance is spilled, a spill-kit should be used to clean up the substance and remove any trace of it from the site, to reduce any slipping hazards. Constant cleaning and tidying up around the site is to be actioned by all employees.

Safety equipment required & number
Personal Protective Equipment (PPE): Steel-Toe Safety Boots Safety Glasses Safety gloves Safety masks Safety ear muffs

6. Geotechnical and Structural

6.1. Site Visit

On Friday 8th May 2015 the geotechnical and structural team from Hydro-Future conducted a thorough site visit of first creek from the concrete box culvert underneath Hackney Road, to its end point, the River Torrens. The site has been broken up into sections as seen in Figure 67.

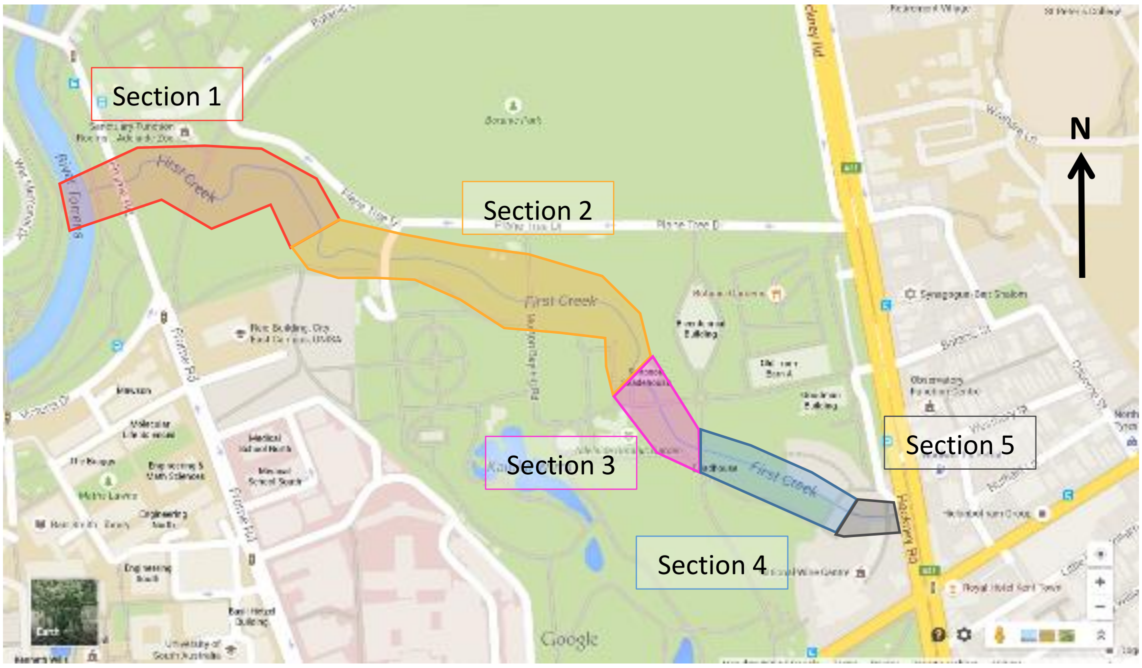


Figure 67: Map of First Creek from Hackney Road to River Torrens, colours highlight the 5 section of the creek as analysed by the team at Hydro-Future (Image Source: Google Maps).

The site visit was conducted between 7:45am and 9:45am, and during the site visit a rain shower occurred allowing the team from Hydro-Future to witness the flows within the creek. The total rainfall for the day was 4.4 mm (BOM, 2015) and this equates to an ARI estimate of less than 1 year event.

All the pavements within the site area have been deemed to be in excellent condition and adequate pedestrian access is available along this section of the creek, with numerous, well maintained pedestrian bridges.

Figure 68: Pavements used by both bicycles and pedestrians within the area are in excellent condition (Hydro-Future 2015)

Figure 69: Example of the many pedestrian bridges that cross first creek (Hydro-Future 2015)

6.1.1. Section 1

The terminal end of first creek enters River Torrens via a culvert underneath Frome Road (Figure 70), prior to this the creek is approximately triangular in shape, with a rock and soil lining, vegetated slopes with numerous trees. Due to the large slopes, wide channels and vegetation the creek's capacity in this section is deemed satisfactory, as shown in Figure 71 and Figure 72.

Figure 70: First Creek as it enters the River Torrens (Hydro-Future 2015)

Figure 71: Typical Setting of First Creek within Section 1 (Hydro-Future 2015)

Figure 72: Typical Setting of First Creek within Section 1 (Hydro-Future 2015)

6.1.2. Section 2

The creek within Section 2 consisted of a concrete lined rectangular channel, which during the observed water flows was satisfactory in its carrying capacity. Numerous stormwater outlets enter First Creek here and water flowed quite rapidly. To assist in decreasing the flow rate a weir has been installed within this section of the creek.

Figure 73: Rectangular Concrete Lined Channel, photo taken before the rain shower (Hydro-Future 2015)

Figure 74: Rectangular Concrete Lined Channel, photo taken after the rain shower (note the rapid water flow) (Hydro-Future 2015)

Figure 75: Photo of the Weir, which is used to decrease the velocity of the water to assist in erosion control (Hydro-Future 2015)

6.1.3. Section 3

During the site visit it was evident that along this 96 metre stretch of the creek that there is substantial erosion of the creek sides, with undercutting of the lawn surface (Figure 76, Figure 77 and Figure 79). The typical cross-sectional profile of the creek is trapezoidal with a vertical wall of the northern side. The creek bed consisted of patchy rocks and soil.

Some Gabion baskets have been placed underneath stormwater outlets as a method of preventing further erosion (Figure 78 & Figure 79), however the foundations underneath these have eroded away. In an attempt to reinforce the soil, wire meshing has been placed, but this too isn't working (Figure 77). This has resulted in an unstable section of the creek, requiring remedial measures. A detailed retaining wall design has been carried out for this section of the creek and can be located in Section 6.2 of this report.

Figure 76: Eroded Creek bank (submerged) and concrete retaining wall (right of picture) typical of Section 3 (Hydro-Future 2015)

Figure 77: Eroded Creek bank (partially submerged) and concrete retaining wall (right of picture) typical of Section 3 (Hydro-Future 2015). Note steel meshing used to reinforce the soil against erosion (Hydro-Future 2015)

Figure 78: Start of Section 3, note the failed gabion basket in the picture's foreground (Hydro-Future 2015)

Figure 79: Eroded Creek bank, concrete retaining wall (right of picture) typical of Section 3. Note the two gabion baskets (Hydro-Future 2015)

6.1.4. Section 4

This section of the creek consisted of a trapezoidal, rock lined channel (Figure 80) that was constructed in 2013 (DEWNR 2014). This section of the creek has adequate capacity for creek flows, including 1 in 100 year flood events, as shown in Figure 81.

Figure 80: Typical Profile of the Creek in Section 4 (Hydro-Future 2015)

Figure 81: Flood level indicator, with red highlighting the 1 in 100 year flood event level. (Hydro-Future 2015)

6.1.5. Section 5

This section of the creek begins at the exit of the culvert underneath Hackney Road (Figure 82), which was the limit of the site investigation. The water flows through a rectangular, concrete lined channel (Figure 83) and goes through a gross pollutant trap (Figure 84), prior to entry into Section 4. The gross pollutant trap at the time of the site visit appeared to be very full and need of maintenance, as much of the litter simply pooling up behind it. This section of the creek showed adequate capacity for design flows.

Figure 82: Concrete Box Culvert beneath Hackney Road, the start of Section 5 (Hydro-future 2015)

Figure 83: Typical Creek Profile in Section 5 (Hydro-Future 2015)

Figure 84: Channel or Creek Net used in the transition from Section 5 to Section 4. (Hydro-Future 2015)

6.2. Gabion Retaining Wall Design

Gabions are simply rock-filled baskets and have been used for centuries (Global Synthetics nd). They have many uses, however for this project they will be used as earth retention structures and designed on the basis of gravity retaining walls. They will also be used for riverbank and riverbed protection from localised scour near the stormwater outlets. The total lineal length of the gabion retaining wall is 96 metres.

Calculations are presented in this section of the report, beginning with foundation design, followed by stormwater outlet protection design and finally retaining wall design.

6.2.1. Foundation Design

First creek is a dynamic energy system, where it is able to erode, and transport and deposit sediments. In section 3 of the creek it is evident that significant creek bank erosion has occurred, with many, seemingly ad-hoc methods being employed to counteract these forces.

Before analysing the retaining wall, we first analyse to the foundation of the structure, with the velocity of the water being a key consideration. For design, when calculating the flow within the creek;

$$V_{avg} = \frac{R^{2/3} {S_0}^{1/2}}{n} \qquad \textbf{Equation 1}$$

Where,

V_{avg} is the average water velocity (in m/s)
R, is the hydraulic radius of the creek (in m)
S_0 is the slope of the creek bed (m/m)
n, is Manning's Roughness Co-efficient

The hydraulic radius is found by Equation 2; where, A is the flow area and P is the wetted perimeter;

$$R = \frac{A}{P} \qquad \textbf{Equation 2}$$

The worst possible flow height has been assumed to occur at a depth of 1.0 metres, which has been assumed from the height of scour, with the typical cross section of the creek profile shown in Figure 85.

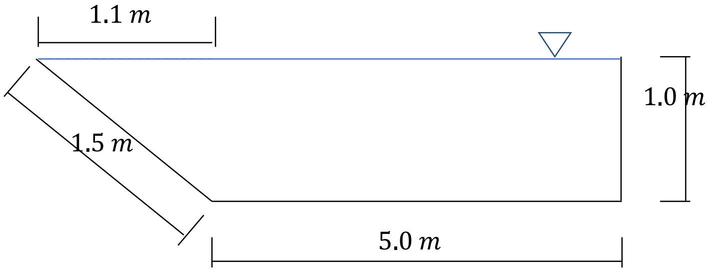


Figure 85: Typical Cross-Section of First Creek for design of Gabion Retaining Wall, as evident from the site visit

$$A = 0.5(1.1)(1.0) + 5.0(1.0) = 5.55\ m^2$$

$$P = 1.0 + 5.0 + 1.5 = 7.5m$$

$$\therefore R = \frac{5.55}{7.5} = 0.74$$

The bed slope has been measured as a drop of 2m over a length of 167 m;

$$\therefore S_0 = \frac{2}{167} = 1.2\%$$

The Manning's Roughness co-efficient for a natural stream, with a rock lining (Archement, Schneider, 1989);

$$n = 0.030$$

$$\therefore V_{avg} = \frac{0.74^{2/3} 0.012^{1/2}}{0.030} = 2.98\ m/s$$

The largest extent of the scour has occurred along the outside of the river bend and in the locations of the stormwater outlets.

To prevent undermining of the gabion retaining wall, the structure needs to incorporate either;

a) Having a foundation at a level of 0.5m below the expected depth of scour, or;
b) Incorporating a mattress apron

The depth of scour in a natural creek bed is difficult to predict (May et al 2002) and mattress aprons are utilised by many engineers to minimise erosion of the foundation, as well as minimise depth of excavation of the creek bed (Global Synthetics nd).

It has been conservatively estimated that the depth of scour of the rock-lined, clay substrate creek bed would be around 0.5 metres. The length of the apron is equal to double the depth of scour, which is 1 metre. The mattress apron rests on the creek bed, as shown in Figure 86

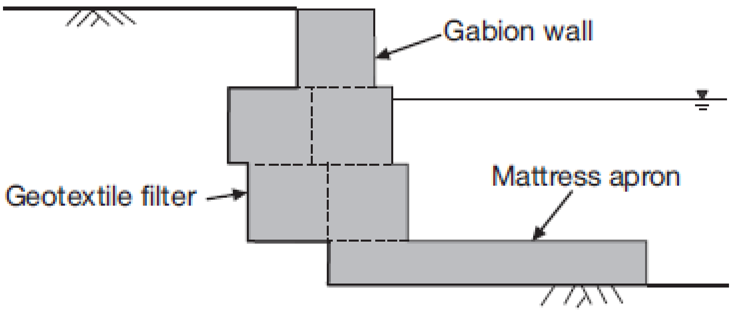


Figure 86: Typical location of mattress apron (Global Synthetic nd)

Based on the critical velocity of the stream (3 m/s), the mattress apron needs to be a minimum of 170 mm thick, with a rockfill size of between 75 and 100 mm (Geofabrics 2013).

A geotextile filter is used underneath the foundation and behind the gabion as to ensure that soil cannot be eroded through the rockfill in the gabions and mattresses, thus protecting the integrity of the structure (Global Synthetics nd)

6.2.2. Stormwater outlets

It was noted during the site visit that extensive scour has occurred in the vicinity of the three stormwater outlets. The detailed design considers the use of a mattress apron beneath the outlet, as well as gabion wingwalls, in a similar arrangement shown in Figure 87

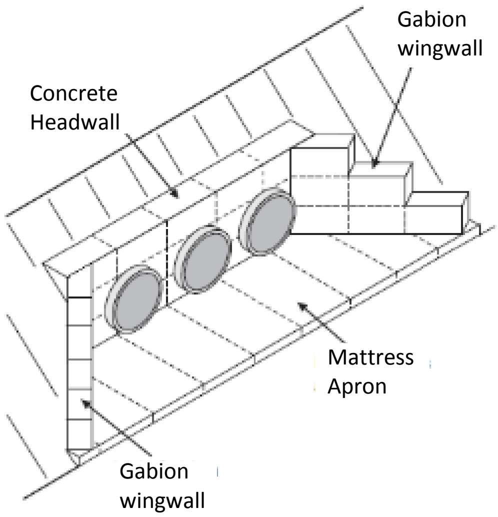


Figure 87: Typical Use of Gabions for Stormwater outlets (Adapted from p 26 Global Synthetic nd)

The diameter of the outlet of the stormwater pipes was 450mm, and the discharge occurs above the water level, as such the water flows are analysed for open channel flow, using Equation 1 and Equation 2. The depth of flow has been assumed to be 225mm, based on the site visit.

$$A = 0.5(\pi)(0.225)^2 = 0.0795\ m^2$$
$$P = \pi(0.225) = 0.707m$$
$$\therefore R = \frac{0.0795}{0.707} = 0.112\ m$$

The pipe slope has been assumed to be;

$$S_0 = 1\%$$

With manning's roughness co-efficient of concrete, $n = 0.013$ (Archement, Schneider, 1989);

$$\therefore V_{avg} = \frac{0.112^{2/3}0.01^{1/2}}{0.013} = 1.8\ m/s$$

Thus the mattress apron needs to be a minimum of 170 mm thick, with a rockfill size of between 75 and 100 mm (Geofabrics 2013). The length of the apron is based on the findings of Vallentine et al (1961) is 3.0D, where D is the pipe diameter. Thus the length of the apron needs to be a minimum of 1.35 metres. It is important that the apron be anchored to the foundation of the stormwater outlet as seen in CAD drawing HF-402.

The wingwalls are analysed on the same basis as the gabion retaining wall.

6.2.3. Retaining Wall Design

The gabion walls have been designed as a gravity retaining wall and are checked against the three limit states of;

- Overturning;
- Sliding, and;
- Bearing Capacity

The design considers the use of sandstone for the rockfill, with a unit weight (γ_s) of 23 kN/m^3. The weight (W) per metre run is calculated by;

$$W = (1 - \upsilon)\gamma_s A \qquad \textbf{Equation 3}$$

Where,

υ, is the void ratio of the rock infill (approximately 0.35)

A, is the cross sectional area of the wall

The soil profile and geotechnical model behind the retaining wall was determined during the Feasibility Study and is presented in Table 25. The design considers the long-term drained analysis of the soil profile, or the c' and Φ' values

Table 25: Geotechnical Model for Earth Pressure Calculations (Hydro-Future 2015)

Depth	Soil	Unit Weight γ (kN/m^3)	Undrained Shear Strength C_u (kPa)	Shear Strength C' (kPa)	Internal Frictional Angle ϕ'
0.0 – 0.45	Silty Sandy CLAY	18.5	85	0 – 10	30
0.45 – 2.0	Silty CLAY	20.5	150	0 – 5	20

6.2.3.1. Vertical Earth Pressures

The vertical stress (σ_z) acting at a depth, h, below the top of the wall is equal to;

$$\sigma_z = \gamma h \qquad \textbf{Equation 4}$$

For the first 0.45m;

$$\sigma_z = 18.5(0.45) = 8.325\ kPa$$

For 0.45 to 1.0m;

$$\sigma_z = 8.325 + 20.5(0.55) = 19.6\ kPa$$

To represent this graphically (Figure 88)

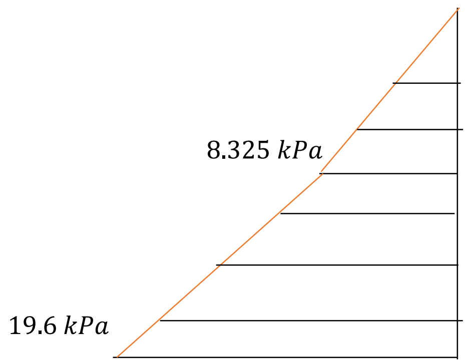


Figure 88: Vertical Earth Pressures using for calculating lateral earth pressures acting on the Gabion Retaining Wall

6.2.3.2. Lateral Earth Pressures

Rankine's theory has been used to determine the lateral earth pressures, with the lateral active pressure coefficient, K_a (Smith 2006);

$$K_a = \frac{1 - \sin\varphi}{1 + \sin\varphi} \qquad \textbf{Equation 5}$$

For the upper 0.45m;

$$K_a = \frac{1 - \sin 30}{1 + \sin 30} = 0.33$$

And for 0.45 to 1.0m;

$$K_a = \frac{1 - \sin 20}{1 + \sin 20} = 0.49$$

The soil exhibits both cohesion and cohesion less properties, as such the active pressure (P_a) (Smith 2006);

$$P_a = K_a \sigma_z - 2c'\sqrt{K_a} \qquad \textbf{Equation 6}$$

For the upper 0.45m;

$$P_a = (0.33)(8.325) - 2(5)\sqrt{0.33} = -3.0\ kPa$$

Negative pressure indicates that this zone of the soil is in suction, however in scenarios such as this it is unwise to assume that any negative pressures exist, due to tension cracking. For design purposes the active pressure is taken as (Smith 2006);

$$P_a = (0.33)(8.325) = 2.75\ kPa$$

At the junction of 0.45m in the silty clay profile;

$$P_a = (0.49)(8.325) - 2(2.5)\sqrt{0.49} = 0.60\ kPa$$

At the base (1.0);

$$P_a = (0.49)(19.6) - 2(2.5)\sqrt{0.49} = 6.10\ kPa$$

Representing this graphically (Figure 89);

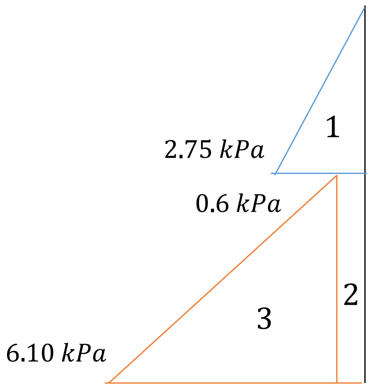


Figure 89: Lateral Earth Pressures acting on the Gabion Retaining Wall, the numbers refer to Table 26 pressure calculations

6.2.3.3. Limit State Checks – Overturning and Sliding

The Gabion Retaining wall has been designed and its dimensions are shown in Figure 90. The construction drawings highlight the exact placement of this structure and come in a variety of lengths, however the cross section remains the same throughout its entire length. Table 26 has the set out of all calculations that have been used for the limit state checks.

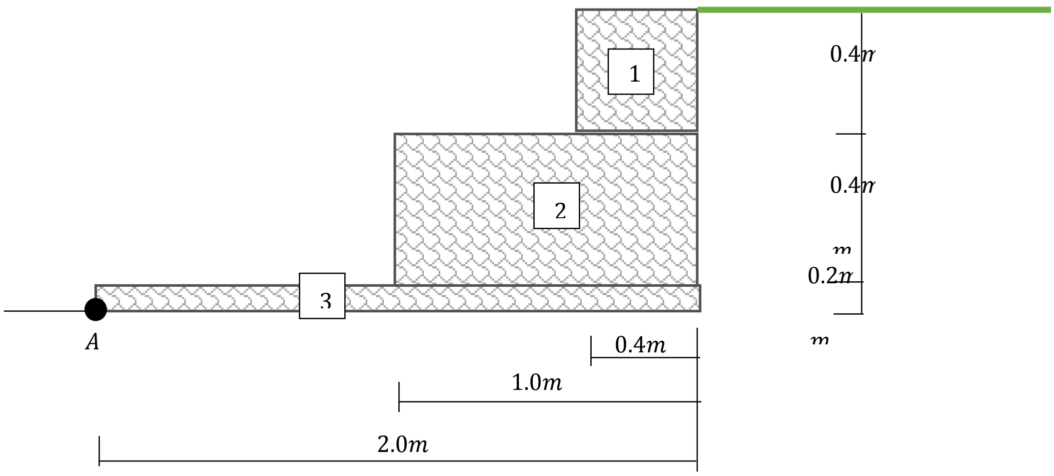


Figure 90: Dimensions for Gabion Structure, allowing the use of limit state design principles

Table 26: Force and Moment Calculations for the Gabion Structure shown in Figure 90

Sections	Force (kN)	Arm (m)	Moment (kNm)
$\boldsymbol{Pa1}$	$0.5(2.75)(0.45) = 0.62$	$0.55 + \frac{1}{3}(0.45) = 0.7$	0.43
$\boldsymbol{Pa2}$	$0.6(0.55) = 0.33$	$0.5(0.55) = 0.275$	0.09
$\boldsymbol{Pa3}$	$0.5(5.5)(0.55) = 1.51$	$\frac{1}{3}(0.55) = 0.18$	0.27
$\boldsymbol{Total}$	$H = 2.5$		$M_o = 0.8$
1	$0.4(0.4)(1 - 0.35)(23) = 2.4$	$0.6 + .5(0.4) = 0.8$	1.9
2	$0.4(1.0)(1 - 0.35)(23) = 5.98$	$0.2 + .5(0.4) = 0.4$	2.4
3	$0.2(2.0)(1 - 0.35)(23) = 5.98$	$0.5(0.2) = 0.1$	0.6
$\boldsymbol{Total}$	$V = 14.4$		$M_R = 4.9$

The factor of safety against overturning;

$$FoS = \frac{M_R}{M_o} \quad \textbf{Equation 7}$$

$$\therefore FoS = \frac{4.9}{0.8} = 6.1\ (\boldsymbol{OK})$$

The friction between the base of the wall and the soil is equal to $\tan\varphi$, or 0.36. The stabilising force at the base, T;

$$T = V\tan\varphi \qquad \textbf{Equation 8}$$

$$\therefore T = 14.4(0.36) = 5.2\ kN$$

The factor of safety against sliding;

$$FoS = \frac{T}{H} \qquad \textbf{Equation 9}$$

$$\therefore FoS = \frac{5.2}{2.5} = 2.1\ (\boldsymbol{OK})$$

The Factor of Safety for Overturning and Sliding needs to be greater than 1.5, which has been shown above.

6.2.3.4. Limit State Check – Bearing Capacity

The ultimate bearing capacity of the foundation soil, a silty clay with an undrained shear strength (Cu) of 150 kPa, is given by (Craig 2004);

$$q_u = (2+\pi)C_u \qquad \textbf{Equation 10}$$

$$\therefore q_u = (2+\pi)(150) = 771\ kPa$$

A factor of safety is applied to the ultimate bearing capacity to give the maximum allowable bearing capacity. The factor of safety is not less than 3 (Smith 2005);

$$\therefore q_{allowable} = \frac{771}{3} = 257\ kPa$$

The base pressure must not exceed the allowable bearing capacity of the supporting soil (257 kPa). To ensure that the base pressure remains constant over its entire base width, the resultant force must act within the middle third of its base. If a linear distribution of pressure is assumed the maximum and minimum base pressures are calculated by (Craig 2004);

$$p = \frac{V}{B}(1 \pm \frac{6e}{B}) \qquad \textbf{Equation 11}$$

Considering moments about point A (Figure 90), the lever arm of the resultant is given by;

$$\frac{\sum M}{V} \qquad \textbf{Equation 12}$$

$$\frac{\sum M}{V} = \frac{(4.9 - 0.8)}{14.4} = 0.28m$$

The eccentricity of the base reaction, e;

$$e = \frac{2}{2} - 0.28 = 0.72\ m$$

The maximum pressure as given by Equation 11;

$$p = \frac{14.4}{2}\left(1 + \frac{6(0.72)}{2}\right) = 22.8\ kPa\ (\boldsymbol{OK})$$

Therefore as it is much less than the allowable bearing capacity it is ok.

6.2.4. Technical Specifications

Rock filled gabions and mattresses are very flexible and require minimal foundation treatment prior to use. They are simple to erect, generally requiring unskilled labour. The gabions are folded flat and are assembled on-site (Global Synthetic nd).

Gabions and mattress are manufactured using woven wire mesh, in a double twist hexagonal pattern. This enables the mesh to maintain its structural integrity under extreme conditions and is not susceptible to unravelling if any individual wires are compromised (Global Synthetic nd).

Gabions and mattress are differentiated according to their geometric shape, with mattresses being considered as "thin" gabions.

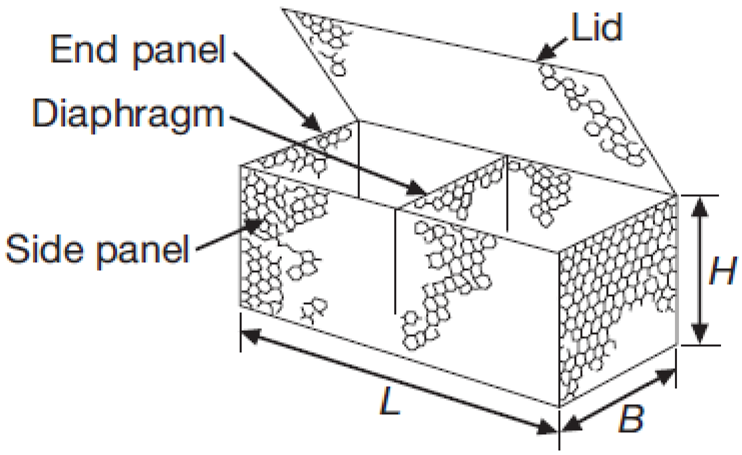


Figure 91: Gabion general configuration (p.4 Global synthetic nd)

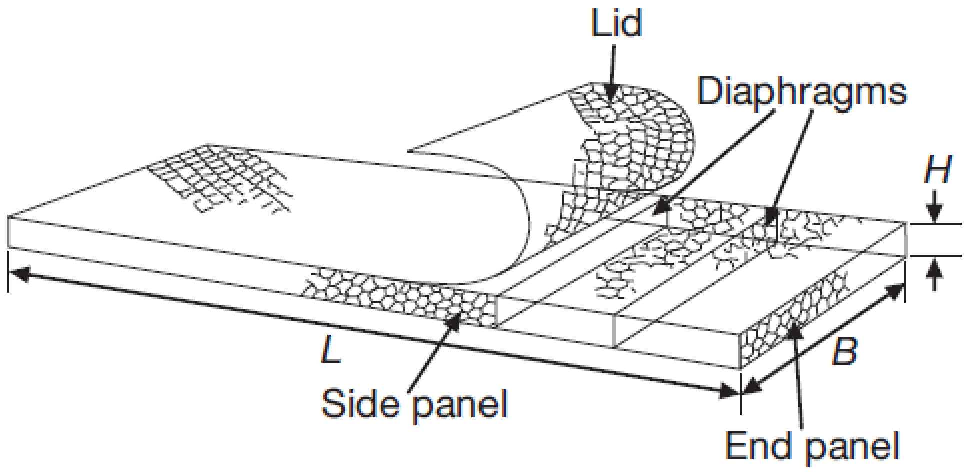


Figure 92: Mattress general configuration (p4. Global Synthetic nd)

6.2.4.1. General Description

The gabions shall be flexible woven, and zinc coated. Gabion sizes are stated in the Bill of Quantities and drawings. Each gabion shall be divided by diaphragms into cells whose length shall not be greater than width of the gabion plus 100mm. (Global Synthetic nd)

6.2.4.2. Zinc Wire

All wire used in the fabrication of the Gabions and in the wiring operations on-site shall be zinc coated in accordance with EN 10244-2:2001. The minimum mass of the zinc coating shall be in accordance with the values listed in Table 27.

Table 27: Minimum mass of Zinc Coating (p5 Global Synthetic nd)

Wire Diameter (mm)	Mass of Zinc Coating (g/m^2)
3.4	265
3.0	255
2.7	245
2.4	230
2.2	230
2.0	215

6.2.4.3. Mesh body wire, lacing wire and selvedge wire

Mesh body wire

The mesh shall be of double twist, hexagonal wove, body wire where the joins are formed by twisting each pair of wire through full turns to make a product with a minimum of three twists at each join. The mesh shall be manufactured to the requirements of EN 102223-3: 1998. Tolerances on mesh openings shall be +16%, -4%. Tolerances shall only apply to mesh openings when measuring distance centre to centre between parallel twists of mesh (Global Synthetics nd)

Gabions have a nominal mesh opening of 80mm and mattresses 60mm. (Global Synthetics nd)

Selvedge Wire

All cut edges of the gabions shall be selvedged with a thicker wire that will minimise ravelling of such ends. All wire sizes are shown in Table 28. (Global Synthetics nd)

Table 28: Gabion and Mattress Body and Selvedge Wire

Body Wire		Selvedge Wire	
Unit	Diameter (mm)	Unit	Diameter (mm)
Gabion	2.7	Gabion	3.4
Mattress	2.0	Mattress	2.4

Lacing Wire

Lacing wire is used to perform the necessary wiring and connecting operations that are associated with these products. Lacing wire is supplied at a rate of 5% of the weight of units supplied. The diameter of the lacing wire shall be 2.2mm for all lacing operations associated with the assembly and construction of the gabion and mattress structures. (Global Synthetics nd)

Pneumatic lacing tools and clips may be used in lieu of lacing wire (Global Synthetics nd), with advice from Hydro-Future sought on correct procedures when using this technique.

Tolerances

Tolerances for wire used in the manufacture of these products shall comply with the requirements of EN 10223:1998. Mesh openings shall be subjected to tolerances as detailed in EN10223:1998. (Global Synthetics nd)

Gabion width, length and height shall be subject to a tolerance of $\pm$5% from the gabion size prior to filling. Mattress units shall be subject to a tolerance of $\pm$5% for the width and length and a tolerance of $\pm$10% for the height prior to filling. (Global Synthetics nd).

6.2.4.4. Sizes of Gabions and Mattresses

Table 29 and Table 30 detail the sizes of the gabions and mattresses.

Table 29: Dimensions of the Gabions

Type	Unit Dimensions (m) Length L	Breadth B	Height H	Mesh Size (mm)	Diaphragms
A	2.0	1.0	0.4	80x100	1
B	1.0	0.4	0.4	80x100	4

Table 30: Dimensions of the mattresses

Type	Unit Dimensions (m) Length L	Breadth B	Height H	Mesh Size (mm)	Diaphragms
1	2.0	2.0	0.20	60x80	2
2	5.2	4.5	0.20	60x80	6
3	4.5	2.7	0.20	60x80	4

6.2.4.5. Geotextile Filter Specification

The geotextile must have specific hydraulic properties that enable it to drain groundwater but prevent soil erosion. It must also be robust to ensure that it is not easily punctured and damaged during handling and placement of the gabion and mattress structures. A generic specification for an appropriate geotextile filter is as follows;

It shall be needle-punched non-woven construction consisting of either polypropylene or polyester fibres and shall exhibit the properties shown in Table 31, with a suitable one being ProFab® AS801 (Global Synthetic nd).

Table 31: Geotextile Properties (p.7. Global Synthetic nd)

Property	Value
CBR puncture resistance (AS 3706.4 : 2000)	Minimum 3,000N
Trapezoidal tear resistance (AS 3706.3 : 2000)	Minimum 450 N
Drop cone h_{50} (AS 3706.5 : 2000)	Minimum 2,500 mm
G Rating (QMRD)	Minimum 2,500
EOS (AS 3706.7 : 1990)	Maximum 0.20 mm
Water Permeability Q_{100} (AS 3706.9 : 1990)	Minimum 50 L/m^2.s
Mass per unit area (AS 3706.1 : 2003)	Minimum 270 g/m^2

6.2.4.6. Rock Fill Specification

The rock used to fill the gabions and mattress shall be dense, hard and durable. It shall be free from fractures or cleavages. It shall be quarried sandstone. The dimensions of the stone shall be cubic, such that the long axis is not more than three times the shortest axis dimension (Global Synthetic nd).

The nominal size of rock used as gabion in-fill shall be 100 to 250 mm with not more than 5% finer by mass than 75 mm. The nominal size of rock used as mattress in-fill shall be 75mm to a maximum 125 mm with no more than 5% by mass finer than 75 mm (Global Synthetic nd).

6.2.5. Construction Guidelines

6.2.5.1. Labour and Plant Requirements

For this project it has been estimated that the gabions will be constructed using a plant operator and 2 labourers. The plant requirement, is best limited to small-medium sizes for ease of manoeuvrability. Using experienced personnel it is envisaged that for prefabrication, filling and closing an allowance of 2 man hours per m3 for the gabions and 0.5 man hours per m3 for the mattresses. (Global Synthetics nd)

6.2.5.2. Assembly

The gabion baskets are supplied from the manufacturer with an integral lid, whilst the mattresses are supplied with a base unit and separate lid, as shown in Figure 93.

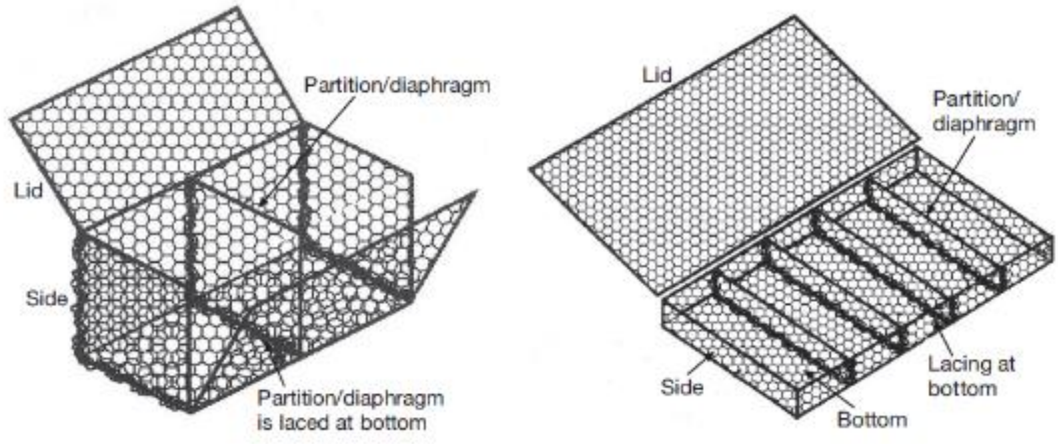


Figure 93: (left) Gabion Components, (right) mattress components (p. 32 Global Synthetics)

The general construction process is;

- The units should be unfolded on level ground and creases removed
- Corners initially joined so that the box remains square
- Join all sides together and each partition within the cage to the side of the cage.
- The partitions or diaphragms are factory connected to the base of the cage at the appropriate position

(Global Synthetics nd)

Joining Connection techniques

Where a connection is to be made, take a pie of tie wire and lace it in and out of the mesh openings at approximately 150mm centres in a continuous manner, alternating between a single loop and double loop around the selvedge wire (See Figure 94)

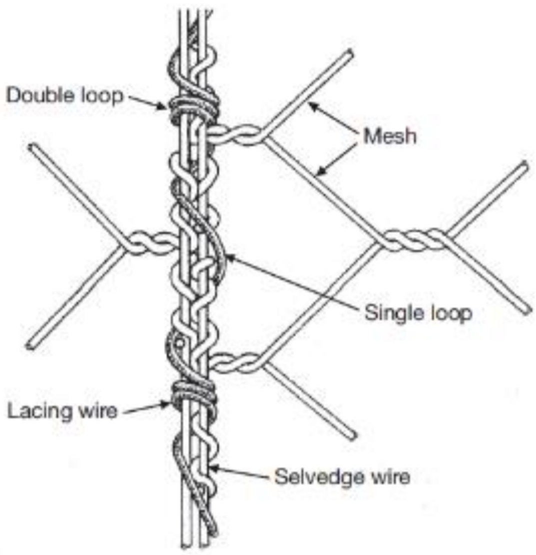


Figure 94: Gabion and Mattress lacing technique (p.33, Global Synthetics nd)

Placement and Filling

As the creek is subject to flows, placement of gabions and mattresses should not occur during rainfall events, and construction timed as to minimise the chances of rainfall events.

Prepare the site where the gabions are to be installed. Any rocks lining the creek bed should be stockpiled for later use. The ground should be relatively smooth and flat. The geotextile filter is installed first under the mattresses and behind the gabions. The fabricated cages are placed in their desired position, as shown on the construction drawings. Adjacent cages are to be connected together in the same manner as shown in Figure 94, to form one large structure (Global Synthetics nd).

Once in position the units should be stretched taught to provide a neat, straight alignment. The lid of the units remain open during the filling process and should be folded back as to allow easier filling. The rock should be place by machine to half height of the gabion, the rock is then placed by hand to position the rock against front face of the cage to ensure a neat, tight finish. Once the rock at this layer is neatly faced, two bracing wires (Figure 95) should be installed from the back face of the cage to the front face to prevent localised bulging. The internal bracing is set at the mid height of the cage (Global Synthetics nd).

All gabions should be overfilled to allow for settlement with time. The top of the gabion should be relatively level and flat to ensure a sound base is available for stacking of additional units. Lastly the lid is pulled into position on the top of the rock and laced closed to the perimeter of the gabion cage (Global Synthetics nd).

For the mattresses the rock is placed by machine, till approximately 10% overfilled, the rock may require manual labour to neatly pack the rock into the cage and level the top surface (Global Synthetics nd).

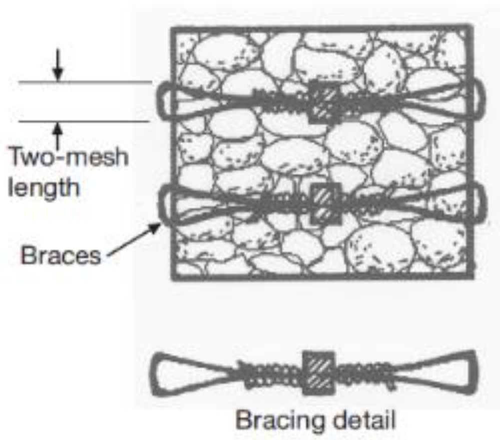


Figure 95: Gabion filling and bracing placement (p.34 Global Synthetics nd)

Costing considerations are detailed within the bill of quantities section of this report and detailed construction drawings have also been produced for the gabion retaining wall.

6.3. Pipe Design

In the design or selection of a concrete pipe an assessment shall be made on the following types of vertical loads;

- Working Load due to fill material
- Working load due to superimposed live loads

To assist with the nomenclature associated with pipe design, bedding configurations, Figure 96 from the standards is presented below

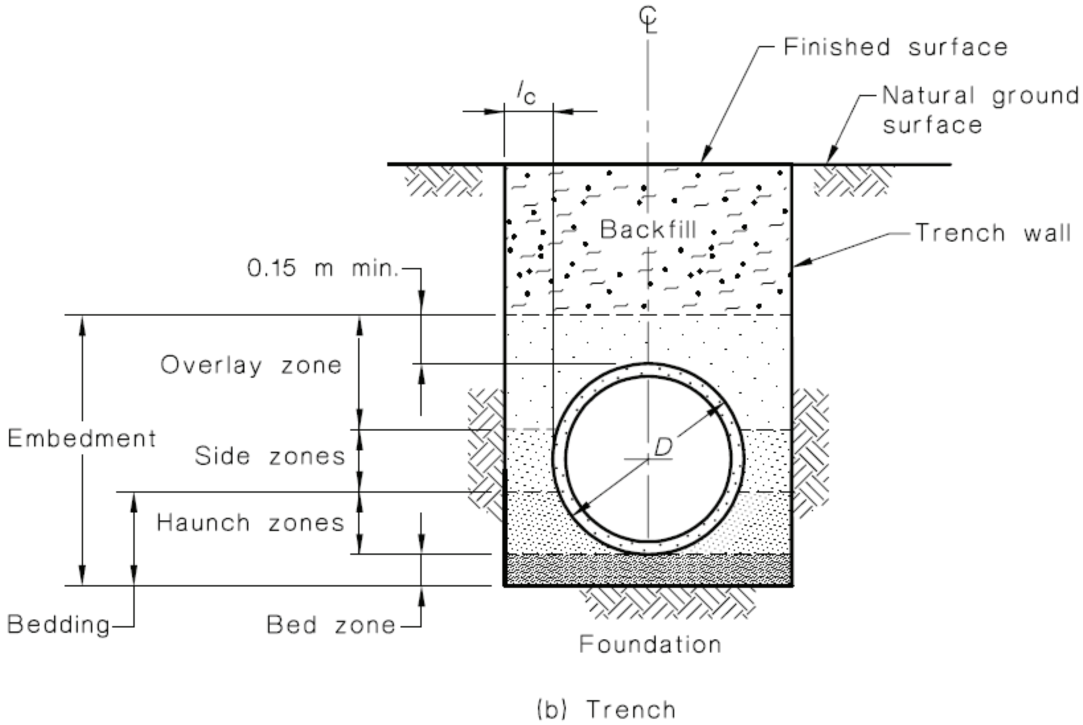


Figure 96: Fill and Pipe Support Terms for Trenching (p. 10, Standards Australia 2007)

6.3.1. Working Load due to fill material

The pipe is placed in a narrow trench that has been dug into the natural undisturbed ground. In this scenario, the load acting on the pipe is considered to be a function of the weight of the fill material in the trench above the pipe. The fill in the trench has a tendency to settle relative to the walls of the trench. This settlement results in frictional forces between the fill and the walls of the trench, which tends to decrease the resultant load of earth acting on the pipe. (CPAA, 2015).

The resultant earth load acting on the pipe in this condition, incorporating the effects of friction developed at the walls of the trench, is calculated in accordance with Clause 6.3 of AS 3725:2007, equation 1;

$$W_g = C_t w B^2 \qquad \textbf{Equation 13}$$

Where;

W_g is the working load due to fill in kN/m

w is the unit weight of the fill material in kN/m^3

B is the trench width in metres.

C_t is the load coefficient proposed by Spangler for the trench condition (generally referred to as the Spangler coefficient for the trench condition) which is calculated by;

$$C_t = \frac{1 - e^{-2Ku'(H/D)}}{2Ku'} \qquad \textbf{Equation 14}$$

Where;

H is the height of fill above the pipe;

D is the external diameter of the pipe;

$$Ku' = (\tan \Phi)\left(\frac{1 - \sin \Phi}{1 + \sin \Phi}\right)$$

Where Φ is the angle of internal friction of the fill soil, which in our case according to DPTI (2013) is a quarry sand type C (Sa – C), to a modified compaction level of 95%, in lifts of no larger than 200mm, which is assumed to have Φ = 33°, and a loose density of 16 kN/m^3.

$$\therefore Ku' = (\tan 33)\left(\frac{1 - \sin 33}{1 + \sin 33}\right) = 0.1914$$

The height of sand back fill, according to the DPTI (2013), must continue until the underside of the reinstated pavement, which according to the DPTI (2012) the reinstated pavement thickness is to be 525mm (North Terrace is Road Number 5221, refer Figure 294 for configuration).

The support provided to the buried concrete pipe acts as both the foundation but also can significantly increase the load carrying capacity of the pipeline. The granular bedding layers limit the load effects (bending moments and shear forces) acting on the wall of the concrete pipe for a given application of external load. Associated with each support type is a numerical measure of this reduction or increase in load carrying capacity of a given pipe installation which is known as the bedding factor.

The bedding factor is a ratio of the bending moment in the wall of the pipe which has been developed during a factory three edge bearing test and the bending moment which will result in the field installation for a given value of external load applied. For our pipe installation, a HS2 type support is required, and has a bedding factor of 2.5 (see technical specifications for more details). This is a haunch and side support, where compacted granular fill is placed in these zones, to varying levels of compaction, and is the recommended option for underneath roadways (CSAA 2015).

6.3.2. Live Loads

Live loads that will apply over the life of the stormwater pipe include loads from regular vehicle traffic. Live loading also needs to consider construction loads for compaction equipment, and other construction equipment such as dump trucks, rollers, diggers, etc. which may be applied to the pipe during installation. Such loads can and are intended to be applied at fill heights lower than the height of fill, H, and are not intended to be permanent design loads.

The SM1600 model traffic design load has been considered for the traffic loading on the pipe (Standards Australia 2004) and includes;

- W80 load - 80 kN load distributed over the tyre contact area (0.5 x 0.2 m)
- A160 load - an individual heavy axle load
- M1600 load - moving traffic load
- S1600 load – stationary traffic load

The critical live load is the M1600 traffic load case, however all the live loads applicable to the structure have been analysed. An example hand calculation for the stormwater pipe at Chainage 20 is presented in Appendix 3.4 to determine the pipe class size as per AS 3725. To allow for the time consuming process with individual hand calculations for each depth of trenching, the computer software, Pipe Class v2.0 has been utilised (CPAA 0215). The critical values from this analysis for the desired chainages is present in

Table 32. The trench width for the 750mm pipe is 1300mm and for the 600mm pipe it is 1100mm. The external diameter of the 750mm pipe is 870mm (Ch. 0 to 245m) and for the 600mm pipe it is 699mm (Ch. 245 to 720m).

Table 32 presents the data relating to the bedding support layers, as depicted in Figure 97.

Table 32: Critical Values utilised in the PipeClass V2.0 (CPAA 2015) software.

Chainage (m)	Depth to underside of pavement (H) mm	W_g (kN/m)	W_q (kN/m)	T_C (kN/m)	Pipe Class
0	2025	39.1	8.1	21	2
20	1725	35.9	8.8	20.2	2
50	1225	30.4	11.0	19.5	2
150/245	825	25.4	13.7	19.3	2
Pipe Size Change to 600mm					
245	825	24.1	12.7	18.1	2
305	675	19.1	12.8	16.2	2
360	455	16.6	14.9	16.6	2
395	1095	23.4	9.9	15.9	2
475	1375	25.9	8.5	16.0	2
555/625/720	675	19.1	12.8	16.2	2

Thus for the 750mm pipe adopt;

750 dia Class 2 Rubber Ring Jointed Reinforced Concrete Pipe (750/2 RRJ) in accordance with AS/NZS 4058: 2007

Thus for the 600mm pipe adopt;

600 dia Class 2 Rubber Ring Jointed Reinforced Concrete Pipe (600/2 RRJ) in accordance with AS/NZS 4058: 2007

6.3.3. Pipe Installation Specifications

Figure 97 highlights the dimension nomenclature for the HS2 support system for both the 600 and 750mm pipe, with the values required for each presented in Table 33. Section 0 provides the necessary technical specifications for the installation of HS2 support systems, as per CPAA (2015)

Table 33: HS2 design Support System values

Pipe Size	Bed Zone (X) mm	Haunch Zone (Y) mm	Side Zone (V) mm	Overlay Zone (O) mm
600	100	210	140	150
750	100	265	170	150

The Pavement Reinstatement depth is constant throughout at 525 mm (See Figure 294), however the dimension that changes is the depth of backfill, or H, for all the desired chainages this depth is highlighted in Table 34, with a maximum lift depth of 200mm.

Table 34: Depth of Backfill of Sa - C type Sand, at 95% modified compaction level

Chainage (m)	Depth of Back-fill (H)	Number of Lifts
0	1905	10
20	1575	8
50	1075	6
150/245	675	4
Change to 600 mm pipe		
245	675	4
305	525	3
360	305	2
395	945	5
475	1225	7
555/625/720	525	3

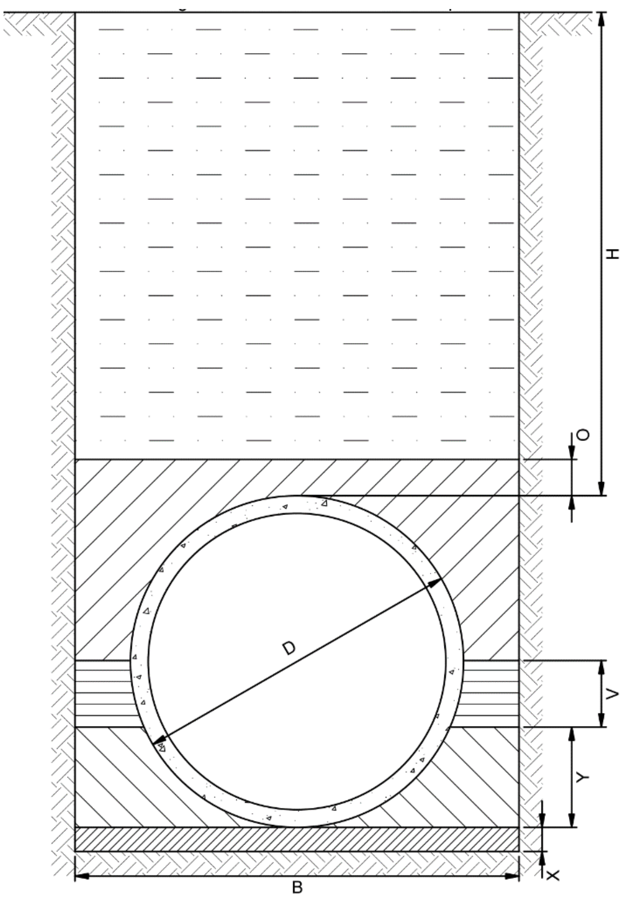


Figure 97: Schematic diagram highlighting the values of the HS2 support system

The type of material that is to be used in the bedding, haunch and side zones is presented in Table 35, as per Cl 9.2.3 of AS 3725 (2007). All bed, haunch and side zone material passing the 0.075mm sieve is to be of low plasticity (AS 1726).

Table 35: Material Grading Requirements for pipe support

	% Passing							
Sieve Size (mm)	75.0	19.0	9.5	2.36	0.60	0.30	0.15	0.075
Bed and Haunch Zones	100	100	100	100-50	90-20	60-10	25-0	10-0
Side Zones	100	100	100-50	100-50	50-15	50-15	50-15	25-0

6.3.4. Installation Specification for Type HS2 Support (as per CPAA 2015)

This specification is prepared to ensure the pipe installation conforms to the requirements of AS/NZS 3725:2007 Design for installation of buried concrete pipes.

Type HS2 supports represents an installation with controlled compaction in the bed zone and haunch zone.

Excavation and Bedding

Prior to completion of excavation, the soil type in which the trench is to be excavated shall be assessed for density and stiffness, to the satisfaction and approval of the superintendent.

If it is established that the natural ground will provide effective side support, the trench width for both trench condition an embankment condition shall be as shown on the drawings.

For an embankment installation, the positive projection of the pipe shall be 0.5 times the pipe outside diameter or less. Where the projection of the pipe above natural ground surface is greater than 0.5 times the pipe outside diameter, it will be necessary to construct the embankment to a height above top of bed level at least equal to 0.5 time pipe outside diameter, prior to laying the pipe, and to a width equal to at least 1.5 times pipe nominal diameter on each side of the proposed trench width.

For a trench installation, if in the opinion of the superintendent, the natural ground is not considered effective in providing the necessary side support, the trench shall be excavated to a width of 4.0 times pipe nominal diameter to the top of bed level.

The wide trench excavation shall then be refilled in accordance with the specification for refilling, side zone.

Refilling in embankment or wide trench shall be placed in layers not exceeding 150mm when compacted and, if cohesive material is used, the moisture content shall be controlled to within the range 85 percent to 115 percent of the optimum moisture content. Compaction by tamping, rolling and/or vibration shall be carried out to achieve a minimum Relative Density (RD) of 90% of standard maximum dry density, or a minimum Density Index (DI) of 60 for cohesionless material.

Density achieved shall be monitored by field testing as directed by the superintendent.

The required trench for the installation, to the width and depth shown on the drawings shall be excavated centrally through the above compacted select fill material.

Excavation shall be to line and level shown on the CAD drawings.

Should the excavation to the required foundation at the bottom of the bed level reveal material, which in the opinion of the superintendent is unsuitable, the trench shall be over-excavated to a depth required to remove the unsuitable material and refilled with compacted material conforming to the requirements for the bed zone.

Bedding

Bed zone material shall be select fill. Select fill as defined in AS/NZS 3725:2007 is material obtained from excavation of the pipe trench or elsewhere with a particle size not greater than 19 mm, and conforms to the following soil classes as defined in Appendix D of AS 1726. (Table 35)

The material passing the 0.075m sieve must have low plasticity as described in Appendix D of AS 1726.

Alternatively select fill as defined in AS/NZS 3725:2007 which does not conform to the above grading limits may be used provided that it is cement stabilized. Where controlled low strength materials are used they should comply with Appendix A of AS/NZS 3725:2007 to achieve 28 day compressive strength in the range of 0.6 to 3.0 MPa.

The bed material shall extend over the full width of the trench and shall be compacted by tamping, rolling and/or vibration to a minimum Density Index (DI) of 60.

Compaction achieved shall be monitored by field testing in accordance with AS 1289.

The bed level shall be graded to provide for a uniform fall to the discharging end of the pipeline, with line and level as shown on the drawings.

For pipes with sockets protruding beyond the barrel outside surface, chases shall be dug into the bed and foundation if necessary, in the appropriate positions, so that each pipe is supported along the full length of the barrel and the socket is not subjected to point loading.

Refilling

The refilling shall be carried out in four stages and these are to be identified as:

- Haunch zone
- Side zone
- Overlay zone
- Backfill or embankment fill

The **haunch zone** shall extend from the top of the bed zone to 0.3 times the pipe outside diameter and shall be fill material complying with the requirements shown above for the bed zone.

The material shall be placed over the full width of the trench either in layers not exceeding 150mm compacted thickness and compacted by conventional methods or compacted in one operation by saturation and vibration to achieve a minimum Density Index (DI) of 60.

Compaction achieved shall be monitored by field testing in accordance with AS 1289.

The **side zone** shall extend from the top of the haunch zone to 0.5 times pipe outside diameter and shall consist of select fill material complying with the following grading as shown in Table 35.

The material shall be placed over the full width of the trench in layers not exceeding 150mm compacted thickness and compacted by tamping, rolling or vibration to a minimum Relative Density (RD) of 90% or a minimum Density Index (DI) of 60.

Compaction achieved shall be monitored by field testing in accordance with AS 1289.

The **overlay zone** shall extend from the top of the side zone to 150mm above the top of the pipe and around the pipe measured radially from any point. The fill material in the overlay zone shall be quarry sand, type Sa – C as per DPTI requirements. It shall be compacted to achieve 95% MDD.

The Sa - C in the overlay zone should be placed and compacted in maximum 200mm layers and be a minimum of 125 mm.

Backfill or Embankment fill is to be the remainder of the refilling and should consist of any available material up to finished levels as shown on the drawings.

The material should be compacted as necessary to prevent excessive settlement in the ground surface level over the installed pipeline.

Refilling of sheeted trenches shall be carried out to the following requirements:

- No struts, walling or other supports shall be removed until the top of the compacted refilling has reached the level of these supports.
- No wall sheeting is to be totally removed from the trench until the level of the compacted refill is within 1500 mm of the surface.

- No wall sheeting is to be removed, in dewatered trenches, until the level in water table between natural ground and refill material is less than 500mm.
- The wall sheeting is to be withdrawn or removed in such a manner that the pipe and compacted bed and haunch support are not disturbed during such withdrawal or removal.

Important Notes:

- Ensure the bed zone is even and well graded to provide uniform support for the pipe.
- Do not compact directly over the pipe.
- Ensure the pipe is appropriately embedded and covered before allowing any construction equipment or plant over the top.
- Compact as you go and ensure that the appropriate levels of compaction are reached.

Test Procedures (DPTI 2013)

The Contractor shall use the following test procedures to verify conformance with the Specification: (refer http://www.dpti.sa.gov.au/contractor_documents)

TEST		PROCEDURE NO.
Sampling of Soil, Aggregates and Rocks		TP 226
Preparation of Samples		AS 1289.1
Site Selection by Stratified Random Technique		AS 1289.1.4.2
Field Density:	Nuclear Method	AS 1289.5.8.1
Moisture Content:	Oven Drying Method Microwave Method	AS 1289.2.1.1 AS 1289.2.1.4
Maximum Dry Density:	Modified Compaction	AS 1289.5.2.1
Dry Density Ratio		TP 320

6.3.5. Trench Excavation and Backfill Specifications (DPTI 2013)

General

This Part specifies the requirements for the excavation and backfill of trenches or similar excavations up to the level of the underside of the pavement (or natural surface outside of pavements).

For the purposes of this part, "**Trench**" includes any excavation constructed for the installation, maintenance or inspection of culverts, drainage structures, cables, conduits, pits and pipes (**"Services"**).

Removal of Existing Pavement

The pavement shall be saw cut or cold planed prior to excavation. Any additional breakage of the existing pavement edge shall be cut out square to the edge of the excavation prior to reinstatement.

All saw cutting shall be dampened by water to reduce dust and any resultant slurry shall be collected and disposed of in accordance with the requirements of the Environmental Protection Act, as outlined in the attached EMP. The slurry must not enter stormwater drainage systems or dry out on the road surface.

Removal of existing pavements shall comply with the following:

(a) Asphalt Surfaced Roads

The trench shall be cut or cold planed to the full depth of the existing asphalt surface. Where cement stabilised pavement exists, the pavement shall be cut to the full depth, or a minimum cut depth of 200 mm, whichever is the lesser. Note that saw cutting or cold planning in addition to that specified by this clause may be required to meet the requirements of Clause 208.4.2 "Asphalt".

Excavation

All excavation shall be of sufficient width to allow for safe and practical working, including the proper placing and subsequent removal of any formwork, shoring or dewatering systems and for the compaction of the backfill.

The depth of the Trench shall be sufficient to achieve the minimum cover of 1.0 m to the Service and the requirements of any applicable Service Authority.

Where excavation take place outside of existing pavement, any topsoil present shall be stripped and stockpiled to a depth of 100mm or other depth specified. Unless specified otherwise, any surplus excavated material not used elsewhere in the Works shall be removed from the site and disposed of by the Contractor in accordance with the requirements of the Environment Protection Act

Use of Steel Plates

If steel plates are used to enable traffic to cross an excavation, the Contractor must ensure that:

(a) the surface of the plate does not create a skidding hazard to motorists;

(b) a speed restriction of 60km/h or less is imposed on the section of road where a steel plate is situated;

(c) there is a smooth transition for traffic from the road surface onto the steel plate by the use of a temporary ramp;

(d) the steel plate is treated with an approved anti-skid compound in accordance with the manufacturer's instructions (the use of checker plate or plain steel alone is insufficient); and

(e) the steel plate is restrained, pinned or anchored to reduce impact noises caused by motorists.

The anti-skid compounds listed in the DPTI Approved Products List, available from: http://www.dpti.sa.gov.au/documents/contractsandtenders/specifications/general are approved for use on steel plates. The Contractor may submit a request for the approval of additional anti-skid products.

The anti-skid compound must be maintained in good order. The skid resistance must exceed 0.5 GN when tested in accordance with DPTI Test Procedure: TP344 "Determination of Skid Resistance with the GripTester", available from: http://www.dpti.sa.gov.au/standards/materials_technology_documents/test_procedures2

Backfill Material

Excavated material shall not be reused for backfill of Trenches below areas of pavement.

(a) <u>Services Installed in Trenches</u>

Backfill material shall be Sa-C Type C Sand in accordance with Part R15 or Controlled Low-Strength Material (CLSM) in accordance with Part R09.

Below pavement, the backfill material shall extend to the underside of the reinstated pavement.

In verges and roadsides (i.e. outside of the pavement area), the backfill material shall extend to a level at least 300 mm above the top of the Service after compaction. Unless specified otherwise, excavated material may be used above this level.

(b) Services Installed Within Fill Locations

Backfill material shall be Sa-C Type C Sand and shall be placed to a level at least 300 mm above the top of the Service after compaction.

Placement of CLSM Backfill

If CLSM is used, it shall be placed in accordance with Appendix K "Controlled Low Strength Materials—CLSM" of AS 2566.2: Buried flexible pipelines - Part 2: Installation.

Placement of Sand Backfill

Sa-C Type C Sand backfill shall be compacted alternately on each side of the Service. Backfill shall not be placed against any cast-in-place concrete within 48 hours of the placing of concrete. Flooding of sand with water is, by itself, not an acceptable method of compaction.

Backfill against Drainage Structures

Backfill placed against drainage structures shall:

(a) be free draining material in locations where it is necessary to prevent the build-up of hydrostatic pressures;

(b) develop sufficient strength to ensure it is stable and does not undergo post construction settlement;

(c) where backfill is to be placed on both sides of wing walls or retaining walls, the backfill shall be brought up level with a maximum height differential of 300 mm;

(d) not be placed against concrete which is less than 48 hours old; and

(e) not be placed against wing walls or retaining walls until all cast in place concrete has reached the 28 day characteristic compressive strength and is at least 14 days old.

Compaction of Backfill

Unless specified otherwise, the backfilling shall be uniformly compacted in horizontal layers not exceeding 200 mm (loose) thickness to the values shown in Table 4.2. Compaction shall be the Dry Density Ratio determined using AS 1289, test method 5.2.1 (modified compaction).

Compaction testing shall comply with Table 36 and Table 37

Table 36: Minimum Backfill Compaction (DPTI 2013)

	Compaction (% modified)		
	Below Sealed Pavement	Below Unsealed Pavement and Shoulder	**Outside of areas of Pavement**
Between 800 mm below finished surface and the underside of pavement	95	92	**90**
More than 800 mm below finished surface	**92**	**92**	**90**

Table 37: Minimum Compaction Test Frequency (DPTI 2015)

Small box culverts and stormwater pipes 1 m or less in diameter:	**1 test per 5 m^3 or part thereof**
Large box culverts and stormwater pipes over 1 m in diameter:	**1 test per 10 m^3 or part thereof**
All other Services:	**1 test per 10 m^3 or part thereof unless an approved compaction methodology has been implemented**

A minimum of 3 compaction tests shall be carried out.

6.4. Trench Stability

As outlined in Section 6.3, the depth of excavation ranges from 1.1 metres to 2.7 metres. The Work Health Safety Regulations (2012), Chapter 6, Division 3, state that any trench that is excavated by more than 1.5 metres deep, must ensure that the work area is secured from unauthorised access, and that to minimise the risk of collapse of the trench that all sides must be adequately supported by the following;

- Shoring by shielding or other comparable means;
- Benching;
- Battering

However, if suitable, written advice is given by a qualified geotechnical engineer that all sides of the trench are safe from collapse, then the supporting methods may not be required.

For the purposes of this project, it is assumed (as per Tim Kerby, Tonkin Consulting) that we are unable to gain the advice from a qualified geotechnical engineer, or apply geotechnical engineering principles of slope stability analysis to the trench stability problem. It has decided to use shoring, by shielding as a method of trenching support for excavations over 1.5 metres.

For the project area the following chainages have excavations in excess of 1.5 metres;

- Ch 0.0 to 138m
- Ch 383.0 to 533m

This next section of the report will provide information on the stability and safety of the trench excavation. It will include details of the soil conditions, trench support requirements and dewatering systems.

6.4.1. Soil Conditions

Before any trenching is to commence, a pre-construction survey and a review of the soil types needs to be undertaken. Many environmental and on-site activates can result in trench collapse, and are to be considered as a part of the pre-construction survey. Typical causes of trench collapse include;

- Failure to install shoring / ground support;
- Mechanical failure of soil (inability to support its own weight);
- Breakdown of soil strength due to moisture;
- Vibration from vehicle and/or plant movement;
- Surcharge of spoil or heavy weights close to trench edge;

- Change in soil composition (sand pockets etc);
- Previously disturbed ground (landfill, old trenches);
- Trench walls being struck by heavy loads;
- Undercutting; and
- Premature removal of shoring. (SA Water 2011)

6.4.1.1. Pre-Construction Survey

The project area incorporates two main soil types, RB5 and AL. The inherent variable nature of alluvium means that high degrees of variability are likely to be encountered, therefore during construction test holes, along the proposed alignment should be dug as to observe ground conditions, and to see if they differ from the assumptions made during this design.

It has been assumed that the soil conditions at the site up to a depth of 2.7 metres are;

- RB5
  - Silty Clay
  - medium to high plasticity
  - Moisture content less than the plastic limit
  - No water table
- AL
  - Sandy gravelly Clay (to 1.25 metres)
    - low plasticity,
    - MC less than the plastic limit
  - Silty Gravelly Sand (1.25 to 2.7 metres)
    - Fine to coarse
    - Medium Dense to Dense
    - Dry to Moist Condition
  - No Water Table

If however, during construction a water table is encountered, especially near the culvert, the consideration of a dewatering system is presented in Section 6.4.4.

All services have been located and are available on the construction drawings.

6.4.2. Trench Support Requirements

The type of material that is to be excavated has a large influence on the design of shoring/shielding. Loose or unstable ground requires closed sheeting systems, where hard, compact ground can utilise open sheeting systems (SA Water 2012). Due to the variable nature, and uncertainty around the alluvial material, a closed sheeting system has been designed. The type of system that has been chosen is a 'trench box' or metal shield.

Shields don't support the trench sides, but does ensure worker safety. When a trench wall collapses, it does so against the side of the shield, leaving the worker safe inside. They are particularly useful when long and large diameter pipes are to be installed, over varying ground conditions (SA Water 2012), such as this project.

They are designed and installed based on their ability to remain intact when impacted/pressurised by a fall of soil. Due to their versatility they are also available for hire from many companies, at a relatively low cost. For ease of construction, the shield has been designed as a drag box (SA Water 2012).

6.4.3. Trench Box Design

A trench box is assembled using pre-fabricated parts, incorporating a system of panels and struts, and generally comprises lower and upper modules (Figure 98). A drag box, is an extension of a trench box, but has the benefit of being dragged horizontally by its leading edge during the excavation process (Figure 99) (Standards Australia 2000).

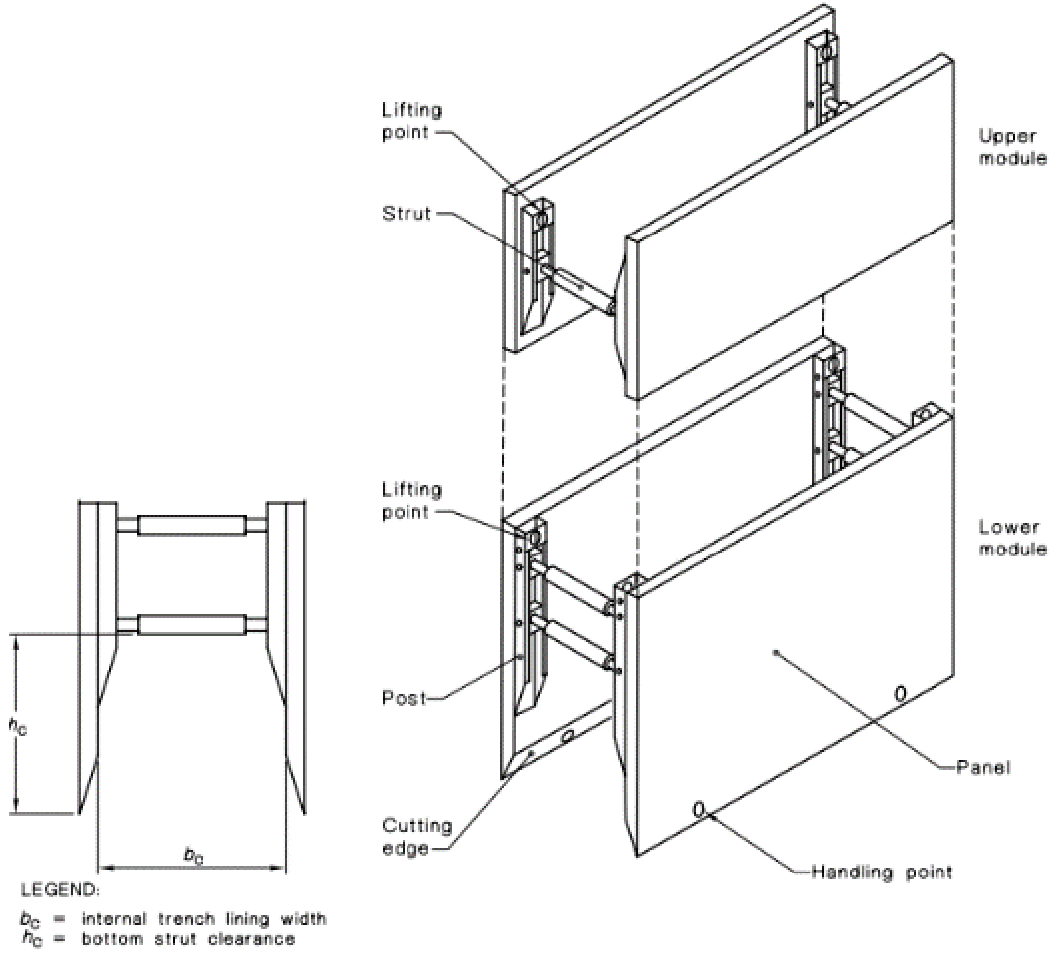


Figure 98: Example of a typical trench box (p.9 Standards Australia 2000)

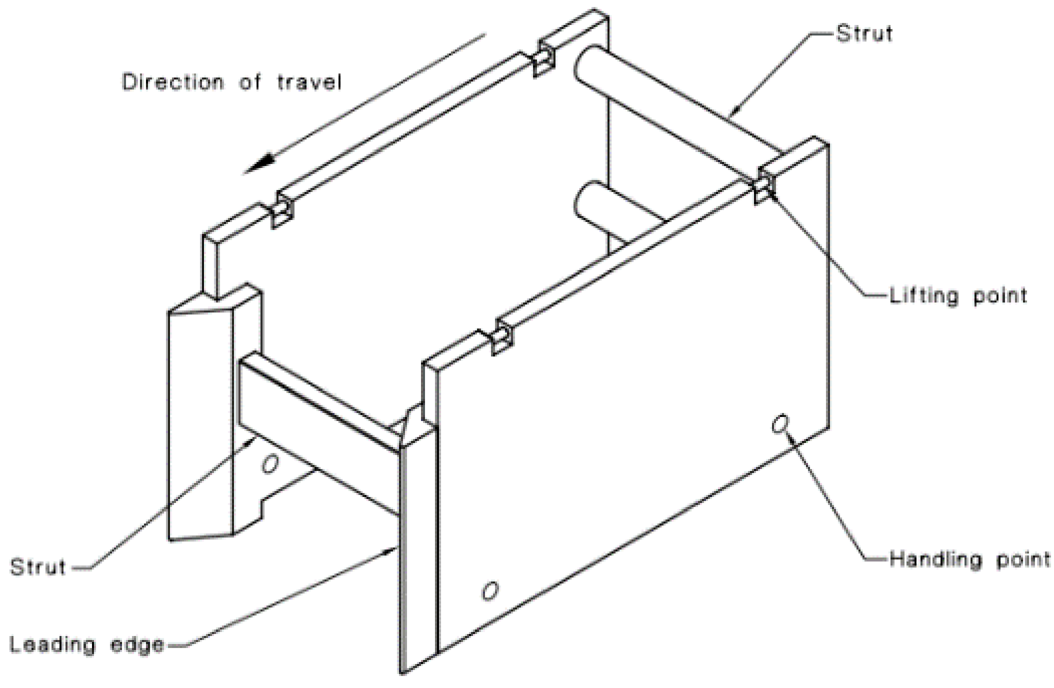


Figure 99: Example of a drag box (p. 12, Standards Australia 2000)

6.4.3.1. Supporting Components

The components that make up the drag box are shown in Figure 98 and Figure 99. The metal sheeting is relatively flexible and is placed against the excavation walls, the lateral thrust on the sheeting is resisted by the horizontal members in compression, struts. The leading edge provides a cutting edge to smooth off the edges of the excavated walls as it is dragged, often, by an excavator bucket (Standards Australia 2000)

6.4.3.2. Lateral Earth Pressures

Rankine, or Coulumb's lateral earth pressure theory can't be used in the computation of lateral earth pressure on sheetings, as those theories assume rotation about the base of rigid retaining walls about the base. For design purposes a trapezoidal distribution is assumed after the work of Terzaghi and Peck in 1967(Smith 2006).

The design of struts is semi-empirical, with the pressure envelopes used in brace design shown in Figure 100.

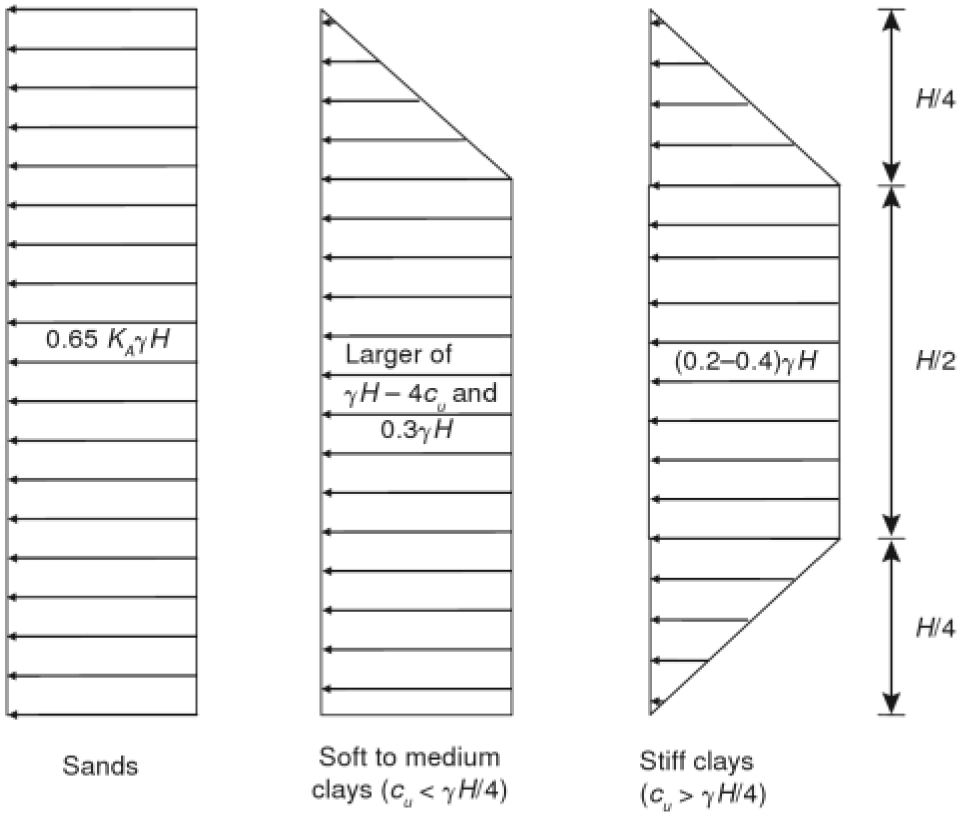


Figure 100: Pressure Envelopes for Braced Excavations (Sivakugan, Das, 2009)

For the RB5 profile;

$H = 2.0\ m$ (Deepest excavation for the profile)

$\gamma = 20\ kN/m^3$

$\gamma H = 2(20) = 40\ kPa$

$C_u = 125\ kPa$

$$\therefore P_a = (0.3)40 = 12\ kPa$$

For the AL profile, it consists of mixture of sands and clays, with the critical loads occurring for a pure sand profile. The active pressure coefficient, Ka, is given in Equation 15

$\gamma = 20\ kN/m^3$

$\Phi' = 30°$

$$K_a = \frac{1 - \sin \Phi}{1 + \sin \Phi} \qquad \textbf{Equation 15}$$

$$\therefore K_a = 0.33$$

$$\therefore P_a = 0.65(0.33)(20)(2.7) = 12\ kPa$$

It is evident that the worst pressure envelope occurs for the AL profile. During construction additional loads from plant, soil heaps and adjacent structures can impact on the trench shoring. Appendix E of AS 4744.1 (p33, Standards Australia 2000), gives the minimum uniform surcharge for plant up to 30 tonne loaded weight of 10 kPa.

Due to the narrow confines of the trenching operation, from a traffic management perspective an 8T excavator is considered, similar to a Komatsu PC88MR-8 (Figure 101), however the trench shoring loads, considers plant up to 30T.

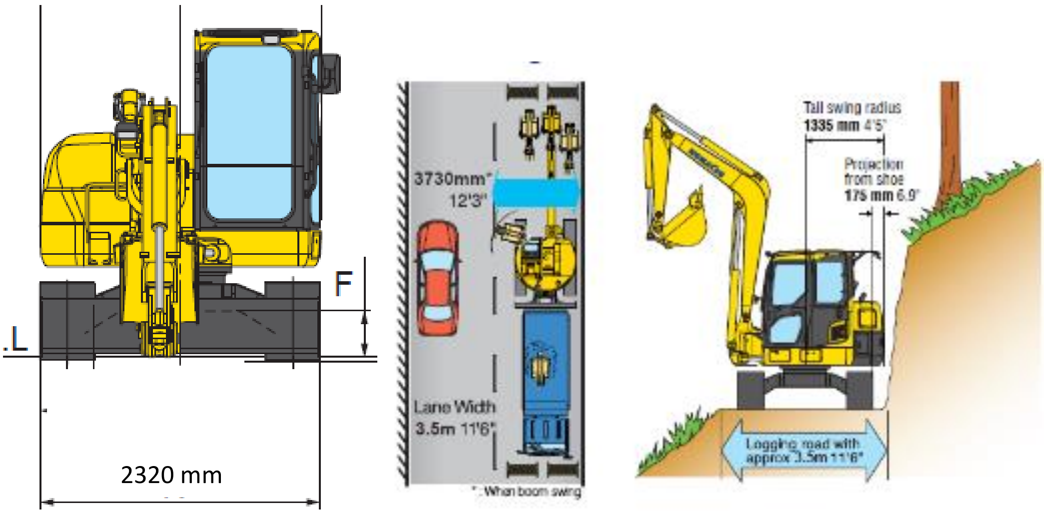


Figure 101: (Left) Komatsu PC88MR-8 (8 Tonne) Excavator Track Dimensions. (Right) Working width required of 3730mm (Komatsu 2015)

The surcharge load is assumed to occur directly next to the trench, as such the total lateral earth pressure, is rectangular, and uninform throughout and is;

$$P_a = 12 + 10 = 22\ kPa$$

6.4.3.3. Strut Design Loads

The worst possible design load case occurs for a trench depth of 2.7 metres, and has a uniform pressure of 22 kPa.

Hand Calculations

To determine the compression forces within the struts, some hand calculations have been performed. The lateral pressures are assumed to contribute to half of the total force to the front and rear struts. i.e 22x2.5=55kPa. The front strut of the drag box is 1250mm in height from the base of the excavation, and is assumed to be a hinge (Figure 102).

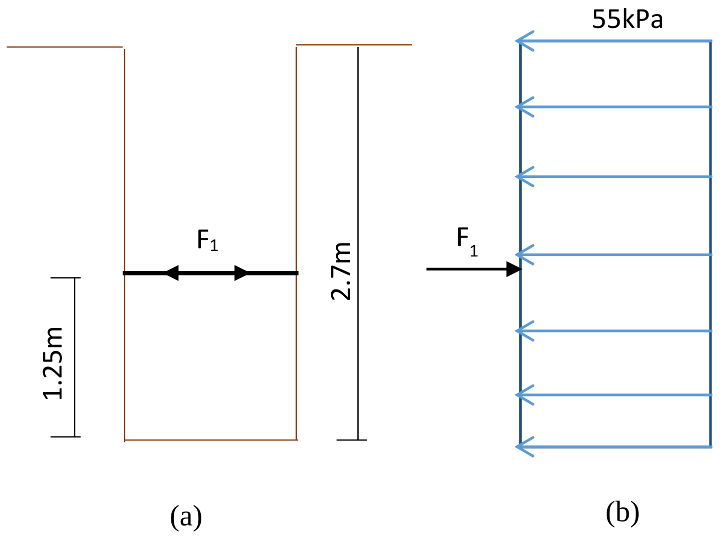


Figure 102: Force Diagram for Front Strut of Drag Box

For the equilibrium conditions;

$$\sum Horizonal\ Forces = 0 \rightarrow F_1 = 55 \times 2.7 = 148.5\ kN$$

For the rear struts, they lie at 1500 and 2400mm respectively from the base of the excavation as shown in Figure 103.

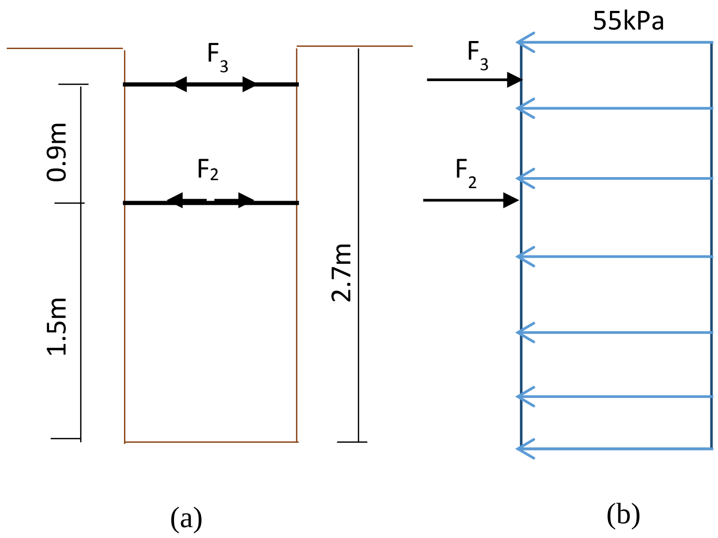


Figure 103: Force Diagram for Rear Struts of Drag Box

$$\sum Moments_{F_2} = 0 \rightarrow 0.9(F_3) = 1.2(55)(0.6) - 1.5(55)(0.75) \rightarrow F_3 = \frac{-22.3}{0.9} = -24.8\ kN$$

$$\sum Horizonal\ Forces = 0 \rightarrow F_2 = (55 \times 2.7) + 24.8 = 173.3\ kN$$

Steel Design

With the forces now determined the steel for the bracing has been designed, the trench widths are 1100, 1300 and for the GPT 3400mm. The drag box is customisable, with the interchanging of the bracing members, increasing or decreasing the external box width for the trench.

$$N_t = A_g f_y \qquad \textbf{Equation 19}$$

$$N_t = 0.85 k_t A_n f_u \qquad \textbf{Equation 20}$$

Table 38 has the critical values used in the steel design, according the AS 4100, for the front strut and Table 39 for the rear struts. For compression members the section must satisfy both Equation 16 and Equation 17, for tension members it must satisfy both equations Equation 19 and Equation 20

$$N^* \leq \phi N_s, and$$

$$N^* \leq \phi N_c$$

$$N^* \leq \phi N_t$$

$$N_s = k_f A_n f_y \qquad \textbf{Equation 16}$$

$$N_c = \alpha_c N_s \qquad \textbf{Equation 17}$$

$$\lambda_n = \left(\frac{l_e}{r}\right)\sqrt{k_f}\sqrt{\frac{f_y}{250}}$$ **Equation 18**

$$N_t = A_g f_y$$ **Equation 19**

$$N_t = 0.85 k_t A_n f_u$$ **Equation 20**

Table 38: Front Strut Calculation Values

	1100 external width	1300 external width	3400 external width
l_e (mm)[1]	1000	1200	3300
Section	RHS	RHS	RHS
Dimensions (dxbxt)mm	500x200x9	500x200x9	500x200x9
k_f	1.0	1.0	1.0
A_n (mm^2)	53772	53772	53772
f_y (MPa)	350	350	350
N_s (kN)	18820	18820	18820
α_b	-1.0	-1.0	-1.0
r (mm)	184.7	184.7	184.7
λ_n	6.4	7.7	21.1
α_c[2]	1.0	1.0	1.0
ϕN_c (kN)	16938	16938	16938

Table 39: Rear Strut Calculation Values

	1100 external width	1300 external width	3400 external width
l_e (mm)[3]	1000	1200	3300
Section	SHS	SHS	SHS
Dimensions (dxbxt)mm	200x200x9	200x200x9	200x200x9
k_f	1.0	1.0	1.0
A_n (mm^2)	6024	6024	6024
A_g (mm^2)	6600	6600	6600

[1] No lateral restraint by inspection, need to find Nc, also the aluminium plate is 50mm thick
[2] As per table 6.3.3(3) (p86. Standards Australia, 1998)
[3] No lateral restraint by inspection, need to find Nc

f_y (MPa)	350	350	350
f_u (MPa)	430	430	430
N_s (kN)	2310	2310	2310
α_b	-1.0	-1.0	-1.0
r (mm)	77.1	77.1	77.1
λ_n	15.3	18.4	50.6
α_c[4]	1.0	1.0	0.942
ϕN_c (kN)	2079	2079	1958
ϕN_t (kN)	2079	2079	2079

Therefore adopt for the front strut an **RHS 500x200x9** and the rear struts adopt a **200x200x9 SHS**

6.4.3.4. Finite Element Model

To determine the thickness of the aluminium plate, a finite element model was created using Strand7. The tensile yield strength of Aluminium Alloy (Al 6061-T6) is 290 MPa (Aloca 2010). This model was modified until the maximum von Mises stress was below 290 MPa. This was found to occur for a thickness of 50mm. Figure 104 shows the model, with the maximum stresses (221 MPa) shown in Figure 105 and the maximum displacement (1.01mm) in Figure 108. The bending moment distribution is shown in Figure 106 and Figure 107.

[4] As per table 6.3.3(3) (p86. Standards Australia, 1998)

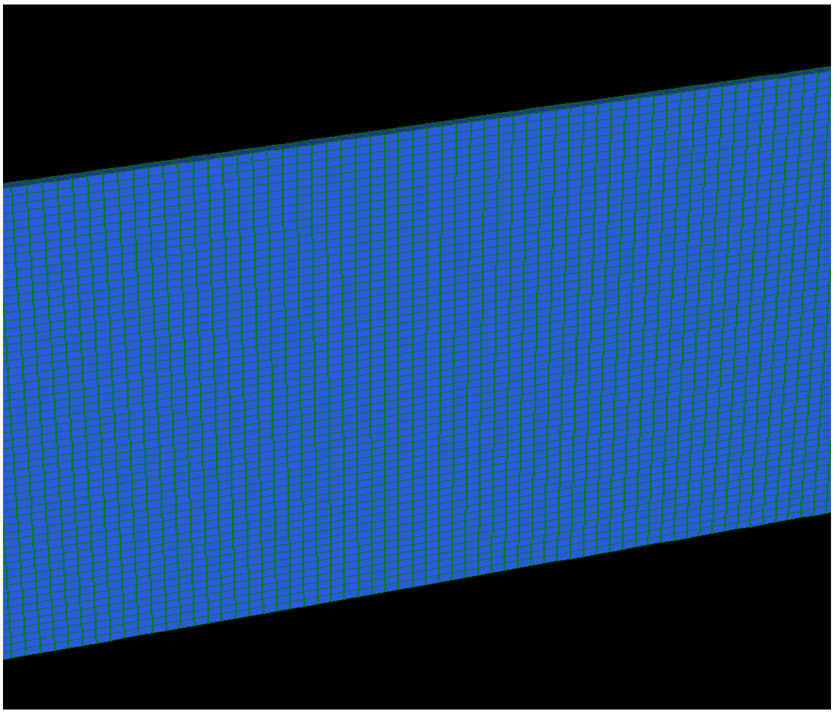


Figure 104: Strand 7 model, pink and white lines show the position of the strut bracing

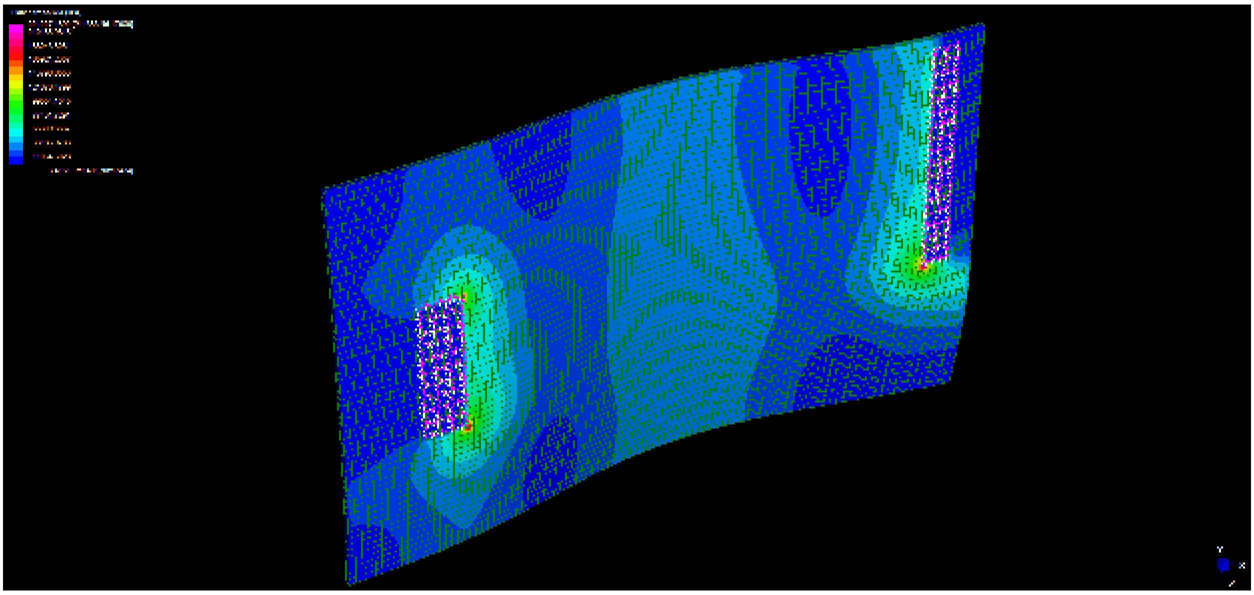


Figure 105: von Mises stress plot for the 50mm aluminium plate, note the concentration of forces around the struts

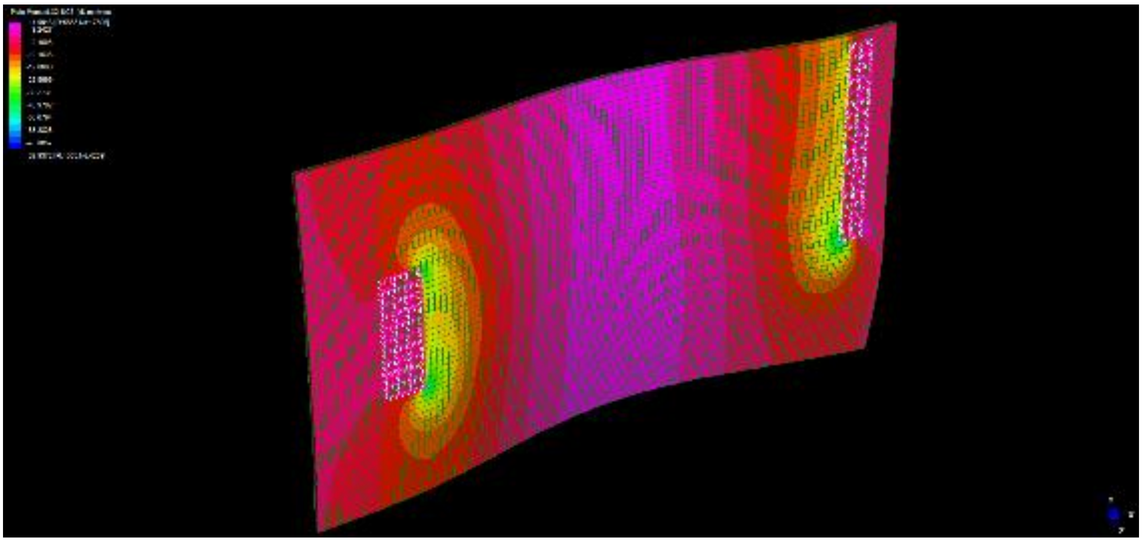


Figure 106: Bending Moment Contour Plot for the y-y axis

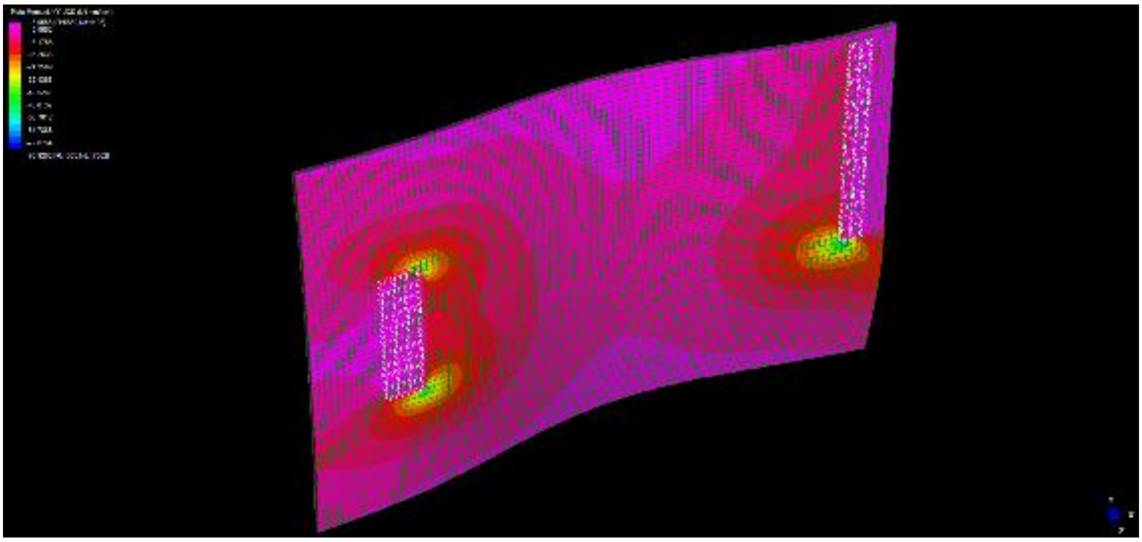


Figure 107: Bending Moment Contour Plot for the x-x axis

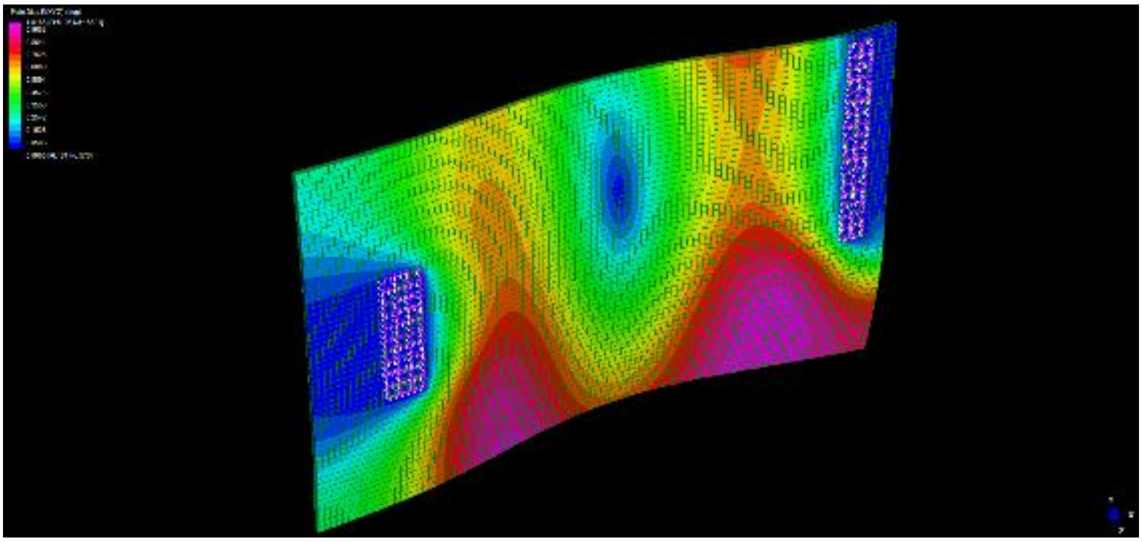


Figure 108: Displacement plot for R(x,y,z) maximum value was 1.01mm

The dimensions of the drag box are shown on the construction drawings. For construction purposes a custom aluminium and steel drag box has been designed, however due to the wide availability of trench boxes in the market place for hire, any drag box may be considered as long as it can withstand a minimum lateral earth pressure of 22 kPa.

6.4.3.5. Dragbox Usage

In the event of trench wall instability the Dragbox is designed to carry a uniform lateral earth pressure of 22 kPa. If depths of greater than 2.7 metres are required, further analysis of lateral earth pressures is required prior to use of the dragbox. Table 40 shows the sizes and quantities of struts required to make up the particular sizes of the drag box

Table 40: Sizes, Quantities and Weight of the Drag Boxes

	Front Strut		Rear Strut		Sides	
External Width (mm)	Length	Qty	Length	Qty	Dimensions (lxhxt)	Total Weight (kg)
1100	1000	1	1000	2	5000x3000x50	4328
1300	1200	1	1200	2	5000x3000x50	4352
3400	3300	1	3300	2	5000x3000x50	4620

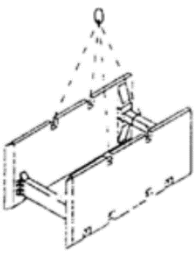
Off Loading, Site Handling and Assembly of Dragbox

Plant for Lifting

A suitable appliance is required for offloading, as the box comes preassembled. The machine lifting capacity and clearance under the lifting point should be checked against the sizes and weights of the dragbox to be lifted/assembled.

Slinging of Dragbox

Lifting of the preassembled box must be in the upright position using the 4 lifting points provided, as shown in the image below.



(Mabey 2013)

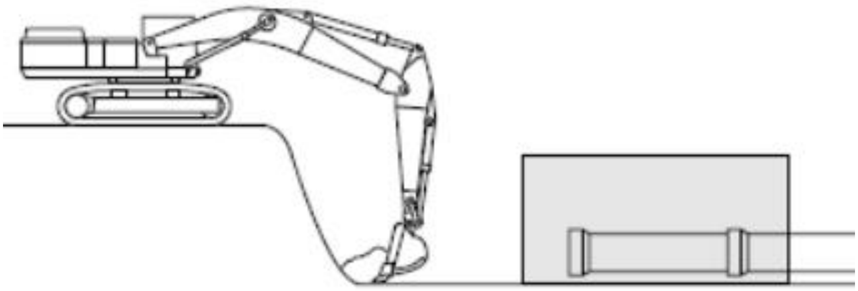
Installation

The installation of shields should be conducted by suitably qualified crane/ excavator and dogger personnel. When working in trenches with shields, it should be remembered that shields are the only means of protection. Workers should not (SA Water 2012):

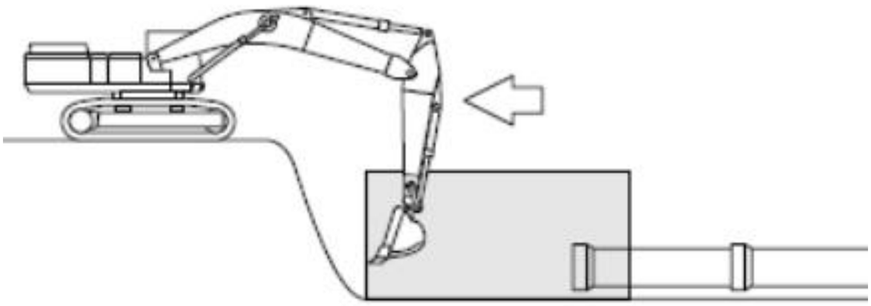
- enter the excavation area prior to the installation of the shield;
- work outside the shield area;
- enter an excavation after the shields have been removed; or
- enter a shield other than by a ladder.

The normal method used for drag boxes is as follows:

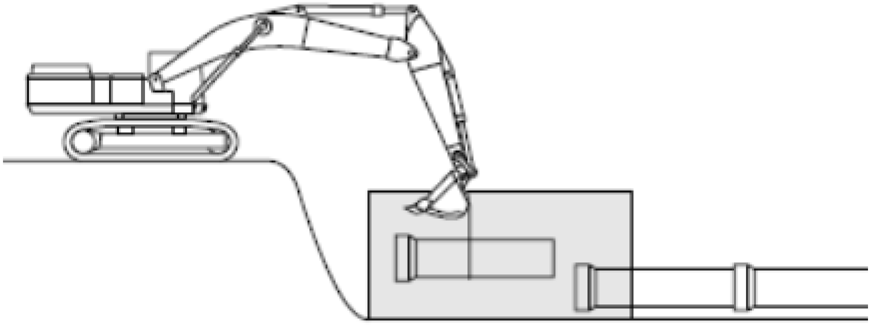
1. Excavate in front of the box. Excellent access and visibility allows for accurate digging and thus minimises hard work after the excavator has finished.



2. Pull the box forward. This is carried out by putting the bucket behind the front strut and pulling it forward using the digging action of the excavator



3. Position the pipe. After pouring and levelling the pipe bedding material, the driver again has good visibility for safe handling of the pipe. Some backfilling may be carried out before the box is moved forward.



For sandy soils, which are likely to be encountered between chainages 0 and 138 m, the trench should be excavated to approximately 1 metre deep, and the trench box placed within the trench and the excavation completed within the box. For clay soils, which are likely to be encountered between chainages 383 and 533 m, the trench may be dug to the desired depth and the trench box lowered into position (SA Water 2012).

6.4.4. Dewatering

It is not anticipated that the excavation will encounter groundwater, however a dewatering process must be allowed for, especially if a heavy rainfall event occurs during construction, due to the presence of relatively impermeable clay in the base of the excavations.

The purpose of dewatering is to;

- Establish stable ground conditions;
- To make working conditions in an excavation more acceptable;
- To provide a suitable floor on which the pipes can be founded.

A simple method of removing water from the trench is to utilise a dewatering pump, or sump pump, sometimes a hole is dug below the level of excavation and allowed to fill with water and a pump used to remove it. (SA Water 1996). Any water that is recovered from the dewatering process must be disposed of in the appropriate manner, as highlighted by the EPA Act (1993), it may need to be chemically tested (SA Water 1996).

6.4.5. Work Health and Safety

When assessing the risks associated with excavation work we should consider things such as:

- local site conditions, including access, ground slope, adjacent buildings and structures, water courses (including underground) and trees
- depth of the excavation
- soil properties, including variable soil types, stability, shear strength, cohesion, presence of ground water, effect of exposure to the elements

- fractures or faults in rocks, including joints, bedding planes, dip and strike directions and angles, clay seams
- any specialised plant or work methods required (e.g. ground support) „ the method(s) of transport, haul routes and disposal
- what exposures might occur, such as to noise, ultra violet rays or hazardous chemicals
- the number of people involved
- the possibility of unauthorised access to the work area
- local weather conditions
- the length of time that the excavation will be open. (Safe Work Australia 2012)

The following table lists the common hazards associated with excavation work, with some examples of control measures;

Table 41: Potential Hazards and Control Measures

Potential Hazards	Control Measure
Ground Collapse	Installation of ground supports
Water inrush	Pumps or other dewatering systems to remove water and prevent build up
Falls	Ramps, Steps, or other appropriate access into the excavation
Hazardous Manual Tasks	Rotating tasks between workers
Airborne Contaminants	Mechanical Ventilation to remove airborne contaminants
Buried Contaminants (e.g. asbestos)	Training to identify buried contaminants and what action to take
Underground Services	Obtain information from local authorities on the location of underground services, using DBYD and Potholing

Mechanical plant and vehicles can be located in the 'zone of influence' of the excavation, as the ground support system installed has been designed to carry such loads. The zone of influence will depend on the ground conditions. It is the zone in which there may be an influence on the excavation including possible ground collapse (see Figure 109).

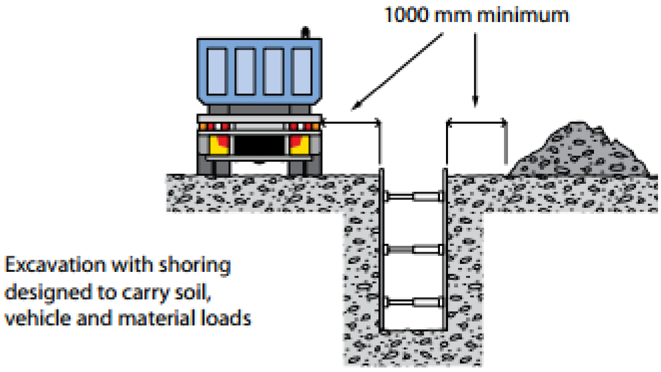


Figure 109: Zone of Influence on excavation (p.14, Safe Work Australia 2012)

It is important that materials are not placed or stacked near the edge of any excavation, as this increases the loading on the support. This has not been designed for in the drag box calculations. As such, this would put persons working in the excavation at risk. For example, the placement of material near the edge of an excavation may cause a collapse of the side of the excavation. (Safe Work Australia 2012)

To reduce the risk of ground collapse, excavated or loose material should be stored away from the excavation. Excavated material should be placed outside the zone of influence (Safe Work Australia 2012).

6.4.5.1. Plant

Excavation work cannot be carried out safely unless the plant being used is appropriate for the work and maintained in good condition. A range of plant and equipment may be used for excavation work including:

- powered mobile plant;
- air compressors;
- electric generators;
- jack hammers;
- hydraulic jacks;
- oxy-acetylene (gas cutting/welding);
- scaffolding;
- ladders; and
- any types of handheld plant such as shovels, picks, hammers, hydraulic jacks and pinch/lever bars. (Safe Work Australia 2012)

It is required that;

- plant is used and operated by a competent person;

- that appropriate guards and operator protective devices are fitted;
- that the safe working load is displayed and any load measurement devices are operating correctly; and
- plant is maintained in accordance with the manufacturer/supplier's instructions or relevant Australian Standards. (Safe Work Australia 2012)

Operators of powered mobile plant can often have severely restricted visibility of ground workers or nearby pedestrians, particularly those close to the plant (Safe Work Australia 2012). Figure 110 shows some of the blind spots for operators of typical excavation equipment used for this project.

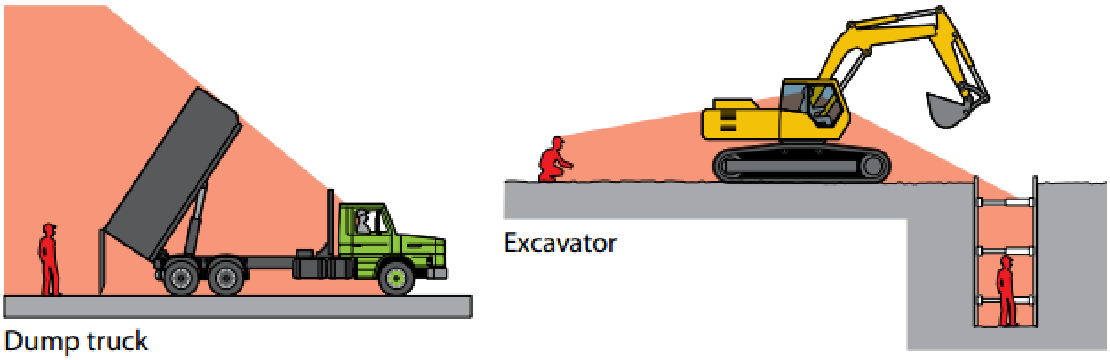


Figure 110: Blind spots of Mobile Plant Operators (Safe Work Australia 2012)

Powered mobile plant operating near ground personnel or other powered mobile plant should be equipped with warning devices (for example reversing alarm and a revolving light). Effective communication systems must be established between plant operators and ground crew, before work commences. Mobile plant operators and ground workers should be provided with and required to wear high-visibility clothing (Safe Work Australia 2012).

6.4.5.2. Falls

In managing the risks of falls, the WHS Regulations require the following specific control measures to be implemented where it is reasonably practicable to do so:

- carry out the work on solid construction that includes a safe means of access and egress;
- if a fall risk cannot be eliminated, minimise the risk of fall by providing and maintaining a safe system of work including;
  - using fall prevention devices (e.g. temporary work platforms and guard rails) or
  - work positioning systems (e.g. industrial rope access systems), or
  - fall arrest systems such as catch platforms. (Safe Work Australia 2012)

Control measures include:

- the support system itself, for example using trench box extensions or trench sheeting that is longer than the trench depth (see Figure 111);
- installing guard rails or covers on trench shields (see Figure 111);
- securing ladders to trench shields;
- installing effective barriers or barricades;
- providing clearly defined pedestrian detours;
- provision of alternative access and egress points to the excavation for emergency use; and
- backfilling the excavation as work progresses (Safe Work Australia 2012)

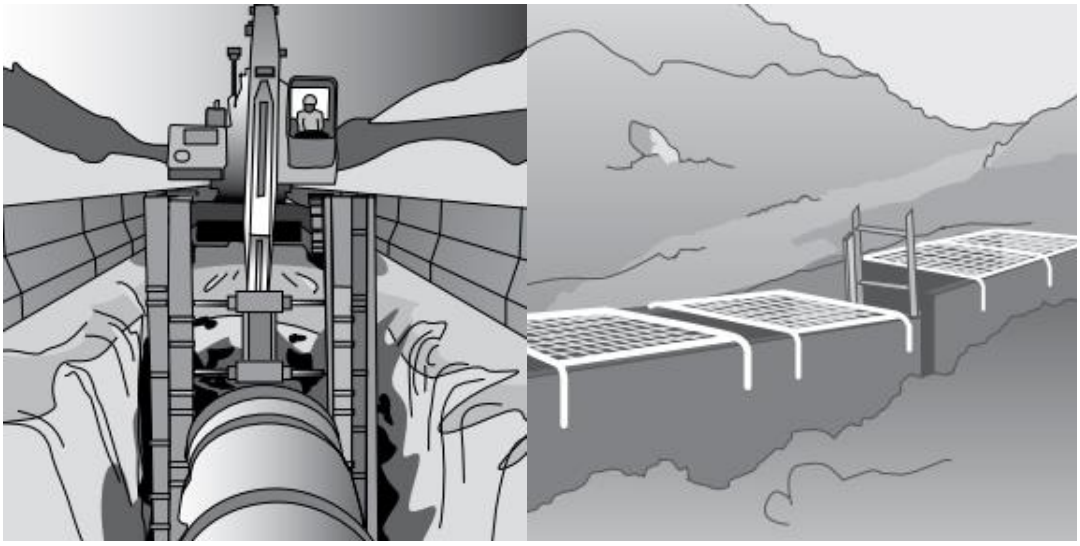


Figure 111: (left) Example of Extending the trench box above the excavation. (right) Steel mesh covers over the trench box (p. 22, Safe Work Australia 2012)

6.4.5.3. Atmospheric conditions and ventilation

Gases and fumes heavier than air can collect in excavations, with the risk of atmospheric contamination through a build-up of gases and fumes. This risk must be controlled in excavation work. Gases may include;

- sulphur dioxide;
- engine fumes (such as carbon monoxide and carbon dioxide);
- leakage from;
    - gas bottles
    - fuel tanks

    - LPG tanks (Safe Work Australia 2012)

Plant that uses a combustion engine (for example air compressors, electrical generators) should never be used in a confined excavation such as a trench if workers are in the trench. The build-up of exhaust gases in the excavation, particularly carbon monoxide, can cause death. (Safe Work Australia 2012)

Other methods of controlling the risks associated with atmospheric contamination include:

- pre-start checks of atmospheric conditions;
- using gas monitors including workers' wearing personal monitors near their airways;
- ensuring adequate ventilation (either natural or mechanical);
- working in pairs, with one person as a safety observer at the surface to monitor conditions;
- ensuring familiarity with rescue procedures; and
- using PPE (Safe Work Australia 2012)

6.4.5.4. Manual Work

An allowance for an excavator to do the majority of the earthmoving has been made, some of the trench excavation will require manual work, such as trimming from the outside of the trench by shovelling. The manual work involves pushing the side material to the bottom of the trench, where it can be picked up by the excavator. Risks associated with falls must be controlled. (Safe Work Australia 2012)

Task Specific Safety Assessment Form	
Department/Section:	**Structural and Geotechnical**
Task/ stage Name:	Trenching
Brief Description of works to be undertaken	
- Excavate to required depth using plant - Install trench box where excavation depth greater than 1.5m - Manual Trimming of sides of trench	
Summary of major risks or hazards	
- Trench collapse - Falls - Slips - Trips - Confined spaces - Moving plant - Water inrush - Underground services - Buried contaminants	
Mitigation strategies	
- Shielding (Engineering) - Barriers (Isolation) - Communication strategies (Administrative)	
Safety equipment required & number	
- Hard Hats - High Visibility clothing - Warning signs - Audible signals for moving plant - Ladders - Fencing - Trench box	

6.5. Rainwater Tank Design

During the rainwater tank analysis it was found that 4 categories of tanks are required to be structurally designed, and are summarised in Table 42. These are of a circular cylindrical shape.

Table 42: Market tank capacity and corresponding dimensions for 4 categories

Category	Volume	Diameter (mm)	Height (mm)
1	4000	1830	1750
2	5500	1800	2300
3	8000	2350	2200
4	9500	3090	1580

The tanks have been designed to rest on the ground, with the walls subjected to a hydrostatic pressure from the water inside, while the base is subject to the weight of the water and the self-weight of the structure itself.

- The walls are subjected to water pressure
- The base to carry the load of water and tank load (such as footing ; slab concrete)
- The slab (footing) has to carry load of water and tank
- It is also required to consider wind loads (for empty tank, maintenance)

The concrete slab will be designed in a similar fashion to a floor in buildings for bending moments due to water load, self-weight (dead load), live and wind loads, with the ultimate bending moments and shear will be taken into account.

6.5.1. Description

The work to be performed under these specifications includes, all labours, materials, tools and equipment necessary to design, construct, inspect and test a welded steel elevated water storage tank supported on a concrete support structure, including the foundation and accessories as shown on the drawings and specified.

6.5.2. Materials

- Rainwater Tank
    - Design considers the use of Bluescope AQUAPLATE® steel
        - Laminated galvanized steel product
        - Meets the stringent quality requirements for the storage of drinking water (Standards Australia 2014)

Figure 112: Aquaplate Steel Substrates (WaterPlex 2015)

The thicknesses of the AQUAPLATE steel are shown in

Table 43.

Table 43: Thicknesses of AQUAPLATE steel used in design

Water Capacity (Litres)	Steel thickness (mm)		
	Shell	Bottom	Top
< 4000	2.36	2.36	2.18
> 4000	4.24	6.09	2.92

6.5.3. Design Criteria

- Dead Load will be the estimated weight of all permanent construction and fittings.
- Water load must be the weight of water when the tank is filled to the overflow.
- Wind load must be based on a basic wind speed, V, and exposure category 2, in accordance with AS/NZ Standers for residential building.

6.5.4. Loading

The calculations for the wind loads are presented in Appendix 3.3.The design of the tanks considers the following loads;

- Wind Load, W (dynamic)
    - 0.5 kPa
- Dead loads or Permanent Actions

6.5.4.1. Dead Load (G)

The dead loads considered are the weight of water and the self-weight of the tank .i.e. to determine the dead load physical constants can be used in calculation:

- Water load is considered to be critical when the tank is full.

- Weight (mass) of water = 1000 kg/m³ (1000 L = 1000 kg, i.e. the volume of tank, units = L = kg).
- Weight (mass) of tank (πdht).
- Weight (mass) of concrete (in air) = 2400 kg/m³.

6.5.4.2. Determining of water pressure

The physical and hydraulic behaviour of water is important for system design, and it requires to calculate other variables. So hydrostatic pressure in a liquid (water) can determined using the following equation:

$$P = h \rho g$$

Where

p = pressure (N/m^2, Pa)

h = height of the tank (m)

ρ = density of water= 1000 kg/m^3

g = the gravitational constant (9.81 m/s^2)

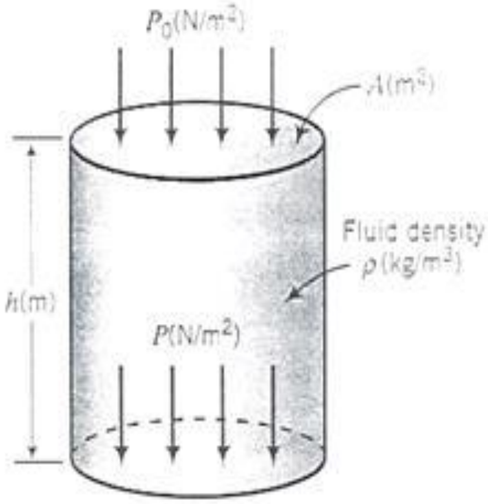


Figure 113: Pressure at the base of fluid column (hydrostatic pressure) (Larapedia 1998)

Table 44 shows the values of the pressures for all the water tanks, with an example calculation shown in Appendix 3.3.1.6.

Table 44: Hydrostatic Water Pressure values for all water tanks

Category	Litres	Height (mm)	Pressure (kPa)
1	4000	1750	17.15
2	5500	2300	22.54
3	8000	2200	21.56
4	9500	1580	15.484

6.5.4.3. Thin wall analysis

The pressure applies a force normal to the inside walls of the tank, then at the bottom it acts downwards, with the pressure increasing linearly with depth. Due to the circular nature of the tank walls, this lateral pressure places the steel into hoop tension (University of Washington n.d.).

Two types of analysis are commonly applied to pressure wall, hoop and longitudinal stresses. In the case of a simple cylinder, the tensile stress acts around the cylinder and is called "hoop stress or circumferential stress. This stress can be found using the equation:

$$\sigma_H = \frac{Pd}{2t}$$

Where:

σ_H= is the hoop stress

P =is the water pressure

d = is the tank diameter (Inner diameter)

t = is the wall thickness

The other stress is Longitudinal or axial stress which is usually smaller than hoop stress. This stress can be determine by using the equation:

$$\sigma_L = \frac{Pd}{4t}$$

Where:

σ_L= is longitudinal or axial stress

P = is the water pressure

d = is the tank diameter (Inner diameter)

t = is the wall thickness

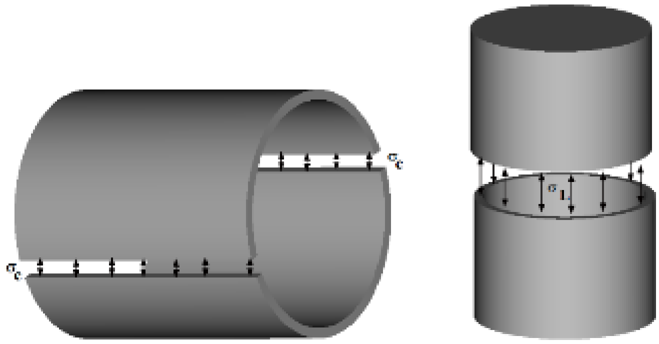


Figure 114: Cylindrical Thin-Walled Pressure. (left) hoop stress (right) longitudinal stress (University of Washington

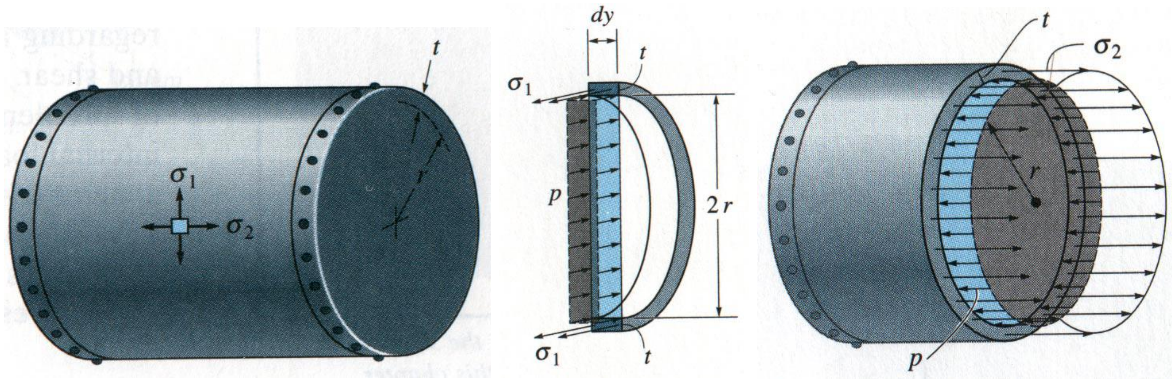


Figure 115: Free Body diagram, outlining the internal hoop stresses and axial stresses (University of Washington n.d)

nd)

All the values for the hoop stress and axial stress are presented in Table 45, it is apparent that the maximum stresses do not exceed the allowable, 250 × 0.9 = 225 MPa.

Table 45: Maximum hoop stress and axial stress on the water tanks

Volume	Maximum hoop stress (MPa)	Maximum axial stress (MPa)
4000	6.6	3.3
5500	4.8	2.4
8000	6.0	2.1
9500	5.6	2.8

6.5.5. Site Preparation

Correct site preparation is essential to ensure a long trouble free life for Rainwater tank. It is important to have the water tank on a flat level compacted base, the base of water tank should be fully supported at all times (Team Poly 2015).

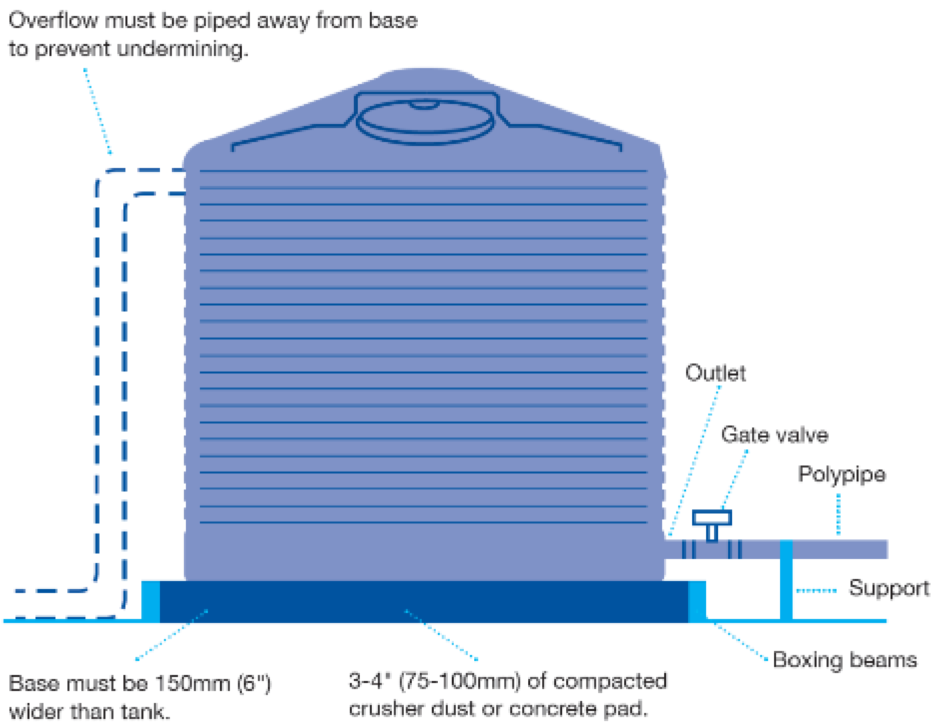


Figure 117: Common dimensions of tank base (Team Poly 2015)

6.5.6. Concrete Slab

Concrete slab should be at least 25 MPa and 100 mm thick with F62 (also called SL62) mesh halfway through the mix (compliant with Australian standards code; AS 2870 Residential Slab footings and AS 3600 Concrete Structures). The slab must be flat, smooth and level with no high or low spots. The following criteria will be needs to be met for slab design:

- Earthwork including levelling and compacting
- 100 mm concrete slab thickness, reinforced with steel wire mesh
- The tank base must be 150 mm wider than tank diameter

Associated Standards:

- AS 2870 Residential Slab footings
- AS 3600 Concrete Structures
- AS 4100 Steel Structures
- AS/NZS Structural design actions

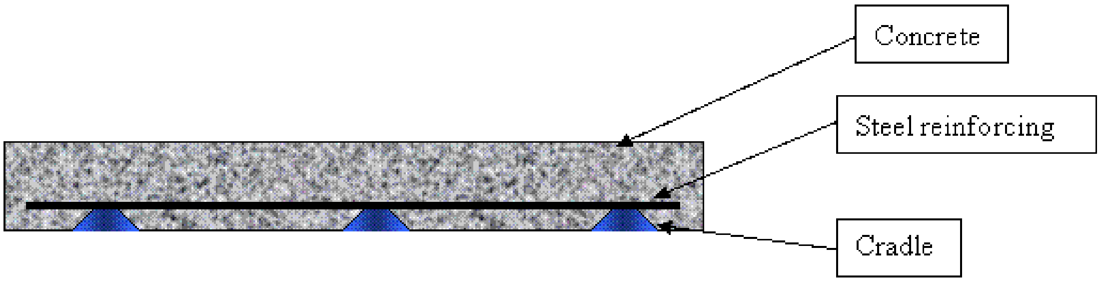


Figure 118: Typical of the reinforced concrete slab (NSW HSC nd)

Table 46: Reinforcement

Description	Length (m)	Width (m)	Wire spacing (mm)	Wire Diameter (mm)	Wight (kg)	Price ($)
SL62	6	2.4	200 × 200	6	33	$66.06

This mesh is used for reinforcing thin concrete elements such as floor slabs where more strength is required in one direction than the other. It rectangular shape is manufactured which are welded together, i.e. previously called F62 (now SL 62) = fabric 6mm bars welded together in a 200mm square grid. Sheet size 6m X 2.4m.

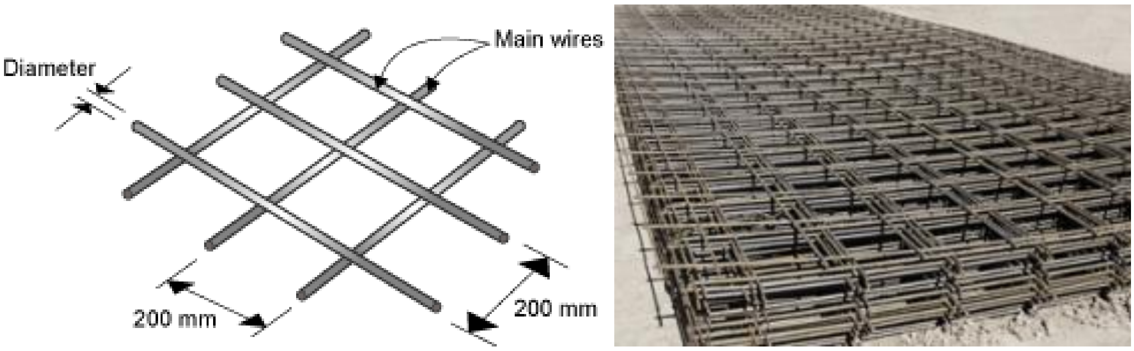


Figure 119: Typical reinforcing mesh use for concrete slab of the base of the water tank (Onesteel 2014)

The tables below shows the output of the calculations of the loadings for the steel tank and water inside the tank.

Table 47 presents the relevant properties of the steel tank used for the design calculations.

Table 47 Properties of Steel for the tank

Property of steel	symbol	value
Young's Modulus of Elasticity	E	200×10^3 MPa
Shear Modulus of Elasticity	G	80×10^3 MPa
Density	ρ	7850 kg/m³

Table 48 is presents the output calculations for the area and weight of the tank containing water.

Table 48: Determine the weight of the steel tanks

category	Height (m)	Diameter (m)	Area (m²)	Weight (kg)
1	1.75	1.83	15.26	282.7
2	2.30	1.80	17.97	623.6
3	2.20	2.35	24.78	859.8
4	1.58	3.09	30.19	1047.5

The table below illustrates the calculated values for the ultimate loading produced by the water and steel weight.

Table 49 Output calculation for ultimate loading using total weight (water + steel)

category	Steel weight (kg)	Water weight (kg)	Total weight (kg)	Ultimate Load (kN)
1	282.7	4000	4282.7	42
2	623.6	5500	6123.6	60
3	859.8	8000	8859.8	87
4	1047.5	9500	10547.5	103

The pressures above and below the slab are effectively balance out, therefore the ultimate moment in the slab is very small and can be ignored. The ultimate flexural moment capacity of the slab is not calculated for any of the slab sections due to this load balancing effect.

The dimensions of the reinforced concrete slab is presented in the tables below, these values are calculated; all calculations are available in Appendix 3.3.1.6.

The output for the calculations of the determined allowable dimension (B) for every tank considered is presented in Appendix 3.3.1.6. The output of the results is shown in the table below.Table 50 Allowable dimensions for every slab and tank configuration

Dimension	Allowable B (m)
Tank 1	0.63
Tank 2	0.74
Tank 3	0.87
Tank 4	0.94

The results presented in the table above shows the allowable dimension, B. This dimension is used to check whether the chosen dimensions for each tank and slab configuration is ok. This check is completed for every tank below.

Tank 1 (4000L):

In reference to Figure 117, the dimensions of the slab are calculated below. The diameter of the first tank, as illustrated in Table 42 is 1830mm. The dimensions of the slab must be increase by 150mm in length and width as per Figure 117. The calculation for the final dimensions of the slab is shown below.

Calculations:

$$Slab = 1.83m + (0.15m) = 1.98\,m = 2.00\,m$$

The chosen reinforcement for this slab is selected based off the OneSteel catalogue and relevant Australian Standards introduced in section 6.5.5.

Table 51 Tank 1 base dimensions and specifications

Specified dimensions	Values
Tanks	
Volume	4000 L
Diameter	1.83 m
Height	1.75 m
Tank base	
Elevation above ground	100 mm
Plan	2.00 m × 2.00m
Tank base foundation	

Slab	2.00m × 2.00m × 100 mm
Reinforcing bar	6 mm Ø SL62 mesh

Tank 2 (5500L):

The calculations are repeated as per tank 1 descriptions.

Calculations:

$$Slab = 1.8m + (0.15m) = 1.95\,m$$

Table 52 Tank 2 base dimensions and specifications

Specified dimensions	Values
Tanks	
volume	5500 L
Diameter	1.80 m
Height	2.30 m
Tank base	
Elevation above ground	100 mm
Plan	1.95 m × 1.95 m
Tank base foundation	
Slab	1.95m × 1.95m × 100 mm
Reinforcing bar	6 mm Ø SL62 mesh

Tank 3 (8000L):

The calculations are repeated as per tank 1 descriptions.

Calculations:

$$Slab = 2.35m + (0.15m) = 2.5\,m$$

Table 53 Tank 3 base dimensions and specifications

Specified dimensions	Values
volume	8000 L
Diameter	2.35 m
Height	2.2 m
Tank base	
Elevation above ground	100 mm
Plan	2.5 m × 2.5 m

Tank base foundation	
Slab	2.5m × 2.5m × 100 mm
Reinforcing bar	6 mm Ø SL62 mesh

Category 4:

The calculations are repeated as per tank 1 descriptions.

Calculations:

$$Slab = 3.09m + (0.15m) = 2.94\ m = 2.95\ m$$

Table 54 Tank 4 base dimensions and specifications

Specified dimensions	Values
Tanks	
volume	9500 L
Diameter	3.09 m
Height	1.58 m
Tank base	
Elevation above ground	100 mm
Plan	2.95 m × 2.95 m
Tank base foundation	
Slab	2.95m × 2.95m × 100 mm
Reinforcing bar	6 mm Ø SL62 mesh

6.6. Sandstone Arch Culvert

6.6.1. Introduction

The termination point of the stormwater system occurs at the base of a sandstone arch, maintaining the integrity of the arch is paramount and this section of the report details this connection, and the associated temporary support frame. Figure 120 shows the dimensions of the brick arch, as highlighted in the Feasibility study.

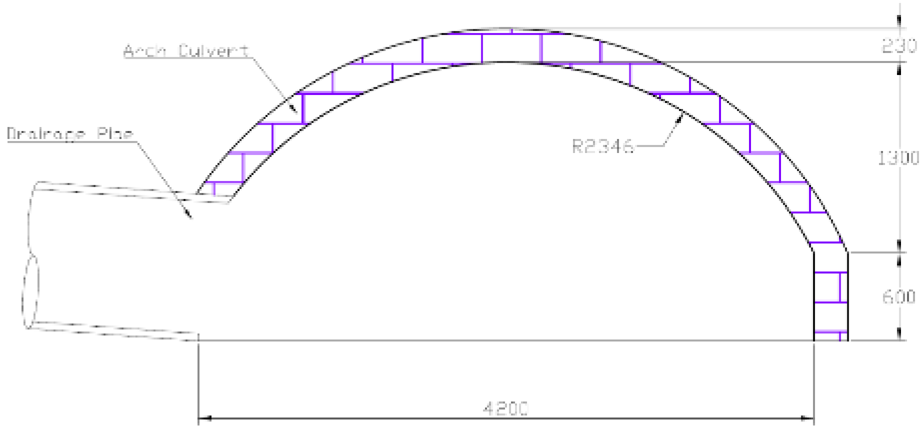


Figure 120: Sandstone Arch Culvert

6.6.2. Design Loads

The design loads on the culvert include the self-weight of the structure, live traffic loads and vertical and horizontal earth pressures. Due to the shape of the culvert, vertical and horizontal loadings vary along its width and height, thus five evenly spaced points along the arch (A, B, C, D and E) have been evaluated. The location of these five points can be seen below in Figure 121.

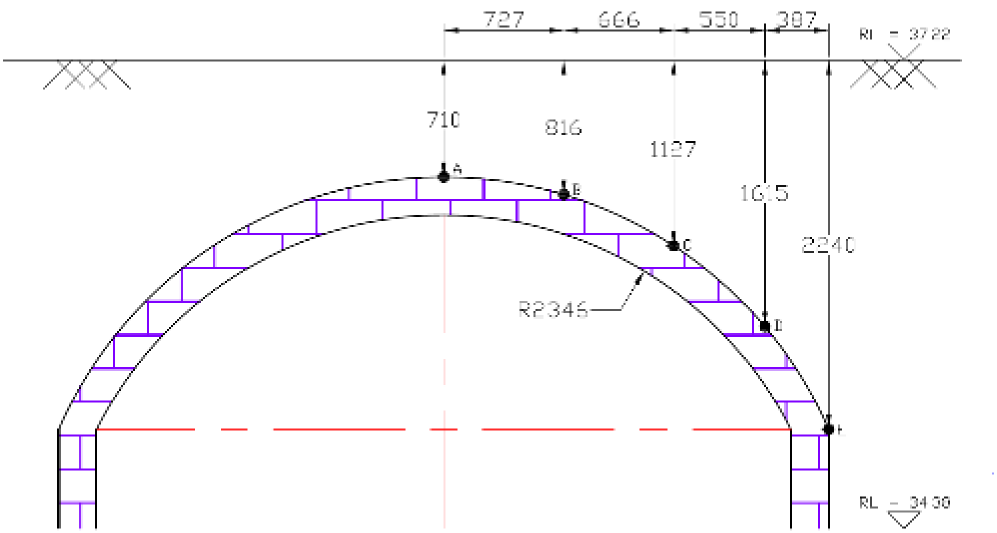


Figure 121: Load Points

6.6.2.1. Critical Conditions

Case 1 – Symmetrical Loading

All load combinations have been analysed and are presented in Appendix 3.6, with Figure 122 and Figure 123 showing the critical vertical and horizontal loadings for a symmetrical loading, or for a live load occurring directly above the arch culvert.

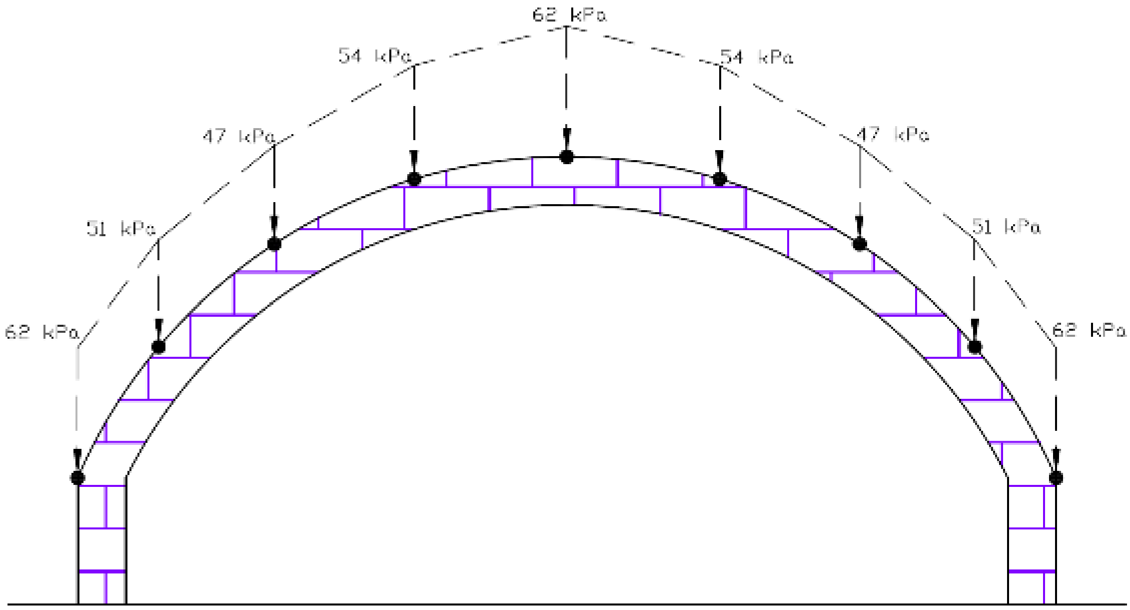


Figure 122: Critical Vertical Loadings - Case 1

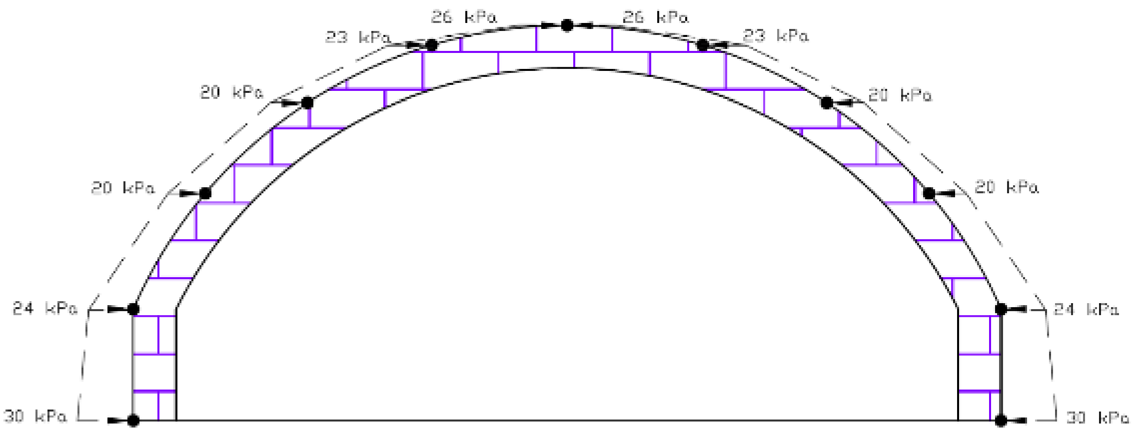


Figure 123: Critical Horizontal Loadings - Case 1

Case 2 – Asymmetrical Loading

For an asymmetrical loading, or live load occurring off centre to the arch, the critical loads are shown in Figure 124 and Figure 125.

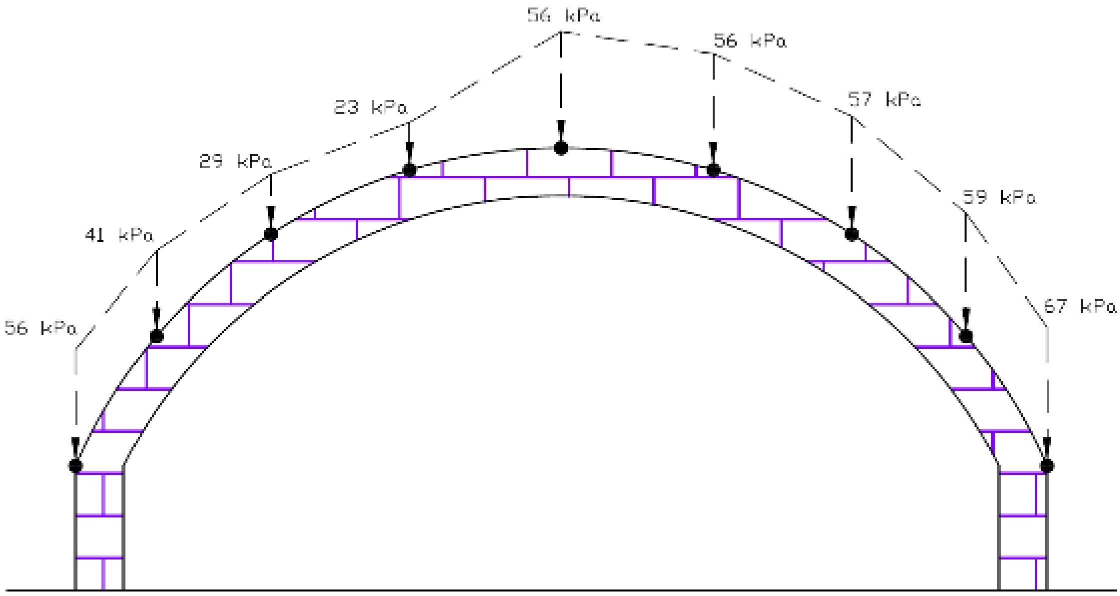


Figure 124: Critical Vertical Loadings - Case 2

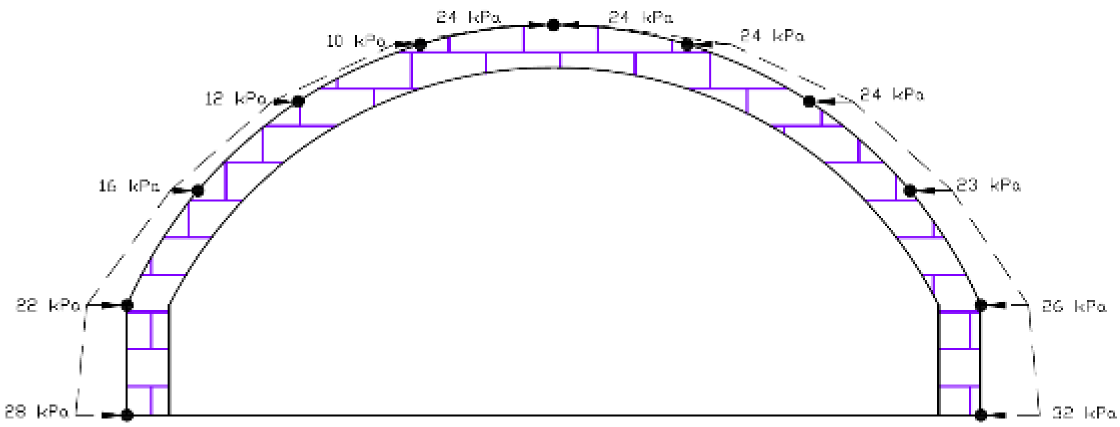


Figure 125: Critical Horizontal Loadings - Case 2

6.6.3. Drainage Pipe Connection

6.6.3.1. Introduction

As stated in section 6.6.1 and shown in Figure 120, a 750mm drainage pipe is to be connected straight into the arch culvert. Hydro-Future has decided to use a reinforced concrete section around the drainage pipe to protect the arches longevity and decrease the loadings on the pipe.

6.6.3.2. Finite Element Analysis

Hydro-Future has produced a finite element model (Strand7) using the factored design load for symmetrical and asymmetrical vertical and horizontal loading conditions as seen in section 6.6.2. The analysis of the arch culvert as a 3D brick and as a 2D beam were both undertaken to give Hydro-Future a clear understanding of the stress concentrations, bending moments, shear forces and axial forces to be used when designing the reinforced concrete section.

6.6.3.3. 3D Brick Model

The arch culvert was modelled as a 3D brick element for a length of 7 meters, which is the load width for 2 trafficable lanes. Figure 126 is the longitudinal view of the arch culvert showing the connection point. The 3D brick element was modelled to understand the stresses for which 2 trafficable lanes would induce on the culvert.

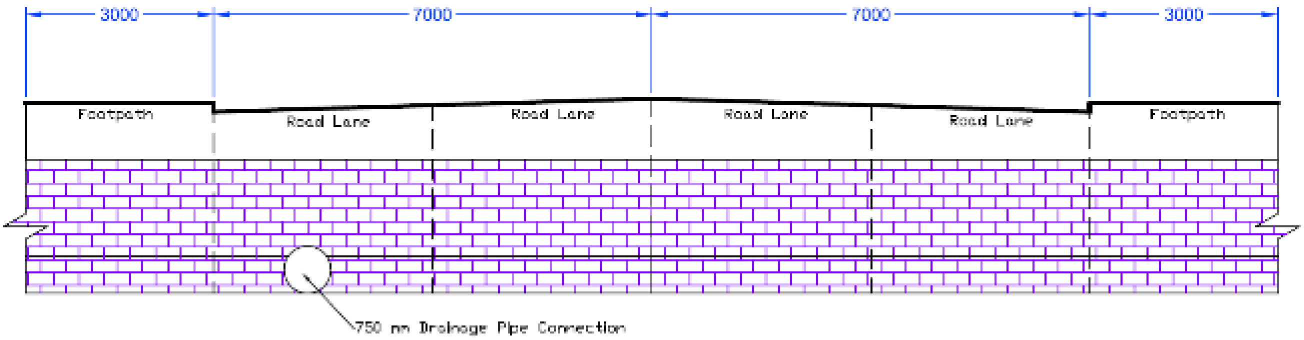


Figure 126: Arch Culvert Longitudinal View

Load Case 1 – Symmetrical

Figure 127 to Figure 134 are the finite element meshes showing the stress distribution of live and dead loads throughout the arch culvert and the drainage pipe connection. It can be seen that there is a high amount of stress located in and around the drainage pipe void.

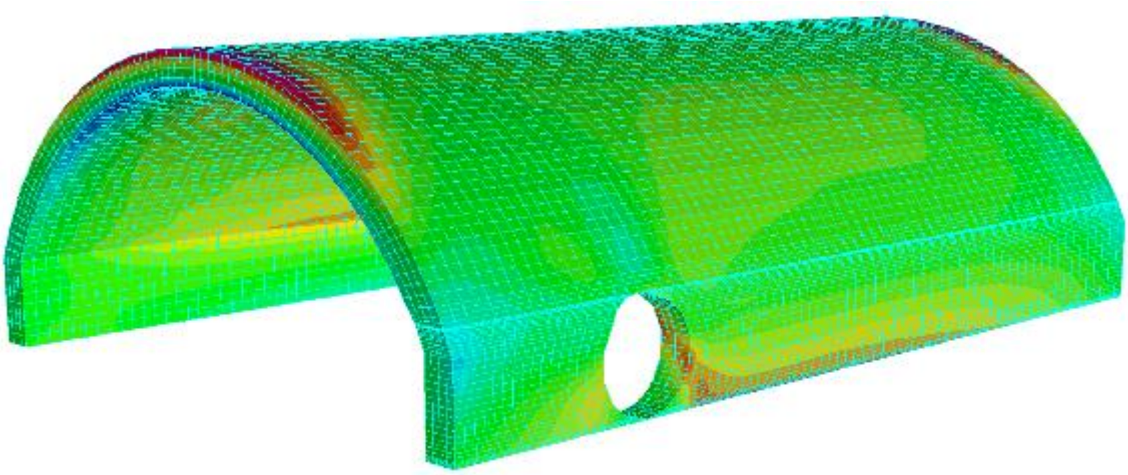


Figure 127: Arch Culvert Mesh (Mean Stress)

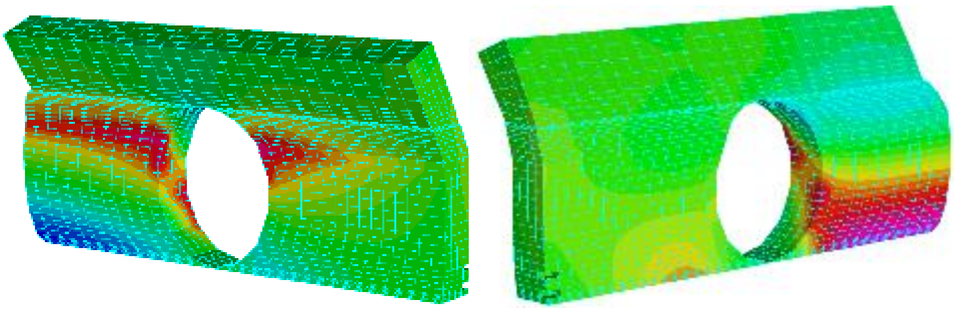


Figure 128: Stress Concentration (XX)

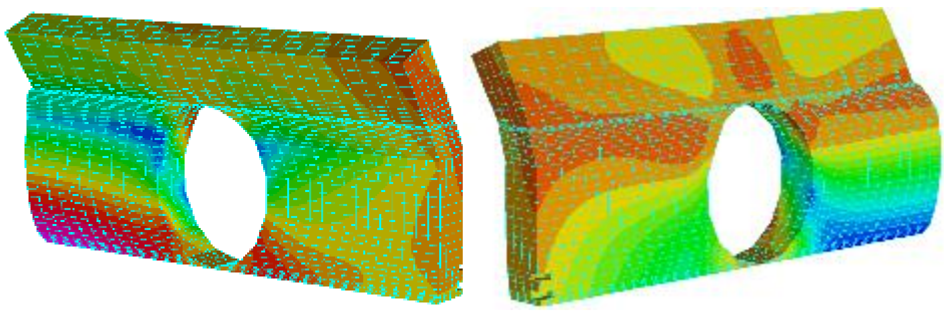


Figure 129: Stress Concentration (YY)

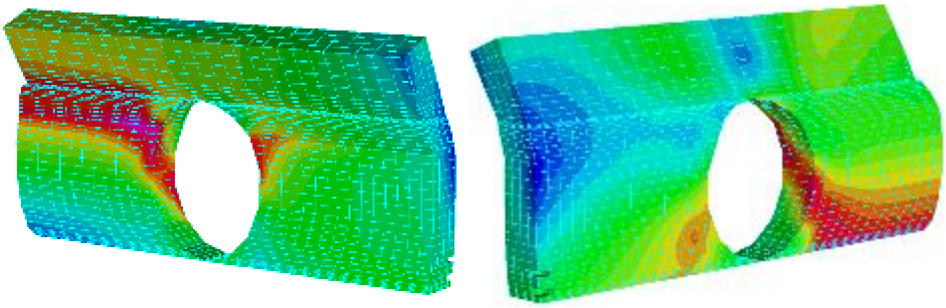


Figure 130: Stress Concentration (ZZ)

Table 55 shows the critical stresses occurring in the XX, YY and ZZ plane for loading case 1.

Table 55: Critical Stresses - Load Case 1

Stress	Critical Positive (kPa)	Critical Negative (kPa)
XX	$9.7 * 10^2$	$-5.4 * 10^2$
YY	$8.35 * 10^2$	$-1.57 * 10^3$
ZZ	$7.19 * 10^2$	$-4 * 10^2$

Load Case 2 – Asymmetrical

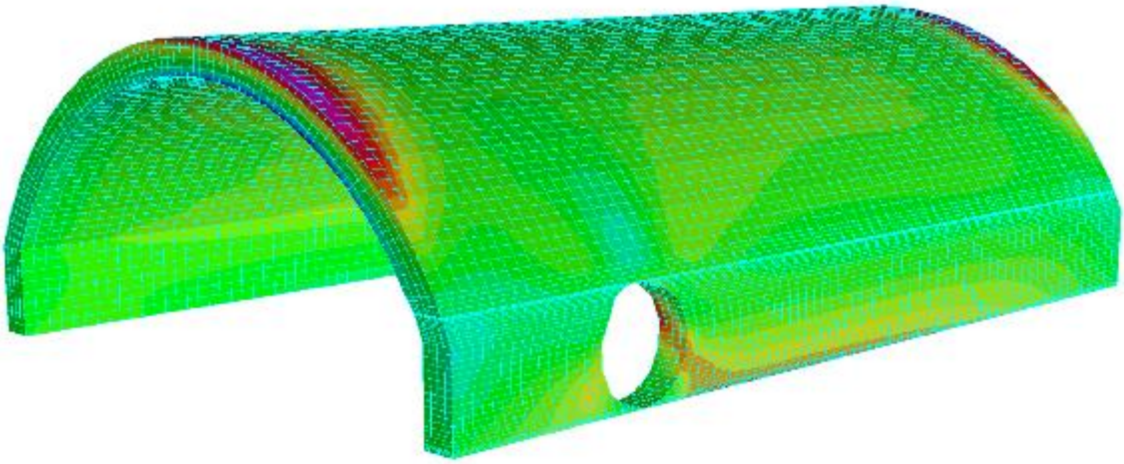


Figure 131: Arch Culvert Mesh

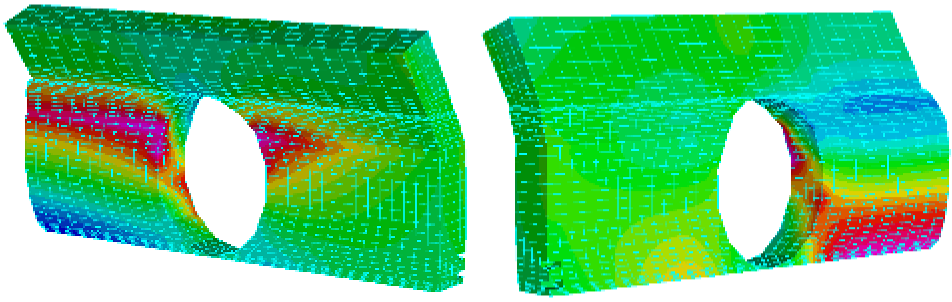


Figure 132: Stress Concentration (XX)

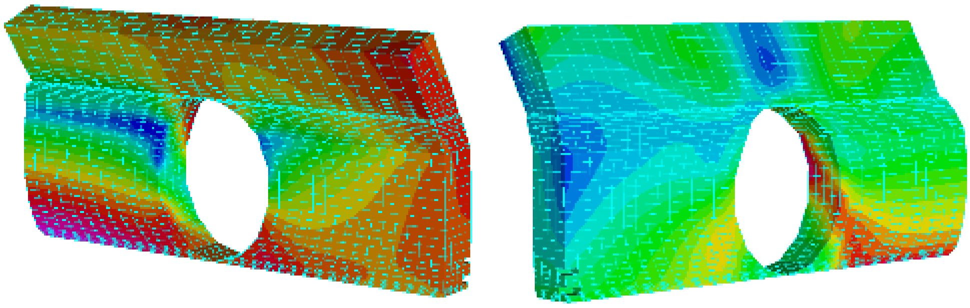


Figure 133: Stress Concentration (YY)

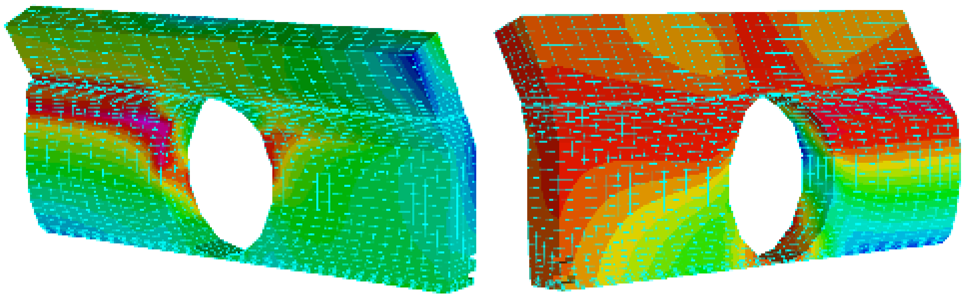


Figure 134: Stress Concentration (ZZ)

Table 56: Critical Stresses - Case 2

Stress	Critical Positive (kPa)	Critical Negative (kPa)
XX	$1 * 10^3$	$-4.45 * 10^2$
YY	$6.7 * 10^2$	$-1.7 * 10^3$
ZZ	$8.5 * 10^2$	$-4 * 10^2$

As observed throughout the strand7 models, there are high stress concentrations occurring in and around the drainage pipe hole. This is a large concern for Hydro-Future as this analysis shows the hole will attract higher stresses which are a structural hazard considering the region will have unrestrained bricks and mortar. A solution has been devised to ensure stresses occurring around the drainage pipe hole are absorbed by a modern structure and to ensure the section is sealed to ensure soil or groundwater erosion does not occur.

6.6.3.4. 2D Beam Model

A 2D beam element was modelled to understand the shear force, bending moments and axial forces at a cross sectional level of the arch culvert as seen below in Figure 135. The cross section was modelled per meter width of the arch culvert and the ultimate vertical and horizontal pressures were treated as uniformly distributed loadings.

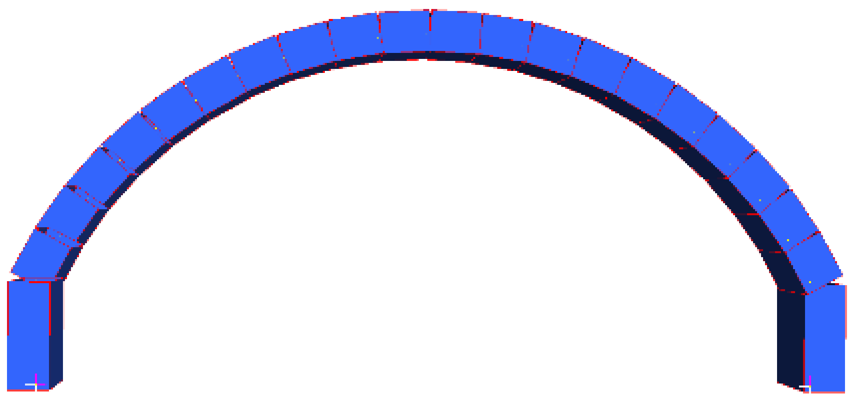


Figure 135: 2D Beam Element

Load Case 1 – Symmetrical

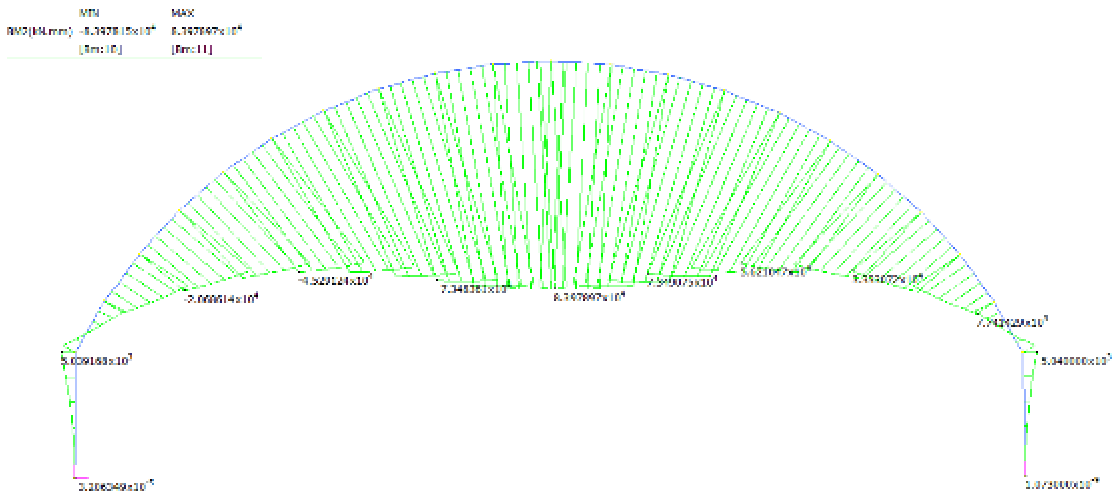


Figure 136: BMD, Case 1

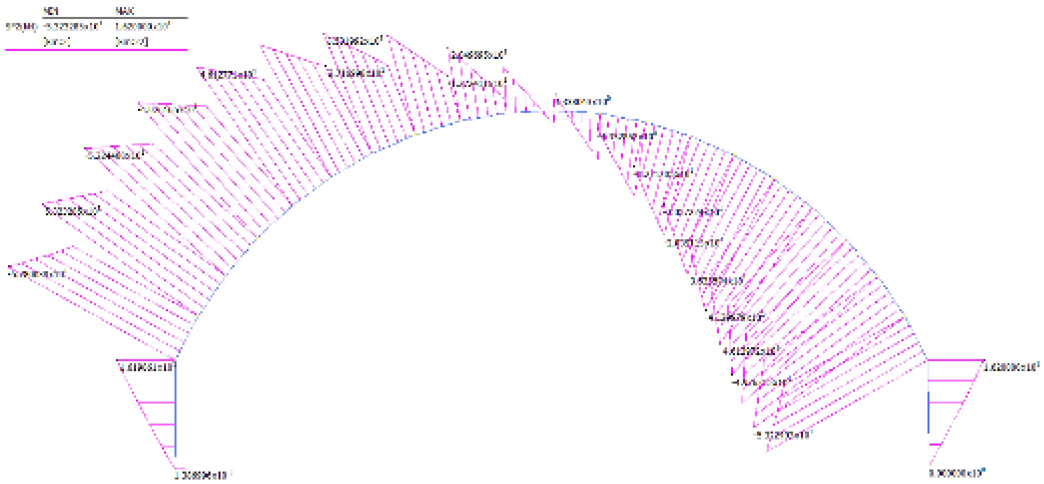


Figure 137: SFD, Case 1

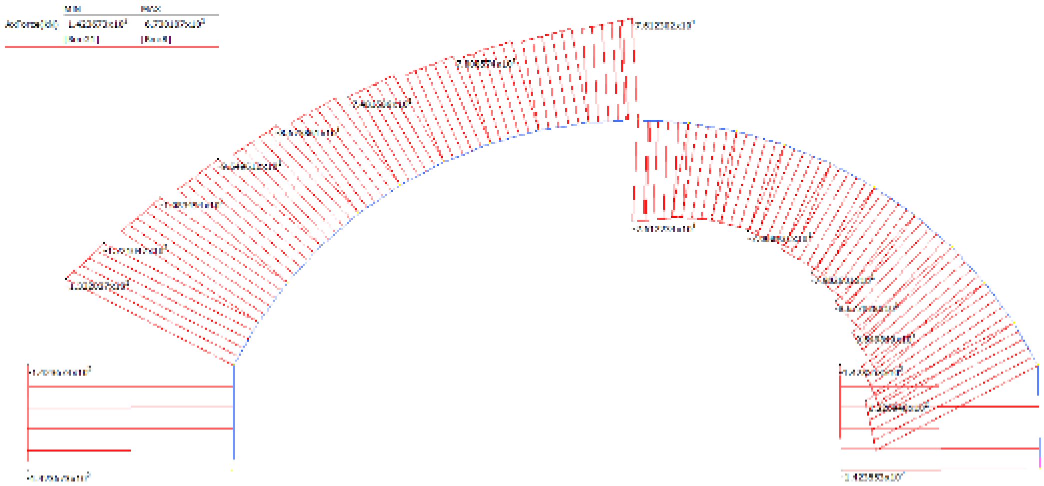


Figure 138: AFD, Case 1

Load Case 2 – Asymmetrical

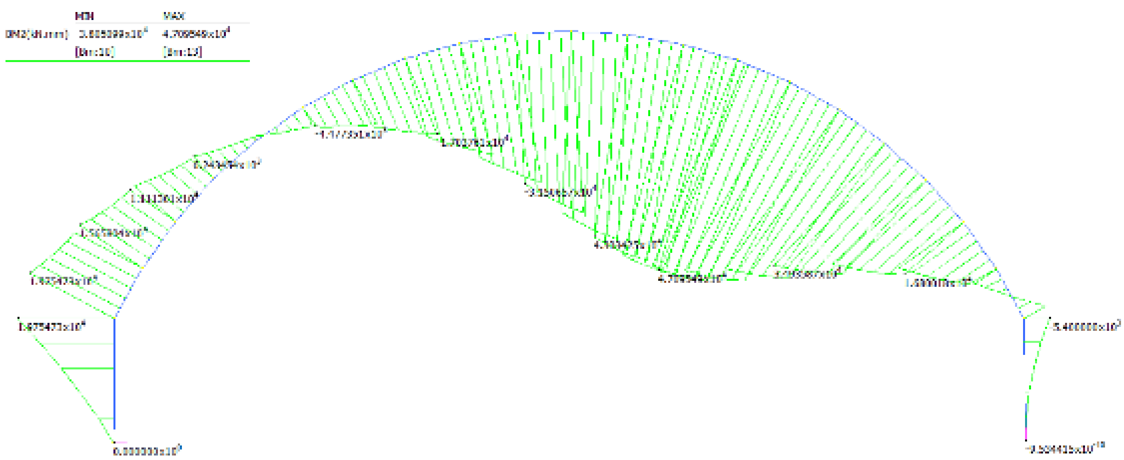


Figure 139: BMD, Case 2

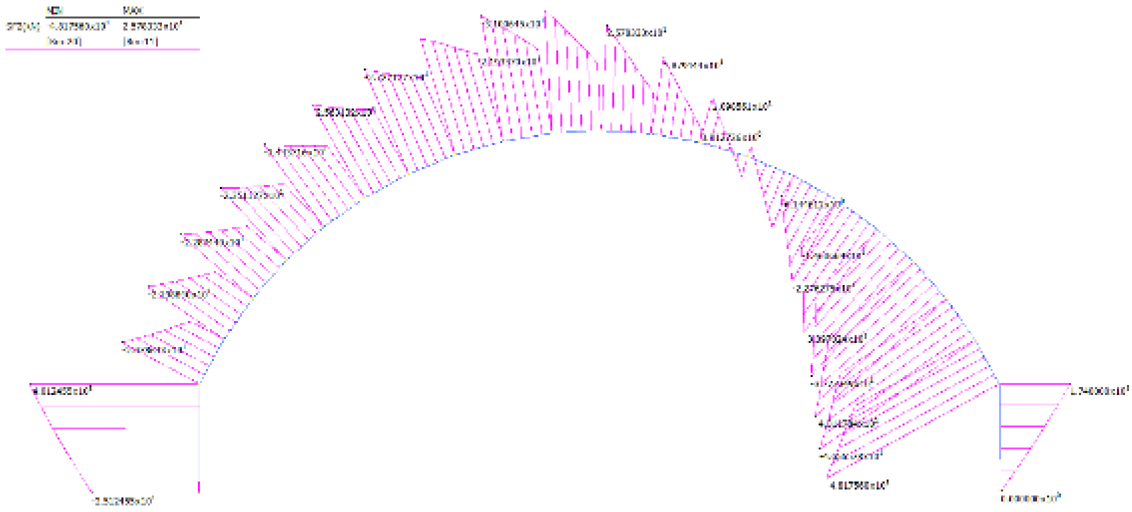


Figure 140: SFD, Case 2

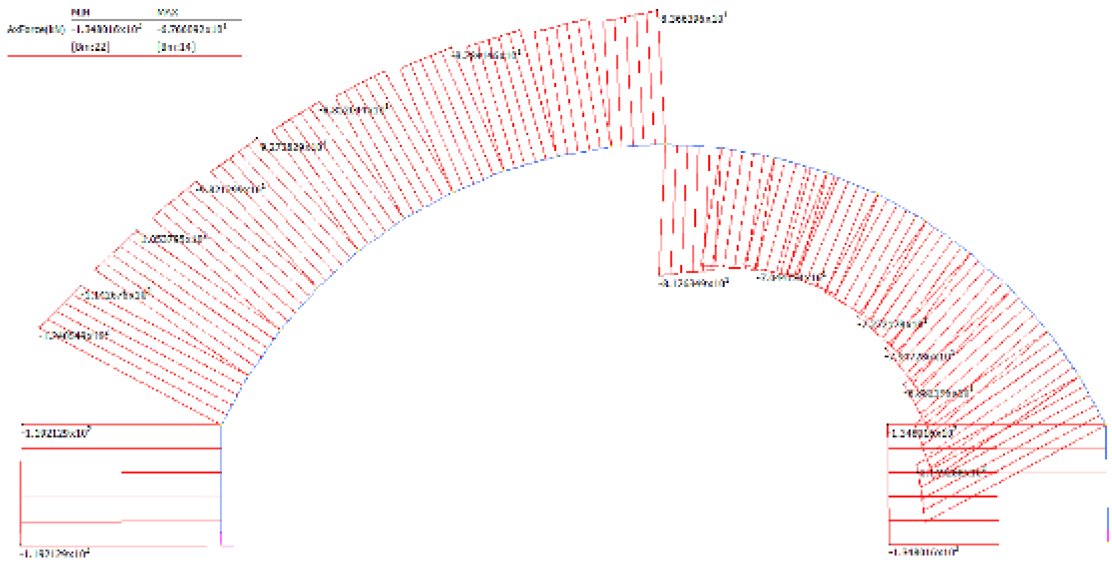


Figure 141: AFD, Case 2