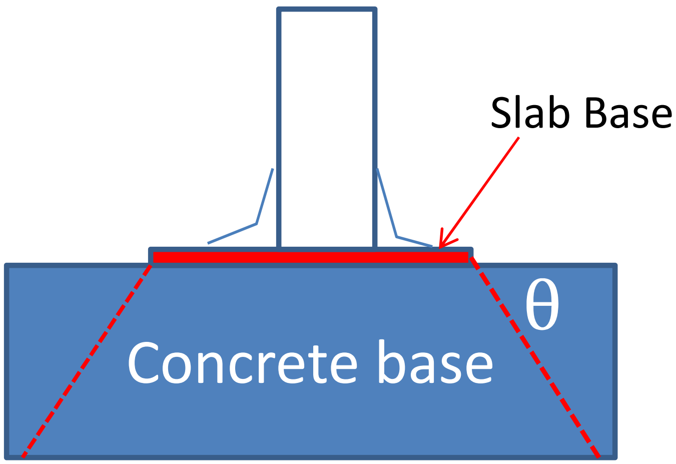
DESIGN OF
STEEL STRUCTURES

Design of slab and Gusset bases:-

Slab Base or Base Plate:-

- It is a steel plate placed between column base and concrete base.



Area of Slab base = $\dfrac{P}{\sigma_c}$

P – Axial load in the column

σ_c - Permissible compressive stress in concrete

$$\theta \leq \tan^{-1}\left(0.9\sqrt{\frac{100\, q^0}{f_{ck}} + 1}\right)$$

where

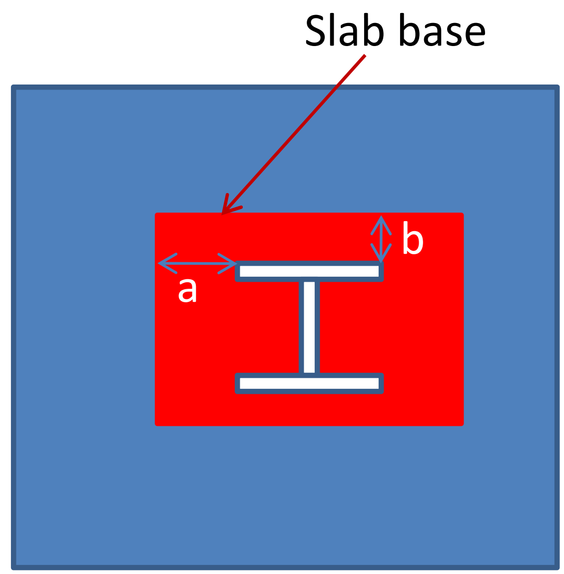
q^0 = calculated maximum bearing pressure at the base of the pedestal in N/mm^2

f_{ck} = characteristic strength of concrete at 28 days in N/mm^2.

➢ Thickness of a rectangular slab base

From, IS 800 : 1984, Clause 5.4.3, Pg - 44

$$t = \sqrt{\frac{3}{\sigma_{bs}} w \left(a^2 - \frac{b^2}{4} \right)} mm$$



where

t = the slab thickness, in mm;

w = the pressure or loading on the underside of the base in MPa

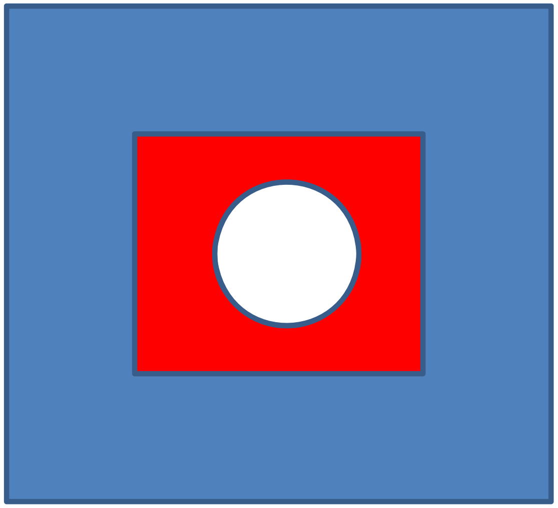
a = the greater projection of the plate beyond column in mm

b = the lesser projection of the plate beyond the column in mm

σ_{bs} = the permissible bending stress in slab bases (for all steels, shall be assumed as 185 MPa).

➢ Thickness of a square slab base under solid circular column:

$$t = 10\sqrt{\frac{90W}{16\sigma_{bs}} \times \frac{B}{(B - d_0)}}\, mm$$



where

t = the thickness of the plate, in mm.

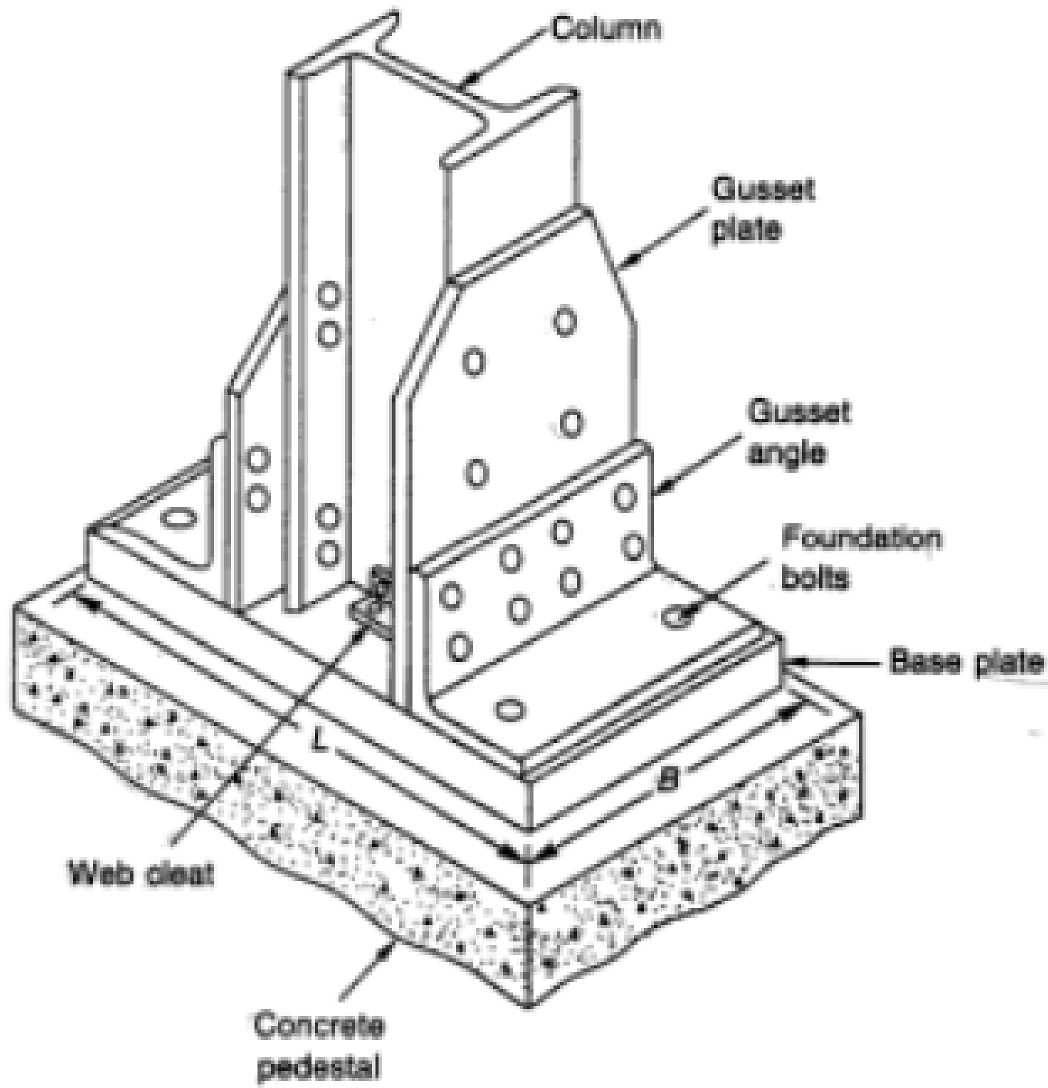
W = the total axial load, in KN

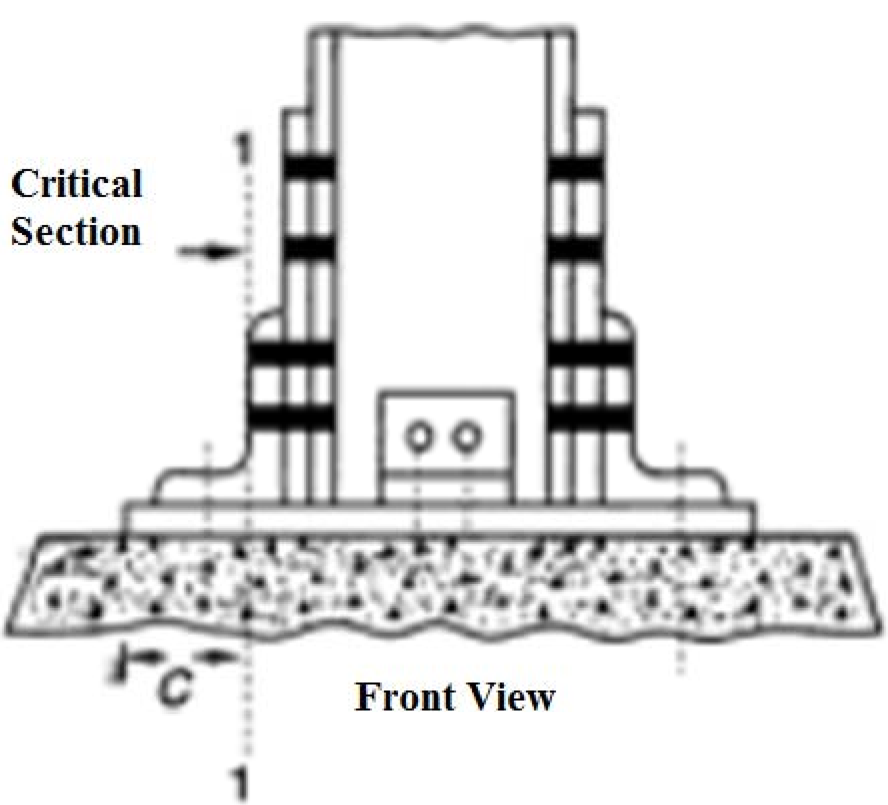
B = the length of the side of cap or base, in mm

σ_{bs} = the permissible bending stress in slab bases (for all steels, shall be assumed as 185 MPa)

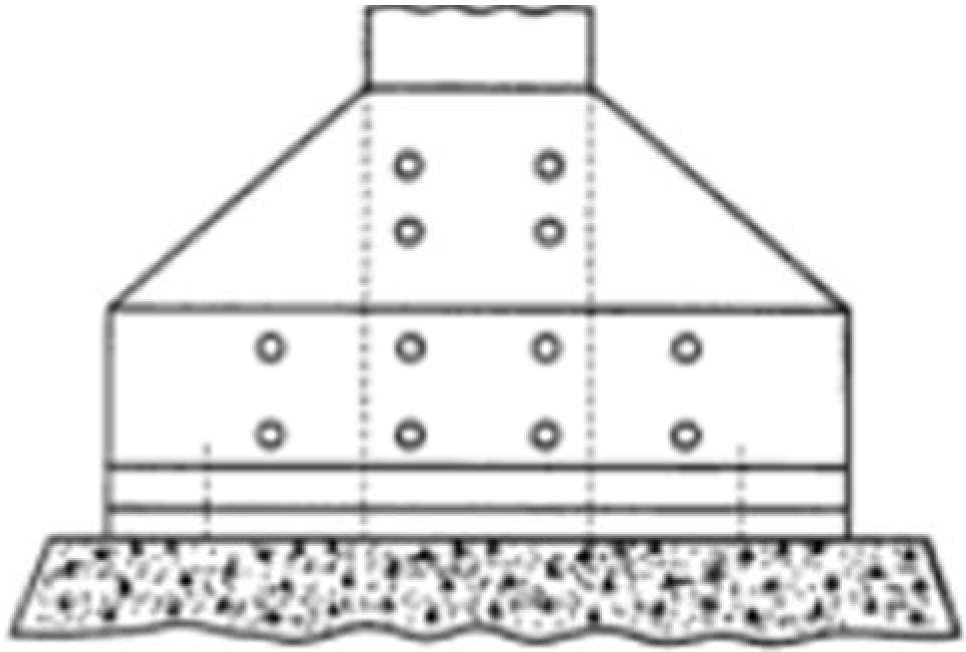
d_0 = the diameter of the reduced end, if any, of the column in mm.

Gusseted base:





Front View



Side View

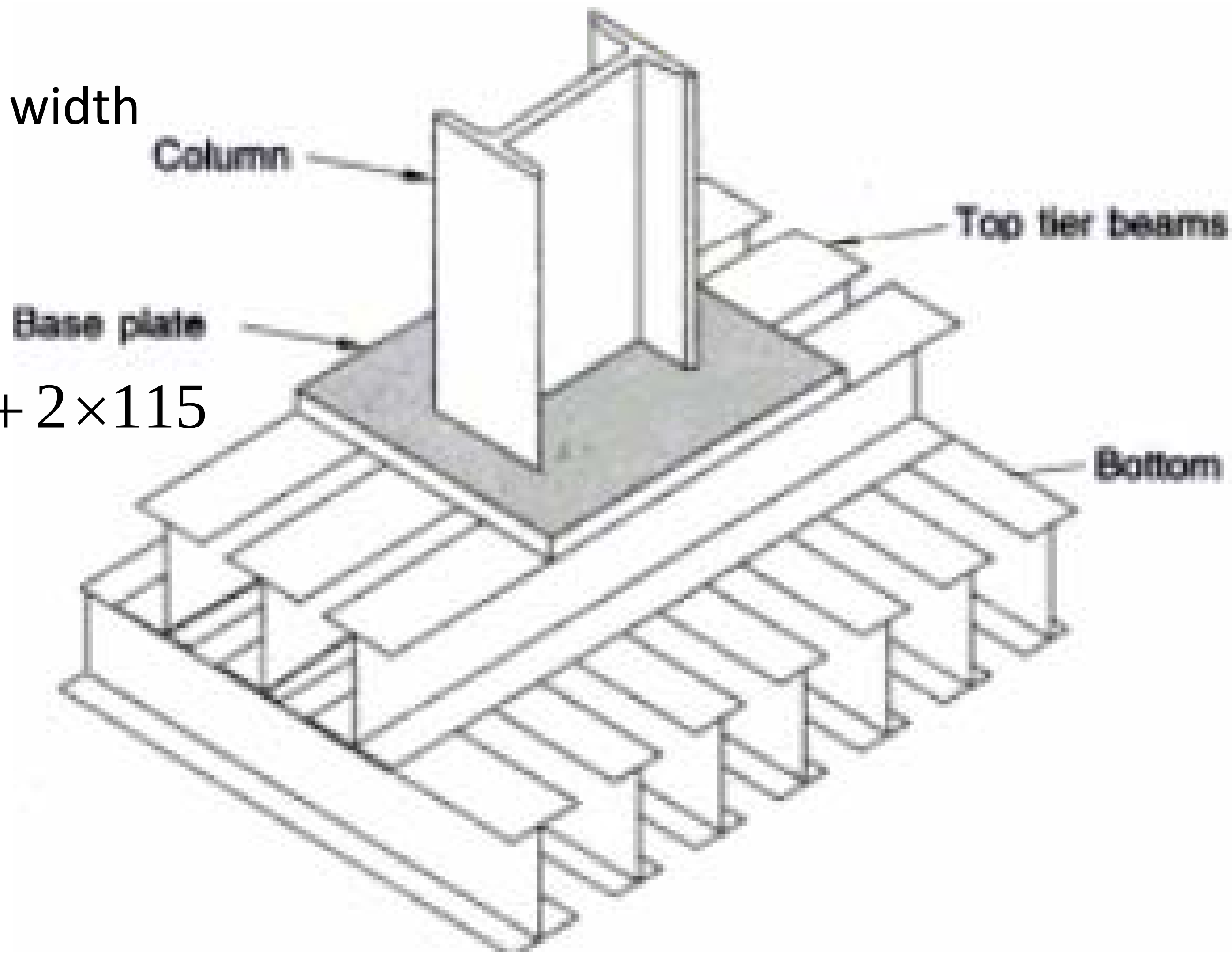
Grillage Foundation:-

- It is provided at shallow depth for column carrying heavy Load on weak soil. It consists of two or more layers of steel Beams completely encased in concrete.
- The permissible stress in grillage beam may be increased by 33.5% (50% in case of WL or EQ). If the following condition are satisfied

1. The beam are unpainted and solidly encased in ordinary dense concrete with 10mm aggregate and minimum f_{ck} = 15 Mpa.
2. Minimum distance between edge of flange is 75 mm.
3. Minimum concrete cover is 100 mm.

Design a grillage foundation for axial load 2500 KN and in column section two ISMC 250, with two cover Plates 300x25 mm.

Minimum required width of slab base



$$w = 250 + 2 \times 25 + 2 \times 115$$

$$w = 530mm$$

Assume,

$$w = 600mm$$

$$l = w + 2\sqrt{3}h_2$$

Assume self weight of footing is 15% of axial load

Total load = 1.15x2500 = 2875 KN

Area of bottom footing = $\frac{2875}{250} = 11.5 m^2$

Provide a 3.4x3.4m grillage foundation

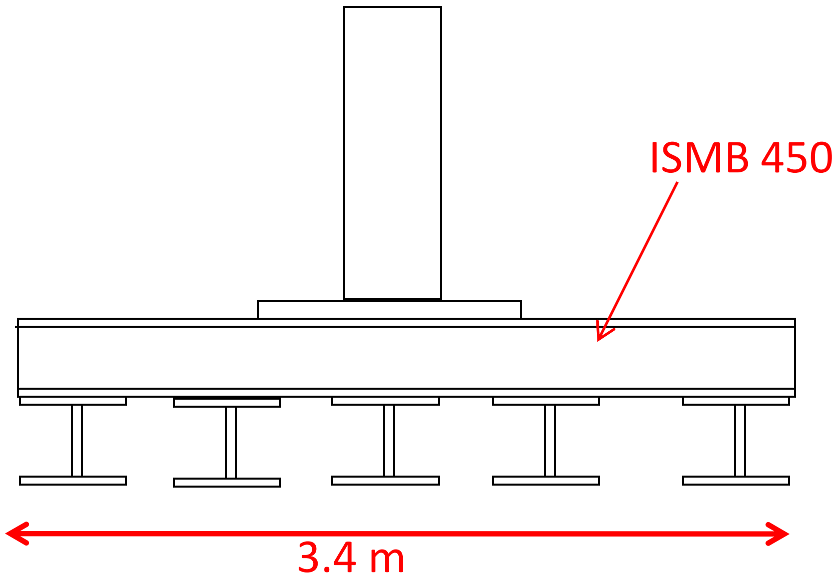
Maximum bending moment.

$$M = \frac{P(L-l)}{8}$$

$$= \frac{2500\ (3.4 - 0.6)}{8} = 875\ KN - m$$

$$Z_{req} = \frac{M}{1.33\sigma_{bc}} = \frac{875 \times 10^6}{1.33 \times 165} = 3987 \times 10^3 mm^3$$

$$Z_{req,each} = \frac{3987 \times 10^3}{3} = 1329 \times 10^3 mm^3$$



Try with ISMB450@72.4Kg/m

C/S area required at critical web section

$$A = \frac{P}{1.33 \times \sigma_{pt}} = \frac{2500 \times 10^3}{1.33 \times 189} = 9945.49\, mm^2$$

$$h_2 = 35.4mm$$

$$l = 600 + 2\sqrt{3} \times 35.4 = 722.62\, mm$$

Required total web thickness :

$$w' = \frac{A}{l} = \frac{9945.49}{722.62} = 13.67\, mm$$

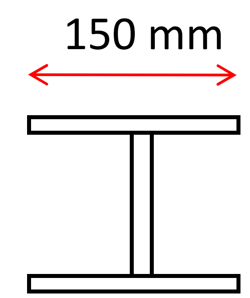
If we provide 3 No of beam then web thickness of each beam

$$t_{w,req} = \frac{w'}{3} = \frac{13.76}{3} = 4.58mm < t_w(9.4mm), OK$$

Maximum shear force = $\dfrac{P(L-l)}{2L} = \dfrac{2500(3.4-0.6)}{2\times 3.4} = 1030KN$

$$\tau_{av,cal} = \frac{1.03\times 10^{6}}{3\times 450\times 9.4} = 81N/mm^{2} < \tau_{av}(100\,N/mm^{2})$$

Minimum required length of slab base



$$L = 3\times 150 + 2\times 75 = 600\ mm$$

Let’s provide 600 mm

Thus.

$$a = 150\ mm\ ,\ b = 150\ mm$$

Thickness of slab base,

$$t = \sqrt{\frac{3\times w}{\sigma_{bs}}\left(a^{2} - \frac{b^{2}}{4}\right)}$$

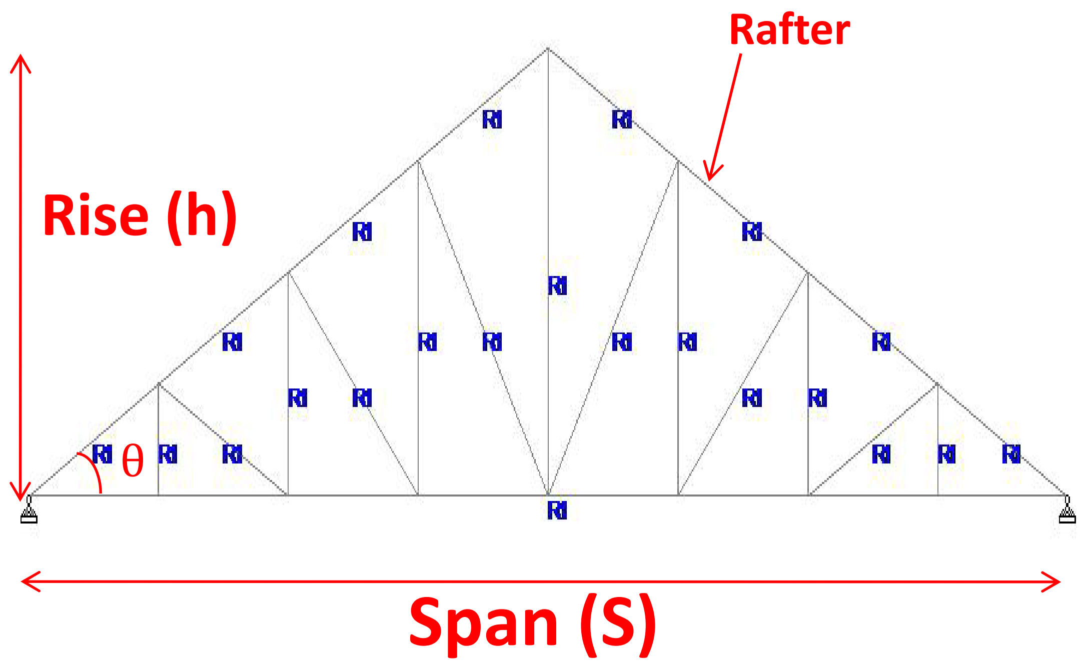
$$w = \frac{2500\times 10^{3}}{600\times 600} = 6.94N/mm^{2}$$

$$t = 43.58\,mm \approx 4.5cm$$

➢ ROOF TRUSS:

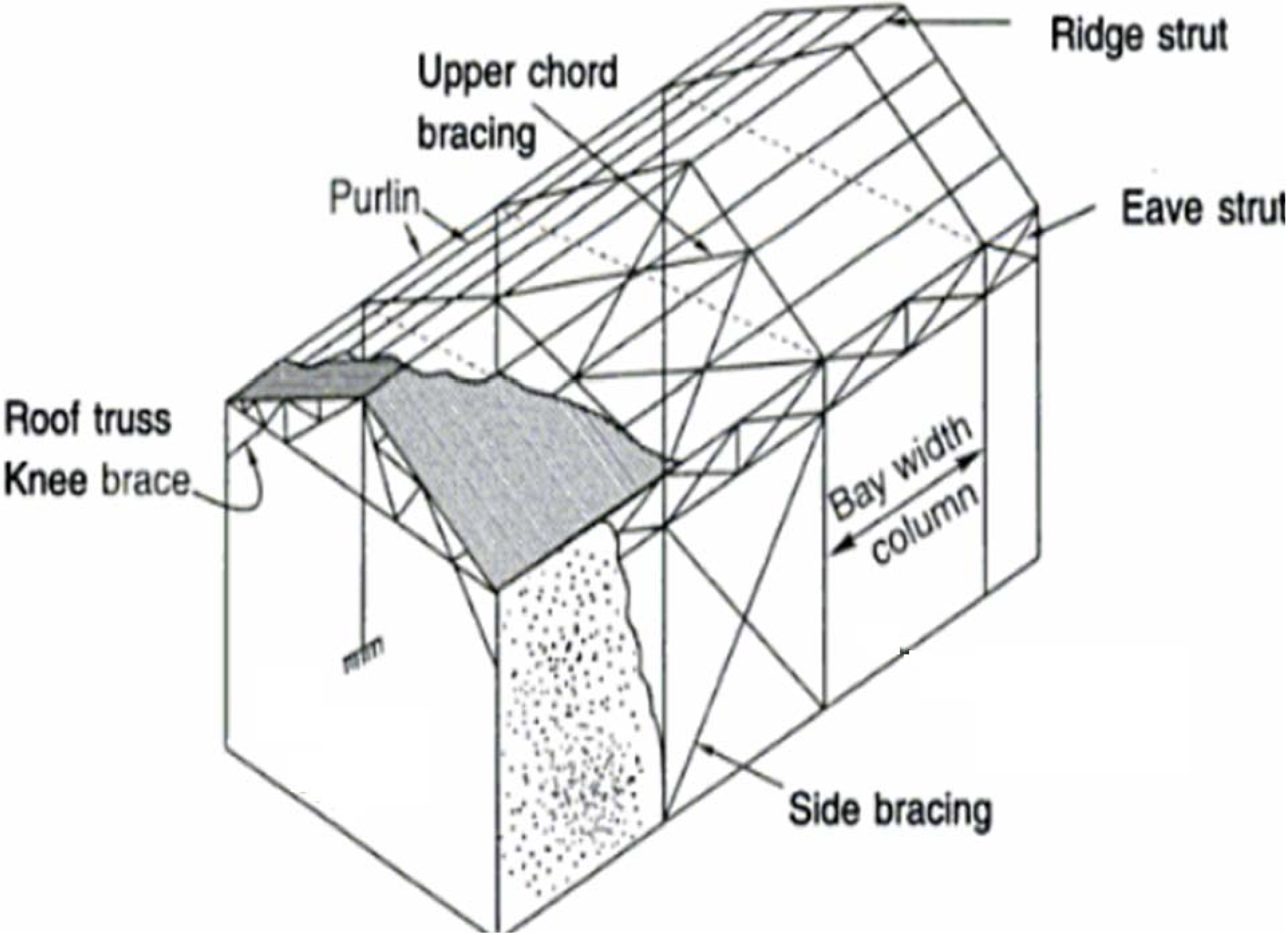
$$\text{Pitch of roof truss} = \frac{h}{S}$$

$\theta \longrightarrow$ Slope of truss



Bay : distance between adjacent truss.

Eaves : the bottom edges of an inclined roof surface.


Ridge strut
Upper chord bracing
Purlin
Eave strut
Roof truss
Knee brace
Bay width
column
Side bracing

Load on roof truss:- DL, LL, WL/SL

Dead Load (DL)

Weight of roof covering sheeting =150 N/m^2

Weight of bracings =15 N /m^2

Weight of purling = 100 N/m^2

Self Weight of truss = $10\left(\frac{Span\ (m)}{3}+5\right)N/m^2$

From IS 875 –II, Table -2

Live Load (LL) $= 750 - [(\theta - 10) \times 20]N/m^2 \geq 400\ N/m^2$

$= 1500\ N/m^2$ For Accessible roof

If value of LL is less than 400 N/m^2 then take LL = 400 N/m^2

Note: For truss analysis take LL as 2/3 of total LL

For purlins analysis take LL as total LL

Wind Load (WL): IS 875 - III

It depend upon wind speed and height of the structure.

Basic Wind speed (V_b) - It is determine on the based on statically Data of return period 50 year and at height 10 m above the ground Surface.

Design wind speed (V_z) – $K_1\ K_2\ K_3\ V_b$

K_1 – Probability/Risk factor, depend upon class and design life of the structure. Given in Table - 1

TABLE 1 RISK COEFFICIENTS FOR DIFFERENT CLASSES OF STRUCTURES IN DIFFERENT WIND SPEED ZONES

(*Clause* 5.3.1)

Class of Structure	Mean Probable Design Life of Structure in Years	k_1 Factor for Basic Wind Speed (m/s) of					
		33	39	44	47	50	55
All general buildings and structures	50	1·0	1·0	1·0	1·0	1·0	1·0
Temporary sheds, structures such as those used during construction operations (for example, form-work and falsework), structures during construction stages and boundary walls	5	0·82	0·76	0·73	0·71	0·70	0·67

K_2 – Depend upon height, terrain category and class of the structure. Given in Table – 2,3.

TABLE 3 FETCH AND DEVELOPED HEIGHT RELATIONSHIP

(*Clause* 5.3.2.4)

FETCH (x) km	DEVELOPED HEIGHT, h_x IN METRES			
	Terrain Category 1	Terrain Category 2	Terrain Category 3	Terrain Category 4
(1)	(2)	(3)	(4)	(5)
0·2	12	20	35	60
0·5	20	30	35	95
1	25	45	80	130
2	35	65	110	190
5	60	100	170	300
10	80	140	250	450
20	120	200	350	500
50	180	300	400	500

Class of structure

If any maximum dimension of the structure (Length, width, height)

< 20 m – A, 20 – 50 m – B, > 50 m - C

TABLE 2 k_2 FACTORS TO OBTAIN DESIGN WIND SPEED VARIATION WITH HEIGHT IN DIFFERENT TERRAINS FOR DIFFERENT CLASSES OF BUILDINGS/STRUCTURES

(*Clause* 5.3.2.2)

HEIGHT	TERRAIN CATEGORY 1 CLASS			TERRAIN CATEGORY 2 CLASS			TERRAIN CATEGORY 3 CLASS			TERRAIN CATEGORY 4 CLASS		
m	*A*	*B*	*C*	*A*	*B*	*C*	*A*	*B*	*C*	*A*	*B*	*C*
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
10	1·05	1·03	0·99	1·00	0·98	0·93	0·91	0·88	0·82	0·80	0·76	0·67
15	1·09	1·07	1·03	1·05	1·02	0·97	0·97	0·94	0·87	0·80	0·76	0·67
20	1·12	1·10	1·06	1·07	1·05	1·00	1·01	0·98	0·91	0·80	0·76	0·67
30	1·15	1·13	1·09	1·12	1·10	1·04	1·06	1·03	0·96	0·97	0·93	0·83
50	1·20	1·18	1·14	1·17	1·15	1·10	1·12	1·09	1·02	1·10	1·05	0·95
100	1·26	1·24	1·20	1·24	1·22	1·17	1·20	1·17	1·10	1·20	1·15	1·05
150	1·30	1·28	1·24	1·28	1·25	1·21	1·24	1·21	1·15	1·24	1·20	1·10
200	1·32	1·30	1·26	1·30	1·28	1·24	1·27	1·24	1·18	1·27	1·22	1·13
250	1·34	1·32	1·28	1·32	1·31	1·26	1·29	1·26	1·20	1·28	1·24	1·16
300	1·35	1·34	1·30	1·34	1 32	1·28	1·31	1·28	1·22	1·30	1·26	1·17
350	1·37	1·35	1·31	1·36	1·34	1·29	1·32	1·30	1·24	1·31	1·27	1·19
400	1·38	1·36	1·32	1·37	1·35	1·30	1·34	1·31	1·25	1·32	1·28	1·20
450	1·39	1·37	1·33	1·38	1·36	1·31	1·35	1·32	1·26	1·33	1·29	1·21
500	1·40	1·38	1·34	1·39	1·37	1·32	1·36	1·33	1·28	1·34	1·30	1·22

K_3 – Topographic factor, depend upon slope of ground wind flow direction. The effect of topography will be significant at a site when the upwind slope (θ) is greater than about 3^0, If slope of ground is less than 3^0 $K_3 = 1$, for other slope K_3 is Given in Appendix - C

Design Wind Pressure (P_Z)

$$P_Z = 0.6 \times V_Z^2 N / mm^2$$

Wind Load on Individual Members (F)

$$F = \left(C_{pe} - C_{pi}\right) A_e P_Z$$

C_{pe} - is external pressure coefficient (for ***Pitched roofs Table - 5***)

C_{pi} - is internal pressure coefficient (***clause 6.2.3***)

Internal Pressure Coefficients, C_{pi}	
Wall opening	C_{pi}
Upto 5% of wall area	± 0.2
5% - 20 % of wall area	± 0.5
>20 % of wall area	± 0.7

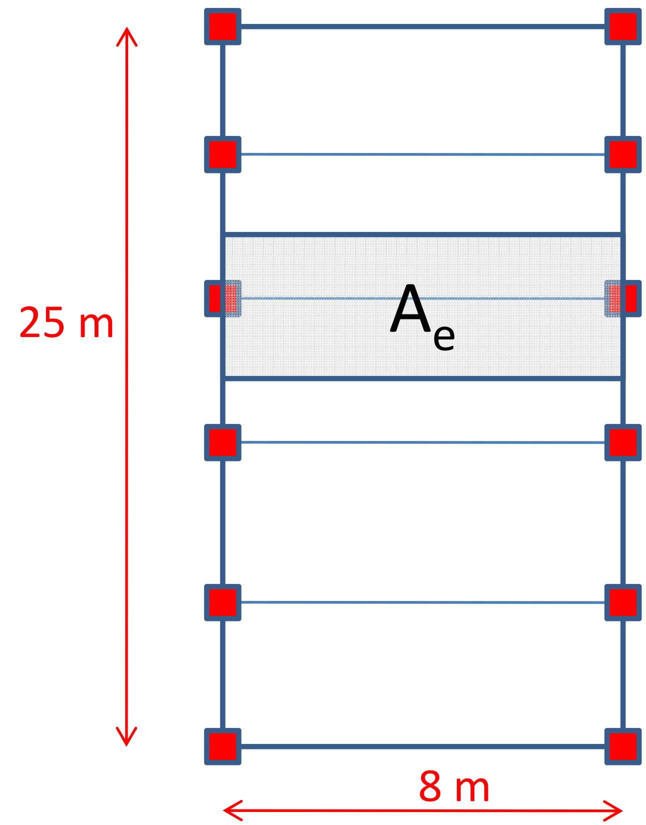
A_e – effective area

Design an industrial shed for following dimension

Span Length (m)	Pitch of roof truss	Spacing of truss (m)	Location.	Eaves Height (m)
8	0.25	5	Patna	6

Assume number of way is five.

Draw plan view in scale



Height of roof truss (h)

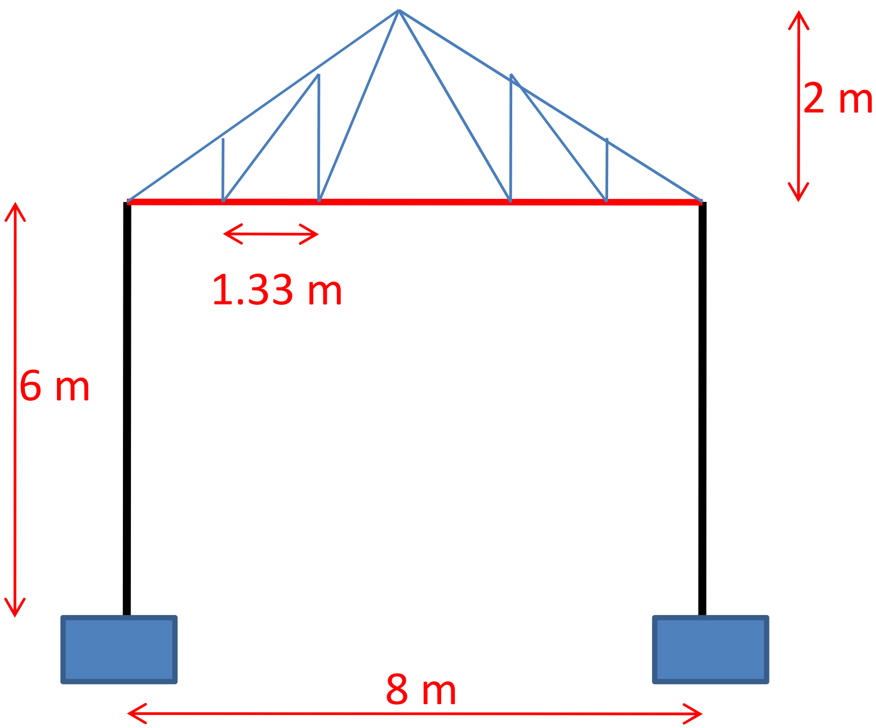
$$h = 0.25 \times 8 = 2\,m$$

Effective area (A_e)

$$A_e = 8 \times 5 = 40 m^2$$

Slope of the roof truss:

$$\theta = \tan^{-1}\left(\frac{2}{4}\right) = 26.56^{0}$$



Load calculation:

Dead Load (DL)

Weight of roof covering sheeting =150 N/m^2

Weight of bracings =15 N /m^2

Weight of purling = 100 N/m^2

Self Weight of truss = $10\left(\frac{8}{3} + 5\right) = 76.67\ N/m^2$

Total DL intensity = 341.67 N/m^2

DL = $A_e \times 341.67 = 40 \times 341.67 = 13666.8N$

Live Load intensity (LL) $= 750 - [(\theta - 10) \times 20]$

$= 418.69\ N/m^2 \geq 400\ N/m^2$

Total LL on a truss = $\frac{2}{3} \times 418.69 \times 40 = 11165.3N$

Total DL+LL = 24.83 KN

Wind Load (WL):

Basic Wind speed for Patna (V_b) = 47 m/s

Design wind speed (V_z) = $K_1\ K_2 K_3 V_b$

Assume design life of the structure is 50 year

So, $K_1 = 1$

Terrain category – 2, Class of structure – B

So, $K_2 = 0.98$

Slope of ground is less than 3^0

So, $K_3 = 1$

Design wind speed (V_z) = 1x0.98x1x 47 = 46.06 m/s

Design Wind Pressure (P_Z) = $0.6 \times 46.06^2 = 1272.91$ N/m^2

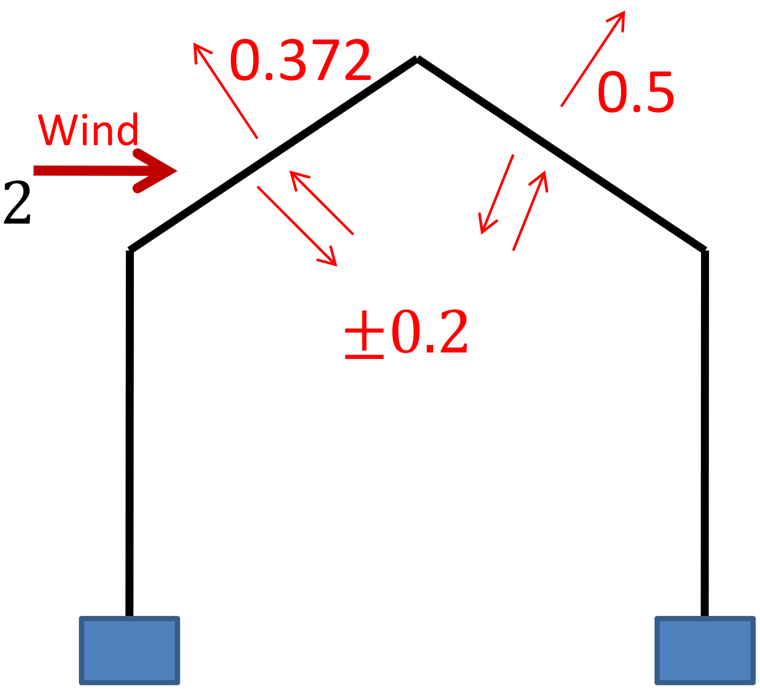
Wind Force (F) $= (C_{pe} - C_{pi})A_e P_Z$

External pressure coefficient on roof, IS 875 (III), Table - 5

$$\frac{1}{2} < \frac{h}{w} = \frac{6}{8} = 0.75 < \frac{3}{2}$$

Roof angle (θ) =26.56^0

Case – I, $\theta = 90^0$



Assume wall opening is less then 5%

Internal pressure coefficient (C_{pi}) = ± 0.2

Wind – ward direction:

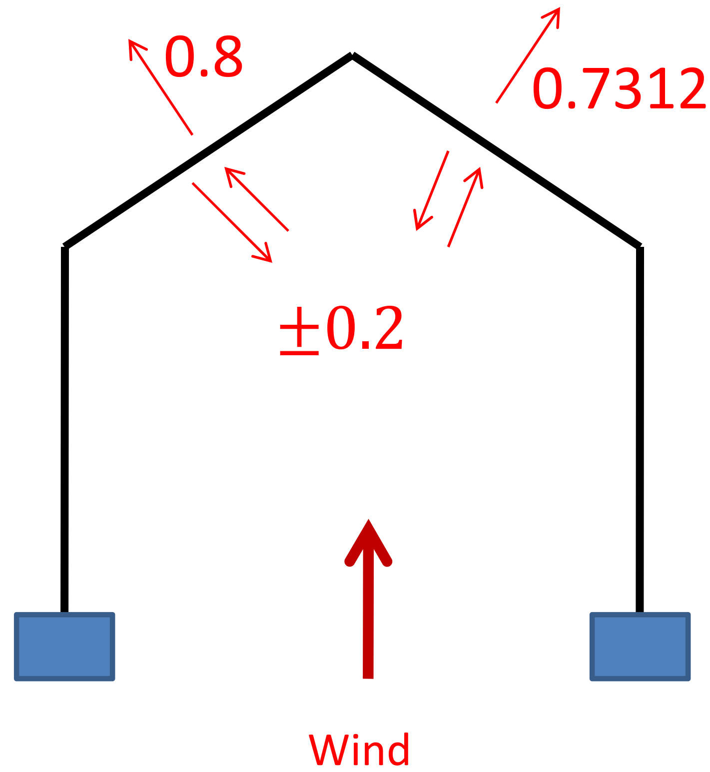
$$= -\left(0.7 - 0.5 \times \frac{6.56}{10}\right) = -0.372$$

$$\left(C_{pe} - C_{pi}\right)_{\max} = -0.572$$

Lee – ward direction= - 0.5

$$\left(C_{pe} - C_{pi}\right)_{\max} = -0.7$$

Case – II, $\theta = 0^0$



Wind – ward direction $= -0.8$

$$\left(C_{pe} - C_{pi}\right)_{\max} = -1.0$$

Lee – ward direction:

$$= -\left(0.6 + 0.2 \times \frac{6.56}{10}\right) = -0.7312$$

$$\left(C_{pe} - C_{pi}\right)_{\max} = -0.9312$$

Thus maximum value in all cases = - 1.0

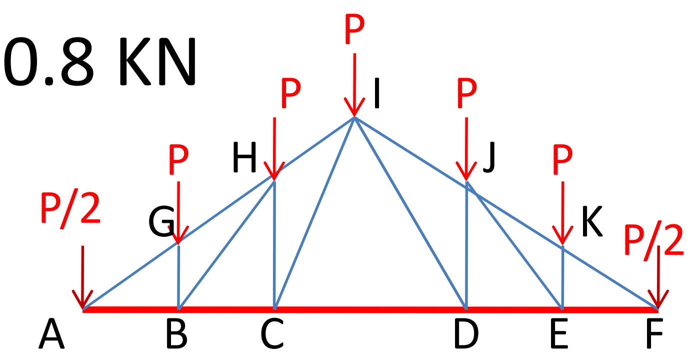
Design wind load on roof truss (F)

$$F = \left(C_{pe} - C_{pi}\right) A_e P_Z = -1.0 \times 40 \times 1.27 = -50.8 KN$$

Truss analysis :

Design load combination

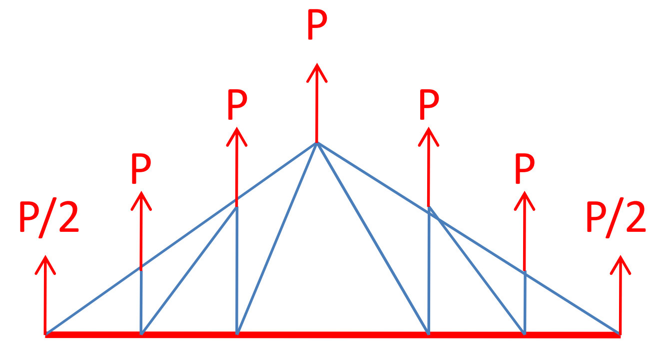
DL = 13.67 KN, LL = 11.16 KN, WL = -50.8 KN



DL + LL = 24.38 KN ↓ Down ward

$$P = \frac{24.38}{6} = 4.06KN$$

DL +WL = 37.12 KN ↑ Upward



$$P = \frac{37.12}{6} = 6.18KN$$

DL +LL analysis:

Member Type	Member	Length (m)	Forces (KN)
Rafter	AG	1.486	- 22.7
	GH	1.486	- 22.7
	HI	1.486	- 18.15
	IJ	1.486	- 18.15
	JK	1.486	- 22.7
	KF	1.486	- 22.7
Bottom Chord	AB	1.33	20.3
	BC	1.33	16.26
	DE	1.33	16.26
	EF	1.33	20.3
	CD	2.67	12.21
Vertical	GB	0.664	- 4.06
	EK	0.664	- 4.06
	CH	1.329	- 6.087
	DJ	1.329	- 6.087
Diagonal	BH	1.88	5.74
	JE	1.88	5.74
	CI	2.4	7.31
	ID	2.4	7.31

Design force in rafter is 22.7 KN compressive

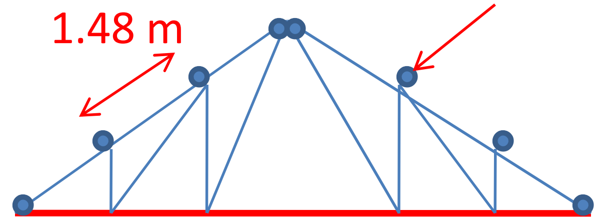
DL + WL analysis:

Member Type	Member	Length (m)	Forces (KN)
Rafter	AG	1.486	34.598
	GH	1.486	34.598
	HI	1.486	27.69
	IJ	1.486	27.69
	JK	1.486	34.598
	KF	1.486	34.598
Bottom Chord	AB	1.33	- 30.94
	BC	1.33	-24.76
	DE	1.33	-24.76
	EF	1.33	- 30.94
	CD	2.67	- 18.59
Vertical	GB	0.664	6.18
	EK	0.664	6.18
	CH	1.329	9.27
	DJ	1.329	9.27
Diagonal	BH	1.88	- 8.74
	JE	1.88	- 8.74
	CI	2.4	- 11.13
	ID	2.4	- 11.13

Minimum size of member is ISA 50x50x6

Design of Purlins:

- It act as continuous beam



Spacing of purlins = 1.48 m

Weight of roof covering sheeting =150 x1.48 = 222 N/m

Weight of purling = 100x1.48 = 148 N/m

Total DL = 370 N/m

LL = 418.69 x 1.48 *cos* (26.56^0) = 554.26 N/m

WL = - 1272.91x1.48 = - 1883.9 N/m

DL + LL = 924.26 N/m

DL +WL = -1513.9 N/m

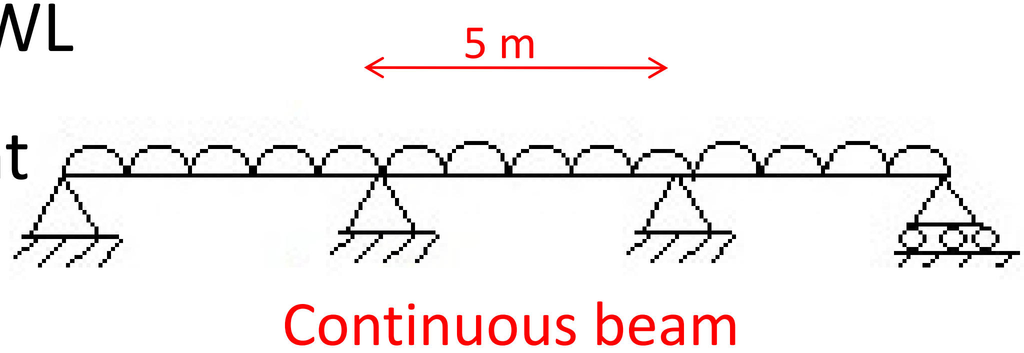
Foe WL case permissible stress is increased by 33 %

If we considered permissible is same, we can take effective load in DL + WL case

$$DL + WL = \frac{1513.9}{1.33} = 1138.27\ N/m > DL + LL\ (924.26\ N/m)$$

Thus, Design load is DL + WL

Maximum bending moment



From, IS 456 : 2000, Table - 12

$$M = \frac{wl^2}{10} = \frac{1138.27 \times 5^2}{10} = 2845.67\ N-m$$

$$Z_{req} = \frac{M}{\sigma_{bc/bt}} = \frac{2845.67 \times 10^3}{165} = 17.25 \times 10^3\ mm^3$$

Minimum depth = L/45 , Width = L/60

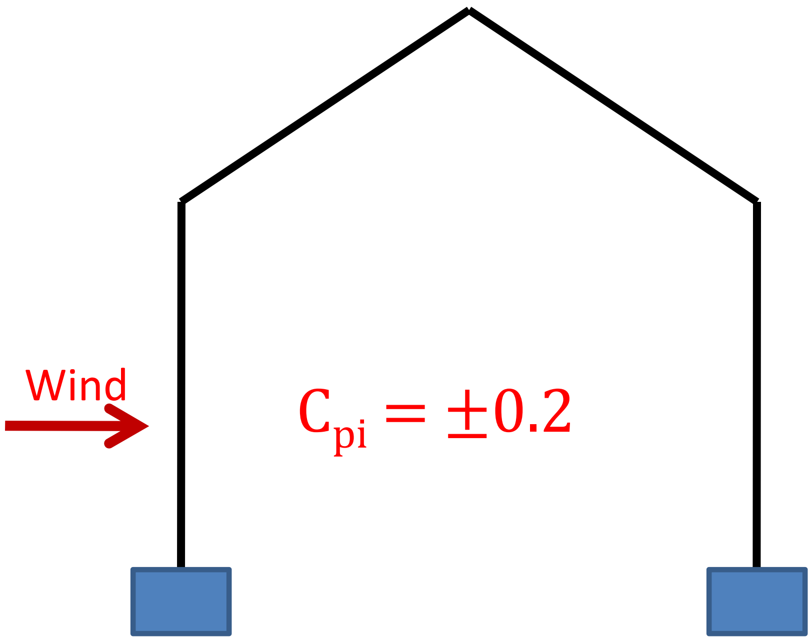
Design of Column

DL + LL = 24.38 KN Down ward

Axial load on column $=\frac{24.38}{2}=12.19KN$

DL +WL = 37.12 KN Upward

Axial load on column $=\frac{37.12}{2}=18.56KN$



Transverse load due to wind

From IS 875 (III), Table - 4

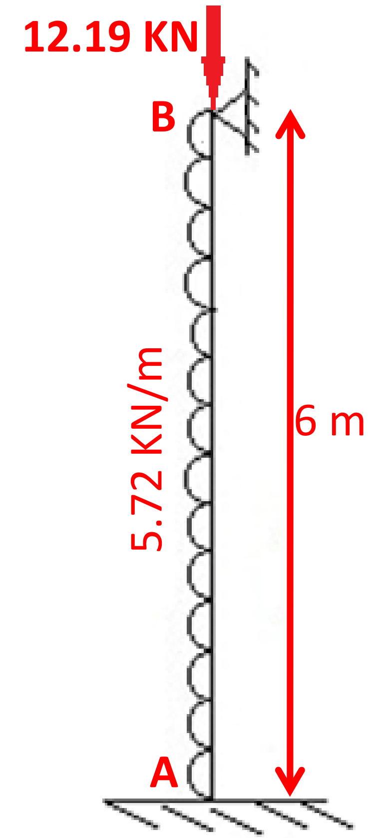
$$\frac{1}{2} < \frac{h}{w} = \frac{6}{8} = 0.75 < \frac{3}{2}, \frac{3}{2} < \frac{l}{w} = \frac{25}{8} = 3.125 < 4$$

$$\left(C_{pe} - C_{pi}\right)_{\max} = 0.9$$

Transverse load intensity

$$= 0.9 \times 5 \times 1272.91 N/m = 5.72 KN/m$$

Design column and Foundation for the load as shown



5.5 Angle Struts

5.5.1 *Single Angle Struts*

a) Single angle discontinuous struts connected by a single rivet or bolt may be designed for axial load only provided the compressive stress does not exceed 80 percent of the values given in Table 5.1 in which the effective length '*l*' of the strut shall be taken as centre-to-centre of intersection at each end and '*r*' is the minimum radius of gyration. In no case, however, shall the ratio of slenderness for such single angle struts exceed 180.

- **Radius of gyration and slenderness ratio :**

The radius of gyration is given by -

$$r = \sqrt{\frac{I}{A}}, \; r_{xx} = \sqrt{\frac{I_{xx}}{A}}, \; r_{yy} = \sqrt{\frac{I_{yy}}{A}}, \; r_{\min} = \sqrt{\frac{I_{\min}}{A}}$$

Where,

I – Moment of inertia

A – Cross-sectional area of the section