Examples

Example1 Basic wind pressure - calculation

A Power house building 25m high is to be designed in Darbhanga city. Compute the basic wind pressure.

Basic wind speed in Darbhanga (from appendix A)

P. 53 Code $V_b = 55$m/sec

An industrial building can be grouped under all general buildings and structures so should be designed for 50 years of design life

Risk coefficient from table 1. P. 11 code

$k_1 = 1$

Assuming the terrain is in city industrial area with numerous closely spaced obstructions. It can be grouped under category 3. P.8 code. Since the height of the building is 25m this falls under class B P.11 code. The terrain factor k_2 can be got from table 2 P.12 code. For category 3, class B interpolating between 20m and 30m

$k_2 = 1.005$

The ground is assumed to be plain so the topography factor k_3 is 1 + cs P. 56 code

where c = Z / L

Since the terrain assumed is plain. Read clause 5.3.3.1 P.12 code

$k_3 = 1$

Design wind speed $(V_z) = V_b\, k_1\, k_2\, k_3$

= 55 (1) (1.005) (1)

= 55.275 m/sec

Design wind pressure = $0.6\ V_Z^2$

$= 0.6\ (55.275)^2$

$= 1833.2\ N/m^2$

Example2

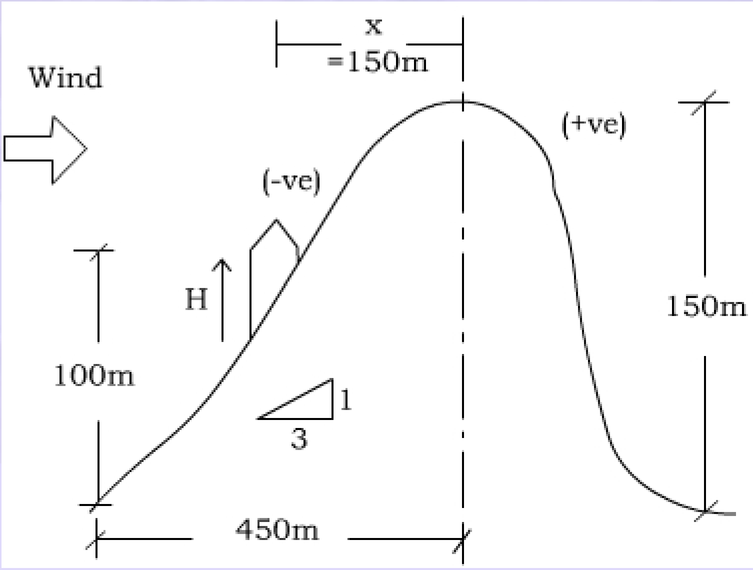
If the above building has to be constructed on a hillock where the height of the hill is 150m having a slope of 1:3 and the building is proposed at a height of 100m from the base on hte windward side, find the design wind

Basic wind speed at Darbhanga = 55m/sec

Risk coefficient k_1 =1

Terrain factor k_2 = 1.005

To find the topography factor k_3 Ref. appendix C. P. 56 code



Z = height of the hill (feather) = 150m

θ = slope in 3 $\tan^{-1}(1/3) = 18.43^o$

L = Actual length of upwind slope in the wind direction = 150(3) = 450m

L_e = Effective horizontal length of the hill for $\theta > 17^o$ $L_e = Z / 0.3 = 150 / 0.3 = 500m$

Values of C for $\theta = 18.43^o$ (i.e.) $> 17^o$

$C = 0.36$

Height of the building = 25m

To find x (i.e) the horizontal distance of the building from the crest measured +ve towards the leeward side and -ve towards the windward side.

$k_3 = 1 + cs$

To get s Fig 14 and 15 are used

$x = -150m$

$x / L_e = -150 / 500 = -0.3$ $H / L_e = 25 / 500 = 0.05$

Referring to figure 15 hill and ridge for $x / L_e = -0.3$ and $H / L_e = 0.05$ on the upwind direction

$s = 0.58$

$k_3 = 1 + (0.36)(0.58)$

$k_3 = 1.21$

Design wind speed $V_z = V_b k_1 k_2 k_3$

$= 55 (1) (1.005) (1.21)$

$= 66.9$ m/sec

Design wind pressure $P_Z = 0.6 V_Z^2$

$= 0.6 (66.9)^2$

$= 2685.4 N/m^2$

Example 3:

A memorial building is proposed at Sriperumbudur - Madras on a hill top. The size of the building is 40m x 80m and height is 10m. The hill is 300m high with a gradiant of 1 in 5. The building is proposed at a distance of 100m from the crest on the downwind slope. Calculate the design wind pressure on the building.

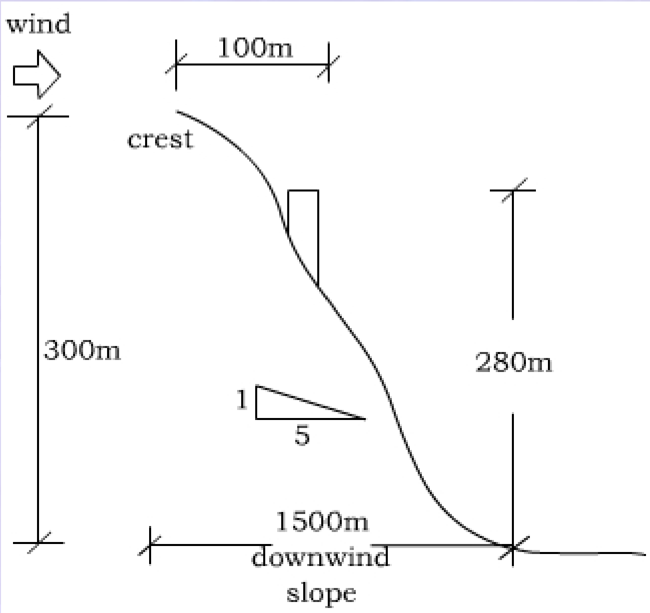
Basic wind velocity at madras is 50m/sec Ref. Appendix A. P.53 code

Risk coefficient k_{s1} = 1.08 for a memorial building of 100 years design life

Terrain factor k_2 for category 3 and class C since dimension of building 750m

k_2 = 0.82

Topography factor k_3



Z = effective height of the hill = 300m

θ = 1 in 5 $\tan^{-1}$ (1/5) = 11.31^o

L = Actual length of upward slope in the wind direction = 1500m

L_e = effective horizontal length of the hill

For θ = 11.31^o L_e = L = 1500m

Topography factor $k_3 = 1 + cs$

where $c = 1.2\ (Z/L)$ since $\theta = 11.31^o$ $3^o < \theta < 17^o$

$c = 1.2\ (300/1500) = 0.24$

x is the distance of the building from the crest + on downwind side

- on upward side $x = +100m$

The non dimensional factors are

$x / L_e = 100 / 1500 = 0.067$; $H / L_e = 10 / 1500 = 0.0067$

s = 1 from fig 15. P.57

$k_3 = 1 + (0.24)\ (1)$;

$k_3 = 1.24$

Design wind speed $V_z = V_b\ k_1\ k_2\ k_3$

$= 50\ (1.08)\ (0.82)\ (1.24)$

$= 54.91$ m/sec

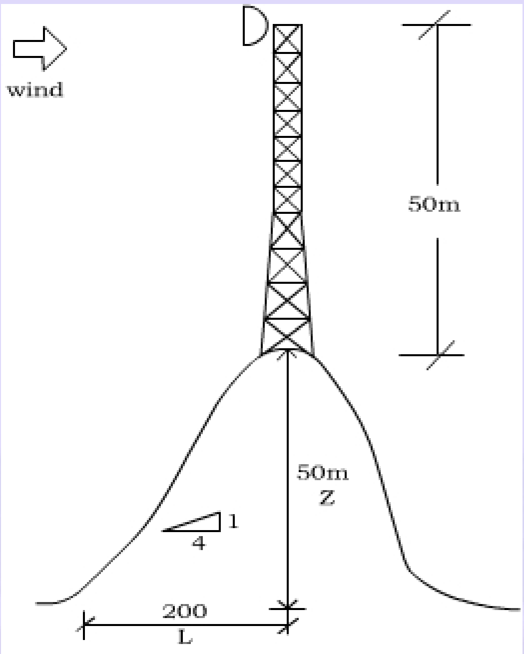
Design wind pressure $P_Z = 0.6\ V_Z^2$

$= 0.6\ (54.91)^2$

$= 1809.1\ N/m^2$

Example 4: Wind pressure on tower on a hill

A microwave tower of 50m height is proposed over a hill top. The height of the hill is 50m with a gradiant of 1 in 4. The terrain category is 3. The tower is proposed at coimbatore. Compute the design wind pressure:



Basic wind speed at CBE is 39m/sec

Risk factor $k_1 = 1.06$

Terrain factor (k_2) for category 3 class B - height between 20 and 50

$k_2 = 1.09$ table 2, P.12

Topography factor (k_3) Ref. P.56

Z - effective height of the hill = 50m

θ - slope 1 in 4 $\tan^{-1}(1/4) = 14.04^{o}$

L - Actual length of the upwind slope = 200m

L_e - Effective horizontal length of the hill $\theta = 14.04^{o} < 17$

$L_e = L = 200$m

$k_3 = 1 + cs$

$\theta < 17$, $c = 1.2\,(Z/L) = 1.2\,(50/200) = 0.3$

$x / L_e = 0/200 = 0$; $H / L_e = 50/200 = 0.25$

Ref. Fig.15 $s = 0.6$; $k_3 = 1 + (0.3)(0.6)$

$k_3 = 1.18$

Design wind speed $V_z = V_b\, k_1\, k_2\, k_3$

= 39 (1.06) (1.09) (1.18)

= 53.17 m/sec

Design wind pressure $P_Z = 0.6\ V_Z^2$

$= 0.6\ (53.17)^2$

$= 1696.23\ N/m^2$

Example 5:

If the 50m tower given in previous example is mounted with a hollow hemispherical dome of 2m diameter weighing 10kN. Compute the forces and stresses in members of various panels. The elevation of the tower is as shown below

Data given: Height of the tower = 50m

Base width = 6m

Top width = 2m

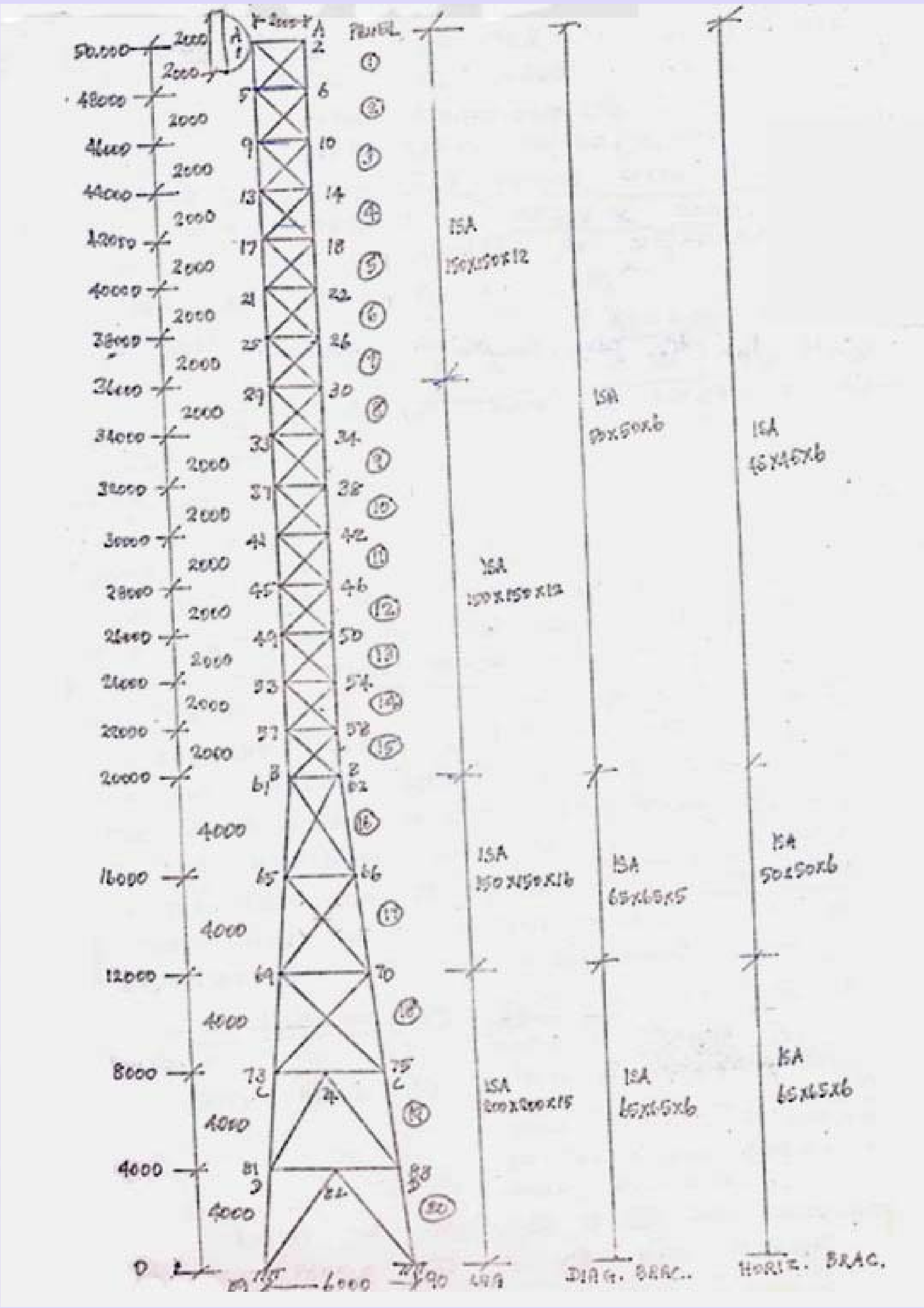
No. of panels = 20

Disk size = 2m diameter

Step 1: Wind force - From the previous example

Basic wind speed = 39m/sec

Risk coefficient (k_1) = 1.06



Topography factor (k_3) = 1.2

Terrain factor (k_2), varies with the height of the tower Ref, P.12 Table 2 code

The design wind pressures at different heights are computed as

$$P_Z = 0.6\ V_Z^2$$

$$= 0.6\ (39 \times 1.06 \times 1.2 \times k_2)^2$$

$$=1476.6\ k_2^2\ N/m^2$$

The values of k_2 at different height is chosen from Table 2

Step2: Basic assumptions:

1. Self weight of the members are equally distributed to the two joints connected by the members

2. No load is applied at the middle of the k-braced joint but allocated to column joint

3 Dead and wind loads are increased by 15% for each joints to account for Gussets, bolts and nuts

4. Secondary members are assumed to be provided in the panel where batter starts (below the waist level in our case panels 16 to 20. So an additional load of 10% is accounted for in the case of provision of secondary members

5. The wind loads on the members are equally distributed to the connecting joints.

Step3: Calculation of solidity ratios: Ref P.7 code

Solidity ratio for different panels are calculated

$$\text{Solidity ratio}\,(\phi) = \frac{\text{Projected area of all the individual elements}}{\text{Area enclosed by the boundary of the frame normal to the wind direction}}$$

Solidity ratios of panel 1 to 15 are calculated once as panels 1 to 15 are similar

$$\phi_{1-15} = \frac{15\,x\,2\left(2\,x\,0.15\right)+15\,x\,2\left(\sqrt{2}\,x\,2\,x\,0.05\right)+16\,x\,2\,x\,0.045}{30\,x\,2}$$

$$\phi_{1-15} = 0.245 \text{ Similarly for } \phi_{16}$$

$$\phi_{1-16} = \frac{2\,x\,4.04\,x\,0.15 + 2\,x\,4.68\,x\,0.065 + 2.8\,x\,0.05}{\left(\frac{2+2.8}{2}\right)x\,4}$$

$$\phi_{16} = 0.204$$

$$\phi_{17} = \frac{2\,x\,4.04\,x\,0.15 + 2\,x\,5.14\,x\;0.065 + 1\,x\,3.6\,x\,0.065}{\left(\frac{2+3.6}{2}\right)x\,4}$$

$$\phi_{17} = 0.165$$

$$\phi_{18} = \frac{2\,x\,4.04\,x\,0.2 + 2\,x\,5.67\,x\;0.065 + 1\,x\,4.4\,x\,0.065}{\left(\frac{3.6+4.4}{2}\right)x\,4}$$

$$\phi_{18} = 0.165$$

$$\phi_{19} = \frac{2\,x\,4.04\,x\,0.2 + 2\,x\,4.79\,x\;0.065 + 1\,x\,5.2\,x\,0.065}{\left(\frac{4.4+5.2}{2}\right)x\,4}$$

$$\phi_{19} = 0.134$$

$$\phi_{20} = \frac{2\,x\,4.04\,x\,0.2 + 2\,x5.016\,x\;0.065}{\left(\frac{5.2+6}{2}\right)x\,4}$$

$$\phi_{20} = 0.101$$

Step4 : Calculation of bowl wind pressure

Ref. Fig6 P.44 code. Bowl wind coeffs. are

c_f = 1.4 for wind from front

c_f = 0.4 for wind from rear

wind pressure at 50m above GL

Design wind pressure P_Z =1476.6 $(1.09)^2$

= 1.754 kN/m^2

Wind loads on dish are on front face F_{DISH} 1 = $c_f.A_e.p_d$

Ref. P.36 clause 6.3 code

F_{DISH} 1 = 1.4 x π/4 x 2_2 x 1.754

= 7.71 kN

On rear face

F_{DISH} 2 = 0.4 x π/4 x 2_2 x 1.754

= 2.20 kN

Step5:

The terrain factor (k_2), the solidity ratio and the design wind pressures at various heights are tabulated as shown - category 3 class B

Panel from top	Height in 'm' from top	Terrain size,HT. coeff. k_2	Design wind pressure P_Z =1476.6 (k_2^2) N/m^2	Solidity ratio	Overall force coeff. c_f Table30 P.47	$P_Z . c_f$ N/m^2
1 to 5	10	1.09 = 1.075 1.06	1706.4	0.245	3.075	5247.2
6 to 10	20	1.06 = 1.045 1.03	1612.5	0.245	3.075	4958.4
11 to 15	30	1.03 = 1.005 0.98	1491.4	0.245	3.075	4586.1
16	34	0.98 = 0.964 0.948	1372.2	0.204	3.28	4500.8
17	38	0.948 = 0.926 0.904	1266.1	0.165	3.475	4399.7
18	42	0.904 = 0.88 0.856	1143.5	0.165	3.475	3975.7
19	46	0.856 = 0.832 0.808	1022.1	0.134	3.630	3710.2
20	50	0.808	964.0	0.101	3.795	3658.4

Step6: Calculation of forces at different joints

The forces from the dish are transferred to two top most joints 1 and 4. The dish weight and wind force on the dish are equally distributed at the two joints.

Panel 1 Leg: Length of the leg = 2m

Width of the leg = 0.15m

Since 4 Nos of ISA 150 x 150 x 12 @ 0.272 kN/m

Self weight of legs = 4 x 2 x 0.272 = 2.176 kN

No. of legs exposed to wind = 2

Wind obstruction area = 2 x 2 x 0.15

= 0.6 m^2

wind load on leg = 0.6 x 5247.2

= 3.148 kN

Diagonal bracing : No. of diagonal bracings = 8

No. of obstructing wind = 2

Size of diagonal bracing ISA 50 x 50 x 6 @ 0.045 kN/m.

Self weight = 8 x $\sqrt{2}$ x 2 x 0.045

= 1.018 kN

Wind obstruction area = 2 x $\sqrt{2}$ x 2 x 0.05

= 0.283 m^2

Wind load on diag. Brac = 0.283 x 5247.2

= 1.485 kN

Horizontal bracing: ISA 45 x 45 x 6

No. of horizontal bracings = 8

No. of obstructing wind = 2

Self weight of horizontal bracing = 8 x 2 x 0.04

= 0.64 kN

Wind obstruction area = 2 x 2 x 0.045

= 0.18m^2

Wind load on horizontal brac = 0.18 x 5247.2

= 0.945 kN

Total self weight of leg, diag. brac and horizontal brac

F_v = 2.176 + 1.018 + 0.64 = 3.834 kN

Total wind load on leg, diag and Hor. bracs

F_H = 3.148 + 1.485 + 0.945 = 5.578 kN

These load are to be distributed to all the 8 joints connecting the elements (i.e. joints 1 to 8)

Load at each joint is increased by 15% to account for gussets, bolts and washers

F_{v1} vertical load on joints 1 to 8 = 1.15 x 3.834 / 8

= 0.551 kN

F_{H1} wind load on joints 1 to 8 = 1.15 x 5.576 / 8

= 0.802 kN

The self weight of the dish is shared by joints 1 and 4

$F_{V\ DISH}$ = 10/2 kN = 5kN

Wind load on the dish is shared by joints 1, 2, 3 and 4, $F_{H\ DISH}$ = 7.71 / 4 = 1.93 kN

Panel 2: Self weight of legs = 2.176 kN

wind load on legs = 3.148 kN

Self weight of diag. Bracs = 1.018 kN

Wind load on Diag. Brac = 1.485 kN

No. of horizontal bracings = 4

No. of obstructing wind = 4

Self weight of horizontal bracing = 4 x 2 x 0.04

= 0.32 kN

Wind obstruction area = 1 x 2 x 0.045

= 0.09 m^2

Wind load on hor. brac. = 0.09 x 5247.2 = 472.2 N

Vertical load due to leg and diag. brac carried by joints 5 to 12 = 1.15 (2.176 + 1.018) / 8

= 0.46 kN

Vertical load due to hor.brac. carried by joints 9, 10, 11 and 12 = 1.15 x (0.32)/4 = 0.092 kN

Wind load carried by joints 5 to 12 = 1.15 (3.148 + 1.485) / 8

= 0.666 kN

Wind load carried by joints 9, 10, 11 and 12 = 1.15 x 0.472/4

= 0.136 kN

Computation of loads at different joints are made for panel to panel from panel 2 to panel 5 are tabulated

Panel 6: Self weight of legs = 4 x 2 x 0.272 = 2.176 kN

Wind load = 0.6 x 4958.4 = 2.975 kN

Self weight of Diag. Brac. = 1.018 kN

Wind load = 0.283 x 4958.4 = 1.403 kN

Self weight of hor. bracings = 0.32 kN

Wind load = 0.09 x 4958.4 = 0.446 kN

Vertical load carried by joints 21 to 28 = (2.176 + 1.018) 1.15 / 8

= 0.46 kN

Wind load carried by joints 21 to 28 = (2.975 + 1.403) 1.15 / 8

= 0.63 kN

Vertical load due to Hor. Brac. carried by joints 25, 26, 27 and 28 = 1.15 x (0.32)/4

= 0.092 kN

Wind load carried by joints 25, 26, 27 and 28 = 1.15 x (0.446)/4

= 0.128 kN

Computations of loads at different joints were done from 6 to 10 and are tabulated.

Panel 11: Vertical load carried by joints 41 to 48 = 0.46 kN

Wind load on the legs = 0.6 x 4586.1

= 2.75 kN

Wind load on the Diag. Brac. = 0.283 x 4586.1

= 1.3 kN

Vertical load due to Hor. Brac carried by joints 45, 46, 47 and 48 = 0.092 kN

Wind load carried by joints 41 to 48 = 1.15 (2.75 + 1.3)/8

= 0.582 kN

Wind load carried by joints 45 to 48 due to Hor. Brac. = (0.09 x 4586.1)/4

Computation of loads at different joints were done from panel 11 to 15 and are tabulated

Panel 16: Leg: ISA 150 x 150 x 15 @ 0.336 kN/m

Length of the leg (L) = 4.04m

Width of the leg (B) = 0.15m

Self weight of legs = 4 x 4.04 x 0.336

= 5.43 kN

No. of legs exposed to wind = 2

Wind obstruction area = 2 x 4.04 x 0.15

= 1.212 m^2

Wind load on leg = 1.212 x 4500.8

= 5.454 kN

Diag. Brac: ISA 65 x 65 x 5 @ 0.049 kN/m

No. of bracing = 8

No. of obstructing wind = 2

Self weight of diagonal brac. = 8 x 4.68 x 0.049

= 1.835 kN

Wind obstruction area = 2 x 4.68 x 0.065

= 0.6084 m^2

Wind load on Diag. Brac = 0.6084 x 4500.8

= 2.74 kN

Horizontal Brac: ISA 65 x 65 x 5 @ 0.045 kN/m

No. of bracing = 4

No. of obstructing wind = 1

Self weight of Hor. brac. = 4 x 2.8 x 0.045

= 0.504 kN

Wind obstruction area = 1 x 2.8 x 0.050

= 0.14 kN

Wind load on Hor. Brac = 0.14 x 4500.8

= 0.63 kN

Secondary bracings are accounted for so DL and WL is increased by 10%

Vertical load carried by joints 61 to 68 = (1.25 / 5.43 + 1.835)/8

= 1.135 kN

Vertical load carried by joints 65 to 68 due to Hor. Brac. = 1.25 (0.504)/4

= 0.158 kN

Wind load carried by joints 61 to 68 = 1.25 (5.454 + 2.74)/8

= 1.28 kN

Wind load carried by joints 65 to 68 due to Hor. Brac = 1.25 (0.63) / 4

= 0.197 kN

Panel 17: Leg: ISA 150 x 150 x 16 @ 0.336 kN/m

Self weight of legs = 4 x 4.04 x 0.336

= 5.43 kN

Wind obstruction area = 2 x 4.04 x 0.15

= 1.212 m^2

Wind load on leg = 1.212 x 4399.7

= 5.332 kN

Diag. Brac: ISA 65 x 65 x 5 @ 0.049 kN/m

Self weight of diagonal brac. = 8 x 5.14 x 0.049

= 2.015 kN

Wind obstruction area = 2 x 5.14 x 0.065

= 0.6682 m^2

Wind load on Diag. Brac = 0.6682 x 4399.7

= 2.94 kN

Horizontal Brac: ISA 65 x 65 x 6 @ 0.058 kN/m

Self weight of Hor. brac. = 4 x 3.6 x 0.058

= 0.835 kN

Wind obstruction area = 1 x 3.6 x 0.065

= 0.234 m^2

Wind load on Hor. Brac = 0.234 x 4399.7

= 1.03 kN

Secondary bracings should be accounted for in this panel

Vertical load carried by joints 69 to 72 = 1.25 (5.43 + 2.015)/8

= 1.163 kN

Vertical load carried by (Due to horizontal brac.) joints 69 to 72 = 1.25 (0.835)/4

= 0.261 kN

Wind load carried by joints 65 to 72 = 1.25 (5.332 + 2.94)/8

= 1.29 kN

Wind load carried by joints 69 to 72 due to Hor. Brac = 1.25 (1.03) / 4

= 0.332 kN

Panel 18 : Leg: ISA 200 x 200 x 15 @ 0.454 kN/m

Self weight of legs = 4 x 4.04 x 0.454

= 7.34 kN

Wind obstruction area = 2 x 4.04 x 0.2

= 1.616 m^2

Wind load on leg = 1.616 x 3973.7

= 6.42 kN

Diag. Brac: ISA 65 x 65 x 6 @ 0.058 kN/m

Self weight of diagonal brac. = 8 x 5.67 x 0.058

= 2.63 kN

Wind load on Diag. Brac = 2 x 5.67 x 0.065 x 3973.7

= 2.93 kN

Horizontal Brac: ISA 65 x 65 x 6 @ 0.058 kN/m

Self weight of Hor. brac. = 4 x 4.4 x 0.058

= 1.02 kN

Wind load on Hor. Brac = 1 x 4.4 x 0.065 x 3973.7

= 1.14 kN

Vertical load carried by joints 69 to 79 except 74, 76, 78, 80 = 1.25 (7.34 + 2.68)/8

= 1.56 kN

Vertical load carried by joints 73, 75, 77, 79 (Due to horizontal brac.) = 1.25 (1.02)/4

= 0.32 kN

Wind load carried by joints 65 to 79 except 74, 76, 78, 80 = 1.25 (6.42 + 2.93)/8

= 1.46 kN

Wind load carried by joints 73, 75, 77, 79 due to Hor. Brac = 1.25 (1.14) / 4

= 0.356 kN

Panel 19: Leg: ISA 200 x 200 x 15 @ 0.454 kN/m

Self weight of legs = 4 x 4.04 x 0.454

= 7.34 kN

Wind load on leg = 2 x 4.04 x 0.2 x 3710.2

= 6 kN

Diag. Brac: ISA 65 x 65 x 6 @ 0.058 kN/m

Self weight of diagonal brac. = 8 x 4.79 x 0.058

= 2.22 kN

Wind load on Diag. Brac = 2 x 4.79 x 0.065 x 3710.2

= 2.31 kN

Horizontal Brac: ISA 65 x 65 x 6 @ 0.058 kN/m

Self weight of Hor. brac. = 4 x 5.2 x 0.058

= 1.21 kN

Wind load on Hor. Brac = 1 x 5.2 x 0.065 x 3710.2

= 1.254 kN

Vertical load carried by joints 73 to 88 except 74, 76, 78, 80, 82, 84, 86, 88 = 1.25 (7.34 + 2.22)/8

= 1.494 kN

Vertical load carried by joints 81, 83, 85, 87 (Due to horizontal brac.) = 1.25 (1.21)/4

= 0.378 kN

Wind load carried by joints 73, 75, 77, 79, 81, 83, 85, 87 = 1.25 (6 + 2.31)/8

= 1.3 kN

Wind load carried by joints 81, 83, 85, 87 due to Hor. Brac = 1.25 (1.254) / 4

= 0.392 kN

Panel 20: Leg: ISA 200 x 200 x 15 @ 0.454 kN/m

Self weight of legs = 4 x 4.04 x 0.454

= 7.34 kN

Wind load on leg = 2 x 4.04 x 0.2 x 3658.4

= 5.91 kN

Diag. Brac: ISA 65 x 65 x 6 @ 0.058 kN/m

Self weight of diagonal brac. = 8 x 5.02 x 0.058

= 2.33 kN

Wind load on Diag. Brac = 2 x 5.02 x 0.065 x 3658.4

= 2.39 kN

Vertical load carried by joints 81, 83, 85, 87, 89, 90, 91, 92 = 1.25 (7.34 + 2.33)/8 = 1.51 kN

Wind load carried by joints 81, 83, 85, 87, 89, 90, 91, 92 = 1.25 (5.91 + 2.39)/8 = 1.3 kN

Computation of loads at different joints are made panel by panel and the nodal loads are superposed and tabulated in the following sections. The tower is symmetrically loaded in the XY plane and so nodal loads are tabulated for joints which are in the front plane.

Calculation of forces in the members

By symmetry the two planes are identical the front plane is analysed and forces are resolved. The tower is analysed for three basic static loads

- Self weight of the tower
- Superimposed load from Hemispherical Dome
- Wind Loads

- Acting parallel to face
- Acting diagonal to the tower

Tabulation of joint forces

Joint No	Self WT.(kN)	Wind load (kN)	Joint No	Self WT (kN)	Wind load (kN)
1	5 + 0.551 = 5.551	0.802 + 1.93 = 2.732	2	0.551	0.802 + 1.93 = 2.732
5	0.551 + 0.46 = 1.011 6.562	0.802 + 0.666 = 1.468	6	0.551 + 0.46 = 1.011 1.562	0.802 + 0.666 = 1.468
9	0.46 + 0.092 + 0.46 = 1.012 7.574	0.666 + 0.136 + 0.666 = 1.468	10	0.46 + 0.092 + 0.46 = 1.012 2.574	0.666 + 0.136 + 0.666 = 1.468
13	0.46 + 0.092 + 0.46 = 1.012 8.586	0.666 + 0.136 + 0.666 = 1.468	14	0.46 + 0.092 + 0.46 = 1.012 3.586	0.666 + 0.136 + 0.666 = 1.468
17	0.46 + 0.092 + 0.46 = 1.012 9.598	0.666 + 0.136 + 0.666 = 1.468	18	0.46 + 0.092 + 0.46 = 1.012 4.598	0.666 + 0.136 + 0.666 = 1.468
21	0.46 + 0.092 + 0.46 = 1.012 10.61	0.666 + 0.136 + 0.63 = 1.432	22	0.46 + 0.092 + 0.46 = 1.012 5.61	0.666 + 0.136 + 0.63 = 1.432
25	0.46 + 0.092 + 0.46 = 1.012 11.622	0.63 + 0.128 + 0.63 = 1.388	26	0.46 + 0.092 + 0.46 = 1.012 6.622	0.63 + 0.128 + 0.63 = 1.388
29	0.46 + 0.092 + 0.46 = 1.012 12.634	0.63 + 0.128 + 0.63 = 1.388	30	0.46 + 0.092 + 0.46 = 1.012 7.634	0.63 + 0.128 + 0.63 = 1.388
33	0.46 + 0.092 + 0.46 = 1.012 13.646	0.63 + 0.128 + 0.63 = 1.388	34	0.46 + 0.092 + 0.46 = 1.012 8.646	0.63 + 0.128 + 0.63 = 1.388
37	0.46 + 0.092 + 0.46 = 1.012 14.658	0.63 + 0.128 + 0.63 = 1.388	38	0.46 + 0.092 + 0.46 = 1.012 9.658	0.63 + 0.128 + 0.63 = 1.388
41	0.46 + 0.092 + 0.46 = 1.012 15.67	0.63 + 0.128 + 0.63 = 1.34	42	0.46 + 0.092 + 0.46 = 1.012 10.67	0.63 + 0.128 + 0.63 = 1.34
45	0.46 + 0.092 + 0.46 = 1.012	0.582 + 0.103 + 0.582 = 1.267	46	0.46 + 0.092 + 0.46 = 1.012	0.582 + 0.103 + 0.582 =

	16.682			11.682	1.267
49	0.46 + 0.092 + 0.46 = 1.012 17.694	0.582 + 0.103 + 0.582 = 1.267	50	0.46 + 0.092 + 0.46 = 1.012 12.694	0.582 + 0.103 + 0.582 = 1.267
53	0.46 + 0.092 + 0.46 = 1.012 18.706	0.582 + 0.103 + 0.582 = 1.267	54	0.46 + 0.092 + 0.46 = 1.012 13.706	0.582 + 0.103 + 0.582 = 1.267
57	0.46 + 0.092 + 0.46 = 1.012 19.718	0.582 + 0.103 + 0.582 = 1.267	58	0.46 + 0.092 + 0.46 = 1.012 14.718	0.582 + 0.103 + 0.582 = 1.267
61	0.46 + 0.092 + 1.135 = 1.687 21.405	0.582 + 0.103 + 1.28 = 1.965	62	0.46 + 0.092 + 1.135 = 1.687 16.405	0.582 + 0.103 + 1.28 = 1.965
65	1.135 + 0.158 + 1.163 = 2.456 23.861	1.28 + 0.197 + 1.29 = 2.767	66	1.135 + 0.158 + 1.163 = 2.456 18.861	1.28 + 0.197 + 1.29 = 2.767
69	1.163 + 0.261 + 1.56 = 2.984 26.845	1.29 + 0.322 + 1.46 = 3.072	70	1.163 + 0.261 + 1.56 = 2.984 21.845	1.29 + 0.322 + 1.46 = 3.072
73	1.56 + 0.32 + 1.494 = 3.374 30.219	1.46 + 0.356 + 1.3 = 3.116	75	1.56 + 0.32 + 1.494 = 3.374 25.219	1.46 + 0.356 + 1.3 = 3.116
81	1.494 + 0.378 + 1.51 = 3.382 33.601	1.3 + 0.392 + 1.3 = 2.99	83	1.494 + 0.378 + 1.51 = 3.382 28.601	1.3 + 0.392 + 1.3 = 2.99
89	1.51 35.111	1.3	90	1.51 30.111	1.3

Panel 15: 1. Considering self weight of the tower

The leg ISA 150 x 150 x 12 will be maximum stressed in this panel. So this panel is chosen. The self weight acting on joints 61 and 62 is taken.

The leeward leg 2 will be in compression and also the windward leg 1

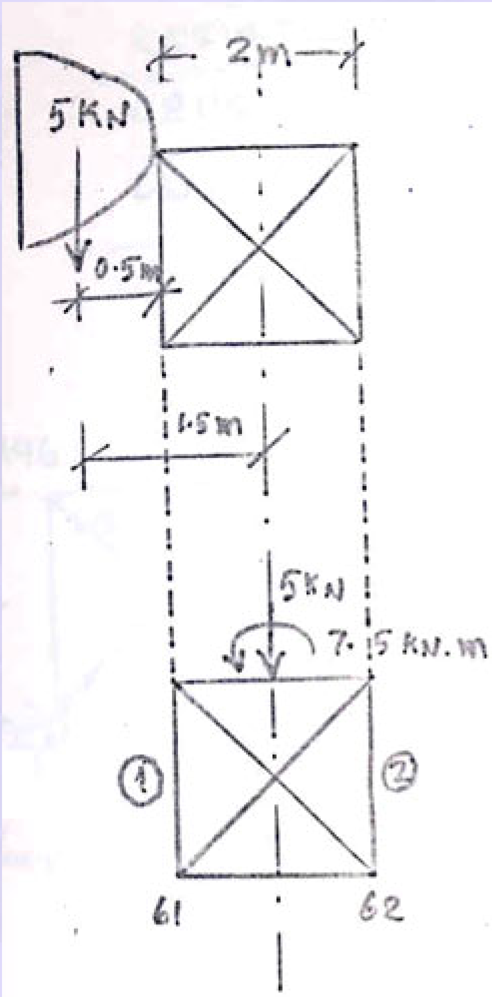
$F_1 = F_2 = 16.405$ kN (compression)

2. Considering superimposed load from hemispherical dome:

The front plane takes half the self weight = 5kN

The self weight of the dome will create a moment with respect to centre of planar truss. The eccentric load of 5 kN is transferred as a concentric load of 5 kN acting at the centre of planar truss and an anticlockwise moment of 7.5 kN.m as shown. Due to self weight both the legs F_1 and F_2 will be in compression

$$F_1 = F_2 = 2.5 \text{ kN (compression)}$$



The moment will cause compression on the windward side and tension on the leeward side.

$F_1 = 7.5 / 2 = 3.75$ kN (compression)

$F_2 = 7.5 / 2 = 3.75$ kN (tension)

Net force on F_1 = 3.75 + 2.5 = 6.25 kN (compression)

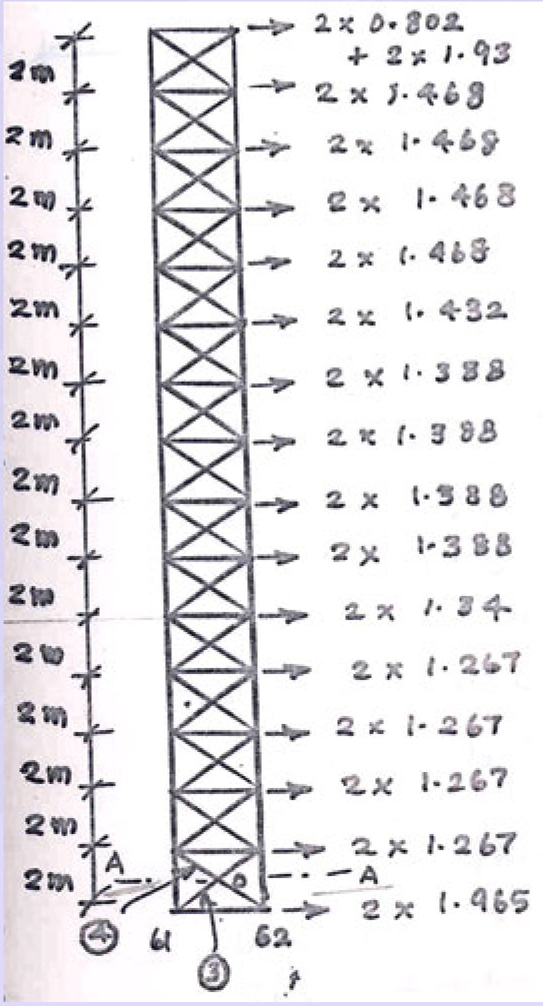
Net force on F_2 = -3.75 + 2.5 = 1.25 kN (tension)

The moment due to dome and self weight are carried entirely by legs.

3. Considering wind load condition

(i) Wind parallel to the face of the frame

The sum of the wind forces upto panel 15 and also the bending moment due to wind load about point 0 (the point of intersection of Diag. Brac.) is taken



Total wind load above the level 'AA'

F_{LAT1} = 2 x 0.802 + 2 x 1.93 + 4 x 2 x 1.468 + 2 x 1.432 + 4 x 2 x 1.388 + 2 x 1.34 + 4 x 2 x 1.267

F_{LAT1} = 43.992 kN

Moment due to wind

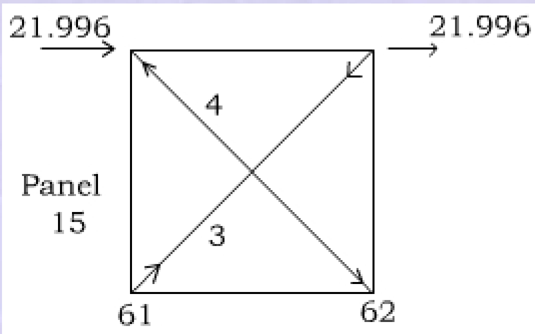
M_{W1} = (1.604 + 3.86) x 29 + 2.936 x 27 + 2.936 x 25 + 2.936 x 23 + 2.936 x 21 + 2.864 x 19 + 2.776 (17 + 15 + 13 + 11) + 2.68 x 9 + 2.534 (7 + 5 + 3 + 1)

M_{W1} = 714.85 kN.m

This external wind moment has to be resisted by internal couple. this moment will cause tension of the windward leg and comp on the leeward leg

$F_1 = M_{W1} / 2$ = 714.85 / 2 = 357.43 kN

F_1 = 357.43 kN (tension) F_2 = 357.43 kN (compression)



The lateral force of 43.992 kn is shared by the diagonal bracings equally and the tension diagonal is considered as effective taking moment about joint 62

$43.992 = \sqrt{2}\, F_3$

F_3 = 31.11 kN tension

F_4 = 31.11 kN compression

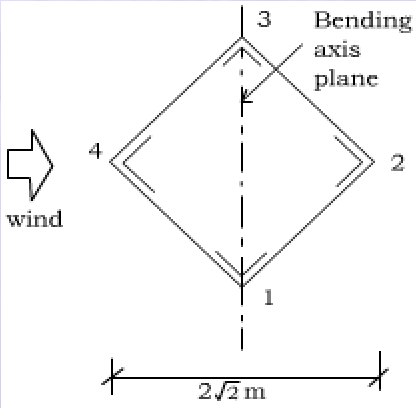
(ii) Wind wards acting along diagonal:

when the wind is parallel to the diagonal, the wind pressure coeff. is taken 1.2 times that of parallel to the plane Ref. clause 6.3.3.5 P.47 - IS 875

However the wind pressure on hte dish is reduced as the wind is at 45^o to the front of the dish.

Wind pressure on the dish = 2 x 3.86 x Sin 45^o

= 5.46 kN



Considering the tower as a space frame:

The wind load on the four joints together can be obtained. By multiplying the loads by 1.2

So total horizontal load due to wind

$F_{LAT\ 2}$ = 5.46 + 1.2 x 2 (43.992 - 3.86)

$F_{LAT\ 2}$ = 101.78 kN

Similarly the bending moment of all the wind forces along the diagonal about point 0

M_{W2} = 1.2 x 2 {714.85 - (3.86 x 29)} + 5.46 x 29

M_{W2} = 1605.32 kN.m Since the legs are upright, the horizontal force is registered by the braces and the forces in the braces will be equal and opposite.

The forces have to be resolved in the horizontal plane and then parallel to the diagonal.

Let F_D = force in each brace (tension or compression)

The total force from braces in the horizontal plane along the tower diagonal is

$= 8\ F_D \cos 45^o . \sin 45^o$

$= 4\ F_D$

Equilibrium in the horizontal direction gives

$4\ F_D = 101.78$ kN

$F_D = 25.45$ kN

This value is less than that of case 1. Therefore the forces in braces are controlled by the load condition wind parallel to the frame. The bending moment is resisted by the pair of extreme legs 2 and 4. Forces in legs 3 and 1 will be zero as they lie in the bending axis Ref. Fig.

$F_1 = F_3 = 0$

$F_2 = M_{W2} / 2\sqrt{2} \qquad = 1605.32 / 2\sqrt{2}$

$F_2 = 567.57$ kN (compression)

$F_4 = 567.57$ kN (tension)

Maximum compressive force on the leg = 567.57 + 16.405 - 1.25

= 582.73 kN

Leg ISA 150 x 150 x 12 @ 0.272 kN / m

$A = 3459\ mm^2$; $r_{min} = 29.3$ mm

$L_{eff} = 0.85 \times 2000 = 1700$mm; $L_{eff} / r_y = 1700 / 29.3 = 58.02$

σ_{ac} from table 5.1 = 124 N/mm^2 can be raised by 25%. Since wind is considered: σ_{ac} = 1.25 x 124 = 155 N/mm^2

Actual stress σ_c = (582.73 x 10^3) / 3459 = 168.5 N/mm^2

Diag. Brac: The tension member is considered effective.

Force in the bracing = 31.11 kN

Size ISA 50 x 50 x 6 mm A = 568 mm^2

Check the adequacy of the section as a tension member

Panel 20: Leg: ISA 200 x 200 x 15 @ 0.454 kn/m

1. Self weight acting at the bottom most panels

$F_1 = F_2$ = 30.111 kn (compression)

The leg is checked at the mid height as buckling will occur midway between the nodes

2. Considering superimposed load from hemispherical dome

Due to moment F_1 = 7.5 / 5.6 = 1.34 kn (compression)

F_2 = 1.34 kN (tension)

Due to self weight F_1 = 2.5 kN (compression)

F_2 = 2.5 kN (compression)

Net forces F_1 = 1.34 + 2.5 = 3.84 kN (compression)

F_2 = -1.34 + 2.5 = 1.16 kN (compression)

3. Considering wind load condition:

(a) Wind parallel to the face of the frame:

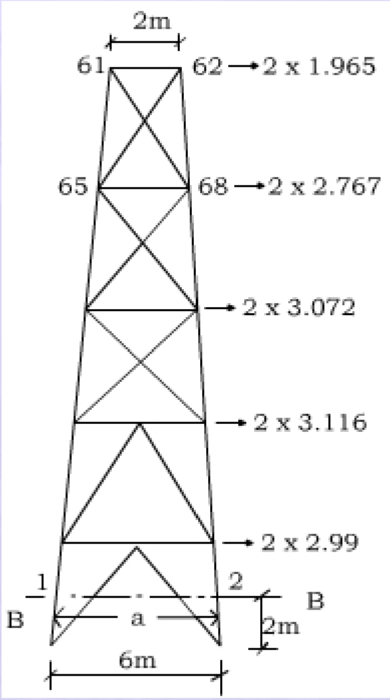
Total wind load above level 'BB'

$F_{LAT\ 3}$ = 43.992 + 2 x 1.965 + 2 x 2.767 + 2 x 3.072 + 2 x 3.116 + 2 x 2.99

$F_{LAT\ 3}$ = 71.812 kN

M_{W3} = (1.604 + 3.86) x 48 + 2.936 (46 + 44 + 42 + 40) + 2.864 x 38 + 2.776 (36 + 34 + 32 + 30) + 2.68 x 28 + 2.534 (26 + 24 + 22 + 20) + 3.93 x 18 + 5.534 x 14 + 6.144 x 10 + 6.232 x 6 + 5.98 x 2

M_{W3} = 1809.704 kN.m



Force in the legs and braces

$F_1 = M_{W3}$ / a = 1809.704 / 5.6 = 323.16 kN

F_1 = 323.16 kN (tension)

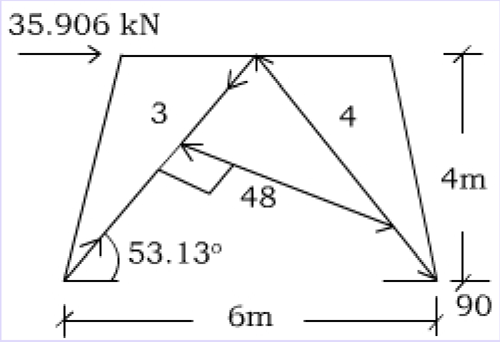
F_2 = 323.16 kN (compression)

The lateral force of 71.812 kN is shared by the diagonal bracings equally and the tension diagonal is considered effective taking moment about joint 90

35.906 x 4 = F_3 x 4.8

F_3 = 29.92 kN (tension)

F_4 = 29.92 kN (compression)



(b) Wind acting parallel to the diagonal:

Wind load is increased by 1.2 times that of parallel to the frame. P.47 code. However wind pressure on the dish is reduced as the wind is 45^o to the front of the dish

Wind pressure on dish = 5.46 kN

Considering the tower as a space frame the wind load on the four joints together can be obtained by multiplying the load by 1.2

So, total horizontal load due to wind

$F_{LAT\ 4}$ = 5.46 + 1.2 x 2 (71.812 - 3.86)

$F_{LAT\ 4}$ = 168.55 kN

Similarly the bending moment of all the wind forces along section 'BB'

M_{W4} = 1.2 x 2 {1809.704 - (3.86 x 48)} + 5.46 x 48

M_{W4} = 4160.7 kN.m

The horizontal forces are resisted by the braces these forces have to be resolved in the horizontal plane and then parallel to the diagonal.

Let F_d be the force in each brace tension or compression. The total force is resisted by these 8 braces

$4F_d \cos 53.13^o (\cos 37.47^o + \cos 52.59^o) = 168.55$

F_d = 50.12 kN (tension or compression)

This is more than the value with wind parallel to the frame. The bending moment M_{W4} is resisted by the pair of extreme legs which does not lie on the bending axis

$F_1 = F_3 = 0$

$F_2 = M_{W4} / a\sqrt{2} = 4160.7 / 5.6\sqrt{2} = 525.4$ kN

$F_2 = 525.4$ kN (compression)

$F_4 = 525.4$ kN (tension)

Maximum compressive force will be on leg 2

$= 30.111 + 1.16 + 525.4$

$F_2 = 556.67$ kN (compression)

Leg ISA 200 x 200 x 15 @ 0.454 kN/m

$A = 5780$ mm^2; $r_y = 39.1$ mm

$L_{ef} = 0.85 \times 4040 = 3434$mm

$L_{ef} / r_y = 3434 / 39.1 = 87.83$ Refer Table 5.1

$\sigma_{ac} = 86$ N / mm^2

Since wind is considered allowable stresses are raised by 25%. So $\sigma_{ac} = 1.25 \times 86 = 107.5$ N / mm^2

Actual stress $\sigma_c = 556.67 / 5780 = 96.31$ N / mm^2

σ_{ac} and σ_c Safe