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Appendix

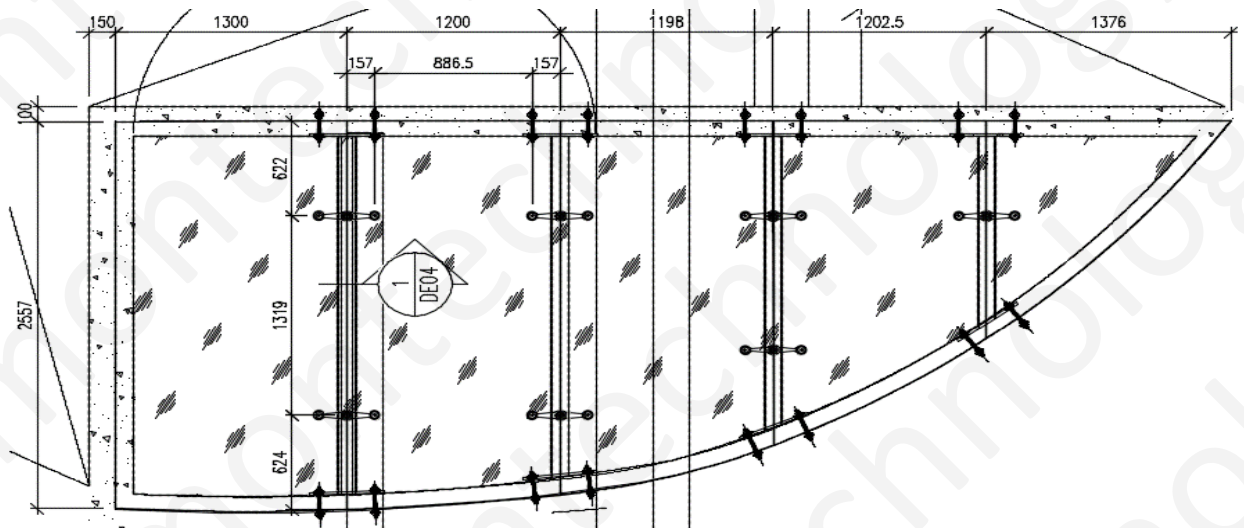
## 1. Introduction


There are 2 types of glass panel. The overall size of first type is 1300x2557mm with 2 point & 2 sides supports. The overall size of second type is 2565x1200mm with 4 point supports.

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



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## 2. Design Code, Design Data

### 2.1 Design Code

- Hong Kong Building (Construction) Regulation 1990 (Amendment 2011).
- Code of Practice on Wind Effect 2008, Macau.
- Code of Practice for the Structural Use of Steel 2011, Hong Kong
- Code of Practice for the Structural Use of Concrete 2004, Hong Kong
- Code of Practice for Dead and imposed Loads 2011, Hong Kong
- Code of Practice for the Structural Use of Steel GB 50017-2003
- 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


		1.4401 (316 S31)
stainless steel grade		X5CrNiMo17-12-2
0.2% proof stress	(N/mm <sup>2</sup> )	220
ultimate tensile strength, min.	(N/mm <sup>2</sup> )	510
Modulus of elasticity	(N/mm <sup>2</sup> )	200000
Design strength	(N/mm <sup>2</sup> )	220
Design stress of fillet weld	(N/mm <sup>2</sup> )	220

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

steel grade		E50xx
design strength of filled weld	(N/mm <sup>2</sup> )	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 <sup>2</sup> )	450
ultimate tensile strength	(N/mm <sup>2</sup> )	700
design tensile strength	(N/mm <sup>2</sup> )	373
design shear strength	(N/mm <sup>2</sup> )	280
design bearing strength	(N/mm <sup>2</sup> )	805
stainless steel grade		A4 / 316
design bearing strength on connected part	(N/mm <sup>2</sup> )	479

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### 2.2.4 Concrete

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

### 2.2.5 The data refer to SAP2000 program.

## 3. Design Load

### 3.1 Wind load

Wind pressure,  $q_z = 2.12$  kPa (height above ground level  $\leq 20m$ )

Pressure coefficient,  $= 2$  (open frame)

Design wind load  
 $= 2.12 \times 2.0$   
 $= 4.24$  kPa

### 3.2 Live load

Live load  $= 0.75$  kN/m<sup>2</sup>

### 3.3 Dead Load

Weight of glass	$= (12 + 12) \times 26.5 / 1000$	$= 0.636$	kPa
Others,	$=$	$= 0.064$	kPa
		<b>Total</b>	$= 0.7$ kPa

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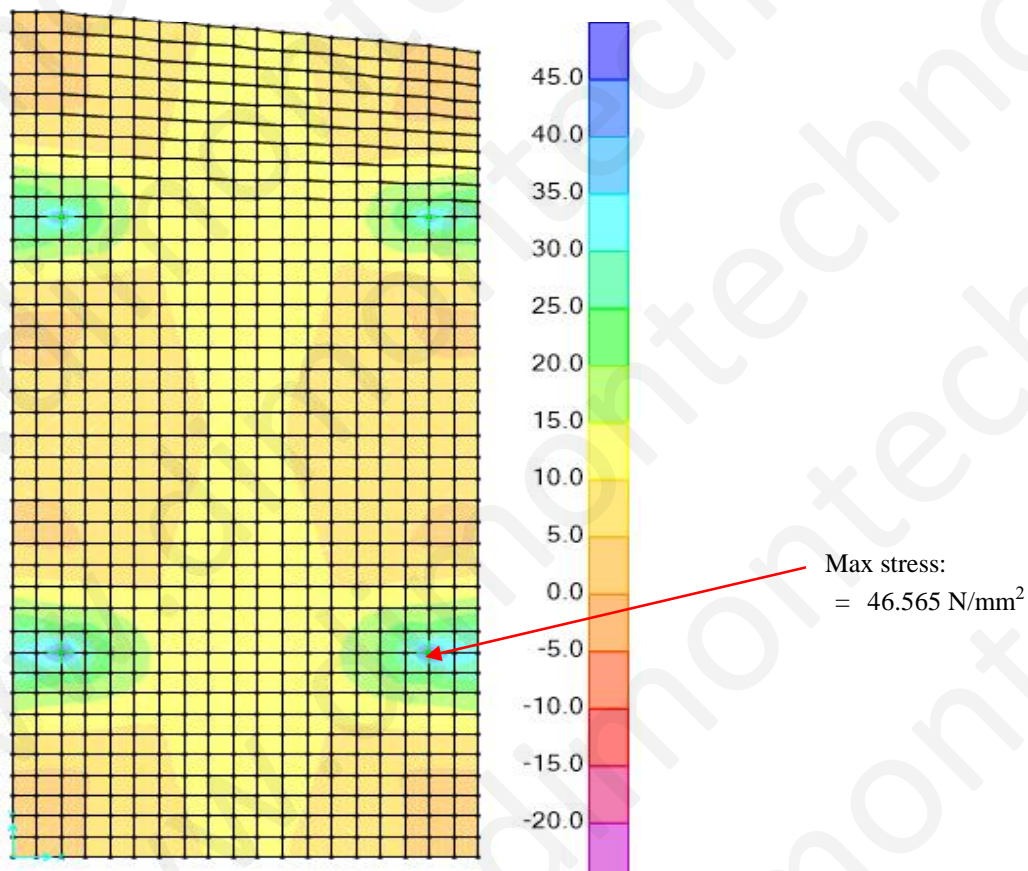
#### 4. Check for laminated tempered glass

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

Glass density,	$\rho = 2650$	$\text{kg/m}^3$		
Nominal thickness of glass pane 1	= 12	mm	Min. thickness of glass pane 1,	$t_1 = 11.91$ mm
Nominal thickness of glass pane 2	= 12	mm	Min. thickness of glass pane 2,	$t_2 = 11.91$ mm
Glass type for pane 1 & 2	: tempered		Ultimate design strength,	$p_y = 80$ N/mm <sup>2</sup>
Load duration	: short term		Reduction factor,	$\gamma_d = 1$
Surface treatment	: clear		Reduction factor,	$\gamma_s = 1$
			Material factor,	$\gamma_m = 1$
Ultimate resistance strength,	$R_{ult} = p_y \gamma_d \gamma_s / \gamma_m$			= 80 N/mm <sup>2</sup>

Case 1: 4 point supports

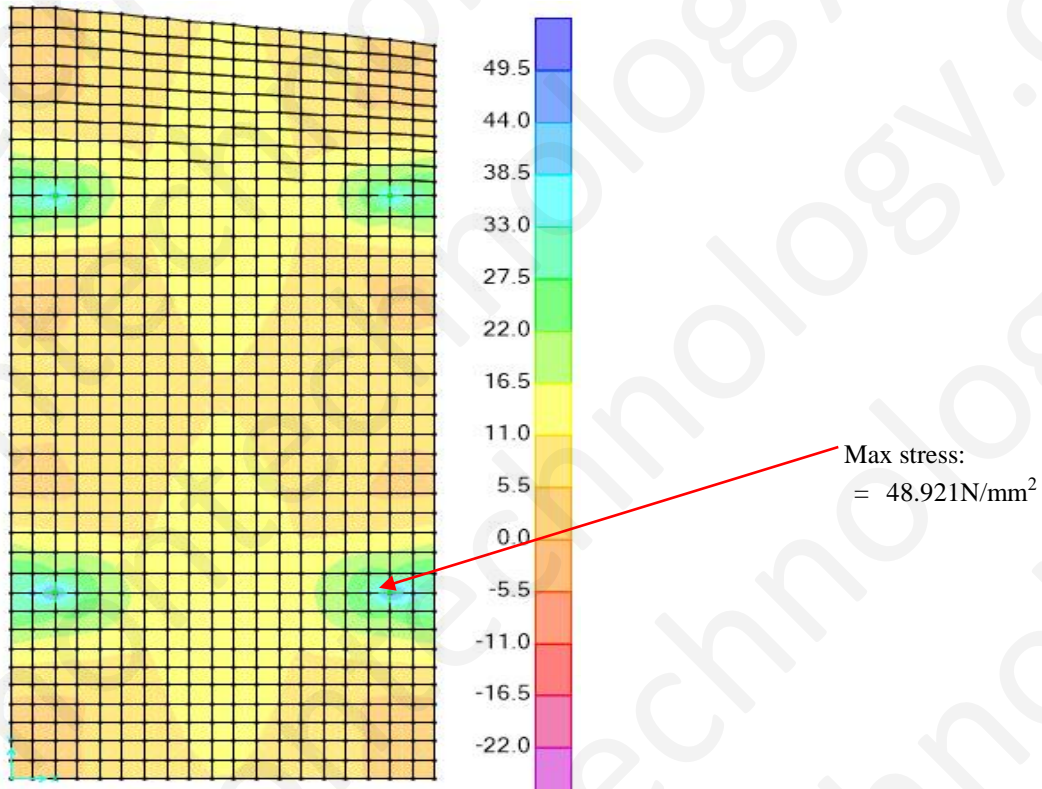
Comb 1: 1.3DL + 1.05LL + 0.9WL;



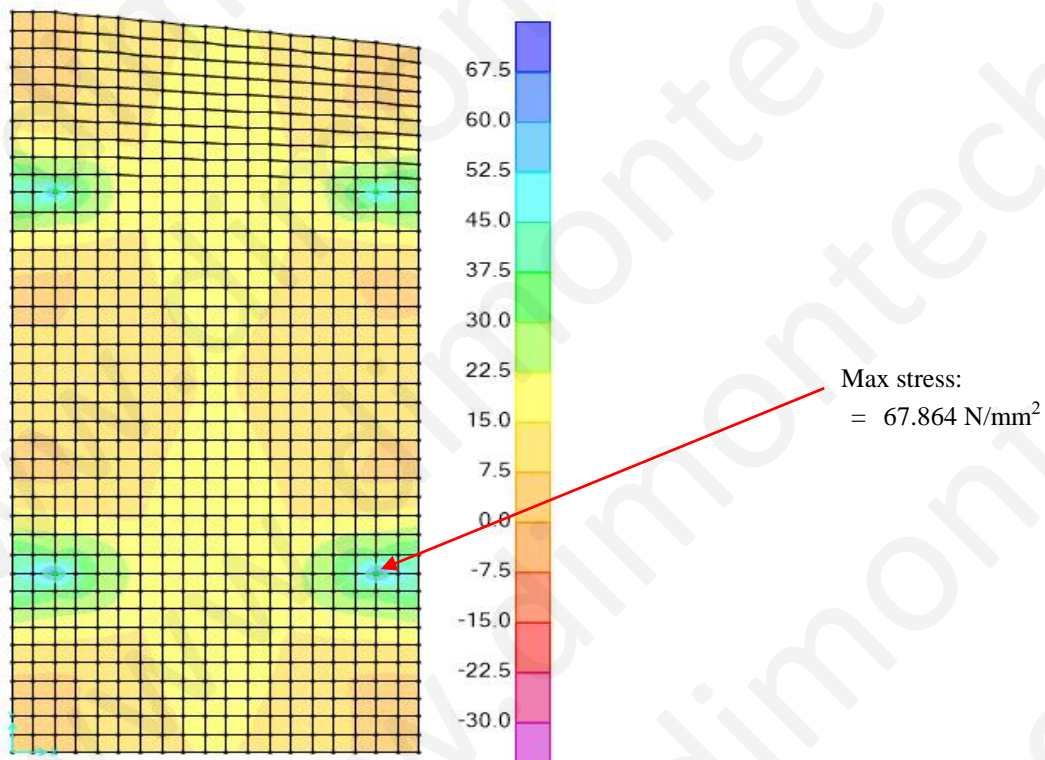
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Comb 2: 1.2DL + 1.5LL + 0.9WL;

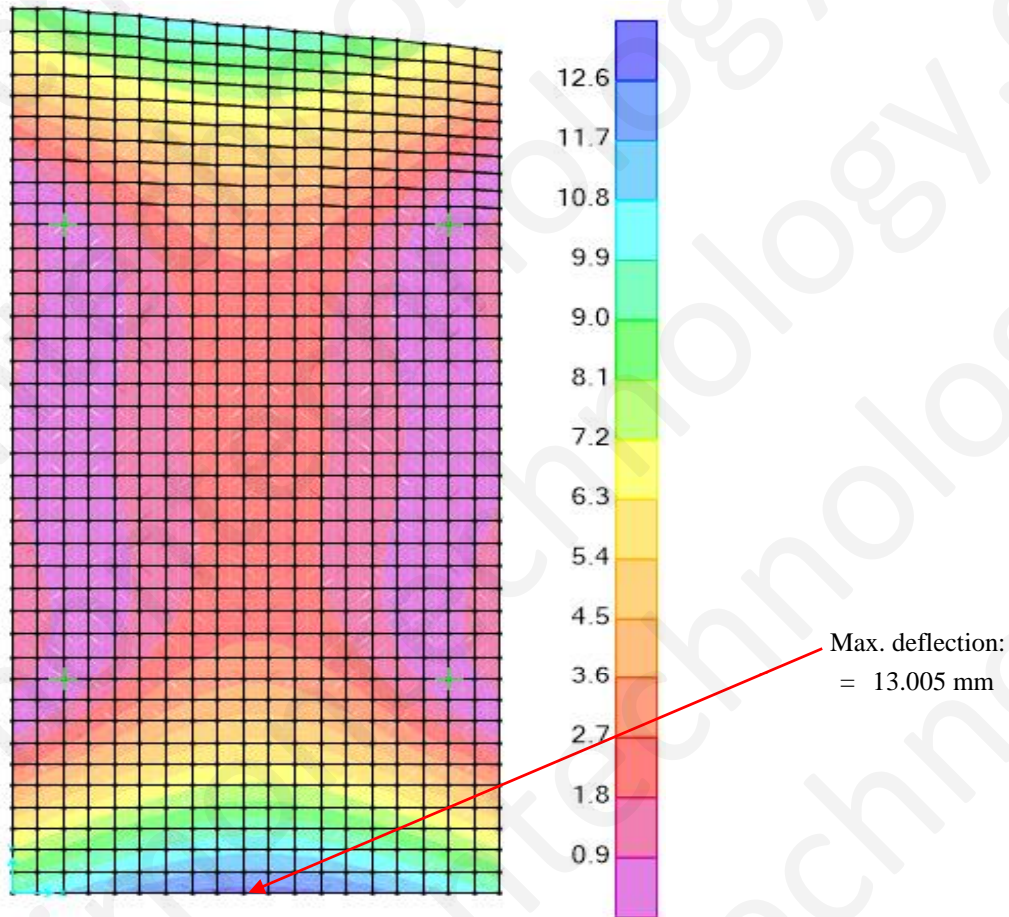


Comb 3: 1.2DL + 1.05LL + 1.5WL;



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Comb 4: 1.0DL + 1.0WL; (for deflection checking)



Max. bending stress of  
glass pane,

$$\sigma_{c1} = 67.864 \quad \text{N/mm}^2$$

$$\leq 80 \quad \text{N/mm}^2 \quad \text{O.K.}$$

Max. deflection,

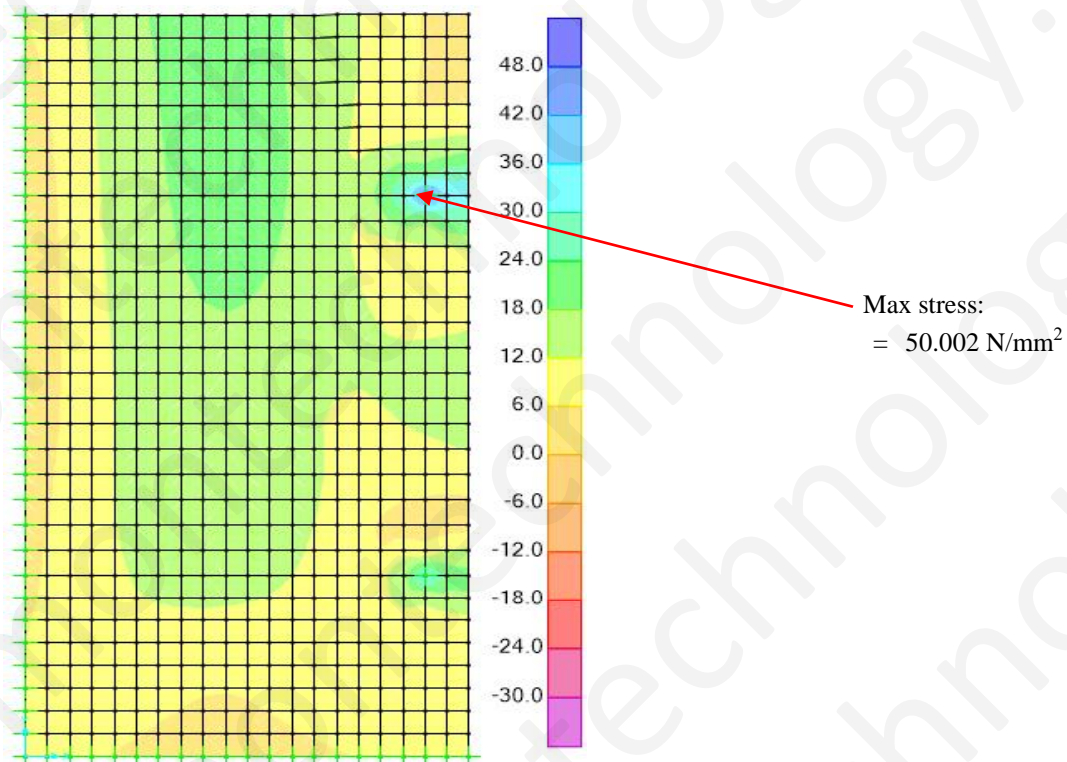
$$\delta_c = 13.005 \quad \text{mm}$$

$$\leq 950 / 60 = 15.83 \quad \text{mm} \quad \text{O.K.}$$

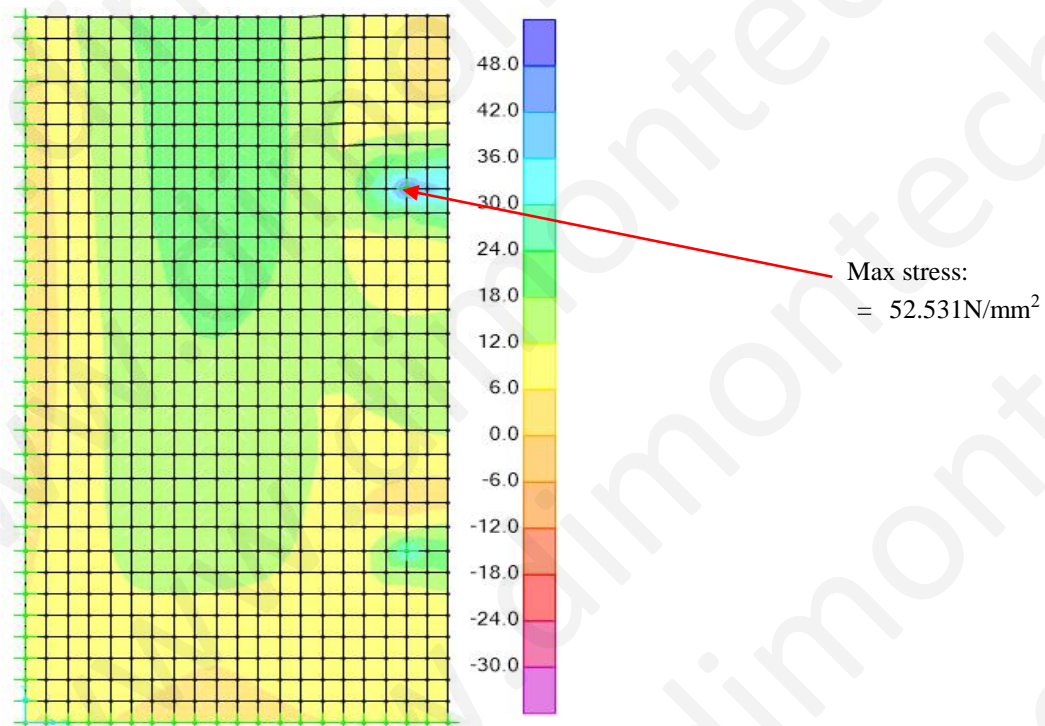
<h1 style="margin: 0;">DIMON</h1> <p style="margin: 0;">TECHNOLOGY</p>		Sheet No.	7
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Case 2: 2 point supports + 2 side supports

Comb 1: 1.3DL + 1.05LL + 0.9WL;



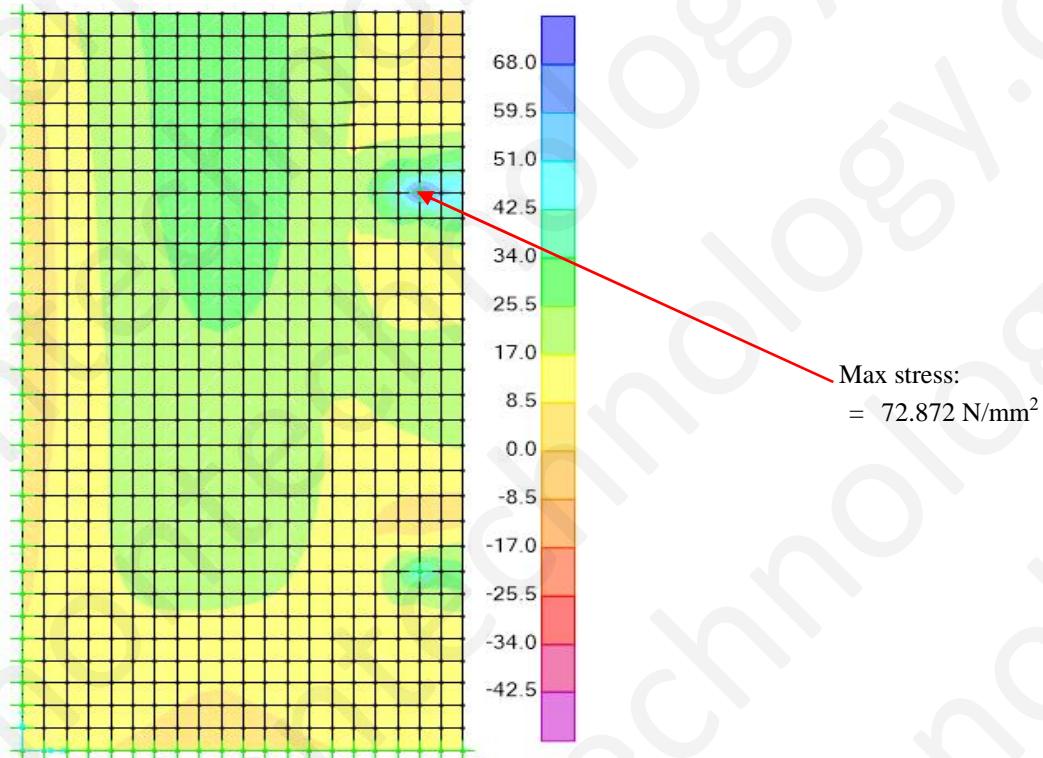
Comb 2: 1.2DL + 1.5LL + 0.9WL;



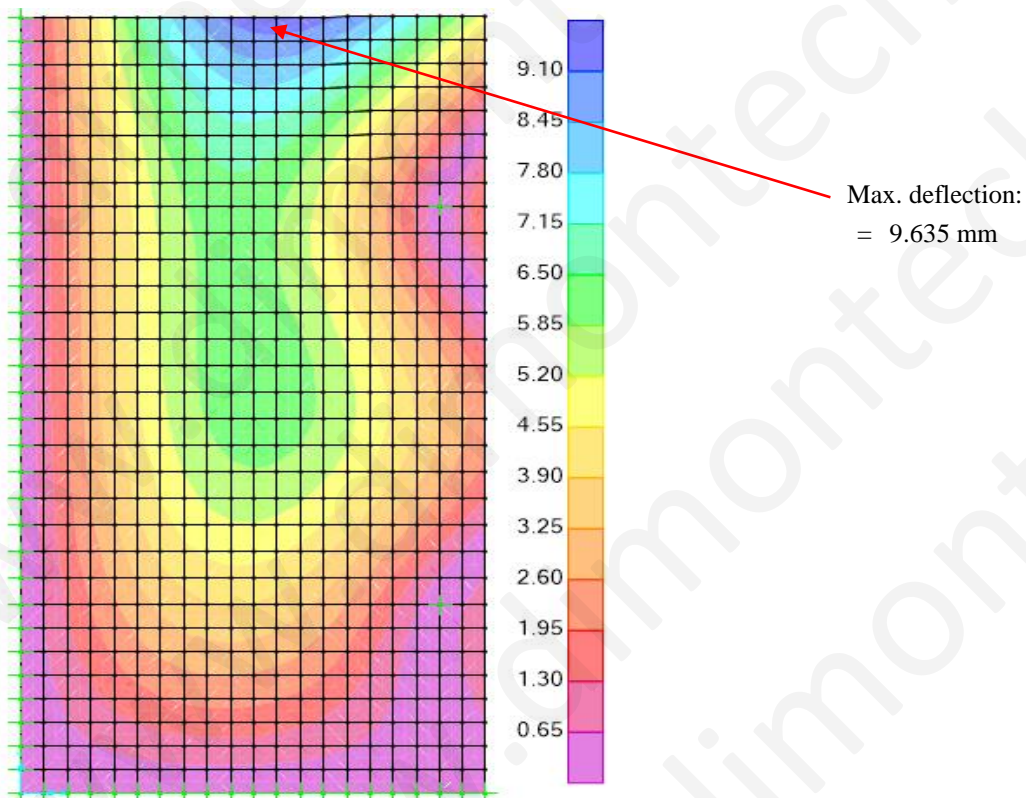



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Comb 3: 1.2DL + 1.05LL + 1.5WL;



Comb 4: 1.0DL + 1.0WL; (for deflection checking)



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Max. bending stress of  
glass pane,

$$\begin{aligned}\sigma_{c1} &= 72.872 \quad \text{N/mm}^2 \\ &\leq 80 \quad \text{N/mm}^2\end{aligned}$$

O.K.

Max. deflection,

$$\begin{aligned}\delta_c &= 9.635 \quad \text{mm} \\ &\leq 1175 / 60 = 19.58 \quad \text{mm}\end{aligned}$$

O.K.

Check for structural sealant

$$\begin{aligned}\text{Structural sealant bite required} &= 4.24 \times 1.3 / 2 / 0.138 \\ &= 19.97 \quad \text{mm} \\ &\leq 26 \quad \text{mm}\end{aligned}$$

O.K.

#### 4.2 Check for flat cap routel (Kin Long TF12)

Case 1: 4 point supports

**Table: Joint Reactions**

Joint	Output Case	CaseType	F1	F2	F3	M1	M2	M3
			KN	KN	KN	KN-m	KN-m	KN-m
9	COMB1	Combination	0	0	2.112	0	0	0
9	COMB2	Combination	0	0	2.219	0	0	0
9	COMB3	Combination	0	0	3.078	0	0	0
9	COMB4	Combination	0	0	1.897	0	0	0
10	COMB1	Combination	0	0	2.023	0	0	0
10	COMB2	Combination	0	0	2.126	0	0	0
10	COMB3	Combination	0	0	2.949	0	0	0
10	COMB4	Combination	0	0	1.817	0	0	0
15	COMB1	Combination	0	0	2.124	0	0	0
15	COMB2	Combination	0	0	2.231	0	0	0
15	COMB3	Combination	0	0	3.095	0	0	0
15	COMB4	Combination	0	0	1.907	0	0	0
16	COMB1	Combination	0	0	1.914	0	0	0
16	COMB2	Combination	0	0	2.011	0	0	0
16	COMB3	Combination	0	0	2.789	0	0	0
16	COMB4	Combination	0	0	1.719	0	0	0

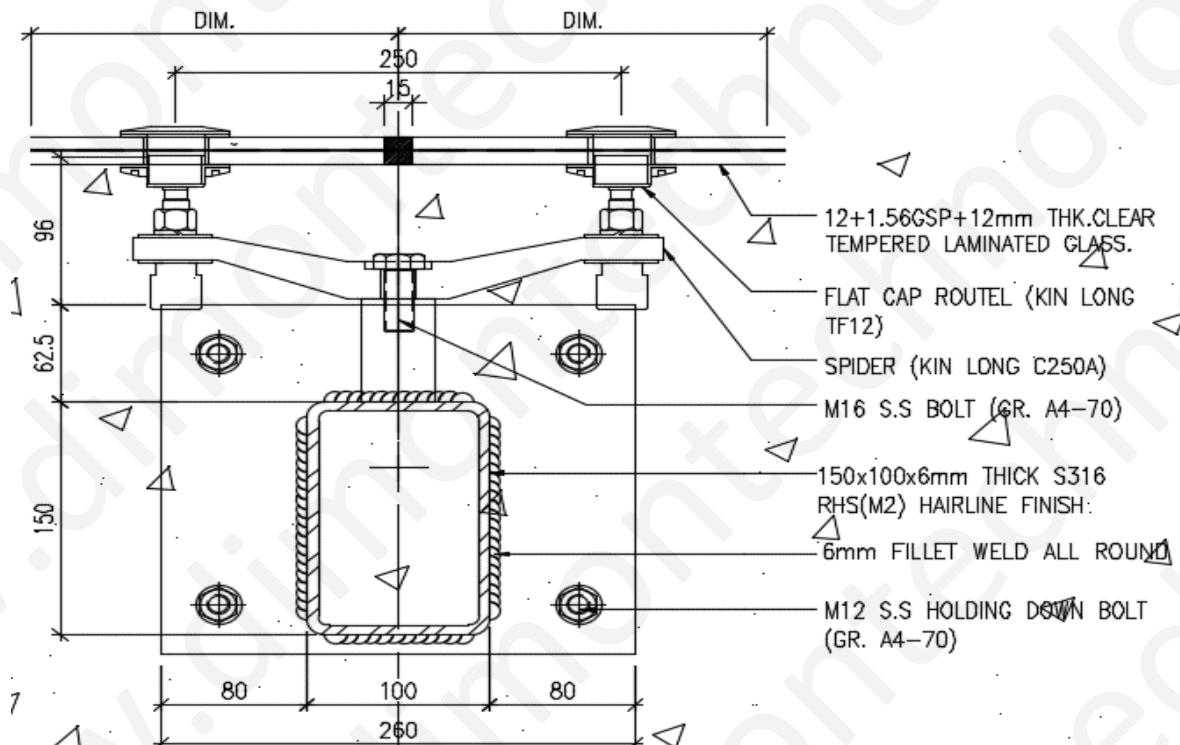
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Case 2: 2 point supports + 2 side supports

Table: Joint Reactions

Joint	Output Case	CaseType	F1	F2	F3	M1	M2	M3
			KN	KN	KN	KN-m	KN-m	KN-m
1282	COMB1	Combination	0	0	2.058	0	0	0
1282	COMB2	Combination	0	0	2.163	0	0	0
1282	COMB3	Combination	0	0	3	0	0	0
1282	COMB4	Combination	0	0	1.849	0	0	0
1283	COMB1	Combination	0	0	2.682	0	0	0
1283	COMB2	Combination	0	0	2.818	0	0	0
1283	COMB3	Combination	0	0	3.909	0	0	0
1283	COMB4	Combination	0	0	2.409	0	0	0


By comparison, the max. reaction load is 3.909 kN



$$\begin{aligned} \text{Vertical load} &= 0.7 + 0.75 + 4.24 && \text{(DL+LL + WL)} \\ &= 5.69 \quad \text{kN/m}^2 && \text{(unfactored)} \end{aligned}$$

$$\begin{aligned} &= 1.2 \times 0.7 + 1.05 \times 0.75 + 1.5 \times 4.24 && \text{(1.2DL+1.05LL +1.5 WL)} \\ &= 7.99 \quad \text{kN/m}^2 && \text{(factored)} \end{aligned}$$

$$\begin{aligned} \text{Vertical load on routel at centre} &= 3.909 \times 5.69 / 7.99 \\ &= 2.78 \quad \text{kN} \end{aligned}$$

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Reaction on each routel point  
tension/compression = 2.78 kN (refer to Appendix)  
< 4.5 kN O.K.

#### 4.3 Check for spider (Kin Long C250A)

Vertical load on spider = 2.78 kN (refer to item 4.2)

Reaction on each spider  
tension/compression = 2.78 kN (refer to Appendix)  
< 4 kN O.K.

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

Vertical load = 2 x 3.909  
= 7.818 kN (refer to item 4.2)

Tensile area of M16 bolt = 156 mm<sup>2</sup>  
Tensile strength of M16 bolt = 373 N/mm<sup>2</sup>  
Shear strength of M16 bolt = 280 N/mm<sup>2</sup>

Tensile capacity of bolt = 156 x 373 / 1000  
= 58.19 kN  
> 7.818 kN O.K.

Pull out capacity on connected  
part, = 16 x 3.1416 x 10 / 2 x 127 / 1000  
= 31.919 kN  
> 7.818 kN O.K.

#### Check for 5mm fillet weld connector channel to steel frame

Horizontal load, = 7.818 kN

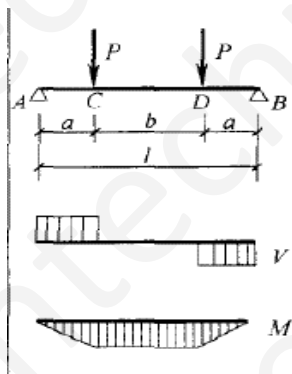
Effective length of fillet weld,  
= 2 x 3.14 x 23  
= 144.44 mm

Capacity of fillet weld = 160 x 144.44 x 5 x 0.7 / 1000  
= 80.886 kN  
> 7.818 kN O.K.

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## 5. Check for steel frame

### 5.1 Check for 150x100x6mm s.s. RHS



$$R_A = R_B = P$$

$$A \sim C \text{ 段: } M_x = Px$$

$$C \sim D \text{ 段: } M_x = M_{\max} = Pa$$

A - C 段:

$$x < a$$

$$w_x = \frac{Px}{6EI}(3la - 3a^2 - x^2)$$

$$x = a \quad w_c = \frac{Pa^2}{6EI}(3l - 4a)$$

C - D 段:  $a < x < a + b$ :

$$w_x = \frac{Pa}{6EI}(3lx - 3x^2 - a^2)$$

$$w_{\max} = \frac{Pa}{24EI}(3l^2 - 4a^2)$$

Vertical load,	$P = 2 \times 3.909$ $= 7.818$	kN	(refer to item 4.2) (factored)
Reaction at support	$R_A = R_B = 7.818$	kN	
Max bending moment	$= 7.818 \times 0.521$ $= 4.07$	kNm	
Shear stress	$= 7.818 \times 1000 / (2 \times 6 \times 150)$ $= 4.34$	N/mm <sup>2</sup>	$< 0.6 \times 127 = 76.2$ N/mm <sup>2</sup> (low shear)
Moment capacity	$= \min(1.2 p_y Z_y, p_y S_y)$ $= (1.2 \times 220 \times 115 / 1000, 220 \times 141 / 1000)$ $= 30.36$ $> 4.07$	kNm	(factored) O.K.
Max deflection	$= 2.78 \times 2 \times 1000 \times 521 \times (3 \times 2362^2 - 4 \times 521^2) / (24 \times 200000 \times 8620000)$ $= 1.096$	mm	(unfactored) $< 2362 / 250 = 9.45$ mm O.K.

#### Check for lateral torsional buckling

Design strength,	$p_y = 220$	N/mm <sup>2</sup>	Parameter,	$\varepsilon = (275 / p_y)^{0.5} = 1.118$
Modulus of Elasticity,	$E = 200000$	N/mm <sup>2</sup>		
Overall width,	$B = 100$	mm	Overall depth,	$D = 150$ mm
Wall thickness,	$t = 6$	mm	Area,	$A = 28.2$ cm <sup>2</sup>
Moment of inertia,	$I_x = 862$	cm <sup>4</sup>	Moment of inertia,	$I_y = 456$ cm <sup>4</sup>
Section modulus,	$Z_x = 115$	cm <sup>3</sup>	Section modulus,	$Z_y = 91.2$ cm <sup>3</sup>
Plastic modulus,	$S_x = 141$	cm <sup>3</sup>	Plastic modulus,	$S_y = 106$ cm <sup>3</sup>

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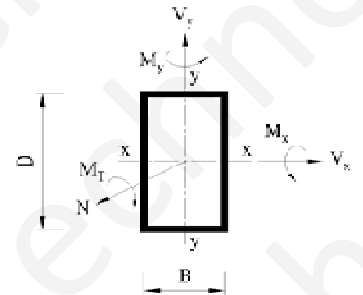
Torsional constant,	$J = 946$	$\text{cm}^4$	Radius of gyration,	$r_y = 4.02$	$\text{cm}$
Effective length,	$L_E = 2362$	$\text{mm}$			
Slenderness ratio,	$\lambda = L_E / r_y$				
	$\gamma_b = (1 - I_y / I_x) [1 - J / (2.6 I_x)]$				
Buckling index,	$\phi_b = [S_x^2 \gamma_b / (A J)]^{0.5}$				
Ratio,	$\beta_w = 1$				for plastic section
Equivalent slenderness,	$\lambda_{LT} = 2.25 (\phi_b \lambda \beta_w)^{0.5}$				
	$P_E = \pi^2 E / \lambda_{LT}^2$				
	$\alpha_{LT} = 7$				
	$\lambda_{L0} = 0.4 (\pi^2 E / p_y)^{0.5}$				
Perry factor,	$\eta_{LT} = \alpha_{LT} (\lambda_{LT} - \lambda_{L0}) / 1000$				$< 0$
	$\phi_{LT} = [p_y + (\eta_{LT} + 1) P_E] / 2$				
Bending buckling strength,	$p_b = P_E p_y / (\phi_{LT} + (\phi_{LT}^2 - P_E p_y)^{0.5})$				
	$= 270.82$	$\text{N/mm}^2$			
	$> 220$	$\text{N/mm}^2$			

## 5.2 Check for 6mm fillet weld all round connection to base plate

Vertical load = 7.818 kN (refer to item 5.1)

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


Breadth,	$B = 100$	$\text{mm}$
Height,	$D = 150$	$\text{mm}$
Area,	$A = 2(B + D)$	$= 500 \text{ mm}^2$
Moment of inertia,	$I_x = B D^2 / 2 + D^3 / 6$	$= 1687500 \text{ mm}^4$
Moment of inertia,	$I_y = B^2 D / 2 + B^3 / 6$	$= 916667 \text{ mm}^4$
Polar moment of inertia,	$J = I_x + I_y$	$= 2604167 \text{ mm}^4$



Shear load,	$V_x = 0$	$\text{kN}$	Moment,	$M_x = 0$	$\text{kNm}$
Shear load,	$V_y = 7.818$	$\text{kN}$	Moment,	$M_y = 0$	$\text{kNm}$
Tensile load,	$N = 0$	$\text{kN}$	Torsional moment,	$M_T = 0$	$\text{kNm}$

Leg length of fillet weld,  $t = 6 \text{ mm}$

Shear stress,	$\tau_x = V_x / (0.7 t A) + M_T (D / 2) / (0.7 t J)$		$= 0$	$\text{N/mm}^2$
Shear stress,	$\tau_y = V_y / (0.7 t A) + M_T (B / 2) / (0.7 t J)$		$= 3.72$	$\text{N/mm}^2$
Tensile stress,	$\sigma = N / (0.7 t A)$		$= 0$	$\text{N/mm}^2$
Tensile stress,	$\sigma_x = M_x D / (2 I_x) / (0.7 t)$		$= 0$	$\text{N/mm}^2$
Tensile stress,	$\sigma_y = M_y B / (2 I_y) / (0.7 t)$		$= 0$	$\text{N/mm}^2$

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Resultant ,

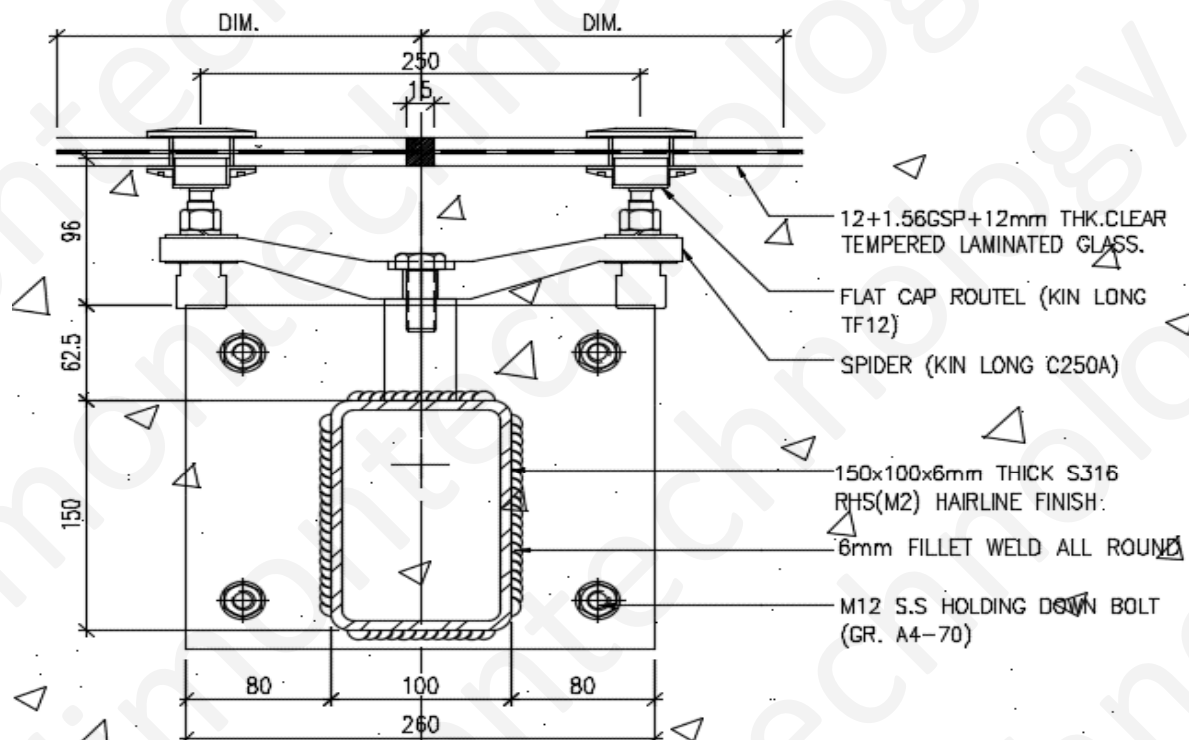
$$f_w = [\tau_x^2 + \tau_y^2 + (\sigma + \sigma_x + \sigma_y)^2]^{0.5}$$

$$= 3.72 \quad \text{N/mm}^2$$

$$\leq 200 \quad \text{N/mm}^2$$

O.K.

### 5.3 Check for M12 holding down bolt, A4-70



Vertical load = 7.818 kN (refer to item 5.1)

Shear load on each bolt = 7.818 / 4 = 1.95 kN

Tensile area of M12 bolt = 84 mm<sup>2</sup>  
Tensile strength of M12 bolt = 373 N/mm<sup>2</sup>  
Shear strength of M12 bolt = 280 N/mm<sup>2</sup>

Shear capacity of bolt = 84 x 280 / 1000 = 23.52 kN  
> 1.95 kN

O.K.