3.5.2.2. Strand7 Results

Within strand 7 the brick arch was modelled up until beam – B to determine the forces transferred to the top of the beam from the centre of the arch as seen below in Figure 319 and Figure 320. Forced below beam B were neglected due to their minimal effect on the structure of interest.

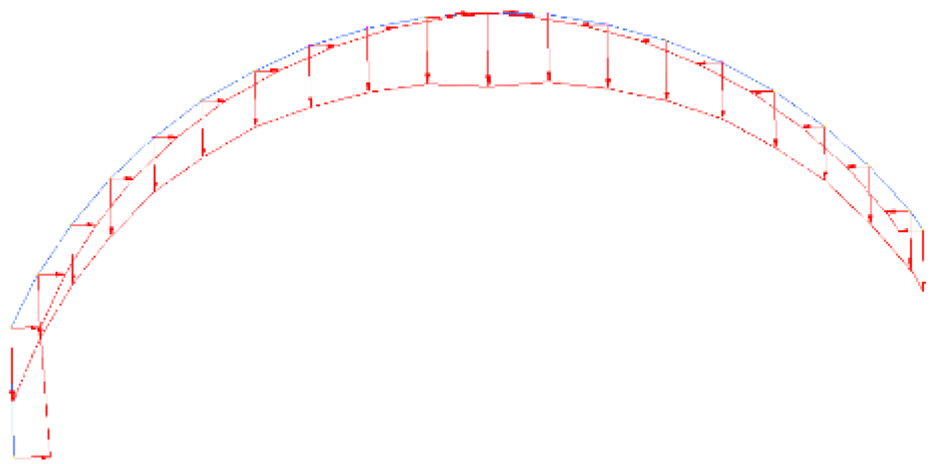


Figure 319: Case 1, Beam B

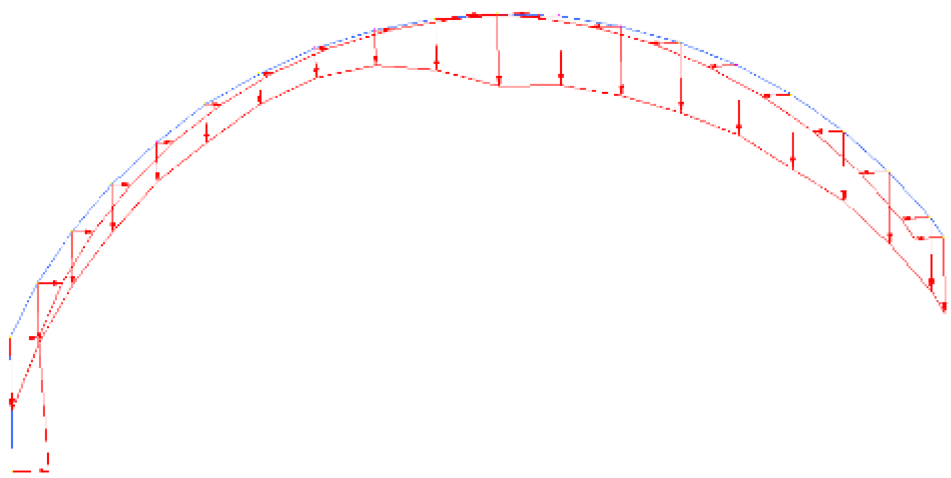


Figure 320: Case 2, Beam B

3.5.2.3. Resultant Forces

As seen below in Table 121 are the X and Y component forces at the top of beam B. The resultant force in terms of the X and Y components which is perpendicular to beam B will be determined to find the maximum bending moment throughout the section. The component forces and beam angle can be seen below in Figure 321

Table 121: Force Components

Load Combinations	F_x (kN)	F_y (kN)
Case 1	$-2.59 * 10^{-10}$	$1.27 * 10^{2}$
Case 2	$-4.317 * 10^{-10}$	$1.17 * 10^{2}$

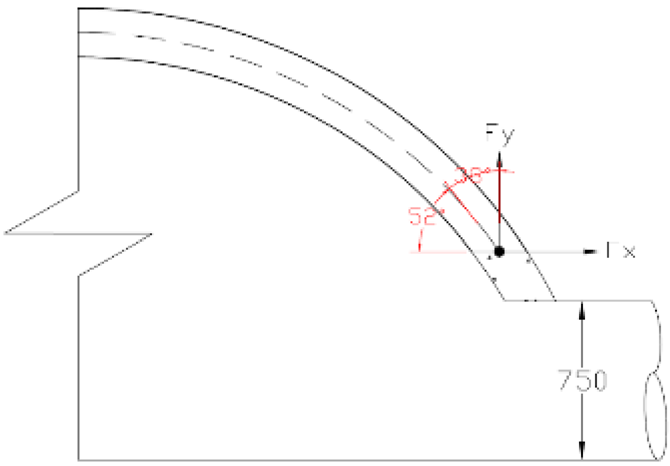


Figure 321: Resultant Forces, Beam B

Resultant Force - Case 1:

$$Cos(52) = \frac{-F_x}{F}$$

$$Cos\ (38) = \frac{F_y}{F}$$

$$Resultant\ Force\ (F) = \frac{-F_x}{Cos(52)} + \frac{F_y}{Cos(38)} = \frac{-2.59 * 10^{-10}}{Cos(52)} + \frac{1.27 * 10^{2}}{Cos(38)} = 161\ kN$$

Resultant Force - Case 2:

$$Cos(52) = \frac{-F_x}{F}$$

$$Cos\ (38) = \frac{F_y}{F}$$

$$Resultant\ Force\ (F) = \frac{-F_x}{Cos(52)} + \frac{F_y}{Cos(38)} = \frac{-4.317 * 10^{-10}}{Cos(52)} + \frac{1.17 * 10^2}{Cos(38)} = 148\ kN$$

As seen above case 1 has a critical load on beam B of *161 kN*, however as the surface between beam B and the sandstone culvert is assumed to be flush this load must be treated as a uniformly distributed load throughout the entirety of the beams length.

$$w^* = \frac{161}{Beam\ Length} = \frac{161}{1.21} = 133\ kN/m$$

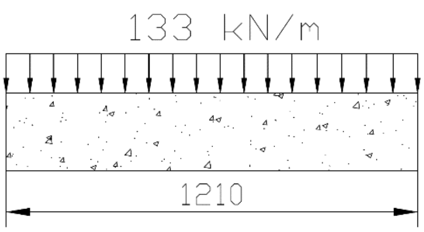


Figure 322: Beam B - UDL

3.5.2.4. Design for Flexure

The ultimate flexural strength of a concrete member in flexure will be design in accordance with section 8 of AS 3600.

$$M^* = \frac{w^* l^2}{8} = \frac{(133)(1.21)^2}{8} = 24.3\ kNm$$

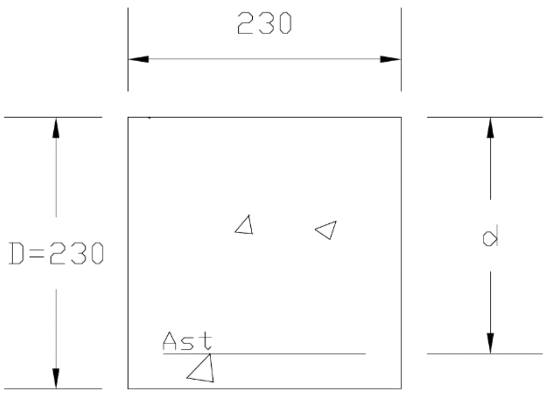


Figure 323: Beam B Cross Section

Ultimate Bending Capacity (ϕM_u)

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

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

$$M_u = \frac{24.3}{0.8} = 30.4\ kNm$$

Effective Depth (d)

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.

$$d = D - cover - ligature\ diamater - \left(\frac{1}{2}\right) bar\ diamater = 230 - 30 - 12 - \left(\frac{12}{2}\right) = 182mm$$

Calculate Tensile Steel ($\mathbf{A_{st}}$)

Assuming the moment arm between the tension steel and the compression flange of the concrete beam = $0.925d$:

$$M_u = T * Z_u$$

$$T = \frac{Mu}{Zu} = \left(\frac{30.4 * 10^6}{0.925(182)}\right) * 10^{-3} = 180.6\ kN$$

$$T = A_{st} f_{sy}$$

f_{sy} = Class N reinforcement from Table 3.2.1 of AS 3600. Deformed reinforcing bars in accordance with AS/NZS 4671 will be used with a yield strength of $500\ MPa$.

$$A_{st} = \frac{T}{f_y} = \frac{180.6 * 10^3}{500} = 361\ mm^2$$

According to the ARC Reinforcement Handbook: $Adopt\ 2N16, Ast\ =\ 400\ mm^2$. As the bar size has changed from N12 to N16 the effective depth will need to be recalculated:

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

Equilibrium:

Assuming all tension forces are equal to compressive forces:

$T = C$

$T = A_{st} f_{sy}$

$A_{st} f_{sy} = \alpha_2 f_c' \gamma K_u db$

$C = \alpha_2 f_c' \gamma K_u db$

$\rightarrow (400)(500) = (0.85)(40)(0.77)(K_u)(180)(230)$

$\rightarrow K_u = 0.18,$ Ductility Okay

Calculate the ultimate flexure capacity (ϕM_u):

$$Z_u = d - \left(\frac{1}{2}\right)\gamma K_u d = 180 - \left(\frac{1}{2}\right)(0.77)(0.18)(180) = 167.5\ mm$$

$$\phi M_u = (0.8)TZ_u = (0.8)(400)(500)(167.5) = 26.8\ kNm$$

As $26.8 > 24.3, \phi M_u > M^*$, 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.

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

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

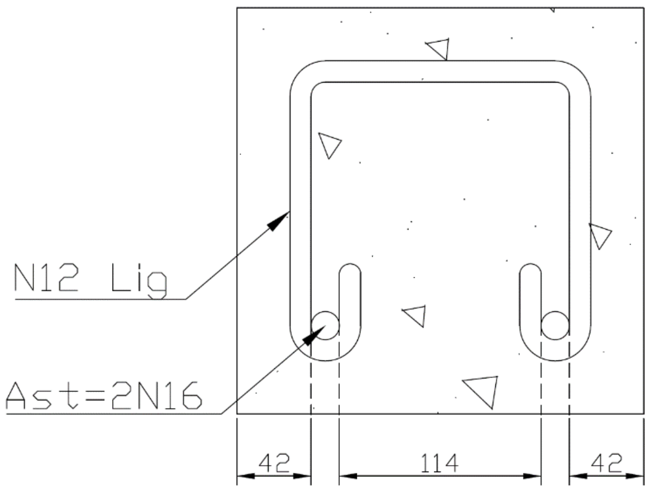


Figure 324: Beam B, Flexural Design

3.5.2.5. Design for Shear

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

$$V^* = \frac{wL}{2} = \frac{(133)(1.21)}{4} = 80.5\ kN$$

3.5.2.5.1.1. 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:

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

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

$$d_o = d = 180$$

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

$\beta_2 = 1$, *No axial forces present*

$$\beta_3 = 1$$

$$b_v = b = 230\ mm$$

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

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

$$V_{uc} = (1.562)(1)(1)(230)(180)(3.4)\left(\frac{400}{(230)(180)}\right)^{\frac{1}{3}} = 46.8\ kN$$

$$0.5\phi V_{uc} = 0.5 * 0.7 * 46.8 = 16.4\ kN$$

As $16.4 < 40.2$, $0.5\phi V_{uc} < V^*$, shear capacity not okay

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.

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

$$V_{u,min} = (46.8)(10^3) + (0.1)\left(\sqrt{(40}\right)(230)(180) \geq (46.8)(10^3) + (0.6)(230)(180)$$

$$V_{u,min} = 73 \ \geq 71.6$$

$$V_{u,min} = 73$$

$$\phi V_{u,min} = (0.7)(73) = 51.1\ kN$$

As $51.1 < 80.5,\ \phi V_{u,min} < V^*$, concrete with minimum shear steel capacity not okay

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

Shear Steel Design

The shear reinforcement will be designed in accordance with section 8.2.10 of AS 3600.

$$V^* < \phi(V_{uc} + V_{us})$$

$$\rightarrow V_{us} = \frac{V^*}{\phi} - V_{uc} = \frac{80.5}{0.7} - 46.8 = 68.2\ kN$$

$$V_{us} = \frac{A_{sv} f_{sy} d_o cot\theta_v}{s}$$

$cot\theta_v = 1$, conservative approach

$$(68.2)(10^3) = (A_{sv})\frac{(500)(180)(1)}{s}$$

$$\frac{A_{sv}}{s} = \frac{(68.2)(10^3)}{(500)(180)(1)} = 0.757\ mm^2/mm$$

Try N12 ligatures, $A_{sv} = 2 * 110 = 220\ mm^2$

$$\frac{220}{s} = 0.757$$

$$s = 290\ mm$$

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

$$0.75D = 172.5\ mm$$

$s = 172.5\ mm$

Adopt N12 ligatures at 170 cts. As the length of the beam is 1210 mm and assuming the ligatures will be spaced evenly and they will have a minimum cover of 30 mm from each end of beam B:

$$Amount\ of\ Ligatures = \frac{1210 - (2)(30)}{172.5} = 6.67 = 8\ ligatures$$

$$s = \frac{1210 - (2)(30)}{8 - 1} = 164.3 \approx 165\ mm$$

Adopt 8N12 ligatures at 165 cts.

3.5.2.6. Dowel Bars

Shear Force for Case 1:

$$N_{Rc} = \frac{161}{1.21} * \frac{1.21}{2} = 80.5\ kN$$

$N_s = 140\ kN$, *Figure 138: AFD, Case 1*

$$V^* = 140 - 80.5 = 59.5\ kN$$

Shear Force for Case 2:

$$N_{Rc} = \frac{148}{1.21} * \frac{1.21}{2} = 74\ kN$$

$N_s = 134\ kN$, *Figure 141: AFD, Case 2*

$$V^* = 134 - 74 = 60\ kN$$

Hence load case 2 is critical with a shearing resistance of 60 kN between the reinforced section and the sandstone culvert.

The dowel bars are to be designed in accordance with AS 4100 – 1998 section 5.11:

$V^* < \phi V_v \quad where \quad V_v = V_u$ If the shear distribution is approximately uniform. Assuming a uniform stress distribution throughout the dowel bars:

$$V_u = V_w \quad if \quad \frac{d_p}{t_w} < \frac{82}{\sqrt{\frac{f_y}{250}}}$$

Assuming N12 reinforcing bars are to be used:

$$\frac{12}{12} < \frac{82}{\sqrt{\frac{500}{250}}} = 1 < 58$$

Hence $V_u = V_w$ where V_w will be the nominal shear yield capacity of the section which is undergoing shear and will be designed in accordance with section 5.11.4 of AS 4100 – 1998:

$$V_w = 0.6 f_y A_w$$

$$A_w = 1N12 = 110\ mm^2$$

$$V_w = (0.6)(500)(110) = 33\ kN$$

Try 3N12 bars, $V_w = (3)(33) = 99\ kN$

$\phi = 0.9$, in accordance with table 3.4 of AS 4100 – 1998

$$\phi V_w = (0.9)(99) = 89.1\ kN$$

$A\ 89.1 > 60,\ \phi V_w > V^*$ Shear Capacity Okay

As the sandstone arch culverts structural integrity and longevity is a high priority to Hydro – Future, the dowel bars will be designed conservatively to ensure a clean connection between the RC concrete and the sandstone:

Adopt 5N12 dowel bars at 175 cts for column A and C

Adopt 6N12 dowel bars at 175 cts for beam B

The dowel bars will connect into a hole in the sandstone arch which compromises of a $30mm$ diameter at a depth of 50 mm with an additional $50mm$ hole at a diameter of $12mm$ to ensure the dowel bar is kept central within the $100mm$ deep hole. The dowel bars will be adhesively connected to the sandstone using epoxy grout designed in accordance with section 17.1.8 of AS 3600.

3.5.2.7. Design Summary

Hydro-Future has ensured the reinforced concrete section which has been designed as 2 columns and a beam is able to withstand the live and dead loads which result in flexure, shear and axial stresses. The design summary for each member can be seen below in Table 122.

Table 122: Member Reinforcement Design Summary

Design Action	Column A			Column C			Beam B		
	A_{st}	A_{sc}	A_{sv}	A_{st}	A_{sc}	A_{sv}	A_{st}	A_{sc}	A_{sv}
Flexural and Axial	2N16	2N16	0	2N16	2N16	0	0	0	0
Flexural	2N16	2N16	0	2N16	2N16	0	2N16	0	0
Shear	0	0	7N12 @165	0	0	7N12 @165	0	0	8N12 @165

The connection between the reinforced concrete section and sandstone culvert was designed to resist shear loading and to ensure compatibility between the 2 structures. The connection design summary can be seen below in Table 123.

Table 123: Connection Reinforcement Design Summary

Design Action	Column A	Column C	Beam B
Shear	5N12 @ 175	5N12 @ 175	6N12 @ 175

3.5.2.8. Reinforcement Detailing

To ensure all the designed steel reinforcement is practical and constructible whilst still maintaining the design requirements including spacing, cover and length a reinforcement detailing plan has been devised by Hydro-Future to maintain the structures strength and allow for installation of the reinforcement.

3.5.2.9. Column A & C

Column A and C span 600 mm vertically including a 407 mm span into the arch and beam B however the critical span which will undergo shear and bending can be seen below in Figure 325. The connection between the beam and column will not be of a critical concern due to the structural stability of two members meeting and the change in bending and shear which occur at the connection point.

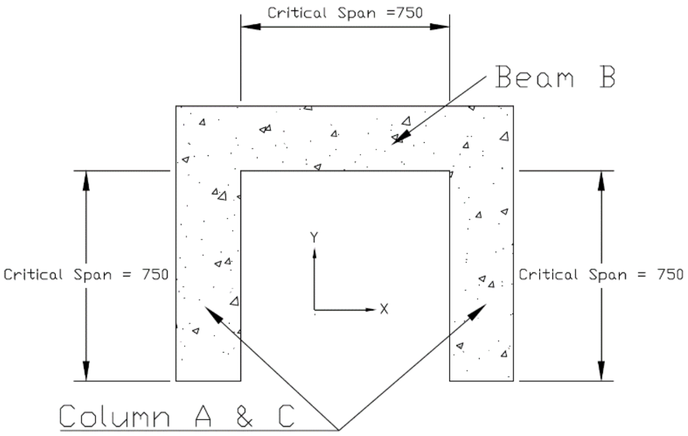


Figure 325: Critical Spans

Bar Bending

As beam B is on an angle due to its location throughout the arch the reinforcement will be required to be bent to ensure the reinforcement flows throughout column A and C. The reinforcing bars will be bent in accordance with section 17.2.3.(a) of AS 3600, which states that the reinforcement will be bent cold around a pin of diameter specified in accordance with clause 17.2.3.2 to ensure no mechanical weakening of the bar is induced. In accordance with clause 17.2.3.2(c) the pin used to bend a reinforcing bar of diameter less than or equal to $16mm$ will be:

$$4d_b = (4)(16) = 64\,mm$$

The dimensions of the reinforcing bend will be specified in the reinforcement design drawings as stated in clause 17.2.3.2(c).

Reinforcement Lapping

The longitudinal reinforcement located in columns A and C will be extended into the sandstone base $100mm$ as specified for the dowel bars to ensure a sufficient connection at the bottom and top of the column. The extension of longitudinal reinforcing bars into the sandstone will also ensure the steel is held in place during concrete placement. However as the longitudinal reinforcing bars throughout the column will extend into the sandstone arch the reinforcement will need to be installed in 2 sections and lapped at the columns centre, as the critical flexure design caused bending about the X axis the reinforcement will be lapped in the X direction to ensure the flexural capacity of the column is not affected. The lap development length of the reinforcement bars will be designed in accordance with section 13.1.5 of AS 3600 with the assumption that the longitudinal reinforcing bars with primarily be in compression.

Basic Development Length:

$$L_{sy.cb} = \frac{(0.22)(f_{sy})}{\sqrt{f_c'}} d_b > 0.0435 f_{sy} d_b \ or\ 200mm$$

$$L_{sy.cb} = \frac{(0.22)(500)}{\sqrt{40}}(16) > (0.0435)(500)(16)\ or\ 200mm$$

$$L_{sy.cb} = \frac{(0.22)(500)}{\sqrt{40}}(16) > (0.0435)(500)(16)\ or\ 200mm$$

$$L_{sy.cb} = 280 > 348\ or\ 200$$

$$L_{sy.cb} = 348\ mm$$

Refined Development Length:

$$L_{sy.c} = k_6 L_{sy.cb}$$

If $\frac{\Sigma A_{tr}}{s} > \frac{A_s}{600}$ then $k_6 = 0.75$ otherwise $k_6 = 1.0$

$$\Sigma A_{tr} = Shear\ Reinforcment\ Along\ L_{sy.cb} = 2N12 = 220\ mm^2$$

$$s = Shear\ Reinforcment\ Spacing = 160\ mm$$

$$A_s = Steel\ Reinforcement = 4N16 = 800\ mm^2$$

As $\rightarrow \frac{220}{160} > \frac{800}{600} = 1.375 > 1.33$ hence $k_6 = 0.75$

$$L_{sy.c} = (0.75)(348) = 261\ mm$$

Hence the lapping length of reinforcement will be taken at 300mm in the x direction.

3.5.2.10. Beam B

As beam B intersects columns A and C the longitudinal reinforcement in will need to be detailed to ensure it does not conflict at the intersection of beam B and column A and C. As the critical bending direction was assumed to be around the Z axis which is perpendicular to the specified x and y axis in Figure 325, the reinforcement cover will be moved along the Z axis to ensure it does not intersect the column reinforcement nor hinder the bending capacity of beam B.

$$New\ Cover = column\ cover + ligature\ diamater + bar\ diameter$$

$$New\ Cover = 30 + 12 + 16 = 58\ mm$$

$$Recalculate\ Spacing\ S = 230 - (2)(58) - (2)(16) = 82mm$$

New longitudinal reinforcement spacing along beam B is sufficient. As the spacing has not changed in the Y direction along beam B the flexural and shear capacity will not be affected.

As shown in Figure 325, beam B has a critical span in which bending and shear will occur over$750mm$. The longitudinal bars through beam B will extend into the sandstone arch as specified for the longitudinal reinforcement of the columns to ensure an adequate connection. Additional top reinforcement which was not required during the design for flexure and shear will be added and extended through beam B and into the sandstone arch for strength and serviceability requirements.

The top and bottom longitudinal reinforcement will extend into the sandstone culvert $100mm$ which includes a $50mm$ deep hole at a diameter of $30mm$ which will be grouted and a $50\ mm$ chamfer hole at a diameter of $16mm$ to ensure the reinforcement is held in place and kept central to allow an even distribution of grout around the steel.

As the longitudinal reinforcement will extend into the sandstone arch, they will be required to be installed in 2 sections with an overlap as specified for the columns. The lap development length of the reinforcement bars will be designed in accordance with section 13.1.2 of AS 3600 with the assumption that the longitudinal reinforcing bars with primarily be in tension.

Basic Development Length:

$$L_{sy.tb} = \frac{0.5k_1k_3f_{sy}d_b}{k_2\sqrt{f_c'}} > 29k_1d_b$$

$$k_1 = 1.0$$

$$k_2 = \frac{132 - d_b}{100} = \frac{132 - 16}{100} = 1.16$$

$$k_3 = 1 - \frac{0.15(c_d - d_b)}{d_b}\ where\ 0.7 < k_3 < 1.0$$

$c_d = \min(c_1, c, \frac{a}{a})$, figure 13.1.2.3(A) of AS 3600

$$c_d = \min(42, 42, 57) = 42$$

$$k_3 = 1 - \frac{(0.15)(41 - 16)}{16} = 0.75625$$

$$L_{sy.tb} = \frac{(0.5)(1.0)(0.75625)(500)(16)}{(1.16)(\sqrt{40})} > (29)(1.0)(16)$$

$$L_{sy.tb} = 412 > 464$$

$$L_{sy.tb} = 464\ mm$$

Refined Development Length:

$$L_{sy.t} = k_4 k_5 L_{sy.tb}$$

$$k_4 = 1 - K\lambda$$

$K = 0.1$, Figure 13.1.2.3(B) of AS 3600

$$\lambda = \frac{\Sigma A_{tr} - \Sigma A_{tr.min}}{A_s}$$

$$\Sigma A_{tr} = Shear\ Reinforcment\ Along\ L_{sy.ct} \approx 3N12 = 330\ mm^2$$

$$\Sigma A_{tr.min} = 0.25 A_s$$

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

$$\Sigma A_{tr.min} = (0.25)(800) = 200 mm^2$$

$$\lambda = \frac{330 - 200}{800} = 0.1625$$

$$k_4 = 1 - (0.1)(0.1625) = 0.98375$$

$$k_5 = 1.0 - 0.04\rho_p$$

$$\rho_p = \frac{133}{464} = 0.29\ MPa$$

$$k_5 = 1.0 - (0.04)(0.29) = 0.9884$$

$$L_{sy.t} = (0.98375)(0.9884)(464) = 450 mm$$

Hence the longitudinal reinforcement throughout beam B will be lapped 450mm.

3.6. Calculations for the Structural Support System

Client	Tonkin Consulting	Date: 1/06/2015
Project	North Terrace Drainage System	Pages: 513-575
Subject	Support System Analysis	By: Michael Renko
Reviewed By:	David Argent	Date: 1/06/2015
Approved By:	Eriny Abdelraouf	Date: 7/06/2015

3.6.1. Initial Design Dimensions

The preliminary design drawings of the dimensions for the timber support system is illustrated in Figure 326 and Figure 327, these designs drawings are created in reference to the preliminary designs given in the detailed design brief i.e. the arrow, and steel/ timber beam & purlin support system.

All calculations are completed in reference to these drawings, however these dimensions change depending on the calculated capacity of the chosen members.

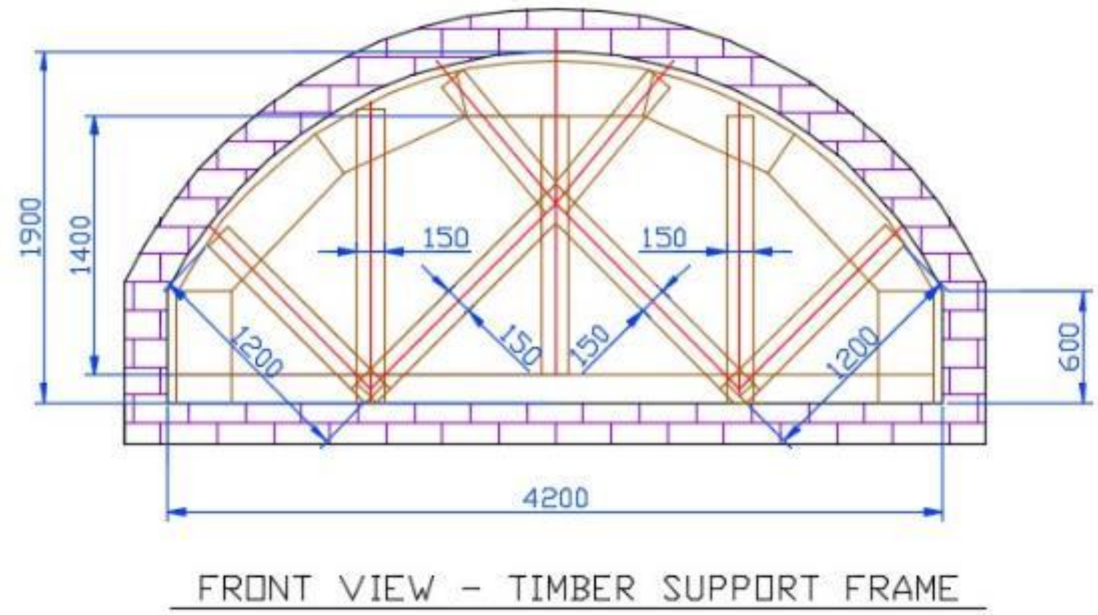


Figure 326 Preliminary design drawing; front view, timber support frame

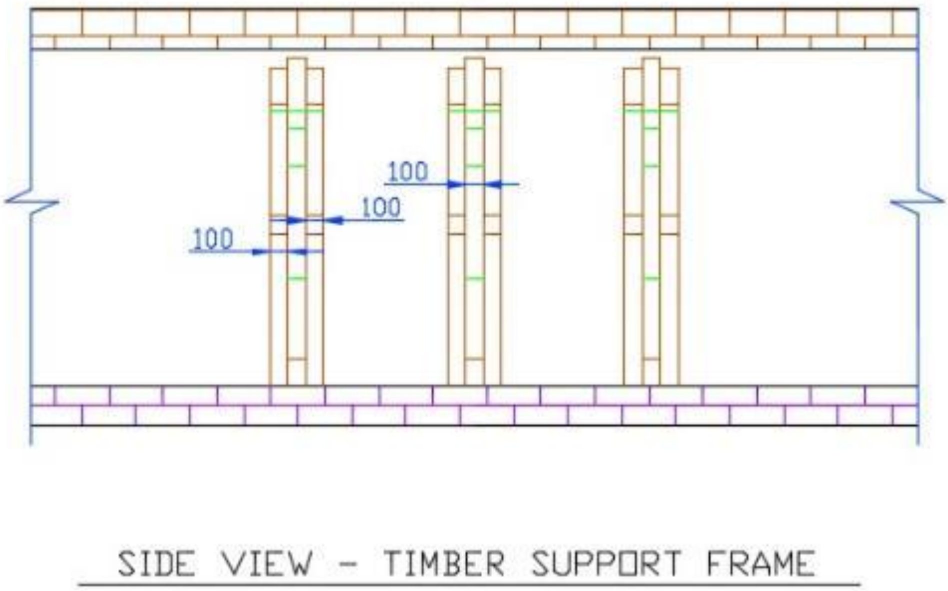


Figure 327 Preliminary design drawing; front view, timber support frame

3.6.2. Detailed Design

3.6.2.1. Analysis Methods

A statically determinate analysis method is utilised for the calculation of ultimate shear forces, bending moments, axial compression and tension forces of the support system.

The arch section of the frame could be simplified to a 3-point, statically determinate arch, however this method is inaccurate due to the complexity of the frame connections to the arching timber sections and is not considered.

A simple structural pinned and roller frame is used for this analysis, it is assumed that the structure is supporting reconfigured load width UDL point loads exerted from the purlins onto the upper most; curved members of the supporting frame. These forces are distributed throughout the structure and can be analysed quickly using finite element software such as Space Gass or Strand7 (Xing-Ma 2015). A sketch of the frame that is utilised for the finite element analysis is presented in Figure 328

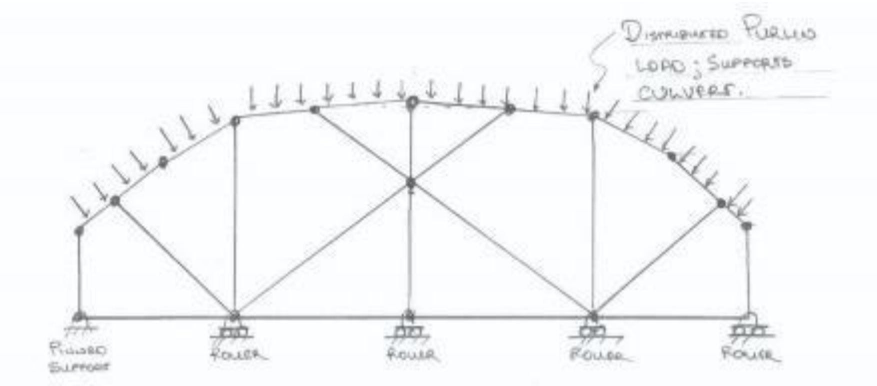


Figure 328 Sketch of input for frame into finite element program (Hydro-Future)

The finite element program used for this project is Space Gass, the input and output from this program is presented in the following sections.

3.6.2.2. Space Gass Input

The simple structural frame detailed in Figure 329 is replicated in Space Gass based off the structural frame dimensions presented in Figure 330 this Space Gass design input is shown in Figure 331 and .

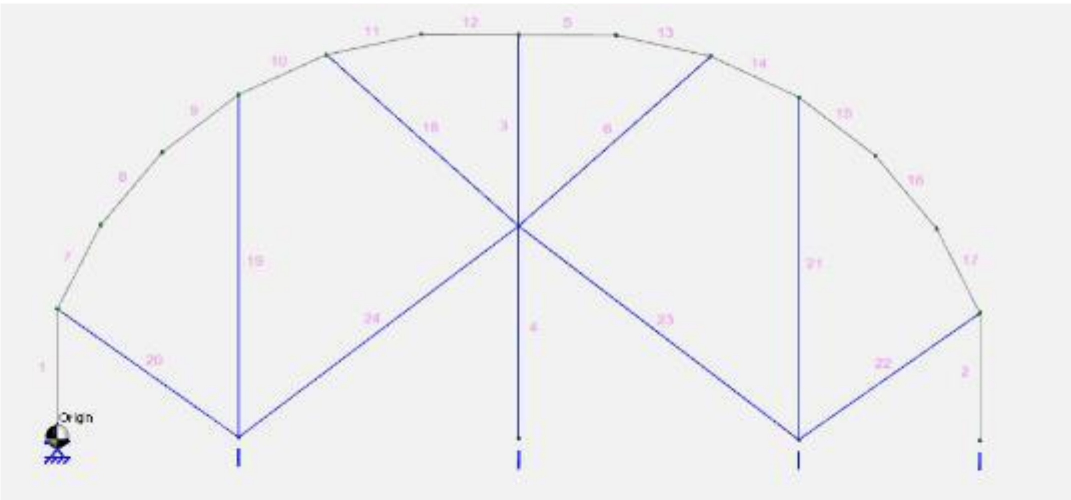


Figure 329 Space Gass input of support frame 1

The support system shown represents a section which is dedicated for construction personnel to use when inserting the pipe into the arch culvert.

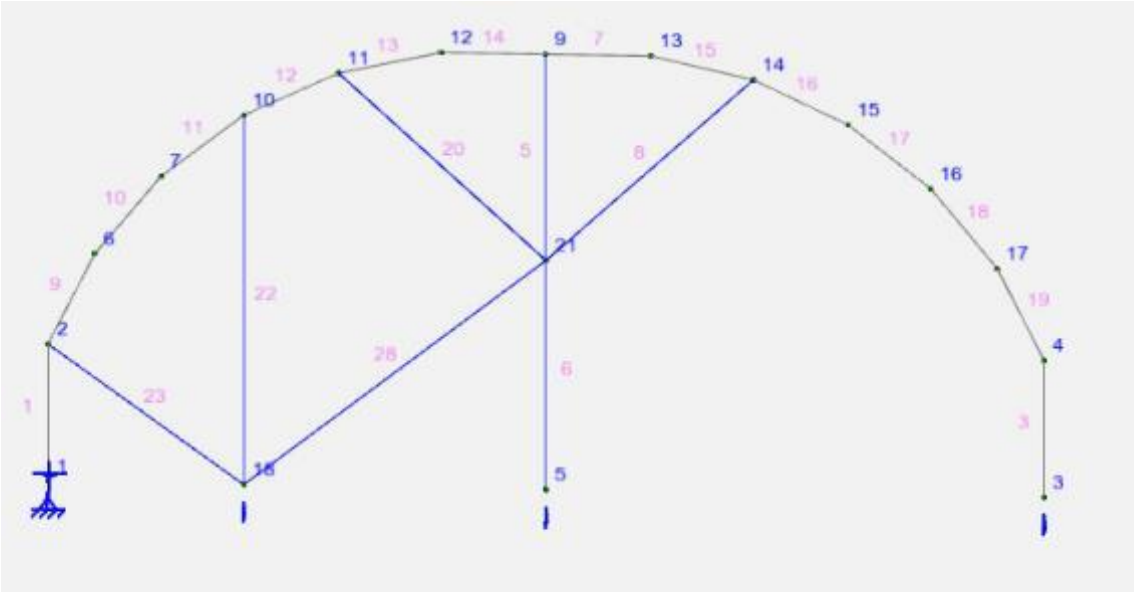


Figure 330 Space Gass input of support frame 2

The loading transferred from the culvert is distributed to the top section of the frame as a UDL.

Figure (5) shows the inputs used to activate the self-weight of the slab in Space Gass.

Case	X Acceleration (g's)	Y Acceleration (g's)	Z Acceleration (g's)
Click here to add a new row			
1	0	-1	0

Figure 331 Activating gravity, for self-weight of support structure (Space Gass 12; Hydro-Future)

The strength combinations used for the design are shown below. All combinations are given under AS/NZS 1170.0:2002, CL4.2.2 under pg 16. These strength combinations are used to design the timber members.

$$Pu1 = 1.35G\ (Permanent\ Load\ i.e.\ k_1 = 0.57)$$

$$Pu2 = 1.2G + 1.5\psi_l Q\ (Long - term\ Combination\ i.e.\ k_1 = 0.8)$$

$$Pu3 = 1.2G + 1.5Q\ (Short - Term\ combination\ i.e.\ k_1 = 0.94)$$

Out of the three load combinations presented above, $P_{u\ (3)}$ is the only combination which is applicable to the short term design of the temporary structure. The final loading acting on the culvert are created in space gass by setting up several load cases and then combining them into one ultimate load. The common set-up for these combination load cases are presented below.

$$Load\ Case\ \#1 = Self - weight + Soil\ Weight$$

$$Load\ Case\ \#2\ =\ Live\ Load, Traffic$$

$$Load\ Case\ \#10\ =\ Load\ Case\ \#1\ +\ Load\ Case\ \#2$$

The load case inputs used in Space Gass are presented in the tables shown below. These input values are derived from the sandstone arch culvert loading calculations presented in section 6.6.2. All loadings are calculated with respect to a 1m load width and are therefore converted from kPa to kN/m. The soil loadings are calculated with respect to the given soil property data available in Appendix 3.4.1.2.

Table 124 - (Dead Load) Soil, vertical and horizontal loads

Location	Soil Pressure Coefficient (k_a)	Load – UDL (kN/m) Vertical	Horizontal
A	0.42	15	6
B	0.42	19	8
C	0.42	24	10
D	0.39	34	13
E	0.39	47	18
BASE	0.39	0.6m x 20 kN/m^3 + 47 = 59	23

Table 125 - (Live Load) M1600 Traffic Case 1, vertical and horizontal loads

Location	Soil Pressure Coefficient (k_a)	Load – UDL (kN/m) Vertical	Horizontal
A	0.42	29	12
B	0.42	21	9
C	0.42	12	5
D	0.39	7	3
E	0.39	4	2
BASE	0.39	4	2

For this analysis, it is conservatively assumed that the sandstone material has a unit weight of 24kN/m^3. The self-weight of the culvert structure per metre of load width is calculated below. The thickness of the culvert wall is 230mm

$$Culvert\ Self-Weight\ (kPa)\ =\ 0.24\ kN/m3\ x\ 0.23m$$

$$Culvert\ Self-Weight\ (kPa)\ =\ 0.0552$$

$$And\ per\ metre\ width\ (load\ width\ =\ 1m)$$

$$Culvert\ Self-Weight\ (kPa)\ \approx\ \mathbf{0.06\ kN/m}$$

Table 126 Space Gass input load cases

Load Case	Load – UDL (kN/m)	
	Fx	***Fy***
1 (Support Sys. Self-Weight) G	Self-Generated	
2 (Soil Loading) G	Use Appendix 3.4.1.2	
3 (Live Load, Traffic) LL	Use Appendix 3.4.1.3 input	
4 (Culvert Self-Weight) G	-	0.06
10 (Combined Loading)	$1.2G\ +\ 1.5Q$	

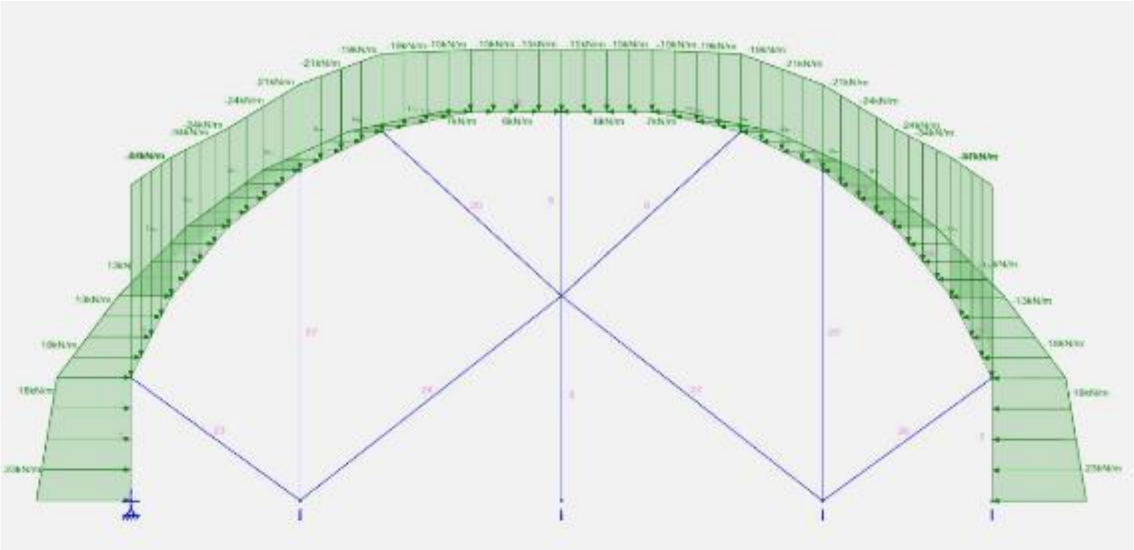


Figure 332 Space Gass load combination 2; Soil Loading (Xing-Ma)

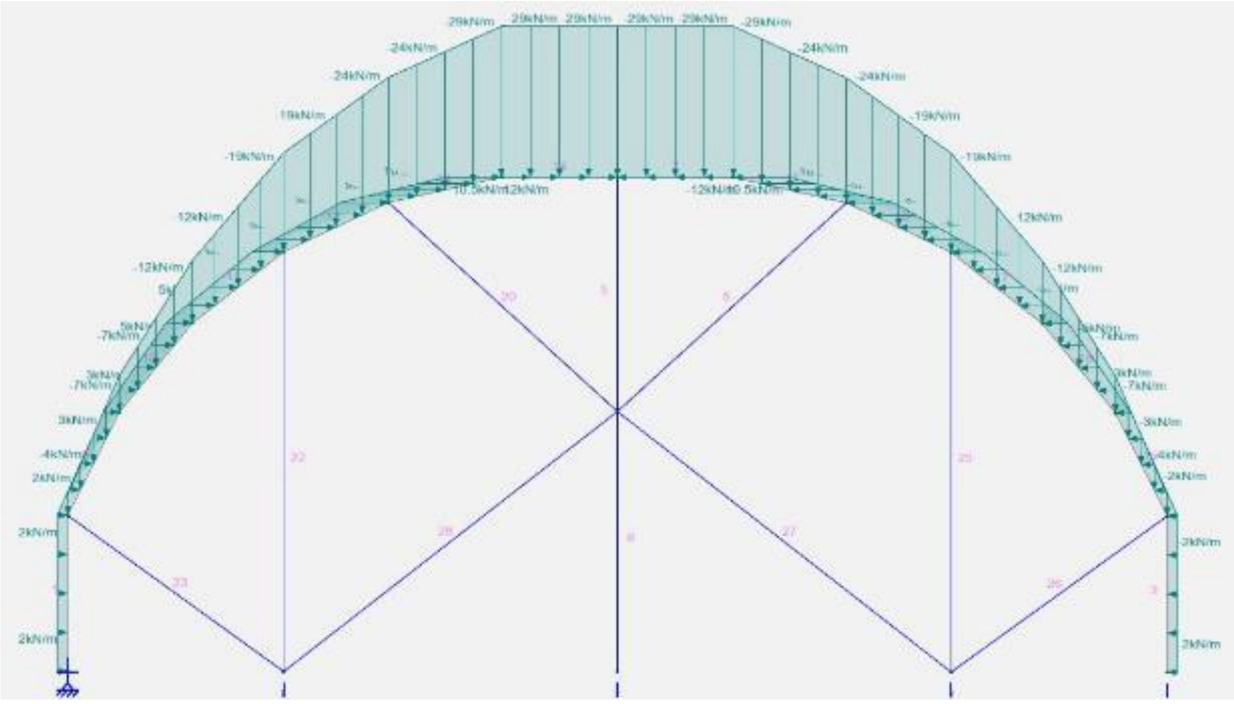


Figure 333 Space Gass load combination 3; Traffic (LL)

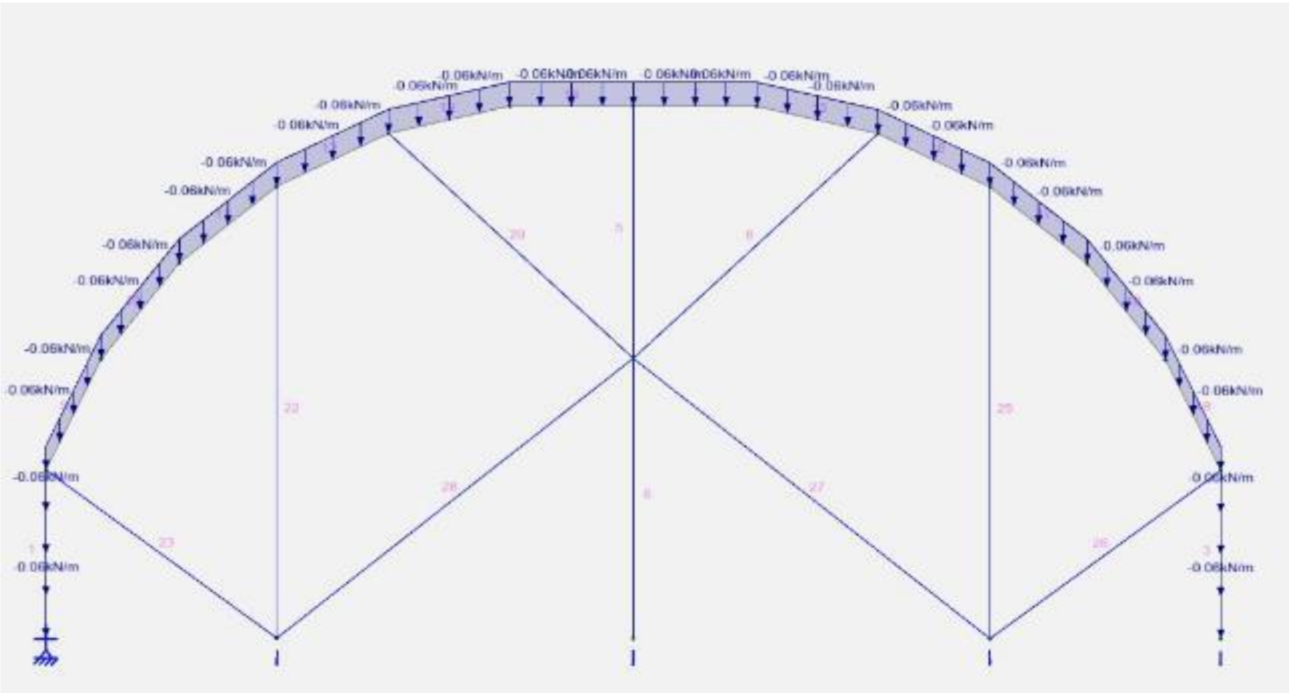


Figure 334 Space Gass load combination 4; Culvert Self-weight (Xing-Ma)

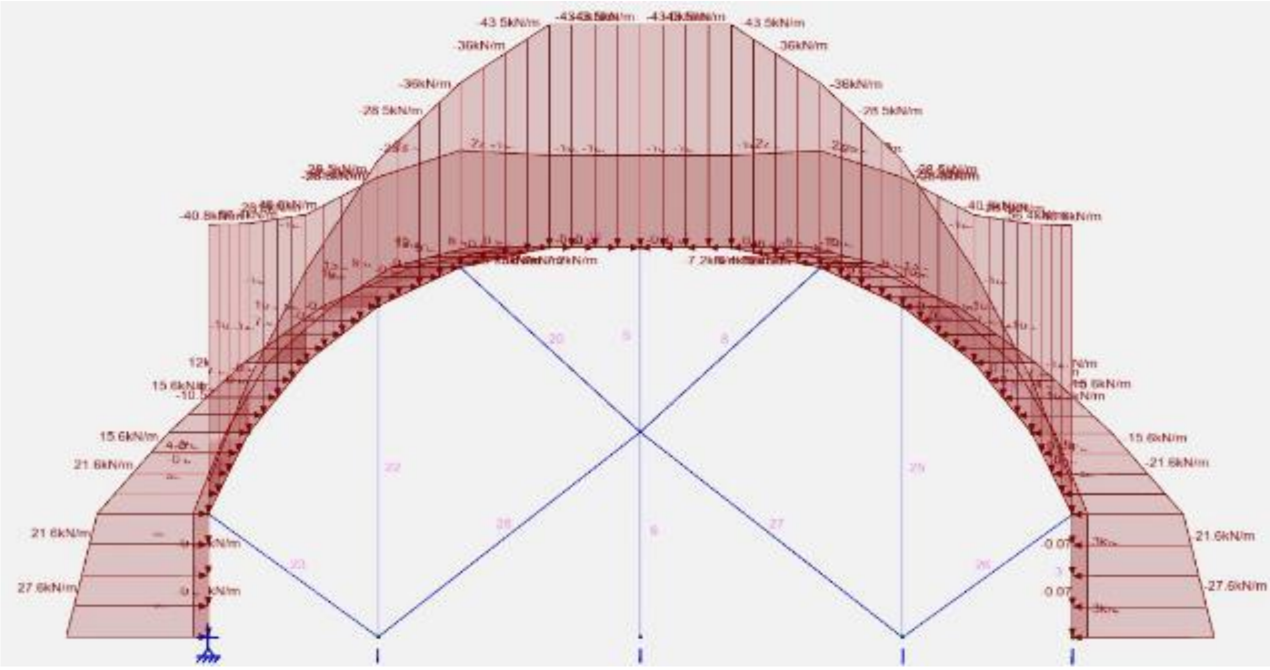


Figure 335 Space Gass load combination 10; Combined loading (1.2G + 1.5Q)

The analysis of structural members in this Space Gass project is completed by trial and error; for the first trial, the input dimensions i.e. width and breadth of each timber member is presented in Table 127

Table 127 Section properties of support system (Space Gass 12; Hydro-Future)

Section	Dimensions (mm)	Area (mm^2)	Ixx (mm^4)	Iyy (mm^4)
1 (Grey)	225 x 100	22 500	1.875×10^7	9.492×10^7
2 (Blue)	200 x 100	20 000	1.667×10^7	6.667×10^7

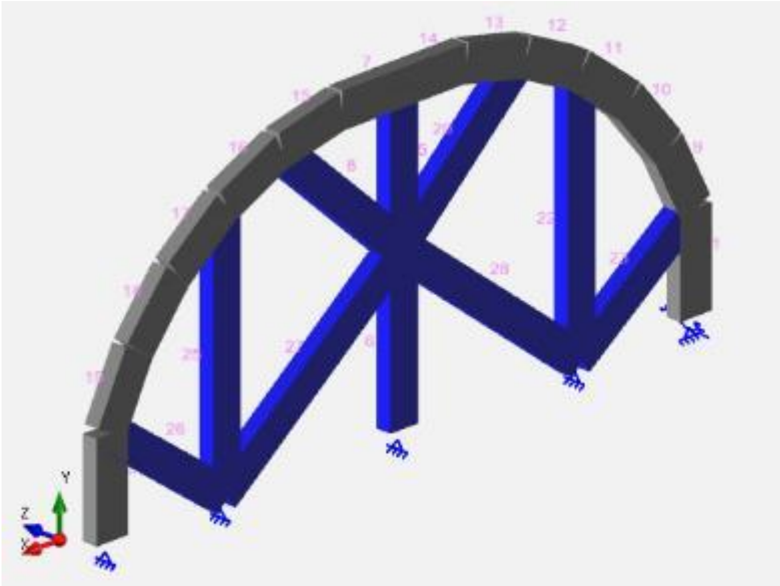


Figure 336 Space Gass rendered frame support system

The results from the linear static analysis presented in the following sections show the behaviour exhibited by the loaded frame structure. A conclusion is drawn based on the generated shear forces, bending moments and deflections.

3.6.2.3. Analysis Results

The results from the Space Gass linear static analysis has given the output for shear forces, bending moments, and axial forces of the timber structure. These results are available in the tables below illustrate the critical loading outputs for frame 1 and 2

Table 128 Output loadings for Frame 1 with a 1m load width

Section	Member	Nodes	Moment (kNm)	Shear Force (kN)	Axial Force (kN)
Side Timber Support	1	1	5.2	0.0	46.7
		2	0.0	16.5	48.9
Internal Timber	6	21	0.0	0.0	66.6
		5	0.0	0.0	66.9
Internal Timber	25	15	0.5	-0.5	43.4
		19	-0.6	-0.5	43.4
Timber Arch	19	17	0.9	-5.1	36.8
		4	-5.7	-25.2	55.1

Table 129 Output loadings for Frame 2 support 1m load width

Section	Member	Nodes	Moment (kNm)	Shear Force (kN)	Axial Force (kN)
Side Timber Support	3	4	5.2	0.0	85.8
		3	0.0	23.4	86.0
Internal Timber	6	21	0.0	0.2	97.0
		5	0.0	0.0	97.3
Timber Arch	19	17	0.6	3.3	65.9
		4	-5.2	23.4	84.2
Timber Arch	9	6	Max (6.06)	6.7	29.8
		2		26.9	48.1

The Space Gass results shows that the second frame system, sustains the highest and most critical loading, these loadings are used for design calculations of the timber frame system.

The critical loadings were calculated using an applied load width of 1m, since the pipe being connected to the arch culvert is approximately 900mm wide, a larger load width should be used to provide more space for construction personnel. The recalculated loadings using a load width of 2m is shown below.

Table 130 Critical forces in Frame 2 supporting 2m load width

Section	Member	Nodes	Moment (kNm)	Shear Force (kN)	Axial Force (kN)
Side Timber Support	3	4	-10.3	0.0	171.6
		3	0.0	-33.1	172.1
Internal Timber	6	21	-0.4	0.0	194.0
		5	0.0	0.0	194.5
Timber Arch	19	17	1.2	-6.5	131.8
		4	-10.4	-46.8	168.4
Timber Arch	9	6	Max (12.12)	13.5	59.7
		2		53.8	96.2

The doubling the load width from 1m to 2m has approximately doubled the loadings on the arch culvert system. For simplicity, the member capacities will be check w.r.t. a load width of 1m; a final summary of results and costing for each member will indicate which load width should be chosen for the final design of the support system.

3.6.3. Support System Design

The calculations completed in this section are in accordance to AS1720.1. The clauses utilised from this standard are:

- *Clause 3.3.1 – Compression strength design*
  - *Clause 3.3.2 – Slenderness coefficient for lateral buckling under compression*
- *Clause 3.2.1 – Bending strength design*
- *Clause 3.2.5 – Flexural shear strength design*
- *Clause 3.2.6 – Bearing capacity check*

3.6.3.1. Derived Factors from AS1720.1 i.e. k, ∅ etc

The factors required for calculation of shear, moment and axial compression capacity are determined below.

Stress Grade Factor - ∅

The value of ∅ is derived from Table 2.1 of AS 1720.1-2010, from row 'Stress grades: higher F-Grades.' The structure must fulfil essential services and therefore critical category 3 is chosen.

$$\emptyset = 0.75$$

Duration of load factor, k_1

Since the structure is temporary and will be dismantled after connection of the stormwater pope has been made to the arch culvert, it is assumed that the following value for k_1 should be chosen (as per 2.4.1.1(g) of AS 1720.1-2010):

$$k_1\ (5\ days, short\ term) = 0.94$$

Partial Seasoning Factor, k_4

The factor of k_4 represents the strength due to seasoning. As the design member is unseasoned, and it is unknown whether the full design load with be applied after the member has become partially seasoned, the following value is used (as per 2.4.2.2. of AS 1720.1-2010):

$$k_4 = 1.0$$

Temperature Factor, k_6

This modifier allows for the effects of high temperature on exposed, seasoned, timber. As the member will most likely not season over time, the following k value is used (as per 2.4.3 of AS 1720.1-2010):

$$k_6 = 1.0$$

Strength Sharing Factor, k_9

The k9 factor represents the sharing of load between parallel members, the member is considered to have insufficient sharing since the distribution and sharing of loads in this case is not possible; the spans from edge to edge have no internal member restraints in the lateral direction, and as such the value of k9 is taken as 1.0 for all members.

$$k_9 = 1.0$$

Bending, Stability/Slenderness Factor, k_{12}

The calculation of k_{12} requires the formula of S_1, this is given by equation 3.2(5) of AS 1720.1-2010. The results from this formula will vary for each timber member being analysed. A sample calculation of S_1 for 'Side Timber Support Member' is shown below:

$$S_1 = 1.25\frac{d}{b}\left(\frac{L_{ay}}{d}\right)^{0.5}$$

The values used for this member are: d = 222mm, and b = 97mm. L_{ay} = 600mm:

$$S_1 = 1.25\frac{222mm}{97mm}\left(\frac{600\text{mm}}{222mm}\right)^{0.5}$$

$$= 4.70$$

The value of ρ_b for F34 unseasoned hardwood, the critical value, is 1.21, therefore:

$$\rho_b S = 4.70\ x\ 1.21$$

$$= 5.69$$

Therefore, as per 3.2.4(a) of AS 1720.1-2010 as 4.74<10, k_{12} is taken as 1.0. The 'Timber Arch' member will be restraint along the compression face, it is assumed the purlins are to be space at 200 cts, therefore Lay is set to 200mm for Timber Arch. Take note, dimensions of unseasoned timber members are reduced by 3mm.

Table 131 k_{12} Bending Stability Factor Calculation

K_{12} results for all timber members in bending					
Member:	Dimensions (d x b)	L_{ay}	S_1:	$\rho_b S$:	k_{12}
Side Timber Support	225mm x 100mm	600mm	4.70	5.69	1.0
Internal Support	200mm x 100mm	1,600mm	7.23	8.75	1.0
Timber Arch	125mm x 100mm	200mm	2.01	2.44	1.0

Compression, Stability/Slenderness Factor, k_{12}

The calculation of k_{12} requires the formula of S_3 and $S_{4,}$ this is given by equations 3.3(5) to 3.3(9) in AS 1720.1-2010. A sample calculation of S_3 and S_4 for 'Side Timber Support Member' are shown below:

$L_{ax} = L_{ay} = 600mm$, i.e. no restraints along the length of the column

The values used for this member are: d = 222mm, and b = 97mm. L_{ay} = 600mm:

Solving for S_3:

$$S_3 = L_{ax}/d$$

$$S_3 = \frac{600mm}{222mm}$$

$$S_3 = 2.70$$

Or

$$S_3 = g_{13}L/d$$

Set g_{13} to 1.5; Restrained in one end in direction and position and other end partially restrained, as per Table 3.2; AS1720.1-2010

$$S_3 = 1.5\left(\frac{600mm}{222mm}\right)$$

$$S_3 = 4.05$$

Take the lesser value of S_3 as the final answer, i.e. 2.07

Solving for S_4:

$$S4 = Lay/b$$

$$S4 = \frac{600mm}{97mm}$$

$$S4 = 6.19$$

$$Or$$

$$S4 = g13L/b$$

$$S4 = 1.5\,x\,6.19$$

$$S4 = 9.28$$

Take the lesser value of S_4 as the final answer, i.e. 6.19

Of the two calculated values; S_3 and S_4, S_4 = 6.19 (larger value) is the critical slenderness factor used to calculate $\rho_c S$

Factor k_{12} requires ρ_c from Table 3.3 of AS 1720.1-2010. For unseasoned timber this value is equal to 1.34 and therefore $\rho_c S$ equal to 8.29. Since $\rho_c S$ < 10, the factor is equal to:

$$k_{12} = 1.0$$

Table 132 - k_{12} Compression Stability Factor Calculation

			K12 results for all timber members			
Member:	Dimensions (dxb)	L_{ay}	Lower S_3:	Lower S_4:	$\rho_c S$:	k_{12}
Side Timber Support	225mm x 100mm	600mm	2.70	6.19	8.29	1.0
Internal Support	200mm x 100mm	1,600mm	5.69	11.55	7.62	1.0
Timber Arch	125mm x 100mm	200mm	1.15	1.44	1.54	1.0

The values of stability factor, k_{12} vary depending on the chosen member size. It should be noted that if any size were to reduce, the slenderness values would be recalculated to check if the stability factor changes i.e. $\rho_c S$ or $\rho_b S$ > 10.

3.6.3.2. Side timber support design

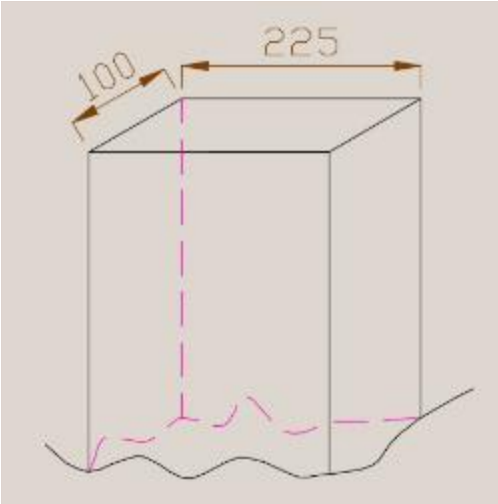
The Space Gass output loadings for the side timber support member is presented in *Table 133*

Table 133 Space Gass output loadings; Side timber support member

Section	Member	Nodes	Moment (kNm)	Shear Force (kN)	Axial Force (kN)
Side Timber Support	3	4	5.2	0.0	88.3
		3	0.0	16.6	88.5

Table 134 Side Timber Support Member Properties

Side Timber Support Member Properties	
Length	≈ 0.6m | 600mm
Area	225mm x 100mm
Bending Strength	84 MPa
Shear Strength	6.1 MPa
Tension, parallel to grain	Hardwood; 51 MPa
Compression	63 MPa
Elastic Modulus	21.5 GPa
Rigidity Modulus	1.43 GPa



Compression Capacity Check

Compression members must be checked for stability also i.e. the slenderness; modifier k_{12}. The compressive capacity formula given in AS 1720.1-2010 is shown below.

$$\emptyset Nd.c. = \emptyset k_1 k_4 k_6 k_{12} f'c\, A'c$$

The cross sectional area of the member for compression, A_c, is equal to 222mm x 97mm (Reduced by 3mm due to unseasoned timber), equals 21,534mm^2; f_c' is given by Table H2.1 of the AS 1720.1-2010 as 63MPa (for F34 grade hardwood).

The capacity is calculated below.

$$\emptyset Nd.c. = \ 0.75\,x\,0.94\,x\,1.0\,x\,1.0\,x\,1.0\,x\,64\,MPa\,x\,21{,}534mm^2$$

$$\emptyset Nd,c = \ 971.6kN \ > \ 103.6kN$$

The capacity in compression for this member is almost 8 times larger than the ultimate loading. This member is overdesigned and must be reduced in size.

The size of the member is reduced to 125 x 100mm; this size is effectively the same as the 'Timber Arch Member'. Take note, stability factor for bending and compression are equal to 1.0 for this size.

$$\emptyset Nd.c. = \ 0.75\,x\,0.94\,x\,1.0\,x\,1.0\,x\,1.0\,x\,64\,MPa\,x\,(122x97)\,mm^2$$

$$\emptyset Nd.c. = \ 534.0\,kN \ > \ 103.6kN$$

The capacity in compression for this member is almost 4 times larger than the ultimate loading. This member is overdesigned and must be reduced in size. The reduction in size will undoubtedly decrease stability performance (k_{12}) in bending and compression. The stability factors are calculated in Excel with regard to a range of timber cross-sectional sizes in the tables below with the best member cross-section size chosen for the design.

Table 135 Bending Stability (k_{12}) calculation for different sized members

Member	d (mm) Actual	b (mm) Actual	d (mm) Reduced(3mm)	b (mm) Reduced(3mm)	S_1	$\rho_b S$	k_{12}
Side Timber Support	225	100	222	97	4.70	5.69	1.0
	100	50	97	47	6.42	7.76	1.0
	75	50	72	47	5.53	6.69	1.0
Internal Support	200	100	197	97	7.23	8.75	1.0
	100	50	97	47	10.48	12.68	**Not 1.0**
	75	50	72	47	9.03	10.92	**Not 1.0**
Timber Arch	125	100	122	97	2.01	2.44	1.0
	100	50	97	47	3.70	4.48	1.0
	75	50	72	47	3.19	3.86	1.0

Table 136 Compression Stability (k_{12}) calculation for different sized members (AS1720.1, Excel 2015; Hydro-Future)

Member	d (mm) Actual	b (mm) Actual	G_{13}	S_3	$G_{13}S_3$	S_4	$G_{13}S_4$	$\rho_c S$	Lay	Lax	k_{12}
Side Timber Support	**225**	100	1.5	2.70	4.05	6.19	9.28	8.29	600	600	1.0
	100	50	1.5	6.19	9.28	12.77	19.15	17.11			**Not 1.0**
	75	50	1.5	8.33	12.50	12.77	19.15	17.11			**Not 1.0**
Internal Support	200	100	0.7	8.12	5.69	16.49	11.55	15.47	1600	1600	**Not 1.0**
	100	50	0.7	16.49	11.55	34.04	23.83	31.93			**Not 1.0**
	75	50	0.7	22.22	15.56	34.04	23.83	31.93			**Not 1.0**
Timber Arch	125	100	0.7	3.69	1.15	2.06	1.44	1.93	200	450	1.0
	100	50	0.7	4.64	1.44	4.26	2.98	3.99			1.0
	75	50	0.7	6.25	1.94	4.26	2.98	3.99			1.0

From analysis of bending and compression slenderness/stability it is concluded that the most appropriate members to be tested for further detailed design are as follows:

- 75x50mm for Side Timber Member
- 75x50mm for Internal Member
- 75x50mm for Timber Arch Member

Reducing the members to a single size improves the speed of calculations i.e. replicating results from one section to another.

The compression capacity for the side timber support is calculated below.

$$\emptyset Nd.c. = \; 0.75\ x\ 0.94\ x\ 1.0\ x\ 1.0\ x\ 1.0\ x\ 64\ MPa\ x\ (73x47mm)$$

$$\emptyset Nd,c = \; 154.8kN \; > \; 103.6kN;\; OK$$

The capacity in compression for this member is satisfactory, adopt 75x50mm. This member is fairly strong in resisting the applied forces, however the uncertainty surrounding its deflection, moment and shear capacity dictates the necessity to not reduce the size any further.

Shear Capacity Check

The beam design shear capacity is calculated using the following formula given under AS/NZS 1720.1:2010, page 33:

$$Vd \;=\; \emptyset k_1 k_4 k_6 f's\, A's$$

The design is safe if the capacity equation satisfies the following expression below

$$V^* \leq V_d$$

The ultimate shear force is located $1.5d$ from either of member supports. The shear force for this member is equal to -16.5 kN at Node 2, and the length is 600mm. The depth of the member is 75mm, therefore 1.5d = 112.5, the shear force at this point is calculated below via equal triangles.

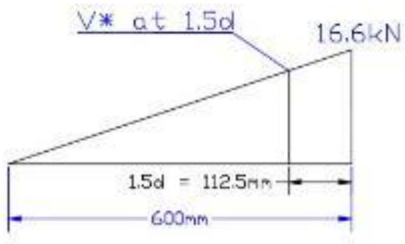


Figure 337: Equal triangle for shear calculation

$$\text{V* at 1.5d from support} = \text{V* x}\ \frac{600mm-112.5mm}{600mm}$$

$$\text{V* at 1.5d from support} = \text{16.6kN}\ \left(\frac{600mm-112.5mm}{600mm}\right)$$

$$\text{V* at 1.5d from support} = \text{13.5kN}$$

The K factors given in the equation modify the design strength capacity based on the same factors used to calculate compression capacity. For F34 timber, the value of f'_s is equal to 6.1 *MPa*. The equation used to calculate shear area is shown below.

$$A_s = \left(\frac{2}{3}\right) bd$$

Recall that the dimensions *b* and *d* have shrunk by 3mm since the member is unseasoned.

$$A_s = \left(\frac{2}{3}\right)(72mm \times 47mm)$$

$A_s = 2,256mm^2$

Substituting all values into shear capacity equation and solving for V_d

$$V_d = 0.75\ x\ 1.0\ x\ 1.0\ x\ 1.0\ x\ 6.1\ MPa\ x\ 2,256\ mm^2$$

$$V_d = 10,828.9N$$

Therefore,

$$V^* > V_d$$

The design capacity for shear doesn't withstands the ultimate shear load. The size for this member must be increased to 100x100mm.

$$As = \left(\frac{2}{3}\right)(97mm\ x\ 97mm)$$

$$As = 6,272.7\ \text{mm}^2$$

$$V_d = 0.75\ x\ 1.0\ x\ 1.0\ x\ 1.0\ x\ 6.1\ MPa\ x\ 6,272.7\ mm^2$$

$$V_d = 28.7\text{kN}$$

V_d <V*, V* = 28.4kN; OK

Design capacity met, adopt 100x100mm from this point forth

Flexural Bending Capacity Check

The beam design bending strength capacity is calculated using the following formula given under AS/NZS 1720.1:2010, page 27:

$$Md = \emptyset k_1 k_4 k_6 k_{12} f'b\ Z$$

Where f'b = 84 MPa

The design is safe if the capacity equation satisfies the following expression below

$$M^* \leq M_d$$

Calculating the section modulus, Z for the member using the following equation listed below

$$Z = \frac{bd^2}{6}$$

$$Z = \frac{(97\ mm\ x\ (97mm)^2)}{6}$$

$$Z = 152,112.2\text{mm}^3$$

The value of k_{12} is equal to 1.0

Substituting all values into design capacity equation and solving for M_d

$$M_d = 0.75 \, x \, 1.0 \, x \, 1.0 \, x \, 1.0 \, x \, 84 \, MPa \, x \, 152{,}112.2 \, mm^3$$

$$M_d = 9.58 \, kNm$$

Therefore, $M_d > M^*$, *where* $M^* = 5.2$ kNm

Adopt 100 x 100mm for the 'Side Timber Support' member.

Deflection Serviceability Check

Deflection can cumulate to cause creep over time. The deflection of beams under a given load combination is calculated using the formula shown below:

For point loads

$$\delta_s = \sum(j_2 \, \frac{PL^3}{48EI})$$

Where the value of j_2 is considered as a 'Duration of Loading' factor and is given under table 2.4 on pg 22; *E* is the modulus of elasticity of timber given under table H2.1, pg 155; *I* is the second moment of inertia; *L* is the length of the member; *P* is the load combination used to check serviceability deflection.

The timber member is unseasoned, the j_2 factor is equal to 1.0, as per table 2.4 on pg 22. The value of E derived from table H2.1 is equal to 21.5 GPa, the value of $E_{0.05}$ is derived as illustrated under Appendix B on pg 104, therefore $E_{0.05} = 0.5E_{ave}$, $E_{0.05}$ = 10.75 GPa.

The value of I is given as the second moment of inertia which is calculated using the equation shown below

$$I_x = \frac{bh^3}{12}$$

Where b = 50mm and h = 125mm. Since the member is unseasoned, 3 mm is removed from both breadth and height of the member. The calculation for second moment of inertia is shown below

$$I_x = \frac{(97mm)(97mm)^3}{12}$$

$$I_x = 7.382 \times 10^6 \text{ mm}^4$$

As specified under AS1170.0:2002, if the serviceability deflection exceeds the limit state deflection of $\frac{SPAN}{200}$, then the selected timber member does not provide adequate conditions to be considered fit for human occupation.

Therefore, calculating allowable deflection

Allowable Serviceability Deflection = Allow δ

$$Allowable\ \delta = \frac{SPAN}{200}$$

$$Allowable\ \delta = \frac{600mm}{200}$$

$$Allowable\ \delta = 3mm$$

As per page 101, Table B1, the deflection is calculated using the following combination only i.e. G + Q = 88.3 *kN*. Length of member, L = 0.6m, $E_{0.05}$ = 21.5 GPa and the area is equal 122mm x 47mm = 5,734mm^2

$$\delta_s = (1.0)\frac{(88{,}300\text{N})(600\text{mm})^3}{48(21{,}500MPa)(7.382 \text{ x } 10^6\text{mm}^4)}$$

$$\delta_s = 2.5 \text{ mm}$$

$$\delta_s > Allow\ \delta$$

Therefore, the serviceability deflection successfully satisfies deflection conditions.

Adopt 100 x 100 mm for the 'Side Timber Support' member.

3.6.3.3. Internal timber Member design

The Space Gass output loadings for the internal timber support member is presented in *Table 137*

Table 137 Space Gass output loadings for Frame 1; Side timber support member

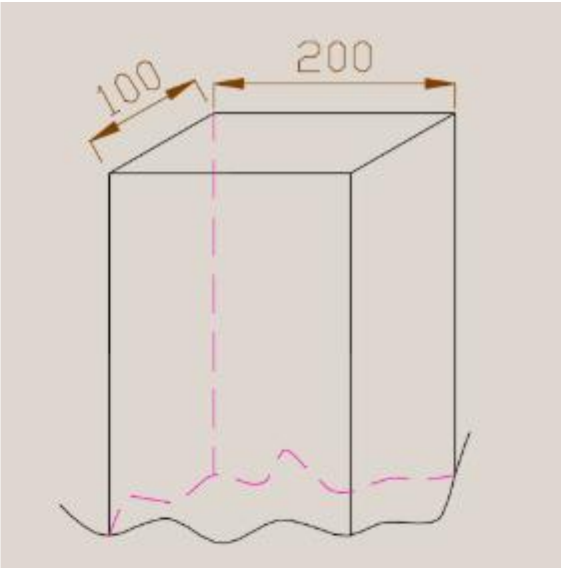
Section	Member	Nodes	Moment (kNm)	Shear Force (kN)	Axial Force (kN)
Internal Timber	6	21	0.0	0.0	66.6
		5	0.0	0.0	66.9
Internal Timber	25	15	0.5	-0.5	43.4
		19	-0.6	-0.5	43.4

Table 138 Space Gass output loadings for Frame 2; Side timber support member

Section	Member	Nodes	Moment (kNm)	Shear Force (kN)	Axial Force (kN)
Internal Timber	6	21	0.0	0.0	94.0
		5	0.0	0.0	94.3

Table 139 Side Timber Support Member Properties

Side Timber Support Member Properties	
Length	≈ 1.6m | 1600mm
Area	200mm x 100mm **(Reduced to 100mm x 100mm; Based on Appendix 3.6Design outcomes)**
Bending Strength	84 MPa
Tension, parallel to grain	Hardwood; 51 MPa
Compression	63 MPa
Elastic Modulus	21.5 GPa



Compression Capacity Check

$$\emptyset Nd.c. = \emptyset k_1 k_4 k_6 k_{12} f'c\, A'c$$

A_c, is equal to 197mm x 97mm (Reduced by 3mm due to unseasoned timber), equals 19,109mm^2;The value of k_{12} is not equal to 1.0.The calculation for k_{12} is shown below.

The calculated value of *PcS* is equal to 15.47. Since *PcS > 10*, the following formula is used to calculate k_{12}:

$$k_{12} = 1.5 - 0.05(PcS)$$

$$k_{12} = 1.5 - 0.05(15.47)$$

$$k_{12} = 0.73$$

The capacity is calculated below.

$$\emptyset N_{d.c.} = 0.75 \times 0.94 \times 1.0 \times 1.0 \times 0.73 \times 64 \text{ MPa} \times 19{,}109\text{mm}^2$$

$$\emptyset N_{d,c} = 629.4\text{kN} > 94.3 \text{ kN; OK}$$

Decreasing size to 100x100mm

The capacity is calculated below.

$$\emptyset N_{d.c.} = 0.75 \times 0.94 \times 1.0 \times 1.0 \times 0.73 \times 64 \text{ MPa} \times (97\text{x}97\text{mm})\text{mm}^2$$

$$\emptyset N_{d,c} = 310.0\text{kN} > 94.3 \text{ kN; OK}$$

Decreasing size to 50x100mm

$$\emptyset N_{d.c.} = 0.75 \times 0.94 \times 1.0 \times 1.0 \times 0.73 \times 64 \text{ MPa} \times (97\text{x}47\text{mm})\text{mm}^2$$

$$\emptyset N_{d,c} = 191.3\text{kN} > 94.3 \text{ kN; OK}$$

Design capacity met, adopt 50x100mm from this point on

Shear Capacity Check

The beam design shear capacity is calculated using the following formula given under AS/NZS 1720.1:2010, page 33:

$$Vd \;=\; \emptyset k_1 k_4 k_6 f's\, A's$$

The design is safe if the capacity equation satisfies the following expression below

$$V^* \leq V_d$$

Conservatively take shear as ultimate load i.e. 0.5 kN. Recall that the dimensions *b* and *d* have shrunk by 3mm since the member is unseasoned.

$$As \;=\; \left(\frac{2}{3}\right)(47mm\ x\ 97mm)$$

$$A_s = 3{,}039.333mm^2$$

Substituting all values into shear capacity equation and solving for V_d

$$V_d \;=\; 0.75\ x\ 1.0\ x\ 1.0\ x\ 1.0\ x\ 6.1\ MPa\ x\ 3{,}039.3\ mm^2$$

$$V_d \;= 13.9kN;\ V_d > V^*,\ \text{OK}$$

Therefore, design capacity met, continue using 50x100mm

Flexural Bending Capacity Check

The beam design bending strength capacity is calculated using the following formula given under AS/NZS 1720.1:2010, page 27:

$$Md = \emptyset k_1 k_4 k_6 k_{12} f'b\ Z;\ Where\ f'b = 84\ MPa$$

The design is safe if the capacity equation satisfies the following expression below

$$M^* \leq M_d$$

Calculating the section modulus, Z for the member using the following equation listed below

$$Z = \frac{bd^2}{6}$$

$$Z = \frac{(47\ mm\ x\ (97mm)^2)}{6}$$

$$Z = 73{,}703.83\ \text{mm}^3$$

The value of k_{12} is not equal to 1.0

The calculation of S_1 for is shown below:

$$S_1 = 1.25 \frac{d}{b} \left(\frac{L_{ay}}{d}\right)^{0.5}$$

The values used for this member are: d = 97mm, and b = 47mm. L_{ay} = 1600mm:

$$S_1 = 1.25 \frac{97mm}{47mm} \left(\frac{1600\text{mm}}{97mm}\right)^{0.5}$$

$$= 10.48$$

The value of ρ_b for F34 unseasoned hardwood, the critical value, is 1.21, therefore:

$$\rho_b S = 10.48x\ 1.21$$

$$= 12.68$$

Therefore, as per 3.2.4(a) of AS 1720.1-2010 as 4.74>10, k_{12} must be calculated using:

$$k_{12} = 1.5 - 0.05\ \rho_b S$$

$$k_{12} = 1.5 - 0.05(12.68)$$

$$k_{12} = 1.5 - 0.05(12.68)$$

$$k_{12} = 0.866$$

It may also be assumed that the 'Timber Arch' member is restraint along the compression face, and that purlins are spaced at a maximum of 200 cts, therefore Lay is set to 200mm for Timber Arch, however it is conservative to adopt the k_{12} value as it is more critical.

Substituting all values into design capacity equation and solving for M_d

$$M_d = 0.75\ x\ 1.0\ x\ 1.0\ x\ 0.866\ x\ 84\ MPa\ x\ 73{,}703.8\ mm^3$$

$$M_d\ =\ 4.01\ kNm;\ M_d > \mathrm{M}^*$$

Adopt 50mmx100mm for the 'Internal' member.

Deflection Serviceability Check

$$\delta_s = \sum(j_2\ \frac{PL^3}{48EI})$$

The timber member is unseasoned, the j_2 factor is equal to 1.0, as per table 2.4 on pg 22. The value of E derived from table H2.1 is equal to 21.5 GPa.

Where b = 50mm and h = 75mm. Since the member is unseasoned, 3 mm is removed from both breadth and height of the member. The calculation for second moment of inertia is shown below

$$I_x = \frac{(47mm)(97mm)^3}{12}$$

$$I_x = 3.574\text{x}10^6\ \text{mm}^4$$

Therefore, calculating allowable deflection

Allowable Serviceability Deflection = Allow δ

$$Allowable\ \delta = \frac{SPAN}{200}$$

$$Allowable\ \delta = \frac{1600mm}{200}$$

$$Allowable\ \delta = 8mm$$

Calculating deflection under dead load combination only i.e. G + Q = 94,300*N*. Length of member, L = 0.6m, $E_{0.05}$ = 21.5 GPa

$$\delta_s = (1.0)\frac{(94.300\text{N})(1600\text{mm})^3}{48(21{,}500MPa)(3{,}574\ \text{x}\ 10^6\text{mm}^4)}$$

$$\delta_s = 104\ \text{mm}$$

$$\delta_s < Allow\ \delta$$

Therefore, the serviceability deflection does not satisfy deflection conditions. Therefore, increasing size back up to 225 x 100mm.

$$I_x = \frac{(97mm)(222mm)^3}{12}$$

$$I_x = 88.44 \times 10^6\ mm^4$$

$$\delta_s = (1.0)\frac{(94.300\mathrm{N})(1600\mathrm{mm})^3}{48(21{,}500MPa)(88.44\ \mathrm{x}\ 10^6\mathrm{mm}^4)}$$

δ_s = 4.2 mm > Allow δ; **Adopt 225 x 100mm for 'internal timber members'**

3.6.3.4. Timber arch member design

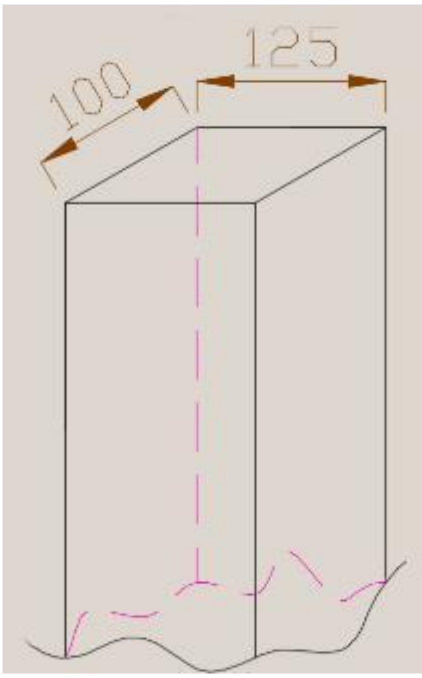
The Space Gass output loadings for the timber arch support member is presented in *Table 140*

Table 140 Space Gass output loadings for Frame 2; Side timber support member

Section	Member	Nodes	Moment (kNm)	Shear Force (kN)	Axial Force (kN)
Timber Arch	19	17	1.1	4.4	68.1
		4	-5.2	24.5	86.4
Timber Arch	16	14	Max (3.22)	28.4	99.2
		15		1.6	103.6

Table 141 Side Timber Support Member Properties

Timber Arch Support Member Properties	
Length (Lay)	≈ 0.2m
Length (Lax)	≈ 0.45m
Area	125mm x 100mm **(Increased to 225mm x100mm; Previous section deflection design outcomes)**
Bending Strength	84 MPa
Tension, parallel to grain	Hardwood; 51 MPa
Compression	63 MPa
Elastic Modulus	21.5 GPa



Rigidity Modulus	1.43 GPa

Compression Capacity Check

The compression capacities are identical to whatever compression capacity is achieved by the side timber member. The calculation for the original size of the member (125x100mm) is shown below.

$$\emptyset Nd.c. = \ 0.75\ x\ 0.94\ x\ 1.0\ x\ 1.0\ x\ 1.0\ x\ 64\ MPa\ x\ \ 11{,}834\ \text{mm}^2$$

$$\emptyset Nd.c. = \ 534.0kN\ >\ 103.6kN$$

Shear Capacity Check

The beam design shear capacity is calculated using the following formula given under AS/NZS 1720.1:2010, page 33:

$$Vd\ =\ \emptyset k_1 k_4 k_6 f's\ A's$$

The design is safe if the capacity equation satisfies the following expression below

$$V * \leq\ Vd$$

The ultimate shear force is located 1.5d from either of member supports. The shear force for this member is equal to -25.2 kN, and the length is 450mm. The depth of the member is 100mm, therefore 1.5d = 150, the shear force at this point is calculated below via equal triangles.

$$V *\ at\ 1.5d\ from\ support\ =\ V *\ x\ \frac{450mm - 150mm}{450mm}$$

$$\text{V*}\ at\ 1.5d\ from\ support\ =\ (28.4kN\ -\ 1.6kN) x (\frac{450mm - 150mm}{450mm}) + 1.6\text{kN}$$

$$V *\ at\ 1.5d\ from\ support\ =\ 17.87kN\ +\ 1.6kN$$

$$V *\ at\ 1.5d\ from\ support\ =\ 19.5kN$$

Recall that the dimensions *b* and *d* have shrunk by 3mm since the member is unseasoned.

$$As\ =\ \left(\frac{2}{3}\right)(97mm\ x\ 122mm)$$

$$As\ =\ 7{,}889.3\ \text{mm}^2$$

Substituting all values into shear capacity equation and solving for V_d

$$V_d\ =\ 0.75\ x\ 1.0\ x\ 1.0\ x\ 1.0\ x\ 6.1\ MPa\ x\ 7{,}889.3\ mm^2$$

$V_d\ = 36{,}093N$ > V*, Shear capacity met; however too strong, reduce size to 75x50mm

V_d = 10.32kN < V*, Not OK, increase size to 100x100mm

V_d = 28.7kN > V* (1m Load Width), OK; Adopt 100x100mm from this point on

Extra size calculation; increase size to 225x100mm

$$Vd = 65.7kN$$

Flexural Bending Capacity Check

The beam design bending strength capacity is calculated using the following formula given under AS/NZS 1720.1:2010, page 27:

$$Md = \emptyset k_1 k_4 k_6 k_{12} f'b\, Z$$

Where f'b = 84 MPa

The design is safe if the capacity equation satisfies the following expression below

$$M^* \leq M_d$$

Calculating the section modulus, Z for the member using the following equation listed below

$$Z = \frac{bd^2}{6}$$

$$Z = \frac{(97\ mm\ x\ (97mm)^2)}{6}$$

$$Z = 152{,}112\ \text{mm}^3$$

The value of k_{12} is equal to 1.0

Substituting all values into design capacity equation and solving for M_d

$$M_d = 0.75\ x\ 1.0\ x\ 1.0\ x\ 1.0\ x\ 84\ MPa\ x\ 152{,}112.2\ mm^3$$

$$M_d = 9.58\ kNm > \text{M*}$$

Adopt 75mmx50mm for the 'Internal' member from this point on.

Deflection Serviceability Check

$$\delta_s = \sum(j_2 \frac{PL^3}{48EI})$$

The timber member is unseasoned, the j_2 factor is equal to 1.0, as per table 2.4 on pg 22. The value of E derived from table H2.1 is equal to 21.5 GPa.

Where b = 100mm and h = 225mm. Unseasoned, remove 3mm.

$$I_x = \frac{(97mm)(97mm)^3}{12}$$

$$I_x = 7.377x10^6 \text{ mm}^4$$

Therefore, calculating allowable deflection

Allowable Serviceability Deflection = Allow δ

$$Allowable\ \delta = \frac{SPAN}{200}$$

$$Allowable\ \delta = \frac{450mm}{200}$$

$$Allowable\ \delta = 2.3mm$$

Calculating deflection under dead load combination only i.e. G + Q = 103.6 *kN*. Length of member, L = 0.6m, $E_{0.05}$ = 21.5 GPa

$$\delta_s = (1.0)\frac{(103{,}600\text{N})(450\text{mm})^3}{48(21{,}500MPa)(7.38 \text{ x } 10^6\text{mm}^4)}$$

$$\delta_s = 1.2\text{mm}$$

$$\delta_s > Allow\ \delta$$

Adopt 100 x 100mm for 'timber arch member'

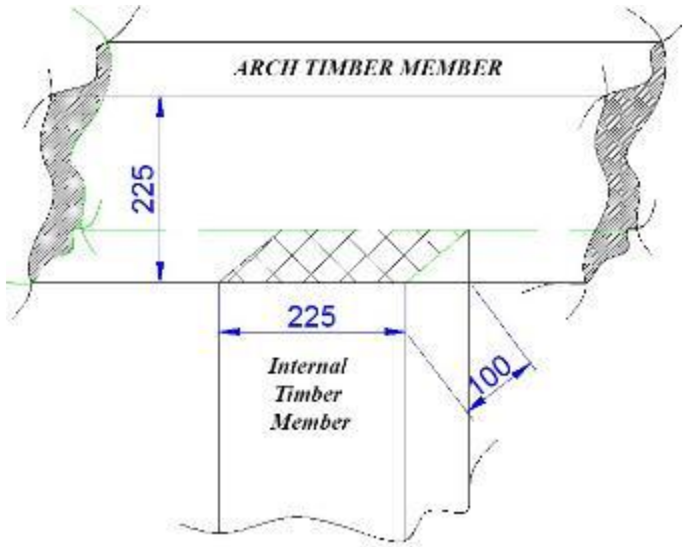
3.6.3.5. Bearing Capacity Design

Flat End Bearing Check

The Space Gass output loadings for the internal timber support member is presented in Table 142

Table 142 Space Gass output loadings; Member 5

Section	Member	Nodes	Moment (kNm)	Shear Force (kN)	Axial Force (kN)
Internal Timber	5	21	0.0	0.0	104.8
		9	0.0	0.0	104.3



The internal member is currently pressing against the surface of the top arching timber member, the bearing capacities for this member must be calculated using Clause 3.2.6; the formula is shown below.

$$Ndp = \emptyset k_1 k_4 k_6 k_7 f'p\ A'p$$

Where, $Nd,p > Np\ ^*Np^*$ = Design load effect in bearing

Where: $\emptyset$ = 0.75, k_1 = 0.94, k_4 = 1, k_6 = 1.0

Timber is non-ash, hardwood, as per Table H2.3, the joint group is J2 and strength group is S4, therefore the bearing characteristic strength is (<u>$f'p$</u>) 8.6 MPa. K_7 was then determined using Table 2.6 the depth bearer member, 225mm, therefore K_7 is equal to 1.

The final modifier, A_p, was then determined. This modifier takes into account the contact area between the bearer and the arch timber member, the contact area between these is:

A_p = 222mm x 97mm (allowing for 3mm shrinkage)

A_p = 21,534mm^2

$$Ndp = \emptyset k_1 k_4 k_6 k_7 f'p\, A'p = 0.75\ x\ 0.94\ x\ 1\ x\ 1\ x\ 1\ x\ 8.6MPa\ x\ 21{,}534\text{mm}^2$$

$$Nd,p = 130{,}560.6N > 104.8kN$$

The bearing capacity between the internal member and the top arching section has been satisfied.

Notched End Bearing Check

The Space Gass output loadings for the internal timber (25), notched between member 16 and 17 is presented in Table 143

Table 143 Space Gass output loadings; Member 25

Section	Member	Nodes	Moment (kNm)	Shear Force (kN)	Axial Force (kN)
Internal Timber	25	15	0.97	-0.9	86.7
		19	0.54	-0.9	87.6

The figure below presents the notched member and its angle to the arching timber member

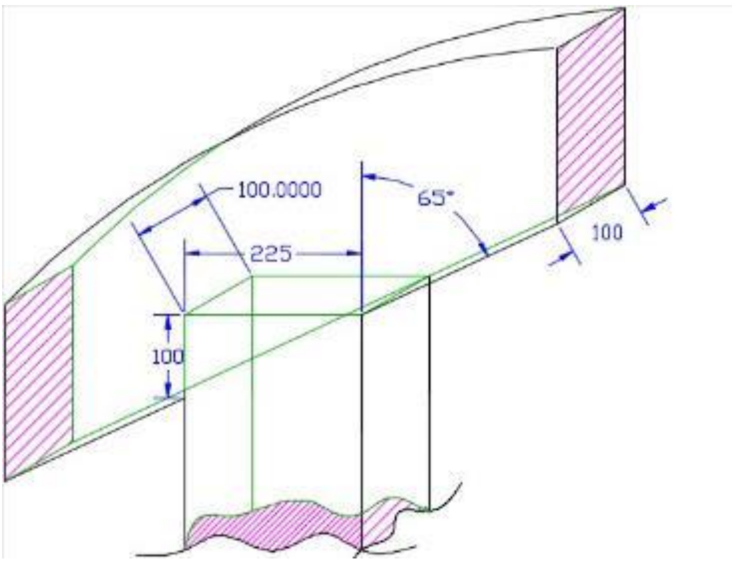


Figure 338 Notched bearing section detail

The design bearing capacity at an angle to the grain is calculated using Clause 3.2.6.3, where

$N_{d.l} > N_l^*$

$$Ndl \; = \; \emptyset k_1 k_4 k_6 f'l \, A'l$$

Where, $f'l$ is the chara. Capacity parallel to grain, A_l = bearing area for loading parallel to grain.

In this case, let the *main member* of the connection be the arch timber member.

Therefore, the capacity is calculated using:

$$N_{d\emptyset} = \frac{N_{d.l}N_{d.p}}{N_{d.l}Sin^2\emptyset + N_{d.p}Cos^2\emptyset}$$

Where, $N_{d,p}$ = 130,560.6N, Al = 222mm x 97mm (allowing for 3mm shrinkage) = 21,534mm^2, and $f'l$ = 23MPa (Table H2.2)

$N_{d.l}$ = $\emptyset k_1 k_4 k_6 f'_l A_{lp}$ = 0.75 x 0.94 x 1 x 1 x 1 x 23MPa x 21,534 mm^2

$N_{d.l}$ = 349.2kN

$$N_{d\emptyset} = \frac{(349.2kN)(130.56kN)}{(349.2kN)Sin^2(65)+(130.56)Cos^2(65)}$$

$N_{d\emptyset}$ = 146.9kN > 87.6kN

The bearing capacity of the notched connection is OK

3.6.3.6. *Bolt Design and Bearing Capacity Check*

Bolt Design; Internal Section

This connection sustains the highest loads in the support frame and therefore it is conservatively assumed that all bolt connections should adopt the final result from this critical case.

The design of the connection at the internal section must be considered as per AS1720-2010. The applicable standards and formulas used to design this connection are:

- AS1720-2012:
  - Clause 4.4.2.4; Equation 4.4(AS1720.1-2010) – Hankinson's Formula

$$Qsk = \frac{Qskl \ x \ Qskp}{Qskl(Sin^2\emptyset) + \ Qskp(Cos^2\emptyset)}$$

  - Table 4.10 A and B; and Table 4.9 A and B – Unseasoned Connection
  - Clause 4.4.4; Spacing, edge and end distance for bolts

The internal member joint currently holds 3 timber members together through an effective timber thickness (b_{eff}) of 100mm; for bolts parallel to grain - Table 4.9A and perpendicular to the grain - Table 4.10A of AS1720.1. The sketch below represents the connection of the internal members at the approximate centre point of the support structure.

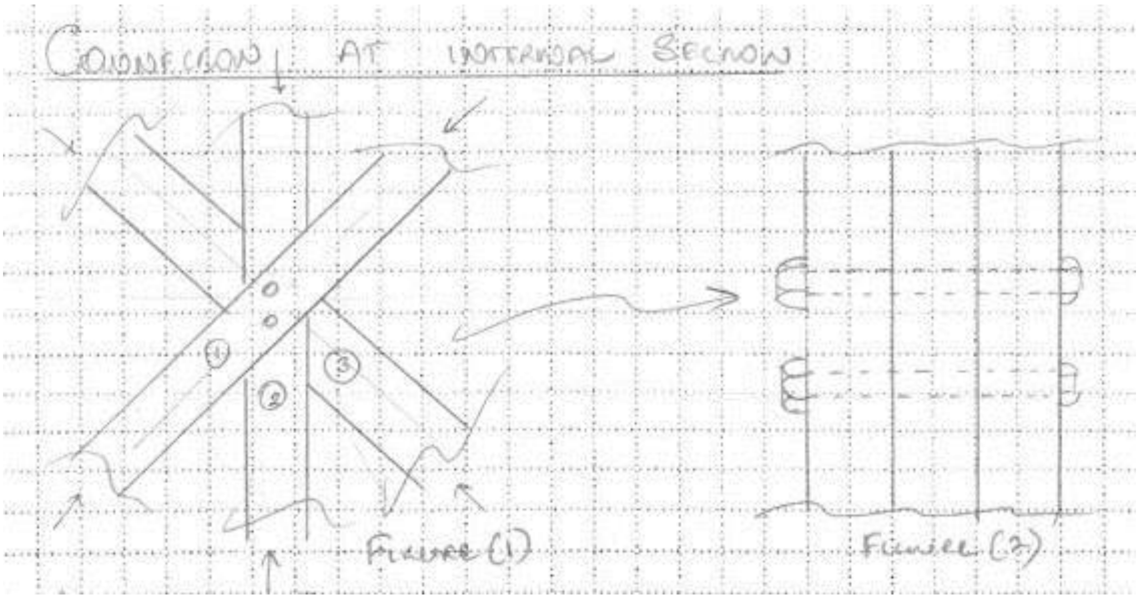


Figure 339 Sketch of internal member connection

All member thickness are 100mm. The loadings on these members are simplified by removing smaller axially loaded members from the figure, this simplification of the highest loadings acting on the connection are illustrated in Figure 340

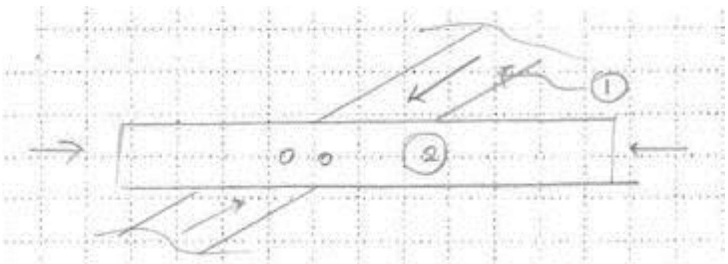


Figure 340 Simplification of joint connection via largest loadings

The highest loadings in member 1 and 2 are 53.5kN and 104.7kN respectively

Error! Reference source not found. The sketches shown below are used to determine the required bolt connection capacity. These sketches shown the angles between the inclined member and the straight member.

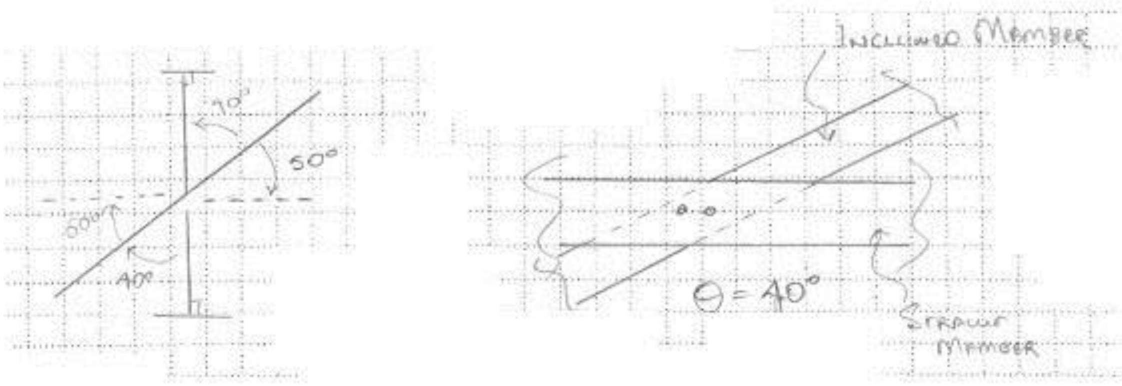


Figure 341 Sketch of internal joint with angles of members

Determining factors required for calculation of Equation 4.4(AS1720.1-2010).

- k_{16} depends on the No. of side plates given in the design, which is set to 1.0
- k_{17} depends on the No. and size of the chosen bolt, for this trial calculation, choosing M24 bolts in unseasoned timber and *n* is the number of bolts used, set n to 4; k_{17} 1.0, as per table 4.12.

The formula of $Q_{sk}Ø$ requires calculations for Qskp (Perpendicular to the grain) and Qskl (Parallel to the grain). Timber is non-ash, hardwood, as per Table H2.3, the joint group is J2 and strength group is S4.

The values of Q_{kl} are given in table 4.9(B) for unseasoned timber, where Q_{skl} = 2Qkl

Q_{skl} = 2(44,400)

Q_{skl} = 88,800N

Determining loading capacity perpendicular to the grain (Qskp)

Where Q_{kp} = Q_{sk}, as per Table 4.10(B)

Q_{skp} = 21,000N

Therefore, the characteristic capacity is calculated using Hankinson's Formula:

$$Qsk = \frac{Qskl\ x\ Qskp}{Qskl(Sin^2Ø) +\ Qskp(Cos^2Ø)}$$

Where Ø = 40°, Qskp = 26,880N and Qskl = 39,400N

$$Qsk = \frac{88{,}800\text{N}\ x\ 21{,}000\text{N}}{88{,}800\text{N}(Sin^2 40) + \ 21{,}000\text{N}(Cos^2 40)}$$

$Qsk =$ 38.05kN; this answer is used to evaluate maximum capacity.

The final capacity is calculated with regards to orientation of the member, furthermore the capacity of the timber member depends on whether the forces are acting parallel or perpendicular to the grain. For this case, the loading is acting parallel and at an incline to the horizontal/vertical. The capacities for the inclined and parallel direction are shown below.

The final capacity parallel to the grain:

$ØN_{d,j} = Øk_1k_{16}k_{17}nQsk$; Qsk = 21,000N

$ØN_{d,j}$ = 0.85 x 0.94 x 1.0 x 1.0 x 4 x 21,000N

$ØN_{d,j}$ = 67,116N > 53.5kN; OK for 4-M24 bolts

The final capacity inclined to the grain:

$ØN_{d,j} = Øk_1k_{16}k_{17}nQsk$; Qsk = 38.05N

$ØN_{d,j}$ = 0.85 x 0.94 x 1.0 x 1.0 x 4 x 38,050N

$ØN_{d,j}$ = 121,607.8N > 104.7kN; OK for 4-M24 bolts

Adopt 4-M24 Bolts for connection.

Bolt Design; Detailing

Bolt spacing, edge and end distances comply with Clauses 4.4.4.2 to 4.4.4.4 of AS1720.1-2010.

In this case, Cl4.4.4.4 states for loads acting 30 to 90 degrees shall be taken as perpendicular to the beam i.e. Cl4.4.4.3. Distance *a* is 2.5D for a b/D ratio of 2, and increased proportionately to 5D for a b/D of 6 or more.

Bolted Connection Detail for Internal Connection

Choose 225x100mm for all timber members, as per conclusion statement.

$$\frac{b}{D} = \frac{100}{24} = 4.167 < 6, therefore\ a \approx 3.85x24mm \approx 90mm$$

Edge distance shall be $4D = 4\ x\ 24mm = 96mm$

The figure below shows the allowable bolt zone for M16 Bolts, the figure displays the allowable zone for bolts to be connected in.

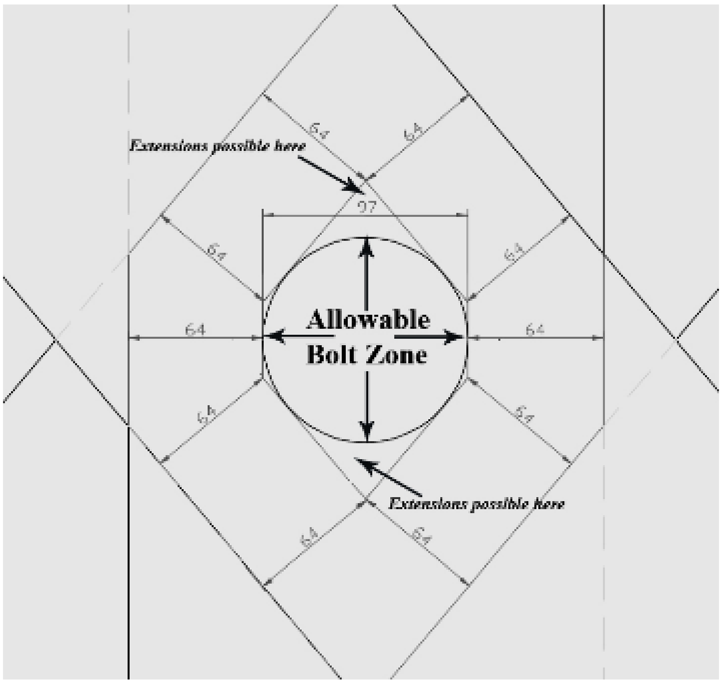


Figure 342 Internal Bolt, Allowable Zone for M16

This would not accommodate 6M16 to resist the applied capacity. It is recommended that 250x100mm internal members be used to provide enough room for bolt connections.

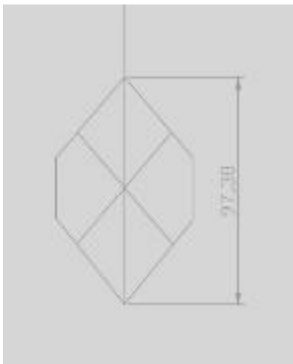


Figure 343 Internal Bolt, Allowable Zone for M24 for 250x100mm

The figure below shows that the spacing is insufficient for the M24 bolt diameter and a wider timber member. In this case, a wider timber member must be chosen – 300mmx100mm.

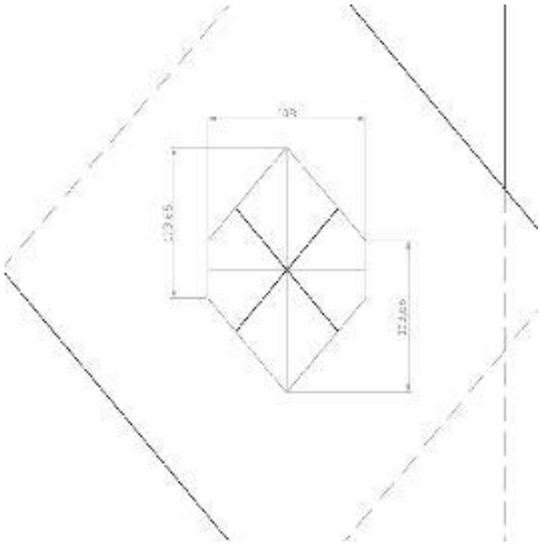


Figure 344 Internal Bolt, Allowable Zone for M24 for 300x100mm

The figure below shows that the spacing is OK, therefore adopt 300mmx100mm for all internal members from this point onward.

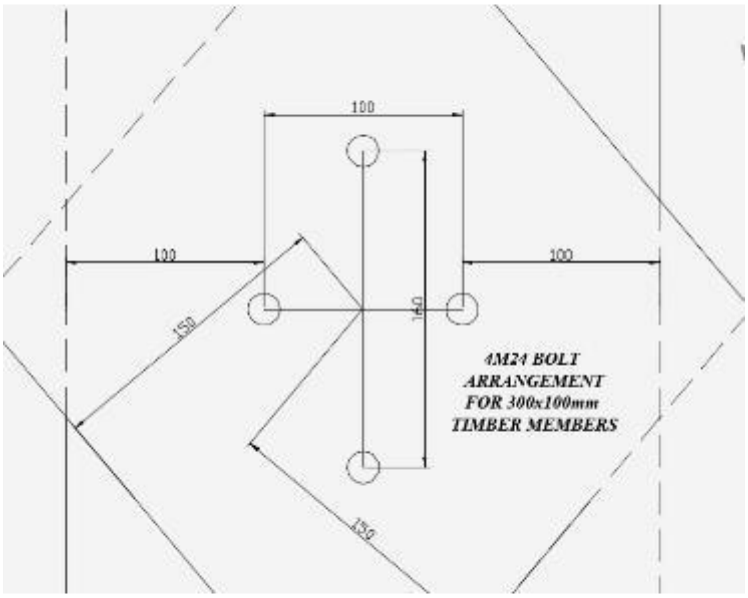


Figure 345 Internal Section Bolt Connection Detail for M24 for 300x100mm

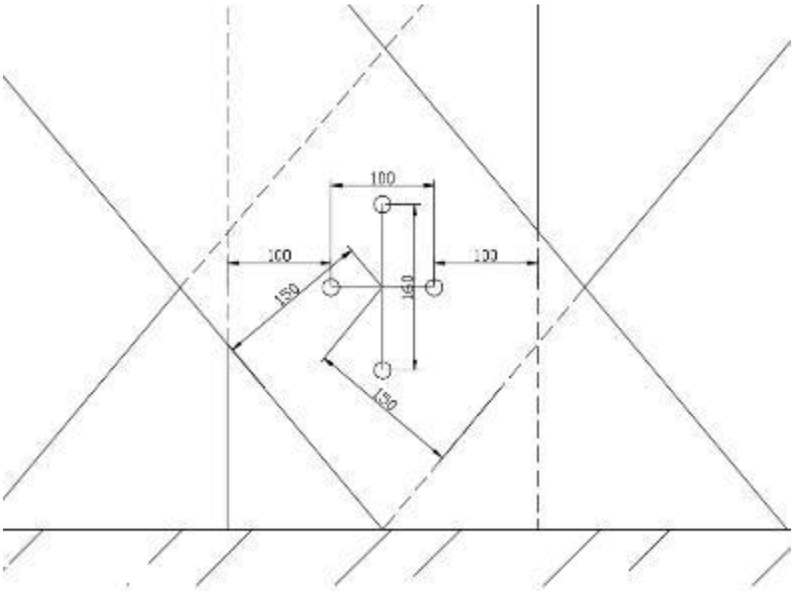


Figure 346 Support Bolt Connection Detail for M24 for 300x100mm

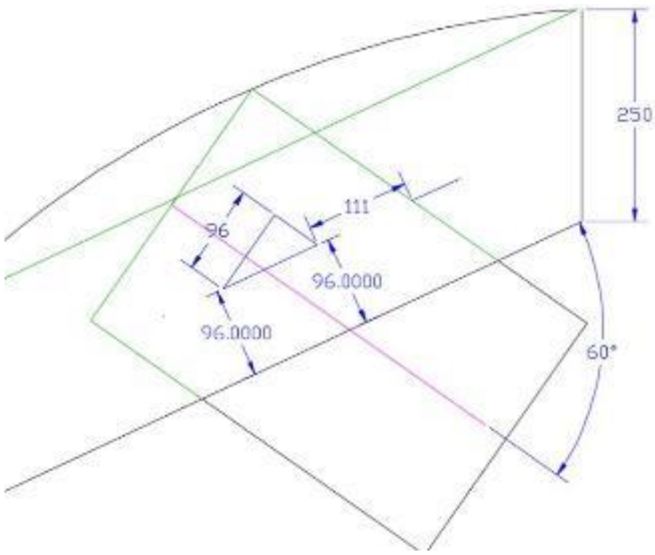


Figure 347 Arch and Internal Member, Allowable Zone for M24 for 300x100mm

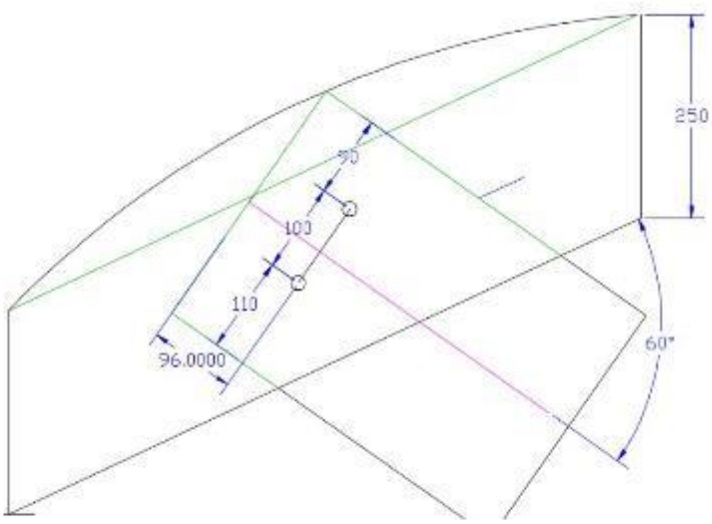


Figure 348 Arch and Internal Member Connection Detail for M24 for 300x100mm

3.6.3.7. Purlin/Joist Design

There exists several types of purlin design that can be utilised for the support system, these designs are shown below and are dependent on the chosen load width.

The diagram below shows the purlin and arch structure configuration for a 1m load width. Purlins are placed on the surface of three bearers i.e. arch timber members, over a total span of 2m. Two, Frame 2s are used for the middle and side arch structures to allow for 1m clearance for pipe installation.

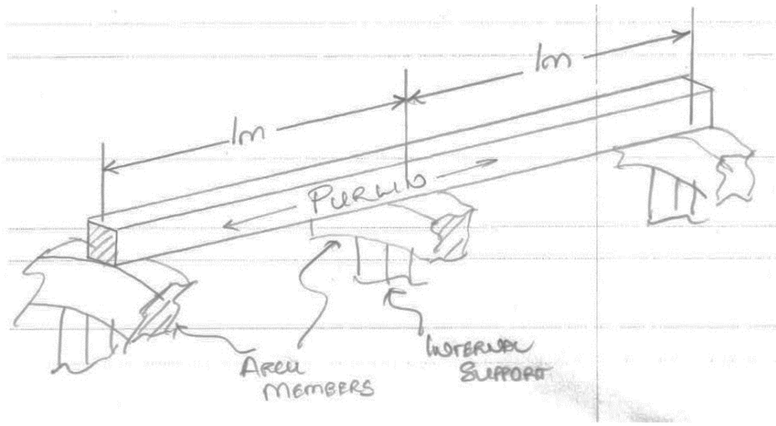


Figure 349 Purlin design for 1m load width (Hydro-Future)

The diagram below shows the purlin and arch structure configuration for a 2m load width. Similar to the 1m load width, the purlins replicate the same function of joists placed on the surface of two bearers i.e. arch timber members. This design consists of a two arch structures made from one Frame 1 and one Frame 2 supports.

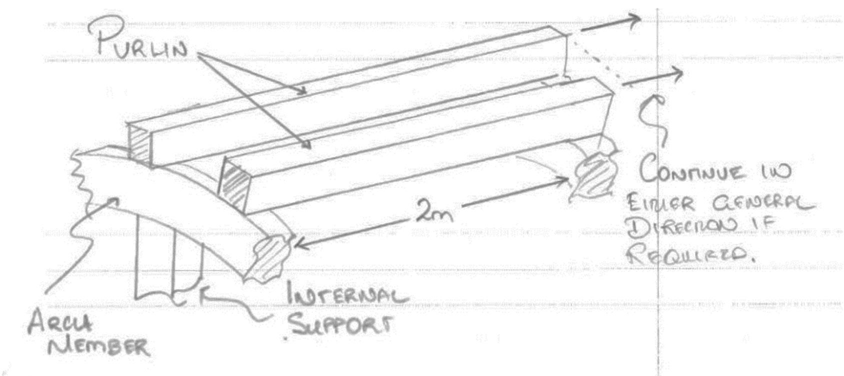


Figure 350 Purlin design for 2m load width

Applied Loadings

The applied loadings on both the 1m and 2m load widths shall fall within a load area envelope of *0.45m x LW* for each joist end. The length, 0.45m, is the distance between each node of the arch member given in Space Gass. It is assumed that placing a purlin at each of these nodes locations will increase simplicity of calculations. The diagram below shows the applied load envelop on each section of the 1m and 2m load widths.

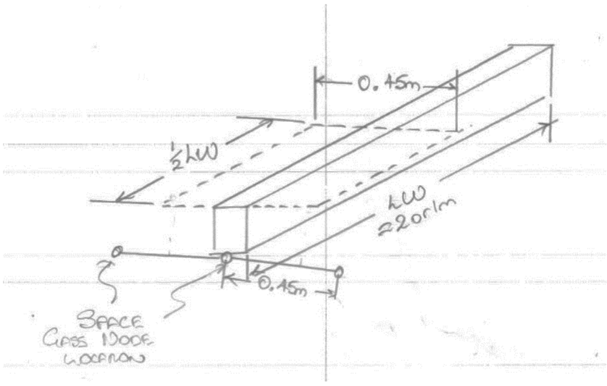


Figure 351 Load Area/Envelope for each joist

It is assumed that the purlin design is dependent upon the highest applied loading on the culvert structure within the load area shown in *Figure 352* The node locations are presented in the figure below.

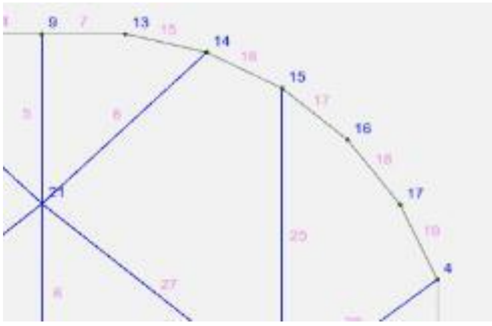


Figure 352 Node locations along arch of support system

The loadings at two node locations; 9 and 17, are shown in the tables below where the highest loading is chosen for the design of the purlin.

Table 144 - (Dead Load) Node 9 and 17; Soil, vertical and horizontal loads

Location	Soil Pressure Coefficient (k_a)	Load – UDL (kPa) Vertical	Horizontal
A≈9	0.42	15	6
E≈17	0.39	47	18

Table 145 - (Live Load) Node 9 and 17; M1600 Traffic Case 1, vertical and horizontal loads

Location	Soil Pressure Coefficient (k_a)	Load – UDL (kPa) Vertical	Horizontal
A≈9	0.42	29	12

E≈17	0.39	4	2

The summation of the vertical and horizontal pressures have the highest loadings are located at E or node 17.

The self-weight of the culvert is equal to 0.06 kN/m (kPa) for the vertical direction. The density of F34 Ash-Hardwood is 1.15 kN/m^3. The loadings acting at node 17 are shown in *Table 146*

Table 146 Vertical and horizontal loads at Node 17

Location	Load – UDL (kPa)	
	Vertical	*Horizontal*
Soil Load (G)	4	2
Traffic Load (LL)	47	18
Culvert Self-Weight (G)	0.06	-
Purlin Self-Weight (G)	0.05m x 1.15 kN/m^3 ≈ 0.06	-

For design simplification, it is assumed that the higher vertical loadings are acting perpendicular to the purlin. The ultimate load calculation for these loadings is shown below

$$Wu = 1.2G + 1.5Q$$

$$Wu = 1.2(4kPa + 2x0.6kPa) + 1.5(47kPa)$$

$$\boldsymbol{Wu = 76.7\ kPa}$$

The UDL acting on a load width of 0.45m (i.e. distance between nodes and purlins) is:

$$UDL = LW\ x\ 76.7\ kPa$$

$$UDL = 0.45m\ 76.7\ kPa$$

$$\boldsymbol{UDL = 34.5\ kN/m}$$

Decreasing the distance between purlins will decrease applied UDL i.e. decrease to 0.2m

$$UDL = 0.1m\ 76.7\ kPa$$

$$UDL = 0.1m\ 76.7\ kPa$$

$$\boldsymbol{UDL = 7.7\ kN/m}$$

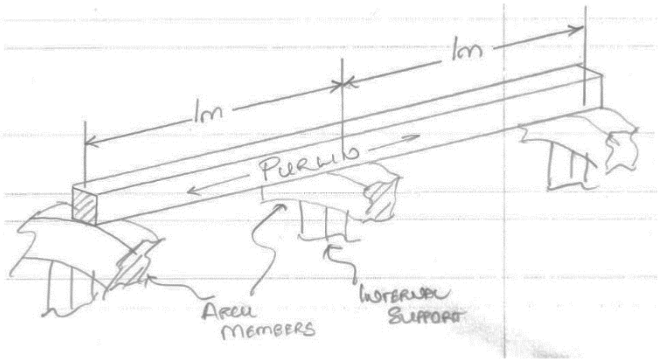
The distance between purlins is directly proportional to the calculated UDL.

Design Calculations

The preliminary size for all purlin timber members is 50x50mm. Assume that purlin is glued to the surface of arch culvert.

The first calculation is for a LW of 1m, as per figure below.

Table 147 Figure of design scenario, LW = 1m



Since the purlin is statically indeterminate, the support system must be analysed using a finite element software i.e. Space Gass.

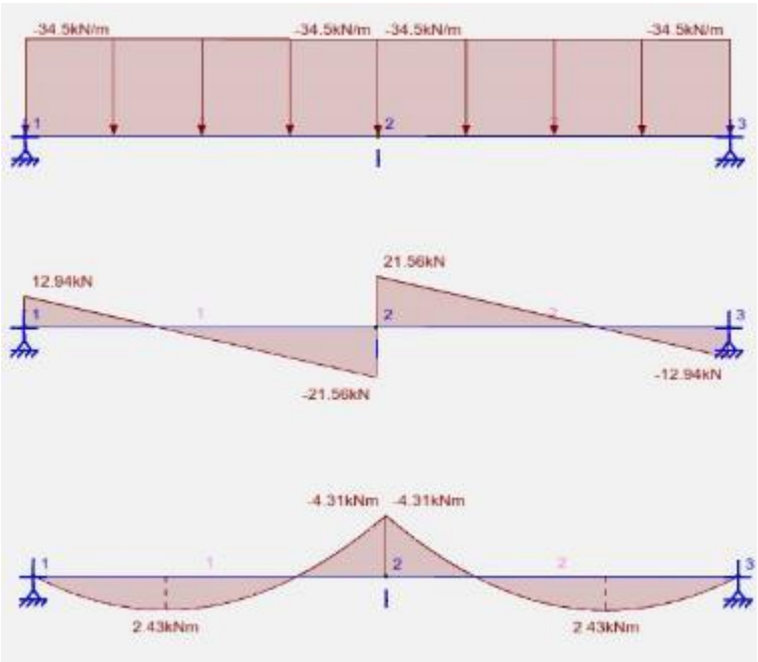


Figure 353 Purlin SFD and BMD

Solving for serviceability deflection

As per Appendix G of AS1720.1-2010, Table G1 gives the load combinations for specific applications and actions. The load combination used to determine serviceability deflection is shown below.

$S = 1.2G + 1.5Q$ (Short Term Imposed Action)

Therefore, S = W_u = 34.5 kN/m

The length between supports is 1m, E = 21,500 MPa, Ix = 520,833.33mm^4 (For 50x50mm) and J_2 = 1.0 for short term loadings. The maximum deflection of a joist ranges from span/400 to span/250, in this case choose span/250 = 1000mm/250 = 4mm

$$\delta_s = \sum (j_2 \frac{5}{384} \frac{wL^4}{EI})$$

$$\delta = 1 \text{ x } \frac{5(34.5kN/m)(1000mm)^4}{384(21{,}500MPa)(520{,}833.3mm^4)}$$

δ =200mm – Not satisfactory.

The deflection is too large and must be reduced. Deflection is reduced by decreasing distance between purlins or increasing member size, however an increased purlin size could change the dimensions of the entire arch culvert system and is therefore not recommended.

Decreasing distance between purlins to 0.2m i.e. UDL = 15.3 kN/m

$$\delta = 1 \text{ x } \frac{5(15.3kN/m)(1000mm)^4}{384(21{,}500MPa)(520{,}833.3mm^4)}$$

δ = 17.8mm < 4mm, Not acceptable, however since the structure is temporary and is not required for aesthetic improvement of its surroundings, a value for serviceability can be assumed i.e. 20mm. In such a case, this serviceability deflection is OK.

Solving for Flexural Capacity

$Md = \emptyset k_1 k_4 k_6 k_9 k_{12} f'b\ Z$, *Where f'b = 84 MPa*

The factors for F34 timber in a short-term case scenario are: $\emptyset = 0.75$, $k_1 = 0.94$, k_4 *= 1.0 and* k_6 *= 1.0*. All other factors are calculated below.

Strength Sharing Factor, k_9 is calculated using Clause 2.4.5.3, where:

$$k_9 = g31 + (g32 - g31)[1 - 2s/L],$$

where g_{31} and g_{32} are derived from Table 2.7, s = centre to centre spacing, and L is effective span of parallel members. In this case, there exist approximately 10 nodes thus 20 purlins in parallel that sustain the loading, thus n_{com} = 1 and n_{mem} = 20. From Table 2.7 of AS1720, g_{31} = 1.0 and g_{32} = 1.33, s = 200mm and L = 1m

k_9 = 1.0 + (1.33 – 1.0)[1 – 2(200mm)/1000mm],

k_9 = 1.2

Stability Factor, k_{12} is calculated using Clause 3.2.4,, where:

$$S_1 = 1.25\frac{d}{b}\left(\frac{L_{ay}}{d}\right)^{0.5}$$

The values used for this member are: d = 222mm, and b = 97mm. L_{ay} = 600mm:

$$S_1 = 1.25\frac{50mm}{50mm}\left(\frac{1000mm}{50mm}\right)^{0.5}$$

$$= 5.6$$

The value of ρ_b for F34 unseasoned hardwood, the critical value, is 1.21, therefore:

$$\rho_b S = 5.6\ x\ 1.21 = 6.7$$

k_{12} is taken as 1.0.

Therefore:

$$Md = \emptyset k_1 k_4 k_6 k_9 k_{12} f'b\ ZM_d =$$

$$0.75\ x\ 1.0\ x\ 1.0\ x\ 1.2\ x\ 1.0\ x\ 84\ MPa\ x\ \frac{(47mm\ x\ (47mm)^2)}{6} mm^3$$

$$M_d = 1.3\ kNm$$

Therefore, $M_d < M^*$, *where* M^* = 4.31 kNm

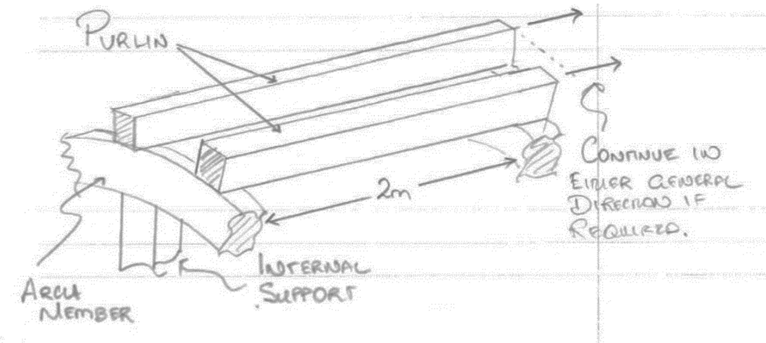
Not good, reduce distance between purlins to 100mm i.e. UDL = 0.1m x 76kPa.

SpaceGass output for M* = -0.96 kNm < M_d

Therefore, for 1m load width purlin configuration design, adopt 50x50mm at 100cts.

The first calculation is for a LW of 1m, as per figure below.

Table 148 Figure of design scenario, LW = 2m



All results are replicated with respect to a 2 m length purlin instead of a 1m length purlin. The final capacity and chosen member size is shown below. No calculations are repeated

LW = 100mm i.e. 100 cts, therefore UDL = 7.7 kN/m

$M^*_{(SpaceGass)}$ = 3.85kNm

M_d for 50x50 not enough, choosing 50x100

$$M_d = 0.75\ x\ 1.0\ x\ 1.0\ x\ 1.2\ x\ 1.0\ x\ 84\ MPa\ x\ \frac{(57mm\ x\ (97mm)^2)}{6} mm^3$$

$$M_d = 5.57\ \text{kNm} > M^*$$

Therefore, for 2m load width purlin configuration design, adopt 50x100mm at 100cts.

Connection between Arch and Purlin Members

3.6.3.8. Column to Base Design

The base of the arch culvert has a maximum stress capacity and therefore must incorporate a base structure to resist the loadings that are acting on it. The connection from the arch culvert members to the base is shown in the figure below.

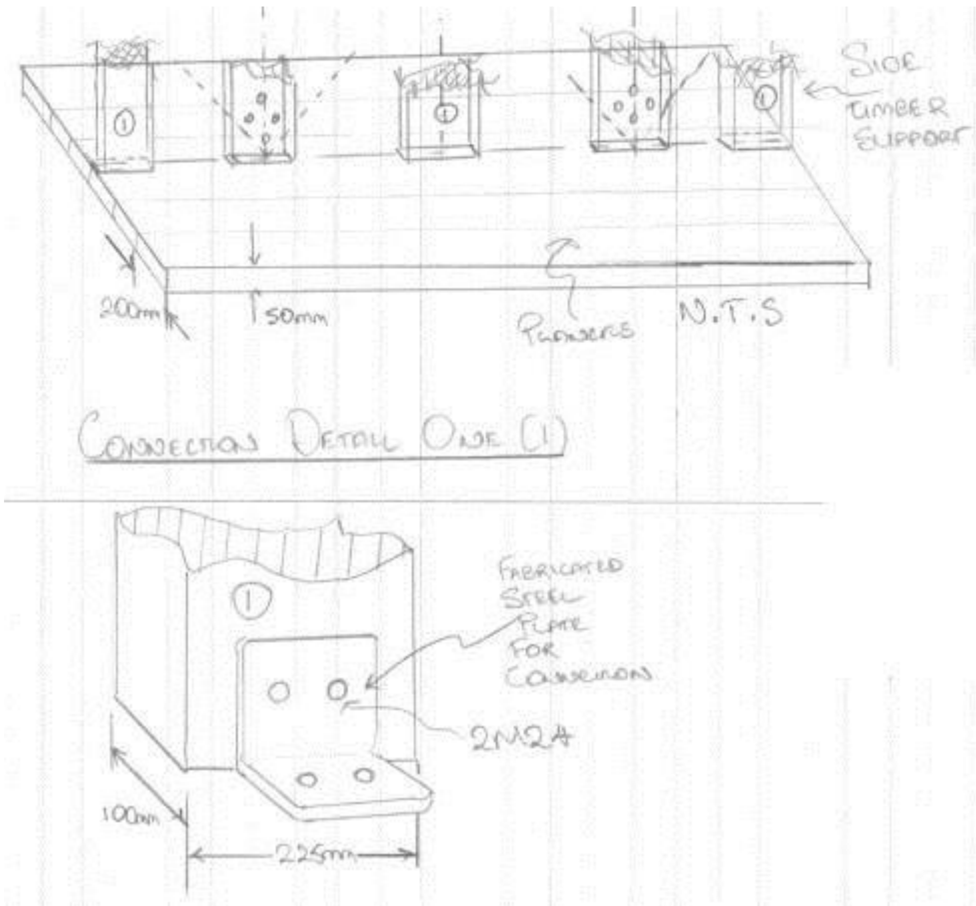


Figure 354 Sketch of connection detail for column to base

All calculations for bearing capacity of the timber members, which states that the bearing capacity achieved was 130,560.6N perpendicular to the grain for 225x100mm. The chosen section sizes for the internal members are 300x100mm, therefore the new capacity in bearing is approximately 174kN, which satisfies all axial ultimate loadings.

The output reactions at the base connection is shown in the figure below.

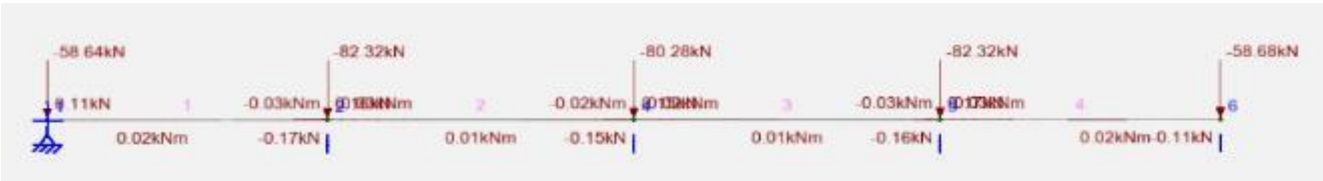


Figure 355 Output Reaction Forces, BMD and SFD on base support plank of wood

These reaction forces are distributed onto a *single* section of F34, Ash timber, approximately 250x25mm in size. The output bending moment diagram and shear force diagram are relatively small i.e. < 0.03kNm and < 0.17kN, therefore it is assumed that the chosen member is sufficiently strong enough to induce the applied loadings. The final drawing for the connection at the positions shown in Figure 355 is presented in the figure below.

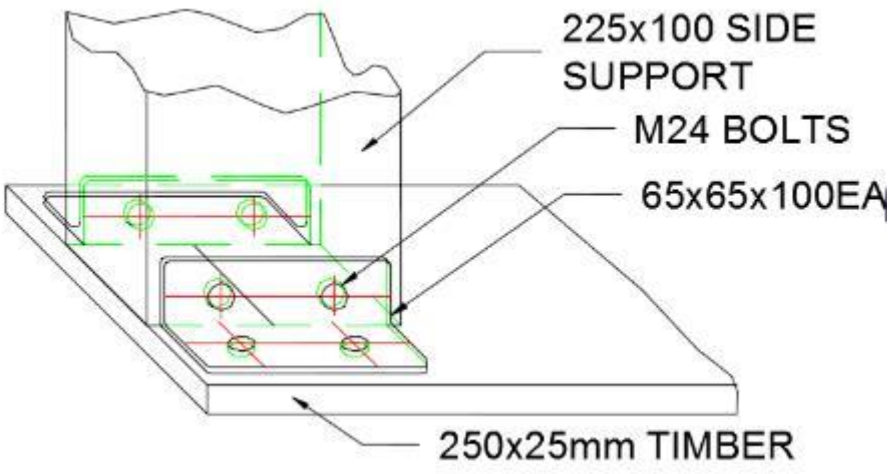


Figure 356 Column to base connection detail

3.6.4. Summary of Design Capacity Results

3.6.4.1. Frame 1 Capacity Results

Table 149 Side Timber Support Member Capacities (FRAME 1)

Side Timber Support Member Capacities (FRAME 1)			
Capacity Tested	**Ultimate Load**	**Dimensions Tested**	**Capacity Achieved**
Compression	1m Load-width (Nc* = 48.9 kN) 2m Load-width (Nc* = 97.8kN)	225x100mm 125x100mm 75x50mm	971.6kN; OK for 1m and 2m 534.0kN; OK for 1m and 2m 154.8kN; OK for 1m and 2m
Shear	1m Load-width (Ns* = 16.5kN) at 1.5d from support = 13.5kN 2m Load-width (Ns* = 33.1kN)	125x100mm 75x50mm 100x100mm 225x100mm	36.1kN; OK for 1m and 2m 10.32kN; NOT OK 28.7kN; OK for 1m 65.7kN; OK for 1m and 2m
Flexural Bending	1m Load-width (M* = 5.2kNm) 2m Load-width (M* = 10.3kNm)	100x50mm 125x50mm	4.64kNm; NOT OK 7.45kNm; OK for 1m
Deflection	Deflection (Max = 2mm)	125x50mm	1.6mm Deflection; OK

Table 150 Internal Timber Support Member Capacities (FRAME 1)

Internal Timber Support Member Capacities (FRAME 1)			
Capacity Tested	**Ultimate Loads**	**Dimensions Tested**	**Capacity Achieved**
Compression	1m Load-width (Nc* = 94.3kN) 2m Load-width (Nc* = 133.7kN)	200x100mm 100x100mm 50x100mm	629.4kN; OK for 1m and 2m 310.0kN ; OK for 1m and 2m 191.3kN; OK for 1m
Shear	1m Load-width (Ns* = 0.5kN) 2m Load-width (Ns* = 0.9kN)	125x100mm 75x50mm 100x100mm 225x100mm	36.1kN; OK for 1m and 2m 10.32kN; OK for 1m and 2m 28.7kN; OK for 1m and 2m 65.7kN; OK for 1m and 2m
Flexural Bending	1m Load-width (M* = -0.6kNm) 2m Load-width (M* = -0.9kNm)	50x100mm	4.01 kNm; OK for 1m and 2m
Deflection	(Max = 8mm)	50x100mm 225x100mm	104 mm Deflection; NOT OK 4.3mm Deflection; OK

Table 151 Timber Arch Support Member Capacities (FRAME 1)

Timber Arch Member Capacities (FRAME 1)			
Capacity Tested	**Capacity Tested**	**Dimensions Tested**	**Capacity Achieved**
Compression	1m Load-width (Nc* = 55.1kN) 2m Load-width (Nc* = 103.6kN)	225x100mm 125x100mm 75x50mm	971.6kN; OK for 1m and 2m 534.0kN; OK for 1m and 2m 154.8kN; OK for 1m and 2m
Shear	1m Load-width (Ns* = 25.2kN) at 1.5d = 19.5kN 2m Load-width (Ns* = 50.41kN) at 1.5d = 33.1kN	125x100mm 75x50mm 100x100mm 225x100mm	$36.1kN$; OK for 1m 10.32kN; NOT OK 28.7kN; OK for 1m 65.7kN; OK for 1m and 2m
Flexural Bending	1m Load-width (M* = 5.7kNm) 2m Load-width (M* = 11.3kNm)	100x100mm	9.58kNm; OK for 1m
Deflection	(Max = 2.3mm)	100x100mm	1.2mm Deflection; OK

3.6.4.2. Frame 2 Capacity Results

Table 152 Side Timber Support Member Capacities (FRAME 2)

Side Timber Support Member Capacities (FRAME 2)			
Capacity Tested	**Ultimate Load**	**Dimensions Tested**	**Capacity Achieved**
Compression	1m Load-width (Nc* = 88.5 kN) 2m Load-width (Nc* = 172.1kN)	225x100mm 125x100mm 75x50mm	831.6 kN; OK for 1m and 2m 393.9 kN; OK for 1m and 2m 86.9kN; OK for 1m
Shear	1m Load-width (Ns* = 16.5kN) 2m Load-width (Ns* = 33.1kN)	125x100mm 75x50mm 100x100mm 225x100mm	$36.1kN$; OK for 1m and 2m 10.32kN; NOT OK 28.7kN; OK for 1m 65.7kN; OK for 1m and 2m
Flexural Bending	1m Load-width (M* = 5.2kNm) 2m Load-width (M* = 10.3kNm)	100x50mm 125x50mm	4.64kNm; NOT OK 7.45kNm; OK for 1m
Deflection	(Max = 2.3mm)	100x100mm	1.2mm Deflection; OK

Table 153 Internal Timber Support Member Capacities (FRAME 2)

Internal Timber Support Member Capacities (FRAME 2)			
Capacity Tested	**Capacity Tested**	**Dimensions Tested**	**Capacity Achieved**
Compression	1m Load-width (Nc* = 94.0kN) 2m Load-width (Nc* = 194.5kN)	200x100mm 100x100mm 50x100mm	527.2kN; OK for 1m and 2m 207.6kN; OK for 1m and 2m 100.6kN; OK for 1m
Shear	1m Load-width (Ns* = 0.0kN) 2m Load-width (Ns* = 0.0kN)	125x100mm 75x50mm 100x100mm 225x100mm	$36.1kN$; OK for 1m and 2m 10.32kN; OK for 1m and 2m 28.7kN; OK for 1m and 2m 65.7kN; OK for 1m and 2m
Flexural Bending	1m Load-width (M* = 0.4kNm) 2m Load-width (M* = 0.0kNm)	50x100mm	4.01 kNm; OK for 1m and 2m
Deflection	(Max = 8mm)	50x100mm 225x100mm	104 mm Deflection; NOT OK 4.3mm Deflection; OK

Table 154 Timber Arch Support Member Capacities (FRAME 2)

Timber Arch Member Capacities (FRAME 2)			
Capacity Tested	**Capacity Tested**	**Dimensions Tested**	**Capacity Achieved**
Compression	1m Load-width (Nc* = 55.1kN) 2m Load-width(Nc* =168.4kN)	225x100mm 125x100mm 75x50mm	831.6 kN; OK for 1m and 2m 393.9 kN; OK for 1m and 2m 86.9kN; OK for 1m
Shear	1m Load-width (Ns* = 25.2kN) at 1.5d = 19.5kN 2m Load-width (Ns* = 56.7kN) at 1.5d = 37.2kN	125x100mm 75x50mm 100x100mm 225x100mm	$36.1kN$; OK for 1m and 2m 10.32kN; NOT OK 28.7kN; OK for 1m 65.7kN; OK for 1m and 2m
Flexural Bending	1m Load-width (M* = 3.2kNm) 2m Load-width(M* =10.4kNm)	100x100mm 125x100mm	9.58kNm; OK for 1m 11.98kNm; OK for 1m and 2m
Deflection	(Max = 1.5mm)	100x100mm	0.7mm Deflection; OK

3.6.4.3. Summary Conclusion

The summary of results illustrated in the sections above shows that there isn't much difference between Frame 1 and Frame 2 ultimate loadings, and that the capacities at a 1m load width for most of the dimensions tested was satisfactory. The ultimate loadings for the larger; 2m load width indicated that most trial member sizes had failed in shear and compression capacity checks.

The minimum size for all timber members for both 1m and 2m load widths and frame 1 and 2 are summarised in the tables below.

Table 155 Frame 1 Minimum dimensions (Hydro-Future)

Frame 1 Minimum Dimensions			
Member:	**1m Load Width Dimensions:**	**2m Load Width Dimensions:**	**Governing Calculation**
Side Timber Support	100x50mm	225x100mm	Compression
Internal Support	300x100mm	300x100mm	Deflection & Bolt Connection
Timber Arch Support	125x100mm	225x100mm	Compression & Shear

Table 156 Frame 2 Minimum dimensions (Hydro-Future)

Frame 2 Minimum Dimensions			
Member:	**1m Load Width Dimensions:**	**2m Load Width Dimensions:**	**Governing Calculation**
Side Timber Support	100x100mm	225x100mm	Compression
Internal Support	300x100mm	300x100mm	Deflection & Bolt Connection
Timber Arch Support	125x100mm	225x100mm	Compression & Shear

The minimum sizes displayed in the tables above show that the member sizes between an applied load width of 1 and 2 metres does not vary drastically. Since these sizes remain fairly consistent, it is assumed that adopting the dimensions for Frame 2; Table 156 is acceptable for the final design. Load width 1m required more material than load width 2m, therefore load width 2m is chosen for the final design which incorporates dimensions shown in Table 156.

3.7. Space Gass Output

The output from space gass for the critical, 2m load widths for both frames is presented below.

3.7.1. FRAME 1 Space Gass Output (2m Load Width)

```
SPACE GASS 12.23 - STUDENT VERSION - NOT FOR COMMERCIAL USE
Path: C:\Users\Michael Renko\Desktop\STRUCTURAL SUPPORT SYSTEM 0.5
Designer:  Date: Friday, May 29, 2015 12:26 AM Page: 1

ANALYSIS STATUS REPORT
----------------------

Job name ...... STRUCTURAL SUPPORT SYSTEM 0.5
Location ...... C:\Users\Michael Renko\Desktop

Length units .......................... m
Section property units ................ mm
Material strength units ............... MPa
Mass density units .................... T/m^3
Temperature units ..................... Celsius
Force units ........................... kN
Moment units .......................... kNm
Mass units ............................ T
Acceleration units .................... g's
Translation units ..................... mm
Stress units .......................... MPa

Nodes ..............................       19  (  32765)
Members ............................       24  (  32765)
Plates .............................        0  (  32765)
Restrained nodes ...................        5  (  32765)
Nodes with spring restraints .......        0  (  32765)
Section properties .................        4  (   5000)
Material properties ................        1  (    999)
Constrained nodes ..................        0  (  32765)
Member offsets .....................        0  (  32765)

Node loads .........................        0  ( 250000)
Prescribed node displacements ......        0  ( 250000)
Member concentrated loads ..........        0  ( 250000)
Member distributed forces ..........       66  ( 250000)
Member distributed torsions ........        0  ( 250000)
Thermal loads ......................        0  ( 250000)
Member prestress loads .............        0  ( 250000)
Plate pressure loads ...............        0  ( 250000)
Self weight load cases .............        1  (  10000)
Combination load cases .............        1  (  10000)
Load cases with titles .............        5  (  10000)
Lumped masses ......................        0  ( 250000)
Spectral load cases ................        0  (  10000)

Static analysis ....................        Y
Dynamic analysis ...................        N
Response analysis ..................        N
Buckling analysis ..................        N
Ill-conditioned ....................        N
Non-linear convergence .............        Y
Frontwidth .........................       34
Total degrees of freedom ...........      102
Static load cases ..................        4  (  10000)
Mass load cases ....................        1  (  10000)


NODE COORDINATES (m)
----------------

                 X          Y          Z
  Node       Coord      Coord      Coord

     1       0.000      0.000      0.000
     2       0.000      0.600      0.000
     3       4.200      0.000      0.000
     4       4.200      0.600      0.000
     5       2.100      0.000      0.000
     6       0.197      0.996      0.000
     7       0.477      1.340      0.000
     9       2.100      1.900      0.000
    10       0.825      1.613      0.000
    11       1.225      1.803      0.000
```

12	1.657	1.900	0.000
13	2.543	1.900	0.000
14	2.975	1.803	0.000
15	3.375	1.613	0.000
16	3.723	1.340	0.000
17	4.003	0.996	0.000
18	0.825	0.000	0.000
19	3.375	0.000	0.000
21	2.100	1.000	0.000

MEMBER DATA (deg,kNm/rad,m)
----------- (F-Fixed, R-Released) (*-Cable length)

Memb	Dir Angle	Dir Node	Dir Axis	Memb Type	Node A	Node B	Sect	Mat	Node A Fixity	Node B Fixity	Length
1	0.00			Norm	1	2	1	1	FFFFFF	FFFFFF	0.600
3	0.00			Norm	3	4	1	1	FFFFFF	FFFFFF	0.600
5	0.00			Norm	21	9	2	1	FFFFFF	FFFFFF	0.900
6	0.00			Norm	21	5	2	1	FFFFFF	FFFFFF	1.000
7	0.00			Norm	9	13	1	1	FFFFFF	FFFFFF	0.443
8	0.00			Norm	21	14	2	1	FFFFFF	FFFFFF	1.187
9	0.00			Norm	2	6	1	1	FFFFFF	FFFFFF	0.443
10	0.00			Norm	6	7	1	1	FFFFFF	FFFFFF	0.443
11	0.00			Norm	7	10	1	1	FFFFFF	FFFFFF	0.443
12	0.00			Norm	10	11	1	1	FFFFFF	FFFFFF	0.443
13	0.00			Norm	11	12	1	1	FFFFFF	FFFFFF	0.443
14	0.00			Norm	12	9	1	1	FFFFFF	FFFFFF	0.443
15	0.00			Norm	13	14	1	1	FFFFFF	FFFFFF	0.443
16	0.00			Norm	14	15	1	1	FFFFFF	FFFFFF	0.443
17	0.00			Norm	15	16	1	1	FFFFFF	FFFFFF	0.443
18	0.00			Norm	16	17	1	1	FFFFFF	FFFFFF	0.443
19	0.00			Norm	17	4	1	1	FFFFFF	FFFFFF	0.443
20	0.00			Norm	21	11	2	1	FFFFFF	FFFFFF	1.187
22	0.00			Norm	10	18	2	1	FFFFFF	FFFFFF	1.613
23	0.00			Norm	18	2	2	1	FFFFFF	FFFFFF	1.020
25	0.00			Norm	15	19	2	1	FFFFFF	FFFFFF	1.613
26	0.00			Norm	19	4	2	1	FFFFFF	FFFFFF	1.020
27	0.00			Norm	21	19	2	1	FFFFFF	FFFFFF	1.620
28	0.00			Norm	21	18	2	1	FFFFFF	FFFFFF	1.620

NODE RESTRAINTS (kN/m,kNm/rad)
---------------- (F-Fixed, R-Released, S-Spring, *-General)

Node	Rest Code	X Axial Stiffness	Y Axial Stiffness	Z Axial Stiffness	X Rotation Stiffness	Y Rotation Stiffness	Z Rotation Stiffness
1	FFFFRR						
3	RFFRRR						
5	RFFRRR						
18	RFFRRR						
19	RFFRRR						

SECTION PROPERTIES (mm,mm^2,mm^4,deg)

Sect	Name	Mark	Shape	Source
1	225x100	S1	Solid rect	AustTimb
2	200x100	S2	Solid rect	AustTimb
3	300x300	S3	Solid rect	AustTimb
4	200x200	S4	Solid square	User

Sect	Area of Section	Torsion Constant	Y-Axis Mom of In	Z-Axis Mom of In	Y-Axis Shr Area	Z-Axis Shr Area	Princ Angle
1	2.2500E+04	5.4068E+07	1.8750E+07	9.4922E+07	Infinite	Infinite	0.00
2	2.0000E+04	4.5775E+07	1.6667E+07	6.6667E+07	Infinite	Infinite	0.00
3	7.5000E+04	7.7484E+08	3.9062E+08	5.6250E+08	Infinite	Infinite	0.00
4	4.0000E+04	2.2496E+08	1.3333E+08	1.3333E+08	Infinite	Infinite	0.00

Sect	Shape	Trans	Mir	Rotate	D	Bt/Bb	Btw/Bbw	Tt/Tb	Tw/Rr
1	Solid rect	No	No	0.00	225.00	100.00	0.00	0.00	0.00
						0.00	0.00	0.00	0.00
2	Solid rect	No	No	0.00	200.00	100.00	0.00	0.00	0.00
						0.00	0.00	0.00	0.00
3	Solid rect	No	No	0.00	300.00	250.00	0.00	0.00	0.00
						0.00	0.00	0.00	0.00
4	Solid square	No	No	0.00	200.00	0.00	0.00	0.00	0.00
						0.00	0.00	0.00	0.00

MATERIAL PROPERTIES (MPa,T/m^3,strain/degC)

Matl	Material Name	Young's Modulus	Poisson's Ratio	Mass Density	Coeff of Expansion	Concrete Strength
1	F34 NonAsh HW U	2.1500E+04	0.37	1.1500E+00	0.000E+00	

MEMBER DISTRIBUTED FORCES (m,kN/m)

Load Case	Memb	Sub Load	Axes Sys	Start Position	Finish Position	X Start/ Finish	Y Start/ Finish	Z Start/ Finish
2	1	1	GI	0.000%	100.000%	23.000	0.000	0.000
						18.000	0.000	0.000
	3	1	GI	0.000%	100.000%	-23.000	0.000	0.000
						-18.000	0.000	0.000
	7	1	GI	0.000%	100.000%	-6.000	-15.000	0.000
						-7.000	-15.000	0.000
	9	1	GI	0.000%	100.000%	18.000	-47.000	0.000
						13.000	-34.000	0.000
	10	1	GI	0.000%	100.000%	13.000	-34.000	0.000
						10.000	-24.000	0.000
	11	1	GI	0.000%	100.000%	10.000	-24.000	0.000
						9.000	-21.000	0.000
	12	1	GI	0.000%	100.000%	9.000	-21.000	0.000
						8.000	-19.000	0.000
	13	1	GI	0.000%	100.000%	8.000	-19.000	0.000
						7.000	-15.000	0.000
	14	1	GI	0.000%	100.000%	7.000	-15.000	0.000
						6.000	-15.000	0.000
	15	1	GI	0.000%	100.000%	-7.000	-15.000	0.000
						-8.000	-19.000	0.000
	16	1	GI	0.000%	100.000%	-8.000	-19.000	0.000
						-9.000	-21.000	0.000
	17	1	GI	0.000%	100.000%	-9.000	-21.000	0.000
						-10.000	-24.000	0.000
	18	1	GI	0.000%	100.000%	-10.000	-24.000	0.000
						-13.000	-34.000	0.000
	19	1	GI	0.000%	100.000%	-13.000	-34.000	0.000
						-18.000	-47.000	0.000
3	1	1	GI	0.000%	100.000%	2.000	0.000	0.000
						2.000	0.000	0.000
	3	1	GI	0.000%	100.000%	-2.000	0.000	0.000
						-2.000	0.000	0.000

	7	1	GI	0.000%	100.000%	-12.000	-29.000	0.000
						-10.500	-29.000	0.000
	9	1	GI	0.000%	100.000%	2.000	-4.000	0.000
						3.000	-7.000	0.000
	10	1	GI	0.000%	100.000%	3.000	-7.000	0.000
						5.000	-12.000	0.000
	11	1	GI	0.000%	100.000%	5.000	-12.000	0.000
						6.000	-19.000	0.000
	12	1	GI	0.000%	100.000%	6.000	-19.000	0.000
						9.000	-24.000	0.000
	13	1	GI	0.000%	100.000%	9.000	-24.000	0.000
						10.500	-29.000	0.000
	14	1	GI	0.000%	100.000%	10.500	-29.000	0.000
						12.000	-29.000	0.000
	15	1	GI	0.000%	100.000%	-10.500	-29.000	0.000
						-9.000	-24.000	0.000
	16	1	GI	0.000%	100.000%	-9.000	-24.000	0.000
						-6.000	-19.000	0.000
	17	1	GI	0.000%	100.000%	-6.000	-19.000	0.000
						-5.000	-12.000	0.000
	18	1	GI	0.000%	100.000%	-5.000	-12.000	0.000
						-3.000	-7.000	0.000
	19	1	GI	0.000%	100.000%	-3.000	-7.000	0.000
						-2.000	-4.000	0.000
4	1	1	GI	0.000%	100.000%	0.000	-0.060	0.000
						0.000	-0.060	0.000
	3	1	GI	0.000%	100.000%	0.000	-0.060	0.000
						0.000	-0.060	0.000
	7	1	GI	0.000%	100.000%	0.000	-0.060	0.000
						0.000	-0.060	0.000
	9	1	GI	0.000%	100.000%	0.000	-0.060	0.000
						0.000	-0.060	0.000
	10	1	GI	0.000%	100.000%	0.000	-0.060	0.000
						0.000	-0.060	0.000
	11	1	GI	0.000%	100.000%	0.000	-0.060	0.000
						0.000	-0.060	0.000
	12	1	GI	0.000%	100.000%	0.000	-0.060	0.000
						0.000	-0.060	0.000
	13	1	GI	0.000%	100.000%	0.000	-0.060	0.000
						0.000	-0.060	0.000
	14	1	GI	0.000%	100.000%	0.000	-0.060	0.000
						0.000	-0.060	0.000
	15	1	GI	0.000%	100.000%	0.000	-0.060	0.000
						0.000	-0.060	0.000
	16	1	GI	0.000%	100.000%	0.000	-0.060	0.000
						0.000	-0.060	0.000
	17	1	GI	0.000%	100.000%	0.000	-0.060	0.000
						0.000	-0.060	0.000
	18	1	GI	0.000%	100.000%	0.000	-0.060	0.000
						0.000	-0.060	0.000

MEMBER FORCES AND MOMENTS (kN,kNm)

Load case 1 (Not analysed): Self-Weight, Support (G)

Load case 2 (Not analysed): Soil Load (G)

Load case 3 (Not analysed): Traffic (LL)

Load case 4 (Not analysed): Culvert Self-Weight (G)

Load case 10 (Linear): Combined Loading

Memb	Node	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
1	1	97.750	0.000	0.000	0.000	0.000	0.000
	2	97.298	-33.120	0.000	0.000	0.000	-10.296
3	3	97.750	0.000	0.000	0.000	0.000	0.000
	4	97.298	33.120	0.000	0.000	0.000	10.296
5	21	104.771	0.000	0.000	0.000	0.000	0.000
	9	104.284	0.000	0.000	0.000	0.000	0.000
6	21	133.206	0.000	0.000	0.000	0.000	0.000
	5	133.747	0.000	0.000	0.000	0.000	0.000
7	9	82.222	52.119	0.000	0.000	0.000	-7.347
	13	60.356	-2.668	0.000	0.000	0.000	3.600
8	21	53.510	3.155	0.000	0.000	0.000	-1.290
	14	53.075	2.682	0.000	0.000	0.000	2.175
9	2	110.104	50.409	0.000	0.000	0.000	-11.340
	6	73.559	10.108	0.000	0.000	0.000	1.764
10	6	69.533	26.045	0.000	0.000	0.000	1.764
	7	46.679	-15.195	0.000	0.000	0.000	4.058
11	7	48.878	-4.553	0.000	0.000	0.000	4.058
	10	34.897	-50.565	0.000	0.000	0.000	-7.960
12	10	81.547	37.134	0.000	0.000	0.000	-6.991
	11	77.193	-16.310	0.000	0.000	0.000	-2.184
13	11	50.833	46.207	0.000	0.000	0.000	-4.359
	12	59.464	-10.677	0.000	0.000	0.000	3.600
14	12	60.356	2.668	0.000	0.000	0.000	3.600
	9	82.222	-52.119	0.000	0.000	0.000	-7.347
15	13	59.464	10.677	0.000	0.000	0.000	3.600
	14	50.833	-46.207	0.000	0.000	0.000	-4.359
16	14	77.193	16.310	0.000	0.000	0.000	-2.184
	15	81.547	-37.134	0.000	0.000	0.000	-6.991
17	15	34.897	50.565	0.000	0.000	0.000	-7.960
	16	48.878	4.553	0.000	0.000	0.000	4.058
18	16	46.679	15.195	0.000	0.000	0.000	4.058
	17	69.533	-26.045	0.000	0.000	0.000	1.764
19	17	73.559	-10.108	0.000	0.000	0.000	1.764
	4	110.104	-50.409	0.000	0.000	0.000	-11.340
20	21	53.510	3.155	0.000	0.000	0.000	-1.290
	11	53.075	2.682	0.000	0.000	0.000	2.175
22	10	86.770	0.935	0.000	0.000	0.000	-0.970
	18	87.643	0.935	0.000	0.000	0.000	0.539

23	18	37.881	2.486	0.000	0.000	0.000	-1.265
	2	37.556	2.039	0.000	0.000	0.000	1.044
25	15	86.770	-0.935	0.000	0.000	0.000	0.970
	19	87.643	-0.935	0.000	0.000	0.000	-0.539
26	19	37.881	2.486	0.000	0.000	0.000	-1.265
	4	37.556	2.039	0.000	0.000	0.000	1.044
27	21	38.686	-0.523	0.000	0.000	0.000	0.681
	19	39.227	-1.213	0.000	0.000	0.000	-0.726
28	21	38.686	-0.523	0.000	0.000	0.000	0.681
	18	39.227	-1.213	0.000	0.000	0.000	-0.726

NODE REACTIONS (kN,kNm)

Load case 1 (Not analysed): Self-Weight, Support (G)

Load case 2 (Not analysed): Soil Load (G)

Load case 3 (Not analysed): Traffic (LL)

Load case 4 (Not analysed): Culvert Self-Weight (G)

Load case 10 (Linear): Combined Loading

Node	X-Axis Force	Y-Axis Force	Z-Axis Force	X-Axis Moment	Y-Axis Moment	Z-Axis Moment
1	0.000	97.750	0.000	0.000	0.000	0.000
3	0.000	97.750	0.000	0.000	0.000	0.000
5	0.000	133.747	0.000	0.000	0.000	0.000
18	0.000	137.095	0.000	0.000	0.000	0.000
19	0.000	137.095	0.000	0.000	0.000	0.000
Load	0.000	-603.438	0.000	0.000	0.000	0.000
Reac	0.000	603.438	0.000	0.000	0.000	0.000
Equil	0.000E+00	0.000E+00	1.694E-18			
Resid	1.918E-13	2.203E-13	1.363E-18	2.774E-19	2.875E-19	1.110E-14

BILL OF MATERIALS (m,m^2,T)

Memb	Sect	Qty	Section Name	Unit Length	Total Length	Unit Mass	Total Mass
1	1	2	225x100	0.600	1.200	0.016	0.031
2	2	1	200x100	0.900	0.900	0.021	0.021
3	2	1	200x100	1.000	1.000	0.023	0.023
4	1	12	225x100	0.443	5.313	0.011	0.137
5	2	2	200x100	1.187	2.375	0.027	0.055
6	2	2	200x100	1.613	3.226	0.037	0.074
7	2	2	200x100	1.020	2.041	0.023	0.047
8	2	2	200x100	1.620	3.240	0.037	0.075

Total mass = 0.462
Center of gravity = 2.100,0.957,0.000

3.7.2. FRAME 2 Space Gass Output (2m Load Width)

```
SPACE GASS 12.23 - STUDENT VERSION - NOT FOR COMMERCIAL USE
Path: C:\Users\Michael R...\Desktop\STRUCTURAL SUPPORT SYSTEM (2ND ARCH) 0.5
Designer:  Date: Friday, May 29, 2015 12:41 AM Page: 1

ANALYSIS STATUS REPORT
----------------------

Job name ...... STRUCTURAL SUPPORT SYSTEM (2ND ARCH) 0.5
Location ...... C:\Users\Michael Renko\Desktop

Length units .......................... m
Section property units ................ mm
Material strength units ............... MPa
Mass density units .................... T/m^3
Temperature units ..................... Celsius
Force units ........................... kN
Moment units .......................... kNm
Mass units ............................ T
Acceleration units .................... g's
Translation units ..................... mm
Stress units .......................... MPa

Nodes ..................................    18  (  32765)
Members ................................    21  (  32765)
Plates .................................     0  (  32765)
Restrained nodes .......................     4  (  32765)
Nodes with spring restraints ...........     0  (  32765)
Section properties .....................     4  (   5000)
Material properties ....................     1  (    999)
Constrained nodes ......................     0  (  32765)
Member offsets .........................     0  (  32765)

Node loads .............................     0  ( 250000)
Prescribed node displacements ..........     0  ( 250000)
Member concentrated loads ..............     0  ( 250000)
Member distributed forces ..............    66  ( 250000)
Member distributed torsions ............     0  ( 250000)
Thermal loads ..........................     0  ( 250000)
Member prestress loads .................     0  ( 250000)
Plate pressure loads ...................     0  ( 250000)
Self weight load cases .................     1  (  10000)
Combination load cases .................     1  (  10000)
Load cases with titles .................     5  (  10000)
Lumped masses ..........................     0  ( 250000)
Spectral load cases ....................     0  (  10000)

Static analysis ........................     Y
Dynamic analysis .......................     N
Response analysis ......................     N
Buckling analysis ......................     N
Ill-conditioned ........................     N
Non-linear convergence .................     Y
Frontwidth .............................    30
Total degrees of freedom ...............    98
Static load cases ......................     4  (  10000)
Mass load cases ........................     1  (  10000)

NODE COORDINATES (m)
----------------

                 X          Y          Z
  Node       Coord      Coord      Coord

     1       0.000      0.000      0.000
     2       0.000      0.600      0.000
     3       4.200      0.000      0.000
     4       4.200      0.600      0.000
     5       2.100      0.000      0.000
     6       0.197      0.996      0.000
     7       0.477      1.340      0.000
     9       2.100      1.900      0.000
    10       0.825      1.613      0.000
    11       1.225      1.803      0.000
```

12	1.657	1.900	0.000
13	2.543	1.900	0.000
14	2.975	1.803	0.000
15	3.375	1.613	0.000
16	3.723	1.340	0.000
17	4.003	0.996	0.000
18	0.825	0.000	0.000
21	2.100	1.000	0.000

MEMBER DATA (deg,kNm/rad,m)
----------- (F-Fixed, R-Released) (*-Cable length)

Memb	Dir Angle	Dir Node	Dir Axis	Memb Type	Node A	Node B	Sect	Mat	Node A Fixity	Node B Fixity	Length
1	0.00			Norm	1	2	1	1	FFFFFF	FFFFFF	0.600
3	0.00			Norm	3	4	1	1	FFFFFF	FFFFFF	0.600
5	0.00			Norm	21	9	2	1	FFFFFF	FFFFFF	0.900
6	0.00			Norm	21	5	2	1	FFFFFF	FFFFFF	1.000
7	0.00			Norm	9	13	1	1	FFFFFF	FFFFFF	0.443
8	0.00			Norm	21	14	2	1	FFFFFF	FFFFFF	1.187
9	0.00			Norm	2	6	1	1	FFFFFF	FFFFFF	0.443
10	0.00			Norm	6	7	1	1	FFFFFF	FFFFFF	0.443
11	0.00			Norm	7	10	1	1	FFFFFF	FFFFFF	0.443
12	0.00			Norm	10	11	1	1	FFFFFF	FFFFFF	0.443
13	0.00			Norm	11	12	1	1	FFFFFF	FFFFFF	0.443
14	0.00			Norm	12	9	1	1	FFFFFF	FFFFFF	0.443
15	0.00			Norm	13	14	1	1	FFFFFF	FFFFFF	0.443
16	0.00			Norm	14	15	1	1	FFFFFF	FFFFFF	0.443
17	0.00			Norm	15	16	1	1	FFFFFF	FFFFFF	0.443
18	0.00			Norm	16	17	1	1	FFFFFF	FFFFFF	0.443
19	0.00			Norm	17	4	1	1	FFFFFF	FFFFFF	0.443
20	0.00			Norm	21	11	2	1	FFFFFF	FFFFFF	1.187
22	0.00			Norm	10	18	2	1	FFFFFF	FFFFFF	1.613
23	0.00			Norm	18	2	2	1	FFFFFF	FFFFFF	1.020
28	0.00			Norm	21	18	2	1	FFFFFF	FFFFFF	1.620

NODE RESTRAINTS (kN/m,kNm/rad)
--------------- (F=Fixed, R=Released, S=Spring, *=General)

Node	Rest Code	X Axial Stiffness	Y Axial Stiffness	Z Axial Stiffness	X Rotation Stiffness	Y Rotation Stiffness	Z Rotation Stiffness
1	FFFFRR						
3	RFFRRR						
5	RFFRRR						
18	RFFRRR						

SECTION PROPERTIES (mm,mm^2,mm^4,deg)

Sect	Name	Mark	Shape	Source
1	225x100	S1	Solid rect	AustTimb
2	200x100	S2	Solid rect	AustTimb
3	300x300	S3	Solid rect	AustTimb
4	200x200	S4	Solid square	User

Sect	Area of Section	Torsion Constant	Y-Axis Mom of In	Z-Axis Mom of In	Y-Axis Shr Area	Z-Axis Shr Area	Princ Angle
1	2.2500E+04	5.4068E+07	1.8750E+07	9.4922E+07	Infinite	Infinite	0.00
2	2.0000E+04	4.5775E+07	1.6667E+07	6.6667E+07	Infinite	Infinite	0.00
3	7.5000E+04	7.7484E+08	3.9062E+08	5.6250E+08	Infinite	Infinite	0.00
4	4.0000E+04	2.2496E+08	1.3333E+08	1.3333E+08	Infinite	Infinite	0.00

Sect	Shape	Trans	Mir	Rotate	D	Bt/Bb	Btw/Bbw	Tt/Tb	Tw/Rr
1	Solid rect	No	No	0.00	225.00	100.00	0.00	0.00	0.00
						0.00	0.00	0.00	0.00
2	Solid rect	No	No	0.00	200.00	100.00	0.00	0.00	0.00

						0.00	0.00	0.00	0.00
3	Solid rect	No	No	0.00	300.00	250.00	0.00	0.00	0.00
						0.00	0.00	0.00	0.00
4	Solid square	No	No	0.00	200.00	0.00	0.00	0.00	0.00
						0.00	0.00	0.00	0.00

MATERIAL PROPERTIES (MPa,T/m^3,strain/degC)

Matl	Material Name	Young's Modulus	Poisson's Ratio	Mass Density	Coeff of Expansion	Concrete Strength
1	F34 NonAsh HW U	2.1500E+04	0.37	1.1500E+00	0.000E+00	

MEMBER DISTRIBUTED FORCES (m,kN/m)

Load Case	Memb	Sub Load	Axes Sys	Start Position	Finish Position	X Start/ Finish	Y Start/ Finish	Z Start/ Finish
2	1	1	GI	0.000%	100.000%	23.000	0.000	0.000
						18.000	0.000	0.000
	3	1	GI	0.000%	100.000%	-23.000	0.000	0.000
						-18.000	0.000	0.000
	7	1	GI	0.000%	100.000%	-6.000	-15.000	0.000
						-7.000	-15.000	0.000
	9	1	GI	0.000%	100.000%	18.000	-47.000	0.000
						13.000	-34.000	0.000
	10	1	GI	0.000%	100.000%	13.000	-34.000	0.000
						10.000	-24.000	0.000
	11	1	GI	0.000%	100.000%	10.000	-24.000	0.000
						9.000	-21.000	0.000
	12	1	GI	0.000%	100.000%	9.000	-21.000	0.000
						8.000	-19.000	0.000
	13	1	GI	0.000%	100.000%	8.000	-19.000	0.000
						7.000	-15.000	0.000
	14	1	GI	0.000%	100.000%	7.000	-15.000	0.000
						6.000	-15.000	0.000
	15	1	GI	0.000%	100.000%	-7.000	-15.000	0.000
						-8.000	-19.000	0.000
	16	1	GI	0.000%	100.000%	-8.000	-19.000	0.000
						-9.000	-21.000	0.000
	17	1	GI	0.000%	100.000%	-9.000	-21.000	0.000
						-10.000	-24.000	0.000
	18	1	GI	0.000%	100.000%	-10.000	-24.000	0.000
						-13.000	-34.000	0.000
	19	1	GI	0.000%	100.000%	-13.000	-34.000	0.000
						-18.000	-47.000	0.000
3	1	1	GI	0.000%	100.000%	2.000	0.000	0.000
						2.000	0.000	0.000
	3	1	GI	0.000%	100.000%	-2.000	0.000	0.000
						-2.000	0.000	0.000
	7	1	GI	0.000%	100.000%	-12.000	-29.000	0.000
						-10.500	-29.000	0.000
	9	1	GI	0.000%	100.000%	2.000	-4.000	0.000
						3.000	-7.000	0.000

10	1	GI	0.000%	100.000%	3.000 5.000	-7.000 -12.000	0.000 0.000
11	1	GI	0.000%	100.000%	5.000 6.000	-12.000 -19.000	0.000 0.000
12	1	GI	0.000%	100.000%	6.000 9.000	-19.000 -24.000	0.000 0.000
13	1	GI	0.000%	100.000%	9.000 10.500	-24.000 -29.000	0.000 0.000
14	1	GI	0.000%	100.000%	10.500 12.000	-29.000 -29.000	0.000 0.000
15	1	GI	0.000%	100.000%	-10.500 -9.000	-29.000 -24.000	0.000 0.000
16	1	GI	0.000%	100.000%	-9.000 -6.000	-24.000 -19.000	0.000 0.000
17	1	GI	0.000%	100.000%	-6.000 -5.000	-19.000 -12.000	0.000 0.000
18	1	GI	0.000%	100.000%	-5.000 -3.000	-12.000 -7.000	0.000 0.000
19	1	GI	0.000%	100.000%	-3.000 -2.000	-7.000 -4.000	0.000 0.000
1	1	GI	0.000%	100.000%	0.000 0.000	-0.060 -0.060	0.000 0.000
3	1	GI	0.000%	100.000%	0.000 0.000	-0.060 -0.060	0.000 0.000
7	1	GI	0.000%	100.000%	0.000 0.000	-0.060 -0.060	0.000 0.000
9	1	GI	0.000%	100.000%	0.000 0.000	-0.060 -0.060	0.000 0.000
10	1	GI	0.000%	100.000%	0.000 0.000	-0.060 -0.060	0.000 0.000
11	1	GI	0.000%	100.000%	0.000 0.000	-0.060 -0.060	0.000 0.000
12	1	GI	0.000%	100.000%	0.000 0.000	-0.060 -0.060	0.000 0.000
13	1	GI	0.000%	100.000%	0.000 0.000	-0.060 -0.060	0.000 0.000
14	1	GI	0.000%	100.000%	0.000 0.000	-0.060 -0.060	0.000 0.000
15	1	GI	0.000%	100.000%	0.000 0.000	-0.060 -0.060	0.000 0.000
16	1	GI	0.000%	100.000%	0.000 0.000	-0.060 -0.060	0.000 0.000
17	1	GI	0.000%	100.000%	0.000 0.000	-0.060 -0.060	0.000 0.000
18	1	GI	0.000%	100.000%	0.000 0.000	-0.060 -0.060	0.000 0.000
19	1	GI	0.000%	100.000%	0.000 0.000	-0.060 -0.060	0.000 0.000

SELF WEIGHT (g's)

Load Case	X-Axis Accel'n	Y-Axis Accel'n	Z-Axis Accel'n
1	0.000	-1.000	0.000

COMBINATION LOAD CASES

Load case 10: Combined Loading

2.400 * Load case 1: Self-Weight, Support (G)
2.400 * Load case 2: Soil Load (G)
3.000 * Load case 3: Traffic (LL)
2.400 * Load case 4: Culvert Self-Weight (G)

LOAD CASE TITLES

Load Case	Title
1	Self-Weight, Support (G)
2	Soil Load (G)
3	Traffic (LL)
4	Culvert Self-Weight (G)
10	Combined Loading 1.2G + 1.5Q

NODE DISPLACEMENTS (mm,rad)

Load case 1 (Not analysed): Self-Weight, Support (G)

Load case 2 (Not analysed): Soil Load (G)

Load case 3 (Not analysed): Traffic (LL)

Load case 4 (Not analysed): Culvert Self-Weight (G)

Load case 10 (Linear): Combined Loading

Node	X-Axis Transl'n	Y-Axis Transl'n	Z-Axis Transl'n	X-Axis Rotation	Y-Axis Rotation	Z-Axis Rotation
1	0.000	0.000	0.000	0.000	0.000	0.001
2	-0.333	-0.097	0.000	0.000	0.000	0.000
3	1.256	0.000	0.000	0.000	0.000	0.001
4	0.761	-0.213	0.000	0.000	0.000	0.002
5	-0.477	0.000	0.000	0.000	0.000	0.000
6	-0.034	-0.326	0.000	0.000	0.000	-0.001
7	0.083	-0.472	0.000	0.000	0.000	0.000
9	-0.102	-0.668	0.000	0.000	0.000	0.000
10	-0.054	-0.339	0.000	0.000	0.000	0.000
11	-0.068	-0.454	0.000	0.000	0.000	0.000
12	-0.053	-0.677	0.000	0.000	0.000	0.000
13	-0.151	-0.872	0.000	0.000	0.000	0.000
14	-0.219	-1.012	0.000	0.000	0.000	-0.001
15	-0.490	-1.388	0.000	0.000	0.000	-0.001
16	-0.549	-1.321	0.000	0.000	0.000	0.001
17	-0.023	-0.756	0.000	0.000	0.000	0.002
18	-0.409	0.000	0.000	0.000	0.000	0.000
21	-0.301	-0.452	0.000	0.000	0.000	0.000

MEMBER FORCES AND MOMENTS (kN,kNm)

Load case 1 (Not analysed): Self-Weight, Support (G)

Load case 2 (Not analysed): Soil Load (G)

Load case 3 (Not analysed): Traffic (LL)

Load case 4 (Not analysed): Culvert Self-Weight (G)

Load case 10 (Linear): Combined Loading

Memb	Node	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
1	1	78.669	0.000	0.000	0.000	0.000	0.000
	2	78.217	-33.120	0.000	0.000	0.000	-10.296
3	3	172.047	0.000	0.000	0.000	0.000	0.000
	4	171.595	33.120	0.000	0.000	0.000	10.296
5	21	103.392	0.723	0.000	0.000	0.000	-0.358
	9	102.904	0.723	0.000	0.000	0.000	0.292
6	21	193.993	0.000	0.000	0.000	0.000	0.000
	5	194.534	0.000	0.000	0.000	0.000	0.000
7	9	64.360	49.016	0.000	0.000	0.000	-7.649
	13	42.495	-5.771	0.000	0.000	0.000	1.924
8	21	115.438	-0.321	0.000	0.000	0.000	-0.353
	14	115.003	-0.795	0.000	0.000	0.000	-1.016
9	2	96.210	53.787	0.000	0.000	0.000	-12.115
	6	59.665	13.486	0.000	0.000	0.000	2.484
10	6	55.236	26.283	0.000	0.000	0.000	2.484
	7	32.382	-14.957	0.000	0.000	0.000	4.884
11	7	34.879	-7.466	0.000	0.000	0.000	4.884
	10	20.898	-53.478	0.000	0.000	0.000	-8.425
12	10	69.279	34.321	0.000	0.000	0.000	-6.857
	11	64.925	-19.122	0.000	0.000	0.000	-3.295
13	11	33.734	48.287	0.000	0.000	0.000	-5.107
	12	42.365	-8.597	0.000	0.000	0.000	3.774
14	12	43.219	0.934	0.000	0.000	0.000	3.774
	9	65.084	-53.852	0.000	0.000	0.000	-7.941
15	13	42.723	3.721	0.000	0.000	0.000	1.924
	14	34.092	-53.164	0.000	0.000	0.000	-9.115
16	14	88.862	61.934	0.000	0.000	0.000	-10.131
	15	93.217	8.490	0.000	0.000	0.000	5.263
17	15	89.064	28.791	0.000	0.000	0.000	5.263
	16	103.045	-17.220	0.000	0.000	0.000	7.641
18	16	104.309	5.873	0.000	0.000	0.000	7.641
	17	127.163	-35.367	0.000	0.000	0.000	1.220
19	17	131.828	-6.522	0.000	0.000	0.000	1.220
	4	168.373	-46.823	0.000	0.000	0.000	-10.296
20	21	62.969	2.720	0.000	0.000	0.000	-1.137
	11	62.534	2.246	0.000	0.000	0.000	1.811
22	10	89.900	1.595	0.000	0.000	0.000	-1.568
	18	90.773	1.595	0.000	0.000	0.000	1.005
23	18	50.127	3.658	0.000	0.000	0.000	-1.685
	2	49.802	3.211	0.000	0.000	0.000	1.819
28	21	51.092	-0.338	0.000	0.000	0.000	0.426
	18	51.634	-1.028	0.000	0.000	0.000	-0.681

```
NODE REACTIONS (kN,kNm)
--------------

Load case 1 (Not analysed): Self-Weight, Support (G)

Load case 2 (Not analysed): Soil Load (G)

Load case 3 (Not analysed): Traffic (LL)

Load case 4 (Not analysed): Culvert Self-Weight (G)

Load case 10 (Linear): Combined Loading

        X-Axis     Y-Axis     Z-Axis     X-Axis     Y-Axis     Z-Axis
 Node    Force      Force      Force     Moment     Moment     Moment

    1    0.000     78.669      0.000      0.000      0.000      0.000
    3    0.000    172.047      0.000      0.000      0.000      0.000
    5    0.000    194.534      0.000      0.000      0.000      0.000
   18    0.000    155.886      0.000      0.000      0.000      0.000

 Load    0.000   -601.135      0.000      0.000      0.000      0.000
 Reac    0.000    601.135      0.000      0.000      0.000      0.000

Equil  4.547E-13  0.000E+00  5.898E-18
Resid  2.061E-13  2.984E-13  2.430E-18  1.573E-19  5.278E-19  2.798E-14

BILL OF MATERIALS (m,m^2,T)
-----------------

                                          Unit       Total        Unit       Total
 Memb  Sect   Qty  Section Name          Length      Length       Mass        Mass

    1     1     2  225x100                0.600       1.200       0.016       0.031
    2     2     1  200x100                0.900       0.900       0.021       0.021
    3     2     1  200x100                1.000       1.000       0.023       0.023
    4     1    12  225x100                0.443       5.313       0.011       0.137
    5     2     2  200x100                1.187       2.375       0.027       0.055
    6     2     1  200x100                1.613       1.613       0.037       0.037
    7     2     1  200x100                1.020       1.020       0.023       0.023
    8     2     1  200x100                1.620       1.620       0.037       0.037

Total mass        = 0.365
Center of gravity = 1.797,1.061,0.000
```

12	1.657	1.900	0.000
13	2.543	1.900	0.000
14	2.975	1.803	0.000
15	3.375	1.613	0.000
16	3.723	1.340	0.000
17	4.003	0.996	0.000
18	0.825	0.000	0.000
19	3.375	0.000	0.000
21	2.100	1.000	0.000

MEMBER DATA (deg,kNm/rad,m)
----------- (F=Fixed, R=Released) (*=Cable length)

Memb	Dir Angle	Dir Node	Dir Axis	Memb Type	Node A	Node B	Sect	Mat	Node A Fixity	Node B Fixity	Length
1	0.00			Norm	1	2	1	1	FFFFFF	FFFFFF	0.600
3	0.00			Norm	3	4	1	1	FFFFFF	FFFFFF	0.600
5	0.00			Norm	21	9	2	1	FFFFFF	FFFFFF	0.900
6	0.00			Norm	21	5	2	1	FFFFFF	FFFFFF	1.000
7	0.00			Norm	9	13	1	1	FFFFFF	FFFFFF	0.443
8	0.00			Norm	21	14	2	1	FFFFFF	FFFFFF	1.187
9	0.00			Norm	2	6	1	1	FFFFFF	FFFFFF	0.443
10	0.00			Norm	6	7	1	1	FFFFFF	FFFFFF	0.443
11	0.00			Norm	7	10	1	1	FFFFFF	FFFFFF	0.443
12	0.00			Norm	10	11	1	1	FFFFFF	FFFFFF	0.443
13	0.00			Norm	11	12	1	1	FFFFFF	FFFFFF	0.443
14	0.00			Norm	12	9	1	1	FFFFFF	FFFFFF	0.443
15	0.00			Norm	13	14	1	1	FFFFFF	FFFFFF	0.443
16	0.00			Norm	14	15	1	1	FFFFFF	FFFFFF	0.443
17	0.00			Norm	15	16	1	1	FFFFFF	FFFFFF	0.443
18	0.00			Norm	16	17	1	1	FFFFFF	FFFFFF	0.443
19	0.00			Norm	17	4	1	1	FFFFFF	FFFFFF	0.443
20	0.00			Norm	21	11	2	1	FFFFFF	FFFFFF	1.187
22	0.00			Norm	10	18	2	1	FFFFFF	FFFFFF	1.613
23	0.00			Norm	18	2	2	1	FFFFFF	FFFFFF	1.020
25	0.00			Norm	15	19	2	1	FFFFFF	FFFFFF	1.613
26	0.00			Norm	19	4	2	1	FFFFFF	FFFFFF	1.020
27	0.00			Norm	21	19	2	1	FFFFFF	FFFFFF	1.620
28	0.00			Norm	21	18	2	1	FFFFFF	FFFFFF	1.620

NODE RESTRAINTS (kN/m,kNm/rad)
--------------- (F=Fixed, R=Released, S=Spring, *=General)

Node	Rest Code	X Axial Stiffness	Y Axial Stiffness	Z Axial Stiffness	X Rotation Stiffness	Y Rotation Stiffness	Z Rotation Stiffness
1	FFFFRR						
3	RFFRRR						
5	RFFRRR						
18	RFFRRR						
19	RFFRRR						

SECTION PROPERTIES (mm,mm^2,mm^4,deg)

Sect	Name	Mark	Shape	Source
1	225x100	S1	Solid rect	AustTimb
2	200x100	S2	Solid rect	AustTimb
3	300x300	S3	Solid rect	AustTimb
4	200x200	S4	Solid square	User

Sect	Area of Section	Torsion Constant	Y-Axis Mom of In	Z-Axis Mom of In	Y-Axis Shr Area	Z-Axis Shr Area	Princ Angle
1	2.2500E+04	5.4068E+07	1.8750E+07	9.4922E+07	Infinite	Infinite	0.00
2	2.0000E+04	4.5775E+07	1.6667E+07	6.6667E+07	Infinite	Infinite	0.00
3	7.5000E+04	7.7484E+08	3.9062E+08	5.6250E+08	Infinite	Infinite	0.00
4	4.0000E+04	2.2496E+08	1.3333E+08	1.3333E+08	Infinite	Infinite	0.00

4. Appendix 4

4.1. DPTI Data

4.1.1. Botanic Road/Hackney Road

4.1.1.1. Turning Movement Survey

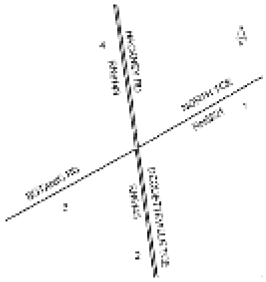
MICHELMO
TV0670 - v10.06

Department of Planning, Transport and Infrastructure
Vehicle Turning Movement Survey

Page 1 of 1
09/04/2015 10:06

Intersection of: HACKNEY ROAD / NORTH TERRACE / DEQUETTEVILLE TERRACE / BOTANIC ROAD
Locality: KENT TOWN
AMG Reference: TG020331
Date of Count: 01/04/2015 Day: Wednesday
Weather: Dry Control: SIGNALS
Survey Status:

Arm	Road Number - Name
1	5221 - NORTH TERRACE
2	6143 DEQUETTEVILLE TERRACE
3	6155 BOTANIC ROAD
4	6143 HACKNEY ROAD

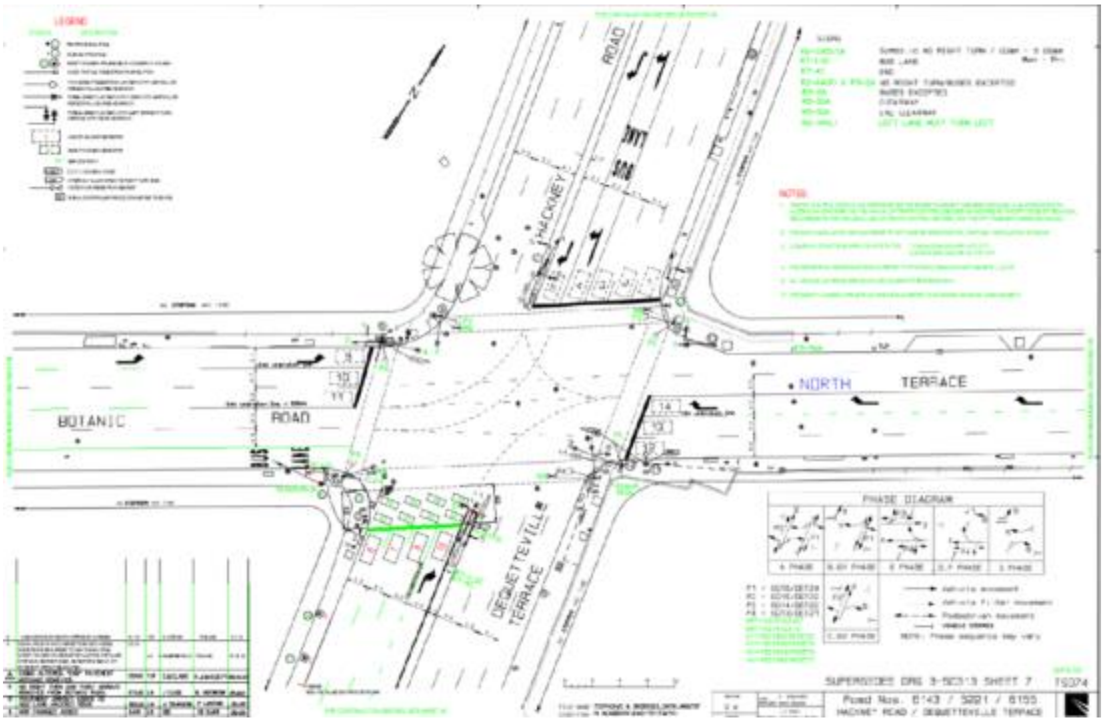


	Arm	1			2			3			4		
	Exit Arm	2 (L)	3	4 (R)	3 (L)	4	1 (R)	4 (L)	1	2 (R)	1 (L)	2	3 (R)
11 hour totals	Cars	652	9684	2668	621	10307	580	2760	7044	3	2751	12568	2684
	CV	23	482	87	31	327	8	485	473	7	84	351	451
	Total	675	10166	2755	652	10634	588	3245	7517	10	2835	12919	3135
AM Peak hour (08:00)	Cars	65	1484	282	67	975	0	168	579	0	196	1850	365
	CV	2	59	4	2	24	0	66	55	1	13	46	85
	Total	67	1543	286	69	999	0	234	634	1	209	1896	450
PM Peak hour (16:45)	Cars	60	720	242	28	1180	58	450	1069	0	339	1171	217
	CV	2	46	5	2	23	0	80	43	0	1	22	51
	Total	62	766	247	30	1203	58	530	1112	0	340	1193	268

		1				2				3				4			
One-way Flows	11 Hour Totals	(IN) 13596		(OUT) 10940		(IN) 11874		(OUT) 13804		(IN) 10772		(OUT) 13953		(IN) 18889		(OUT) 16834	
	AM Peak Hour	08:00	1896	11:45	908	09:00	1129	08:00	1964	11:45	876	08:00	2062	08:00	2556	08:00	1619
	PM Peak Hour	12:15	1173	16:45	1610	16:45	1399	16:45	1379	16:45	1642	14:15	1157	15:45	1934	17:15	1984
Two way Flows	AM Peak Hour	08:00	2739			08:00	3032			08:00	2931			08:00	4074		
	PM Peak Hour	16:45	2586			16:45	2778			16:45	2706			15:45	3806		
All Vehicles	11 Hour Totals	24536	4.7% CV			25478	2.8% CV			24725	7.8% CV			35523	5.0% CV		
	Estimated AADT	31700	SF(1.00) ZF(1.29)			32900	SF(1.00) ZF(1.29)			31900	SF(1.00) ZF(1.29)			45800	SF(1.00) ZF(1.29)		

AADT Annual Average Daily Traffic SF Seasonal Factor ZF Zone Factor CV Commercial Vehicles

4.1.1.2. Intersection Drawing



4.1.1.3. SCATS Summary

TS074 – Botanic Rd / Dequetteville Tce / Hackney Rd / North Tce

PHASING OPERATION:

- Leading Trailing Turn on Hackney Rd and Dequetteville Tce (except AM peak where it runs a leading turn) and Leading Turn from North Tce (east approach)
- Phasing runs A, D, E, G1 in AM peak and A, C, D, E, B at all other times
- Bus turning right from Hackney Rd extends A phase, and calls and extends G1 phase in AM & calls B phase at all other times

TURNING MOVEMENT OPERATION:

- NRT from Dequetteville Tce. M-F 7am-9am – fully controlled at all other times
- RT from Hackney Rd filters M-F 7am-9am
- RT from North Tce into Hackney Rd filters 7-9 am & 4-6pm M-F and late night off peaks
- No right turns from Botanic Rd into Dequetteville Tce. except for buses.

PHASE PERCENTAGE DURING PEAK PERIODS:

- A phase (Hackney Rd / Dequetteville Tce.) is the stretch phase except AM peak
- In the AM peak E phase (Botanic Rd/ North Tce.) is the stretch phase
- Average phase time between May 6th – May 8th 2014 is:

Period	Time	Ave CL	A	B	C	D	E	G1
AM	0745 – 0845	120s	35s	-	-	28s	37s	20s
BUS	1400 - 1500	119s	25s	22s	12s	26s	34s	-
PM	1645 – 1745	120s	27s	16s	12s	18s	46s	-

LINKING:

- TS074 is linked to TS075 (Payneham Rd, Magill Rd, North Tce)
- TS072 (Dequetteville Tce / Rundle Rd) is linked to TS074

INTERGREEN TIME:

- Phase A, B, C & G have 6.5 seconds of intergreen time (Yellow = 4.0s, Red = 2.5s)
- Phase D, E & F have 7.0 seconds of intergreen time (Yellow = 4.0s, Red = 3.0s)

PHASE SKIPPING

- None

CYCLE TIME:

- Maximum cycle time is 120s

WALKING TIME

Pedestrian	Parallel Vehicle Phase	Time Required
P1	A, B, G1	23s
P2	A, C	21s
P3	E	32s
P4	D, E, F	33s

SITE GRAPHICS

TABLE: SCATS Maximum Flow recorded on May 8th, 2014:

Detector No	Maximum Flow
1	1500
2	2000
3	2150
4	1350
5	2250
6	1900
7	2100
8	2150
9	1700
10	1850
11	2000
12	1800
13	1800
14	2250
15	2050

Note: SCATS Maximum Flow is just an indication of the lane Saturation Flow which may vary during time of day and not necessarily same as traditional Saturation Flow (as per definition) used in Modelling Packages

4.1.2. Dequetteville Terrace/Flinders Street

4.1.2.1. Turning Movement Survey

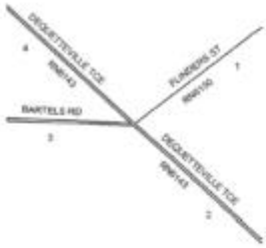
MICHELMO
TV0670 - v10.04

Department for Transport, Energy and Infrastructure
Vehicle Turning Movement Survey

Page 1 of 1
06/07/2011 10:49

Intersection of: DEQUETTEVILLE TERRACE / BARTELS ROAD / FLINDERS STREET
Locality: KENT TOWN
AMG Reference: TG824325
Date of Count: 28/06/2011 Day: Tuesday
Weather: Dry Control: SIGNALS
Survey Status:

Arm	Road Number - Name
1	6150 - FLINDERS STREET
2	6143 - DEQUETTEVILLE TERRACE
3	BARTELS ROAD
4	6143 - DEQUETTEVILLE TERRACE

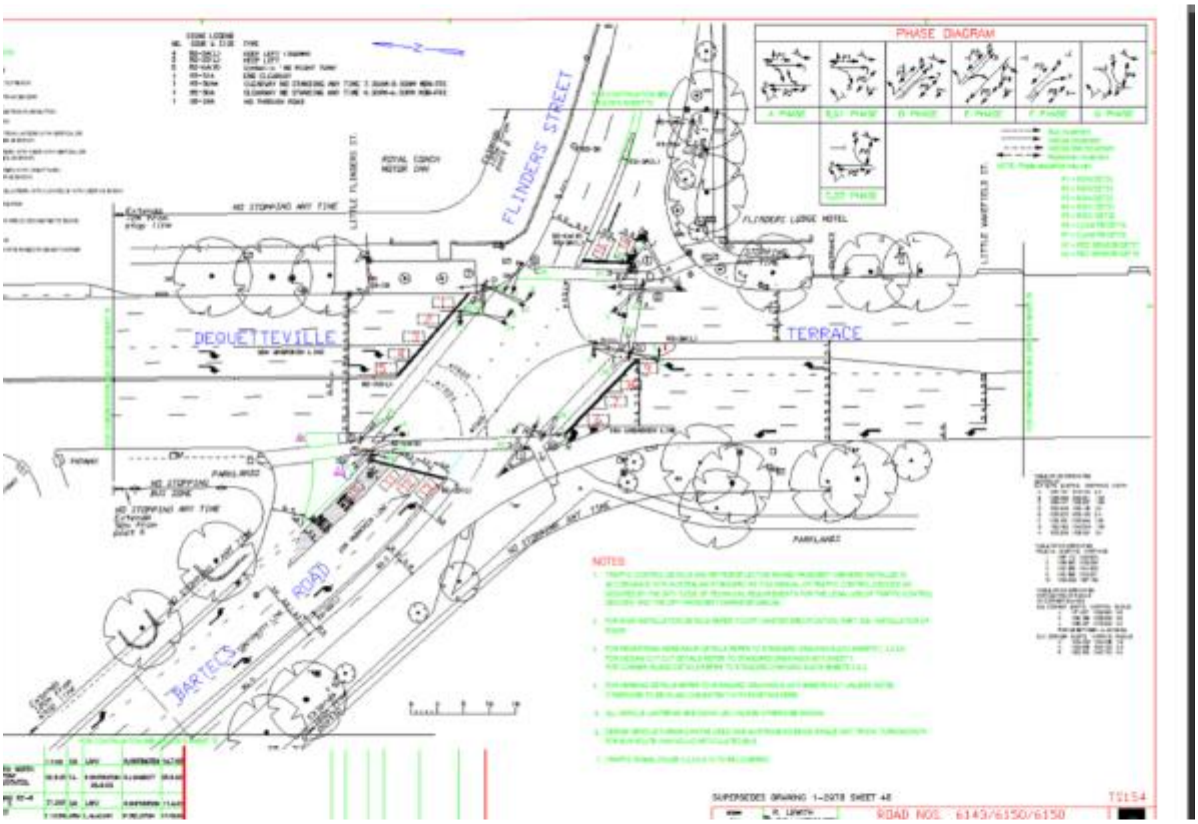


	Arm	1			2			3			4		
	Exit Arm	2 (L)	3	4 (R)	3 (L)	4	1 (R)	4 (L)	1	2 (R)	1 (L)	2	3 (R)
11 hour totals	Cars	149	4945	6	1140	7783	268	2477	6028	1247	313	6806	2642
	CV	12	141	0	9	326	5	26	148	16	7	350	25
	Total	161	5086	6	1149	8109	273	2503	6176	1263	320	7156	2667
AM Peak hour (08:00)	Cars	11	955	2	149	954	28	250	429	106	54	810	431
	CV	0	16	0	0	29	0	4	12	1	2	27	0
	Total	11	971	2	149	983	28	254	441	107	56	837	431
PM Peak hour (17:00)	Cars	16	368	0	124	826	34	373	1028	209	27	740	299
	CV	0	10	0	2	18	0	1	12	0	0	23	2
	Total	16	378	0	126	844	34	374	1040	209	27	763	301

		1	2	3	4
One-way Flows	11 Hour Totals	(IN) 5253 (OUT) 6769	(IN) 9531 (OUT) 8580	(IN) 9942 (OUT) 8902	(IN) 10143 (OUT) 10618
	AM Peak Hour	07:45 1018 11:45 539	07:45 1189 08:00 955	08:15 823 07:45 1597	08:00 1324 08:15 1258
	PM Peak Hour	16:30 420 16:45 1112	14:45 1006 17:00 988	17:00 1623 17:00 805	15:00 1125 16:45 1228
Two-way Flows	AM Peak Hour	08:00 1509	07:45 2120	08:00 2353	07:45 2570
	PM Peak Hour	16:45 1523	17:00 1992	17:00 2428	17:00 2309
All Vehicles	11 Hour Totals	12022 2.6% CV	18111 4.0% CV	18844 1.9% CV	20761 3.5% CV
	Estimated AADT	15700 SF(1.00) ZF(1.31)	23700 SF(1.00) ZF(1.31)	24700 SF(1.00) ZF(1.31)	27200 SF(1.00) ZF(1.31)

AADT - Annual Average Daily Traffic SF - Seasonal Factor ZF - Zone Factor CV - Commercial Vehicles

4.1.2.2. Intersection Drawing



4.1.2.3. SCATS Summary

TS154 – Bartels Rd/ Dequetteville Tce/ Flinders St

SITE DETAILS:

- Running phase sequence B, A, C, D, E (F and G not used)
- Bus (SG8) runs in late start of D or E phase.

RIGHT TURN MOVEMENT OPERATION:

- Bartels Road west approach filters at all times
- Dequetteville Tce south approach filters during standard after hours times.
- Dequetteville Tce north approach does not filter.

PHASE PERCENTAGE DURING PEAK PERIODS:

- A phase (Dequetteville Tce) is the stretch phase
- Average phase time between May 20th – May 22nd, 2014

Period	Time	Ave CL	A	B	C	D	E
AM	0800 - 0900	120s	30s	23s	13s	14s*	40s
BUS	1400 - 1500	102s	31s	17s	15s	16s	23s
PM	1645 - 1745	120s	29s	20s	19s	21s	30s

*D phase skipped every 3rd cycle during AM peak, when D is skipped; C will get D phase time till it gapped out, E will get part of D phase time. Minimum D phase time is **14s**

Bus runs in late start of D/E phase, bus clearance time is 2s before SG3 runs; bus phase minimum green time is 5s, maximum green time is 10s. Bus phase activated **4 times during AM peak; **3** times during BUS peak, and **2** times during PM peak

LINKING:

- TS154 is linked to TS072 (Rundle St / Rundle Rd / Dequetteville Tce).
- In business hours it is not linked to TS072 to allow lower CL
- TS073 (Fullarton Rd / The Parade / Flinders St) is linked to TS154.

INTERGREEN TIME:

- A, B & C Phase has 7 seconds of intergreen time (Yellow = 4s, Red = 3s)

- D & E phase has 8 seconds of intergreen time (Yellow = 4s, Red = 4s)

PHASE SKIPPING:

- D phase skipped every 3rd cycle during AM 0730-0900 Mon-Fri.

CYCLE TIME:

- Maximum cycle time is 120 seconds

PEDESTRIAN TIMES:

Pedestrian	Parallel Vehicle Phase	Time Required
P1	A, B	22s
P2	A, C	27s
P3	D, E	35s
P4	C, E	14s
P5	B, D, E	22s

SITE GRAPHIC

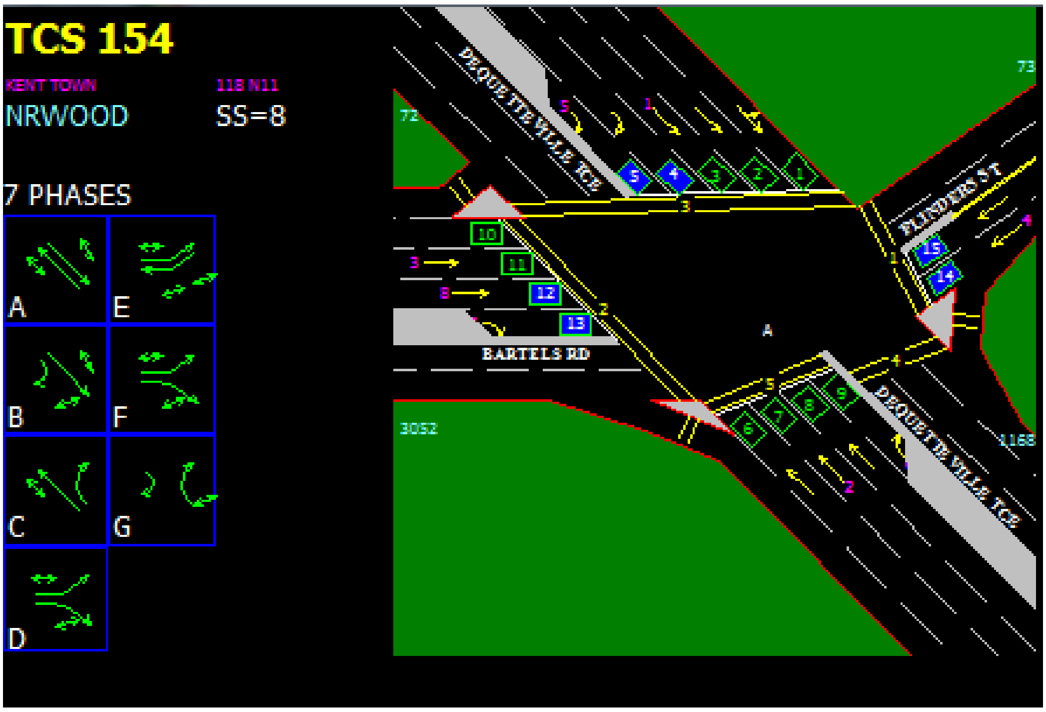


TABLE: SCATS Maximum Flow recorded on 08/05/2014

Detector No	Maximum Flow
1	1850
2	1900

Note: SCATS Maximum Flow is just an indication of the lane Saturation Flow which may vary during time of day and not necessarily same as traditional Saturation Flow (as per definition) used in Modelling Packages.

3	1850
4	2050
5	1850
6	1750
7	1650
8	1600
9	1650
10	2350
11	1550
12	1700
13	1650
14	1750
15	1800

4.1.3. Dequetteville Terrace/Rundle Street

4.1.3.1. Turning Movement Survey

MICHELMO
TV0670 - 10.03

Department of Planning, Transport and Infrastructure
Vehicle Turning Movement Survey

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10/04/2015 15:40

Intersection of: DEQUETTEVILLE TERRACE / RUNDLE ROAD / RUNDLE STREET
Locality: KENT TOWN
AMG Reference: TG821328
Date of Count: 01/04/2015 Day: Wednesday
Weather: Dry Control: SIGNALS
Survey Status:

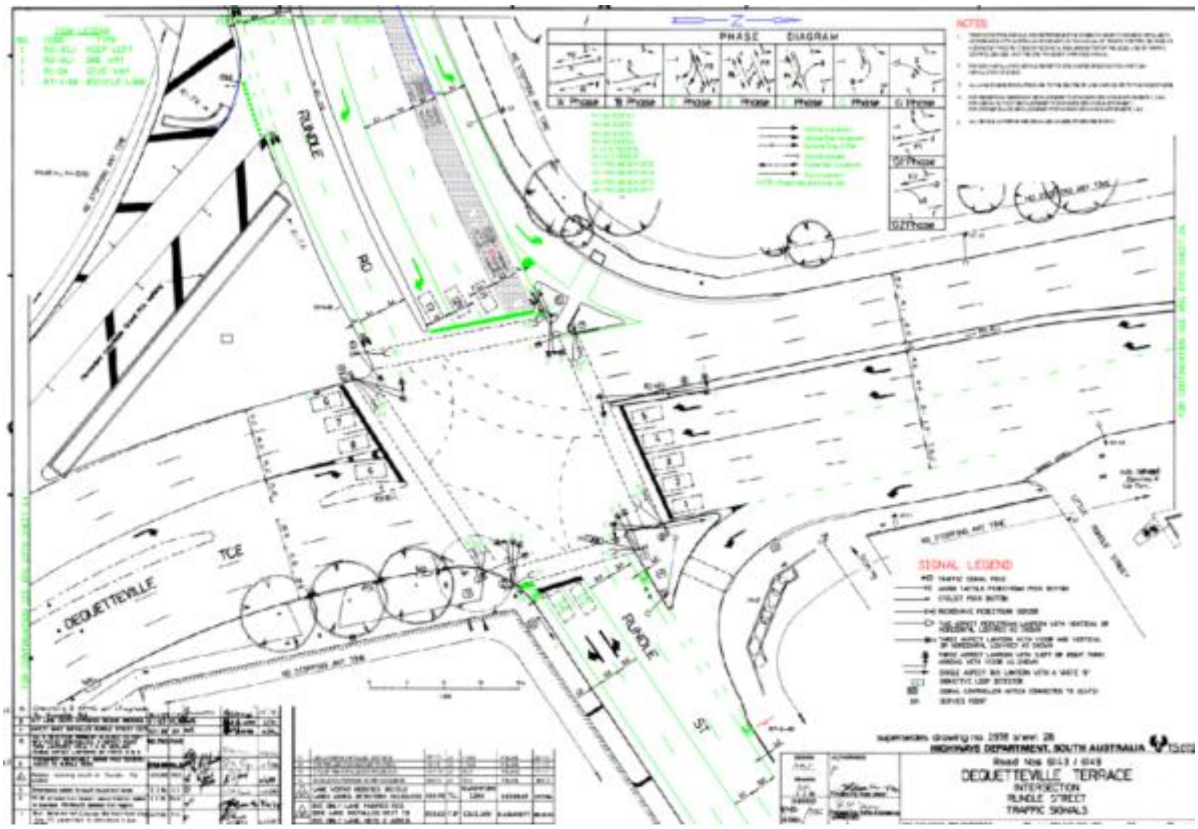
Arm	Road Number - Name
1	6149 - RUNDLE STREET
2	6143 - DEQUETTEVILLE TERRACE
3	6149 - RUNDLE ROAD
4	6143 - DEQUETTEVILLE TERRACE

	Arm	1			2			3			4		
	Exit Arm	2 (L)	3	4 (R)	3 (L)	4	1 (R)	4 (L)	1	2 (R)	1 (L)	2	3 (R)
11 hour totals	Cars	585	2686	1939	1222	8616	726	1531	2246	639	1810	9274	2176
	CV	15	56	23	93	271	6	71	56	84	39	286	107
	Total	600	2742	1962	1315	8887	732	1602	2302	723	1849	9560	2283
AM Peak hour (08:00)	Cars	58	535	254	162	730	96	73	170	46	260	1237	449
	CV	1	9	7	13	10	2	8	4	7	6	33	14
	Total	59	544	261	175	740	98	81	174	53	266	1270	463
PM Peak hour (15:45)	Cars	77	220	215	88	1011	101	239	280	85	164	1014	170
	CV	0	2	2	9	13	0	5	6	9	1	18	16
	Total	77	222	217	97	1024	101	244	286	94	165	1032	186

		1	2	3	4
One-way Flows	11 Hour Totals	(IN) 5304 (OUT) 4883	(IN) 10934 (OUT) 10883	(IN) 4627 (OUT) 6340	(IN) 13692 (OUT) 12451
	AM Peak Hour	08:00 864 08:00 538	07:15 1148 08:00 1382	11:45 337 07:45 1187	08:00 1999 07:15 1160
	PM Peak Hour	16:30 586 17:00 660	15:30 1262 15:45 1203	17:00 813 17:00 641	15:45 1383 15:45 1485
Two-way Flows	AM Peak Hour	08:00 1402	07:45 2452	08:00 1490	08:00 3081
	PM Peak Hour	17:00 1188	15:30 2447	17:00 1454	15:45 2868
All Vehicles	11 Hour Totals	10187 1.9% CV	21817 3.5% CV	10967 4.3% CV	26143 3.0% CV
	Estimated AADT	13100 SF(1.00) ZF(1.29)	28100 SF(1.00) ZF(1.29)	14100 SF(1.00) ZF(1.29)	33700 SF(1.00) ZF(1.29)

AADT - Annual Average Daily Traffic SF - Seasonal Factor ZF - Zone Factor CV - Commercial Vehicl

4.1.3.2. Intersection Drawing



4.1.3.3. SCATS Summary

TS072 – Rundle St / Dequetteville Tce/ Rundle Rd

SITE OPERATION:

- Leading Trailing Turns for Rundle St and Dequetteville Tce.
- Running phase sequence: A, B, D, E, F, C, G2; 'C' is Bus phase
- Bus can run in C and late start of D, F phase

TURNING MOVEMENT OPERATION:

- Rundle St (east) RT filters full time.
- All other RTs are fully controlled

LINKING:

- TS072 is linked to TS074 (Botanic Rd / Dequetteville Tce / Hackney Rd).
- TS154 (Bartels Rd / Dequetteville Tce / Flinders St) is linked to TS072.

PHASE PERCENTAGE DURING PEAK PERIODS:

- B phase is stretch phase during AM peak
- A phase is normally stretch phase at all other times
- Average phase times between May 6th - May 8th 2014:

Period	Time	Ave CL	A	B	*C	D	E	*F	G2
AM	0815 - 0915	120s	26s	24s	-	13s	23s	16s	17s
BUS	1400 - 1500	119s	26s	25s	-	17s	20s	16s	15s
PM	1630 - 1730	120s	35s	19s	-	17s	19s	16s	14s

Note: AM Peak – C ran __3__ times and F ran __16__ times out of 28 cycles; PM - C ran __3__ times out of 29 cycles

Bus phase runs the maximum green time of 5s, and clearance time of 3s. Bus SG is activated **6 times during AM peak, **6** times during BUS peak, and **7** times during PM peak*

INTERGREEN TIME

- A phase has 6.5 seconds of intergreen time (yellow = 4s, red = 2.5s).
- B, C, D, E, F and G2 phase has 7 secs of intergreen time (yellow = 4s, red = 3s).

PHASE SKIPPING

- None

CYCLE TIME

- Maximum cycle time for TS072 is 120 seconds

WALKING TIME:

Pedestrian	Parallel Vehicle Phase	Time Required
P1	A, B	20s
P2	A, G2	26s
P3	D, E	29s
P4	E, F	32s

SCATS GRAPHICS:

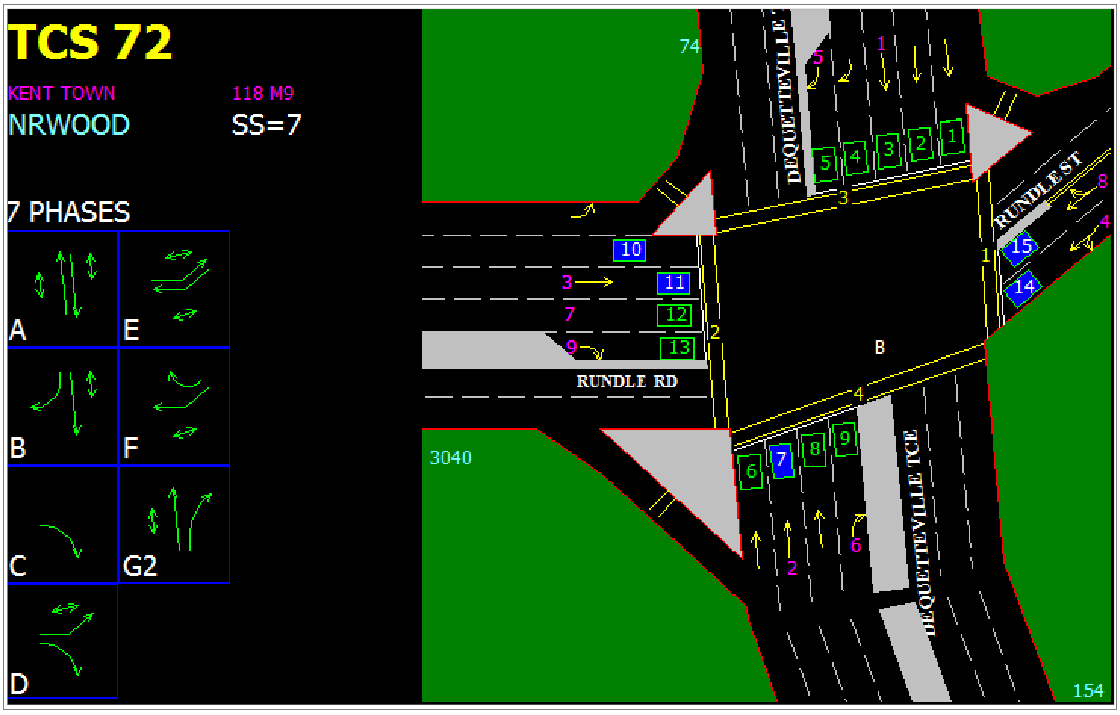


TABLE: SCATS Maximum Flow recorded on 8th May, 2014

Detector No	Maximum Flow
1	1800
2	1650
3	1950

Note: SCATS Maximum Flow is just an indication of the lane Saturation Flow which may vary during time of day and not

4	1600	**necessarily same as traditional Saturation Flow (as per definition) used in Modelling Packages.**
5	1500	
6	1950	
7	2050	
8	2100	
9	1800	
10	1700	
11	1800	
12	1800	
13	1500	
14	1600	
15	1700	

4.1.4. East Terrace/Botanic Road

4.1.4.1. Turning Movement Survey

MICHELMO
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Department of Planning, Transport and Infrastructure
Vehicle Turning Movement Survey

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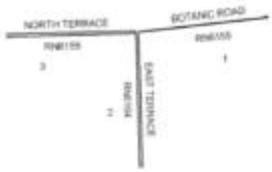
10/04/2015 15:36

Intersection of: BOTANIC ROAD / NORTH TERRACE / EAST TERRACE
Locality: ADELAIDE
AMG Reference: TG816329
Date of Count: 01/04/2015 Day: Wednesday
Weather: Dry Control: SIGNALS
Survey Status:

Arm	Road Number - Name
1	6155 - BOTANIC ROAD
2	6164 - EAST TERRACE
3	6155 - NORTH TERRACE

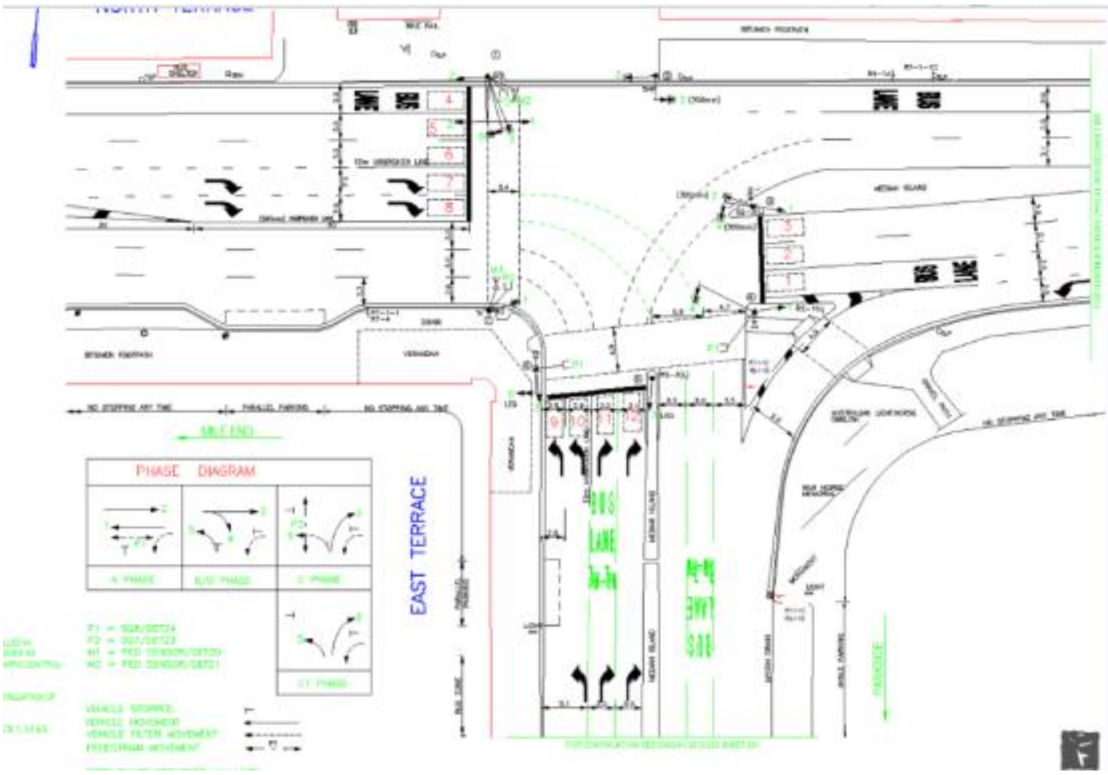


	Arm	1		2		3	
	Exit Arm	2 (L)	3	3 (L)	1 (R)	1	2 (R)
11 hour totals	Cars	4106	8883	2223	1935	7872	2317
	CV	200	764	373	223	742	216
	Total	4306	9647	2596	2158	8614	2533
AM Peak hour (08:00)	Cars	664	1252	215	164	583	174
	CV	33	113	42	45	77	21
	Total	697	1365	257	209	660	195
PM Peak hour (16:45)	Cars	338	627	222	277	1242	274
	CV	38	61	39	29	94	23
	Total	376	688	261	306	1336	297

		1	2	3
One-way Flows	11 Hour Totals	(IN) 13953 (OUT) 10772	(IN) 4754 (OUT) 6839	(IN) 11147 (OUT) 12243
	AM Peak Hour	08:00 2062 11:45 876	08:15 489 08:00 892	11:45 995 08:00 1622
	PM Peak Hour	14:15 1157 16:45 1642	17:00 573 17:00 683	16:45 1633 16:30 1059
Two-way Flows	AM Peak Hour	08:00 2931	08:00 1358	08:00 2477
	PM Peak Hour	16:45 2706	17:00 1256	16:45 2582
All Vehicles	11 Hour Totals	24725 7.8% CV	11593 8.7% CV	23390 9.0% CV
	Estimated AADT	31900 SF(1.00) ZF(1.29)	15000 SF(1.00) ZF(1.29)	30200 SF(1.00) ZF(1.29)

AADT - Annual Average Daily Traffic SF - Seasonal Factor ZF - Zone Factor CV - Commercial Vehicl

4.1.4.2. Intersection Drawing



4.1.5. Fullarton Road/The Parade

4.1.5.1. Turning Movement Survey

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TV0670 - 10.03

Department of Planning, Transport and Infrastructure
Vehicle Turning Movement Survey

Page 1 of 1
04/06/2014 15:25

Intersection of: FULLARTON ROAD / THE PARADE / FLINDERS STREET / THE PARADE WEST
Locality: Kent Town
AMG Reference: TG828329
Date of Count: 12/04/2011
Day: Tuesday
Weather: Dry
Control: SIGNALS
Survey Status:

Arm	Road Number - Name
1	6146 - FULLARTON ROAD
2	6027 - THE PARADE
3	6146 - FULLARTON ROAD
4	6150 - FLINDERS STREET
5	THE PARADE WEST

	Arm	1				2				3				4				5			
	Exit Arm	2 (L)	3	4	5 (R)	3 (L)	4	5	1 (R)	4 (L)	5	1	2 (R)	5 (L)	1	2	3 (R)	1 (L)	2	3	4 (R)
11 hour totals	Cars	621	5360	1782	49	1157	3396	2868	864	121	275	5335	1282	209	2639	3839	0	133	2912	0	0
	CV	25	94	6	5	19	104	35	16	2	2	82	23	4	24	124	0	4	46	0	0
	Total	646	5454	1788	54	1176	3500	2903	880	123	277	5417	1305	213	2663	3963	0	137	2958	0	0
AM Peak hour (07:45)	Cars	20	728	274	4	107	797	556	145	4	52	541	60	37	155	249	0	15	273	0	0
	CV	2	9	2	0	3	13	4	0	0	0	4	0	0	3	11	0	0	4	0	0
	Total	22	737	276	4	110	810	560	145	4	52	545	60	37	158	260	0	15	277	0	0
PM Peak hour (16:45)	Cars	57	574	136	6	99	234	231	60	10	24	615	61	15	510	826	0	16	434	0	0
	CV	0	3	1	0	0	10	2	0	0	0	3	0	0	0	7	0	0	8	0	0
	Total	57	577	137	6	99	244	233	60	10	24	618	61	15	510	833	0	16	442	0	0

		1	2	3	4	5
One-way Flows	11 Hour Totals	(IN) 7942 (OUT) 9097	(IN) 8459 (OUT) 8872	(IN) 7122 (OUT) 6630	(IN) 6839 (OUT) 5411	(IN) 3095 (OUT) 3447
	AM Peak Hour	07:30 1080 08:00 888	07:45 1625 11:45 750	08:15 696 07:30 847	08:00 506 07:45 1090	08:00 302 07:45 653
	PM Peak Hour	16:30 786 16:45 1204	15:00 795 17:15 1419	14:45 785 16:30 678	16:45 1358 14:45 477	17:15 466 15:00 370
Two-way Flows	AM Peak Hour	07:30 1928	07:45 2244	07:30 1534	07:45 1545	07:45 945
	PM Peak Hour	16:45 1981	17:00 2032	15:15 1442	16:45 1749	15:15 801
All Vehicles	11 Hour Totals	17039 1.5% CV	17331 2.3% CV	13752 1.6% CV	12250 2.2% CV	6542 1.5% CV
	Estimated AADT	22300 SF(1.00) ZF(1.31)	22700 SF(1.00) ZF(1.31)	18000 SF(1.00) ZF(1.31)	16000 SF(1.00) ZF(1.31)	8600 SF(1.00) ZF(1.31)

AADT - Annual Average Daily Traffic SF - Seasonal Factor ZF - Zone Factor CV - Commercial Vehicl

4.1.5.2. Intersection Drawing