STRUCTURAL DESIGN CALCULATIONS

GAZEBO DESIGN (3.0x4.0m)

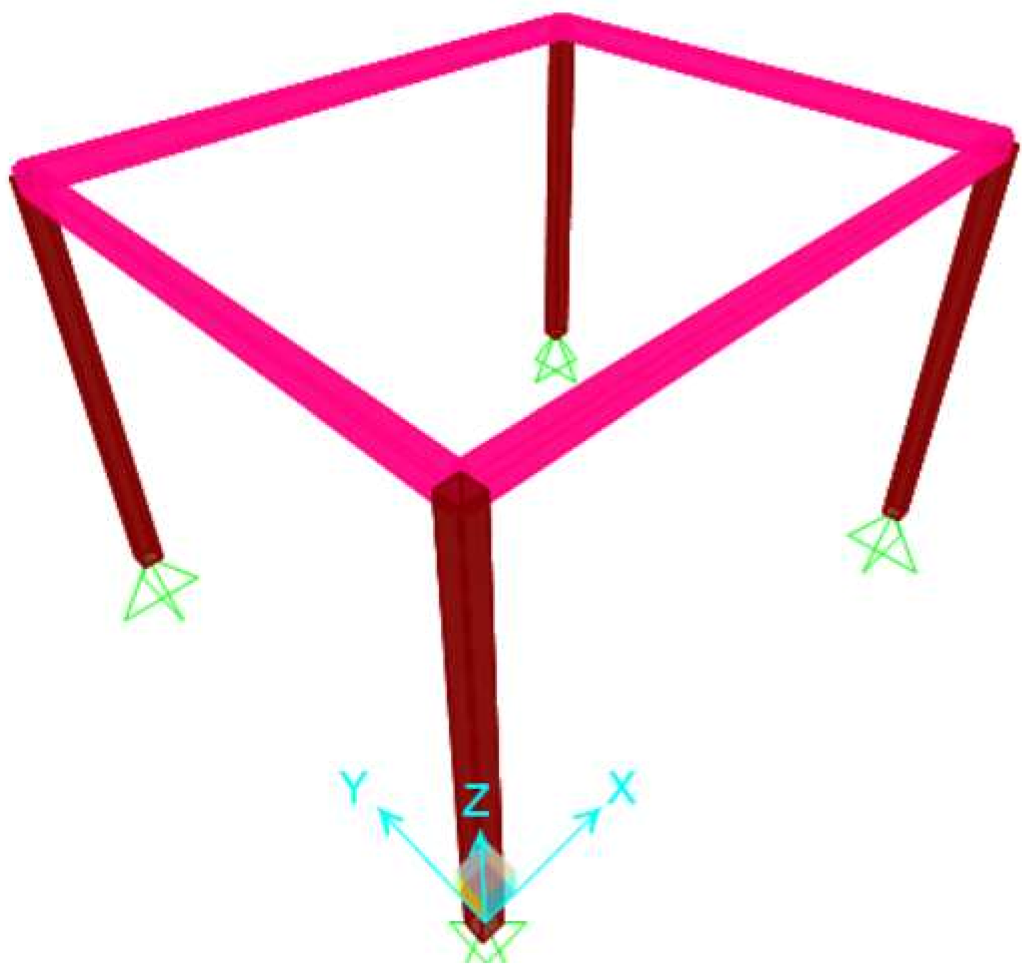
CLIENT: GALE PACIFIC LIMITED

**145 WOODLANDS DRIVE, BRAESIDE,
VICTORIA 3195, AUSTRALIA**

Dated: 19-10-2020

PREPARED BY: Engr. Salman Amjad

1. 3D VIEW OF ANALYSIS MODEL


Y
Z
X

2. INPUT PARAMETERS

2.1. DESIGN LOADINGS & LOAD COMBINATIONS

Following floor loadings have considered for design;

Dead Loadings: Self-weight of Elements

Construction Live Loadings: = 0.250 kN/m^2

Wind Loadings: Design Wind loads = 0.78 kN/m^2

Service Wind loads = 0.365 kN/m^2

Above values includes pressure coefficient (C_{pn})

(Refer to below Wind Calculations)

Load Combinations: Dead Load

Dead Load + Wind Load

1.35 x Dead Load

1.2 x Dead Load + 1.5 x Live Load

1.20 x Dead Load + 1.0 x Wind Load

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			Date:	15/10/2020

Design Calculation sheet

WIND PRESSURE CALCULATIONS

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WIND PRESSURE CALCULATION AS PER AS1170

Design Wind Pressure= $p = (0.5\ \rho_{air})\ [V_{des,\theta}]^2\ C_{fig}\ C_{dyn}$

Design Forces on Surface= $F = \sum(p_z\ A_z)$

where

p_z = design wind pressure in pascals (normal to the surface) at height z, calculated in Clause 2.4.1

NOTE: The sign convention for pressures leads to forces towards the surface for positive pressures and forces away from the surface for negative pressures.

A_z = a reference area, in square metres, at height z, upon which the pressure at that height (p_z) acts

REGIONAL WIND SPEED

TABLE 3.1
REGIONAL WIND SPEEDS

Regional wind speed (m/s)	Region				
	Non-cyclonic			Cyclonic	
	A (1 to 7)	W	B	C	D
V_1	30	34	26	$23\times F_C$	$23\times F_D$
V_5	32	39	28	$33\times F_C$	$35\times F_D$
V_{10}	34	41	33	$39\times F_C$	$43\times F_D$
V_{20}	37	43	38	$45\times F_C$	$51\times F_D$
V_{25}	37	43	39	$47\times F_C$	$53\times F_D$
V_{50}	39	45	44	$52\times F_C$	$60\times F_D$
V_{100}	41	47	48	$56\times F_C$	$66\times F_D$
V_{200}	43	49	52	$61\times F_C$	$72\times F_D$
V_{250}	43	49	53	$62\times F_C$	$74\times F_D$
V_{500}	45	51	57	$66\times F_C$	$80\times F_D$
V_{1000}	46	53	60	$70\times F_C$	$85\times F_D$
V_{2000}	48	54	63	$73\times F_C$	$90\times F_D$
V_{2500}	48	55	64	$74\times F_C$	$91\times F_D$
V_{5000}	50	56	67	$78\times F_C$	$95\times F_D$
V_{10000}	51	58	69	$81\times F_C$	$99\times F_D$
V_R ($R \geq 5$ years)	$67-41R^{-0.1}$	$104-70R^{-0.045}$	$106-92R^{-0.1}$	$F_C\,(122-104R^{-0.1})$	$F_D\,(156-142R^{-0.1})$

V100 =	48.0 m/s	Design Wind Speed
V25 =	39.0 m/s	Serviceiability Wind Speed

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WIND PRESSURE CALCULATION AS PER AS1170

DESIGN WIND SPEED

Constants		
Density of air	1.2	kg/m^3
Location & Hazard Design		
Region Site Exposure Classification	B	Non-cyclonic
Average Recurrence Interval, R	100	years
Terrain category (TC)	1.00	
Probability of exceedance, P=1/R	0.01	
Regional wind speed, V_R	48.0	m/s
Site wind speed, V_site,β	48.0	m/s
Design wind speed, V_des,ϴ	48.0	m/s
Wind Speed Multipliers		
Wind direction multiplier, M_d	1.00	(Likely possible)
	0.99	(Largest possible)
Terrain/height multiplier, M_z,cat	1.00	
Shielding multiplier, M_s	1.00	
Terrain multiplier, M_t	1.00	

SERVICEIABILITY WIND SPEED

Constants		
Density of air	1.2	kg/m^3
Location & Hazard Design		
Region Site Exposure Classification	B	Non-cyclonic
Average Recurrence Interval, R	100	years
Terrain category (TC)	1.00	
Probability of exceedance, P=1/R	0.01	
Regional wind speed, V_R	39.0	m/s
Site wind speed, V_site,β	39.0	m/s
Design wind speed, V_des,ϴ	39.0	m/s
Wind Speed Multipliers		
Wind direction multiplier, M_d	1.00	(Likely possible)
	0.99	(Largest possible)
Terrain/height multiplier, M_z,cat	1.00	
Shielding multiplier, M_s	1.00	
Terrain multiplier, M_t	1.00	

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WIND PRESSURE CALCULATION AS PER AS1170

WIND DIRECTIONALITY MULTIPLIER, Md

TABLE 3.2

WIND DIRECTION MULTIPLIER (M_d)

Cardinal directions	Region A1	Region A2	Region A3	Region A4	Region A5	Region A6	Region A7	Region W
N	0.90	0.80	0.85	0.90	1.00	0.85	0.90	1.00
NE	0.80	0.80	0.80	0.85	0.85	0.95	0.90	0.95
E	0.80	0.80	0.80	0.90	0.80	1.00	0.80	0.80
SE	0.80	0.95	0.80	0.90	0.80	0.95	0.90	0.90
S	0.85	0.90	0.80	0.95	0.85	0.85	0.90	1.00
SW	0.95	0.95	0.85	0.95	0.90	0.95	0.90	1.00
W	1.00	1.00	0.90	0.95	1.00	1.00	1.00	0.90
NW	0.95	0.95	1.00	0.90	0.95	0.95	1.00	0.95
Any direction	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00

Md = 1.00

TERRAIN/HEIGHT MULTIPLIER, Mz,cat

TABLE 4.1

TERRAIN/HEIGHT MULTIPLIERS FOR GUST WIND SPEEDS IN FULLY DEVELOPED TERRAINS—ALL REGIONS

Height (z) m	Terrain/height multiplier ($M_{z,cat}$)			
	Terrain category 1	Terrain category 2	Terrain category 3	Terrain category 4
≤3	0.99	0.91	0.83	0.75
5	1.05	0.91	0.83	0.75
10	1.12	1.00	0.83	0.75
15	1.16	1.05	0.89	0.75
20	1.19	1.08	0.94	0.75
30	1.22	1.12	1.00	0.80
40	1.24	1.16	1.04	0.85
50	1.25	1.18	1.07	0.90
75	1.27	1.22	1.12	0.98
100	1.29	1.24	1.16	1.03
150	1.31	1.27	1.21	1.11
200	1.32	1.29	1.24	1.16

NOTE: For intermediate values of height z and terrain category, use linear interpolation.

Terrain Catagorey= 1

Height, Z (m)= 3

Mz,cat= 1.00

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WIND PRESSURE CALCULATION AS PER AS1170

SHIELDING MULTIPLIER, Ms

Ms = 1.00

TOPOGRAPHIC MULTIPLIER, Mt

Mt = 1.00

DYNAMIC RESPONSE FACTOR, Cdyn

Cdyn = 1.00

EXTERNAL PRESSURE COEFFICENT

TABLE D4(A)

NET PRESSURE COEFFICIENTS ($C_{p,n}$) FOR MONOSLOPE FREE ROOFS—$0.25 \le h/d \le 1$ (see Figure D2)

Roof pitch (α) degrees	θ = 0 degrees				θ = 180 degrees			
	$C_{p,w}$		$C_{p,\ell}$		$C_{p,w}$		$C_{p,\ell}$	
	Empty under	Blocked under	Empty under	Blocked under	Empty under	Blocked under	Empty under	Blocked under
0	−0.3, 0.4	−1.0, 0.4	−0.4, 0.0	−0.8, 0.4	−0.3, 0.4	−1.0, 0.4	−0.4, 0.0	−0.8, 0.4
15	−1.0	−1.5	−0.6, 0.0	−1.0, 0.2	0.8	0.8	0.4	−0.2
30	−2.2	−2.7	−1.1, −0.2	−1.3, 0.0	1.6	1.6	0.8	0.0

Roof Pressure Coefficent, Cpe = (-0.4, 0.4)

External Pressure Coefficent, Cpe = 1.30

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WIND PRESSURE CALCULATION AS PER AS1170

AERO-DYNAMIC SHAPE FACTOR, C_{fig}

$C_{fig,e}$ = $C_{p,e}$ K_a $K_{c,e}$ K_l K_p

Area Reduction Factor, Ka = 1.0

External Combination Factor, Kc,e 1.0

Local Pressure Factor, Kl = 1.0

Net Porosity Factor, Kp = 1.0

$C_{fig,e}$ = 1.0

Design Wind Pressure, pu=

Pu = $p = (0.5\ \rho_{air})\ [V_{des,\theta}]^2\ C_{fig}\ C_{dyn}$

P u= $0.5 \times 1.2 \times 48^2 \times 1.0 \times 1.0 / 1000$

Pu = 1.382 kPa

Service Wind Pressure, ps=

Ps = $p = (0.5\ \rho_{air})\ [V_{des,\theta}]^2\ C_{fig}\ C_{dyn}$

Ps = $0.5 \times 1.2 \times 39^2 \times 1.0 \times 1.0 / 1000$

P s= 0.913 kPa

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WIND PRESSURE CALCULATION AS PER AS1170

Applied Ultimate Wind Pressure, Wu=

Wu, roof = 1.382 x 0.4 = 0.55 kPa

Wu, wall = 1.3 x 1.382 = 1.80 kPa

Applied Service Wind Pressure, Ws=

Ws, roof = 0.913 x 0.4 = 0.365 kPa

Ws, wall = 0.913 x 1.3 = 1.19 kPa

Applied Member Loadings

Column Section=	100 x 100 x 1.4
Main Beam Section=	150 x 62 x 1.3
Secondary Beam Section=	33 x 125 x 1.1

a) Applied Ultimate Wind Loadings

Line Loading on Column =	1.80 x 0.1 =0.18 kN/m
Line Loading on Main Beam =	0.55 x 0.062 =0.0341 kN/m
Side Line Loading on Main Beam =	0.55 x 0.15 =0.082 kN/m
Line Loading on Secondary Beam =	0.55 x 0.125 =0.069 kN/m

b) Applied Service Wind Loadings

Line Loading on Column =	1.19 x 0.1 =0.119 kN/m
Line Loading on Main Beam =	0.365 x 0.062 =0.02262 kN/m
Side Line Loading on Main Beam =	0.365 x 0.15 =0.055 kN/m
Line Loading on Secondary Beam =	0.365 x 0.125 =0.046 kN/m

2.2. MATERIAL STRENGTH

Following material strength have considered for design;

Material Properties of: Alloy 6063-T5

Compressive Yield Strength, f_{cy} = 110 MPa

Tensile Yield Strength, f_{ty} = 110 MPa

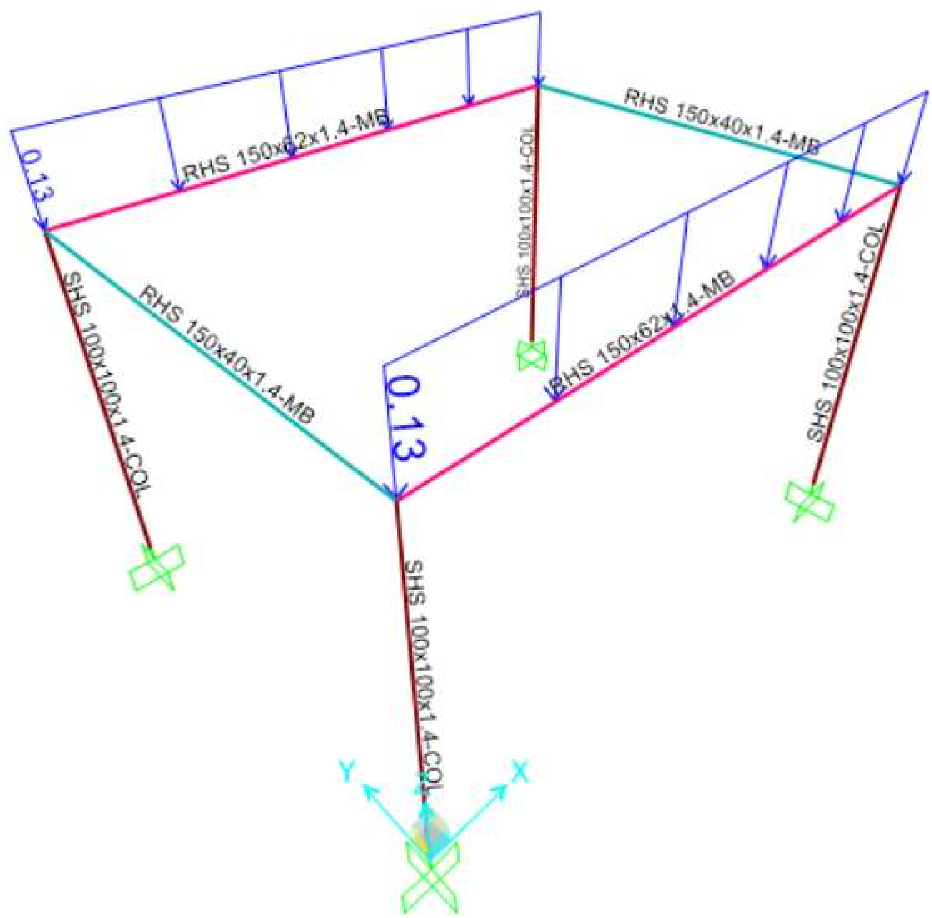
Tensile Ultimate Strength, f_{tu} = 152 MPa

Shear Ultimate Strength, f_{su} = 90 MPa

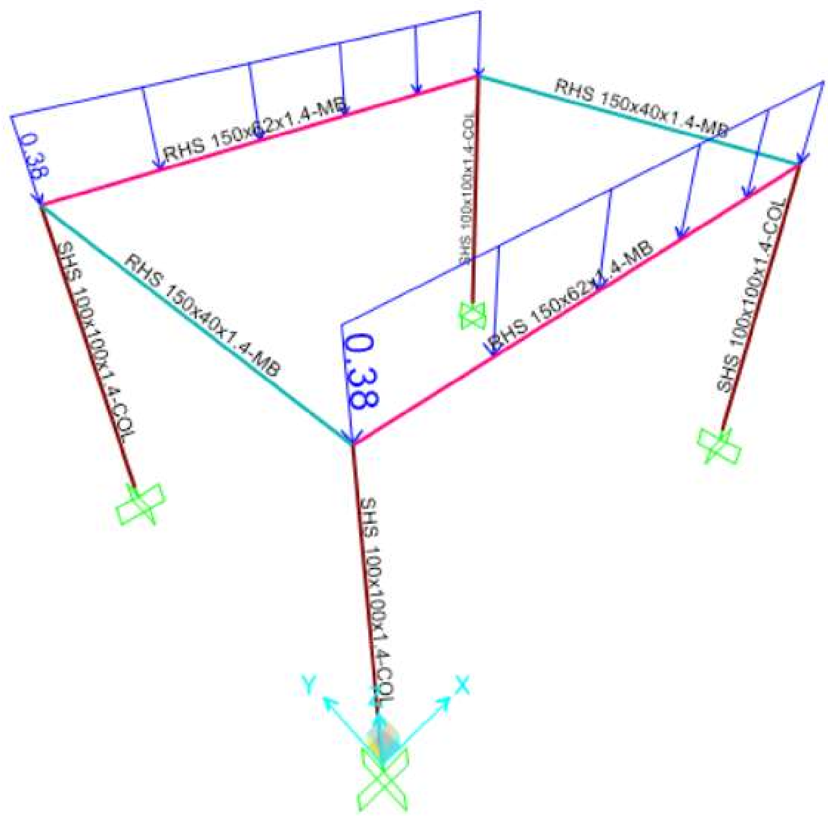
Refer to AS1664.1 table 3.3A

Design Code: AS1664

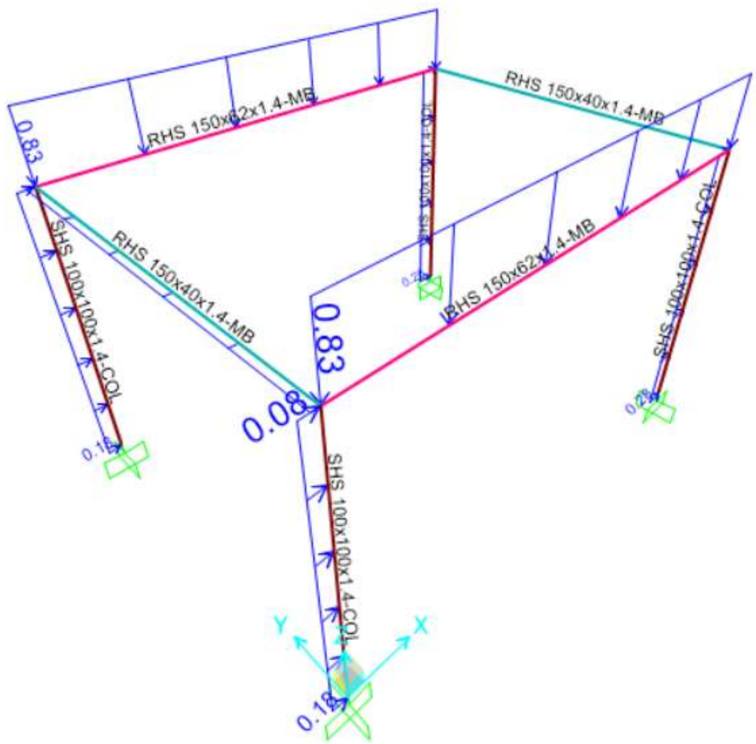
2.3. APPLIED LOADING



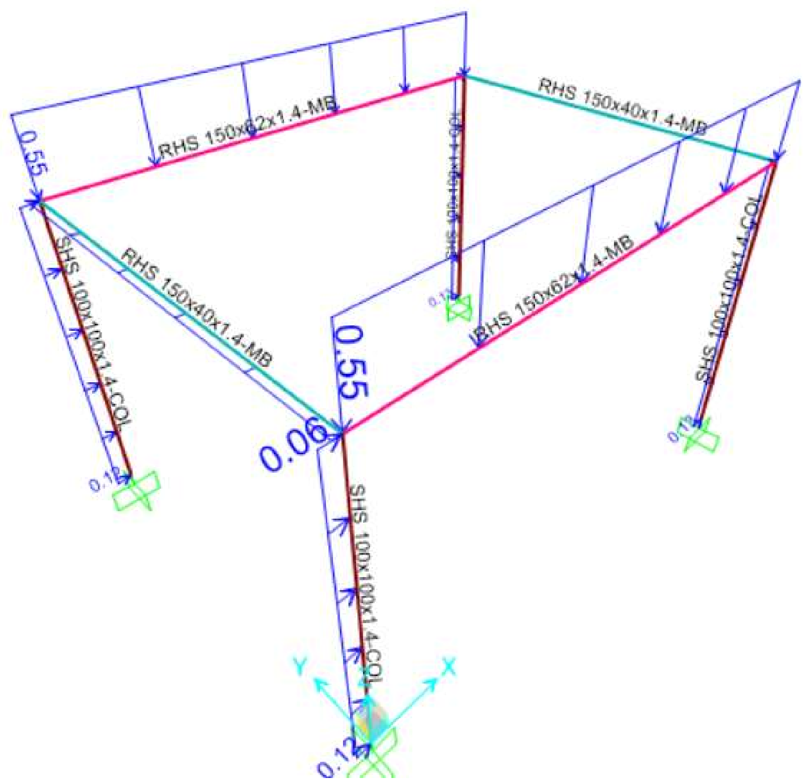
Applied Dead Loadings



Applied Live Loadings



Ultimate Wind Loadings



Service Wind Loadings

3. CRITICAL ELEMENTS DESIGN

3.1. BEAM DESIGN

Member Size = 150 x 62 x 1.4

Member Span = 4.0m

Panel Distributary Width = 1.50m

Dead Load = 0.09 kN/m^2

(From Self-weight of 33.9 x 125 x 1.1)

= 0.09 x 1.5 = 0.135 kN/m

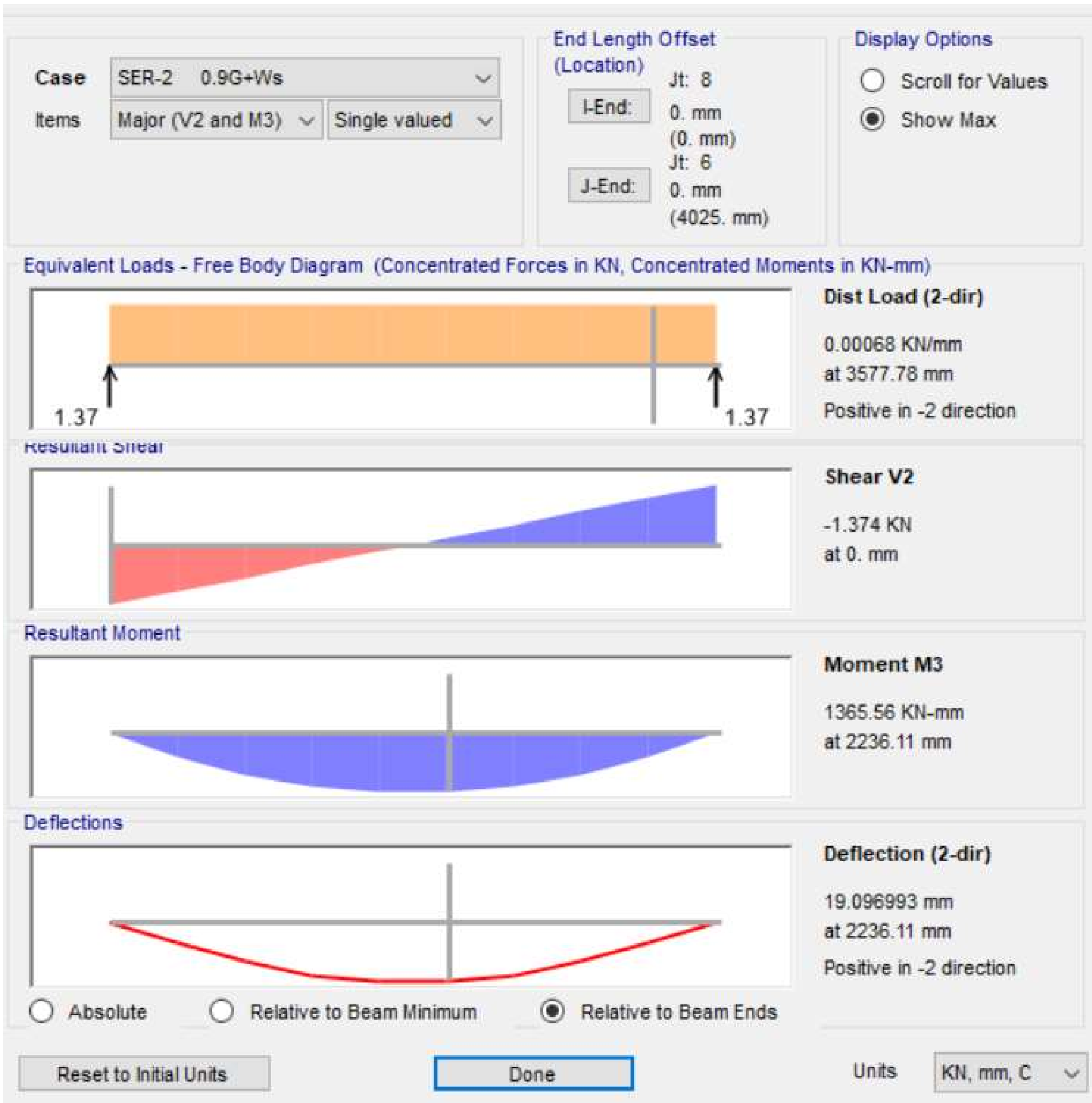
Live Load = 0.25 kN/m^2

= 0.25 x 1.5 = 0.375 kN/m

Ultimate Line Loading = 0.55 x 1.5 = 0.825 kN/m

Service Line Loading = 0.365 x 1.5 = 0.55 kN/m

A) DEFLECTION CHECK



Maximum Deflection Value, δ = 19.09mm

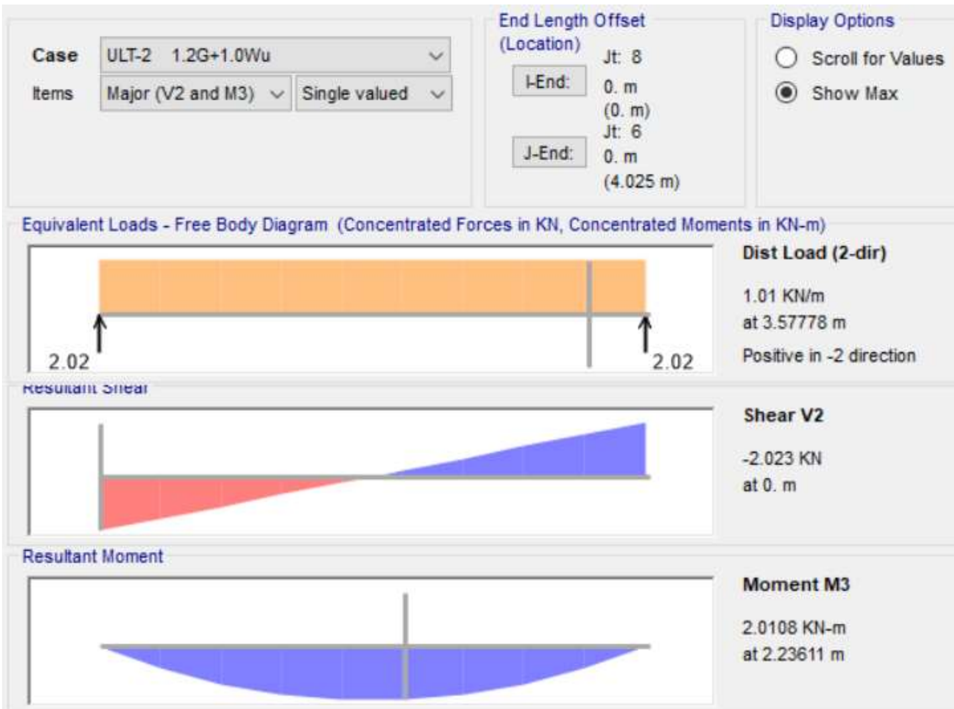
Calculated Deflection Limit = 4000/180 = 22.22mm

Allowable Deflection Limit = L/180

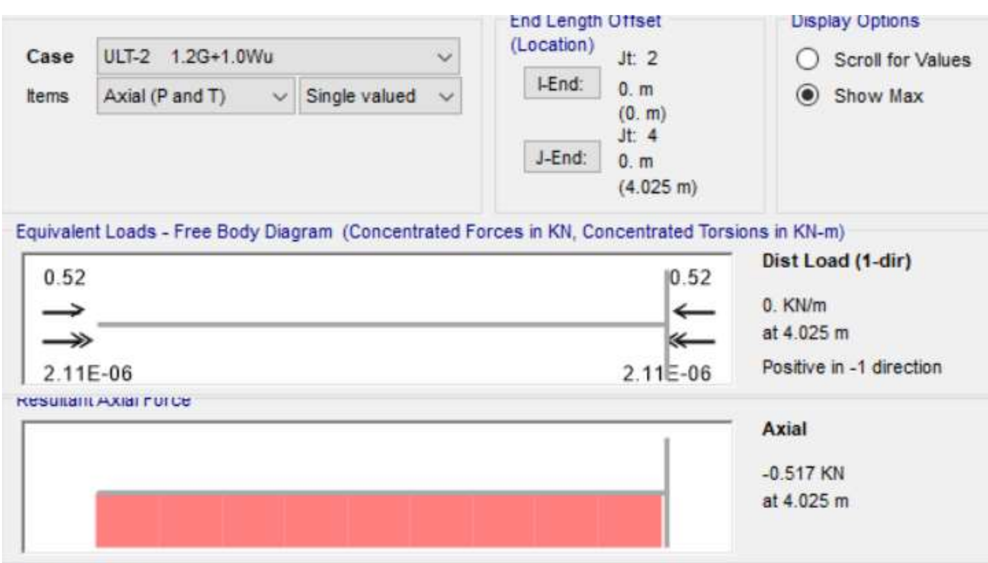
Therefore, member size (150 x 62 x 1.4) is adequate.

B) STRENGTH CHECK

MAJOR DIRECTION BENDING MOMENT AND SHEAR FORCE



AXIAL FORCE DIAGRAM



In conclusion, following are the design forces

Ultimate Bending Moment (Major Direction), Mu	= 2.016 kN-m
Ultimate Shear Force (Major Direction), Vu	= 2.023 kN
Ultimate Axial Force, Pu	= 0.520 kN

C) Design Stresses Check

Bending Stress Check

Gross sectional area, Ag	= 586 mm^2
In plane Elastic Section Modulus, Zy	= 22702 mm^3
Stress from axial force = fa = P/Ag	= 517 /586
	= 0.882 MPa
Stress from in-plane fby = My/Zy	= 2.023x 10^6/22702
	= 89.01 MPa

Compression in beam Eq 3.4.15

Unsupported Length of Member, major = Lmaj	= 4.000 m
Unsupported Length of Member, minor = Lmin	= 4.000/30 = 0.133m
Effective length factor = k	= 1
Radius of gyration about buckling axis (Y) = r_y	= 53.90mm
Radius of gyration about buckling axis (z) = rz	= 27.22mm

Slenderness ratio = kLb/r_y = 4000/53.9 = 74.21

Slenderness ratio = kLb/rz = 133/27.22 = 4.88

Bc = 119.3 MPa REFER AS1664.1 TABLE 3.3D

Dc = 0.492 MPa   REFER AS1664.1 TABLE 3.3D

Cc = 99.38   REFER AS1664.1 TABLE 3.3D

S1 = 21.51

S2 = 3857.96

J = 1085373 mm^4

Iy = 1702603 mm^4

Zc = 14002 mm^3

Lb x Zc/[0.5 x (Iy x J)$^{1/2}$]   = 2.740 < S1 Therefore

ϕFL = ϕ_b x Fcy

= 0.85 x 110 = 93.5 MPa > 89.89 MPa

Utilization Ratio = 89.89 / 93.5 = 0.961

Shear Stress Check

Clear depth = h	= 150mm
Thickness = t	= 1.4mm
h/t = 150/1.4	= 107.2

Bs = 75.83 REFER AS1664.1 TABLE 3.3

Ds = 0.242 REFER AS1664.1 TABLE 3.3

Cs = 128.47 REFER AS1664.1 TABLE 3.3

S1 =34.31 REFER AS1664.1 TABLE 3.3

ϕFL = ϕyFsy   = 0.95 x 62   = 58.9 MPa

Shear Stress, v_u = 2023 / (150 x 1.4 x 2) = 4.82 MPa

As Shear Stress, vu < ϕFL Therefore, the provided section is adequate.

3.2. COLUMN DESIGN

Member Size = 100 x 100 x 1.4

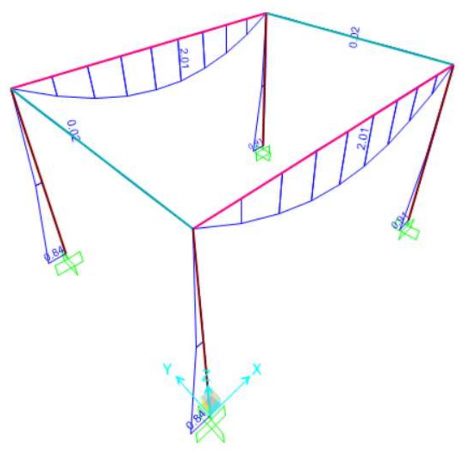
Member Span = 2.50 m

Design Forces

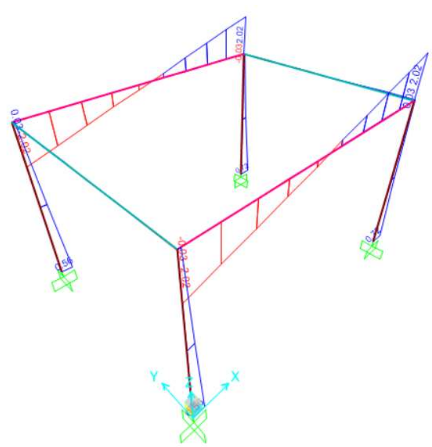
Ultimate Bending Moment (Major Direction), Mu = 0.844 kN-m

Ultimate Shear Force (Major Direction), Vu = 0.56 kN

Ultimate Axial Force, Pu = 2.023 kN



BENDING MOMENT DIAGRAM



SHEAR FORCE DIAGRAM

Bending Stress Check

Gross sectional area, Ag = 552 mm^2

In plane Elastic Section Modulus, Zy = 17897 mm^3

Stress from axial force = fa = P/Ag = 2023 /552

= 3.66 MPa

Stress from in-plane fby = My/Zy = $0.844x\ 10^6/17897$

= 47.16 MPa

Compression in beam Eq 3.4.15

Unsupported Length of Member, major = Lmaj = 2.50 m

Unsupported Length of Member, minor = Lmin = 2.50m

Effective length factor = k = 1

Radius of gyration about buckling axis (Y) = r_y = 40.25mm

Radius of gyration about buckling axis (z) = rz = 40.25mm

Slenderness ratio = kLb/r_y = 2500/40.25 = 62.81

Slenderness ratio = kLb/rz 2500/40.25 = 62.81

Bc = 119.3 MPa REFER AS1664.1 TABLE 3.3D

Dc = 0.492 MPa REFER AS1664.1 TABLE 3.3D

Cc = 99.38 REFER AS1664.1 TABLE 3.3D

S1 = 21.51

S2 = 3857.96

J = 1342019 mm^4

Iy = 894860 mm^4

Zc = 17897 mm^3

Lb x Zc/[0.5 x (Iy x J)$^{1/2}$] = S2 > 81.66 > S1 Therefore

ϕFL = ϕ_b x Fcy

ϕFL = ϕb x (Bc – 1.6Dc x (Lb x Zc/0.5 x (Iy x J)$^{1/2}$)

ϕFL = 0.85 x 61.01 = 51.85 MPa

Total Stresses = 3.66 + 47.16 = 50.82 MPa < 51.85 MPa

Therefore, the provided section is adequate.

Shear Stress Check

Clear depth = h	= 100mm
Thickness = t	= 1.4mm
h/t = 100/1.4	= 71.42

Bs = 75.83 REFER AS1664.1 TABLE 3.3

Ds = 0.242 REFER AS1664.1 TABLE 3.3

Cs = 128.47 REFER AS1664.1 TABLE 3.3

S1 =34.31 REFER AS1664.1 TABLE 3.3

ϕFL = ϕyFsy = 0.95 x 62 = 58.9 MPa

Shear Stress, v_u = 560 / (100 x 1.4 x 2) = 2.00 MPa

As Shear Stress, vu < ϕFL Therefore, the provided section is adequate.

3.3. PIER DESIGN

Vertical Compression Load, Pu = 2.023 kN

Horizontal Shear, Vu = 0.56 kN

Bending Moment, Mu = 0.84 kN-m

Bearing Stress Check

Try ϕ450 x 600 Pier

Stresses due to Axial Forces = 2.023 / (0.159) = 12.72 kPa

Stresses due to Bending Moment = 0.84 x 0.225 / 2.01 x 10^{-03}

= 94.02 kPa

Total Bearing Stress = 12.72 +94.02 = 106.74 kPa < Allowable Ultimate

Bearing Pressure = 150 kPa

Uplift Check

Maximum Uplift Pressure = 0.365 kPa

Total Area = 3.0 x 4.0m

Total Uplift Force = 3.0 x 4.0 x 0.365 = 4.38 kN

Force on One Pier = 1.095 kN

Self-weight of Single Pier = 0.159 x 0.6 x 24 = 2.289 kN > 1.095 kN

Therefore ϕ450 x 600 Pier is Adequate to Bear The Loadings.