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		Prepared by	YSZ
Project		Date	15.06.2020
Title	Hannah @ Chloe Pergola	Revision	-

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1. Introduction

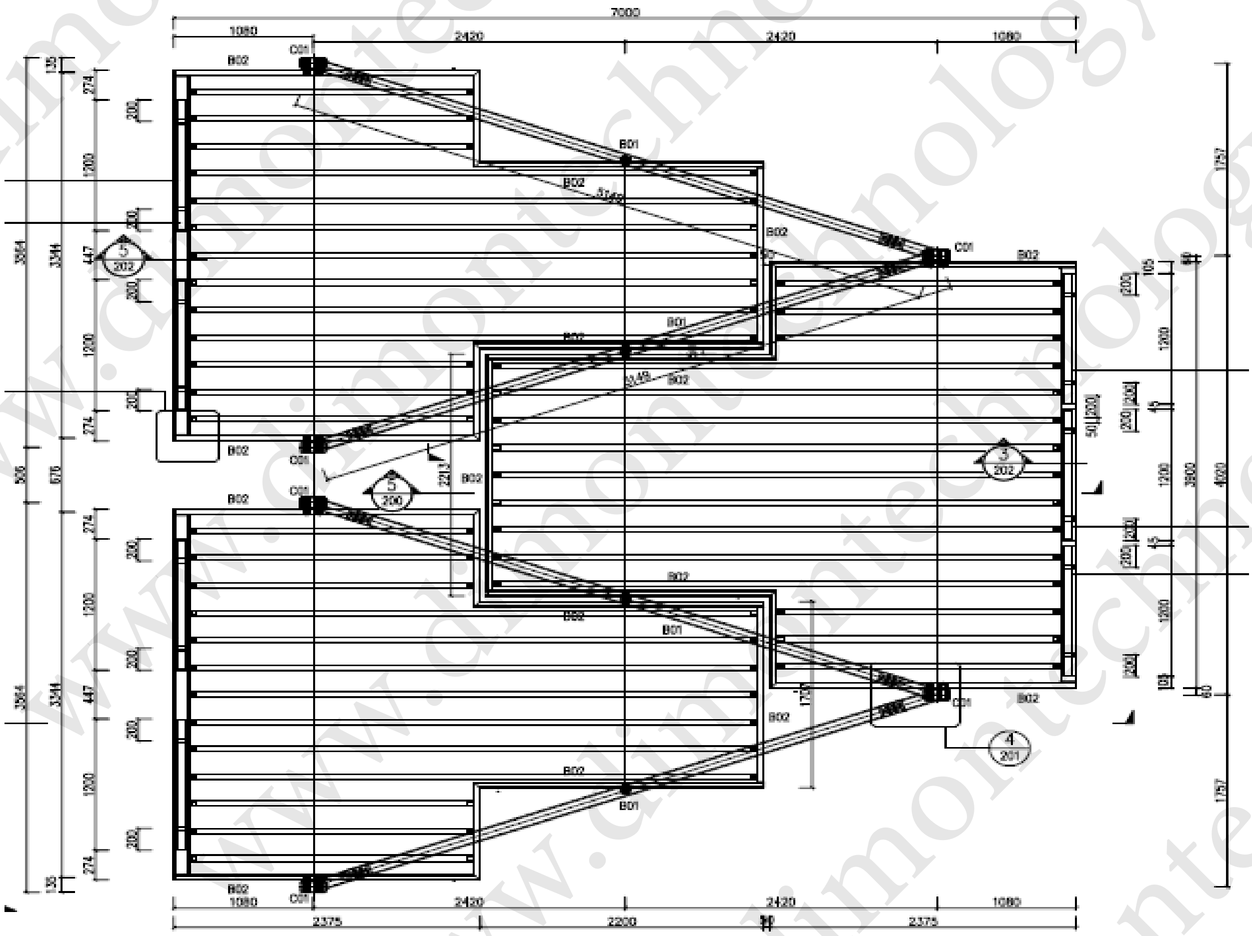
The Covered walkway includes wooden platform and steel frame. The construction of wooden platform it is made of 145x45mm timber and s.s. RHS,it And it's fixed steel frame with bolt. And steel frame with bolt. And steel frame fixed concrete foundation with holdging down bolt

The Covered walkwayis subjected to wind load . The basic wind pressure is taken as 3.1 kPa (72m/s) for the height of the Building is less than 250m. The pressure coefficients actor is taken as 2.0 for open frame The topography factor is taken as 1.0.

In this submission, there is no computer program to be used for the design or structural analysis.

Load path (Vertical wind load) : timber →s.s Beam →steel frame → embed → r.c. structure

Load path (Horizont wind load) : timber /steel frame→ s.s Beam →steel frame → embed → r.c. structure



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2. Design Data

2.1 The design of steelwork to be complied with 'Code of Practice for the Structural Use of Steel 2011, Hong Kong' and 'Code of Practice for the Structural Use of Steel Macau The Steel sections to be grade Q345 complied to comply with GB/t 706-2008 and GB/t 3274-2007.

steel grade			Q345
thickness		(mm)	≤16
ultimate tensile strength,	U_s	(N/mm^2)	400
design strength,	p_y	(N/mm^2)	310
design shear strength,	v_c	(N/mm^2)	180
Modulus of Elasticity,	E_s	(N/mm^2)	206000

2.2 All welding to be complied with complied to GB50661-2011

steel grade		E50xx
design strength of filled weld	(N/mm^2)	200

2.3 ALL ordinary bolts to be complied with BS 4190

grade		8.8
ultimate tensile strength,	(N/mm^2)	800
design strength,	(N/mm^2)	450
design shear strength,	(N/mm^2)	375
design bearing strength,	(N/mm^2)	1000

2.4 All stainless steel bolts or screws to be grade A4 complied to BS EN ISO 3506

class		70
0.2% proof stress	(N/mm^2)	450
ultimate tensile strength	(N/mm^2)	700
design tensile strength	(N/mm^2)	373
design shear strength	(N/mm^2)	280
design bearing strength	(N/mm^2)	805

2.5 Stainless steel to be grade X5CrNiMo17-12-2 complied with BS EN 10088

stainless steel grade		1.4401 (316 S31) X5CrNiMo17-12-2
0.2% proof stress	(N/mm^2)	220
ultimate tensile strength, min.	(N/mm^2)	520
Modulus of elasticity	(N/mm^2)	200000
Design strength	(N/mm^2)	220
Design stress of fillet weld	(N/mm^2)	220

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2.6 145x45mm thick timber (refer to appendix)

	short grain	acorss grain
compressive strength	50 N/mm^2	50 N/mm^2
tensile stress	95 N/mm^2	95 N/mm^2
Modulus of Elasticity,	10.9 N/mm^2	
shear stess,	9.1 N/mm^2	
density	540 kg/m^3	

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3. Design loads

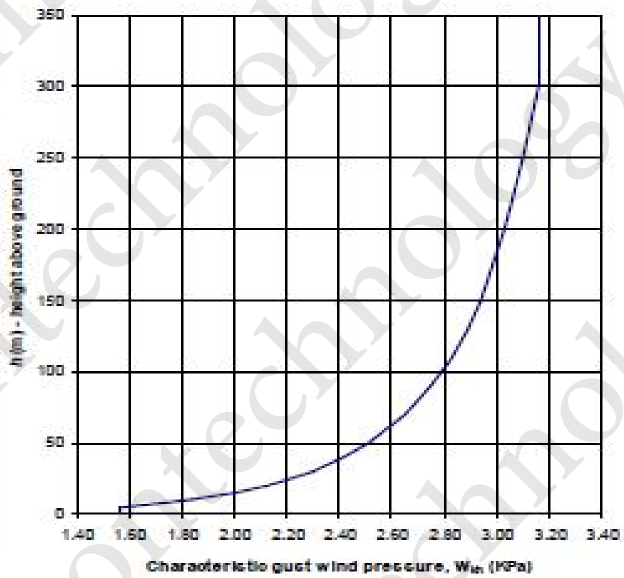
3.1 Dead load

145x45mm thk timber	= 145 x 45 x 5.4 / 1000 / 1000 / 0.245	= 0.144	kN/m^2
Others, say		= 0.156	kN/m^2
	Total	= 0.3	kN/m^2

3.2 Wind load

Table III.2 - Graphic representation and tabulated values of the characteristic gust wind speeds and pressures profile

h (m)	Characteristic gust wind Speed, V_{kh} (m/s)	Characteristic gust wind Pressure, w_{kh} (kPa)
≥300	72.6	3.16
250	**71.9**	**3.10**
200	71.1	3.03
150	70.0	2.94
130	69.4	2.89
110	68.6	2.83
90	67.7	2.75
70	66.5	2.65
50	64.7	2.51
40	63.5	2.42
30	61.9	2.30
20	59.5	2.12
15	57.8	2.00
10	55.4	1.84
≤5	51.2	1.57



Basic wind pressure = 3.1 kPa

Pressure coefficient, C_p = 2 (open frame)

Topography factor S_a = 1

For uplift or downward wind load:
Design wind pressure = 2 x 3.1 x1
= 6.2 kPa

For horizontal wind load:
Design wind pressure = 2 x 3.1 x1
= 6.2 kPa

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3.3 Load combinations

		Downward			Upward	
Wind load	=	6.2	kN/m^2	=	-6.2	kN/m^2
Dead load	=	0.3	kN/m^2	=	0.3	kN/m^2

Load combinations

1.3DL + 1.5WL (downward) + 0.3WL(horizontal)

= 11.55 kN/m^2

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4. Check for wooden platform

4.1 Check for 145x45mm thick timber

<u>Properties of 145x45mm timber</u>

Breadth, $B = 45$ mm — Moment of inertia, $I = 11432344$ mm^4

Minimum thickness, $t = 145$ mm — Elastic modulus, $Z = 157687.5$ mm^3

Area, $A = 6525$ mm

UDL = 11.55 x 45 / 1000 (refer to item 3.3)
= 0.52 kN/m

Spacing of channels = 4560 mm max.

Max bending moment $M = q L^2 / 8$
$= 0.52 \times 4.56^2 / 8$
= 1.352 kNm

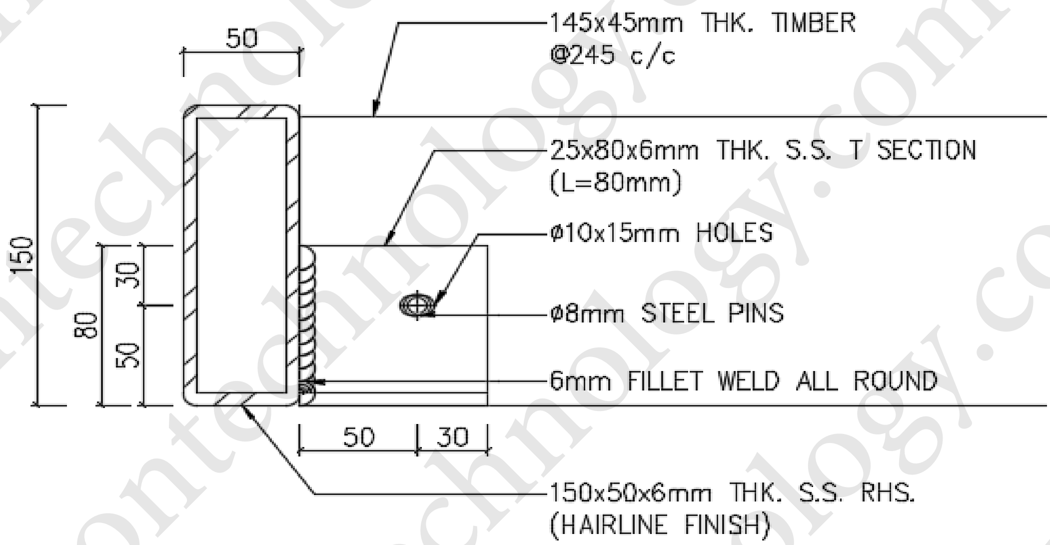
Max shear load $V = q L / 2$
= 0.52 x 4560 / 1000 / 2
= 1.45 kN

Bending stress $f_{bc} = M / Z$
$= 1.352 \times 10^6 / 157687.5$
$= 8.57$ N/mm^2 (factored)
< 50 N/mm^2 O.K.

Shear stress $f_q = V / A$
= 1.45 x 1000 / 6525
$= 0.22$ N/mm^2 (factored)
< 9.1 N/mm^2 O.K.

Deflection $= 5 / 384 \times 0.52 \times 4560^4 / 10900 / 11432344 / 1.5$ (un-factored)
= 17.07 mm
< 4560 / 250 = 18.24 mm O.K.

4.2 Check for 25x80x6mm s.s. T section



Design strength, p_y = 220 N/mm^2 Parameter, $\varepsilon = (275 / p_y)^{0.5}$ = 1.118

Width of flange, B = 25 mm Overall depth, D = 74 mm

Thickness of flange, T = 6 mm Thickness of web, t = 6 mm

b = B / 2 = 12.5 mm

Limiting ratio, b / T β_f = 2.08

= 1.86ε ≤ $\beta_{1f} = 8\varepsilon$ Flange is plastic

Limiting ratio, D / t β_w = 12.33

= 11.03ε ≤ $\beta_{3w} = 18\varepsilon$ Web is semi-compact

Thus, the overall section is semi-compact

member	width w	height h	dist_cg y	qty n	area n w h	moment n w h y	cg z	I_x about its axis $n w h^3 / 12$	I_x about cg $n w h (y - z)^2$
Flange	25	6	3	1	150.0	450.0	32.90	450.0	134092.4
Web	6	74	43	1	444.0	19092.0	32.90	202612.0	45301.5
about x-x					594.0	19542.0	32.90	203062.0	179393.9

member	width h	height w	dist_cg x	qty n	area n h w	moment n h w x	cg z_x	I_y about its axis $n h w^3 / 12$	I_y about cg $n w h (x - z_x)^2$
Flange	6	25	12.5	1	150.0	1875.0	12.50	7812.5	0.0
Web	74	6	12.5	1	444.0	5550.0	12.50	1332.0	0.0
about y-y					594.0	7425.0	12.50	9144.5	0.0

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Width of flange, $B = 25$ mm   Depth of the section, $D = 80$ mm

Thickness of flange, $T = 6$ mm   Thickness of web, $t = 6$ mm

Total area of the section, $A = 594.00$ mm^2

Total moment of inertia, $I_x = 382456$ mm^4

Total moment of inertia, $I_y = 9145$ mm^4

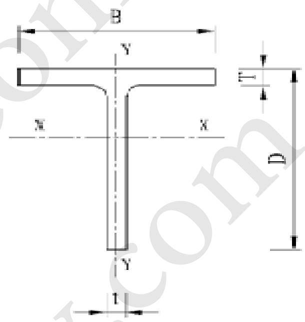
Section modulus, flange $Z_x = 11625.16$ mm^3

Section modulus, toe $Z_x = 8119.91$ mm^3

Section modulus, $Z_y = 731.56$ mm^3

Radius of gyration, $r_x = 25.37$ mm

Radius of gyration, $r_y = 3.92$ mm



Distance, $d_1 = (A / 2 - B\,T) / t = 24.5$ mm

Plastic modulus, $S_x = B\,T\,(T / 2 + d_1) + t\,d_1^2 / 2 + t\,(D - T - d_1)^2 / 2$

$= 13276.5$ mm^3

Design strength, $p_y = 220$ N/mm^2

Parameter, $\varepsilon = (275 / p_y)^{0.5} = 1.118$

Modulus of Elasticity, $E = 200000$ N/mm^2

Effective length, $L_E = 50$ mm

Slenderness, $\lambda = L_E / r_y = 12.74$

Flange ratio, $\gamma = 1 - I_y / I_x = 0.9761$

Warping constant, $H = B^3\,T^3 / 144 + (D - T / 2)^3\,t^3 / 36 = 2762635.5$

$w = 4\,H / [I_y\,(D - T / 2)^2] = 0.2038$

Torsional constant, $J = B\,T^3 / 3 + (D - T)\,t^3 / 3 = 7128$

Torsional index, $x = 0.566\,(D - T / 2)\,(A / J)^{0.5} = 12.581$

$c = z = 32.90$

$y_0 = c - T / 2 = 29.9$

Monosymmetry index, $\psi = \{2\,y_0 - (y_0\,B^3\,T / 12 + B\,T\,y_0^3 + t\,[(c - T)^4 - (D - c)^4] / 4) / I_x\} / (D - T / 2)$

$= 0.8566$

Slenderness factor, $v = 1 / \{[w + 0.05\,(\lambda / x)^2 + \psi^2]^{0.5} + \psi\}^{0.5} = 0.735$

Buckling parameter, $u = \{4\,S_x^2\,\gamma / [A^2\,(D - T / 2)^2]\}^{0.25} = 0.7573$

Ratio, $\beta_w = Z_x / S_x = 0.6116$ for semi-compact section

Equivalent slenderness, $\lambda_{LT} = u\,v\,\lambda\,\beta_w^{0.5} = 5.55$

$P_E = \pi^2\,E / \lambda_{LT}^2 = 64083.14$

$\alpha_{LT} = 7$

$\lambda_{L0} = 0.4\,(\pi^2\,E / p_y)^{0.5} = 37.89$

For $\lambda_{LT} \leq \lambda_{L0}$

Perry factor, η_{LT} = 0

$\o_{LT}$ = $[p_y + (\eta_{LT} + 1) P_E] / 2$ = 32151.57

Bending buckling strength, p_b = $P_E\, p_y / (\o_{LT} + (\o_{LT}{}^2 - P_E\, p_y)^{0.5})$

= 220 N/mm^2

Vertical load = 1.45 kN (refer to item 4.1)

Max. length of cantilever = 50 mm

Max. bending moment at support = 1.45 x 50 / 1000

= 0.07 kNm

Shear stress = 1.45 x 1000 / 6 x 80

= 3.02 N/mm^2

< 0.6 x 127 = 76.2 N/mm^2 (low shear)

Moment capacity = 1.2 p_y Z

= 1.2 x 220 x 8119.9 /10^6

= 2.144 kNm

> 0.07 kNm O.K.

4.3 Check for 6mm fillet weld all round connectiing to 150x50x6mm RHS

Properties for unit throat thickness of fillet weld (effective length)

member	width w	height h	dist_cg y	qty n	area n w h	moment n w h y	cg z	I_x about its axis n w h^3 / 12	I_x about cg n w h (y - z)^2
1	1	74	37	2	148.0	5476.0	46.12	67537.3	12296.8
2	9	1	74	2	18.0	1332.0	46.12	1.5	13996.1
3	1	25	80	1	25.0	2000.0	46.12	1302.1	28704.5
about x-x					191.0	8808.0	46.12	68840.9	54997.5

Overall depth of the section, D = 80 mm

Centre of gravity, z = 46.12 mm

Total area of the section, A = 191.00 mm^2

Total moment of inertia, I_x = 123838.4 mm^4

Section modulus (min), Z_x = 2685.41 mm^3

Section modulus (max), Z_x = 3654.69 mm^3

Radius of gyration, r_x = 25.46 mm

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member	width h	height w	dist_cg x	qty n	area n h w	moment n h w x	cg z_x	I_y about its axis $n h w^3 / 12$	I_y about cg $n w h (x - z_x)^2$
1	1	25	12.5	1	25.0	312.5	12.55	1302.1	0.1
2	1	9	4.5	1	9.0	40.5	12.55	60.8	582.8
3	1	9	21.5	1	9.0	193.5	12.55	60.8	721.4
4	74	1	9.5	1	74.0	703.0	12.55	6.2	687.1
5	74	1	15.5	1	74.0	1147.0	12.55	6.2	645.2
about y-y					191.0	2396.5	12.55	1435.9	2636.6

Overall depth of the section, B = 25 mm
Centre of gravity, z = 12.55 mm
Total area of the section, A = 191.00 mm^2
Total moment of inertia, I_y = 4072.5 mm^4
Section modulus (min), Z_y = 324.58 mm^3
Section modulus (max), Z_y = 327.03 mm^3
Radius of gyration, r_y = 4.62 mm

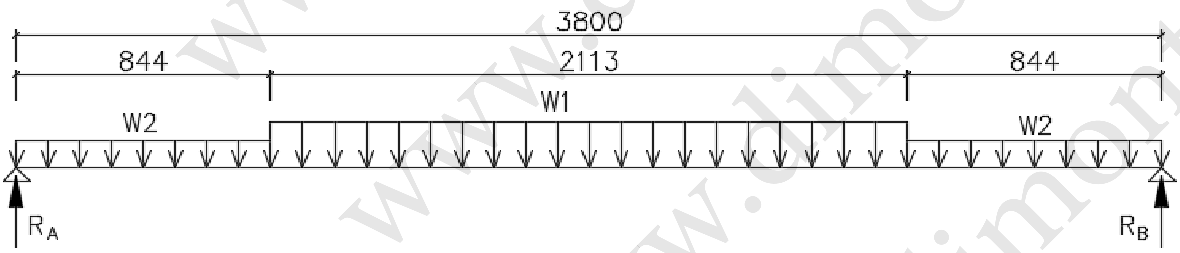
Shear load, V_x = 0 kN   Moment, M_x = 0.07 kNm
Shear load, V_y = 1.45 kN   Moment, M_y = 0 kNm
Tensile load, N = 0 kN   Torsional moment, M_T = 0 kNm

Leg length of fillet weld, t = 6 mm

Shear stress, $\tau_x = V_x / (0.7\ t\ A)$ = 0 N/mm^2
Shear stress, $\tau_y = V_y / (0.7\ t\ A)$ = 1.81 N/mm^2
Tensile stress, $\sigma = N / (0.7\ t\ A)$ = 0 N/mm^2
Tensile stress, $\sigma_x = M_x\ d / (2\ I) / (0.7\ t)$ = 5.38 N/mm^2
Tensile stress, $\sigma_y = M_y\ d / (2\ I) / (0.7\ t)$ = 0 N/mm^2
Shear stress, $\tau_T = M_T\ r / (0.7\ t\ J)$ = 0 N/mm^2

Resultant , $f_w = \{[(\tau_x^2 + \tau_y^2)^{0.5} + \tau_T]^2 + [(\sigma_x^2 + \sigma_y^2)^{0.5} + \sigma]^2\}^{0.5}$
= 5.68 N/mm^2
$\leq$ 200 N/mm^2 O.K.

4.4 Check for 150x120x6mm s.s channel



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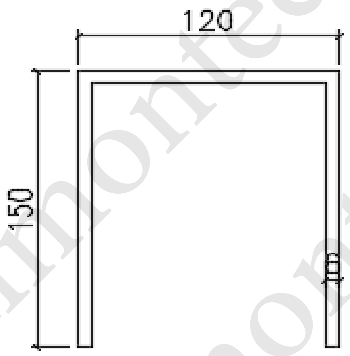
UDL, W1 = 11.55 x 4.56 x 45 / 245 / 2 + 11.55 x 0.12
= 6.22 kN/m

UDL, W2 = 11.55 x 2.28 x 45 / 245 / 2 + 11.55 x 0.12
= 3.8 kN/m

Reaction $R€=R_B$ = (6.22 x 2.113 +3.8 x 2 x 0.844) / 2
= 9.78 kN

Max. Bending moment = 6.22 x 0.844^2 / 2 + (2 - 2.133 / 3.8) x 3.8 x 2.133 x 3.8 / 8
= 10.42 kNm



Area: 2448.0000
Perimeter: 828.0000
Bounding box: X: −60.0000 −− 60.0000
Y: −94.0588 −− 55.9412
Centroid: X: 0.0000
Y: 0.0000
Elastic of modulus: X: 5846967.5294
Y: 6483456.0000
Product of modulus: XY: 0.0000
Radius of gyration: X: 48.8719
Y: 51.4633

Shear stress = 7.16 x 1000 / (2 x 6 x 150)
= 5.43 N/mm^2
< 0.6 x 127 = 76.2 N/mm^2 (low shear)

Moment capacity = 1.2 p_y Z
= 1.2 x 220 x 5846967 / 94.06 /10^6
= 16.411 kNm (factored)
> 10.42 kNm O.K.

Deflection = 3.8 / 1.5 x 844^2 x 3800^2 x (3 - 844^2 / 3800^2) / (48 x 200000 x 5846967.5) +
6.22 / 1.5 x 2133 x 3800^3 x [8 - 4 x $(844/3800)^2$ + $(844/3800)^3$] /
(384x 200000 x 5846967.5)
= 9.815 (unfactored)
< 3800 / 250 = 15.2 mm O.K.

<u>Check for 6mmm fillet weld connecting s.s channel to s.s RHS</u>

Shear force = 9.78 kN (factored)

Effective length of fillet weld = 120 + 2 x 150 - 2 x 6
= 408 mm

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Fillet weld capacity = 6 x 0.7 x 408 x 200 / 1000

= 342.72 kN

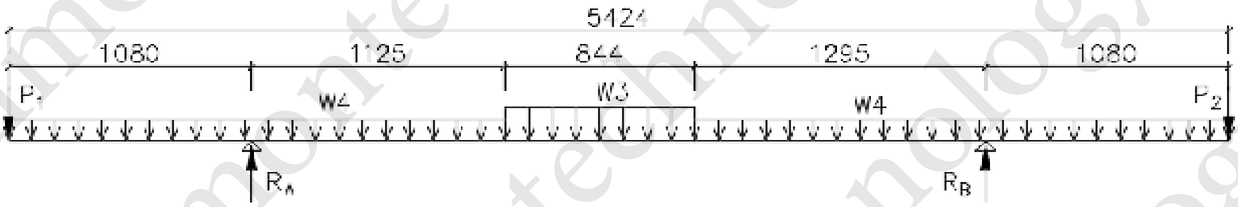
> 9.78 kN O.K.

4.5 Check for 150x50x6mm s.s. RHS

<u>Properties of 150x50x6mm RHS</u>

Moment of inertia, I_x	= 5740272	mm^4	Moment of inertia, I_y	= 931472	mm^4
Elastic modulus, Z_x	= 76536	mm^3	Elastic modulus, Z_y	= 37258	mm^3
Plastic modulus, S_x	= 100332	mm^3	Plastic modulus, S_y	= 43932	mm^3
Radius of gyration, r_x	= 51.44	mm	Radius of gyration, r_y	= 20.32	mm
Area	= 2256	mm^2			

Case 1: Dowunward + Horizontal wind load



UDL, W3 = 11.55 x 2.28 x 45 / 245 / 2 + 11.55 x 0.05

= 3 kN/m

UDL, W4 = 11.55 x 0.05

= 0.58 kN/m

Point load P_1 = (11.55 x 4.56 x 45 / 245 / 2 + 11.55 x 0.05) x 2.213 / 2

= 5.99 kN

Point load P_2 = 9.78 kN (refer to item 4.4)

Reaction at support R€ = [0.58 x 5.424^2 / 2 + (3 - 0.58) x 0.844 x 2.622 + 5.424 x 9.78 - 4.344 x (5.99 + 9.78 + 0.844 x 3 + 4.58 x 0.58)] / 3.264

= 7.39 kN

R_B = 5.99 + 9.78 + 0.844 x 3 + 4.58 x 0.58 - 7.39

= 13.57 kN

Bending moment at middle = 0.58 x 2.375 x 1.609 + 9.78 x 2.797 + 3 x 0.422^2 / 2 - 13.57 * 1.632

= 7.69 kNm

Bending moment due to cantilever $= 9.78 \times 1.08 + 0.58 \times 1.08^2 / 2$
$= 10.9$ kNm (control)

Shear stress $= 13.59 \times 1000 / (2 \times 6 \times 150)$
$= 7.54$ N/mm^2
$< 0.6 \times 127 = 76.2$ N/mm^2 (low shear)

Moment capacity $= 1.2\ p_y\ Z$
$= 1.2 \times 220 \times 76536 / 10^6$
$= 20.206$ kNm (factored)
> 10.9 kNm O.K.

Check for lateral torsional buckling

Design strength, $p_y = 220$ N/mm^2 Parameter, $\varepsilon = (275 / p_y)^{0.5} = 1.118$
Modulus of Elasticity, $E = 200000$ N/mm^2

Overall width,	B	= 50	mm	Overall depth,	D	= 150	mm
Wall thickness,	t	= 6	mm	Area,	A	= 22.56	cm^2
Moment of inertia,	I_x	= 574.03	cm^4	Moment of inertia,	I_y	= 93.15	cm^4
Section modulus,	Z_x	= 76.54	cm^3	Section modulus,	Z_y	= 37.26	cm^3
Plastic modulus,	S_x	= 100.33	cm^3	Plastic modulus,	S_y	= 43.93	cm^3
Torsional constant,	J	= 256.24	cm^4	Radius of gyration,	r_y	= 2.03	cm
Effective length,	L_E	= 3300	mm				

Slenderness ratio, $\lambda = L_E / r_y = 162.6$
$\gamma_b = (1 - I_y / I_x) [1 - J / (2.6\ I_x)] = 0.6939$
Buckling index, $\varnothing_b = [S_x^{\ 2}\ \gamma_b / (A\ J)]^{0.5} = 1.0992$
Ratio, $\beta_w = 1$ for plastic section

Equivalent slenderness, $\lambda_{LT} = 2.25\ (\varnothing_b\ \lambda\ \beta_w)^{0.5} = 30.08$
$P_E = \pi^2\ E / \lambda_{LT}^{\ 2} = 2181.59$
$\alpha_{LT} = 7$
$\lambda_{L0} = 0.4\ (\pi^2\ E / p_y)^{0.5} = 37.89$
Perry factor, $\eta_{LT} = \alpha_{LT}\ (\lambda_{LT} - \lambda_{L0}) / 1000 = -0.0547 < 0$
$\varnothing_{LT} = [p_y + (\eta_{LT} + 1)\ P_E] / 2 = 1141.13$

Bending buckling strength, $p_b = P_E\ p_y / (\varnothing_{LT} + (\varnothing_{LT}^{\ 2} - P_E\ p_y)^{0.5})$
$= 234.36$ N/mm^2
> 220 N/mm^2

Deflection due to cantilever $= 9.78 / 1.5 \times 1000 \times 1080^3 / 3 / 200000 / 5740272$
$= 2.385$ (unfactored)
$< 2 \times 1080 / 250 = 8.64$ mm O.K.

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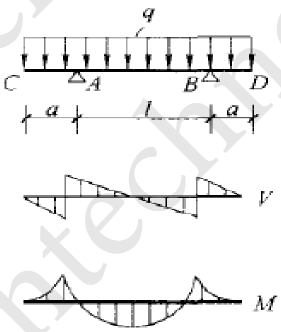
Case 2: Lateral wind load



$$R_A = R_B = q\left(a + \frac{l}{2}\right)$$

$$M_A = M_B = -\frac{qa^2}{2}$$

$$M_{max} = \frac{ql^2}{8}\left(1 - 4\frac{a^2}{l^2}\right)$$

$$w_C = w_D = \frac{qal^3}{24EI}\left(-1 + 6\frac{a^2}{l^2} + 3\frac{a^3}{l^3}\right)$$

$$w_{max} = \frac{ql^4}{384EI}\left(5 - 24\frac{a^2}{l^2}\right)$$

Design wind load = 6.2 kPa

UDL q = 1.5 x 6.2 x 0.15
= 1.4 kN/m (factored)

Reaction R€=R_B = 1.4 x 5.424 / 2
= 3.8 kN

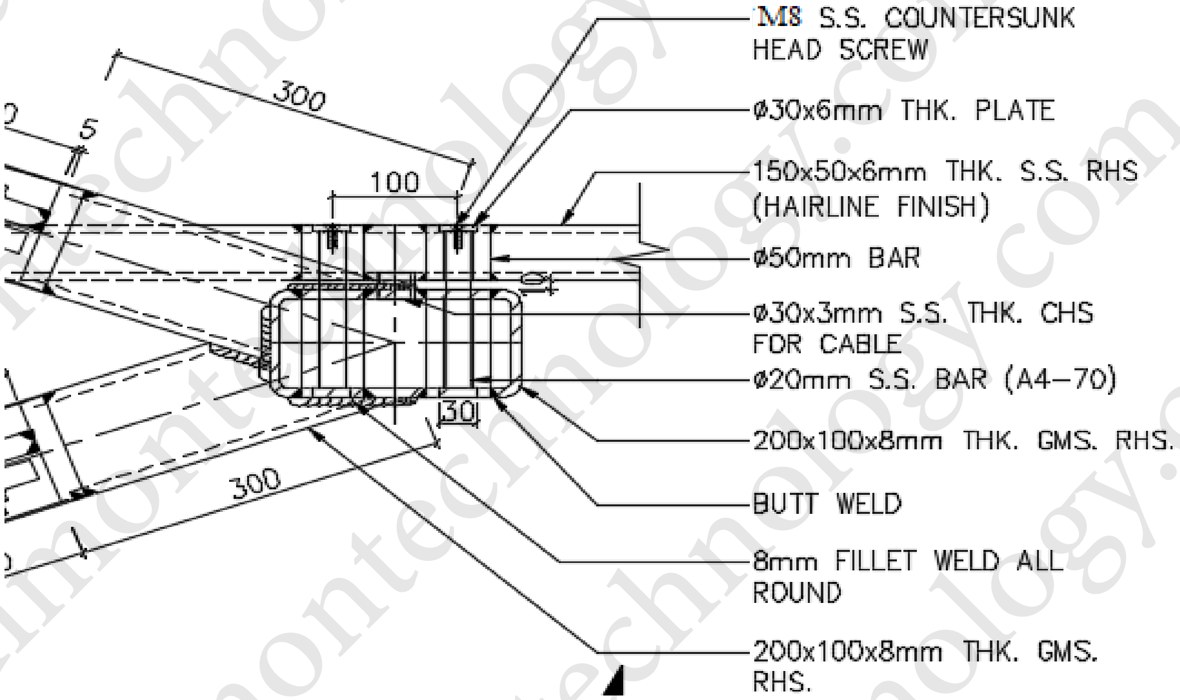
Max. Bending moment = (1 - 4 x 1.08^2 / 3.264^2) x 1.4 x 3.264^2 / 8
= 1.05 kNm

Shear stress = 3.8 x 1000 / (2 x 6 x 50)
= 6.33 N/mm^2
< 0.6 x 127 = 76.2 N/mm^2 (low shear)

Moment capacity = 1.2 p_y Z
= 1.2 x 220 x 37258 /10^6
= 9.836 kNm (factored)
> 1.05 kNm O.K.

Deflection = (5 - 24 x 1.08^2 / 3.264^2) x 1.4 / 1.5 x 3.264^4 / 384 / 200000 / 931472
= 3.51 mm (unfactored)
< 3264 / 250 = 13.06 mm O.K.

4.6 Check for Ø20mm s.s. bar (A4-70)



Vertival load	=	13.57	kN	(refer to item 4.5 case 1)
Lateral load	=	3.8	kN	(refer to item 4.5 case 2)
Eccentricity	=	35	mm	
Moment due to eccentricity on each bar	=	13.57 x 35 / 2 / 1000		
	=	0.237	kNm	
Shear load to each bar	=	13.57 / 2		
	=	6.79	kN	
Tensile load to each bar	=	3.8 / 2		
	=	1.9	kN	
Tensile area of Ø20 bar	=	314	mm^2	
Tensile strength of Ø20 bar	=	373	N/mm^2	
Shear strength of Ø20 bar	=	280	N/mm^2	
Tensile capacity of bolt	=	373 x 314 / 1000		
	=	117.12	kN	
	>	6.79	kN	O.K.

Shear capacity of bolt	= 280 x 314 / 1000		
	= 87.92	kN	
	> 1.9	kN	O.K.
Moment capacity	= 450 x 3.1416 x 10^3 / 32 / 10^6		
	= 0.442	kNm	
	> 0.237	kNm	
Combined tensile & shear loads	= 6.79 / 117.12 +1.9 / 87.92 + 0.237 / 0.442		
	= 0.62		
	< 1		O.K.

4.6.1 Check for M8 counrersunk head screw (A4-70)

Shear load to each bar	= 6.79	kN	
Tensile load to each bar	= 1.9	kN	
Tensile area of M8 screw	= 36.6	mm^2	
Tensile strength of M8screw	= 373	N/mm^2	
Shear strength of M8 screw	= 280	N/mm^2	
Tensile capacity of bolt	= 373 x 36.6 / 1000		
	= 13.65	kN	
	> 6.79	kN	O.K.
Shear capacity of bolt	= 280 x 36.6/ 1000		
	= 10.25	kN	
	> 1.9	kN	O.K.
Combined tensile & shear loads	= 6.79 / 13.65 +1.9 / 10.25		
	= 0.68		
	< 1.4		O.K.
Pull out capacity on connected part,	= 8 x 3.1416 x 15 / 2 x 127 / 1000		
	= 23.94	kN	
	> 6.79	kN	O.K.

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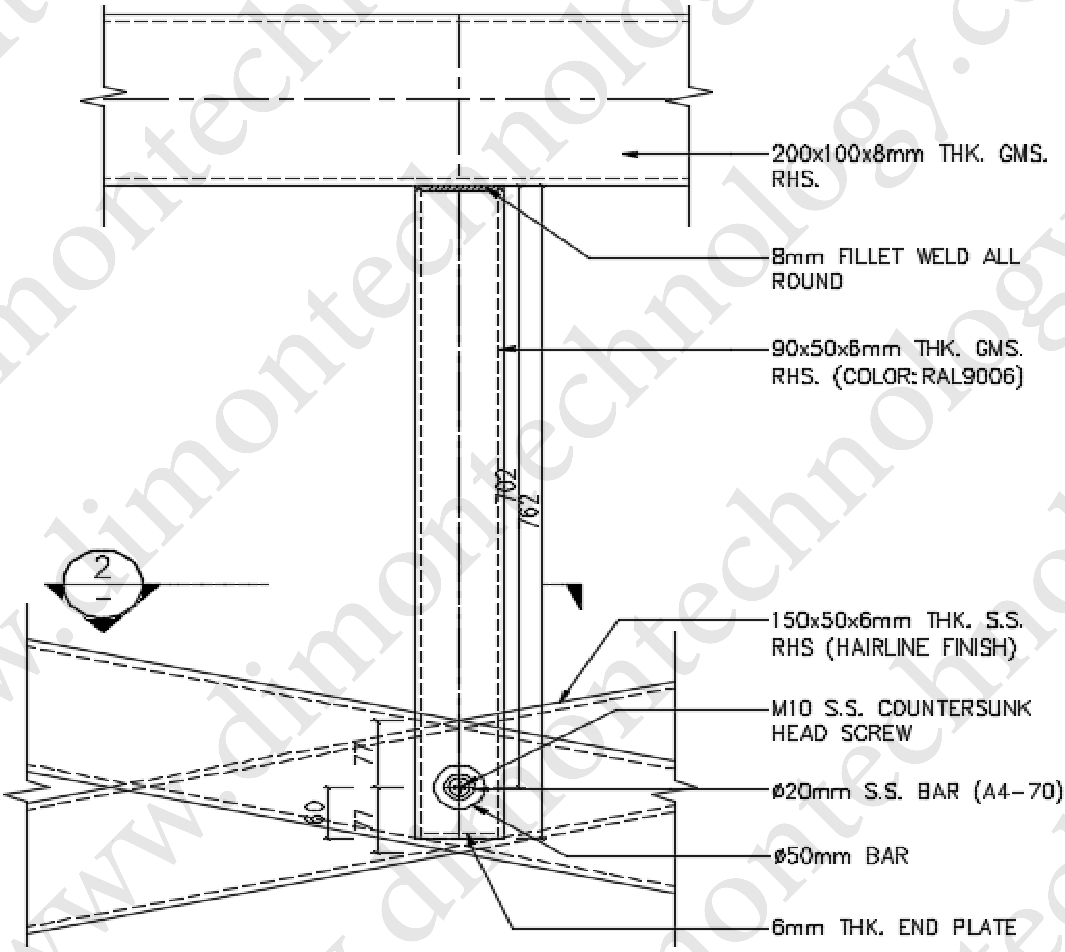
4.7 Check for 90x50x6mm gms. RHS (Q345)

Properties of 90x50x6mm RHS

Moment of inertia, I_x	= 145	cm^4	Moment of inertia, I_y	= 55.4	cm^4
Elastic modulus, Z_x	= 32	cm^3	Elastic modulus, Z_y	= 22.1	cm^3
Plastic modulus, S_x	= 41.6	cm^3	Plastic modulus, S_y	= 27	cm^3
Radius of gyration, r_x	= 3.15	cm	Radius of gyration, r_y	= 1.92	cm
Area	= 15	cm^2			



Csse 1: Lateral wind load

Lateral wind load = 2 x 3.8 (refer to item 4.5 case 2)
= 7.6 kN

Length of cantilever = 702 mm

Bending moment due to cantilever = 7.6 x 702 / 1000
= 5.34 kNm (factored)

Shear stress = 7.6 x 1000 / (2 x 6 x 50)
= 15.83 N/mm^2
< 0.6 x 180 = 108 N/mm^2 (low shear)

Moment capacity = min(1.2 p_y Z_y, p_y S_y)
= (1.2 x 310 x 22.1/1000 , 310 x 27 / 1000)
= 8.221 kNm (factored)
> 5.34 kNm O.K.

Deflection due to cantilever = 7.6 / 1.5 x 1000 x 702^3 / 3 / 206000 / 554000
= 5.12 mm (unfactored)
< 2 x 702 / 250 = 5.62 mm O.K.

Csse 2: Horizontal wind load

Horizonta windl load = 2 x 1.5 x 6.2 x sin13° / 11.55 x 7.39 (refer to item 4.5 case 1)
= 2.68 kN

Length of cantilever = 702 mm

Bending moment due to cantilever = 2.68 x 702 / 1000
= 1.88 kNm (factored)

Shear stress = 2.68 x 1000 / (2 x 6 x 40)
= 5.58 N/mm^2
< 0.6 x 180 = 108 N/mm^2 (low shear)

Moment capacity = min(1.2 p_y Z_y, p_y S_y)
= (1.2 x 310 x 23/1000 , 310 x 30 / 1000)
= 8.556 kNm (factored)
> 1.88 kNm O.K.

Check for lateral torsional buckling

Design strength,	p_y	=	310	N/mm^2	Parameter,	$\varepsilon = (275 / p_y)^{0.5}$	=	0.9419		
Modulus of Elasticity,	E	=	206000	N/mm^2						
Overall width,	B	=	50	mm	Overall depth,	D	=	90	mm
Wall thickness,	t	=	6	mm	Area,	A	=	15	cm^2
Moment of inertia,	I_x	=	145	cm^4	Moment of inertia,	I_y	=	55.4	cm^4
Section modulus,	Z_x	=	32	cm^3	Section modulus,	Z_y	=	22.1	cm^3
Plastic modulus,	S_x	=	41.6	cm^3	Plastic modulus,	S_y	=	27	cm^3
Torsional constant,	J	=	133	cm^4	Radius of gyration,	r_y	=	1.92	cm
Effective length,	L_E	=	702	mm					

Slenderness ratio, $\lambda = L_E / r_y$ = 36.6

$\gamma_b = (1 - I_y / I_x)\,[1 - J / (2.6\, I_x)]$ = 0.3999

Buckling index, $ø_b = [S_x^{\,2}\, \gamma_b / (A\, J)]^{0.5}$ = 0.589

Ratio, $\beta_w = 1$ for plastic section

Equivalent slenderness, $\lambda_{LT} = 2.25\,(ø_b\, \lambda\, \beta_w)^{0.5}$ = 10.45

$P_E = \pi^2\, E / \lambda_{LT}^{\,2}$ = 18618.06

$\alpha_{LT} = 7$

$\lambda_{L0} = 0.4\,(\pi^2\, E / p_y)^{0.5}$ = 32.39

Perry factor, $\eta_{LT} = \alpha_{LT}\,(\lambda_{LT} - \lambda_{L0}) / 1000$ = -0.1536 < 0

$ø_{LT} = [p_y + (\eta_{LT} + 1)\, P_E] / 2$ = 8034.16

Bending buckling strength, $p_b = P_E\, p_y / (ø_{LT} + (ø_{LT}^{\,2} - P_E\, p_y)^{0.5})$

= 367.6 N/mm^2

> 310 N/mm^2

Deflection due to cantilever = $2.68/ 1.5 \times 1000 \times 702^3 / 3 / 206000 / 285000$

= 1.105 mm (unfactored)

< 2 x 702 / 250 = 5.62 mm O.K.

4.7.1 Check for 6mm fillet weld all round connection to 200x100x8mm RHS.

Case 1: Vertical load+Horizontal wind load

Vertical load = 2 x 7.39 (refer to item 4.5 case 1)

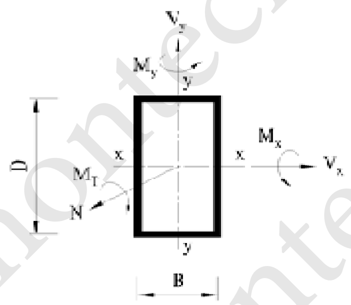
= 14.78 kN

Lateral wind load = 7.6 kN (refer to item 4.6 case 1)

Moment M_y = 5.34 kNm (refer to item 4.6 case 1)

<u>Properties for unit throat thickness of fillet weld (effective length)</u>

Breadth,	B	= 50			mm
Height,	D	= 90			mm
Area,	A	= 2 (B + D)	=	280	mm^2
Moment of inertia,	I_x	= $B\,D^2 / 2 + D^3 / 6$	=	324000	mm^4
Moment of inertia,	I_y	= $B^2\,D / 2 + B^3 / 6$	=	133333	mm^4
Polar moment of inertia,	J	= $I_x + I_y$	=	457333	mm^4



Shear load,	V_x = 7.6	kN	Moment,	M_x	=	0	kNm
Shear load,	V_y = 0	kN	Moment,	M_y	=	5.34	kNm
Tensile load,	N = 14.78	kN	Torsional moment,	M_T	=	0	kNm

Leg length of fillet weld, t = 8 mm

Shear stress,	τ_x	= $V_x / (0.7\ t\ A) + M_T\ (D / 2) / (0.7\ t\ J)$	=	4.85	N/mm^2
Shear stress,	τ_y	= $V_y / (0.7\ t\ A) + M_T\ (B / 2) / (0.7\ t\ J)$	=	0	N/mm^2
Tensile stress,	σ	= $N / (0.7\ t\ A)$	=	9.43	N/mm^2
Tensile stress,	σ_x	= $M_x\ D / (2\ I_x) / (0.7\ t)$	=	0	N/mm^2
Tensile stress,	σ_y	= $M_y\ B / (2\ I_y) / (0.7\ t)$	=	178.8	N/mm^2

Resultant , $f_w = [\tau_x^2 + \tau_y^2 + (\sigma + \sigma_x + \sigma_y)^2]^{0.5}$

= 188.29 N/mm^2

$\leq$ 200 N/mm^2 O.K.

Case 2: Lateral wind load

Vertical load = 2 x 7.39 (refer to item 4.5 case 1)

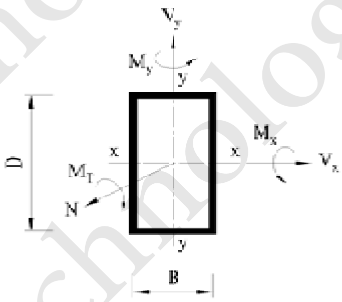
= 14.78 kN

Horizontal wind load = 2.68 kN (refer to item 4.6 case 2)

Moment M_x = 1.88 kNm (refer to item 4.6 case 2)

<u>Properties for unit throat thickness of fillet weld (effective length)</u>

Breadth,	B	= 50			mm
Height,	D	= 90			mm
Area,	A	= $2\ (B + D)$	=	280	mm^2
Moment of inertia,	I_x	= $B\ D^2 / 2 + D^3 / 6$	=	324000	mm^4
Moment of inertia,	I_y	= $B^2\ D / 2 + B^3 / 6$	=	133333	mm^4
Polar moment of inertia,	J	= $I_x + I_y$	=	457333	mm^4



Shear load,	V_x	= 0	kN	Moment,	M_x	=	1.88	kNm
Shear load,	V_y	= 2.68	kN	Moment,	M_y	=	0	kNm
Tensile load,	N	= 14.78	kN	Torsional moment,	M_T	=	0	kNm

Leg length of fillet weld, t = 8 mm

Shear stress,	τ_x	= $V_x / (0.7\ t\ A) + M_T\ (D / 2) / (0.7\ t\ J)$	=	0	N/mm^2
Shear stress,	τ_y	= $V_y / (0.7\ t\ A) + M_T\ (B / 2) / (0.7\ t\ J)$	=	1.71	N/mm^2
Tensile stress,	σ	= $N / (0.7\ t\ A)$	=	9.43	N/mm^2
Tensile stress,	σ_x	= $M_x\ D / (2\ I_x) / (0.7\ t)$	=	46.63	N/mm^2
Tensile stress,	σ_y	= $M_y\ B / (2\ I_y) / (0.7\ t)$	=	0	N/mm^2

Resultant , $f_w = [\tau_x^2 + \tau_y^2 + (\sigma + \sigma_x + \sigma_y)^2]^{0.5}$

= 56.09 N/mm^2

$\leq$ 200 N/mm^2 O.K.

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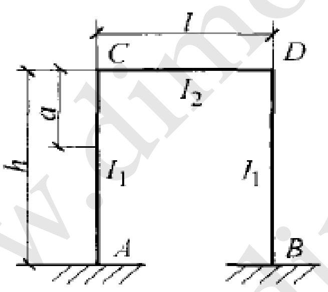
5 Check for main steel frame

5.1 Check for 200x100x8mm gms. RHS

Properties of 200x100x8mm RHS

Moment of inertia, I_x	=	2234	cm^4	Moment of inertia, I_y	=	739	cm^4
Elastic modulus, Z_x	=	223	cm^3	Elastic modulus, Z_y	=	148	cm^3
Plastic modulus, S_x	=	282	cm^3	Plastic modulus, S_y	=	172	cm^3
Radius of gyration, r_x	=	7.12	cm	Radius of gyration, r_y	=	4.08	cm
Area	=	44.8	cm^2				

Vertical load	=	14.78	kN	(factored)
Horizontal load	=	2.68	kN	(factored)
Lateral load	=	7.6	kN	(factored)
Moment	=	1.88	kNm	(factored)
Torsional moment	=	5.34	kNm	(factored)



$$\lambda = \frac{a}{h}$$

$$\mu = \frac{I_2 h}{I_1 l}$$

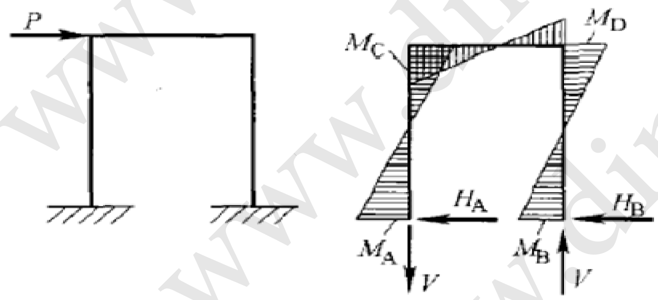
$$K = \frac{1}{2+\mu}$$

$$L = \frac{1}{1+6\mu}$$

h = 3900 mm
l = 5424 mm

μ = 0.719
K = 0.3678
L = 0.1882

case 1:



$$M_C = M_D = 3\mu L \frac{Ph}{2}$$

$$M_A = M_B = (1 - 3\mu L)\frac{Ph}{2}$$

$$H_A = H_B = \frac{P}{2}$$

$$V = 3\mu L \frac{Ph}{l}$$

P = 2.68 kN

Reaction V€ = V_B
= 3μLPh/l
= 0.782 kN

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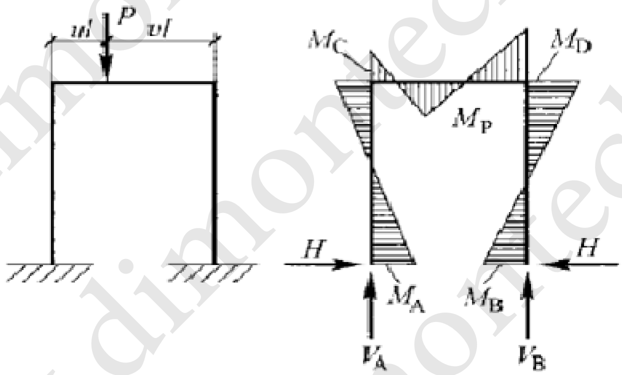
H€ = H_B
= P/2
= 1.34 kN

Moment, M€ = M_B
= (1-3μL)Ph/2
= 3.105 kNm

M_C = M_D
= 3μLPh/2
= 2.121 kNm

case 2:



$$M_C = [0.5(v-u)L + K]Puvl$$
$$M_D = [K - 0.5(v-u)L]Puvl$$
$$M_A = [K - (v-u)L]\frac{Puvl}{2}$$
$$M_B = [K + (v-u)L]\frac{Puvl}{2}$$
$$M_P = [1 - K - 0.5(v-u)^2 L]Puvl$$
$$H = 1.5K\frac{Puvl}{h}$$
$$V_A = [1 + u(v-u)L]Pv$$
$$V_B = [1 - u(v-u)L]Pu$$

P = 14.78 kN

Reaction V€ = [1+u(v-u)L]vP
= 7.39 kN

V_B = [1-u(v-u)L]uP
= 7.39 kN

H€ = H_B
= 1.5KPuvl/h
= 2.835 kN

Moment, M€ = [K-(v-u)L]Puvl/2
= 3.686 kNm

M_B = [K+(v-u)L]Puvl/2
= 3.686 kNm

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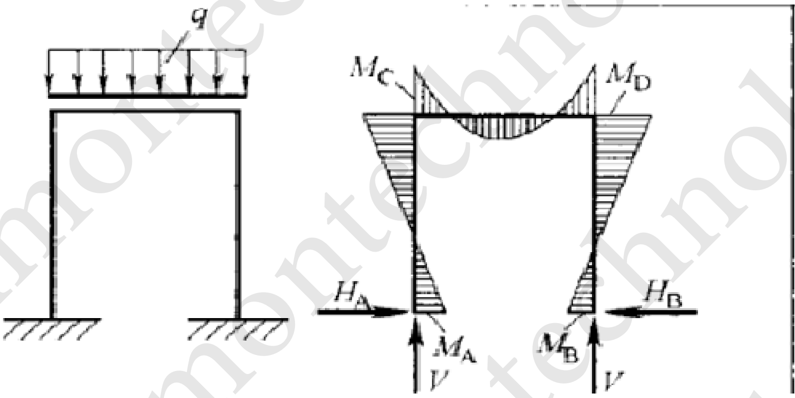
M_C = [0.5(v-u)L+K]Puvl
= 7.371 kNm

M_D = [K-0.5(v-u)L]Puvl
= 7.371 kNm

M_P = [1-K-$(v-u)^2$L]Puvl
= 12.67 kNm

case 3:



$$M_C = M_D = K\frac{ql^2}{6}$$

$$M_A = M_B = K\frac{ql^2}{12}$$

$$H_A = H_B = K\frac{ql^2}{4h}$$

$$V = \frac{ql}{2}$$

q = 1.5 x 6.2 x 0.1
= 0.93 kN/m

Reaction V€ = V_B
= ql/2
= 2.522 kN

H€ = H_B
= $Kql^2/4h$
= 0.645 kN

Moment, M€ = M_B
= $Kql^2/12$
= 0.839 kNm

M_C = M_D
= $Kql^2/6$
= 1.677 kNm

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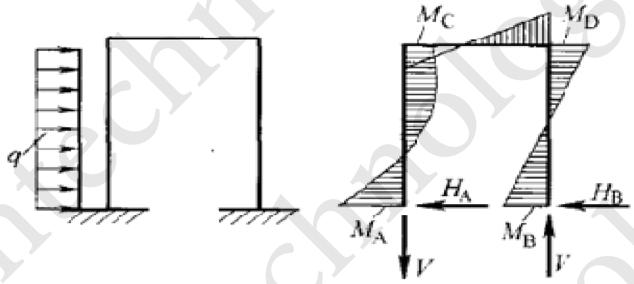
case 4:



$$M_C = \mu(12L - K)\frac{qh^2}{24}$$

$$M_D = \mu(12L + K)\frac{qh^2}{24}$$

$$M_A = [(39 + 19\mu)K - 12\mu L - 12]\frac{qh^2}{24}$$

$$M_B = [(9 + 5\mu)K - 12\mu L]\frac{qh^2}{24}$$

$$H_A = (39 + 18\mu)K\frac{qh}{24}$$

$$H_B = (9 + 6\mu)K\frac{qh}{24}$$

$$V = \mu L\frac{qh^2}{l}$$

q = 1.5 x 6.2 x 0.1
= 0.93 kN/m

Reaction

V€ = V_B
= $\mu Lqh^2/l$
= 0.353 kN

H€ = (39+18μ)Kqh/24
= 2.887 kN

Moment,

H_B = (9+6μ)Kqh/24
= 0.74 kN

M€ = $[(39+19\mu)K-12\mu L-12]qh^2/24$
= 3.386 kNm

M_B = $[(9+5\mu)K-12\mu L]qh^2/24$
= 1.773 kNm

M_C = $\mu(12L - K)qh^2/24$
= 0.801 kNm

M_D = $\mu(12L + K)qh^2/24$
= 1.113 kNm

Conclusions As a resul, the acting force on member:

Shear laod

V_x = 7.6 / 2 + 1.5 x 6.2 x (3.9 x2 + 5.424) / 2 x 0.2
= 16.1 kN

V_y = 0.782 +7.39 + 2.522 + 0.353
= 11.597 kN

T = 1.34 + 2.835 + 0.266 + .645 +2.887+13.57 + 1.88 / 5.424
= 21.624 kN

Moment M_x = 3.105 + 12.67 + 1.677 + .386
= 17.838 kNm

M_y = 5.34 / 2 +1.5 x 6.2 x 5.424 / 2 x 0.2 x 3.9 + 1.5 x 6.2 x 0.2 x 3.9^2 / 2
= 36.488 kNm

Shear stress = 16.1 x 1000 / (2 x 8 x 200) + 11.597 x 1000 / (2 x 8 x 100)
= 12.28 N/mm^2
< 0.6 x 180 = 108 N/mm^2 (low shear)

Moment capacity = min(1.2 p_y Z_x, p_y S_x)
= (1.2 x 310 x 223/1000 , 310 x 282 / 1000)
= 82.956 kNm (factored)
> 17.838 kNm O.K.

Moment capacity = min(1.2 p_y Z_y, p_y S_y)
= (1.2 x 310 x 148/1000 , 310 x 172 / 1000)
= 53.32 kNm (factored)
> 36.488 kNm O.K.

Design strength, p_y = 310 N/mm^2 Robertson constant, α = 3.5
Modulus of Elasticity, E = 206000 N/mm^2 Radius, of gyration, r_y = 40.8 mm
Effective length, L_E = 3900 mm

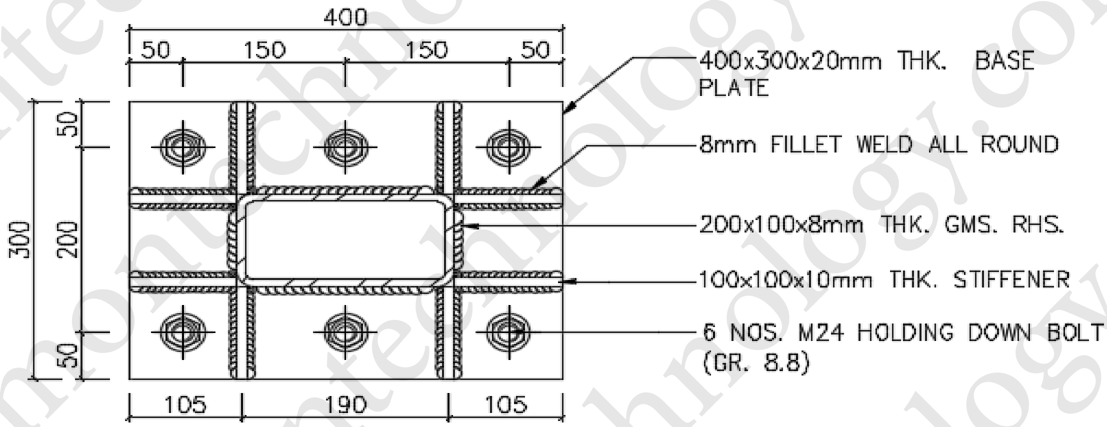
Slenderness, $\lambda = L_E / r_y$ = 95.5882
Limit slenderness, $\lambda_0 = 0.2 (\pi^2 E / p_y)^{0.5}$ = 16.1969
Perry factor, $\eta = \alpha (\lambda - \lambda_0) / 1000$ = 0.2779 $\geq$ 0 O.K.
$P_E = \pi^2 E / \lambda^2$ = 222.5145
$ø_c = [p_y + (\eta + 1) P_E] / 2$ = 297.1756

Compressive buckling strength, $p_c = P_E p_y / [ø_c + (ø_c^2 - P_E p_y)^{0.5}]$
= 158.13 N/mm^2

Compressive resistance = 158.13 x 4480 / 1000
= 708.42 kN
> 21.624 kN O.K.

Combined tensile & shear loads = 17.838 / 82.956 +36.488 / 53.32 +21.624 / 708.42
= 0.93
< 1 O.K.

5.2 Check for 8mm fillet weld all round



Shear load	V_x	=	16.1	kN	(refer to item 5.1)
	V_y	=	11.597	kN	(refer to item 5.1)
Tensile load	T	=	21.624	kN	(refer to item 5.1)
Moment	M_x	=	17.838	kN	(refer to item 5.1)
	M_y	=	36.488	kN	(refer to item 5.1)

<u>Properties for unit throat thickness of fillet weld (effective length)</u>

Breadth, B = 300 mm

Height, D = 400 mm

Area, A = 16 x 100 +2 x (100 + 200) = 2200 mm^2

Moment of inertia, I_x = $B D^2 / 2 + D^3 / 6 + 8 \times 100 \times 150^2 + 4 \times 100 \times 100^2 + 4 \times 100 \times 90^2$
= 56699067 mm^4

Moment of inertia, I_y = $B^2 D / 2 + B^3 / 6 + 8 \times 100 \times 100^2 + 4 \times 100 \times 50^2 + 4 \times 100 \times 40^2$
= 32140000 mm^4

Polar moment of inertia, J = $I_x + I_y$ = 88839067 mm^4

Shear load,	V_x	=	16.1	kN	Moment,	M_x	=	18.708	kNm
Shear load,	V_y	=	11.597	kN	Moment,	M_y	=	36.488	kNm
Tensile load,	N	=	21.624	kN	Torsional moment,	M_T	=	0	kNm

Leg length of fillet weld, t = 8 mm

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Shear stress, $\tau_x = V_x / (0.7\ t\ A) + M_T\ (D / 2) / (0.7\ t\ J)$ = 1.31 N/mm^2

Shear stress, $\tau_y = V_y / (0.7\ t\ A) + M_T\ (B / 2) / (0.7\ t\ J)$ = 0.94 N/mm^2

Tensile stress, $\sigma = N / (0.7\ t\ A)$ = 1.76 N/mm^2

Tensile stress, $\sigma_x = M_x\ D / (2\ I_x) / (0.7\ t)$ = 11.78 N/mm^2

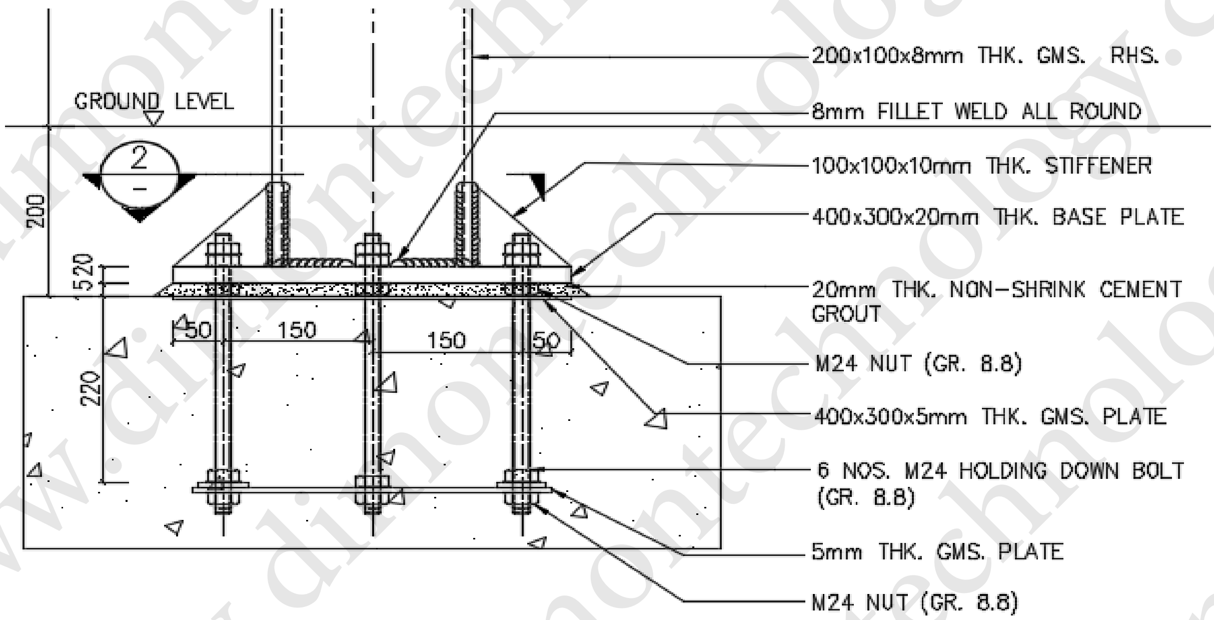
Tensile stress, $\sigma_y = M_y\ B / (2\ I_y) / (0.7\ t)$ = 30.41 N/mm^2

Resultant , $f_w = [\tau_x^{\ 2} + \tau_y^{\ 2} + (\sigma + \sigma_x + \sigma_y)^2]^{0.5}$

= 43.98 N/mm^2

$\leq$ 200 N/mm^2 O.K.

5.3 Check for 6 Nos. M24 holding bolt(gr 8.8)



Shear load V_x = 16.1 kN (refer to item 5.1)

V_y = 11.597 kN (refer to item 5.1)

Tensile load T = 21.624 kN (refer to item 5.1)

Moment M_x = 17.838 kN (refer to item 5.1)

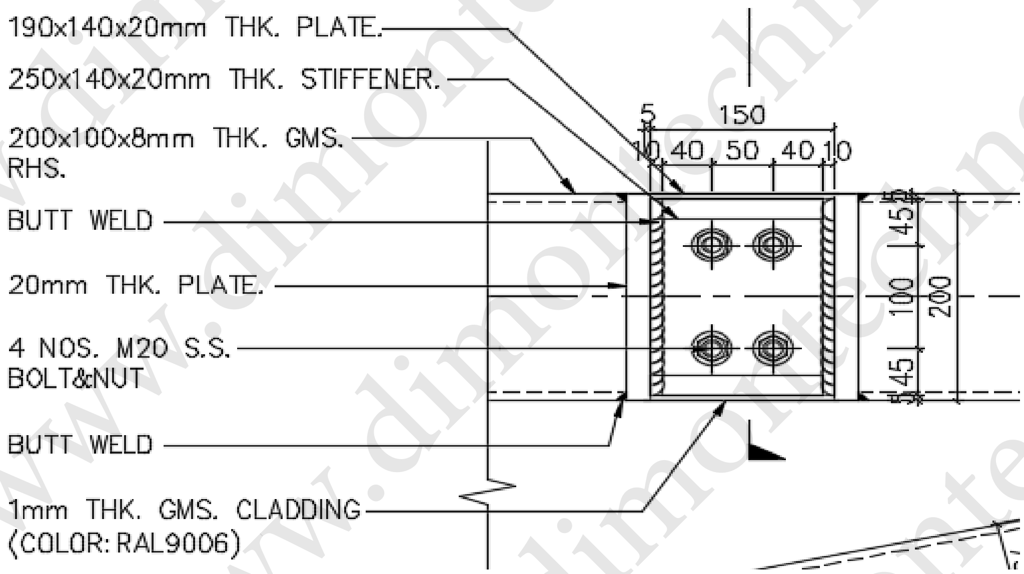
M_y = 36.488 kN (refer to item 5.1)

Shear load to each bar $= (16.1^2 + 11.597^2)^{0.5} / 6$

= 3.307 kN

Tensile load to each bar	=	21.543 / 6 + 36.488 / 0.2 / 3 + 17.838x 1000 x 300 / (2 x 150^2 + 2 x 300^2)		
	=	88.2	kN	
Tensile area of M24 bolt	=	352	mm^2	
Tensile strength of M24 bolt	=	450	N/mm^2	
Shear strength of M24 bolt	=	375	N/mm^2	
Tensile capacity of bolt	=	3450 x 352 / 1000		
	=	158.4	kN	
	>	88.2	kN	O.K.
Shear capacity of bolt	=	375 x 352 / 1000		
	=	132	kN	
	>	3.307	kN	O.K.
Combined tensile & shear loads	=	88.2 / 158.4 +3.307 / 132		
	=	0.58		
	<	1.4		O.K.

5.4 Check for connection part at steel frame

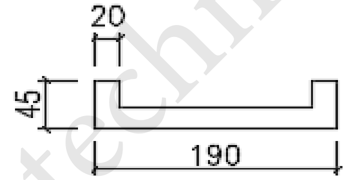


5.4.1 Check for 20mm plate

Case 1: Vertical wind load + Horizontal load

Shear laod	V_y	=	0.782 +7.39 + 2.522 + 0.353 + 1.88 /5.424 (refer to item 5.1)	
		=	11.394	kN

Moment M_x = 2.121 + 7.371 + 1.677 +1.113 (refer to item 5.1 M_C or M_D)
= 12.282 kNm

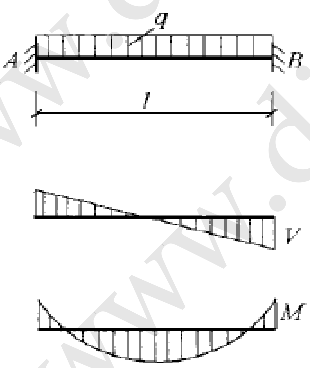


Area:	4800.0000
Perimeter:	520.0000
Bounding box:	X: −95.0000 −− 95.0000
	Y: −14.6875 −− 30.3125
Centiord:	X: 0.0000
	Y: 0.0000
Elastic of modulus:	X: 579531.2500
	Y: 18690000.0000
Product of muduls:	XY: 0.0000
Gyration of radius:	X: 10.9880
	Y: 62.3999

Shear stress = 11.394 x 1000 / （20 x 190）
= 3 N/mm^2
< 0.6 x 180 = 108 N/mm^2 (low shear)

Moment capacity M_{Cx} = 1.2 x 310 x 18690000/ 95 / 10^6
= 73.18611 kNm (factored)
> 12.282 kNm O.K.

Case 2: Lateral wind load



$$R_A = R_B = \frac{ql}{2}$$

$$M_A = M_B = -\frac{ql^2}{12}$$

$$M_x = -\frac{ql^2}{12}\left(1 - \frac{6x}{l} + \frac{6x^2}{l^2}\right)$$

$$M_{max} = \frac{ql^2}{24}\left(x = \frac{l}{2}处\right)$$

反弯点在 $x = 0.211l$ 及 $x = 0.789l$ 处

$$w_x = \frac{ql^2x^2}{24EI}\left(1 - \frac{x}{l}\right)^2$$

$$w_{max} = \frac{ql^4}{384EI}$$

Shear laod V_x = 7.6 / 2 + 1.5 x 6.2 x 5.424 x 0.2 / 2 (refer to item 5.1)
= 8.84 kN

Moment M_y = 5.34 / 2 +1.5 x 6.2 x 0.2 x 5.424^2 x [1- 6 x 0.3 / 5.424 + 6 x (0.3 / 5.424)^2)]/12
= 5.8 kNm (x=0.3m)

Shear stress = 8.84 x 1000 / (2 x 20 x 45）
= 4.91 N/mm^2
< 0.6 x 180 = 108 N/mm^2 (low shear)

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Moment capacity M_{Cx} = 12 x 310 x 579531/ 30.31 / 10^6
= 7.11 kNm (factored)
> 5.8 kNm O.K.

5.4.1 Check for M20 s.s bolt

Case 1: Vertical wind load + Horizontal load

Shear laod V_y = 0.782 +7.39 + 2.522 + 0.353 (refer to item 5.1)
= 11.215 kN

Moment M_x = 2.121 + 7.371 + 1.677 +1.113 (refer to item 5.1 M_C or M_D)
= 12.282 kNm

Shear load to each bar = 11.597 / 4 + 13.152 / 0.1 / 2
= 64.309 kN

Tensile area of M20 bolt = 245 mm^2
Tensile strength of M20 bolt = 373 N/mm^2
Shear strength of M20 bolt = 280 N/mm^2

Shear capacity of bolt = 375 x 352 / 1000
= 68.6 kN
> 64.309 kN O.K.

Case 2: Lateral wind load

Shear laod V_x = 7.6 / 2 + 1.5 x 6.2 x 5.424 x 0.2 / 2 (refer to item 5.1)
= 8.84 kN

Moment M_y = 5.34 / 2 +1.5 x 6.2 x 0.2 x 5.424^2 x [1- 6 x 0.3 / 5.424 + 6 x $(0.3 / 5.424)^2$)]/12
= 5.8 kNm (x=0.3m)

Shear load to each bar = 11.597 / 4 + 13.152 / 0.1 / 2
= 73.579 kN

Tensile load to each bar = 8.84 / 4 + 7.23 / 0.05 / 2
= 58 kN

Tensile area of M20 bolt = 245 mm^2
Tensile strength of M20 bolt = 373 N/mm^2
Shear strength of M20 bolt = 280 N/mm^2

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Tensile capacity of bolt	=	3450 x 352 / 1000		
	=	91.39	kN	
	>	58	kN	O.K.