DIMON TECHNOLOGY		Sheet No.	1
		Prepared by	Y.S.Zhang
Project		Date	16.07.2020
Title	CHLOE @ SKY628	Revision	-

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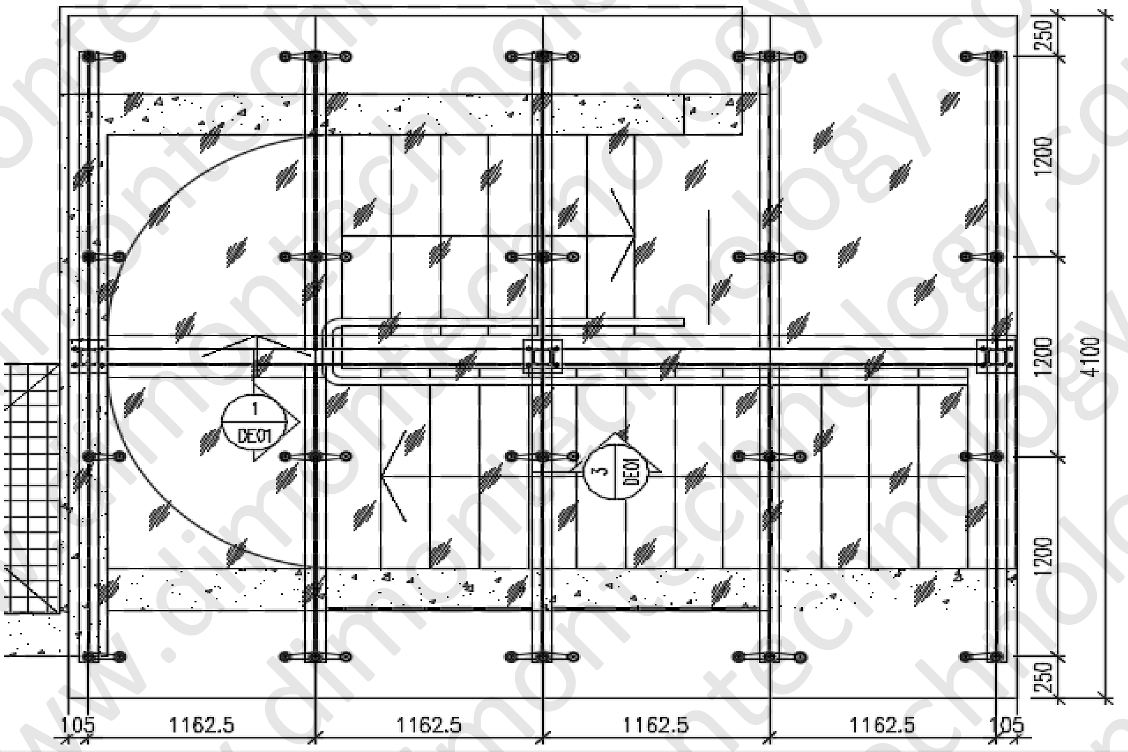
1. Introduction

The skylights are made of 12+12mm thick laminated tempered glass fixed to s.s. RHS by means of stainless steel spider system. The objective of this calculation is to check the design of skylight to be safe against the dead load, live load and wind load.

Load path

Wind load to skylight → glass panels → s.s. spiders → steel RHS frame → gms embeds → r.c. structures



2. Design Code, Design Data

2.1 Design Code

a) Hong Kong Building (Construction) Regulation 1990 (Amendment 2011).
b) Code of Practice on Wind Effect 2008, Macau.
c) Code of Practice for the Structural Use of Steel 2011, Hong Kong
d) Code of Practice for the Structural Use of Concrete 2004, Hong Kong
e) Code of Practice for Dead and imposed Loads 2011, Hong Kong
f) Code of Practice for the Structural Use of Steel GB 50017-2003
g) Load Code of the design of buliding structure GB 50009-2012

2.2 Design Data

2.2.1 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)	510
Modulus of elasticity	(N/mm^2)	200000
Design strength	(N/mm^2)	220
Design stress of fillet weld	(N/mm^2)	220

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

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

2.2.3 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

stainless steel grade		A4 / 316
design bearing strength on connected part	(N/mm^2)	479

2.2.4 Concrete

concrete strength,	f_{cu}	(N/mm^2)	45	
anchorage bond strength, (= $\beta\ f_{cu}^{0.5}$)	f_{bu}	(N/mm^2)	1.878	β = 0.28 for cast-in threaded rod
design concrete shear stress,	v_c	(N/mm^2)	0.4	
compressive strength, (= 0.6 f_{cu})		(N/mm^2)	27	

2.2.5 The date refer to SAP2000 program.

3. Design Load

3.1 Wind load

Wind pressure, q_z = 1.84 kPa (height above ground level ≤10m)

Pressure coefficient, = 2 (open frame)

Design wind load = 2.12 x 2.0
= 3.68 kPa

3.2 Live load

Live load = 0.75 kN/m^2

3.3 Dead Load

Weight of glass	= (12 + 12) x 26.5 / 1000	=	0.636	kPa
Others,	=	=	0.064	kPa
	Total	=	0.7	kPa

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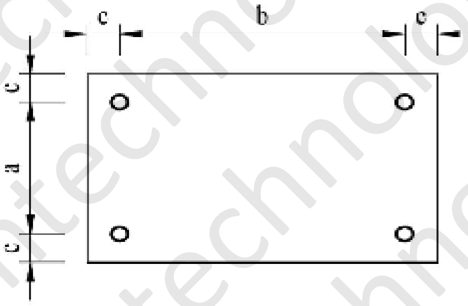
4. Check for laminated tempered glass

4.1 Check for 12mm+1.56PVB+12mm clear laminated tempered glass



Max. bending stress, $\sigma_c = \alpha\, q\, b^2 / t^2$

Max. deflection, $\delta_c = \beta\, q\, b^4 / (E_g\, t^3)$

a/b	β			α		
	b/c=10	b/c=15	b/c=20	b/c=10	b/c=15	b/c=20
1	0.2547	0.2668	0.273	0.8194	0.8719	0.8719
0.95	0.2302	0.2414	0.2472	0.8087	0.843	0.858
0.9	0.2102	0.2206	0.2259	0.7984	0.8307	0.8447
0.85	0.1934	0.203	0.2079	0.7886	0.819	0.832
0.8	0.1801	0.189	0.1935	0.7792	0.8079	0.8199
0.75	0.1693	0.1776	0.1816	0.7703	0.7974	0.8085
0.7	0.1611	0.1688	0.1724	0.762	0.7876	0.7979
0.65	0.1549	0.1619	0.1653	0.7543	0.7786	0.7881
0.6	0.1504	0.157	0.1601	0.7473	0.7703	0.7792
0.55	0.1513	0.1567	0.1593	0.741	0.7629	0.7712
0.5	0.1512	0.1565	0.1588	0.7355	0.7564	0.7641

Glass density, ρ = 2650 kg/m^3

Nominal thickness of glass pane 1 = 12 mm

Nominal thickness of glass pane 2 = 12 mm

Glass type for pane 1 & 2 : tempered

Load duration, self weight : long term

Min. thickness of glass pane 1, t_1 = 11.91 mm

Min. thickness of glass pane 2, t_2 = 11.91 mm

Reduction factor, γ_d = 0.66

<u>Downward load</u>

Basic wind pressure, q_z = 1.84 kN/m^2

Topography factor, S_a = 1

Pressure coefficient, C_P = 2

Design wind pressure, $q_{dn} = q_z\, S_a\, C_P$ = 3.68 kN/m^2

Live load, q_k = 0.75 kN/m^2

Glass weight = $\rho\, \Sigma\, (t / \gamma_d)$, $g_{k,dn}$ = 0.96 kN/m^2

<u>Upward load</u>

Basic wind pressure, q_z = 1.84 kN/m^2

Topography factor, S_a = 1

Pressure coefficient, C_P = -2

Design wind pressure, $q_{up} = q_z\, S_a\, C_P$ = -3.68 kN/m^2

Glass weight = $\rho\, \Sigma\, t$, $g_{k,up}$ = 0.63 kN/m^2

<u>Load combinations (downward)</u>

$1.0\, w_{dn} + 1.0\, q_k$ = 4.43 kN/m^2

$1.3\, g_{k,dn} + 0.9\, w_{dn} + 1.05 q_k$ = 5.35 kN/m^2

$1.2\, g_{k,dn} + 1.5\, w_{dn} + 1.05 q_k$ = 7.46 kN/m^2

$1.2 g_{k,dn} + 0.9\, w_{dn} + 1.5 q_k$ = 5.59 kN/m^2

<u>Load combinations (upward)</u>

$1.0\, w_{up} + 1.0\, g_{k,up}$ = -3.05 kN/m^2

$1.5\, w_{up} + 1.0\, g_{k,up}$ = -4.89 kN/m^2

Serviceability load, $w_{dns} = q_{dn} + q_k$ = 4.43 kN/m^2

Serviceability load, $w_{ups} = q_{up} + g_s$ = -3.05 kN/m^2

Critical combination, w_c = 7.46 kN/m^2 downward

Critical serviceability, w_{cs} = 4.43 kN/m^2 downward (for deflection checking)

Glass type for pane 1 & 2 : tempered

Load duration : short term

Ultimate design strength, p_y = 80 N/mm^2

Reduction factor, γ_d = 1

Surface treatment : clear

Reduction factor, γ_s = 1

Material factor, γ_m = 1

Ultimate resistance strength, $R_{ult} = p_y \gamma_d \gamma_s / \gamma_m$ = 80 N/mm^2

Critical load shared equally to each pane, w_1 = 7.46 / 2 = 3.73 kN/m^2

Critical serviceability load shared equally to each pane, w_{s1} = 4.43 / 2 = 2.215 kN/m^2

The glass panel is simply supported on 4 points.

Longer side of glass pane,	b	=	1200	mm
Shorter side of glass pane,	a	=	918	mm
Edge distance,	c	=	125	mm
Modulus of Elasticity,	E_g	=	70000	N/mm^2

a / b = 0.765　　b / c = 9.6

By interpolation, α = 0.7708

β = 0.1719

Max. bending stress of glass pane, σ_{c1} = 0.7708 x 3.73 / 1000 x 1200^2 / 11.91^2

= 29.19 N/mm^2

$\leq$ 80 N/mm^2　　O.K.

Max. deflection, δ_c = 0.1719 x 2.215 / 1000 x 1200^4 / (70000 x 11.91^3)

= 6.68 mm

$\leq$ 1200 / 60 = 20 mm　　O.K.

4.2 Check for flat cap routel (Kin Long TF12)

Vertical load = 0.7 + 0.75 +3.68　　(DL+LL + WL)

= 5.13 kN/m^2　　(unfactored)

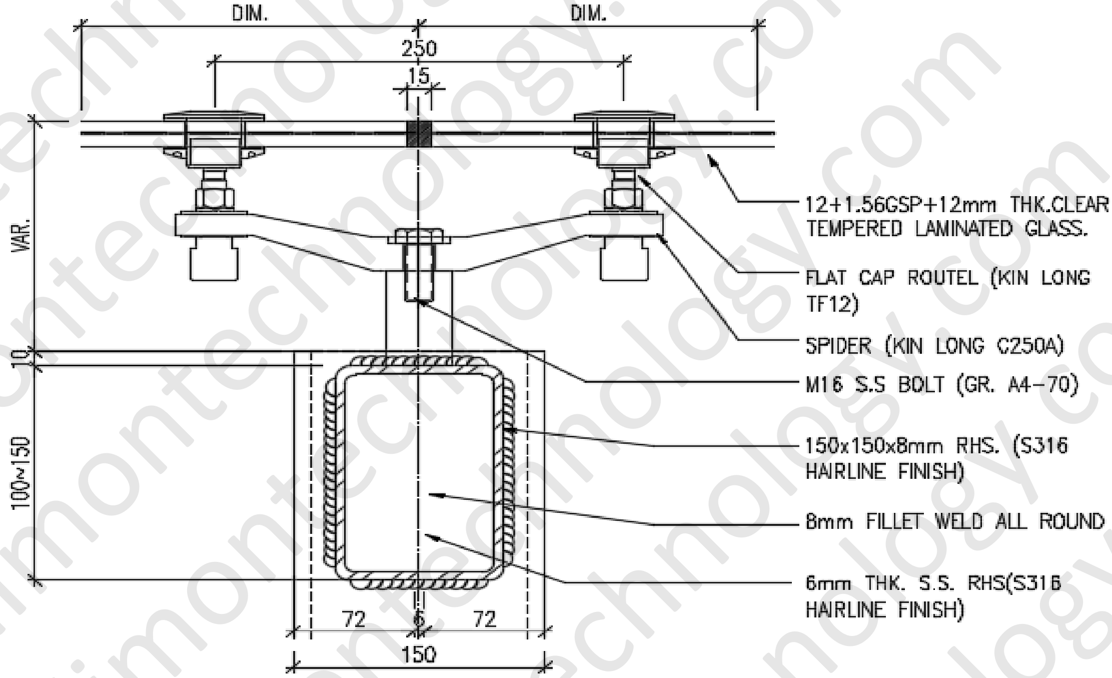
Vertical load on routel at centre = 5.13 x (1200 / 2 + 250) x (1162.5 / 2 + 105) / 10^6

= 2.99 kN

Reaction on each routel point tension/compresion = 2.99 kN　　(refer to Appendix)

< 4.5 kN　　O.K.

4.3 Check for spider (Kin Long C250A)



Vertical load on spider	= 2.99	kN	(refer to item 4.2)	
Reaction on each spider tension/compresion	= 2.99	kN	(refer to Appendix)	
	< 4	kN		O.K.

4.4 Check for M16 s.s. bolt, A4-70

Vertical load	= 2 x 2.909			
	= 5.818	kN	(refer to item 4.2)	
Tensile area of M16 bolt	= 156	mm^2		
Tensile strength of M16 bolt	= 373	N/mm^2		
Shear strength of M16 bolt	= 280	N/mm^2		
Tensile capacity of bolt	= 156 x 373 / 1000			
	= 58.19	kN		
	> 5.818	kN		O.K.
Pull out capacity on connected part,	= 16 x 3.1416 x 10 / 2 x 127 / 1000			
	= 31.919	kN		
	> 5.818	kN		O.K.

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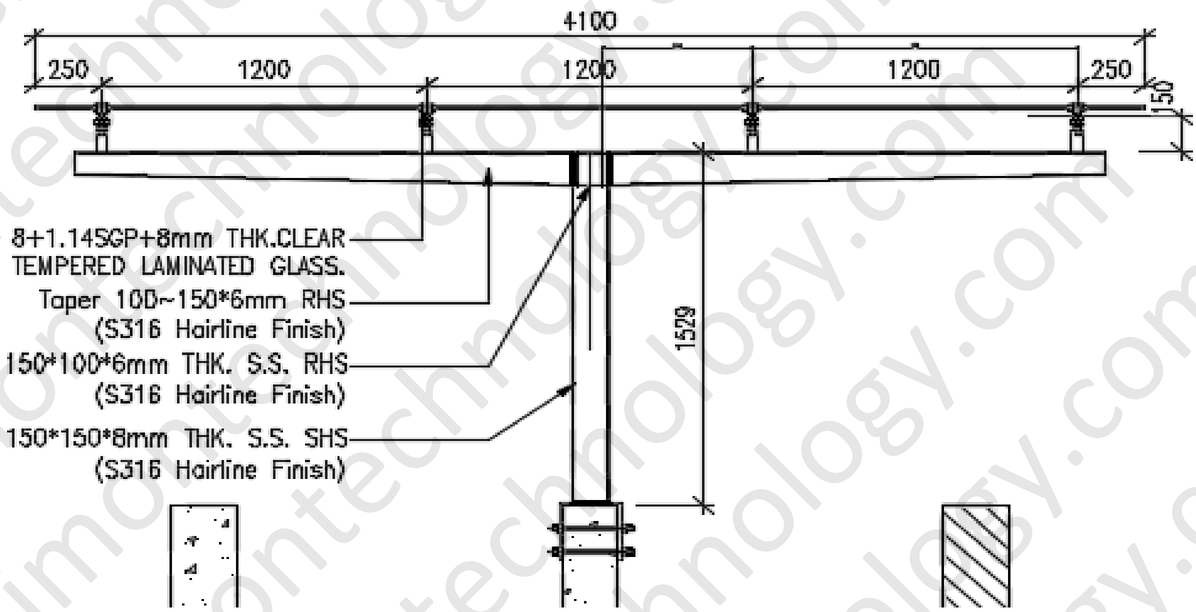
Check for 5mm fillet weld connector channel to steel frame

Horizontal load,	=	5.818	kN	
Effective length of fillet weld,	=	2 x 3.14 x 23		
	=	144.44	mm	
Capacity of fillet weld	=	160 x144.44 x 5 x 0.7 / 1000		
	=	80.886	kN	
	>	5.818	kN	O.K.

5. Check for steel frame



5.1 Check for150~100x100x6mm RHS

Vertical load = 7.46 kN/m^2

Point load P_1 = 7.46 x 1.2 x 1.1625
= 10.407 kN

P_2 = 7.46 x (1.2 / 2 + 0.25)x 1.1625
= 7.371 kN

Length of cantilever = 1850 mm

Reaction at support = 10.407 + 7.371
= 17.778 kN

Max bending moment = 10.407 x 0.55 x 7.371 x 1.75
= 18.62 kNm

Shear stress = 17.778 x 1000 / (6 x 150)
= 19.75 N/mm^2
< 0.6 x 127 = 76.2 N/mm^2 (low shear)

Moment capacity = min(1.2 p_y Z_y, p_y S_y)
= (1.2 x 220 x 115/1000 , 220 x 141 / 1000)
= 30.36 kNm (factored)
> 18.62 kNm O.K.

Max .deflection		=	$P_k\ b^2(3L\text{-}b)/\ (6E\ I)$				
		=	10.407 x 5.13 / 7.46 x 550^2 x (3 x 1850 - 550)/ (6x200000 x 8620000) +				
			7.371 x 5.13 / 7.46 x 1750^2 x (3 x 1850 - 1750)/ (6x200000 x 8620000) +				
		=	6.75	mm		(unfactored)	
		<	2 x 1850 / 250	=	14.8	mm	O.K.

<u>Check for lateral torsional buckling</u>

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	=	100	mm	Overall depth,	D	=	150	mm
Wall thickness,	t	=	6	mm	Area,	A	=	28.2	cm^2
Moment of inertia,	I_x	=	862	cm^4	Moment of inertia,	I_y	=	456	cm^4
Section modulus,	Z_x	=	115	cm^3	Section modulus,	Z_y	=	91.2	cm^3
Plastic modulus,	S_x	=	141	cm^3	Plastic modulus,	S_y	=	106	cm^3
Torsional constant,	J	=	946	cm^4	Radius of gyration,	r_y	=	4.02	cm
Effective length,	L_E	=	2362	mm					

Slenderness ratio,	λ	=	L_E / r_y	=	46	
	γ_b	=	$(1 - I_y / I_x)\ [1 - J / (2.6\ I_x)]$	=	0.2722	
Buckling index,	$\o_b$	=	$[S_x^{\ 2}\ \gamma_b / (A\ J)]^{0.5}$	=	0.4504	
Ratio,	β_w	=	1			for plastic section
Equivalent slenderness,	λ_{LT}	=	$2.25\ (\o_b\ \lambda\ \beta_w)^{0.5}$	=	10.24	
	P_E	=	$\pi^2\ E / \lambda_{LT}^{\ 2}$	=	18824.78	
	α_{LT}	=	7			
	λ_{L0}	=	$0.4\ (\pi^2\ E / p_y)^{0.5}$	=	37.89	
Perry factor,	η_{LT}	=	$\alpha_{LT}\ (\lambda_{LT} - \lambda_{L0}) / 1000$	=	-0.1936	< 0
	$\o_{LT}$	=	$[p_y + (\eta_{LT} + 1)\ P_E] / 2$	=	7700.15	

Bending buckling strength,	p_b	=	$P_E\ p_y / (\o_{LT} + (\o_{LT}^{\ 2} - P_E\ p_y)^{0.5})$	
		=	273.79	N/mm^2
		>	220	N/mm^2

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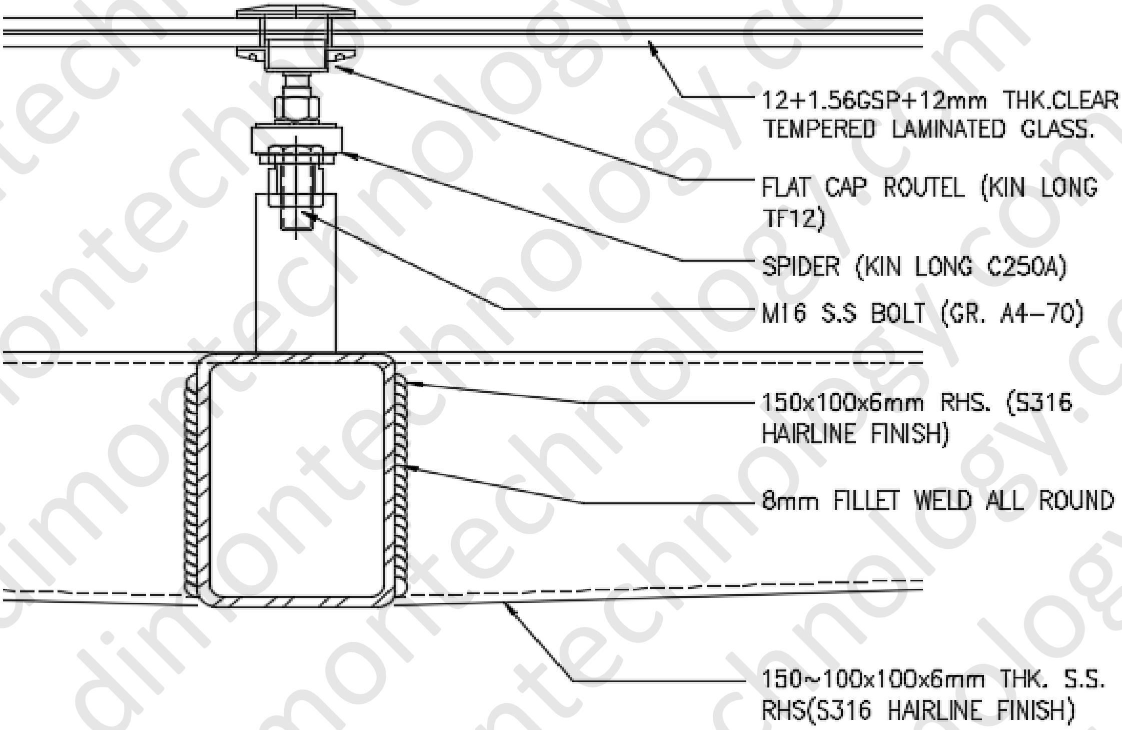
5.2 Check for 8mm fillet weld all round connection to horizontal member



Vertical load = 17.778 kN (refer to item 5.1)

Moment = 18.62 kNm (refer to item 5.1)

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

Breadth, $B = 100$ mm

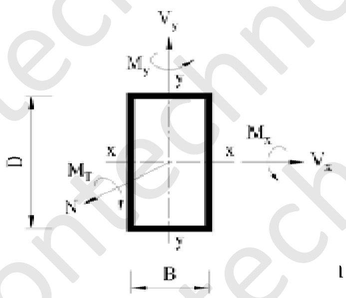
Height, $D = 150$ mm

Area, $A = 2 (B + D) = 500$ mm^2

Moment of inertia, $I_x = B D^2 / 2 + D^3 / 6 = 1687500$ mm^4

Moment of inertia, $I_y = B^2 D / 2 + B^3 / 6 = 916667$ mm^4

Polar moment of inertia, $J = I_x + I_y = 2604167$ mm^4



Shear load, $V_x = 0$ kN Moment,

Shear load, $V_y = 17.778$ kN Moment, $M_y = 0$ kNm

Tensile load, $N = 0$ kN Torsional moment, $M_T = 0$ kNm

Leg length of fillet weld, $t = 8$ mm

Shear stress, $\tau_x = V_x / (0.7\ t\ A) + M_T\ (D / 2) / (0.7\ t\ J) = 0$ N/mm^2

Shear stress, $\tau_y = V_y / (0.7\ t\ A) + M_T\ (B / 2) / (0.7\ t\ J) = 6.35$ N/mm^2

Tensile stress, $\sigma = N / (0.7\ t\ A) = 0$ N/mm^2

Tensile stress, $\sigma_x = M_x\ D / (2\ I_x) / (0.7\ t) = 147.78$ N/mm^2

Tensile stress, $\sigma_y = M_y\ B / (2\ I_y) / (0.7\ t) = 0$ N/mm^2

Resultant , $f_w = [\tau_x^2 + \tau_y^2 + (\sigma + \sigma_x + \sigma_y)^2]^{0.5}$
$= 147.92$ N/mm^2
≤ 200 N/mm^2 O.K.

5.3 Check for horizontal member 150x100x6mm RHS

Vertical load = 2 x 17.778 (refer to item 5.1)
= 34.896 kN

Torsiona moment = 18.62 kNm (refer to item 5.2)

Distance of support = 2325 mm

Reaction at support = 34.896 / 2
= 17.448 kN

Max bending moment = 34.896 x 2.325 / 4
= 20.28 kNm

Shear stress = 17.778 x 1000 / (6 x 150) + 18.62 / 147000
= 19.39 N/mm^2
< 0.6 x 127 = 76.2 N/mm^2 (low shear)

Moment capacity = $\min(1.2\ p_y Z_y, p_y S_y)$
= (1.2 x 220 x 115/1000 , 220 x 141 / 1000)
= 30.36 kNm (factored)
> 20.28 kNm O.K.

Max .deflection = $1/48 P_k L^3 / (E\ I)$
= 34.896 x 5.13 / 7.46 x 1000 x 2325^3 / (48 x200000 x 8620000)
= 3.645 mm (unfactored)
< 2325 / 250 = 9.3 mm O.K.

<u>Check for lateral torsional buckling</u>

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	= 100	mm	Overall depth,	D	= 150	mm
Wall thickness,	t	= 6	mm	Area,	A	= 28.2	cm^2
Moment of inertia,	I_x	= 862	cm^4	Moment of inertia,	I_y	= 456	cm^4
Section modulus,	Z_x	= 115	cm^3	Section modulus,	Z_y	= 91.2	cm^3
Plastic modulus,	S_x	= 141	cm^3	Plastic modulus,	S_y	= 106	cm^3
Torsional constant,	J	= 946	cm^4	Radius of gyration,	r_y	= 4.02	cm

Effective length, $L_E = 2362$ mm   Torsional constant, $C = 147$ cm^3

Slenderness ratio, $\lambda = L_E / r_y = 46$

$\gamma_b = (1 - I_y / I_x)\,[1 - J / (2.6\, I_x)] = 0.2722$

Buckling index, $ø_b = [S_x^2\, \gamma_b / (A\, J)]^{0.5} = 0.4504$

Ratio, $\beta_w = 1$ for plastic section

Equivalent slenderness, $\lambda_{LT} = 2.25\, (ø_b\, \lambda\, \beta_w)^{0.5} = 10.24$

$P_E = \pi^2\, E / \lambda_{LT}^2 = 18824.78$

$\alpha_{LT} = 7$

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

Perry factor, $\eta_{LT} = \alpha_{LT}\, (\lambda_{LT} - \lambda_{L0}) / 1000 = -0.1936 < 0$

$ø_{LT} = [p_y + (\eta_{LT} + 1)\, P_E] / 2 = 7700.15$

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

$= 273.79$ N/mm^2

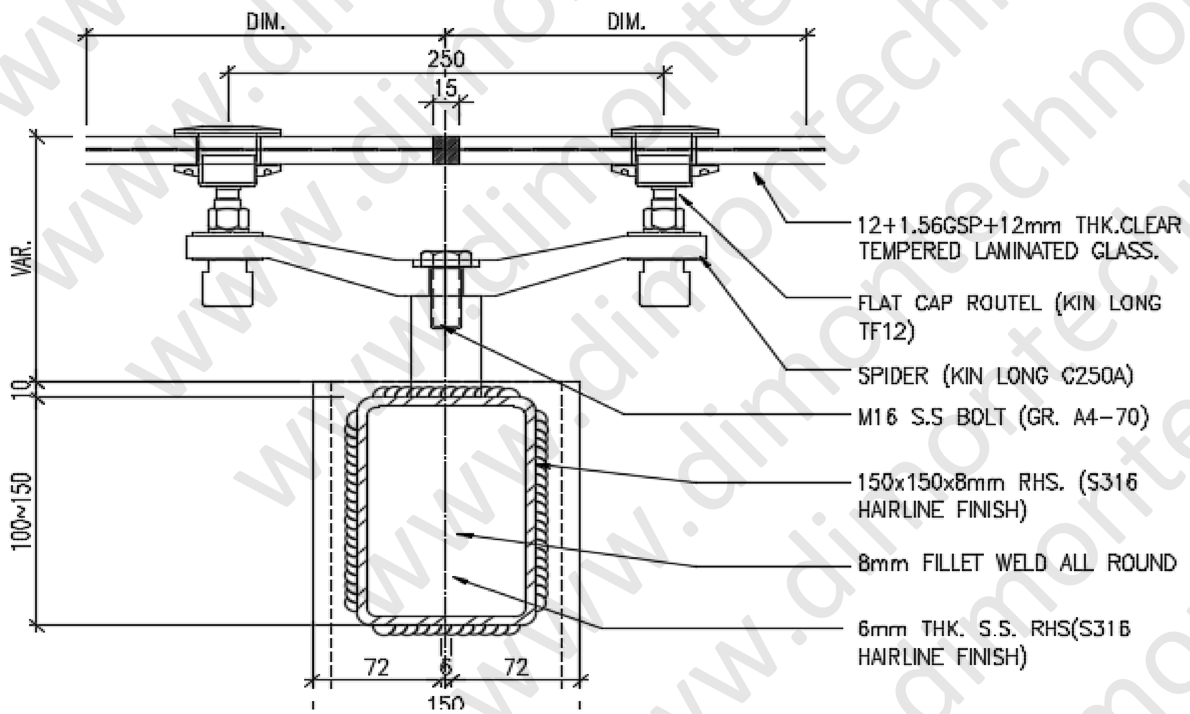
> 220 N/mm^2

Capacity of buckling moment $= 220 \times 141 / 10^3$

$= 31.02$ kNm   (factored)

> 18.62 kNm   O.K.

5.4 Check for 8mm fillet weld all round connection horizontal member to column



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V

Torsional Moment = 18.62 / 2 (refer to item 5.3)
= 9.31 kNm

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

Breadth, B = 100 mm

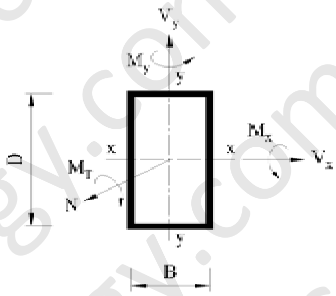
Height, D = 150 mm

Area, A = $2\ (B + D)$ = 500 mm^2

Moment of inertia, I_x = $B\ D^2 / 2 + D^3 / 6$ = 1687500 mm^4

Moment of inertia, I_y = $B^2\ D / 2 + B^3 / 6$ = 916667 mm^4

Polar moment of inertia, J = $I_x + I_y$ = 2604167 mm^4



Shear load, V_x = 0 kN Moment,

Shear load, V_y = 17.448 kN Moment,

Tensile load, N = 0 kN Torsional moment, M_T = 9.31 kNm

Leg length of fillet weld, t = 8 mm

Shear stress, τ_x = $V_x / (0.7\ t\ A) + M_T\ (D / 2) / (0.7\ t\ J)$ = 47.88 N/mm^2

Shear stress, τ_y = $V_y / (0.7\ t\ A) + M_T\ (B / 2) / (0.7\ t\ J)$ = 38.15 N/mm^2

Tensile stress, σ = $N / (0.7\ t\ A)$ = 0 N/mm^2

Tensile stress, σ_x = $M_x\ D / (2\ I_x) / (0.7\ t)$ = 73.89 N/mm^2

Tensile stress, σ_y = $M_y\ B / (2\ I_y) / (0.7\ t)$ = 0 N/mm^2

Resultant , f_w = $[\tau_x^2 + \tau_y^2 + (\sigma + \sigma_x + \sigma_y)^2]^{0.5}$
= 95.96 N/mm^2
$\leq$ 200 N/mm^2 O.K.

5.5 Check for column 150x150x8mm SHS

<u>Properties of gms. 150x150x8mm thk. SHS.</u>

Moment of inertia,	I	=	1491	cm^4	Area,	A	=	44.8	cm^2
Section modulus,	Z	=	199	cm^3	Plastic modulus,	S	=	237	cm^3
Torsional constants of inertia,	J	=	2351	cm^4	Torsional constants of modulus	C	=	291	cm^3
Radius of gyration	r	=	5.77	cm					

Vertical load = 2 x 17.778 (refer to item 5.1)
= 34.896 kN

Moment = 2 x 18.62
= 37.24 kNm (refer to item 5.1)

Moment capacity = $\min(1.2\ p_y\ Z_y,\ p_y\ S_y)$
= (1.2 x 220 x 199 /1000 , 220 x 237 / 1000)
= 52.14 kNm (factored)
> 37.24 kNm O.K.

Design strength,	p_y	=	220	N/mm^2	Robertson constant,	α	=	2	
Modulus of Elasticity,	E	=	200000	N/mm^2	Radius, of gyration,	r_y	=	57.7	mm
Effective length,	L_E	=	1529	mm					

Slenderness, $\lambda = L_E / r_y$ = 26.4991
Limit slenderness, $\lambda_0 = 0.2\ (\pi^2\ E / p_y)^{0.5}$ = 18.9445
Perry factor, $\eta = \alpha\ (\lambda - \lambda_0) / 1000$ = 0.0151 ≥ 0 O.K.
$P_E = \pi^2\ E / \lambda^2$ = 2811.043
$ø_c = [p_y + (\eta + 1)\ P_E] / 2$ = 1536.745

Compressive buckling strength, $p_c = P_E\ p_y / [ø_c + (ø_c{}^2 - P_E\ p_y)^{0.5}]$
= 216.46 N/mm^2

Compression resistance P_c = 216.46 x 44.8 x 100
= 969.741 kN
> 34.896 kN O.K.

Combined loads = 34.896 / 969.741 + 37.24 / 52.14
= 0.75
< 1 O.K.

5.6 Check for 8mm fillet weld all round connection to base plate

Vertical load = 34.896 kN (refer to item 5.5)

Moment = 37.24 kNm (refer to item 5.5)

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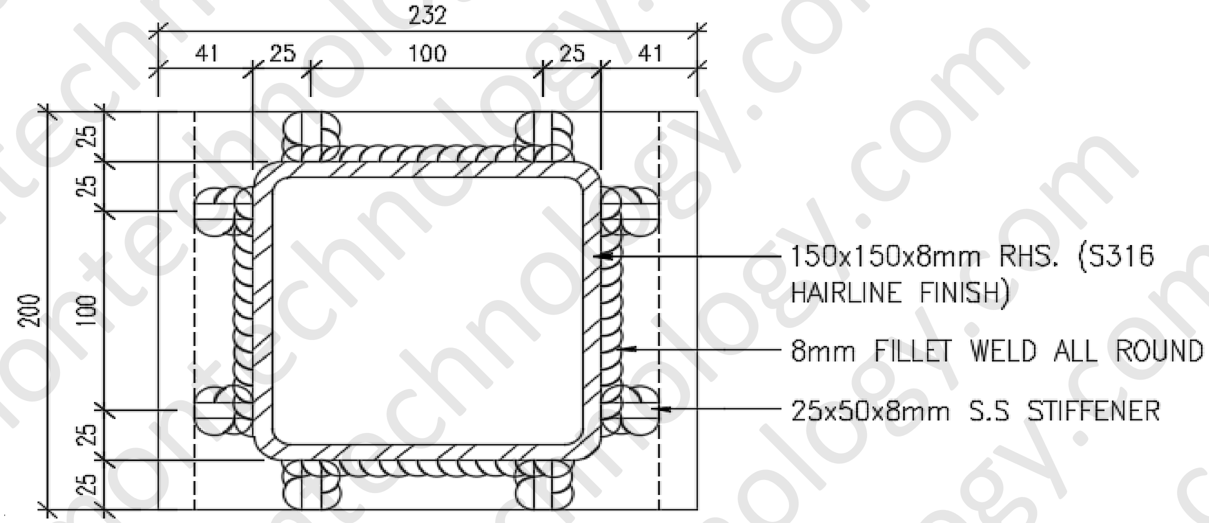

Breadth, $B = 200$ mm

Height, $D = 200$ mm

Area, $A = 16 \times 25 + 4 \times 150 = 1000$ mm^2

Moment of inertia, $I_x = B D^2 / 2 + D^3 / 6 + 8 \times 25 \times 87.5^2 + 4 \times 25 \times 54^2 + 4 \times 25 \times 46^2$

$= 4284450$ mm^4

Moment of inertia, $I_y = B^2 D / 2 + B^3 / 6 + 8 \times 25 \times 87.5^2 + 4 \times 25 \times 54^2 + 4 \times 25 \times 46^2$

$= 4284450$ mm^4

Polar moment of inertia, $J = I_x + I_y = 8568900$ mm^4

Shear load,	$V_x = 0$	kN		Moment,	$M_x = 37.24$	kNm
Shear load,	$V_y = 0$	kN		Moment,	$M_y = 0$	kNm
Tensile load,	$N = 34.896$	kN		Torsional moment,	$M_T = 0$	kNm

Leg length of fillet weld, $t = 8$ mm

Shear stress, $\tau_x = V_x / (0.7\, t\, A) + M_T (D / 2) / (0.7\, t\, J) = 0$ N/mm^2

Shear stress, $\tau_y = V_y / (0.7\, t\, A) + M_T (B / 2) / (0.7\, t\, J) = 0$ N/mm^2

Tensile stress, $\sigma = N / (0.7\, t\, A) = 6.23$ N/mm^2

Tensile stress, $\sigma_x = M_x D / (2 I_x) / (0.7\, t) = 155.21$ N/mm^2

Tensile stress, $\sigma_y = M_y B / (2 I_y) / (0.7\, t) = 0$ N/mm^2

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

$= 161.44$ N/mm^2

≤ 200 N/mm^2 O.K.

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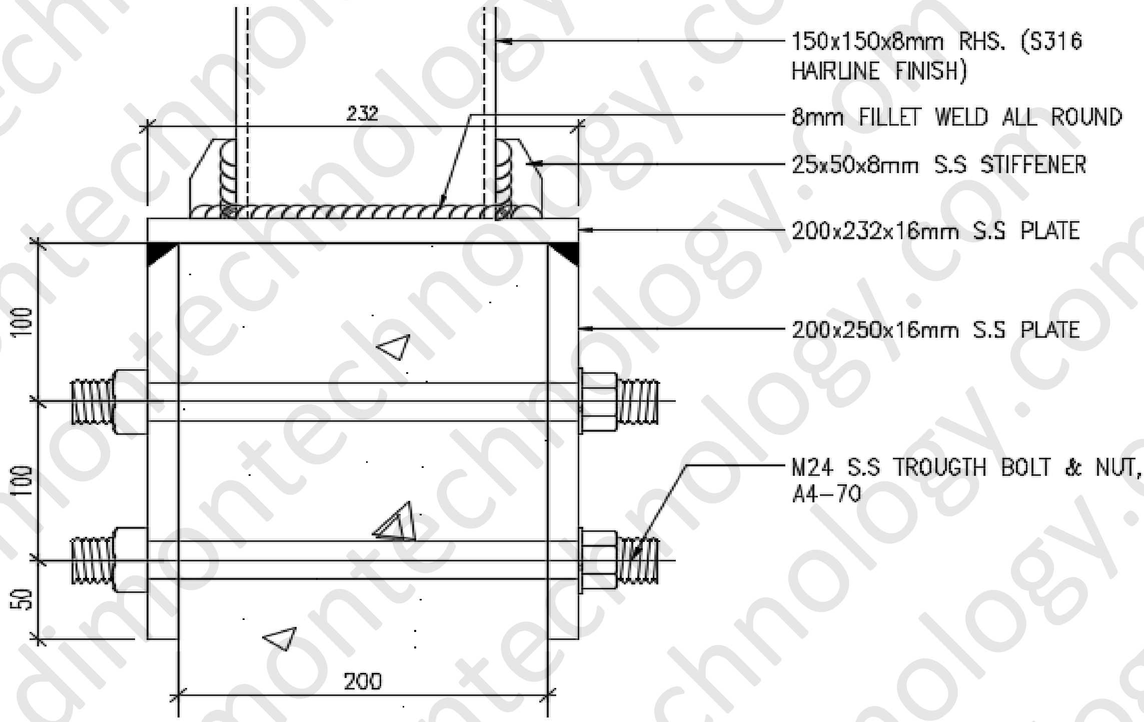
6. Check for embed



Moment	= 37.24	kNm		(refer to item 5.5)	
Shear load on each bolt	= 37.24 / 0.232 / 4 + 34.896 / 2 / 4				
	= 44.49	kN			
Tensile area of M24 bolt	= 352	mm^2			
Tensile strength of M24 bolt	= 373	N/mm^2			
Shear strength of M24 bolt	= 280	N/mm^2			
Shear capacity of bolt	= 352 x 280 / 1000				
	= 98.56	kN			
	> 44.49	kN			O.K.
Bearing pressure on concrete	= 44.49 x 1000 / (24 x 100)				
	= 18.538	N/mm^2			
	< 0.6 x 45	= 27		N/mm^2	O.K.