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MAULANA ABUL KALAM AZAD UNIVERSITY OF TECHNOLOGY
THIRD YEAR- VI SEMISTER

STEEL STRUCTURE DESIGN SESSIONAL
CE(PC) 694

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Subject - Steel Structure Design Sessional

Subject Code - CE (PC) 694.

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Introduction to Industrial Building.

→ The majority of Steel Structures being built are low rise building Industrial building a Subset of low rise. building are normally used for Steel plants automobile industries utility and process industries thermal power Stations, warehouse assembly plants, Storage garages, Small Scale industries etc. these buildings require large Column free areas and adequate head room.

The Structural engineer has to Consider the following points during the planning and design of industrial building.

a) Selection of roofing and wall material.
b) Selection of bay width.
c) Selection of Structural framing System.
d) Roof trusses.
e) Purlins, girts and Sag rods.
f) Bracing System to resist lateral loads.
g) Gantry girders, Columns, base plates, and foundations.

Roofing and wall material.

- Steel or Aluminium Decking / Cladding.
- Galvanized Iron (GI) Sheets.
- Asbestos Cement Sheets.

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Terminology

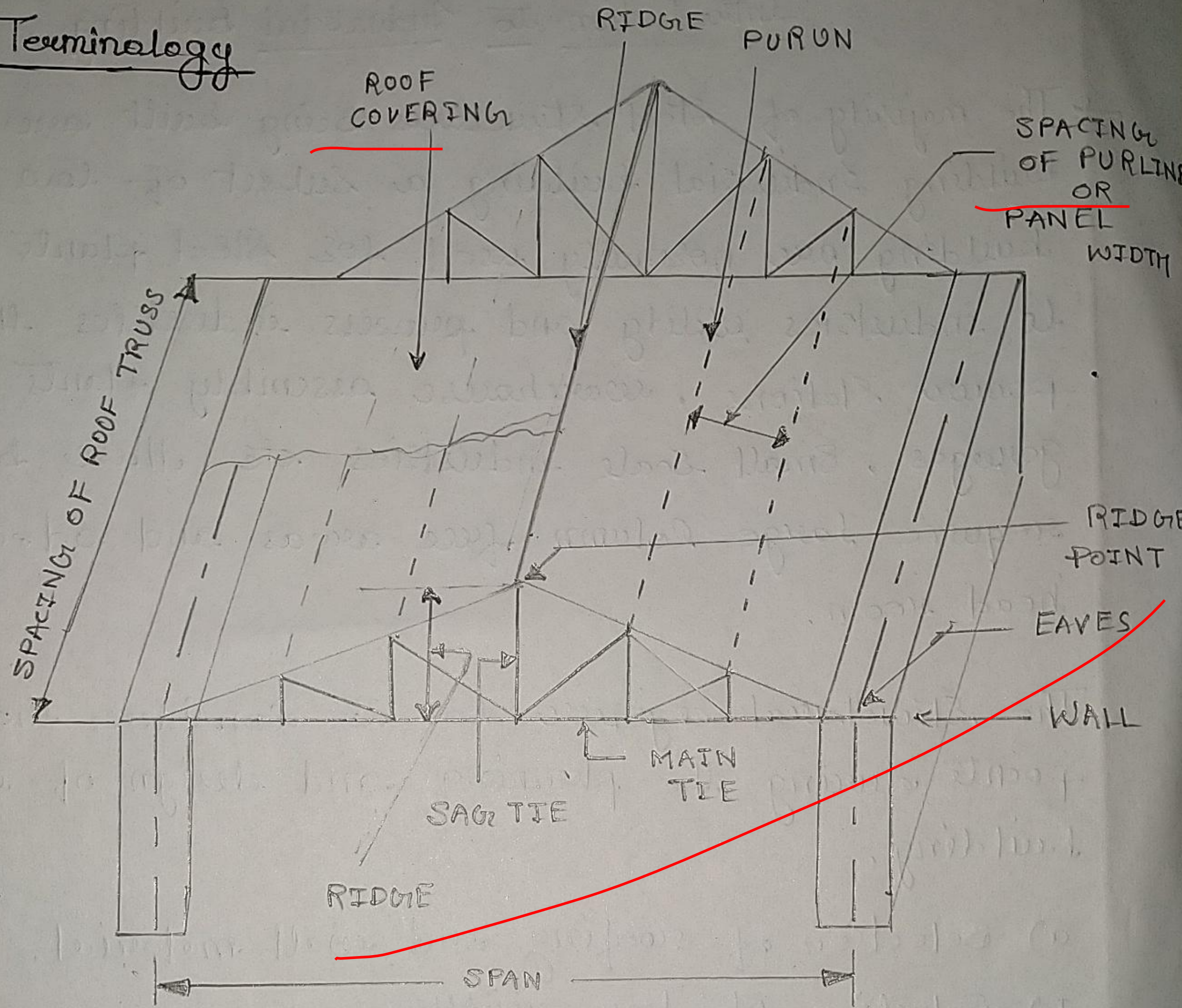

Fig :- Components of Industrial Building.

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Bay :- A bay is defined as the Space between two adjacent bents. the roof truss along with the Columns Constitutes a bent.

Span :- The Span between two rows of Columns of an Industrial building is Called an aisle or Span.

Braced frames : In braced building the trusses rest on Columns with hinge type of Connections and Stability is provided by bracing in the three mutually perpendicular planes.

Unbraced frames :- Unbraced frames are in the form of portal frames the frames Can provide large Column free areas, offering max^m adaptability of the Space inside the building.

Purlin :- A purlin is a horizontal Structural member that supports roof Covering and Carries load to the primary frame.

Girt :- A girt is a horizontal Structural member that is attached to Sidewall or end wall Column and support paneling.

Eave Strut :- Eave Strut member is located at the intersection of the roof and the exterior wall and hence acts as both the first purlin and the last (highest) girt.

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Plane Trusses :- A structure that is composed of a number of line members pin connected at the ends to form a triangulated framework is called a Truss. if all the members lie in a plane, the structure is a planar truss.

Preliminary Calculations.

- Pitch = rise / Span.
- Angle of roof truss = $\tan^{-1}\left(\frac{Rise}{\frac{Span}{2}}\right)$
- length of Principal rafter = $\sqrt{[R^2+(L/2)^2]}$
- Half plan area = $\frac{Span}{2}$ x Spacing of roof truss.
- Half Slop area = length of Principal rafter x Spacing of roof truss.
- loads on Roof Truss.

Load Combination for Design.

1) Dead load + imposed load (live load)
2) Dead load + Snow load.
3) Dead load + wind load (wind dirn being normal to the ridge or parallel to ridge whichever is severe)

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4) Dead load + imposed load + wind load (which may not be Critical in most of the Cases)

Wind Load Calculation.

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As per Clause 5.3, IS 875 (Part 3). 1987

Design wind Speed (m/s) at any height z is

$$V_z = V_b k_1 k_2 k_3.$$

- V_z = design wind speed at any height z in m/s
- k_1 = Probability factor (risk Coefficient) (clause 5.3.1)
- k_2 = terrain height and structure size factor (Clause 5.3.2); and.
- k_3 = topography factor (Clause 5.3.3)

Design wind pressure (Clause 5.4)

$$P_z = 0.6\, V_z^2.$$

Design wind force.

1) Total wind load for a building as a whole.

$$F = C_f A_e P_z$$ (Clause 6.3)

C_f = force coefficient of the building.

A_e = Effective frontal area.

P_z = Design wind pressure.

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2) Wind force on roof and walls is given by.

$$F = (C_{pe} - C_{pi}) A p_z .$$ (Clause 6.2.1)

C_{pe} = External pressure Coefficient.

C_{pi} = Internal pressure coefficient.

A = Surface area of structural element or cladding unit.

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Roof Truss

Design of roof truss

Example

Design a roof truss, rafter bracing, purlin for an industrial located at guwahati with a span of 20m and a length of 50m. the roofing is galvanized iron sheeting. Basic wind speed is 50m/s and the terrain is an open industrial area. Building is class B building with a clear height of 8 m at the eaves.

Soln:- 1) Structural Model.

A trapezoidal truss is adopted with a roof slope of 1 to 5 and end depth of 1m. for this span range the trapezoidal trusses would be normally efficient and economical Approximate span to depth ratio is about $\frac{1}{8}$ to $\frac{1}{12}$. Adopt a depth of 3m at mid-span.

$$\frac{\text{Span}}{\text{depth ratio}} = \frac{20}{3.0} = 6.67$$

Hence, the $\frac{\text{Span}}{\text{depth ratio}}$ is fine.

Truss spacing may be in the range of $\frac{1}{4}$th to $\frac{1}{5}$th of the span length Hence adopt a spacing of

$20/4 = 5m$.

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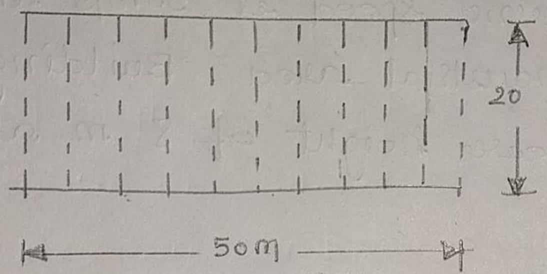
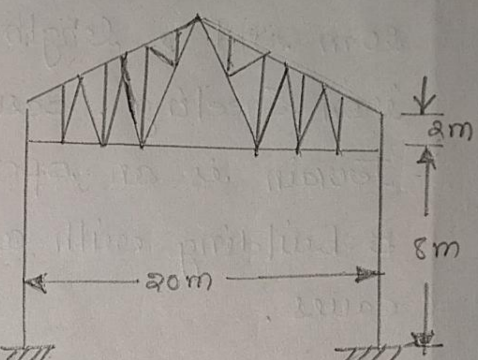
Then,

Number of bays = $\frac{50}{5} = 10$.

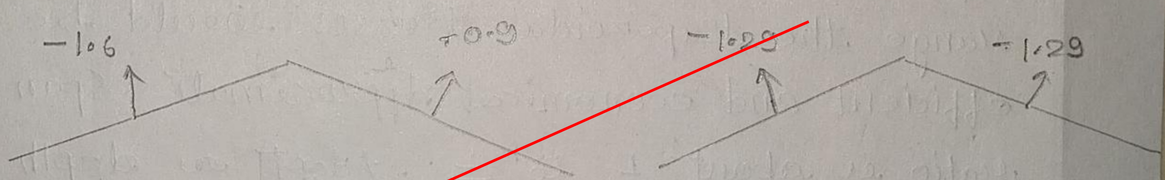
The plan of the building and the elevation of the truss.

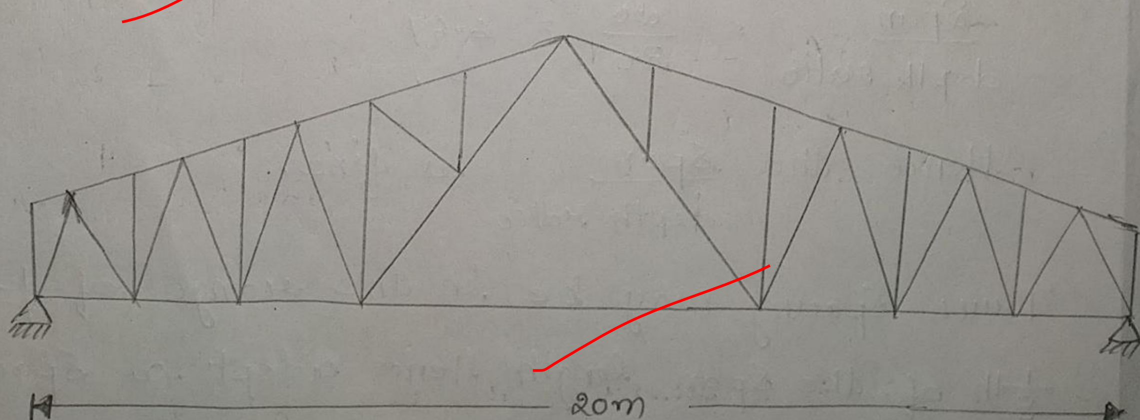


a) Plan.



b) Elevation.





d) configuration of truss with member and joint number adopted for the analysis.

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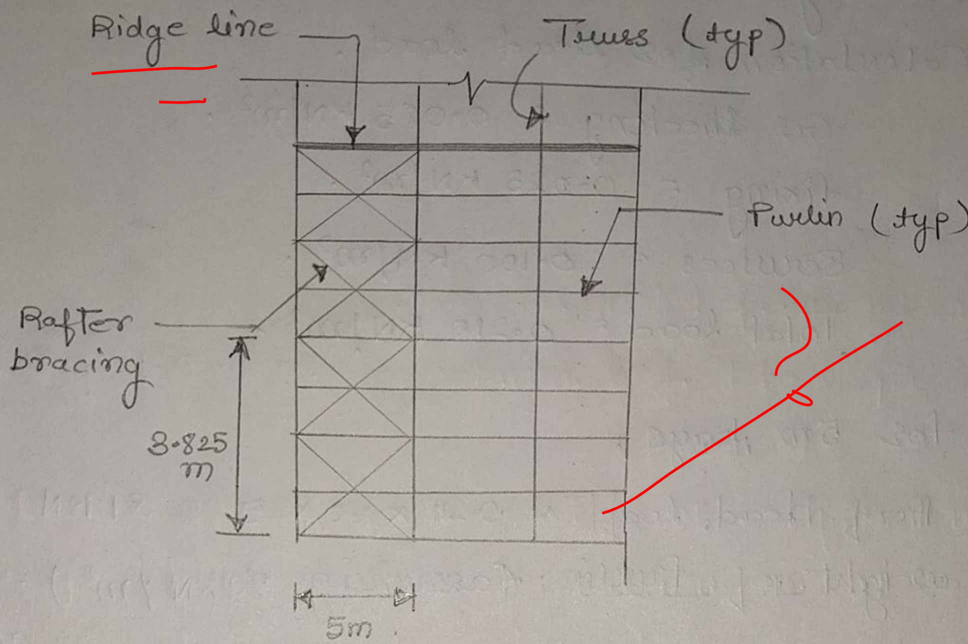
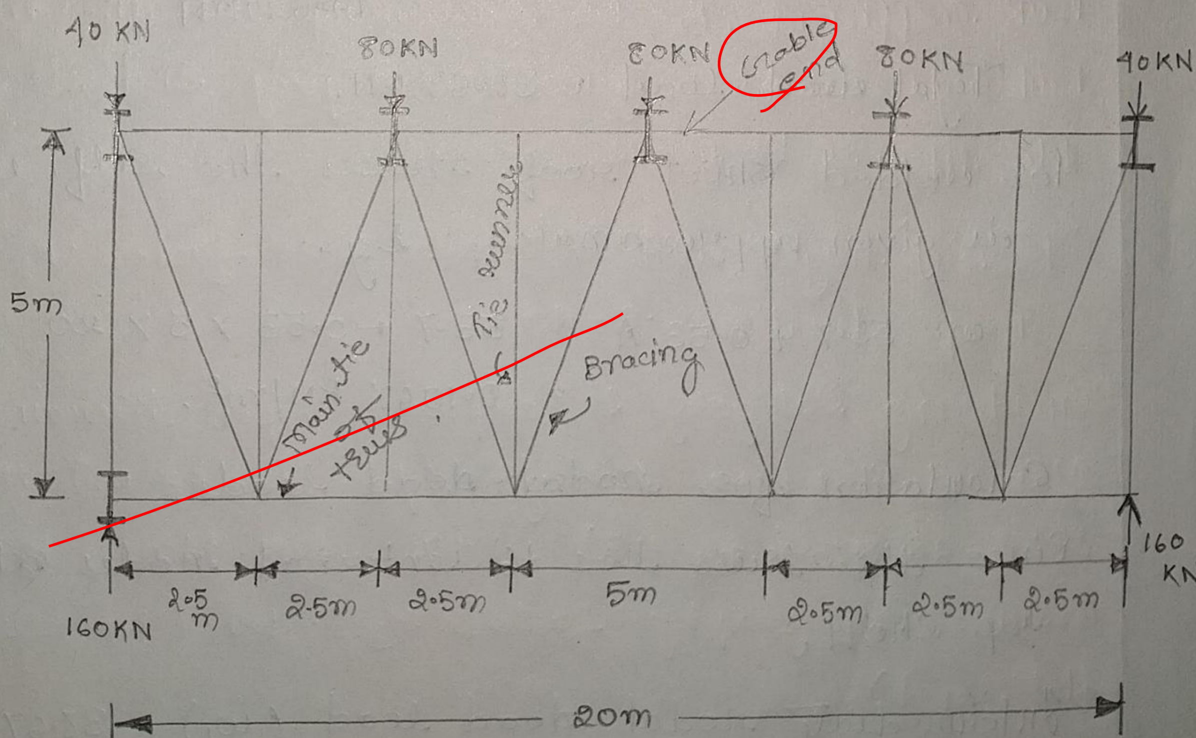


c) Layout of rafter bracing in the edge bay.



f) Configuration of eave girder.

2. loading

Calculation for dead load.

GI sheeting = 0.085 KN/m².

fixing = 0.025 KN/m².

Services = 0.100 KN/m².

Total load = 0.210 KN/m².

for 5m bays.

Roof dead load = 0.21 x 20 x 5 = 21 KN

weight of Purlin (assuming 70 KN/m²) = 0.07 x 5 x 20
= 7KN

Self wt of one truss* = 0.1067 x 5 x 20
= 10.67 KN

Total dead load = 38.67 KN.

for welded sheet roof trusses the self weight is given approximately by.

$w = 53.7 + 0.53 A = 53.7 + 0.53 \times 5 \times 20$
$= 0.1067$ KN/m².

Calculation for nodal dead loads.

Since the truss has 16 internal nodes at the top chord.

Intermediate nodal dead load $(w_1) = \frac{38.67}{16} = 2.42$ KN.

Dead load at end nodes $\left(\frac{w_1}{2}\right) = \frac{2.42}{2} = 1.21$ KN.

Wind load as per IS 875 (Part 3) - 1987.

Basic wind speed in Guwahati = 50m/s.

Wind load F on a roof truss by static wind method is given by (Clause 6.2.1 of IS 875) as follows.

$$F = (C_{pe} - C_{pi}) \times A \times P_d.$$

where C_{pe} and C_{pi} are the force coefficient for the exterior and interior of the building

Value of C_{pi} =

Assume wall opening between 5% - 20% of wall area (Clause 6.2.3.2 of IS 875).

we have

$$C_{pi} = \pm 0.5$$

Value of C_{pe} :

Roof Angle = $\alpha = \tan^{-1}\left(\frac{1}{5}\right) = 11.3°$

Height of the building to eaves $h = 8m$.

Short dimension of the building in plan $w = 20m$.

Building height to width ratio is given by,

$$\frac{h}{w} = \frac{8}{20} = 0.4 < 0.5.$$

wind Angle 0°

for 10° in windward side, $C_{pe} = -1.2$ and leeward side $C_{pe} = -0.4$.

Roof Angle $\alpha = 11.3°$.

Then by interpolation we get.

$C_{pe} = -1.1$ for windward and -0.4 for ~~upward~~ leeward.

wind angle = 90° [Table 5 of IS 875 (Part 3)]

for 10° in windward and leeward, $C_{pe} = -0.8$.

for 20° in windward and leeward, $C_{pe} = -0.7$

for 11.3°, $C_{pe} = -0.79$ for windward and leeward.

Risk Coefficient, $K_1 = 1.0$, assuming that the industrial building is under general category and its proba-ble life is 50 years.

Terrain height and Structure size factor K_2:

Roof elevation 8-11 m.

Considering Category 1 (exposed open terrain) and Class B structure (length b/w 20-25 m) table 2 of IS 875 (Part 3) = 1987, for 11 m, $K_2 = 1.038$

Assume topography factor $K_3 = 1.0$ (because of flat land)

wind pressure Calculation.

Total length of the building = 11m.

Basic wind speed $V_b = 50$ m/s.

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Design wind speed.

$V_z = K_1 \times K_2 \times K_3 \times V_b$

$K_1 = 1.0$; $K_2 = 1.038$; $K_3 = 1.0$.

$V_z = 1.038 \times 1 \times 1 \times 50 = 51.9\ m/s$.

Design wind pressure $P_d = 0.6\ V_z^2 = 0.6 \times (51.9)^2$

$= 1616.17\ N/m^2$

$= 1.616\ KN/m^2$.

Wind load on roof truss.

Wind Angle	Pressure Coefficient C_{pe} Windward	Pressure Coefficient C_{pe} Leeward	C_{pi}	$C_{pe} \pm C_{pi}$ Windward	$C_{pe} \pm C_{pi}$ Leeward	$A \times P_d$ (KN)	Wind Load Windward	Wind Load Leeward
0°	-1.70	-0.4	-0.5	-1.6	-0.9	10.3	-16.48	-9.27
			0.5	-0.6	0.1	10.3	-6.18	1.03
90°	-0.79	-0.79	-0.5	-1.29	-1.29	10.3	-13.29	-13.29
			0.5	-0.29	-0.29	10.3	-2.987	-2.987

The critical wind pressure is shown in fig (c)

3. Design of Purlin.

Span of Purlin = 5m.

Spacing of purlin = 1.275 m

$\theta = 11.3°$

load Calculations.

live load $= 0.75 - (11.3 - 10)\ 0.02$.

$= 0.724\ KN/m^2 > 0.4\ KN/m^2$.

Dead load $= 0.21\ KN/m^2$.

Wind load $= 1.616 \times 1.6 = 2.586\ KN/m^2$.

load Combinations.

1) $DL + LL = 0.21 + 0.724 = 0.934\ KN/m^2$.

2) $DL + WL$ =

Normal to Slope $= -2.586 + 0.21 \cos 11.3$

$= -2.38\ KN/m^2$.

Parallel to Slope $= 0.21 \sin 11.3$

$= 0.041\ KN/m^2$.

a) Load Combination. 1.

DL + LL

$w_z = (0.934 \times \cos 11.3) \times 1.275 = 1.168\ KN/m$.

$w_y = (0.934 \times \sin 11.3) \times 1.275 = 0.233\ KN/m$.

where w_z is the load normal to z-axis w_y is the load normal to y-axes, and 1.275 is the Spacing of the purlin Due to Continuity of Purlins factored bending moments and Shear force are as follows.

$M_z = 1.5 \times 1.168 \times 5^2/10 = 4.38$ KN m.

$M_y = 1.5 \times 0.288 \times 5^2/10 = 0.874$ KN m.

$SF_z = 1.5 \times 1.168 \times 5/2 = 4.38$ KN

Try MC 100 for which the properties are as follow.

$D = 100$ mm, $b_f = 50$ mm, $t_w = 5$ mm, $t_f = 7.7$ mm

$I_{zz} = 192 \times 10^4$ mm^4

$Z_{ez} = 37.3 \times 10^3$ mm^3, $Z_{ey} = 7.71 \times 10^3$ mm^3

$Z_{pz} = 43.83 \times 10^3$ mm^3, $Z_{py} = 16.238 \times 10^3$ mm^3.

Section Classification.

$b/t_f = \frac{50}{7.7} = 6.49 < 9.4$.

$d/t_w = (100 - 2 \times 7.7)/5.0 = 16.92 < 42$.

Hence the section is plastic.

Check for Shear Capacity.

As per Clause 8.4 of IS 800.

$A_v = (100 \times 5.0) = 500$ mm^2.

$$\frac{A_v f_{yw}}{\sqrt{3}\gamma_{mo}} = \frac{500 \times 250}{\sqrt{3} \times 1.10 \times 10^3} = 65.6 \text{ KN} > 4.38 \text{ KN}$$

Hence shear capacity is very large compared to the shear force check for moment capacity.

$$M_{dz} = \frac{b^z p_z f_y}{\gamma_{mo}} = \frac{1 \times 48.83 \times 250 \times 10^3}{1.10 \times 10^6} = 9.96 \text{ KNm}.$$

The above value should be less than.

$$\frac{1.2 \times 37.3 \times 250 \times 10^3}{1.10 \times 10^6} = 10.17 \text{ kNm}.$$

Hence $M_{dz} = 9.96$ KNm $>$ $M_z = 4.38$ KNm.

Hence the assumed section is safe.

$$M_{dy} = \frac{1 \times 16.238 \times 250 \times 10^3}{1.10 \times 10^6} = 3.69 \text{ KNm}.$$

The above value should be less than.

$$\frac{1.2 \times 7.71 \times 250 \times 10^3}{1.1} = 2.10 \text{ KNm}.$$

Hence $M_{dy} = 3.69$ KNm $<$ $M_y = 0.874$ KNm

Hence the section is satisfactory.

Check for biaxial bending.

$$\frac{M_z}{M_{dz}} + \frac{M_y}{M_{dy}} \leq 1$$

Thus, $\frac{4.38}{9.96} \cdot \frac{0.874}{3.69} = 0.68 < 1.0$.

Check for deflection.

Calculation for deflection is based on the Serviceability Cond^n i.e with unfactored imposed loads.

$$W = 1.168 \times 5 = 5.84\ KN.$$

$$\delta = \frac{5WL^3}{384EI_z}$$

$$= \frac{5 \times 5.84 \times 1000 \times 5000^3}{384 \times 2 \times 10^5 \times 192 \times 10^4}$$

$$= 24.75\ mm.$$

As per IS 800, Table 6, deflection limit is $\frac{L}{150}$ $= 33.33\ mm > 24.75\ mm$ Hence the deflection is within allowable limits.

b) load Combination 2: DL + WL.

$$W_z = 2.38 \times 1.275 = 3.035\ KN/m.$$

$$W_y = 0.041 \times 1.275 = 0.052\ KN/m.$$

factored bending moment in this case are,

$$M_z = 1.5 \times 3.035 \times \frac{5^2}{10} = 11.38\ KN\,m > M_{dz} = 9.96\ KNm.$$

$$M_y = 1.5 \times 0.052 \times \frac{5^2}{10} = 0.195\ KN/m < M_{dy} = 3.69\ KNm$$

Hence, the Section is not Safe let us adopt MC 125, which has an.

$I_{zz} = 425 \times 10^4 \ mm^4$.

$Z_{pz} = 77.88 \times 10^3 \ mm^3$ and $Z_{py} = 29.46 \times 10^3 \ mm^3$

$M_{dz} = 1 \times 77.88 \times 250 \times 10^{-3} / 1.1 = 17.7 \ KNm$.

$M_{dy} = 1 \times 29.46 \times 250 \times 10^{-3} / 1.1 = 6.69 \ KNm$.

Thus the Check for biaxial bending is,

$$\frac{11.38}{17.7} + \frac{0.195}{6.69} = 0.67 < 1.0$$

Hence the Section is Safe.

Check for deflection.

$$\delta = \frac{5 \times (3.035 \times 5) \times 1000 \times 5000^3}{384 \times 2.0 \times 10^5 \times 425 \times 10^4}$$

$= 29.06 < 33.33 \ mm$.

4. Truss Analysis and Design.

Tributary area for each node of the truss length of each panel along sloping roof

$$= \frac{1.25}{\cos 11.3^\circ} = 1.275\ m < 1.4m.$$

Spacing of trusses. = 5m.

Tributary area for each node of the truss

$= 5 \times 1.275 = 6.375\ m^2$.

Imposed load calculations.

from IS 875 (Part 2) - 1987

Live load $= 0.75\ kN/m^2$.

Reduction due to slope (See table 2.3 and footnote .3)

$$= (0.75 - 0.02 \times 1.3) \frac{2}{3} = 0.483\ kN/m^2.$$

load at intermediate nodes $W_2 = 0.483 \times 5 \times 1.25$
$= 3.02\ kN$.

load at end nodes $\frac{W_2}{2} = 1.51\ kN$.

(All these load act vertically downwards)

Max^m $C_{pe} \pm C_{pi}$ (Critical wind load to be considered for analysis)

Wind Angle	Windward Side (w_3)		Leeward (w_4)	
	Intermediate nodes w_3	End and Apex $w_3/2$	Intermediate nodes w_4	End and Apex nodes $w_4/2$
0°	-16.48	-8.29	-9.27	-4.64
90°	-13.29	-6.645	-13.29	-6.645

load in KN.

All these loads act perpendicular to the top. Chord member of the truss.

forces in the members the truss has been modelled as a ~~pain~~ pin jointed plane truss as shown in fig 12.15 (d) and analysed using the Software PLTRUSS developed by the author the analysis result are tabulated as follows. [See truss configuration shown in fig 12.15 (d) for member numbers]

load factors and Combinations. (Table 4 of IS 800)

for dead + imposed load.

$1.5 \times D.L + 1.5 \times LL$

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For dead + Wind load.

$1.5 \times DL + 1.5 \times WL$

Dead ~~load~~ + imposed + wind loading case will not be critical as wind loads act in opposite dirn to dead and imposed loads.

<u>Members forces under factored load in KN.</u>

Members Number	Dead Load + ~~Load~~ Load	Dead load + Wind load (0°)	Dead load + Wind load (90°)
1	0	2.472	1.985
2	-97.086	212.066	193.898
3	-97.086	217.01	197.898
4	-124.83	269.25	253.904
5	-124.83	274.20	257.29
6	-124.83	263.50	258.09
7	-128.99	279.45	270.58
8	-128.99	284.39	274.57
9	-128.99	244.35	270.57
10	-128.99	241.57	258.08
11	-124.83	233.41	257.29
12	-124.83	221.17	253.30
13	-124.83	218.39	197.9
14	-97.086	~~218.39~~	193.91
15	-97.086	163.98	1.99
16	0	161.20 1.39	-118.91
17	61.20	-141.17	-219.05
18	113.66	-251.86	-237.61
19	124.67	-261.51	-201.68.
20	108.801	-202.49	

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21	124.67	-212.69	-237.61
22.	113.66	-186.67	-219.06
23	61.20	-97.66	-118.91
24	-4.08	10.79	8.34
25	-4.08	5.27	8.34
26	-86.55	185.54	168.17
27	-86.55	138.114	168.17
28	48.084	-98.26	-92.45
29	48.084	-79.65	-92.45
30	48.084	21.58	16.69
31	-8.16	10.55	16.69
32	-8.16	58.86	59.82
33	-31.76	56.25	59.82
34	15.04	-20.94	-26.92
35	~~15~~.09	-30.85	-26.92
36	-8.16	21.58	16.69
37	-8.16	10.55	76.69
38	-4.67	5.20	5.97
39	-4.67	16.65	5.97
40	-4.67	26.38	12.41
41	-4.67	-2.46	12.41
42	-12.24	32.38	25.03
43	-12.24	15.82	25.03
44	5.225	-13.82	-10.69
45	5.225	-6.75	-10.69
46	8.16	21.58	16.69
47	8.16	10.55	16.69
48	21.245	-72.17	-46.70
49	21.245	-17.793	-46.70
50	27.62	-89.024	-59.74
51	27.62	-26.03	-59.74

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Truss Reaction (KN)

joint number	Case 1 (DL + LL)		Case 2 (DL + WL) 0°		Case 3 (DL + WL) 90°	
	X	Y	X	Y	X	Y
1	0	43.52	11.31	-94.32	0	-84.84
26	0	43.52	0	-68.61	0	-84.84

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Design of Top Chord Members. (member - 8)

factored Compressive force = 128.99 KN

factored tensile force = 284.39 KN

Try two ISA 75 X 75 X 6 mm @ 0.136 KN/m.

Sectional properties :

Area of Cross - Section $A = 2 \times 866 = 1732 \, mm^2$.

Radius of gyration $r_{zz} = 23 mm$.

Assuming 8 mm thick gusset plate.

$I_y = 2 [45.7 + 10^4 + 866 (4 + 20.6)^2]$.

$= 196.21 \times 10^4 \, mm^4$.

$r_y = \sqrt{(196.21 \times 10^4 / 1732)} = 33.66 \, mm$.

Section Classification.

$\varepsilon = \left(\frac{250}{f_y}\right)^{0.5} = \left(\frac{250}{250}\right)^{1/2} = 1.0$

$b/t = \frac{75}{6} = 12.5 < 15.7$

∴ the section is Semi - Compact.

As no member in the section is slender, the full section is effective and there is no need to adopt reduction factor.

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Max^{m} Unrestrained length = L = 1275 mm.

$KL = 0.85 \times L = 0.85 \times 12.75 = 1083.75$ mm.

Note. The effective length of top chord member may be taken as 0.7 - 1.0 times the distance b/w centres of connections as per clause 7.2.4 of IS 800 we have assumed the effective length factor as 0.85.

$$\lambda_y = \frac{1083.75}{23} = 47.12 < 180$$

Hence λ_y is within the allowable limits from table 9c of the code for $\frac{KL}{r} = 47.12$ and $f_y = 250$ MPa.

$f_{cd} = 187.32\ N/mm^2$.

$$\text{Axial Capacity} = 187.32 \times \frac{1732}{1000} = 324.4\ kN$$

$324.4\ kN > 128.99\ kN$.

Hence Section is Safe against axial compression

Axial tension capacity of the Section.

$$= \frac{1732 \times 250}{1.10}$$

$$= 393.64\ kN > 284.39\ kN$$

Hence Section is safe in tension.

Note though a smaller section may be chosen the section is adopted to take care of handling stresses

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Design of Rafters Bracing Members.

Considering the layout of the rafter bracing as shown in fig.

Design wind pressure $= 1.616\ kN/m^2$.

Maximum force coefficient $= -1.6$

factored wind load on rafters bracing

$= 1.5 \times 1.616 \times 1.6 \times 3.825 \times \frac{5}{2} \times \sec 11°3°$

$= 37.8\ kN$.

Length of bracing $= \sqrt{(3825)^2 + 5000)^2} = 6295.29\ mm$

Try $90 \times 90 \times 6$, $A = 1050\ mm^2$.

$r_{min} = 17.5\ mm$, and

$\frac{L}{r} = 6295.29 / 17.5 = 359.7 < 400$.

In the X bracing system as shown in fig. the compression bracing will buckle and only the tension bracing will be effective Also, the bracing members are usually bolted to the trusses at site.

Axial tensile capacity.

Design strength of member due to yielding of gross section.

$T_{dg} = A_g f_y / \gamma_{mo}$

$$\frac{1050 \times \left(\frac{250}{1.1}\right)}{1000}$$

$= 238.64 \text{ KN} > 37.8 \text{ KN}.$

Design strength due to rupture of Critical Section. $T_{dn} = \alpha A_n f_y / \gamma_{m1}$

$\alpha = 0.6$ (Assuming two bolts of 16 mm dia at the ends)

$A_n = 1050 - 18 \times 6 = 942 \text{ mm}^2$

$T_{dn} = 0.6 \times 942 \times \left(\frac{410}{1.25}\right) \times 1000$

$= 185 \text{ KN} > 37.8 \text{ KN}.$

Hence. L 90 X 90 X 6 is Safe the member has been found to be Safe for block Shear failure.

Note. The forces in the bracing members are often Small and rarely govern the design; but their Slenderness limitations decide the Size because of their long length.