1.3. Water Harvesting

1.3.1. Tank Categories

Table 93: Market tank capacity and corresponding dimensions for 4 identified categories (Rainwater tanks direct 2013)

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

Table 94: Typical market tank capacity and corresponding dimensions (Rainwater tanks direct 2013)

Volume (L)	Diameter (mm)	Height (mm)
1000	1100	1300
1500	1150	1700
2000	1200	1900
2500	1450	1700
3000	1410	2200
3500	1500	2150
4000	1830	1750
4500	2400	1530
5000	1840	2000
5500	1800	2300
7200	2170	2320
7500	2180	2300
8000	2350	2200
9000	2570	2300
10000	2200	3000

Figure 261: Typical tank type and dimension (Rainwater tanks direct 2013)

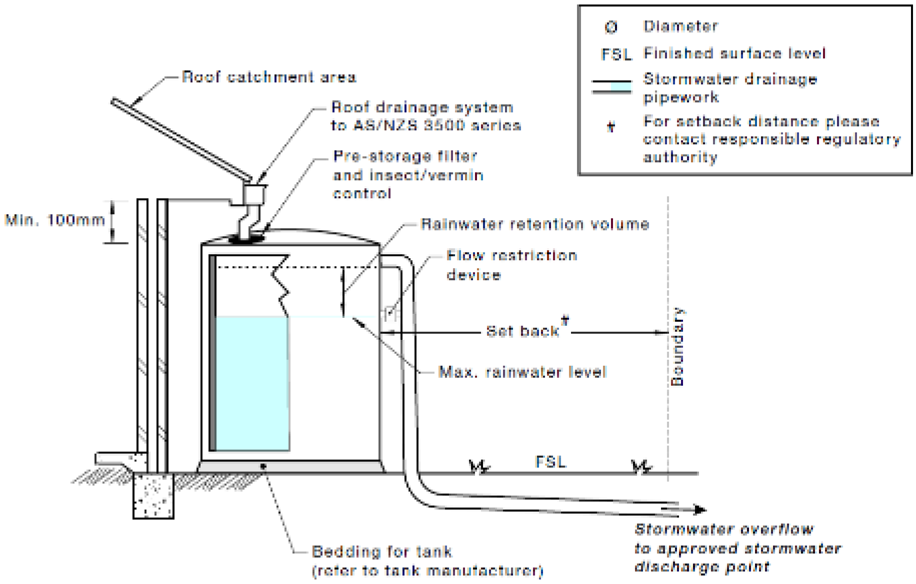

Figure 262: Typical design option of the tank above ground and how rainwater tank can be part of a stormwater system

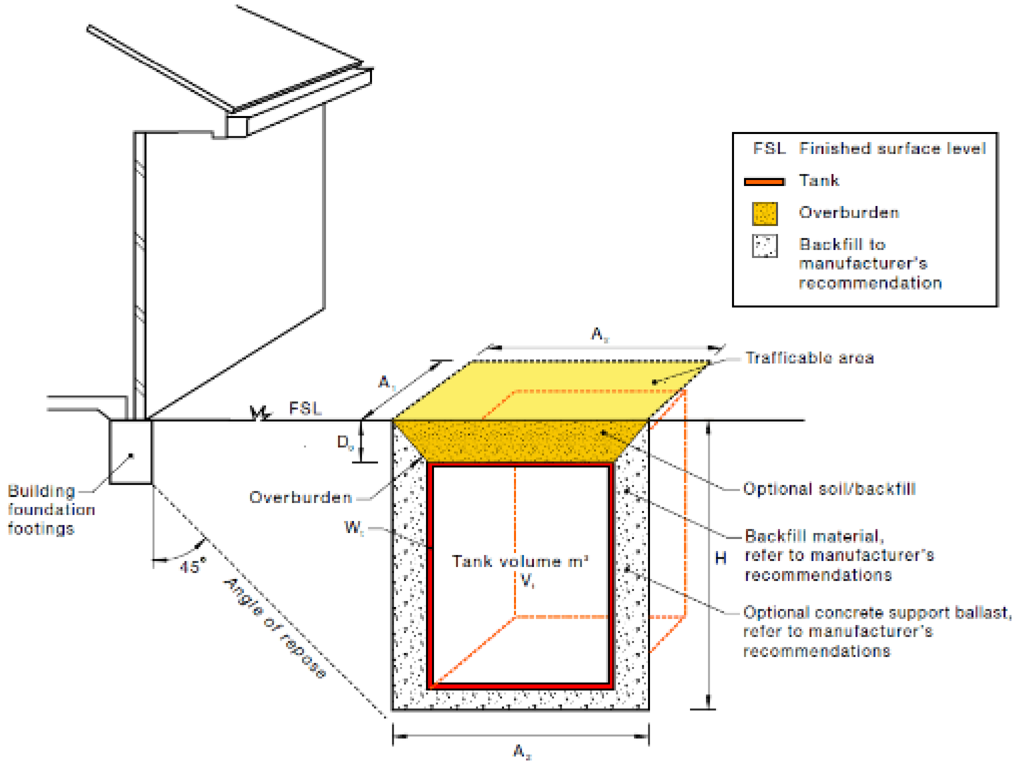

Figure 263: Typical design of the underground option

1.3.2. Used Equations

The following equations were used in pump design using an Excel Spread sheets.

$$Continuity\ equation: Q = A \times V$$

$$Reynolds\ number: Re = \frac{pVD}{\mu}$$

$$Moody\ equation:\ \lambda = 0.0055\left[1 + \left(20000\frac{k}{D} + \frac{10^6}{Re}\right)^{\frac{1}{3}}\right]$$

$$Total\ head:\ H_T = H_D + \frac{Q^2}{12.1}\left(\frac{\lambda_D L_D}{{D_D}^5}\right) + \frac{KQ^2}{12.1{D_D}^4}$$

Client	Tonkin Consulting	Date:26/05/2015
Project	North Terrace Drainage System	Sheet no: 93-101
Subject	Bioretention system Design	By: Menguliu Feng
Reviewed By:	Anne Wickramaratne	Date: 18/05 2015
Approved By:	Eriny Abdelraouf	Date: 22/05/2015

1.4. Bioretention Basin 1 Design

1.4.1. Calculation summary

The table below shows the results of the design calculation.

Table 95: Bioretention basin 1 calculation summaries Appendix C – Bioretention basin 1 design

Bioretention basin 1 Calculation task	Outcome	Unit
1 Identify design criteria		
Conveyance flow standard ARI	5	year
Area of Bioretention	70	m^2
Max extended retention depth	200	mm
Filter media type	36	mm/hr
2 Catchment characteristics		
Road area	5012	m^2
Footpath area	2148	m^2
Fraction impervious		
Road and footpath	0.9	
3 Estimate design flow rates		
Time of concentration	5	min
Identify rainfall intensities		
Station used for IFD data	Kent Town, Adelaide	
100 yr. ARI	182	mm/hr
5 yr. ARI	83.6	mm/hr
Peak design flows		
Calculation task	Outcome	Unit
Q_5	0.142	m^3/s

Q_{100}	0.391	m^3/s
$Q_{infiltration}$	0.00084	m^3/s
4 Perforated collection pipe capacity		
Pipe diameter	100	mm
Number of pipes	1	
Pipe capacity	0.00267	m^3/s
Capacity of perforations	0.00286	m^3/s
Soil media infiltration capacity	0.00084	m^3/s
Check pipe capacity > Soil capacity	YES	
5 Inlet opening width		
Width of opening for inflows	1.25	m
Coarse sediment removal	Forebay	
Coarse sediment removal area	5	m^2
Coarse sediment removal depth	0.5	m
Coarse sediment clean-out frequency	1	year
6 Velocities over vegetation		
Velocity for 5 yr. flow (<0.5 m/s)	0.355	m/s
Velocity for 100 yr. flow (<1 m/s)	0.977	m/s
7 over flow system		
System to convey minor floods	Grated pit 900x900	
8 Surrounding soil check		
Native soil hydraulic conductivities	180	mm/hr
Filter media	36-180	mm/hr
More than 10 times higher than soil?	No (Liner is required)	
9 Filter media specification		
Filtration media	Sandy clay loam	
Transition layer	Coarse sand	
Drainage layer	Fine gravel	
10 Plant selection	Carex Appressa	

1.4.2. Bioretention basin 1- MUSIC output

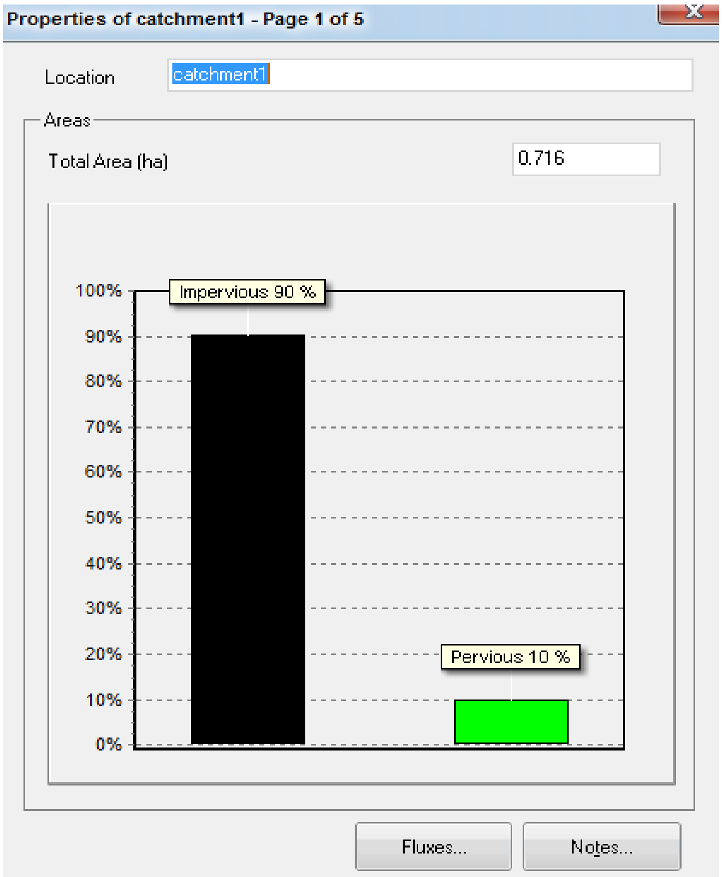

Figure 264: Catchment properties

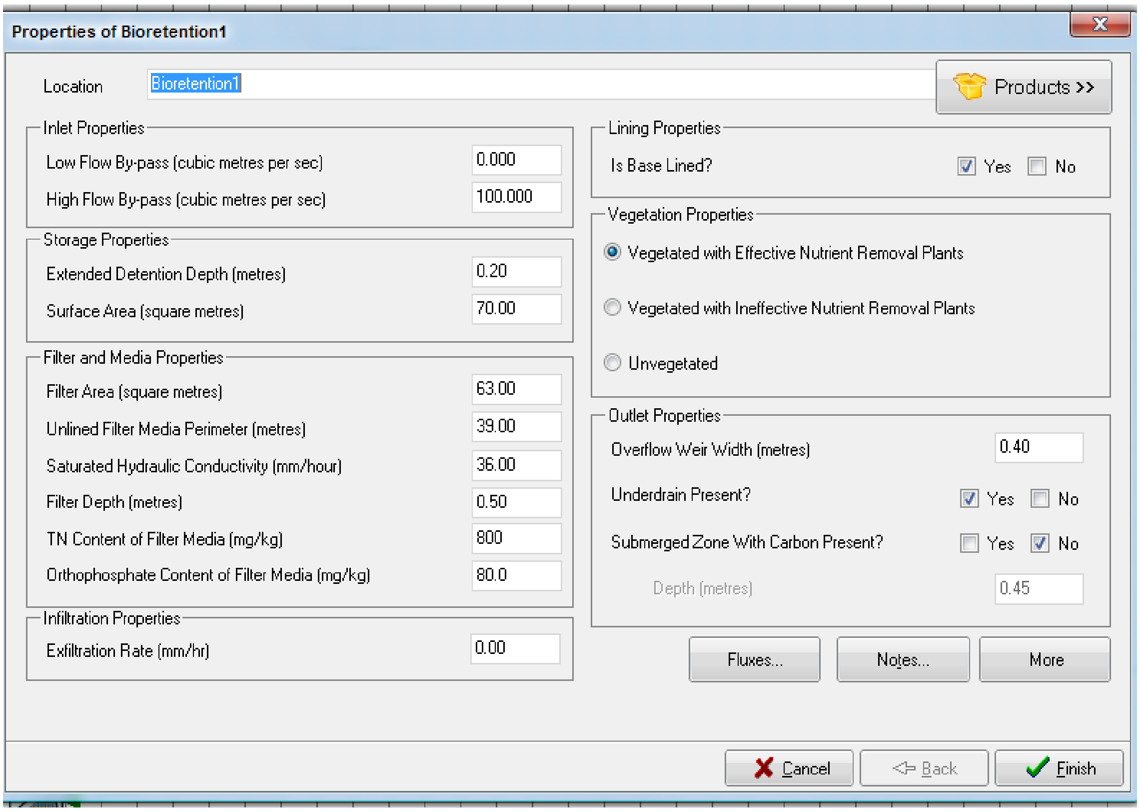

Figure 265: Bioretention basin1 properties

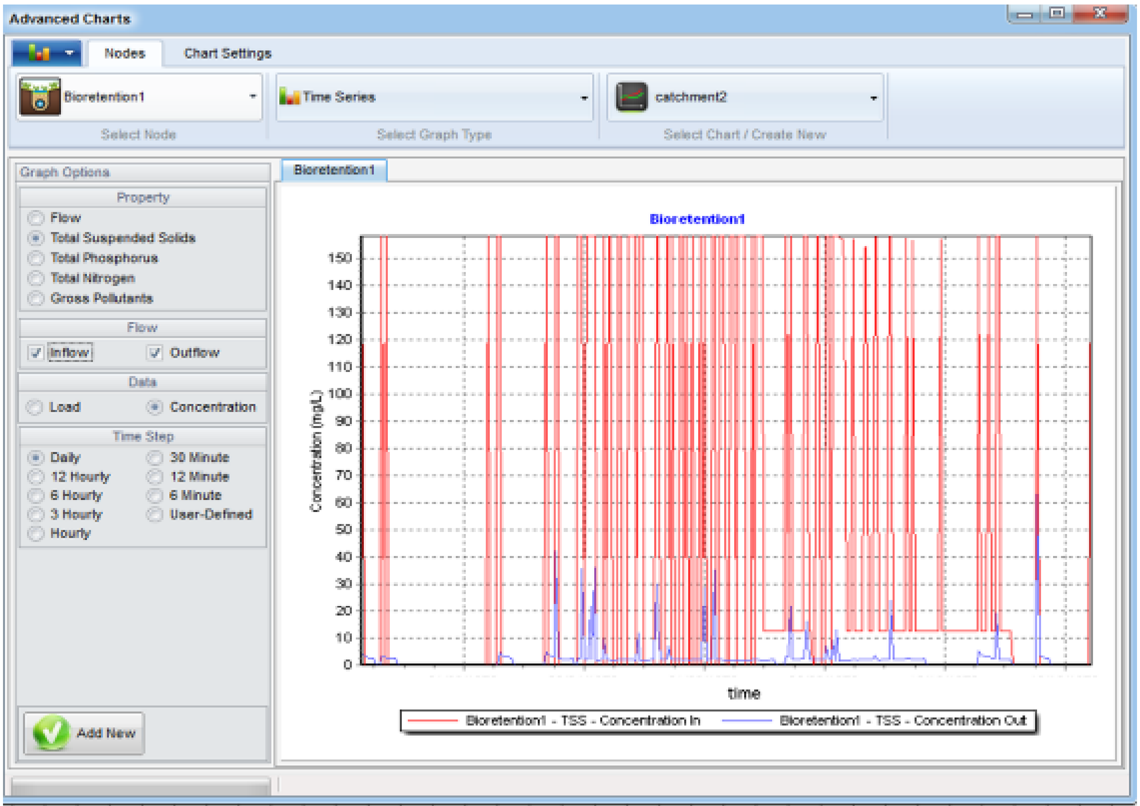

Figure 266: TSS concentration vs. time (Daily)

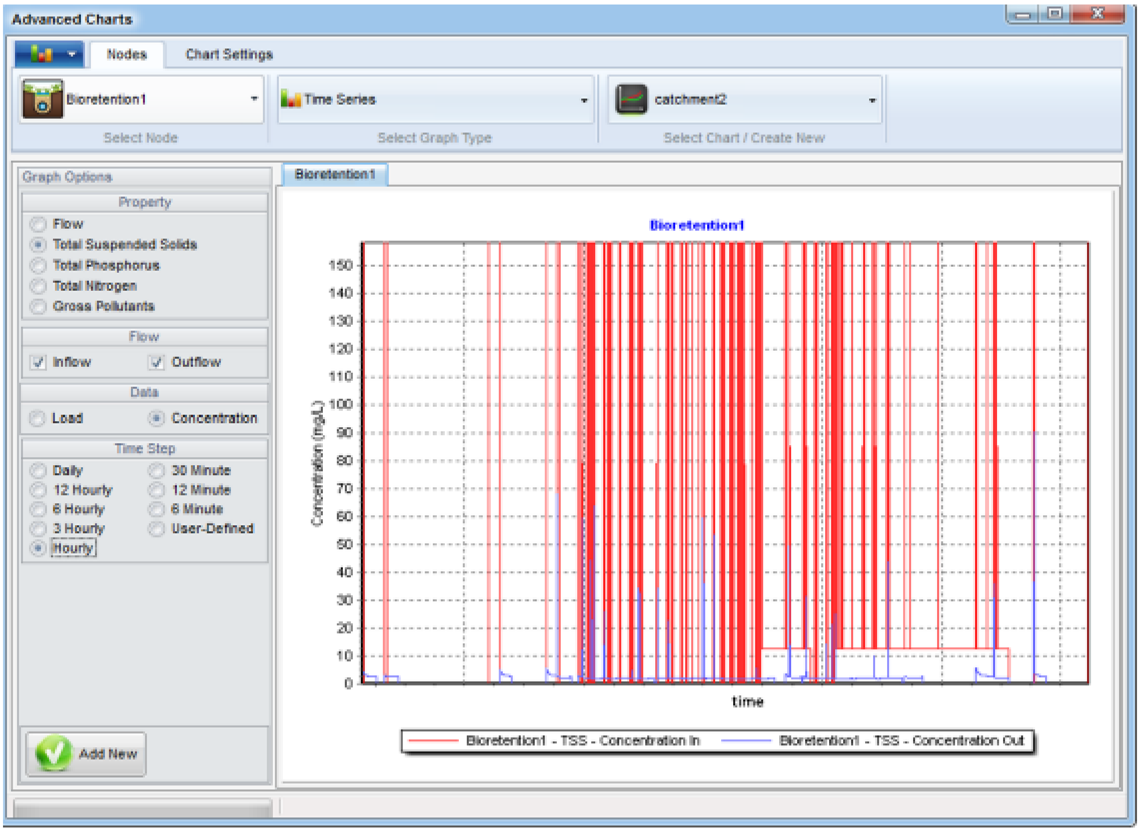

Figure 267: TSS concentration vs. time (Hourly)

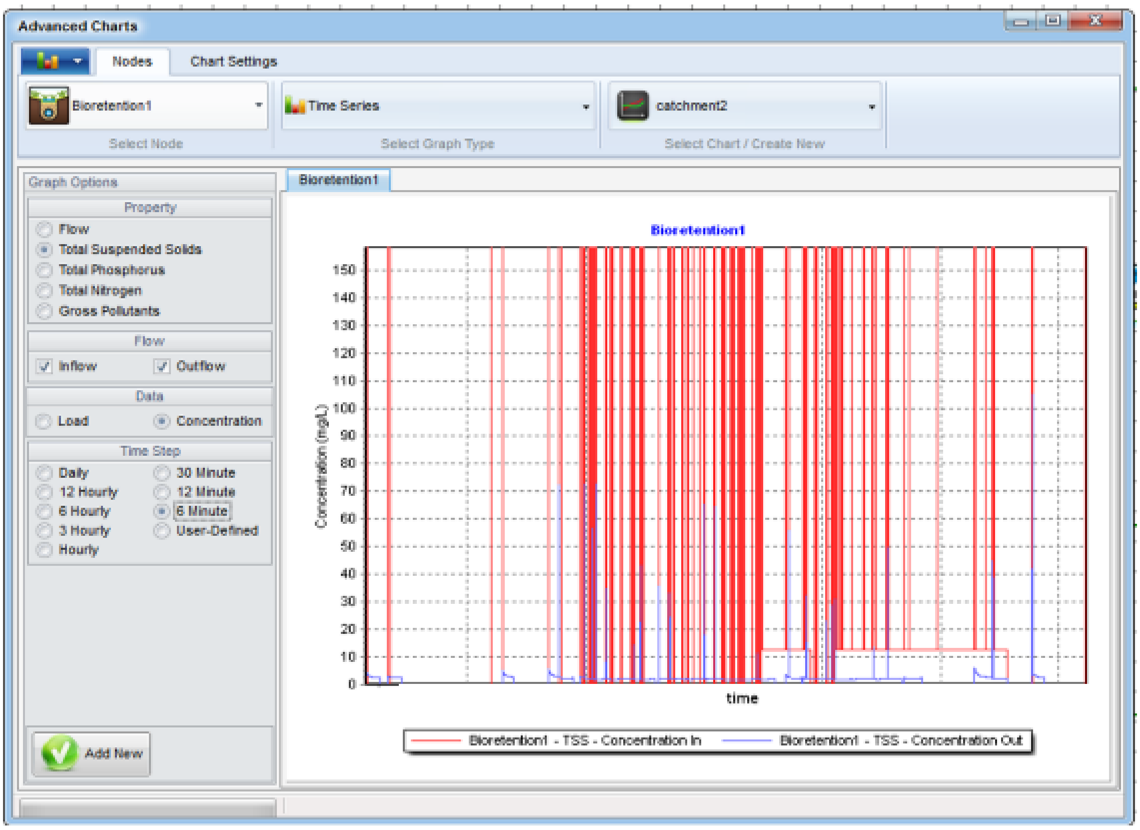

Figure 268: TSS concentration vs. time (6 minute)

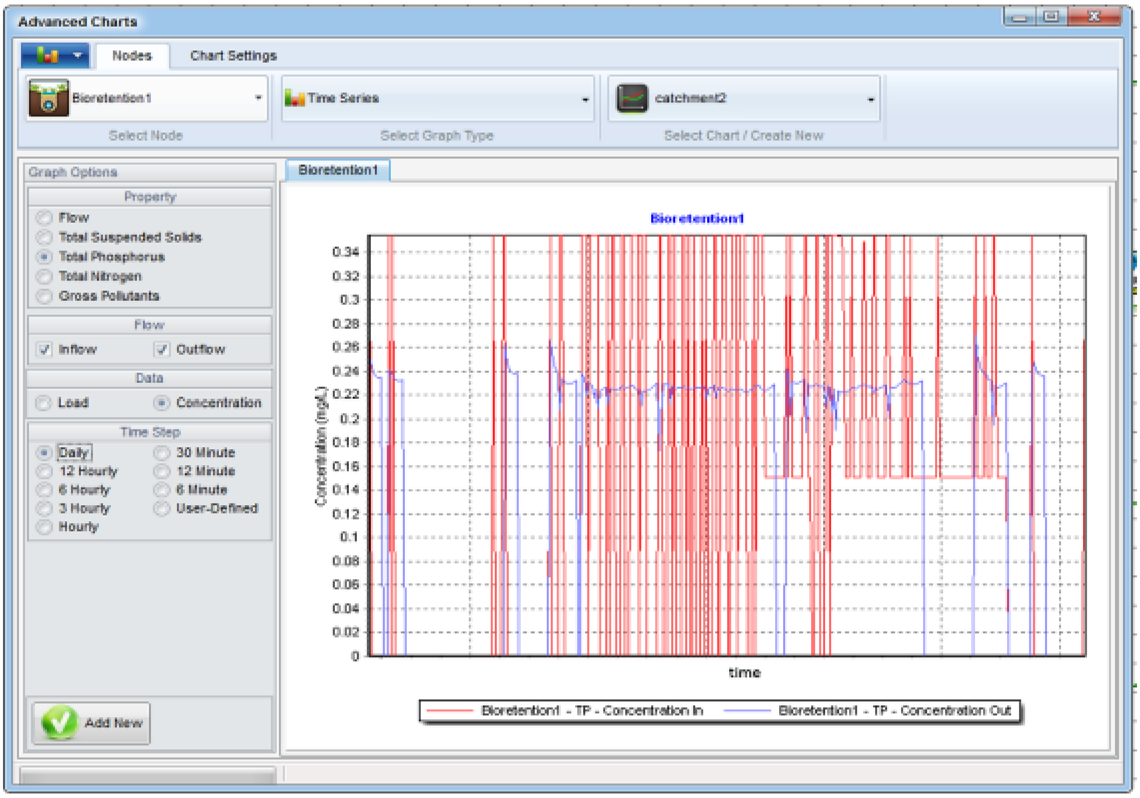

Figure 269: TP concentration vs. time (Daily)

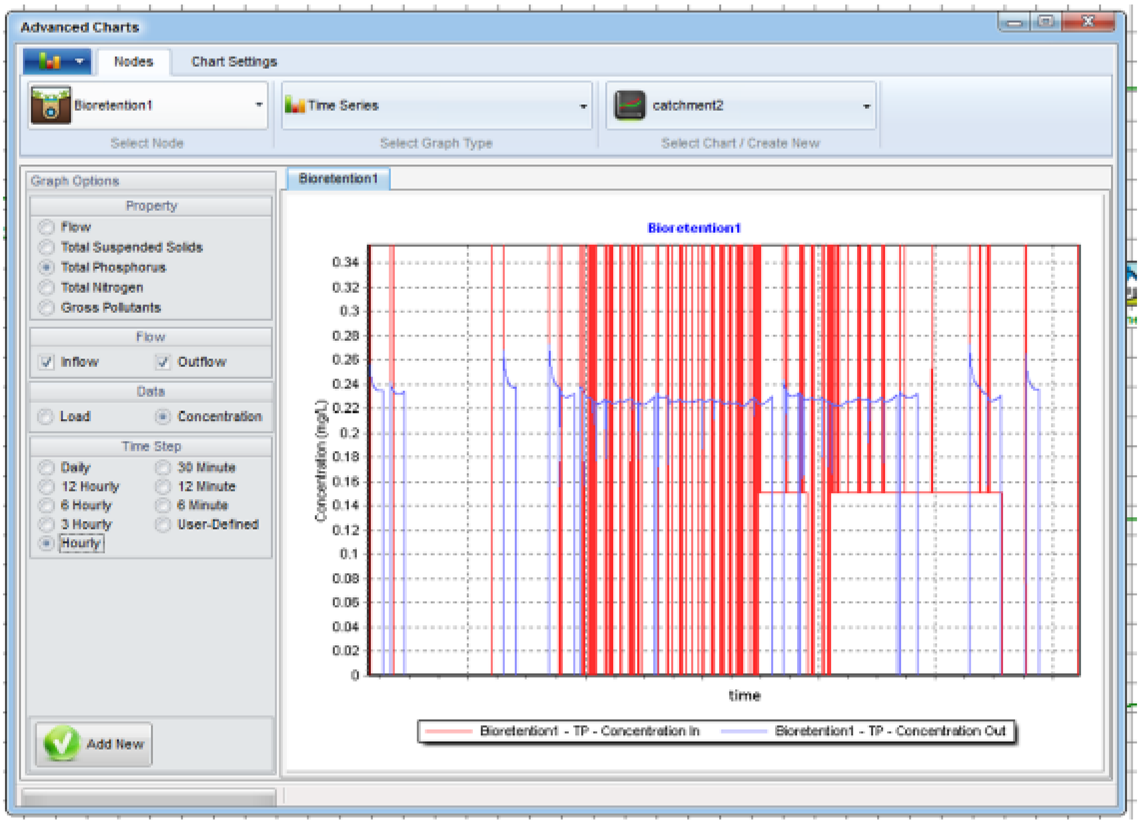

Figure 270: TP concentration vs. time (hourly)

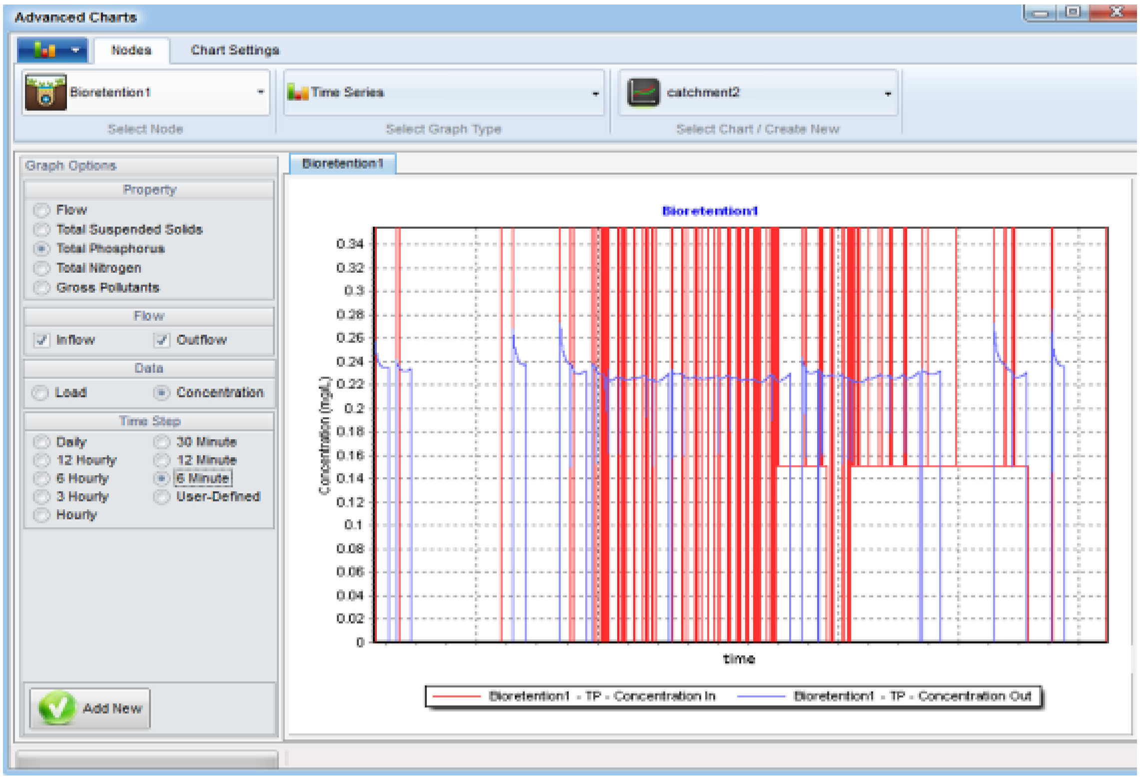

Figure 271: TP concentration vs. time (6 minute)

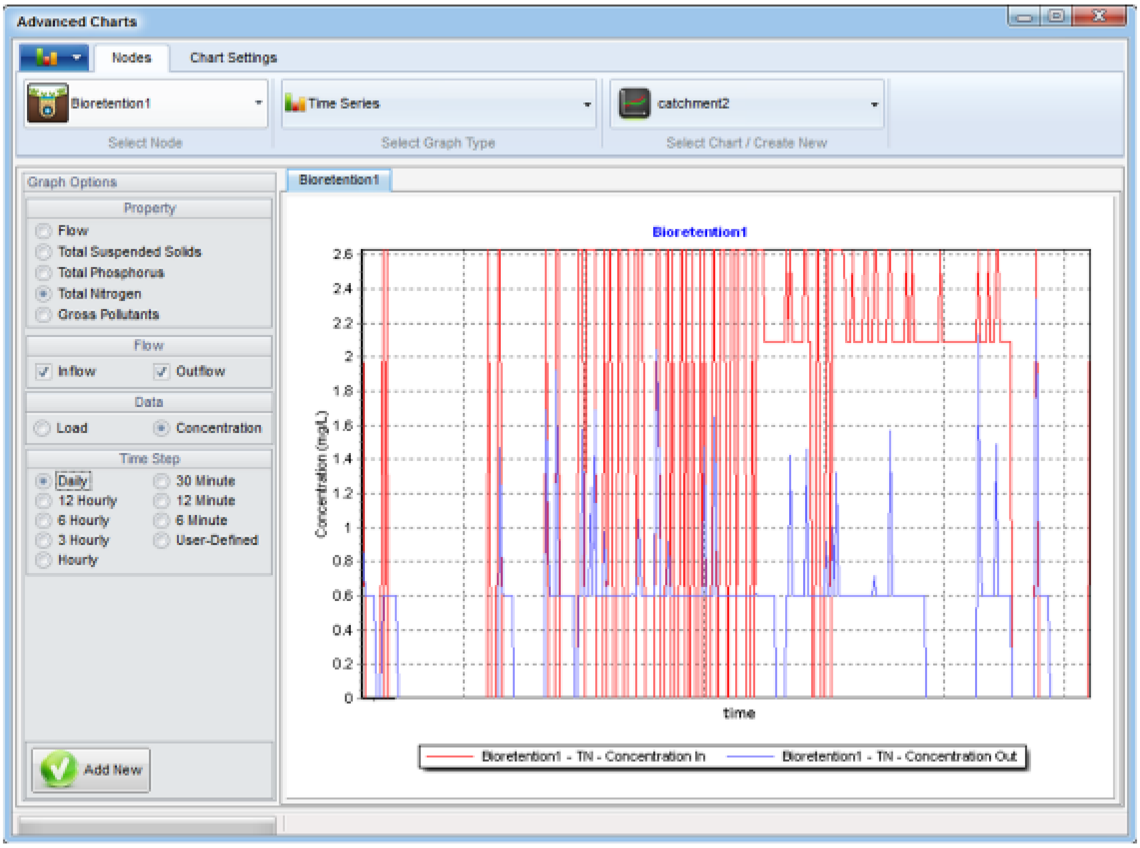

Figure 272: TN concentration vs. time (Daily)

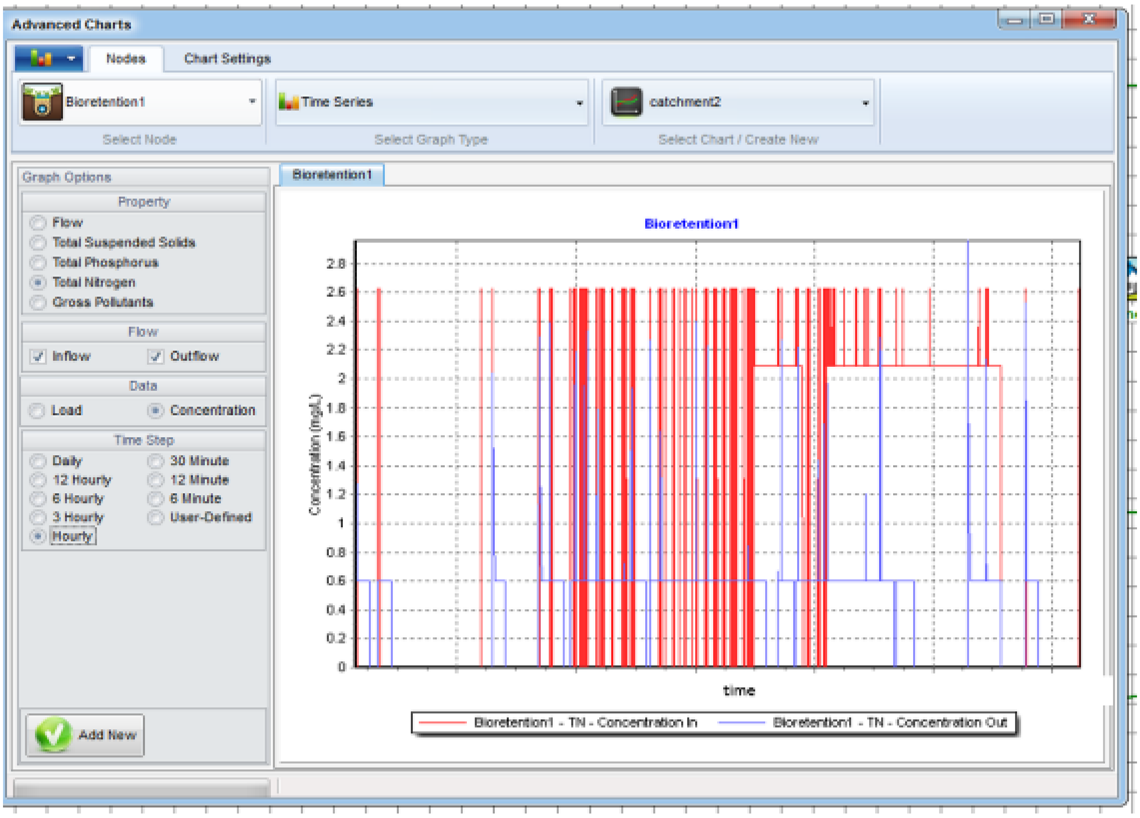

Figure 273: TN concentration vs. time (Hourly)

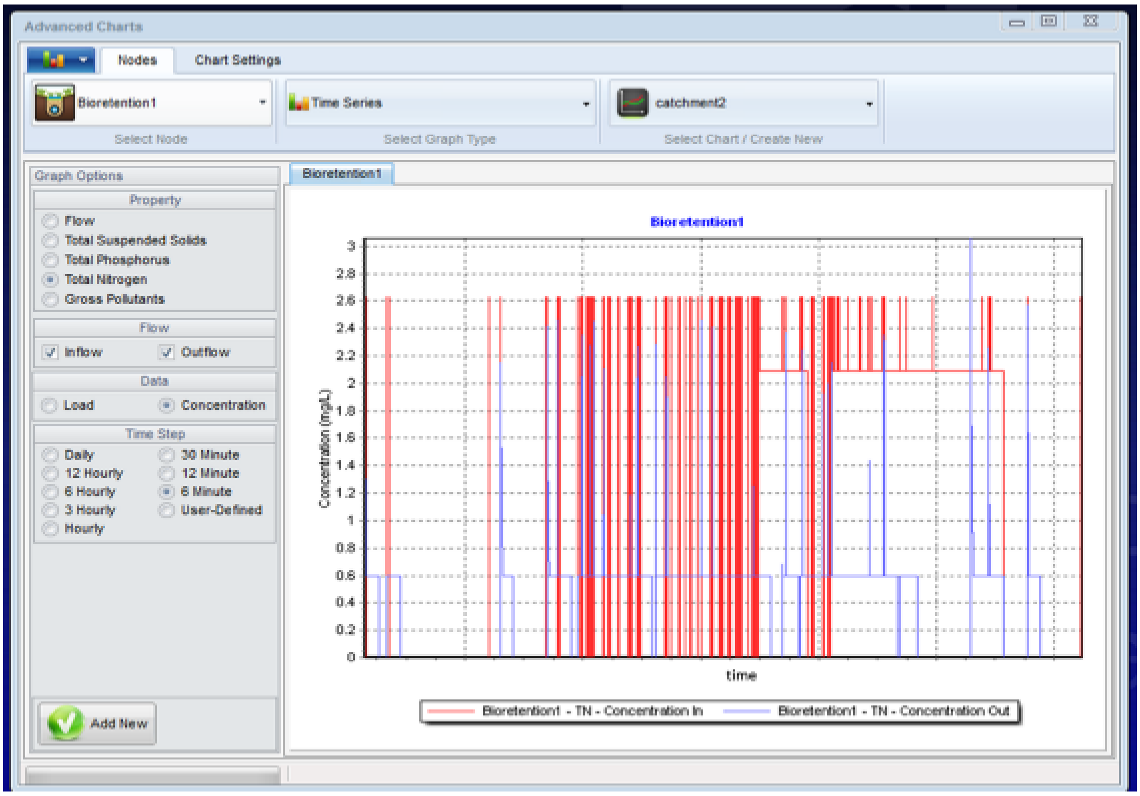

Figure 274: TN concentration vs. time (6 minute)

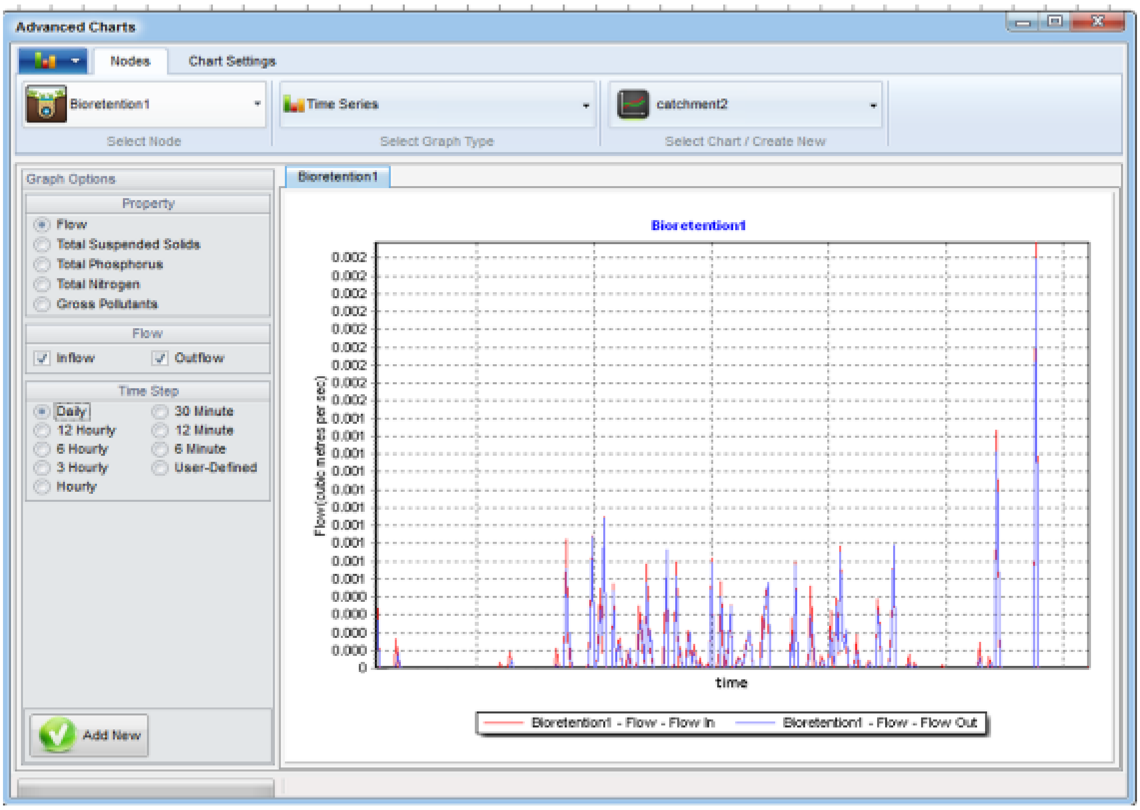

Figure 275: Flow vs. time (Daily)

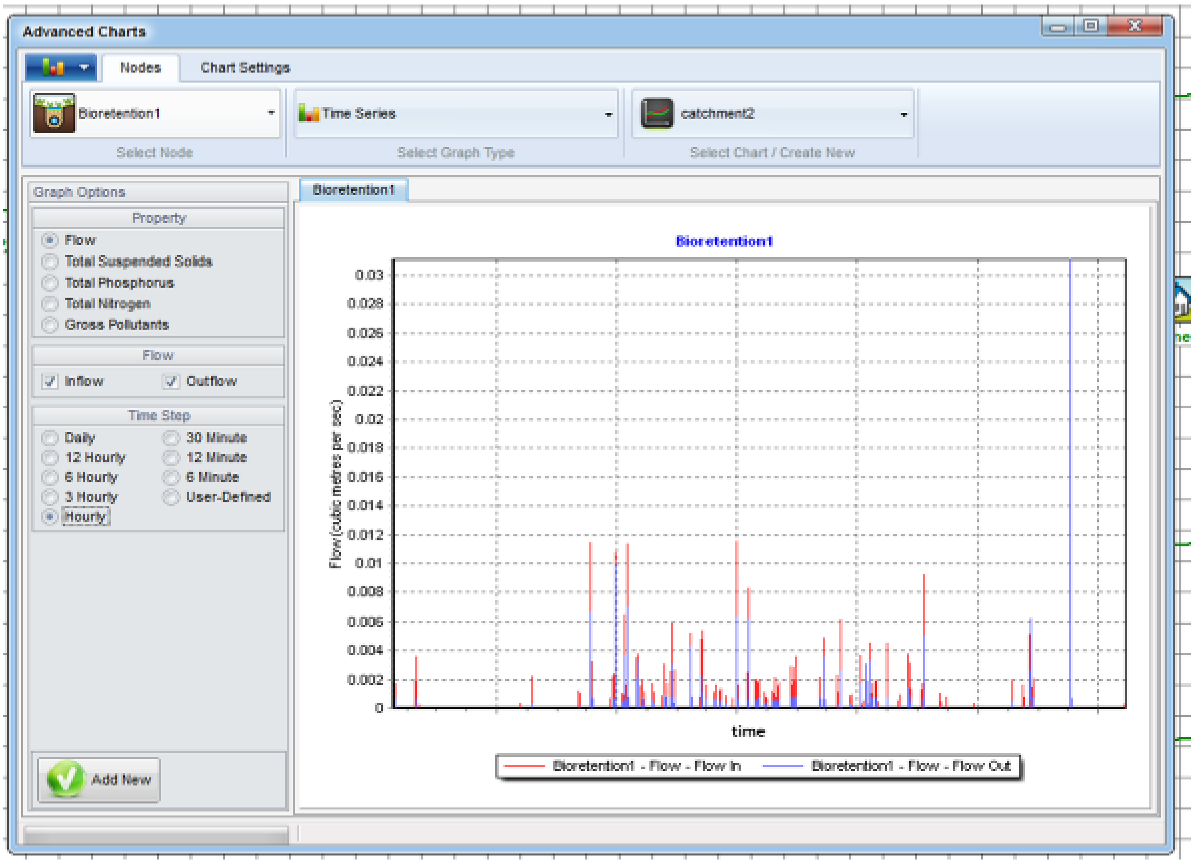

Figure 276: Flow vs. time (Hourly)

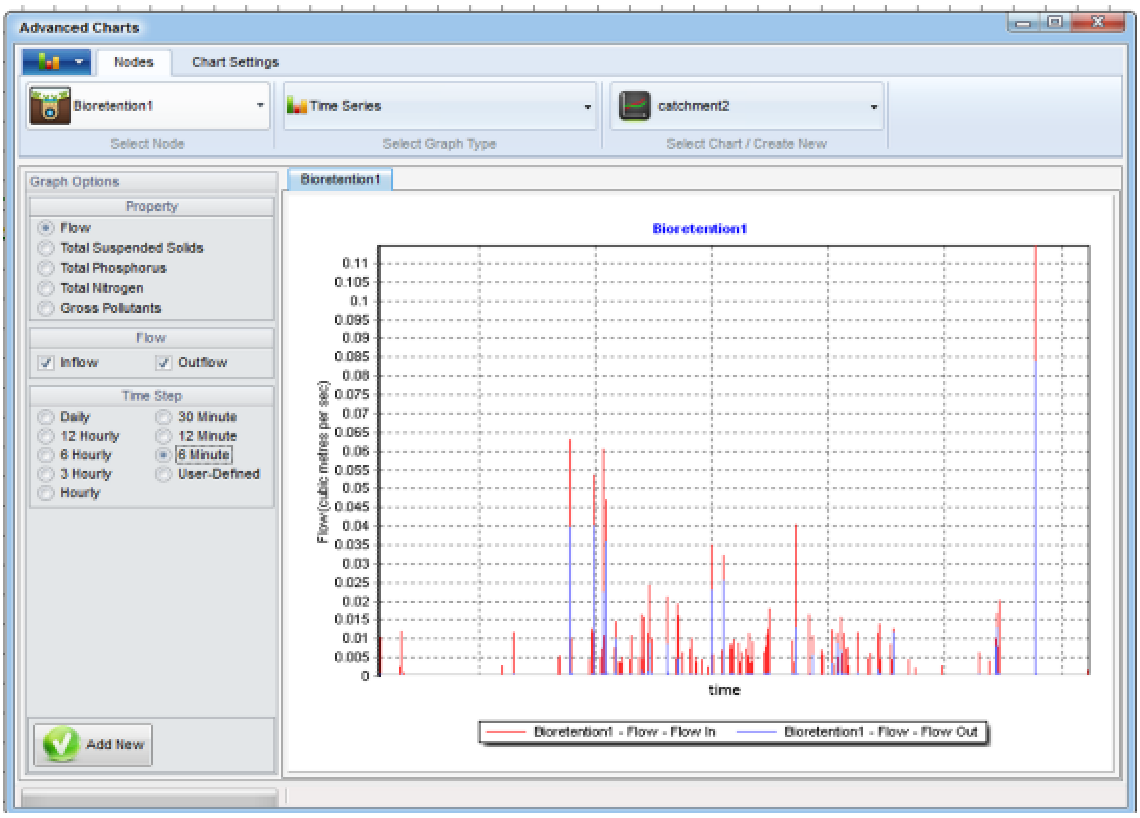

Figure 277: Flow vs. time (6 Minute)

1.5. Bioretention Basin 2 Design Calculations (Optional design)

1.5.1. Concept design

The Bioretention basins are designed according to the following criteria:

- Bioretention basin 2 area of $105\ m^2$;
- Maximum width of the Bioretention basin is to be $3\ m$;
- Extended detention depth is$200\ mm$. $(100\ m - \ 300m\ m)$;
- Filter media shall be a sandy clay loam;
- Coarse sediment forebay is needed as no space is allowed for grass buffer.

1.5.2. Site characteristics

The site characteristics of the Bioretention basin are:

- Urban, paved roads and footpaths;
- Typical overland flow slope of 1%;
- Red brown earth and alluvia soil (sandy loam to light clay loam) are dominated in design project;
- Catchment area: Roads, 0.2968ha; footpath, 0.1272 ha;
- Fraction impervious is: road and footpath, 0.9.

1.5.3. Confirm size for treatment

The MUSIC model is used to check if the concept Bioretention area is able to reach treatment target. More input and output information is provided in 1.5.11.

The contributing area for bioretention area is 0.424 ha. By adjusting filter material depth and extended detention depth the results show $105\ m^2$, 35m x 3m, rectengular bioretention basin with 0.3m sandy clay filter material and 0.2m pounding depth. The reduction can reach 96.7%, 45.8% and 74.7% for TSS, TP and TN.

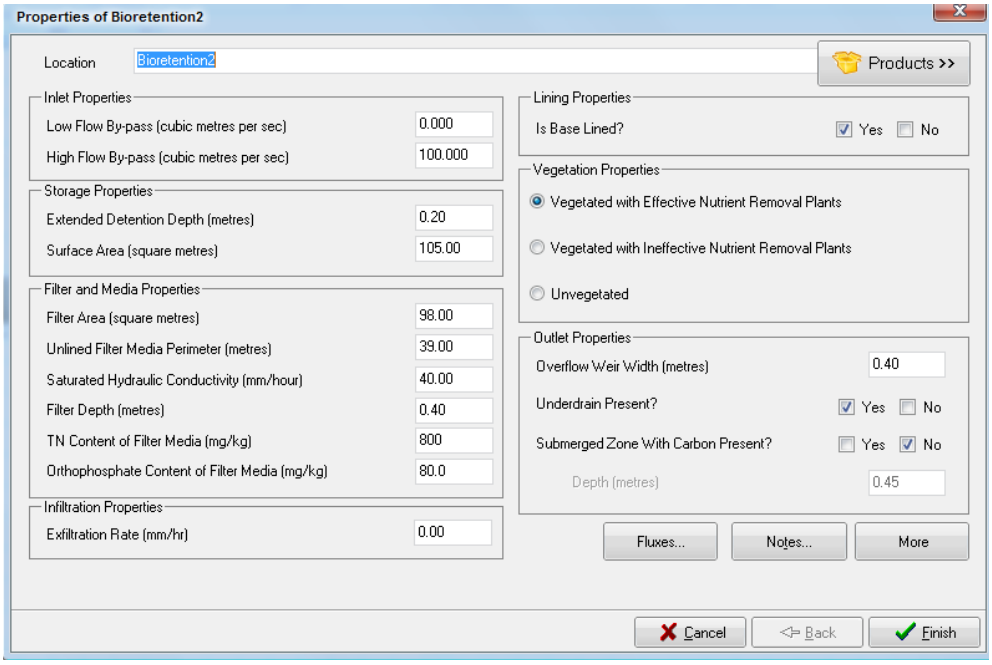

Figure 278: MUSIC modelling input for Bioretention basin 1

Mean Annual Loads - Bioretention2

	Inflow	Outflow	% Reduction
Flow (ML/yr)	1.15	0.948	17.3
Peak Flow (m3/s)	67.9E-3	47.0E-3	30.9
Total Suspended Solids (kg/yr)	181	6.08	96.7
Total Phosphorus (kg/yr)	0.406	0.220	45.8
Total Nitrogen (kg/yr)	3.02	0.762	74.7
Gross Pollutants (kg/yr)	57.4	0.00	100.0

Figure 279: Mean annual loads output for Bioretention basin 2

The modelling indicates that all reduction of pollutants reached design water treatment objectives.

1.5.4. Estimating design flows

1.5.4.1. Design Parameters

- The minor and major storm requirements will be same as for the Bioretention system 1 design.
- Travel time will be 5min as for the standard paved areas.
- The rainfall intensity (I) for 1 in 5 years is 83.6 mm/h and for 1 in 100 years is 182 mm/h.
- Runoff coefficients for 1 in 5 years and 1 in 100 years is shown below.

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

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

1.5.4.2. Peak design flows

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

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

$$A = 0.420$$

$$Q_5 = 0.084 m^3/s$$

$$Q_{100} = 0.232 m^3/s$$

1.5.5. Maximum infiltration rate

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

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

$$Q_{max} = KLW_{base} \frac{h_{max} + d}{d} = 0.0014 m^3/s$$

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

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

L= the length of the bioretention zone (m)

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

1.5.6. Inlet details

1.5.6.1. Coarse sediment forebay

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

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

$$V_s = A_c R L_0 F_c$$

Where $A_c = Area\ of\ catchment$

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

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

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

Required volume of forebay sediment storage $V_{s2} = 0.271\ m^3$,

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

$$A_{s2} = 1.2\ m^2$$

Check:

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

Where $n \ = \ 1.35$ (using λ = 0.26)

$V_s = setting\ velocity\ 0.1$

When $A_{s2} = 3\ m^2, R_2 = 82.5\% > R = 80\%$

Hence, design $A_{s2} = 3\ m^2$, size of coarse sediment forebay: L$\times W = 3m \times 1m, d \ = \ 0.5\ m$

1.5.6.2. Flow width at entry

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

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

$Q_5 = 0.084 m^3/s$, Depth of flow = 0.276m

Width of flow = 3.5m, velocity = 0.383 m/s

1.5.6.3. Inlet opening at entry

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

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

Where, C=1.7 (weir flow coefficient)

$$H = flow\ depth\ from\ previous\ section.$$

$Q_5 = 0.084 m^3/s$, Depth of flow = 0.276m use H=0.28m

Therefore, L = $0.342m$

Hence, a 0.35m-wide opening are adopted for bioretention basin 2 at the inlet under footpath.

1.5.7. Vegetation scour velocity check

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

Area =$width \times detention\ depth$ =3 $\times 0.2 = 0.6\, m^2$

$$V_{5\,average} = \frac{Q_5}{A} = 0140 m/s$$

$$V_{100\,average} = \frac{Q_{100}}{A} = 0.386 m/s$$

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

1.5.8. Sizing of perforated collection pipes

1.5.8.1. Perforations inflow check

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

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

The following are the characteristics of selected slotted pipe

Clear opening = $2100\ mm^2$

Slot width = 1.5 m

Slot Length = 7.5 m

No. of rows, 6

Pipe diameter = 100 mm

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

$$Q_{Pref} = BC_d A\sqrt{2gh}$$

Where C_d = orifice discharge coefficient (0.6)

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

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

A_0 = orifice area (m^2)

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

H=0.2+0.3+0.1+0.2+0.05=0.85m

$Q_{Pref} = 0.00257\ m^3 > Q_{infiltration} = 0.0014 m^3/s$,

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

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

1.5.8.2. Perforated pipe capacity

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

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

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

$k = 0.007m$ (Assumption)

$v = 10^{-6}$

$pipe\ slop = 0.5\%$

$g = 9.81$

The capacity of this pipe is $Q_{pipe} = 0.00267m^3/s$, which exceeds the maximum infiltration rate of bioretention basin 2. Hence, $1\phi 100$ pipe is adopted for perforated collection for the bioretention system.

1.5.8.3. Drainage layer hydraulic conductivity

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

1.5.8.4. Impervious liner requirement

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

1.5.9. High-flow route and bypass design

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

Free overfall conditions:

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

Where, C= weir flow coefficient (1.7)

H = head above weir crest $(0.1m)$

$$L = 1.705m$$

$$\frac{1}{4}L = 0.426\ m$$

Drowned outlet conditions:

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

Where C_d = orifice discharge coefficient (0.6)

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

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

A_0 = orifice area (m^2)

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

$$A_{02} = 0.082m^2$$

$$\sqrt{A_{0_2}} = 0.572\ m$$

Drowned outlet conditions dominate, therefore a 600mm × 600mm grated pits are adopted for both Bioretention basins.

1.5.10. Calculation summary

Table 96: Bioretention basin 2 calculation summary

Bioretention basin 2		
Calculation task	**Outcome**	**Unit**
1 Identify design criteria		
Conveyance flow standard ARI	5	year
Area of bioretention	105	m^2
Max extended retention depth	200	mm
Filter media type	36	mm/hr
2 Catchment characteristics		
Road area	2968	m^2
Footpath area	1272	m^2
Fraction impervious		
Road and footpath	0.9	
3 Estimate design flow rates		
Time of concentration	5	min
Identify rainfall intensities		
Station used for IFD data	Kent Town, Adelaide	
100 yr. ARI	182	mm/hr
5 yr. ARI	83.6	mm/hr
Peak design flows		
Q_5	0.084	m^3/s
Q_{100}	0.232	m^3/s
$Q_{infiltration}$	0.0014	m^3/s
4 Perforated collection pipe capacity		
Pipe diameter	100	mm
Number of pipes	1	
Pipe capacity	0.00267	m^3/s
Capacity of perforations	0.00257	m^3/s
Soil media infiltration capacity	0.0014	m^3/s
Check pipe capacity > Soil capacity	YES	
5 Inlet opening width		
Width of opening for inflows	0.35	m

Coarse sediment removal	Forebay	
Coarse sediment removal area	3	m^2
Coarse sediment removal depth	0.5	m
Coarse sediment clean-out frequency	1	year
7 over flow system		
System to convey minor floods	Grated pit 600x600	
8 Surrounding soil check		
Native soil hydraulic conductivities	180	mm/hr
Filter media	36-180	mm/hr
More than 10 times higher than soil?	No (Liner is required)	
9 Filter media specification		
Filtration media	Sandy clay loam	
Transition layer	Coarse sand	
Drainage layer	Fine gravel	
10 Plant selection	Carex Appressa	

1.5.11. Bioretention basin 2 - MUSIC output

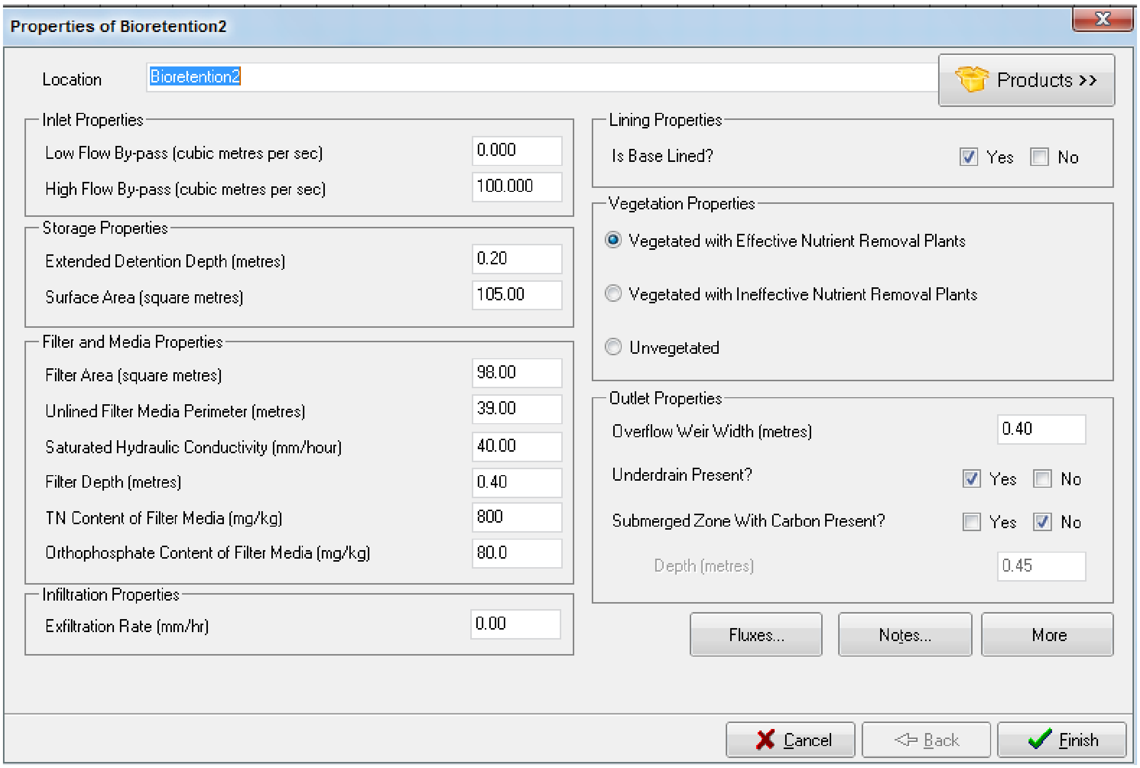

Figure 280: Bioretention basin1 properties

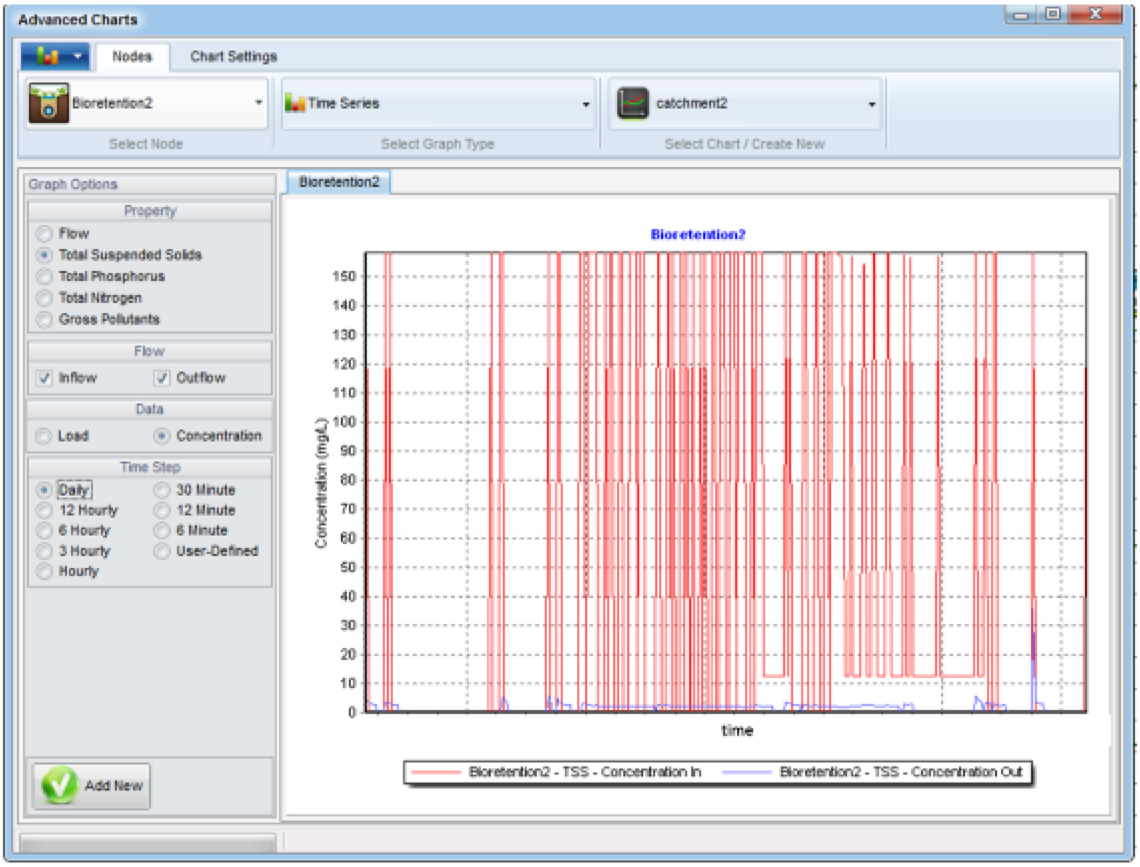

Figure 281: TSS concentration vs. time (Daily)

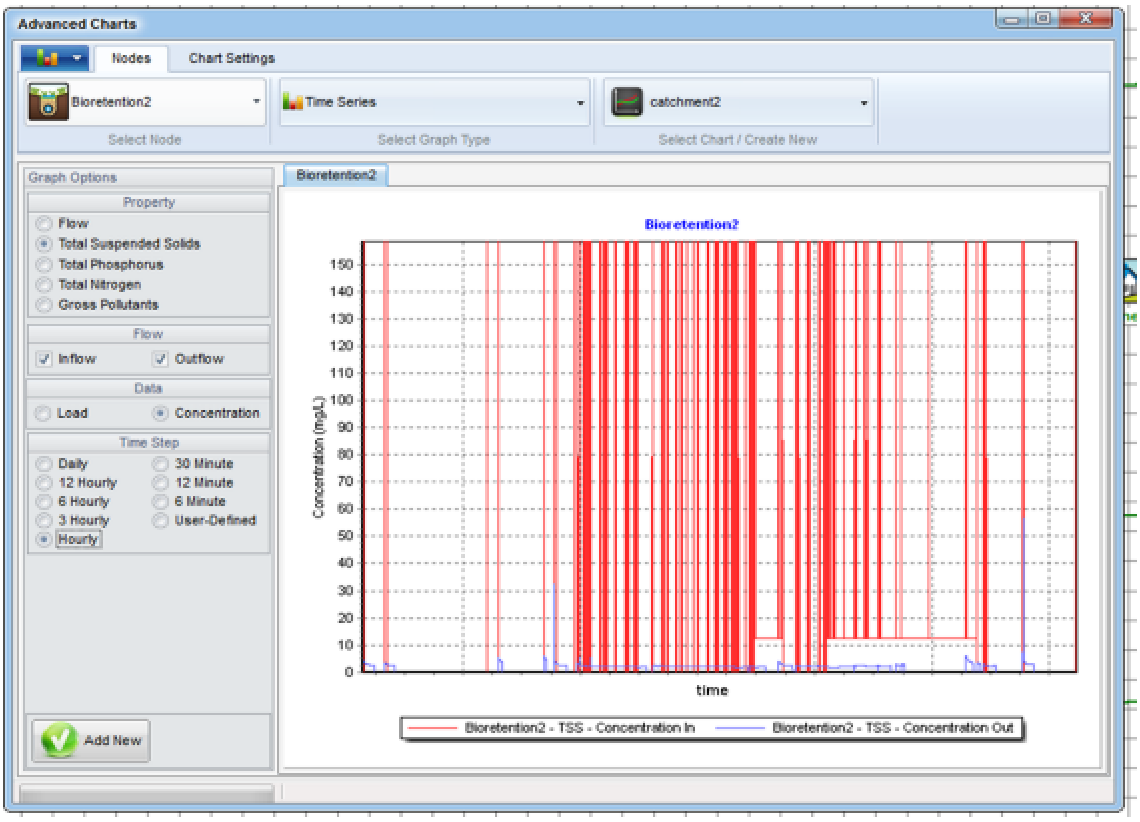

Figure 282:TSS concentration vs. time (Hourly)

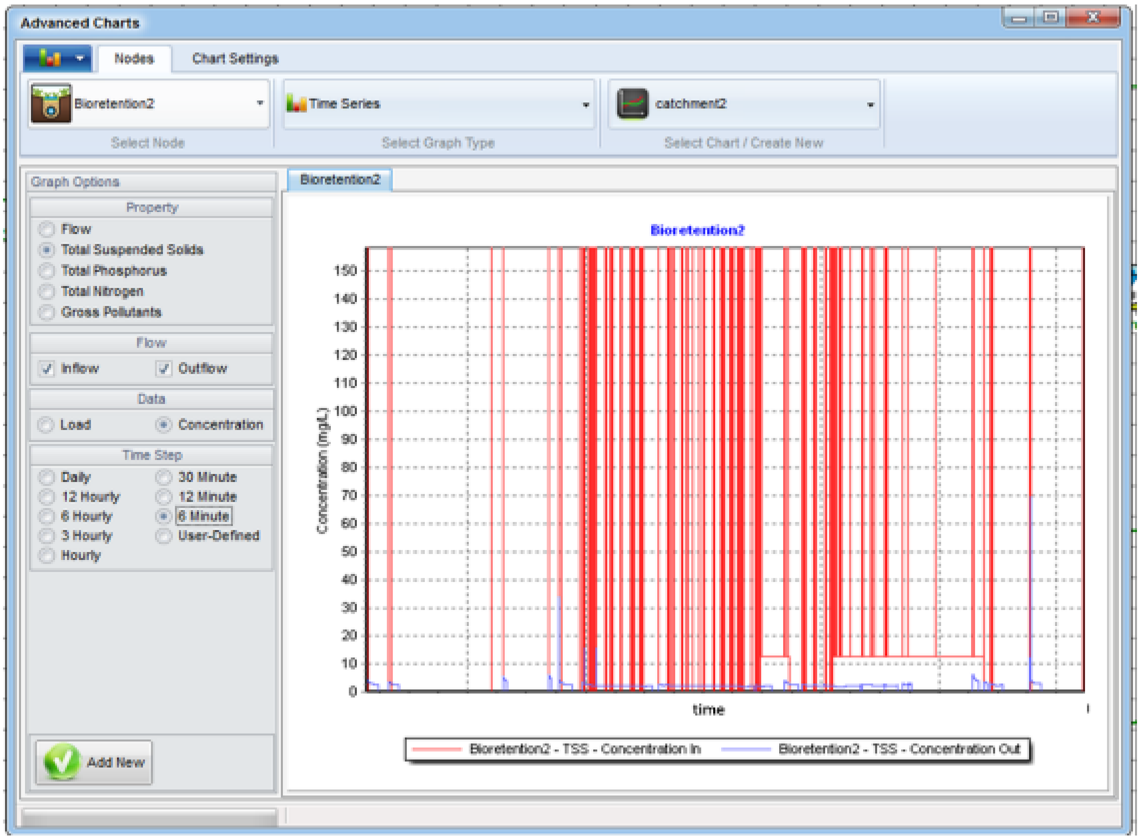

Figure 283:TSS concentration vs. time (6 minute)

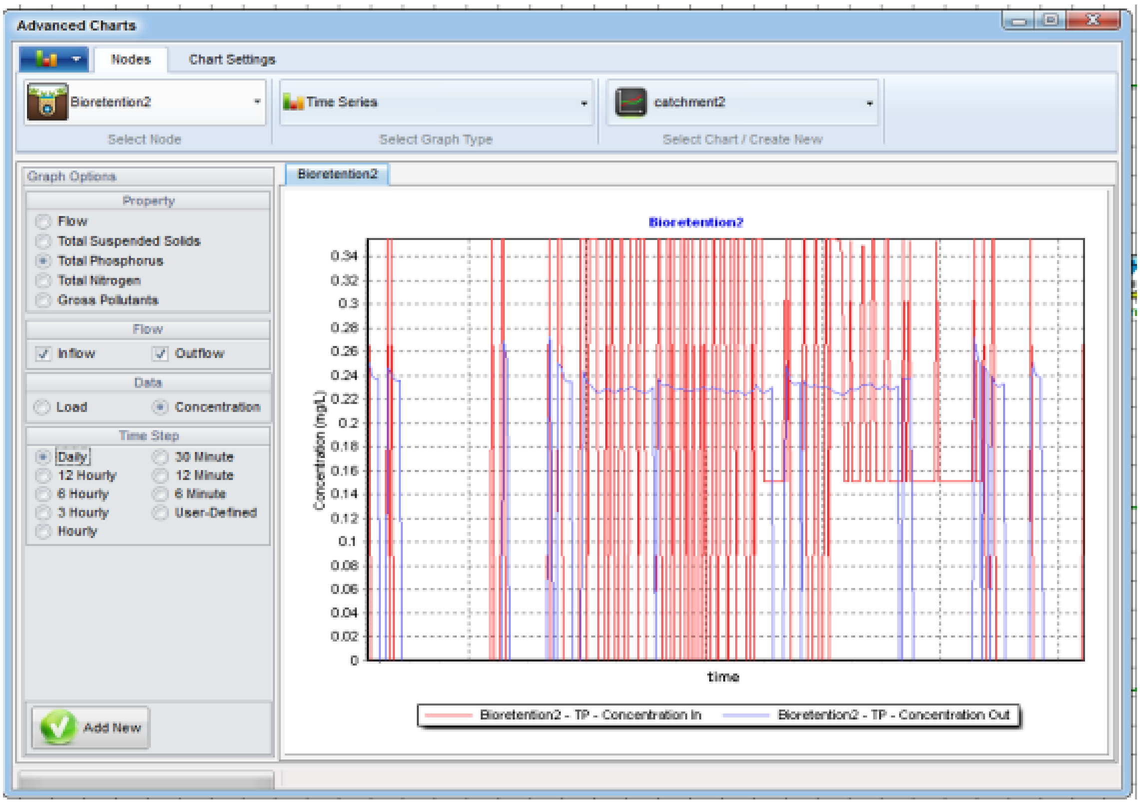

*Figure 284:*TP concentration vs. time (Daily)

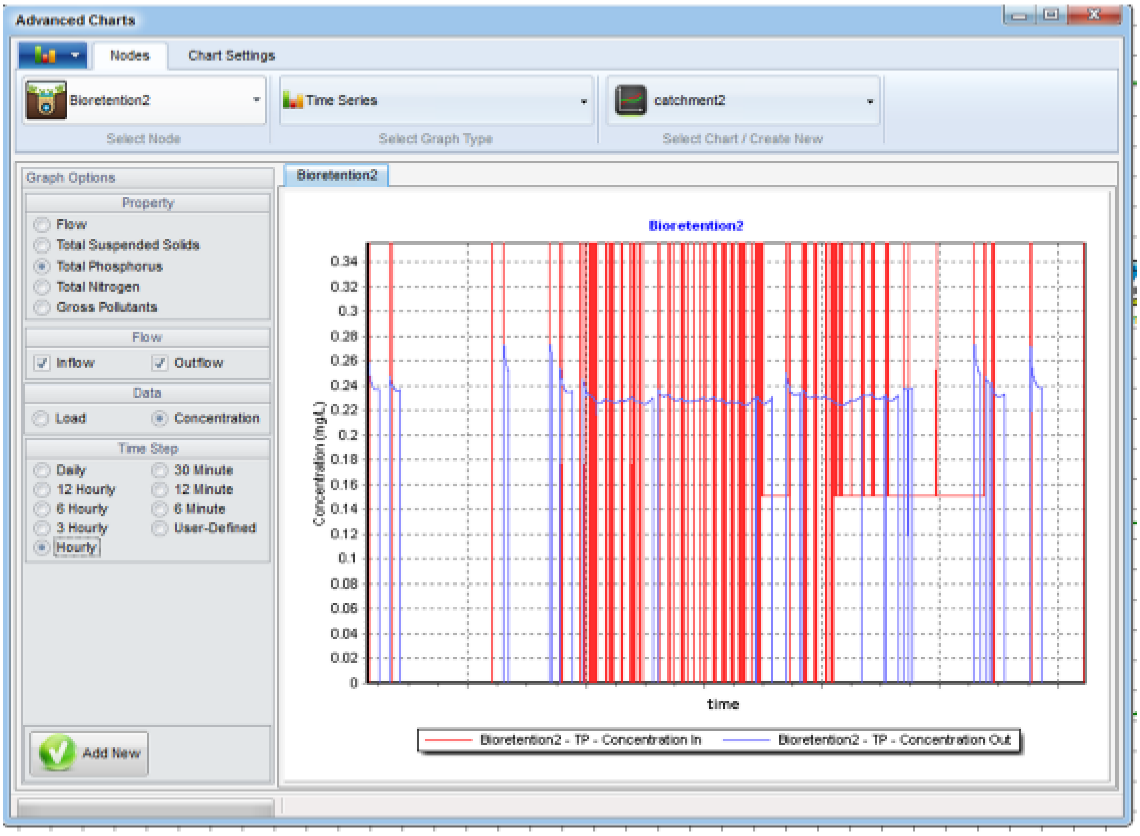

*Figure 285:*TP concentration vs. time (hourly)

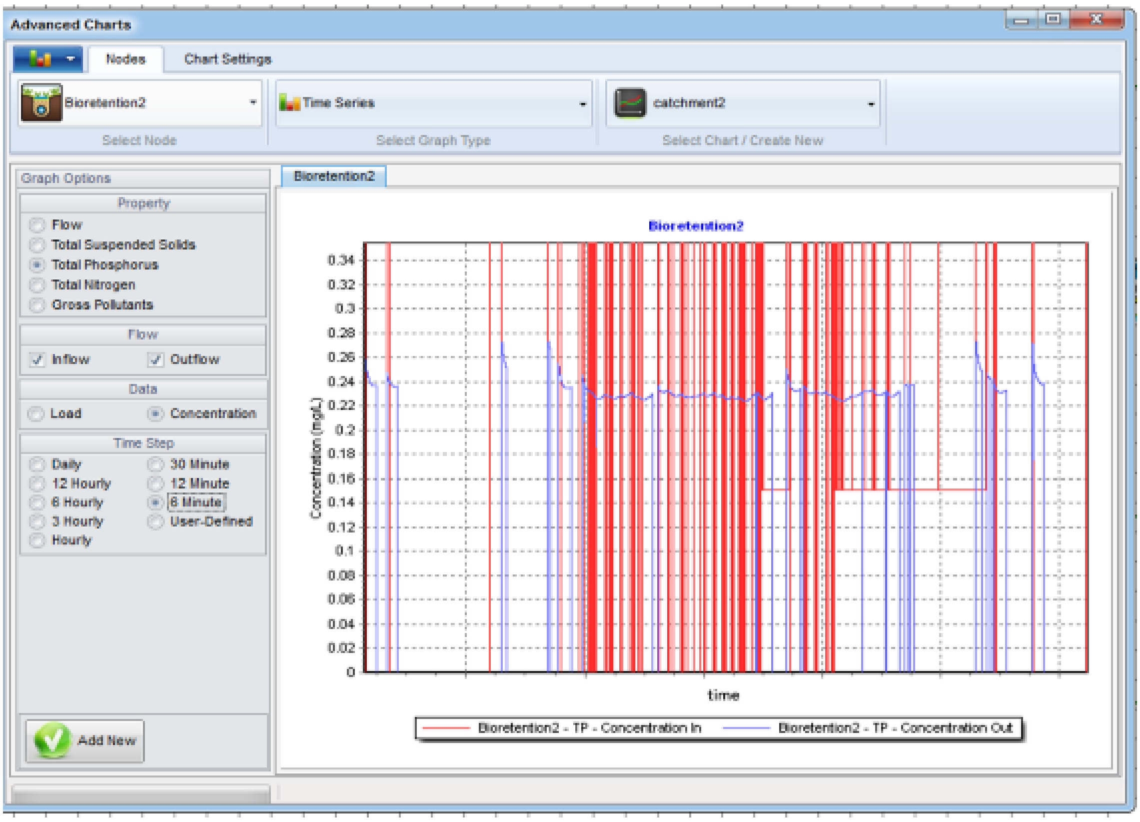

Figure 286: TP concentration vs. time (6 minute)

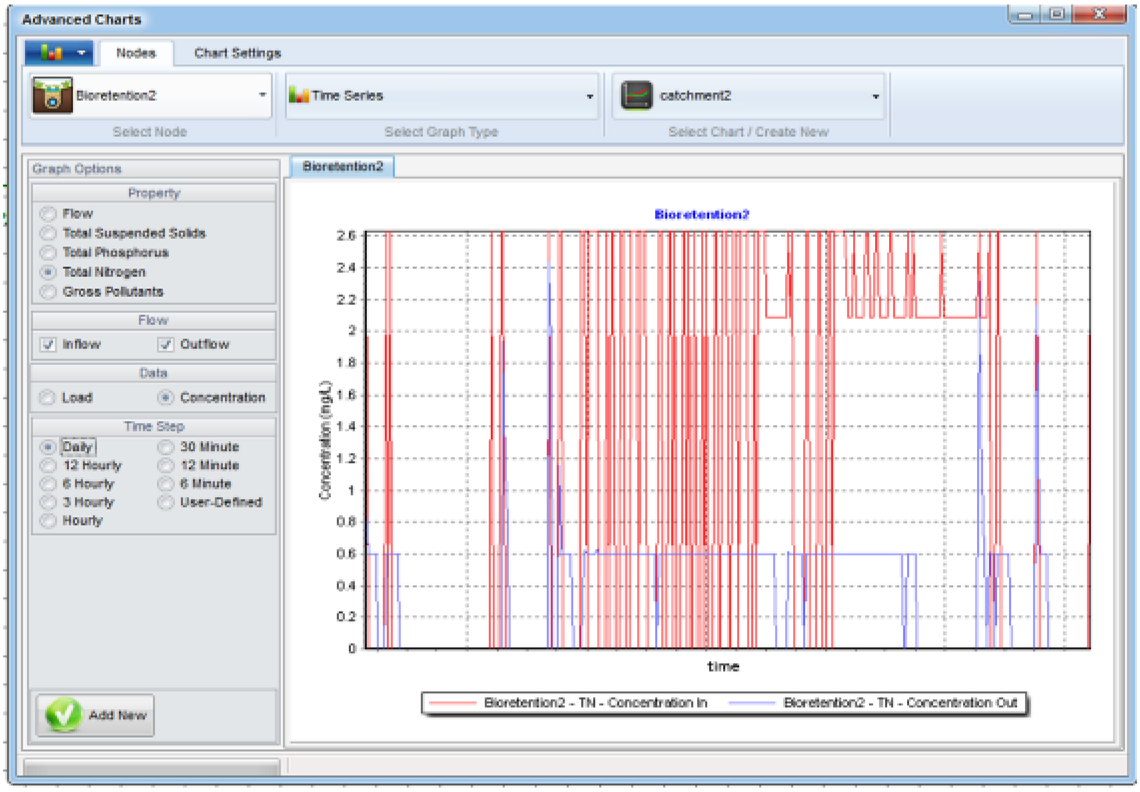

Figure 287: TN concentration vs. time (Daily)

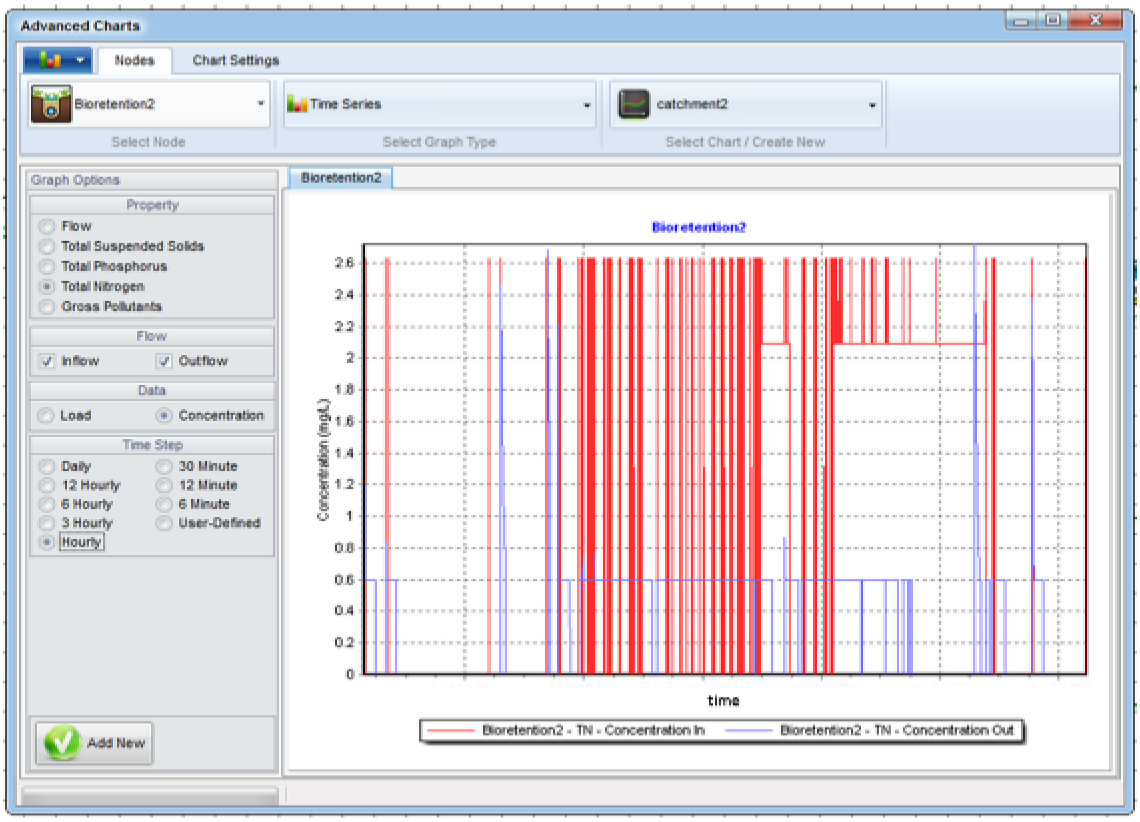

*Figure 288:*TN concentration vs. time (Hourly)

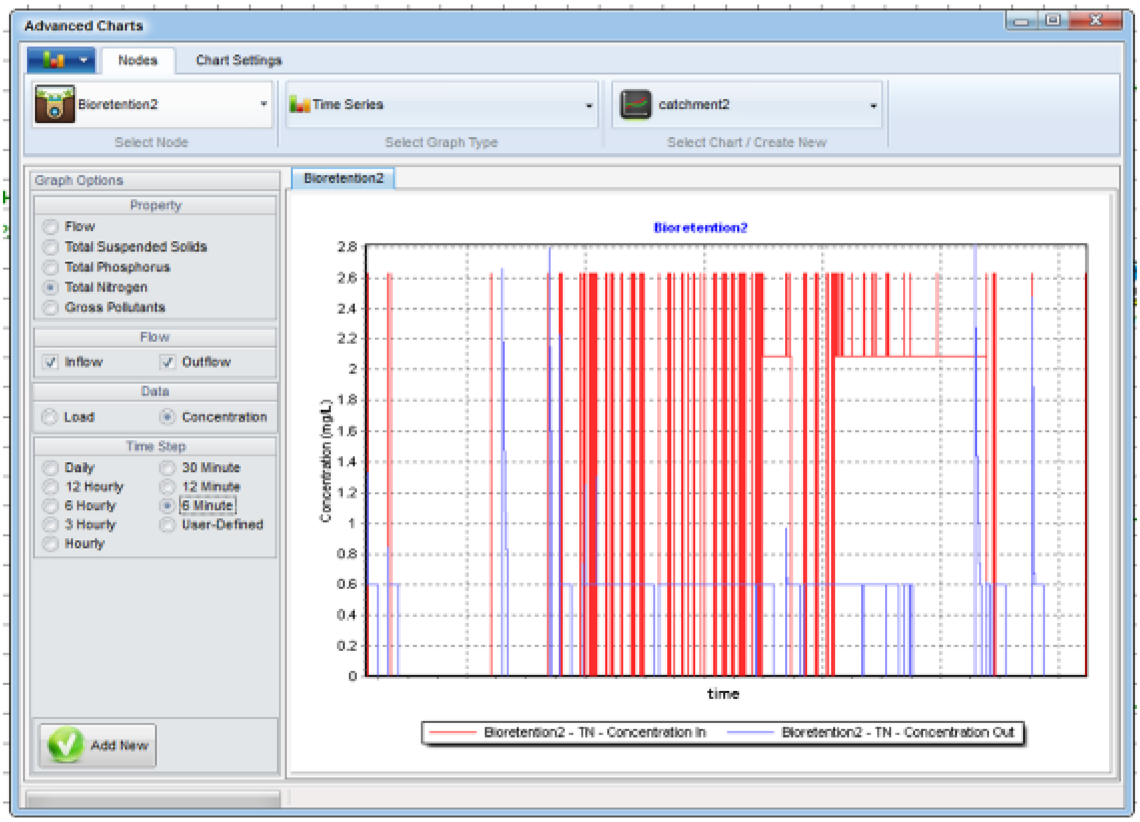

Figure 289: TN concentration vs time (6 minute)

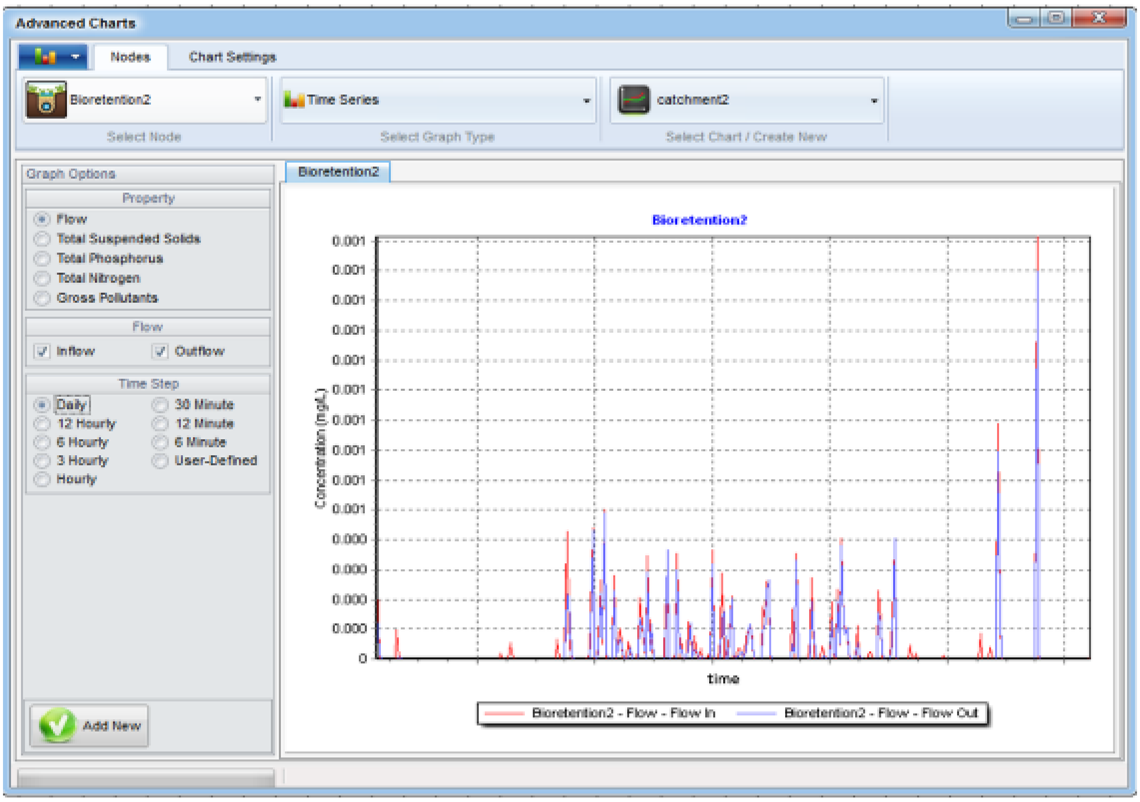

*Figure 290:*Flow vs. time (Daily)

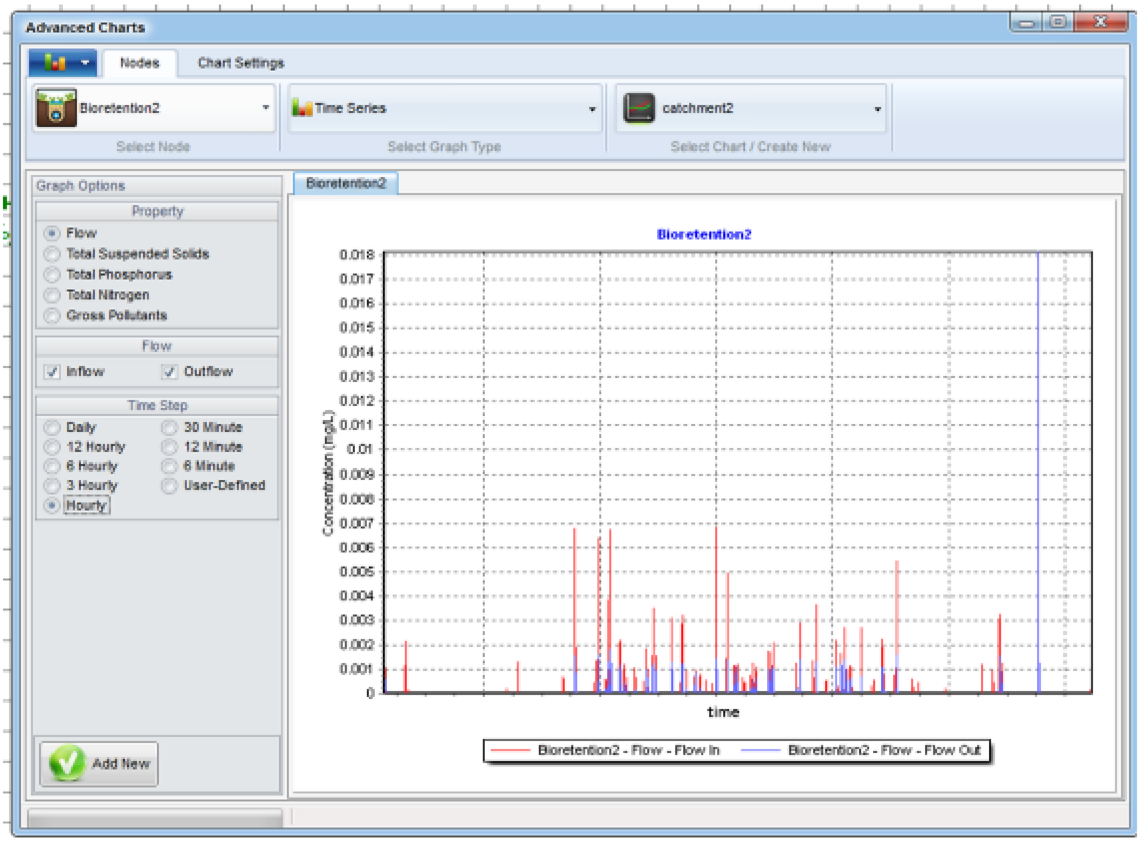

Figure 291: Flow vs. time (Hourly)

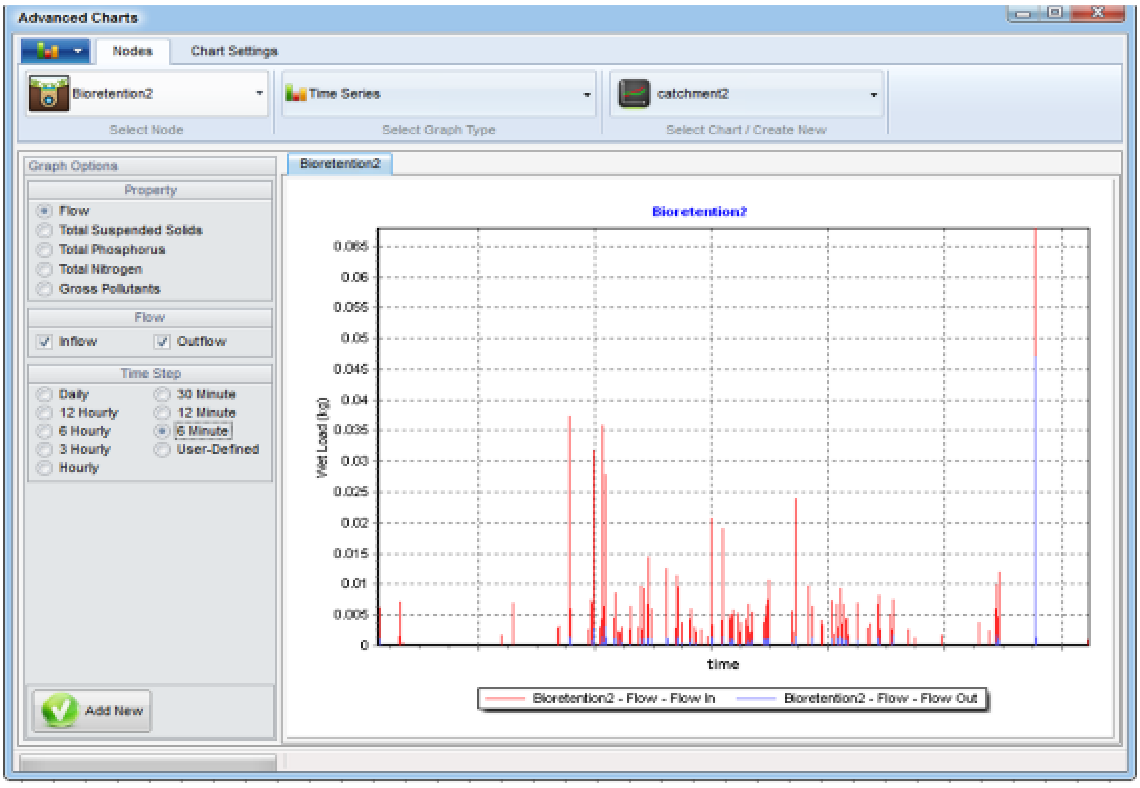

Figure 292: Flow vs. time (6 Minute)

2. Appendix 2

2.1. Gross Pollutant Trap during Construction Inspection Checklist

GROSS POLLUTANT TRAP CONSTRUCTION INSPECTION CHECKLIST (DURING CONSTRUCTION PHASE)

Project Name	Project ID	Inspection Number

Items Inspected	Completed (please tick)	Satisfactory (Y/N)
Preliminary Works		
Traffic and safety control measures in place		
The location reflects plans		
The site is protected from existing stormwater flows		
Earthworks		
Excavation carried out reflects plans and design drawings		
Pre-treatment		
The catchment contributing to GPT is stabilised appropriately		
The catchment contributing to GPT is not collecting excess rubbish/sediments		
Structural Components		
The location and levels of both inflow and outflow pipes reflect plans		
The pipe connections reflect plans		
The concrete components reflect plans		
Sediment and Erosion Control		
Appropriate stabilisation is implemented following earthworks		
If appropriate, temporary protection is in place		
Operation Establishment		
Temporary protection equipment removed		
Diversion for GPT removed		

INSPECTOR DETAILS	
Name	
Date of visit	
Time of visit	
Signature	
Notes and Comments	

2.2. Gross Pollutant Trap Post Construction Inspection Checklist

GROSS POLLUTANT TRAP CONSTRUCTION INSPECTION CHECKLIST (POST CONSTRUCTION PHASE)

Project Name	Project ID	Inspection Number

Items Inspected	Checked (please tick)	Satisfactory (Y/N)
Levels of inlet and outlet conform to plans and drawings		
Traffic control measures in place		
Dimensions of structural components reflect plans and drawings		
Access provided for maintenance and inspections		
Any sediment and debris generated during construction has been removed		

INSPECTOR DETAILS	
Name	
Date of visit	
Time of visit	
Signature	
Notes and Comments	

2.3. Gross Pollutant Trap Maintenance Checklist

NORTH TERRACE UPGRADE PROJECT
GROSS POLLUTANT TRAP MAINTENANCE CHECKLIST

Asset ID	GPT Location	Inspection Number

Please Tick	Inspection Type	Maintenance Requirements	Checked (Please Tick)	Result
	Routine Inspection (once per month)	Amount of debris removed by GPT		__%
		Is trash rack is 50% blocked? If so clean out required		
		Is there any visible damage to the GPT? (If yes, report in notes)		
	Routine Clean Out (a minimum of 8 times per annum)	Volume of debris and sediments removed		__m³
		Is there any visible damage to the GPT? (If yes, report in notes)		
	Annual Inspection	Is there any visible damage to the GPT? (If yes, report in notes)		
		Is water testing required?		

Component Condition	Checked (Please Tick)	Condition	Comments
Inlet			
Outlet			
Concrete Walls			
Trash Rack			
Removable Lids			

CONTRACTOR DETAILS	
Name	
Job Title	
Organisation	
Date	
Signature	
Notes	

3. Appendix 3

Client	Tonkin Consulting	Date: 1/06/2015
Project	North Terrace Drainage System	Pages:448-451
Subject	Geotechnical detailed calculations (Pipe, trench & retaining wall)	By: David Argent
Reviewed By:	David Argent	Date: 1/06/2015
Approved By:	Eriny Abdelraouf	Date: 7/06/2015

3.1. Pipe Class Example Calculation

At chainage 20, a 750mm internal diameter pipe is to be laid in a single trench, beneath North Terrace, a major arterial road. The trench will be excavated in silty slay, which according to the DPTI will not satisfy the requirements for backfilling. The backfilling will consist of 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 modified compaction density of 20 kN/m^3. The pipe invert level is 2.25 metres below the road surface level, requires a HS2 support condition, what pipe class is required?

3.1.1. Calculations

3.1.1.1. Working Load due to fill

The outside diameter (D) of the pipe has been found to be 870mm. From figure 12 of AS 3725, the minimum width of the trench $(B) = D + D/6 + D/6$;

Width of trench $B = 870 + \left(\frac{870}{6}\right) + \left(\frac{870}{6}\right) = 1160\ mm\ (Adopt\ 1300\ mm\ wide\ trench)$

Depth of fill $H = 2.25 - 0.525 = 1.725\ m\ of\ Sa - C\ and\ 0.525m\ pavement$

$$\frac{H}{B} = \frac{1.725}{1.30} = 1.327$$

$$C_t = \frac{1 - e^{-2(0.1914)(1.487)}}{2(0.1914)} = 1.293$$

Using **Equation 13**;

$$W_g = 1.293(20)(1.2)^2 = 37.7\ kN/m\ or;$$

$$W_g = C'_e wDH \qquad \textbf{Equation 21}$$

The projection ratio , $p = H/D = 2.59$ and $r_s p = 0.7$ (figure 7 from AS 3725), and $C'_e = 1.553$

$$W_g = 1.553(16)(0.87)(2.25) = 48.64\ kN/m$$

The convention for design loads is to take the smaller of the two values, thus

$$W_g = 37.7\ kN/m$$

3.1.1.2. Working load due to vehicle load

Since H is greater than 0.4 metres, the intensity of the live load is taken from Table B2, Appendix 2 of AS 3725 (2007), which gives $q = 21\ kPa$ (depth to crown of 1.4m) for the critical value which is for a M1600 wheel load (All other values are below this).

From Clause 6.5.3.2 (Figure 293) and the data in Commentary Paragraph C6.5.3.2 the base of the load prism has dimensions;

$$L_2 = a + 1.45H = 0.2 + 1.45(1.725) = 2.7m$$

$$L_1 = b + 1.45H = 0.5 + 1.45(1.725) = 3.0\ m$$

$$L_e = 3.0 + \left(1.45 + 0.75(0.87)\right) = 5.10m$$

From Clause 6.5.3.4;

$$W_q = qL_1S/L_e \qquad \textbf{Equation 22}$$

$$W_q = 21(3.0)(0.87\,)/5.1 = 10.7\ kN/m$$

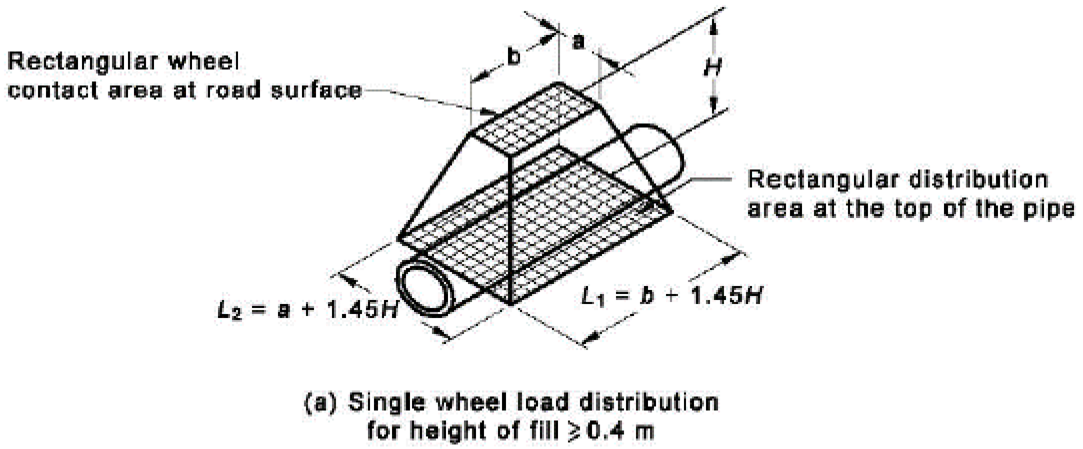

Figure 293: Dimensions used for M1600 wheel load as per Cl 6.5.3.2 of AS 3725 :2007

3.1.1.3. Test Loads

To determine the pipe's external load carrying capacity, when not subjected to internal pressure(as in stormwater pipes) is determined on the basis of performance of sample pipes that have undergone load tests (Standards Australia 2007).

From table 4 (AS 3725 (2007)) for support type HS2, the bedding factor (F) is 2.5. The long term design load (T_c) is calculated from the following equation;

$$T_c = W_g/F + \sum W_q/F_q \qquad \textbf{Equation 23}$$

$$T_c \geq 37.7/2.5 + 10.7/1.5 \geq 22.2\ k\,N/m$$

Utilising the data from Table 2.1, for test loads on Humes Reinforced Concrete Pipes (2009), adopt;

Class 2 750mm RCP RRJ

$$(\boldsymbol{T_c = 32\ kN/m})$$

3.2. Pavement Reinstatement Configuration

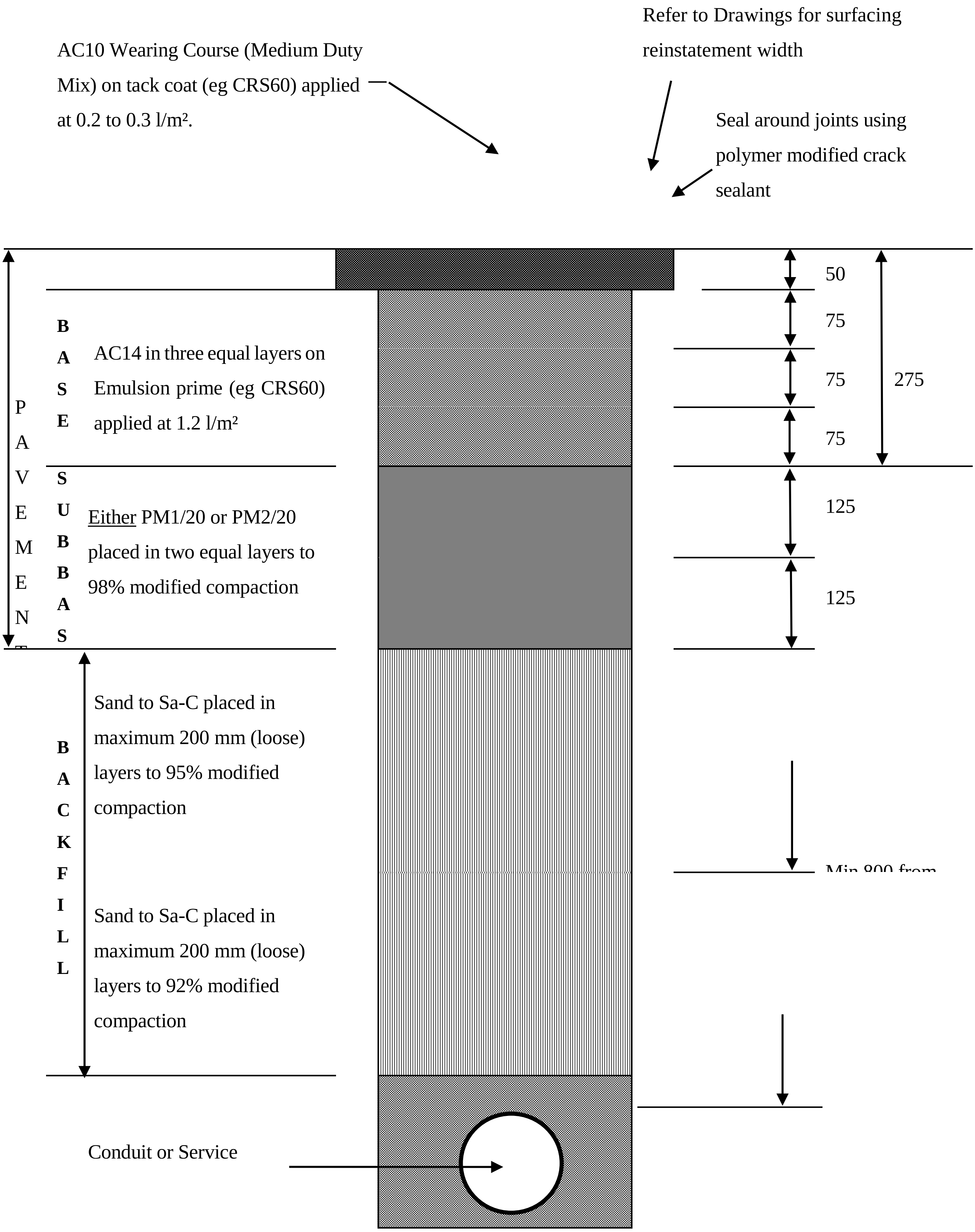

Figure 294: Road Pavement Configuration for reinstatement of North Terrace, Kent Town (DPTI 2012)

3.3. Tank Design

Client	Tonkin Consulting	Date: 1/06/2015
Project	North Terrace Drainage System	Pages:452-458
Subject	Tank Loadings and Calculations	By: Saeed Karevan
Reviewed By:	David Argent	Date: 1/06/2015
Approved By:	Eriny Abdelraouf	Date: 7/06/2015

3.3.1. Wind Load

For the wind load, the Australian Standard was used - Wind Actions (AS 1170.2:2011). There is no specific design for rainwater tank in this standard but it can be assume as normal structural element. The wind loading on the structure (Rainwater Tank) may often to three ultimate wind loads: an upwards wind action, downward wind action and side wind action. Therefore to analyse of this structure wind load is essential. The building is to be built in South Australia. The following calculations were determined based on Australian guidelines for wind actions which give the wind pressure on the structure.

3.3.1.1. Determining site wind speed

The building has an importance level of 2 because it has been designed as building element (Rainwater tank which will be resting on ground on the side of the house) for a normal construction which contains few number of people. Hence the annual probability of exceedance is 1:500 as the region doesn't experience cyclonic conditions. Therefore V_R can then be found by inspecting table 3.1 (Australian Standards 2011) where R is 500 therefore V_{500} in region A is $45ms^{-1}$.

- The site wind speed is given by $V_{sit,\beta}$ where β is the direction for example North, North East, East, etc. The speed is then determined by finding the values of the multipliers in the equation.
- Wind forces in accordance with AS/NZS 1170.2: 2011 on the basis of terrain category 2. Terrain Height Multiplier $M_{z,cat}$ is determined by inspecting the surrounding terrain. As

the height of this building element is less than 5m high therefore the terrain considered as a terrain category 2 for all eight directions.

$V_{sit,\beta} = V_R M_d (M_{z,cat} M_s M_t)$

Site wind speed: $V_{sit,\beta}=V_R M_d (M_{z,cat} M_s M_t)$

V_R = Regional wind speed, form table 3.1, region A1 (1170.2: 2011) for 500 year.

Regional wind speed: V_R = 500 (Regional A1)

V_R = Wind speed = 45 m/s

Table 97: Wind Direction Multipliers (Md)

Direction	Multiplier
N	0.90
NE	0.80
E	0.80
SE	0.80
S	0.85
SW	0.95
W	1.00
NW	0.95
Any direction	1.00

$V_{sit,\beta}$ is then calculated by combining the multiplies and entering them in to the equation $V_{sit,\beta} = V_R M_d (M_{z,cat} M_s M_t)$

Table 98: Vsit, β

	$V_{sit,\beta} = V_R M_d(M_{z,cat} M_s M_t)$	Speed (ms^{-1})
$V_{sit,N}$	= 45 x 0.9 (0.91 x 1 x 1)	36.9
$V_{sit,NE}$	= 45 x 0.8 (0.91 x 1 x 1)	32.8
$V_{sit,E}$	= 45 x 0.8 (0.91 x 1 x 1)	32.8
$V_{sit,SE}$	= 45 x 0.8 (0.91 x 1 x 1)	32.8

$V_{sit, S}$	= 45 x 0.85 (0.91x 1 x 1)	34.8
$V_{sit, SW}$	= 45 x 0.85 (0.91 x 1 x 1)	34.8
$V_{sit, W}$	= 45 x 1 (0.91 x 1 x 1)	41
$V_{sit, NW}$	= 45 x 0.95 (0.91 x 1 x 1)	38.9

Table 99: Parameters to determine site wind speeds for 8 directions

Direction	V_R (ms^{-1})	M_d	$M_{z, cat}$	M_s	M_t	V_{sit} (ms^{-1})
N	45	0.90	0.91	1.0	1.0	36.9
NE	45	0.80	0.91	1.0	1.0	32.8
E	45	0.80	0.91	1.0	1.0	32.8
SE	45	0.80	0.91	1.0	1.0	32.8
S	45	0.85	0.91	1.0	1.0	34.8
SW	45	0.95	0.91	1.0	1.0	34.8
W	45	1.00	0.91	1.0	1.0	41
NW	45	0.95	0.91	1.0	1.0	38.9

Where:

$M_{z,cat}$=Terrain /Height Multipliers, as the structure height is between 1.75 m to 2.3 m, for all directions with a height less than 5 m from table 4.1 (AS/NZS 1170.2: 2011) then:

$M_{Z,Cat} = 0.91$

M_S = Shielding multiplier which is a variable that changes depending on the section of the structure, (shielding is to be ignored, assumed)

$M_S = 1.00$

M_t = Topographic multiplier, because there are no shielding hill in the region, when $H/2L_u < 0.05$, $M_h = 1.00$, then:

$M_t = 1.00$ (AS/NZS 1170.2:2011)

$$V_{sit,\beta}=45 \times 1.0 \times 0.91 \times 1.0 \times 1.0 = 40.95 \approx 41\ m/s$$

3.3.1.2. Wind pressure

P = (0.5 × ρ_{air}) $(V_{des,\theta})^2$× C_{fig}× C_{dyn} (AS/NZS 1170.2:2011)

P= design wind pressure in Pascal's

ρ_{air}= density of air, equal to 1.2 kg/m^3

$V_{des,\theta}$ = building orthogonal design wind speed

C_{fig}= aerodynamic shape factor

C_{dyn}= dynamic response factor

ρ_{air} = 1.2 kg/m^3

C_{dyn} = 1

3.3.1.3. External Pressures (West Wind, Windward side)

$V_{des,\theta} = 41\ m/s$ (Critical wind)

$C_{fig} = 0.7$ (Australian Standard)

P = (0.5 × ρ_{air}) $(V_{des,\theta})^2$× C_{fig}× C_{dyn}

Where:

ρ_{air} = 1.2

V_{des} = 37.4 m/s

C_{dyn} = 1

C_{fig} = $C_{p,e}$ x K_a x K_c x K_L x K_p

= 0.7 x 1 x 0.8 x 1 x 1

= 0.56

Therefore:

P = 0.5 x 1.2 x (41^2) x 0.56 x 1

= 537.6Pa = 0.538 kPa

3.3.1.4. Internal Pressures (West Wind, Windward side)

P = 0.5 x ρ_{air} x (V_{des}^2) x C_{fig} x C_{dyn}

Where:

ρ_{air} = 1.2

V_{des} = 41 m/s

C_{dyn} = 1

C_{fig} = $C_{p,i}$ x K_c

= 0 x 0.8

= 0

Therefore:

P = 0.5 x 1.2 x (41^2) x 0 x 1= 0

External Pressure - Internal Pressure = 537.6 – 0 = 537.8 Pa = 0.538 kPa

3.3.1.5. Sample Calculation for lateral pressures

Sample calculation: for category 1 with 1750 mm

$$P = \rho g h$$

$$P = \rho g z = 1000 \left(\frac{kg}{m^3}\right) \times 9.8 \left(\frac{m}{s^2}\right) \times 1.75\ (m) = 17150 \left(\frac{kg}{m.s^2}\right) or\ Pa = 17.15\ kPa$$

3.3.1.6. Calculations for hoop stress and axial stress

Sample calculation:

4000L

P= 17.15 kPa, D= 1.830m, H= 1.750 m

Shell & bottom thickness = 2.36 mm = 0.00236m

Inner diameter = 1.83 – (2 ×0.00236) = 1.82528 m

For Shell and Bottom thickness:

Hoop stress $\sigma_H = \frac{Pd}{2t} = \frac{17.15 \times (1.82528)}{2 \times 0.00236} = 6632.108\ kPa \approx 6632\ kPa$

Longitudinal or axial stress $\sigma_L = \frac{Pd}{4t} = \frac{17.15 \times (1.82528)}{4 \times 0.00236} = 3316.05 kPa \approx 3316\ kPa$

Therefore:

Hoop stress $\sigma_H = 6.6 MPa$

Axial stress $\sigma_L = 3.3\ MPa$

5500L

P= 22.54 kPa, D= 1.800 m, H= 2.3 m

Shell thickness = 4.24 mm = 0.00424m and Bottom thickness = 6.09 mm = 0.00609 m

Inner diameter = 1.8 – (2 ×0.00424) = 1.79152 m

<u>For Shell thickness:</u>

Hoop stress $\sigma_H = \frac{Pd}{2t} = \frac{22.54 \times (1.79152)}{2 \times 0.00424} = 4761.89\ kPa \approx 4762\ kPa = 4.76\ MPa$

Longitudinal or axial stress $\sigma_L = \frac{Pd}{4t} = \frac{22.54 \times 1.79152}{4 \times 0.00424} = 2380.94\ kPa \approx 2.4\ MPa$

<u>For bottom thickness:</u>

Hoop stress $\sigma_H = \frac{Pd}{2t} = \frac{22.54 \times (1.8-(2\times 0.00609)}{2 \times 0.00609} = 3308.49\ kPa \approx 3308\ kPa = 3.3\ MPa$

8000L

P= 21.56 kPa, D= 2.35 m, H= 2.2 m

Shell thickness = 4.24 mm = 0.00424m and Bottom thickness = 6.09 mm = 0.00609 m

<u>For Shell thickness:</u>

Hoop stress $\sigma_H = \frac{Pd}{2t} = \frac{21.56 \times (2.35-(2\times 0.00424))}{2 \times 0.00424} = 5953.2\ kPa \approx 5953\ kPa = 5.95\ MPa$

Longitudinal or axial stress $\sigma_L = \frac{Pd}{4t} = \frac{21.56 \times (2.35-(2\times 0.00424))}{4 \times 0.00424} = 2069.1\ kPa \approx 2.1\ MPa$

<u>For bottom thickness:</u>

Hoop stress $\sigma_H = \frac{Pd}{2t} = \frac{21.56 \times (2.35-(2\times .00609))}{2 \times 0.00609} = 4138.2\ kPa \approx 4138\ kPa = 4.1\ MPa$

9500L

P= 15.484 kPa, D= 3.09 m, H= 1.58 m

Shell thickness = 4.24 mm = 0.00424m and Bottom thickness = 6.09 mm = 0.00609 m

For Shell thickness:

Hoop stress $\sigma_H = \frac{Pd}{2t} = \frac{15.484 \times (3.09-(2\times 0.00424))}{2 \times 0.00424} = 5626.68\ kPa \approx 5627\ kPa = 5.6\ MPa$

Longitudinal or axial stress $\sigma_L = \frac{Pd}{4t} = \frac{15.484 \times (3.09-(2\times 0.00424))}{4 \times 0.00424} = 2813.3\ kPa \approx 2.8\ MPa$

For bottom thickness:

Hoop stress $\sigma_H = \frac{Pd}{2t} = \frac{15.484 \times (3.09-(2\times 0.00609))}{2 \times 0.00609} = 3912.7\ kPa \approx 3913\ kPa = 3.9\ MPa$

3.3.2. Concrete Loads

Category 1:

Volume = 4000 L

Diameter = 1830 mm

Height = 1750 mm

The mass of the water = 4000 kg

Determine the weight of the steel tank

Calculate the surface area of the cylinder:

Area of the top and bottom = πr^2

Area of the side = $2\pi h$

The formula is: $A = 2\pi r^2 + 2\pi r h$

$r = (1830/2) - 2.36 = 912.64$ mm

$A = 2\pi r^2 + 2\pi r h = 2 \times 3.14 \times (912.64)^2 + 2 \times 3.14 \times 912.64 \times 1750 = 15260599.5\ mm^2$

$A = 15.26\ m^2$

Calculating the weight of cylinder:

Area × thickness × density of steel = 15.26 × 0.00236 × 7850 = 282.71 kg

Category 2:

Volume = 5500 L

Diameter = 1800 mm

Height = 2300 mm

The mass of the water = 5500 kg

Average thickness $= \frac{4.24+6.09+2.92}{3} = 4.42\ mm$

Inner radius = 895.58 mm

$A = 2\pi r^2 + 2\pi r h = 2 \times 3.14 \times (895.58)^2 + 2 \times 3.14 \times 895.58 \times 2300 = 17972716.53\ mm^2$

$A = 17.97\ m^2$

Calculating the weight of cylinder:

Area × thickness × density of steel = 17.97 × 0.00442 × 7850 = 623.599 kg

Category 3:

Volume = 8000 L

Diameter = 2350 mm

Height = 2200 mm

The mass of the water = 8000 kg

Average thickness $= \frac{4.24+6.09+2.92}{3} = 4.42\ mm$

Inner radius = 1170.58 mm

$A = 2\pi r^2 + 2\pi r h = 2 \times 3.14 \times (1170.58)^2 + 2 \times 3.14 \times 1170.58 \times 2200 = 24777950.6\ mm^2$

$A = 24.78\ m^2$

Calculating the weight of cylinder:

Area × thickness × density of steel = 24.78 × 0.00442 × 7850 = 859.79 kg

Category 4:

Volume = 9500 L

Diameter = 3090 mm

Height = 1580 mm

The mass of the water = 9500 kg

Average thickness $= \frac{4.24+6.09+2.92}{3} = 4.42\ mm$

Inner radius = 1540.58 mm

A = $2\pi r^2 + 2\pi r h = 2 \times 3.14 \times (1540.58)^2 + 2 \times 3.14 \times 1540.58 \times 1580 = 30191119.7\ mm^2$

A = 30.19 m^2

Calculating the weight of cylinder:

Area × thickness × density of steel = 30.19 × 0.00442 × 7850 = 1047.5 kg

Determining ultimate load

Sample calculations

Category 1:

Point Load = 4282.7 (kg) × 9.8 (m/s^2) = 41970.46 N = 41.97 kN

The allowable bearing capacity must be a minimum of the rearranged formula of the general pressure equation i.e. Pressure = Force/Area, since both width and length are equal, it is assumed that area = B^2, and so the rearranged formula for minimum slab width and length (B) is calculated below and checked to the chosen slab dimensions.

Pressure = Forces / Area

$$B = \sqrt{Force/Pressure}$$

The calculation for the total dead load is shown below

Dead load (G) = water load (G_W) + tank load (G_T) = 42 kN

Slab self-weight (SW) = 2. 00 × 2.00 × 0.1 × 24 (kN/m^3) = 9.6 KN

Total weight acting on soil = 42 kN + 9.6 KN = 51.6kN

The allowable bearing pressure = 128 kPa

Slab dimensions = 2.00 m × 2.00 m × 0.1 m

1 kPa = 1 KN/m²

The slab being designed for are both square in dimensions, therefore the area of the slab can be simplified to B^2, i.e. $B^2 = L \times W$. in this case the formula is used to determine the required slab dimensions:

Pressure = Forces / Area

$$B = \sqrt{Force/Pressure}$$

$$B = \sqrt{\frac{51.6kN}{128KPa}} = 0.64\text{m}$$

Since the allowable slab dimension, B < 2.0m, it is safe to assume that the chosen size for the slab is appropriate for the final design of the slab.

An excel spreadsheet is used to determine the allowable dimension (B) for every tank considered. The output of the results is shown in the table below.

Table 100 Calculation of allowable dimensions for every slab and tank configuration

Dimension	Weight(kN)	Slab(kN)	Total G (kN)	Allowable B (m)
Tank 1	42	9.6	51.6	0.63
Tank 2	60	9.6	69.6	0.74
Tank 3	87	9.6	96.6	0.87
Tank 4	103	9.6	112.6	0.94

3.4. Design Loads for Brick Arch

Client	Tonkin Consulting	Date: 1/06/2015
Project	North Terrace Drainage System	Pages:462-479
Subject	Vertical & Horizontal Loads on Sandstone Arch	By: Jeremy Bemmerl
Reviewed By:	David Argent	Date: 1/06/2015
Approved By:	Eriny Abdelraouf	Date: 7/06/2015

3.4.1.1. Vertical Pressures - Traffic Loads

The sandstone arch culvert is located directly underneath North Terrace which is a main arterial route into the city. Traffic loads on the culvert will be determined in conjunction with the following standards:

- AS 5100.2 – 2004 Bridge Design Part 2: Design Loads
- AS1597.2 – 2013 Precast Reinforced Concrete Box Culverts

The standard traffic loads on the arch culvert compromise of;

- W80 wheel
- A160 axle, and
- M1600 tri-axle

It has been assumed that the traffic loads will spread on a 1:1 slope throughout the soil.

W80 Wheel Load

Figure 295 is the loading diagram of the W80 wheel load. This load combination allows a point load to be applied at any location ***(x, y)*** along the width or length of the road to create the most adverse or critical effect on the structure. The point load is applied through an initial load area of 250mm by 400mm.

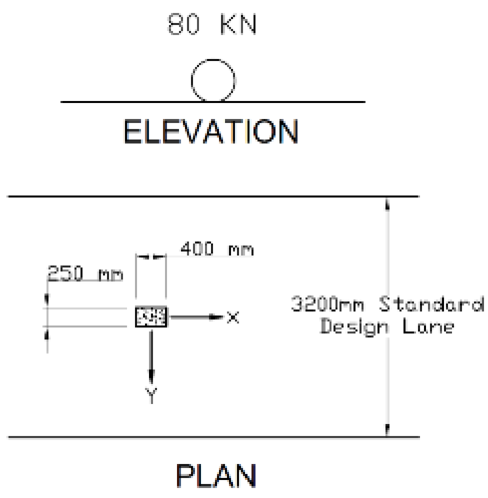

Figure 295:W80 Load Diagram

Load Case 1

The loading path throughout the soil for a point load applied directly above the brick arch can be identified in Figure 296 which shows the dimensions of the load width at points A and B. As the load path does not affect points C, D and E, these locations will not be evaluated for traffic loadings.

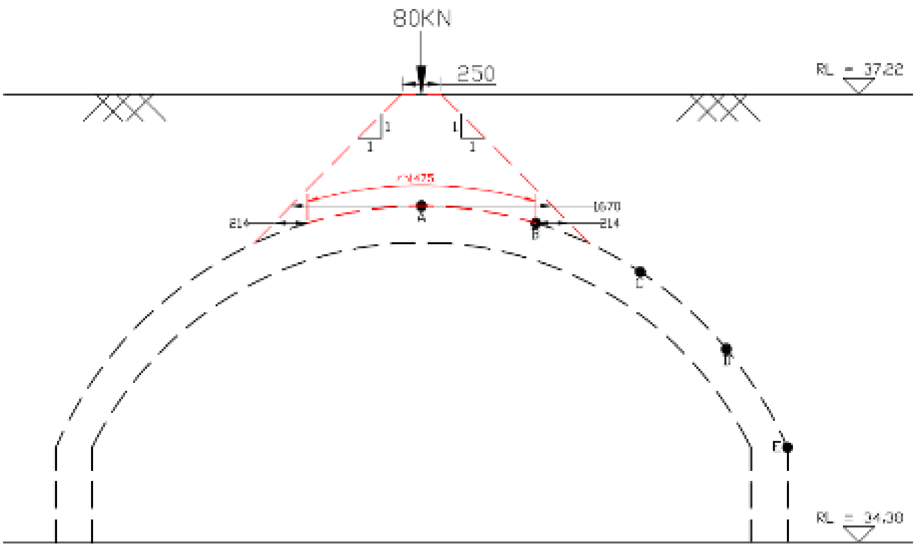

Figure 296: W80 Load Distribution - Case 1

Sample Calculations - Vertical Stress at Point B:

Referring to the dimensions above the pressure at the arch's crest from W80 wheel load can be calculated. The load area will need to be determined at point B longitudinally and laterally along the arch culvert.

$$\sigma_v = \frac{P}{A}$$

$$A = (Longitudinal\ Load\ Width) * (Lateral\ Load\ Width)$$

$$A = (0.4 + 0.816 + 0.816) * (0.214 + 0.214 + 1.475) = 3.86\ m^2$$

$$\sigma_v = \frac{P}{A} = \frac{80}{3.86} = 21\ kPa$$

Results:

As seen below in Table 101 are the vertical pressures for each designated location along the arch.

Table 101: W80 Vertical Pressures - Case 1

Location	Depth (mm)	Longitudinal Load Width (mm)	*Lateral Load Width (mm)*	Vertical Pressure
A	710	1820	1670	26
B	816	2032	1903	21
C	1127	N/A	N/A	0.0
D	1615	N/A	N/A	0.0
E	2240	N/A	N/A	0.0

Load Case 2

The load path throughout the soil for a point load applied directly above point B can be identified in Figure 297 which shows the dimensions of the load width at points A, B, C, D and E. As the load has been applied asymmetrically to determine the difference in pressures.

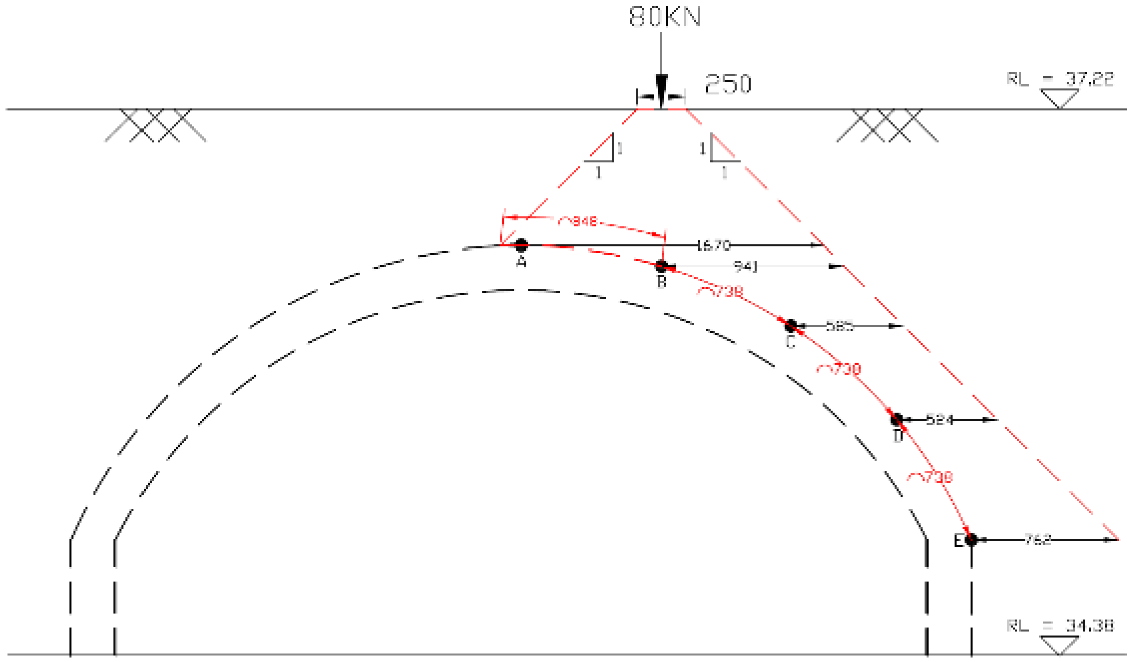

Figure 297: W80 Load Distribution - Case 2

The vertical pressures resulting from load case 2 can be seen in Table 102

Table 102: W80 Vertical Pressures - Case 2

Location	Depth (mm)	Longitudinal Load Width (mm)	*Lateral Load Width (mm)*	Vertical Pressure
A	710	1820	1670	26
B	816	2032	1789	22
C	1127	2654	2171	14
D	1615	3630	2848	8
E	2240	4880	3824	4

A160 Axle Load

The A160 axle loading configuration can be seen below in Figure 298 shows the A160 axle load configuration, which allows for a 160 KN point load to be distributed over a single axle. This is also equivalent to two 80 KN point loads located 2000mm apart. The longitudinal interaction between these loads will need to be accounted for and may reduce the load area significantly.

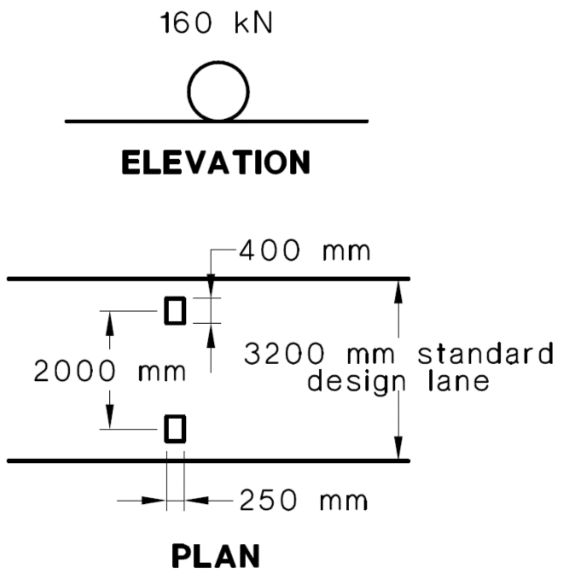

Figure 298: A160 Load Diagram (Standards Australia 2007)

As seen in Figure 299 are the load paths of the two point's loads travelling longitudinally along the sandstone arch culvert. It can be seen that the load paths cross before point B, hence half the load width should not exceed 1000mm for points B, C, D and E. This is likely to increase the vertical pressure acting on the arch.

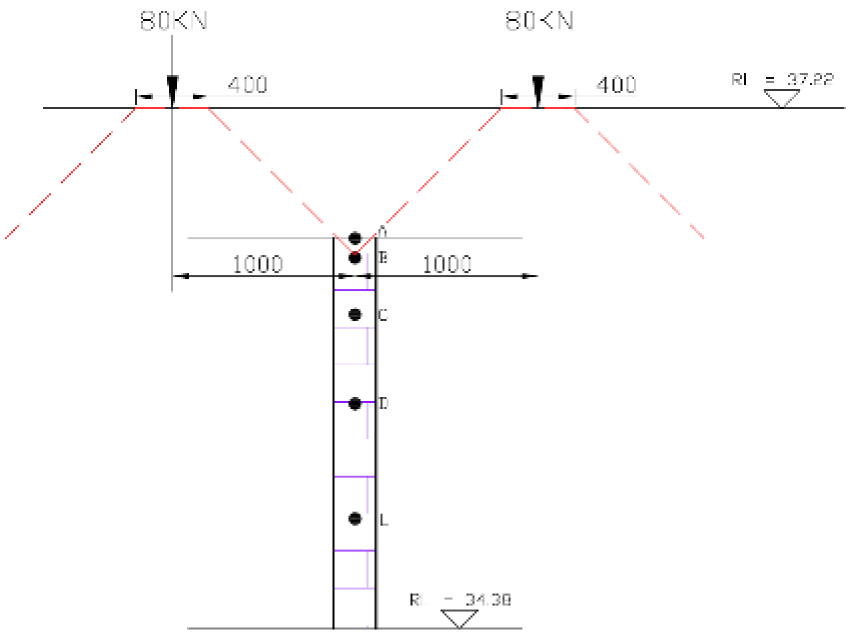

Figure 299: Longitudinal Load Path

Load Case 1

The loading path throughout the soil for a point load applied directly above the brick arch can be identified in Figure 299 which is the same configuration as case 1 for the W80 wheel load however the longitudinal load widths will change during this calculation as seen below in Table 103

Table 103: A160 Vertical Pressure - Case 1

Location	Depth (mm)	Longitudinal Load Width (mm)	*Lateral Load Width (mm)*	Vertical Pressure
A	710	1820	1670	26
B	816	2016	1903	21
C	1127	N/A	N/A	0.0
D	1615	N/A	N/A	0.0
E	2240	N/A	N/A	0.0

Load Case 2

The load path throughout the soil for a point load applied directly above point B is the same configuration as case 2 for the W80 wheel load however the longitudinal load widths will change during this calculation as seen below in Table 104

Table 104: A160 Vertical Pressure - Case 2

Location	Depth (mm)	Longitudinal Load Width (mm)	*Lateral Load Width (mm)*	Vertical Pressure
A	710	1820	1670	26
B	816	2016	1789	22
C	1127	2327	2171	16
D	1615	2815	2848	10
E	2240	3440	3824	6

As seen in the above analysis, the loading has increased by 2 kPa for points C, D and E within case 2. As the configuration within the M1600 tri-axle loadings are similar to the A160 axle loadings, the longitudinal spacing as seen above will be used for the M1600 load class.

M1600 Tri-axle Load

According to AS 1597.2, the M1600 tri-axle loading class will need to be evaluated excluding the uniformly distributed loadings which can be seen in Figure 300 Hence the M1600 tri-axle loading will represent three point loads with a magnitude of 60 KN spaced at 1.25m. The starting load area will be 200mm by 400 mm.

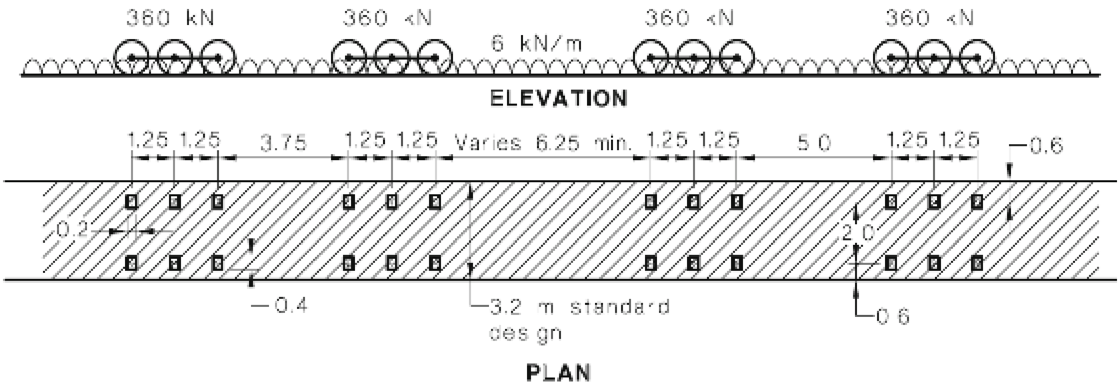

Figure 300: M1600 Load Diagram (Standards Australia 2007)

Load Case 1

The M1600 load path throughout the soil for a symmetrical condition can be identified in Figure 301. As the loading paths connect, this will be taken as a vertical line with the outermost load paths following the slope and increasing the load area with depth.

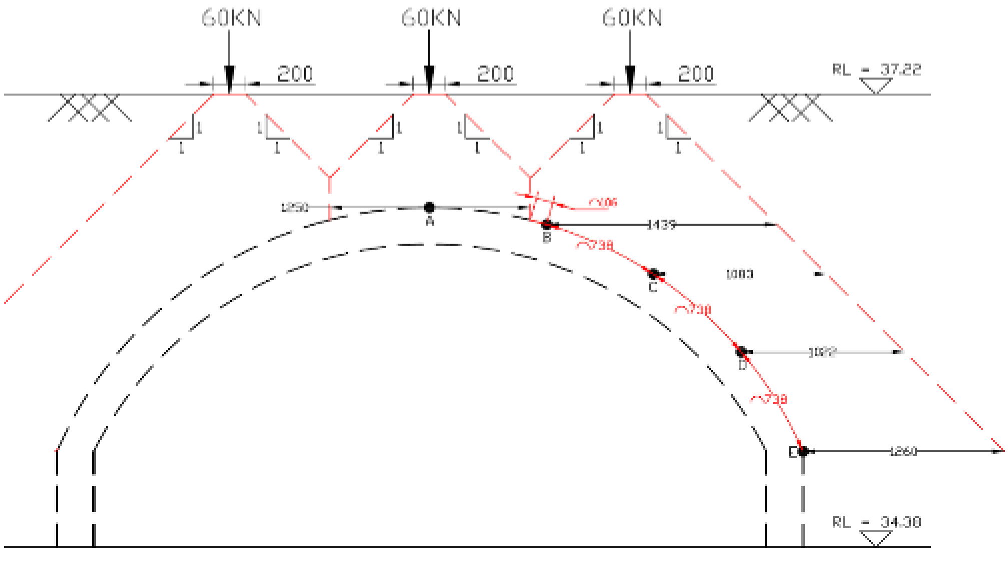

Figure 301: M1600 Load Distribution - Case 1

Table 105: M1600 Vertical Pressures - Case 1

Location	Depth (mm)	Longitudinal Load Width (mm)	*Lateral Load Width (mm)*	Vertical Pressure
A	710	1820	1250	26
B	816	2016	1545	19
C	1127	2327	1927	13
D	1615	2815	2604	8
E	2240	3440	3580	5

Load Case 2

The M1600 load path throughout the soil for the central point load located directly above point D can be identified in Figure 302 The interaction between the load paths will be the same as load case 1.

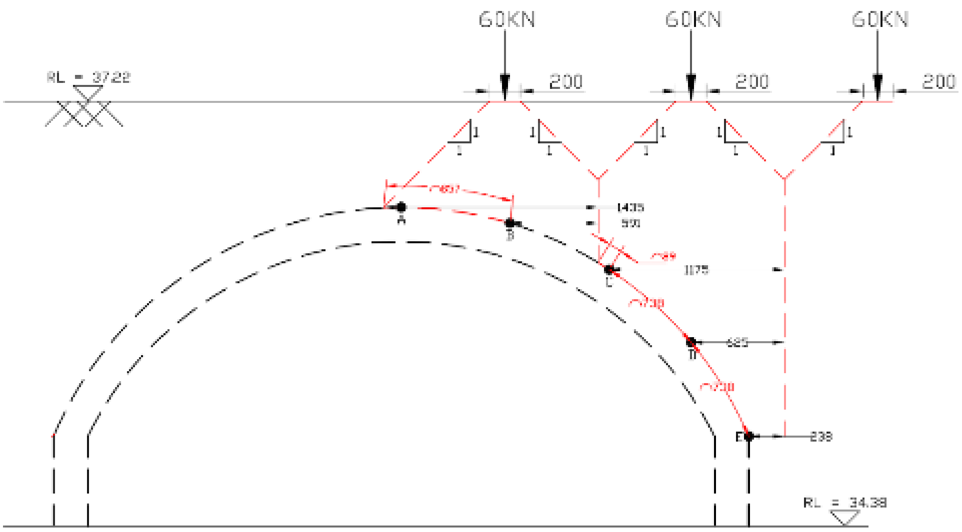

Figure 302: M1600 Load Distribution - Case 2

Table 106: M1600 Vertical Pressures, Case 1

Location	Depth (mm)	Longitudinal Load Width (mm)	*Lateral Load Width (mm)*	Vertical Pressure
A	710	1820	1435	23
B	816	2016	1448	21
C	1127	2327	1264	20
D	1615	2815	1452	15
E	2240	3440	1803	10

3.4.1.2. Vertical Pressures - Soil Loads

AS1597.2 describes the process required to calculate the earth pressures acting on the surface of a culvert structure. The vertical and horizontal earth pressures will be evaluated in terms of the two soil compositions present below in Figure 303. The unit weight and horizontal pressure coefficient used will be the same for points A, B and C and will change for points D and E due to the change in soil composition at the RL of 35.97.

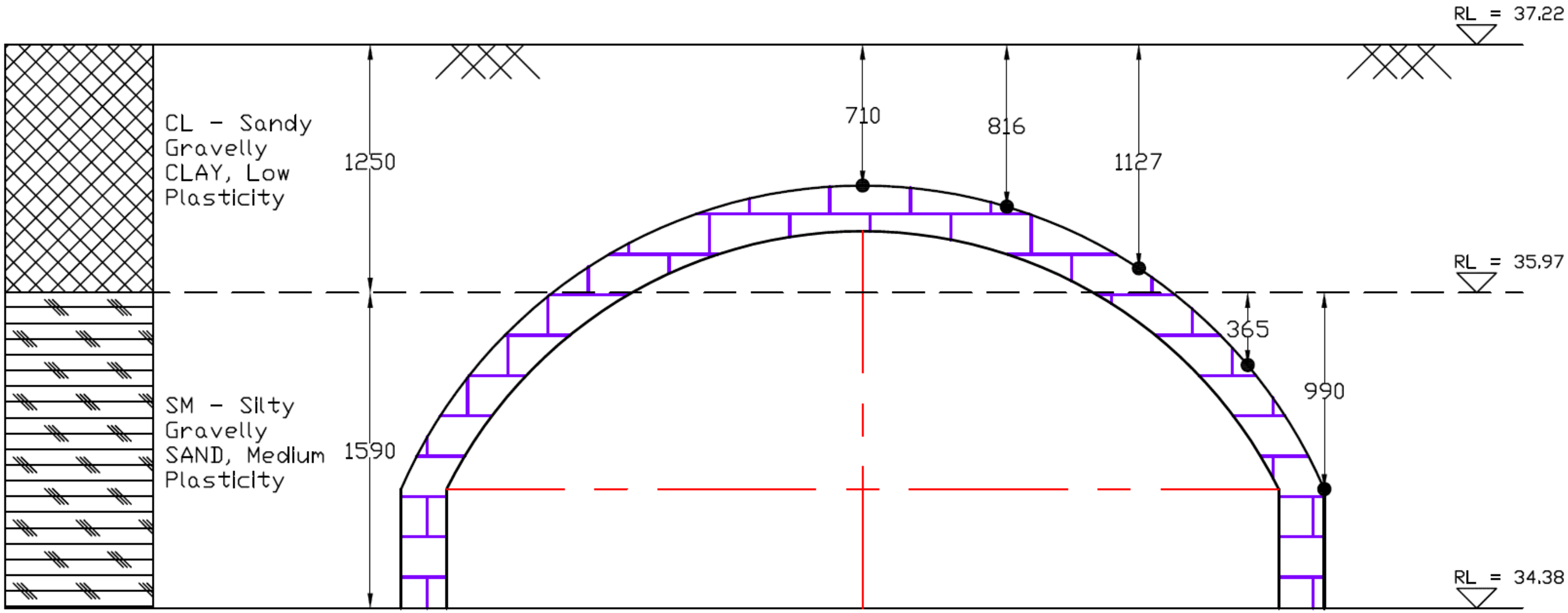

Figure 303: Geotechnical Model

The resultant vertical pressure acting at points A, B, C, D and E can be calculated using the vertical soil pressures and the live load surcharge. The soil strength properties are shown in Table 107

Table 107: Soil Strength Properties

Depth	Soil	Unit Weight γ (kN/m3)	Undrained Shear Strength Cu (kPa)	Shear Strength C' (kPa)
0.0 – 1.25	Sandy gravelly CLAY	21.5	102	0-7
1.25 – 4.5	Silty gravelly SAND	20	156	0-10

Table 108: Soil Loads

Location	Vertical Pressure (kPa)
A	$21.5 * 0.71 = 15$
B	$21.5 * 0.861 = 19$
C	$21.5 * 1.127 = 24$
D	$21.5 * 1.25 + 20 * 0.365 = 34$
E	$21.5 * 1.25 + 20 * 0.99 = 47$

3.4.1.3. Vertical Pressures - Resultant

The resultant vertical pressure can be calculated using the traffic loads and soil loads which were calculated in the previous sections. AS 1170.0 section 4.2 identifies an ultimate limit state for strength $(1.2G + 1.5Q)$ which will require the traffic loads (Q) and the soil loads (G) to be factored accordingly.

W80 Case 1

Location	Traffic Load: Q (kPa)	Soil Load: G (kPa)	Resultant Un-factored Pressure: G+Q (kPa)	Resultant Factored Pressure: 1.2G + 1.5Q (kPa)
A	26	15	41	57
B	21	19	40	54

C	0.0	24	24	29
D	0.0	34	34	41
E	0.0	47	47	56

Table 109: W80 Resultant Vertical Loading - Case 1

W80 Case 2

As this loading condition is an asymmetrical loading condition due to the traffic loads being applied off center to the arch, the vertical loading on each side of the arch's center will change as seen below in Table 110 and Table 111 Although certain loadings are designated to the right and left side of the arch this will likely change in the field due to the traffic directions and the location of the stormwater pipe.

Table 110: W80 Resultant Vertical Loading - Case 2 (Right Side)

Location	Traffic Load: Q (kPa)	Soil Load: G (kPa)	Resultant Un-factored Pressure: G+Q (kPa)	Resultant Factored Pressure: 1.2G + 1.5Q (kPa)
A	26	15	41	57
B	22	19	41	56
C	14	24	38	50
D	8	34	42	53
E	4	47	51	62

Table 111: W80 Resultant Vertical Loading - Case 2 (Left Side)

Location	Traffic Load: Q (kPa)	Soil Load: G (kPa)	Resultant Un-factored Pressure: G+Q (kPa)	Resultant Factored Pressure: 1.2G + 1.5Q (kPa)
A	26	15	41	57
B	0	19	19	23
C	0	24	24	29
D	0	34	34	41
E	0	47	47	56

A160 Case 1

Table 112: A160 Resultant Vertical Loadings - Case 1

Location	Traffic Load: Q (kPa)	Soil Load: G (kPa)	Resultant Un-factored Pressure: G+Q (kPa)	Resultant Factored Pressure: 1.2G + 1.5Q (kPa)
A	26	15	41	57
B	21	19	40	54
C	0.0	24	24	30
D	0.0	34	34	41
E	0.0	47	47	56

A160 Case 2

As this loading condition is asymmetrical the left and right side of the arch will be evaluated separately as seen below in Table 113 and Table 114

Table 113: A160 Resultant Vertical Loadings - Case 2(Right Side)

Location	Traffic Load: Q (kPa)	Soil Load: G (kPa)	Resultant Un-factored Pressure: G+Q (kPa)	Resultant Factored Pressure: 1.2G + 1.5Q (kPa)
A	26	15	41	57
B	22	19	41	56
C	16	24	40	53
D	10	34	44	56
E	6	47	53	65

Table 114: A160 Resultant Vertical Loadings - Case 2 (Left Side)

Location	Traffic Load: Q (kPa)	Soil Load: G (kPa)	Resultant Un-factored Pressure: G+Q (kPa)	Resultant Factored Pressure: 1.2G + 1.5Q (kPa)
A	26	15	41	57
B	0	19	19	23
C	0	24	24	29
D	0	34	34	41
E	0	47	47	56

M1600 Case 1

Table 115: M1600 Resultant Vertical Loadings - Case 1

Location	Traffic Load: Q (kPa)	Soil Load: G (kPa)	Resultant Un-factored Pressure: G+Q (kPa)	Resultant Factored Pressure: 1.2G + 1.5Q (kPa)
A	29	15	44	62
B	21	19	40	54
C	12	24	36	47
D	7	34	41	51
E	4	47	51	62

M1600 Case 2

As this loading condition is asymmetrical the left and right side of the arch will be evaluated separately as seen below in Table 116 and Table 117

Table 116: M1600 Resultant Vertical Loading - Case 2 (Right Side)

Location	Traffic Load: Q (kPa)	Soil Load: G (kPa)	Resultant Un-factored Pressure: G+Q (kPa)	Resultant Factored Pressure: 1.2G + 1.5Q (kPa)
A	25	15	40	56
B	22	19	41	56
C	19	24	43	57
D	12	34	46	59
E	7	47	54	67

Table 117: M1600 Resultant Vertical Loading - Case 2(Left Side)

Location	Traffic Load: Q (kPa)	Soil Load: G (kPa)	Resultant Un-factored Pressure: G+Q (kPa)	Resultant Factored Pressure: 1.2G + 1.5Q (kPa)
A	25	15	40	56
B	0	19	19	23
C	0	24	24	29
D	0	34	34	41
E	0	47	47	56

Evaluation

Hydro-Future has evaluated the traffic and soil loads on the brick arch to determine the maximum factored vertical pressure which may occur. A critical condition will need to be evaluated for all the loading combinations within case 1 and case 2. The worst loading condition from case 1 and case 2 will be evaluated in terms of horizontal pressures.

Case 1

It can clearly be identified (Figure 304) that M1600 will be the critical loading condition with the greatest vertical pressure at points A, B, C, D and E.

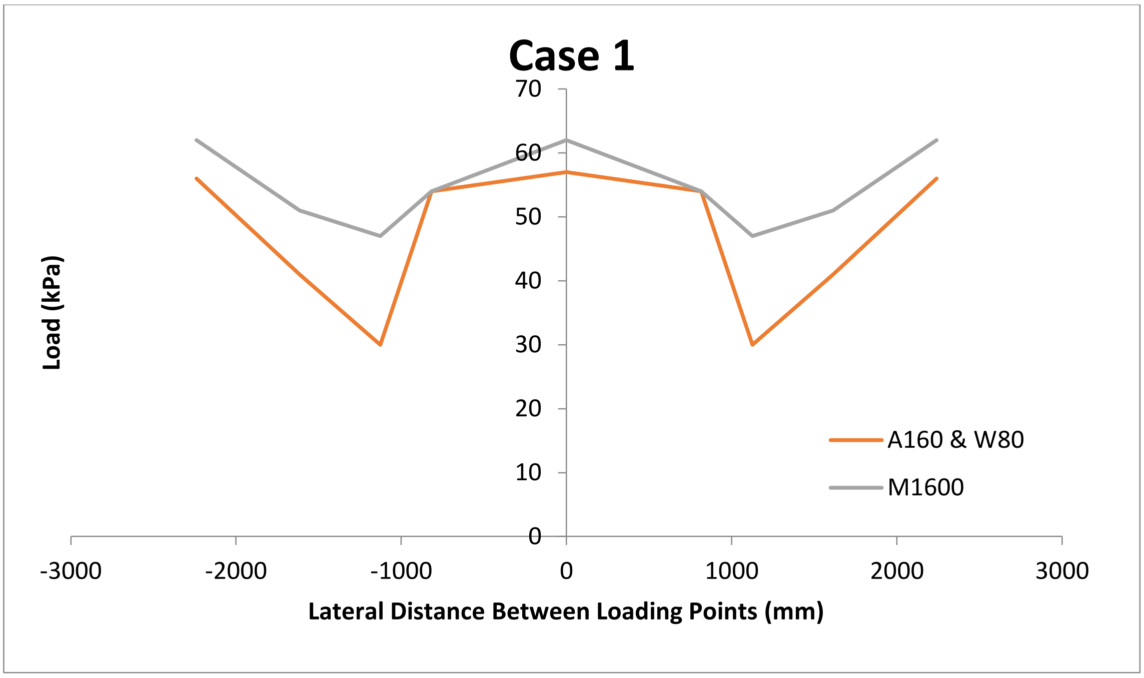

Figure 304: Vertical Pressure Comparison - Case 1

Case 2

As seen below in Figure 305 are the W80, A160 and M1600 factored vertical pressure diagrams for loading case 2. It can clearly be identified that M1600 will be the critical loading condition with the greatest vertical pressure at points B, C, D and E.

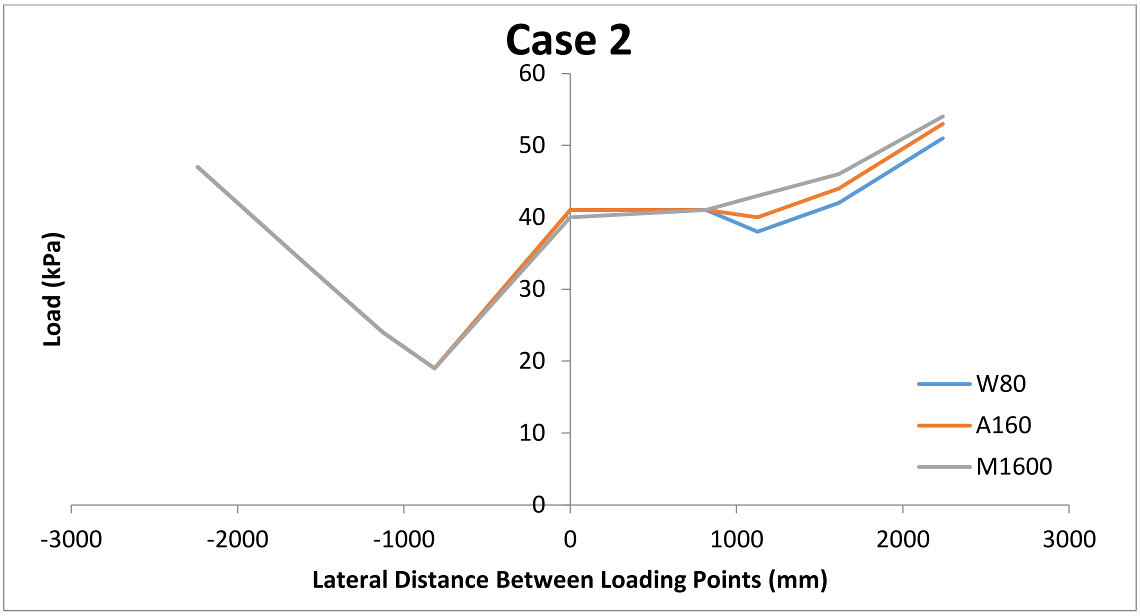

*Figure 305: Vertical Pressure Comparison - Case 2*orizontal Pressures

The earth pressure coefficient (K_a) has been calculated for both soils layers and will be used to determine the horizontal earth pressures for points A, B, C, D and E as well as the pressure at the base of the arch. The resultant horizontal pressure can be calculated using the following equation:

$$\rightarrow \sigma_H = K_a \sigma_v - 2c' \sqrt{K_a}$$

Calculating K_a:

Sandy Gravelly CLAY:

$$K_a = \frac{1 - \sin(\phi)}{1 + \sin(\phi)} = \frac{1 - \sin(17)}{1 + \sin(17)} = 0.42$$

Silty Gravelly SAND:

$$K_a = \frac{1 - \sin(\phi)}{1 + \sin(\phi)} = \frac{1 - \sin(26)}{1 + \sin(26)} = 0.39$$

Case 1

The horizontal pressures acting on the arch will be evaluated for the M1600 tri-axle loading condition, which was deemed critical*:*

$$\rightarrow \sigma_H = K_a \sigma_v - 2c' \sqrt{K_a}$$

$$\rightarrow \sigma_v = 1.2 * (47 + 0.6 * 20) + 1.5 * 4 = 77\ kPa$$

$$\rightarrow \sigma_H = 0.39 * 77 - 2 * 0 * \sqrt{0.42} = 32\ kPa$$

Table 118: M1600 Resultant Horizontal Pressure - Case 1

Location	K_a	Resultant Factored Vertical Pressure: 1.2G + 1.5Q (kPa)	Resultant Horizontal Pressure (kPa)
A	0.42	62	26
B	0.42	54	23
C	0.42	47	20
D	0.39	51	20
E	0.39	62	24
Base	0.39	77	30

Case 2

The horizontal pressures acting on the arch will be evaluated for the M1600 tri-axle loading condition, which was deemed critical. As this loading condition was asymmetrical, the horizontal pressures will vary from each side of the arch will need to be evaluated.

Horizontal Stress at Base:

$\rightarrow \sigma_H = K_a \sigma_v - 2c'\sqrt{K_a}$

$\rightarrow \sigma_v = 1.2 * (47 + 0.6 * 20) + 1.5 * 7 = 81\ kPa$

$\rightarrow \sigma_H = 0.39 * 81 - 2 * 0 * \sqrt{0.42} = 32\ kPa$

Table 119: M1600 Resultant Horizontal Pressure - Case 2(Right Side)

Location	K_a	Resultant Factored Vertical Pressure: 1.2G + 1.5Q (kPa)	Resultant Horizontal Pressure (kPa)
A	0.42	56	24
B	0.42	56	24
C	0.42	57	24
D	0.39	59	23
E	0.39	67	26
Base	0.39	81	32

Horizontal Stress at Base:

$\rightarrow \sigma_H = K_a \sigma_v - 2c'\sqrt{K_a}$

$\rightarrow \sigma_v = 1.2 * (47 + 0.6 * 20) + 1.5 * 0 = 71\ kPa$

$\rightarrow \sigma_H = 0.39 * 71 - 2 * 0 * \sqrt{0.42} = 32\ kPa$

Location	K_a	Resultant Factored Vertical Pressure: 1.2G + 1.5Q (kPa)	Resultant Horizontal Pressure (kPa)
A	0.42	56	24
B	0.42	23	10

C	0.42	29	12
D	0.39	41	16
E	0.39	56	22
Base	0.39	71	28

Table 120: M1600 Resultant Horizontal Pressure - Case 2 (Left Side)

3.5. Calculations for Reinforced Concrete

Client	Tonkin Consulting	Date: 1/06/2015
Project	North Terrace Drainage System	Pages: 483-513
Subject	Reinforced Concrete Pipe Connection	By: Jeremy Bemmerl
Reviewed By:	David Argent	Date: 1/06/2015
Approved By:	Eriny Abdelraouf	Date: 7/06/2015

3.5.1.1. Material Properties

Compressive Strength

The design compressive strength of reinforced concrete section will comply with section 4.3 of AS 3600:

$Exposure\ Classification = B1\ (Surfaces\ of\ members\ in\ water)$

$\rightarrow f_c' = 40\ MPa$

Hence $N40$ normal – class concrete will be used throughout the reinforced concrete section as stated in section 1.3.8 of AS 1379.

Reinforcement Cover

The minimum design reinforcement cover of the reinforced concrete section will comply with section 4.10.3 of AS 3600:

$Exposure\ Classification = B1$, *(Surface of Members in Water)*

$f_c' = 40\ MPa$

$\rightarrow Minimum\ Cover = 30\ mm$, *(Table 4.10.3.2)*

Wet Properties

The concrete slump at the time of pouring will be $110\ mm$ to ensure adequate compaction, constructability and workability as in accordance with section 1.5.3.2 (b) of AS 1379. The specified slump of $110\ mm$ will have allowable tolerances of $\pm 20\ mm$ in accordance with section 5.1 of AS 1379 recorded using a slump test which will be carried out before the concrete pour in accordance with AS 1012.3.1.

The maximum course aggregate size to be used within the concrete mixture will be equal to $20\ mm$ (default value) as stated in section 1.5.3.2. (C) of AS 1397. For all reinforcement spacing calculations the distance between compression, tension and shear steel will not be less than 30 mm to allow for tolerances within the aggregate sizes.

3.5.1.2. Column Design (A & C)

The columns A and C will need to be designed to ensure they can withstand combined loadings, flexure and shear stresses which can be seen in the strand7 results.

Combined Actions

Referring to Figure 136 and Figure 138 for case 1, and Figure 139 and Figure 141 for case 2, there is a combined action acting on the top of the columns in the form of a compressive axial force and a bending moment. Both these loads will need to be considered together to find the most adverse effect on the columns A and C. The combined design actions can be seen below in Figure 306

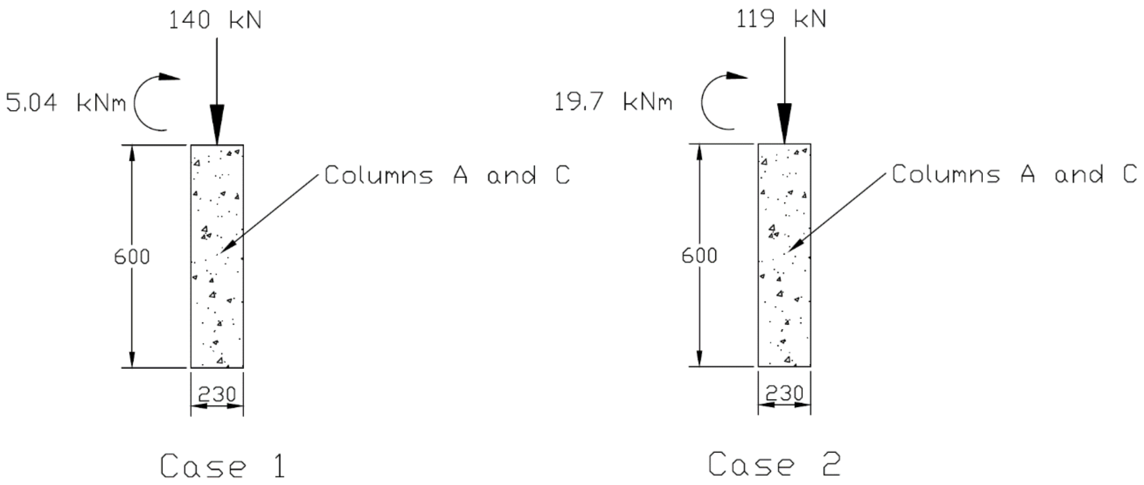

Figure 306: Combined Actions - Case 1 & 2

The design of a reinforced concrete column subject to bending and axial tension or compression will be designed in accordance with section 10 of AS 3600. The failure modes as seen below will need to be determined using a column interaction diagram to check the capacity under combined loadings.

Failure Modes:

- N_{uo}
- M_{uo}
- M_{ub} & N_{ub}
- $\boldsymbol{M_u}$ & $\boldsymbol{N_u}$

Minimum Bending Requirements

The minimum design bending moment about each principle axis will not be less than $N^{*}0.05D$ in accordance with section 10.1.2 of AS 3600.

$$Case\ 1\ M^{*}{}_{min} = \ (140)(0.05)(0.23) = \ 1.16\ kNm$$

Bending moment from Strand7 output is critical: $5.04 > 1.16$

$$Case\ 2\ M^{*}{}_{min} = \ (119)(0.05)(0.23) = \ 1.37\ kNm$$

Bending moment from Strand7 output is critical: $19.7 > 1.37$

Minimum Reinforcement

The minimum reinforcement within a reinforced concrete column will not be less then $0.01A_g$ in accordance with section 10.7 of AS 3600.

$$Minimum\ Reinforcement = (0.01)(230)(230) = 529\ mm^2$$

According to the ARC Reinforcement Handbook, *Adopt 4N16,* $Ast\ = \ 800\ mm^2$

Ultimate Bending Capacity (M_{uo})

The bending strength of the column will be determined with the consideration that the ultimate axial compressive strength is equal to zero:$N_u = 0$. As there is compression as well as tension steel the capacity of the compression steel will need to be taken into account. After several iteration the neutral axis was found to be located at 41.5 mm below the concrete compressive surface, hence the compressive steel (A_{sc}) is assumed to be in tension as seen below in Figure 307.

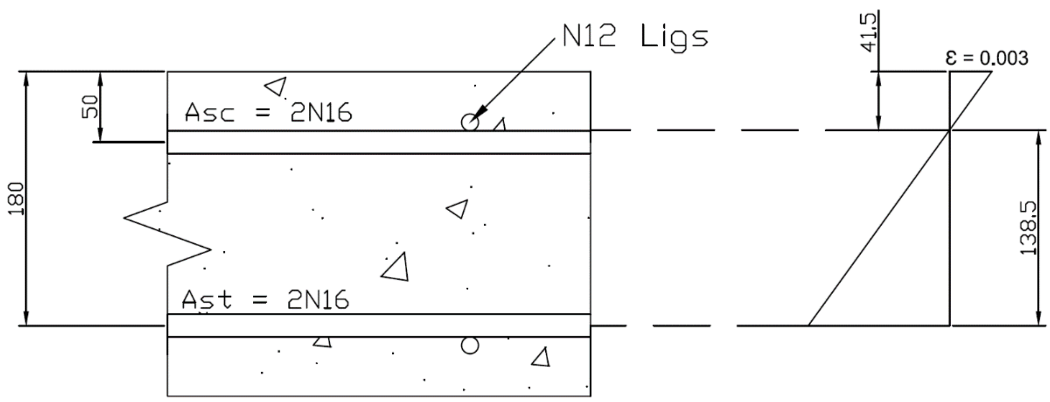

Figure 307: Neutral Axis Depth

Check the strain in the steel:

$$\frac{\varepsilon_c}{50-41.5}=\frac{0.003}{41.5}, \varepsilon_c = 0.000614$$

As $0.000614 < 0.0025$, compression steel is not at yield.

$$\frac{\varepsilon_t}{138.5}=\frac{0.003}{41.5}, \varepsilon_c = 0.01$$

As $0.01 > 0.0025$, tension steel is at yield.

Assuming all tensile forces are equal to compressive forces, the compressive steel is in tension and the neutral axis is located at $41.5\ mm$ below the compression flange:

$$T = C$$

$$\rightarrow A_{st}f_{sy} + A_{sc}E_s\varepsilon_c = \alpha_2 f_c'\gamma K_u db$$

The coefficients $\alpha_2\ and\ \gamma$ will be calculated in accordance with section 8.1.3 of AS 3600.

$$\alpha_2 = 1.0 - 0.003f_c', \qquad 0.67 \le \alpha_2 \le 0.85$$

$$\alpha_2 = 1.0 - 0.003 * 40 = 0.88 = 0.85$$

$$\gamma = 1.05 - 0.007f_c', \qquad 0.67 \le \gamma \le 0.85$$

$$\gamma = 1.05 - 0.007 * 40 = 0.77$$

$$\rightarrow (400)(500) + (400)(200 * 10^3)(0.000614) = (0.85)(40)(0.77)(41.5)(230)$$

$$\rightarrow 249.1 \approx 249.9$$

Neutral axis is at 41.5, hence $K_u d = 41.5$

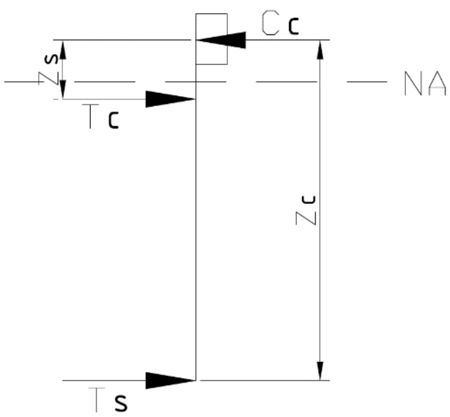

Figure 308: Force Locations & Moment Arms

Referring to Figure 308 the capacity of the column in pure bending can be calculated:

$$M_{uo} = T_s Z_c + T_c Z_s$$

$$T_S = (400)(500)(10^{-3}) = 200\ kN$$

$$T_c = (400)(200 * 10^3)(0.000614)(10^{-3}) = 49.1\ kN$$

$$M_{uo} = (200)(10^3)(180 - (0.5)(0.77)(41.5)) + (49.1)(10^3)(50 - (0.5)(0.77)(41.5)) = 34.5\ kNm$$

Balanced Failure (N_{ub} & M_{ub})

The balanced failure point occurs when a column can either fail in tension or compression based on the magnitude of the axial force and the bending moment. Balanced failure occurs when $K_{ub} = 0.545$:

$$NA = K_{ub} d = (0.545)(180) = 98.1\ mm$$

Hence the compressive steel is in compression as opposed to pure bending when it was subject to tension forces. The location of the new compressive and tensile strains can be seen below in *Figure 309*

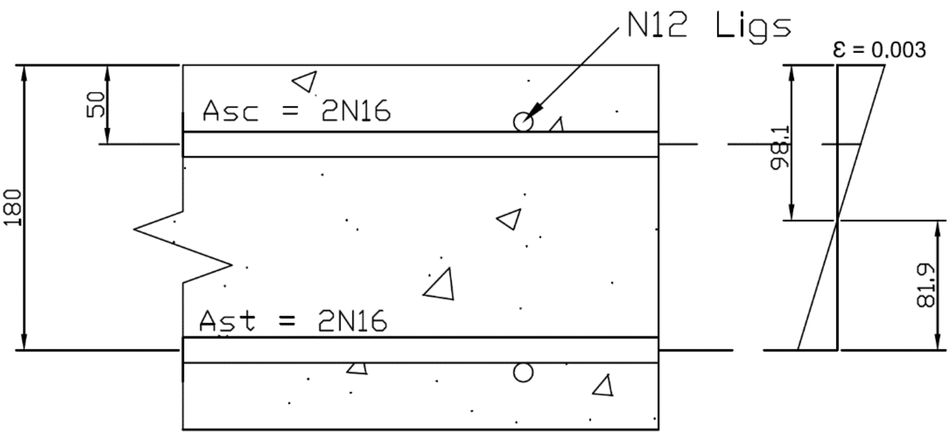

Figure 309: Balanced Failure Neutral Axis

Check strain in steel:

$$\frac{\varepsilon_c}{98.1 - 50} = \frac{0.003}{98.1}, \varepsilon_c = 0.00147$$

As $0.00147 < 0.0025$, compression steel is not at yield.

$$\frac{\varepsilon_t}{89.1} = \frac{0.003}{98.1}, \varepsilon_c = 0.0027$$

As $0.0027 > 0.0025$, tension steel is at yield.

Referring to Figure 309 the location of forces in which balanced failure will occur. As the compressive steel is not at yield, the compressive force will be calculated using the modulus of elasticity.

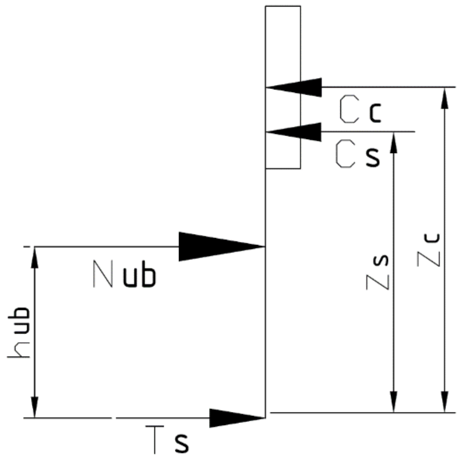

Figure 310: Balanced Failure – Forces

Referring to Figure 310 summing the forces in the x direction:

$\Sigma F_x = 0$

$N_{ub} + T_s - C_s - C_c = 0$

$N_{ub} = -A_{st}f_{sy} + A_{sc}E_s\varepsilon_c + \alpha_2 f_c' \gamma K_u db$

$N_{ub} = -(400)(500) + (400)(200 * 10^3)(0.00147) + (0.85)(40)(0.77)(0.545)(180)(230) = 508\ kN$

Taking the moments about the tensile steel *(T_s):*

$-N_{ub}h_{ub} + C_s(Z_s) + C_cZ_c = 0$

$Z_c = d - 0.5\,\gamma K_u d = 180 - (0.5)(0.77)(98.1)) = 142.2\ mm$

$Z_s = d - d_c = 180 - 50 = 130\ mm$

$(582)(10^3)(h_{ub}) = (400)(200 * 10^3)(0.00147)(130) + (0.85)(40)(0.77)(0.545)(180)(230)(142.2)$

$h_{ub} = 144.5\ mm$

Calculating M_{ub}:

$M_{ub} = N_{ub}e_b$

$e_b = h_{ub} - v$

$v = distance\ from\ T_s\ to\ column\ centroid\ = 180 - \left(\frac{230}{2}\right) = 65mm$

$e_b = 144.5 - 65 = 79.5\ mm$

$M_{ub} = (508)(10^3)(79.5) = 40.4\ kNm$

Decompression Point ($\boldsymbol{N_u}$ & $\boldsymbol{M_u}$)

The decompression point occurs when the neutral axis is located at the tensile steel indicating that$K_u = 1$. The location of the new compressive and tensile strains can be seen in Figure 311

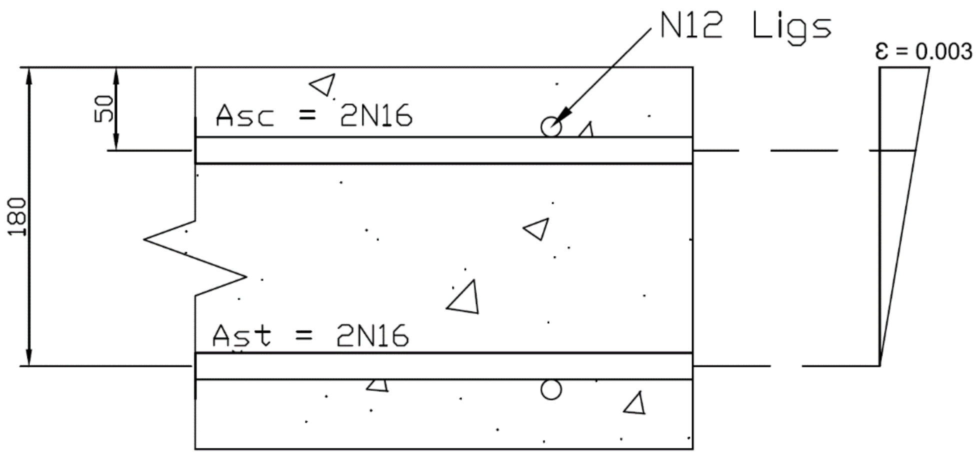

Figure 311: Decompression Point - Neutral Axis

Check strain in steel:

$$\frac{\varepsilon_c}{180 - 50} = \frac{0.003}{180}, \varepsilon_c = \ 0.00216$$

As $0.00216 < 0.0025$, compression steel is not at yield.

$\varepsilon_t = 0$, neutral axis at tension steel.

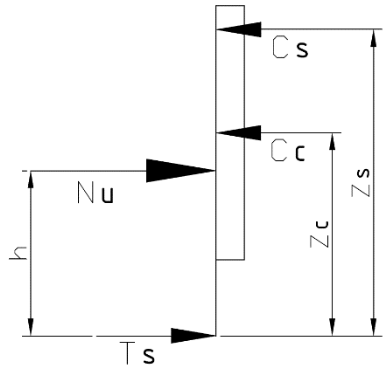

Figure 312: Decompression Point – Forces

Referring to Figure 312 summing the forces in the x direction:

$\Sigma F_x = 0 \;\; And \;\; T_s = 0$

$N_u - C_s - C_c = 0$

$N_u = A_{sc} E_s \varepsilon_c \; + \; \alpha_2 f_c' \gamma K_u d b$

$N_u = (400)(200 * 10^3)(0.00216) + (0.85)(40)(0.77)(1)(180)(230) = \; 1257 \; kN$

Taking the moments about the tensile steel *(T_s):*

$-N_u h + C_s(Z_s) + C_c Z_c = 0$

$Z_c = d - 0.5 \, \gamma K_u d = 180 - (0.5)(0.77)(180)) = 110.7 \; mm$

$Z_s = d - d_c = 180 - 50 = 130 \; mm$

$(1257)(10^3)(h) = (400)(200 * 10^3)(0.00216)(130) + (0.85)(40)(0.77)(1)(180)(230)(110.7)$

$h = 113.3 \; mm$

Calculating M_{ub}*:*

$M_u = N_u e_b$

$e_b = h - v$

$v = distance \; from \; T_s \; to \; column \; centroid \; = 180 - \left(\frac{230}{2}\right) = 65 \; mm$

$e_b = 113.3 - 65 = 48.3 \; mm$

$M_u = (1257)(10^3)(48.3) = 60.7 \; kNm$

Squash Load (N_{uo})

The squash load will be calculated to understand the columns capacity to undergo pure axial compression force, within this calculation $M = 0$.

$$N_{uo} = \alpha_1 f_c' A_g + f_{sy} A_s$$

$$A_g = 230 * 230 = 52900\ mm^2$$

$$A_s = 4N16 = 800mm^2$$

The coefficient α_1 will be calculated in accordance with section 10.6.2.2 of AS 3600.

$$\alpha_1 = 1.0 - 0.003 f_c', \qquad 0.72 \leq \alpha^2 \leq 0.85$$

$$\alpha_1 = 1.0 - 0.003 * 40 = 0.88 = 0.85$$

$$N_{uo} = (0.85)(40)(52900) + (500)(800) = 2199\ kN$$

Interaction Diagram

To check the capacity of the column under combined actions the ultimate bending capacity, balance failure, decompression point and the squash load are required to be factored:

$$As\ 1348 > 582, N_u > N_{ub}, hence\ \phi = 0.6$$

Squash Load:

$$\phi N_{uo} = 0.6 * 2199 = 1319\ kN$$

Ultimate Bending Capacity:

$$\phi M_{uo} = 0.6 * 34.5 = 20.7\ kNm$$

Balance Failure:

$$\phi M_{ub} = 0.6 * 40.4 = 24.2\ kNm$$

$$\phi N_{ub} = 0.6 * 508 = 305\ kN$$

Decompression Point:

$$\phi M_u = 0.6 * 60.7 = 36.4\ kNm$$

$$\phi N_u = 0.6 * 1257 = 754\ kN$$

As seen below in Figure 313 is the factored column interaction diagram which shows the capacity of columns A and C and the combined action effect of case 1 and case 2. It can be seen that both cases fall within the diagram indicating the chosen design is sufficient.

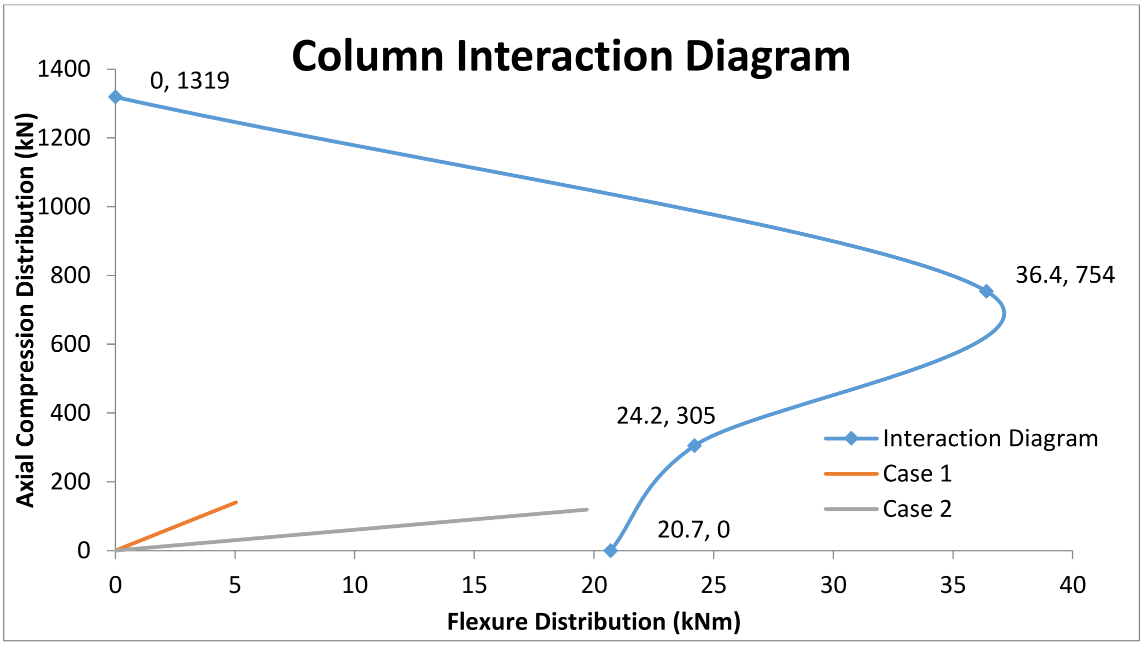

Figure 313: Column Interaction Diagram

Design for Flexure

The ultimate flexural strength of a concrete member in flexure will be design in accordance with section 8 of AS 3600. As these columns contain compression and tension steel the flexural design will be undertaken as a dual reinforced beam.

$M^* = 19.7\ kNm$, (*Figure 139: BMD, Case 2)*

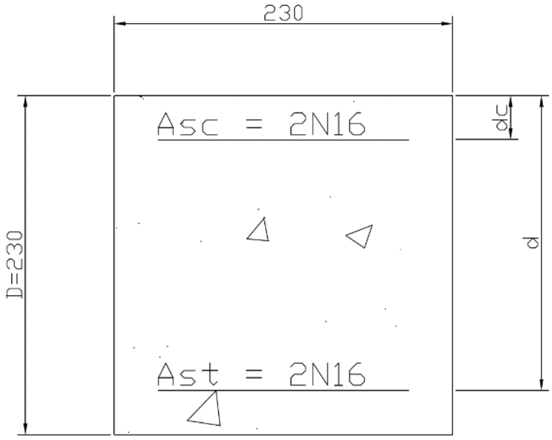

Figure 314: Beam Cross Section

Ultimate Bending Capacity $(\phi \boldsymbol{Mu})$

$$M_u = \frac{M^*}{\phi}$$

Where $\phi = 0.8$ in accordance with Table 2.2.2 of AS 3600.

$$M_u = \frac{19.7}{0.8} = 24.6\ kNm$$

Effective Depths $(\boldsymbol{d}\ \&\ \boldsymbol{d_c})$

The diameter of the ligatures will be assumed to be N12 in the effective depth calculation. An assumption that N12 steel reinforcing bars will be used.

$$d = D - cover - ligature\ diamater - \left(\frac{1}{2}\right) bar\ diamater = 230 - 30 - 12 - \left(\frac{16}{2}\right) = 180mm$$

$$d_c = cover + ligature\ diamater + \left(\frac{1}{2}\right) bar\ diamater = 30 + 12 + \left(\frac{16}{2}\right) = 50mm$$

Equilibrium

As this is a dual reinforced beam the neutral axis depth is unknown, hence several iterations were performed until the neutral axis was found to be located at 41.5 mm below the concrete compressive surface, hence the compressive steel (A_{sc}) is assumed to in tension as seen in Figure 315.

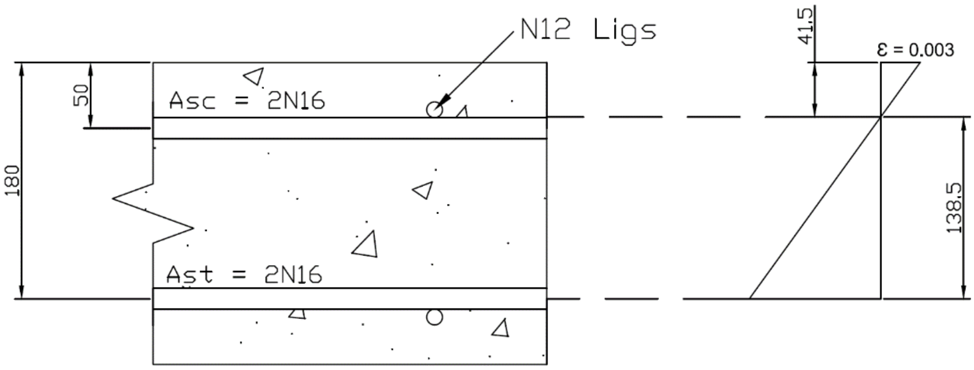

Figure 315: Neutral Axis Depth

Referring to Figure 315, check the strain in the steel:

$$\frac{\varepsilon_c}{50 - 41.5} = \frac{0.003}{41.5}, \varepsilon_c = 0.000614$$

As $0.000614 < 0.0025$, compression steel is not at yield.

$$\frac{\varepsilon_t}{138.5} = \frac{0.003}{41.5}, \varepsilon_c = 0.01$$

As $0.01 > 0.0025$, tension steel is at yield.

Assuming all tensile forces are equal to compressive forces, the compressive steel is in tension and the neutral axis is located at $41.5\ mm$ below the compression flange:

$$\rightarrow A_{st}f_{sy} + A_{sc}E_s\varepsilon_c = \alpha_2 f_c' \gamma K_u db$$

$$\rightarrow (400)(500) + (400)(200 * 10^3)(0.000614) = (0.85)(40)(0.77)(41.5)(230)$$

$$\rightarrow 249.1 \approx 249.9$$

$K_u = \frac{41.5}{180} = 0.23$, ductility okay

Neutral axis is at 41.5, hence $K_u d = 41.5$

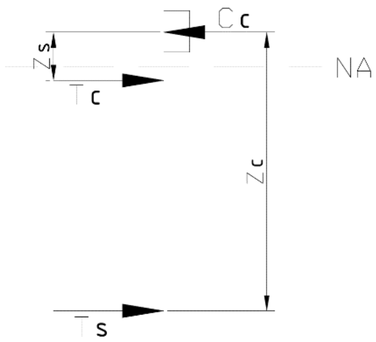

Figure 316: Force Locations & Moment Arms

Referring to Figure 316 the capacity of the column in pure bending can be calculated:

$M_u = T_s Z_c + T_c Z_s$

$T_S = (400)(500)(10^{-3}) = 200\ kN$

$T_c = (400)(200 * 10^3)(0.000614)(10^{-3}) = 49.1\ kN$

$M_u = (200)(10^3)(180 - (0.5)(0.77)(41.5)) + (49.1)(10^3)(50 - (0.5)(0.77)(41.5)) = 34.5\ kNm$

$\phi M_u = 0.8 * 34.5 = 27.6 kNm$

As $27.6 > 19.7, \phi M_u > M^*$*, flexural strength okay*

Bar Spacing

The reinforcing bar spacing will need to be wide enough to allow room for the course aggregates within the concrete.

$$s = \frac{b - (No. Bars)(Bar\ Diamater) - (2)(Cover) - (2)(Ligature\ Diamater)}{(No. Bars) - 1}$$

$s = \frac{230-(2)(16)-(2)(30)-(2)(12)}{(2)-1} = 114mm$, Spacing Okay

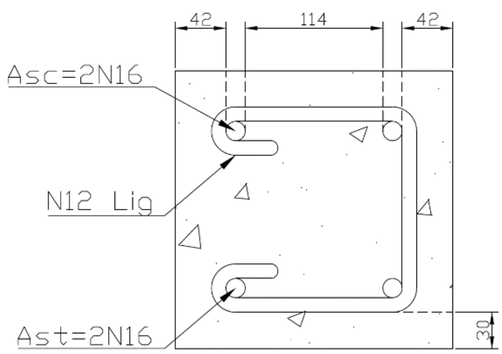

Figure 317: Column A & C, Flexural Design

Design for Shear

The ultimate shear strength of a reinforced concrete member will be designed in accordance with section 8.2 of AS 3600.

$V^* = 40\ kN$, (*Figure 140: SFD, Case 2*)

Concrete Shear Capacity

The shear capacity of a concrete member excluding shear reinforcement will be calculated in accordance with section 8.2.7 of AS 3600:

$$V_{uc} = \beta_1\beta_2\beta_3 b_v d_o f_{cv}\left(\frac{A_{st}}{(b_v)(d_o)}\right)^{\frac{1}{3}}$$

$$\beta_1 = 1.1\left(1.6 - \frac{d_o}{1000}\right)$$

$$d_o = d = 192$$

$$\beta_1 = 1.1\left(1.6 - \frac{180}{1000}\right) = 1.562$$

$$\beta_2 = 1 + \left(\frac{N^*}{14Ag}\right) = 1 + \frac{119 * 10^3}{14 * 52900} = 1.16$$

$$\beta_3 = 1$$

$$b_v = b = 230\ mm$$

$$f_{cv} = f_c'^{\frac{1}{3}} \leq MPa$$

$$f_{cv} = (40)^{\frac{1}{3}} = 3.4\ MPa$$

$$V_{uc} = (1.562)(1.16)(1)(230)(180)(3.4)\left(\frac{400}{(230)(180)}\right)^{\frac{1}{3}} = 54.3\ kN$$

In accordance with section 8.25 of AS 3600, $V^* \leq 0.5\phi V_{uc}$ where $\phi\ =\ 0.7$ from table 2.2.2.

$$0.5\phi V_{uc} = 0.5 * 0.7 * 54.3 = 19kN$$

As $19 < 40,\ 0.5\phi V_{uc} < V^*$, concrete shear capacity not okay

As the concrete member does not have enough shear capacity, the minimum shear reinforcement requirements will need to be met.

Minimum Shear Steel Capacity

A reinforced concrete member containing minimum shear reinforcement will be designed in accordance with section 8.2.9 of AS 3600.

$$V_{u,min} = V_{uc} + 0.1\sqrt{f_c'}b_v d_o \geq V_{uc} + 0.6 b_v d_o$$

$$V_{u,min} = (54.3)(10^3) + (0.1)\left(\sqrt{(40})(230)(180) \geq (54.3)(10^3) + (0.6)(230)(180)$$

$$V_{u,min} = 80.5 \geq 79.1$$

$$V_{u,min} = 80.5$$

$$\phi V_{u,min} = (0.7)(80.5) = 56.4\ kN$$

As $56.4 > 40$, $\phi V_{u,min} > V^*$, concrete with minimum shear steel capacity okay

As the concrete member containing the minimum shear reinforcement does have enough shear capacity, the minimum shear reinforcement will need to be designed.

Shear Steel Design

The minimum shear reinforcement will be designed in accordance with section 8.2.8 of AS 3600.

$$A_{sv,min} = \frac{0.06\sqrt{f_c'}b_v s}{f_{sy}} \geq \frac{0.35 b_v s}{f_{sy}}$$

$$\frac{A_{sv,min}}{s} = \frac{(0.06)\sqrt{40}(230)}{50} \geq \frac{(0.35)(230)}{500}$$

$$\frac{A_{sv,min}}{s} = 0.174 \geq 0.161$$

$$\frac{A_{sv,min}}{s} = 0.174 mm^2/mm$$

$$Try\ 1N12\ ligature = 2 * 110 = 220\ mm^2$$

$$\frac{220}{s} = 0.174$$

$$s = 1264\ mm$$

$$Max\ Longitudinal\ Spacing\ s = \min(500\ or\ 0.75D)$$

$$0.75D = 172.5\ mm$$

$$s = 172.5$$

Adopt N12 ligatures at 172.5 cts. As the full length of the column extends into the arch the ligatures will be spaced for the full length of the column at $1006\,mm$ and not just the vertical length of $600\,mm$. Assuming the ligatures will be spaced evenly and they will have a minimum cover of 30 mm from the top and bottom of the column:

$$Amount\ of\ Ligatures = \frac{1066 - (2)(30)}{172.5} = 5.48 = 7\ ligatures$$

$$s = \frac{1066 - (2)(30)}{7 - 1} = 167.6\ \approx 165\,mm$$

Adopt 7N12 ligatures at 165 cts.

3.5.2. Beam Design (B)

3.5.2.1. Finite Element Analysis

A finite element analysis was conducted to calculate the load transfer through the arch and onto the top of beam- B as seen below in Figure 318.

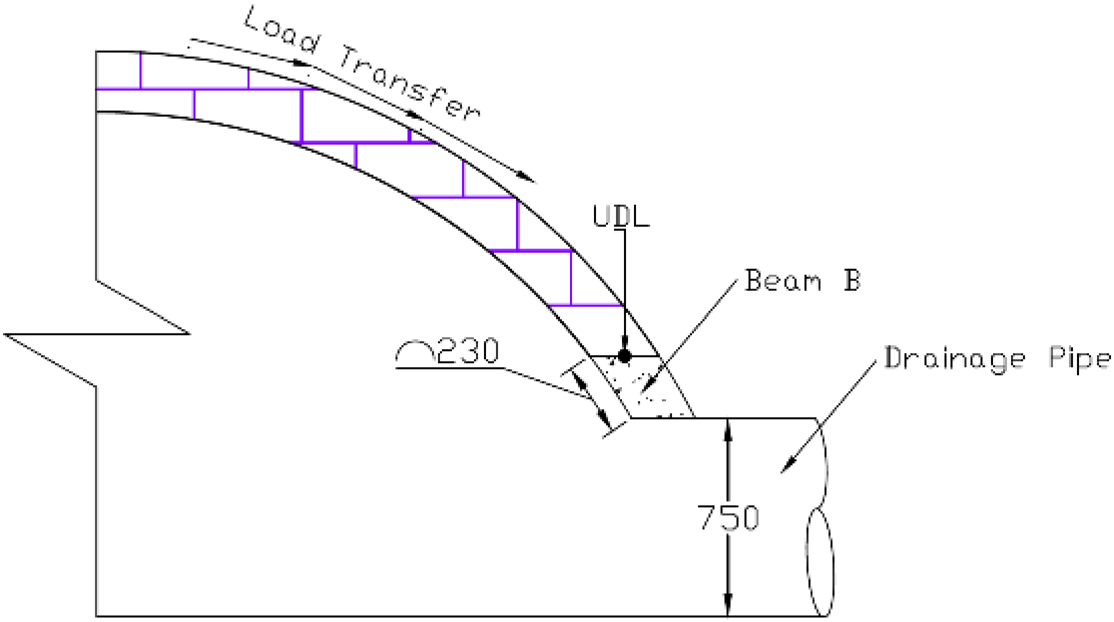

Figure 318: Beam B Load Transfer