INDIAN INSTITUTE OF TECHNOLOGY GANDHINAGAR

PROJECT: INDUSTRIAL SHED DESIGN

INSTRUCTOR - PROF. DHIMAN BASU

1.0 PROBLEM STATEMENT

The objective of this project is to design an industrial shed located in Ahmedabad. The shed must be designed to accommodate the following:

1. **Site and Dimensional Requirements**:
    - Plot Area: 2000 m^2
    - Aspect Ratio (L/W): 2.0
    - Column Height: 2.5 m
    - Truss Height: 3 m
    - Road alignment along the longer dimension of the plot.
2. **Structural Design Considerations**:
    - Dead Load (Self-weight and imposed dead loads).
    - Live Load.
    - Equivalent Static Wind Load.
    - Spacing between trusses: Width/5.
3. **Design Components**:
    - Tension Member
    - Compression Member

- Purlins
- Base
- Columns
- Baseplate
- Bolts

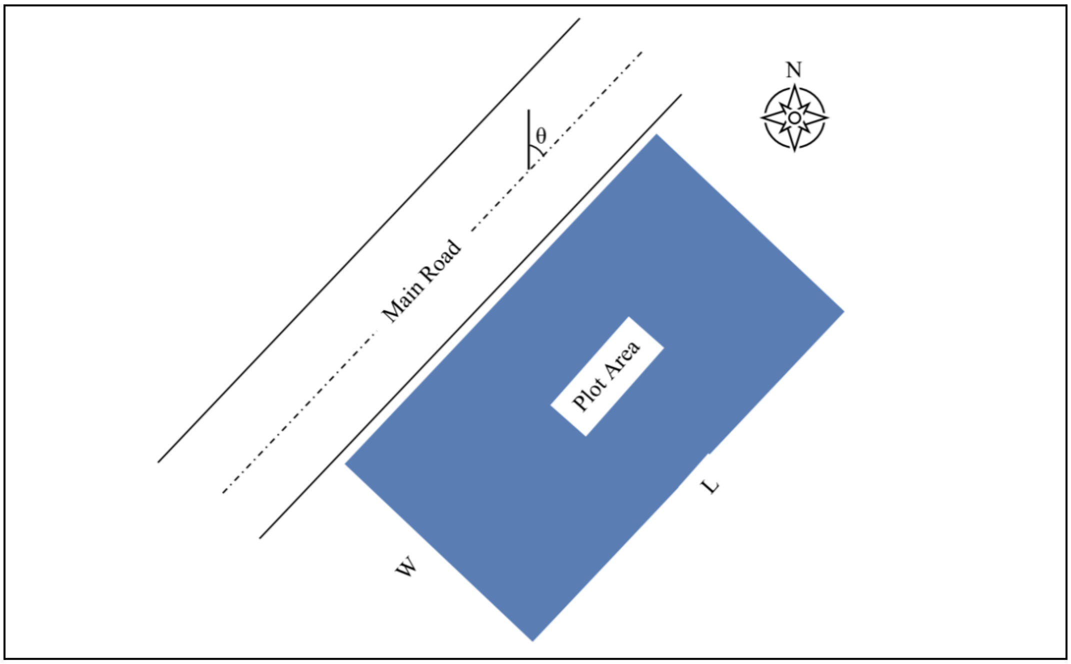


Fig 1: *Design an industrial shed*

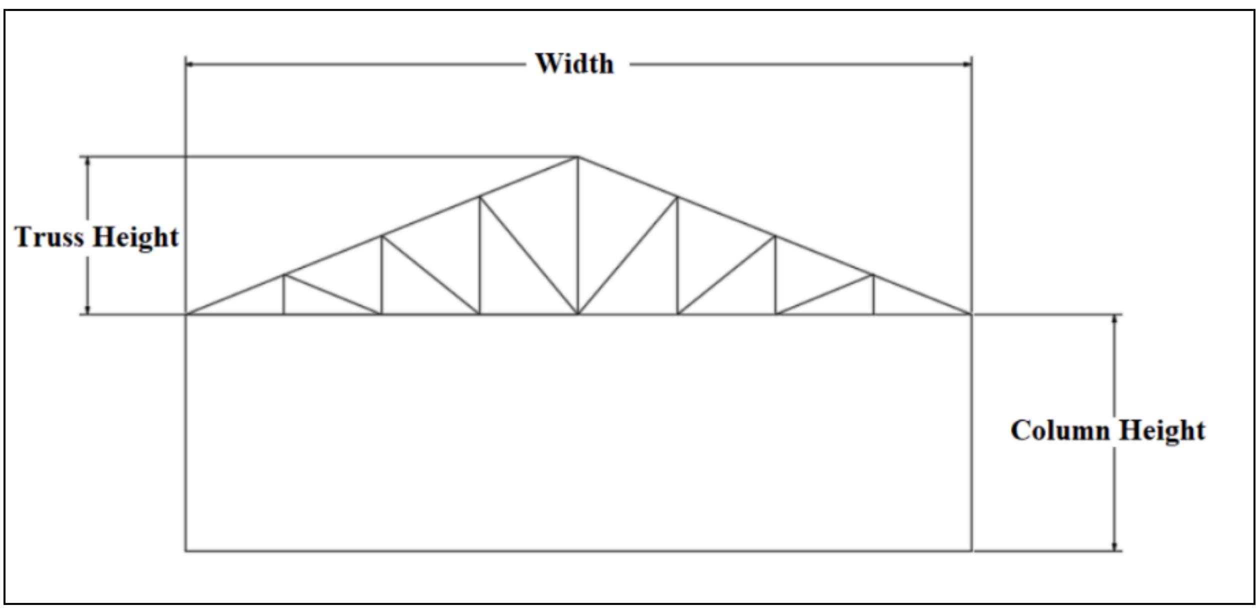


Fig 2: *Front Elevation of Industrial Shed*

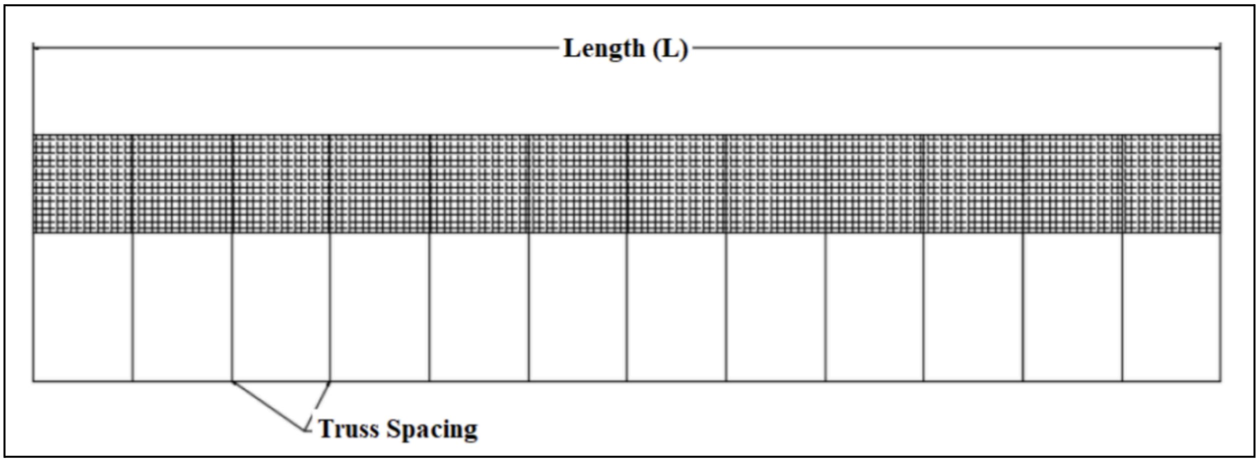


Fig 3: *Side Elevation of Industrial Shed*

2.0 INTRODUCTION

The design and construction of steel structures require a robust understanding of structural behavior and observation with design standards to ensure safety, functionality, and cost-effectiveness. This project emphasizes the practical application of steel design principles, including the design of trusses, columns, purlins, and column bases.

The industrial shed will span a **2000 m² plot** with an **aspect ratio of 2.0**, ensuring the longer side of the shed aligns with the road, as per the site requirements. The structural elements should be designed to withstand **dead loads, live loads**, and **wind loads** by the **Indian Standards Code**. This project provides a prospect to explore theoretical and practical aspects of steel structures.

3.0 LOAD CALCULATION

3.1 DEAD LOAD (As per IS 875: Part 1)

Weight of sheeting	0.15	kN/m^2
Self-weight factor	1	
Area	103.46064	m^2
Total Dead Load	15.519096	kN
DL at IPP	3.879774	kN
DL at EPP	1.939887	kN

3.2 LIVE LOAD (As per IS 875: Part 2)

Angle	14.945	degrees
Live Load	0.6511	kN/m^2
Area	103.46	m^2
Total Live Load	67.36	kN
LL at Per meter length	10.65	kN/m

3.3 WIND LOAD (As per IS 875: Part 3)

Basic wind speed, Vb		39	m/s
Risk coefficient, k1	1		(For 50 years of life span)
Terrain roughness and Height factor, k2	1.057		(Considering structure in Category 2)
Topography factor, k3	1		
Importance factor, k4	1		
Design wind speed, Vz	Vz = Vb * k1 * k2 * k3 * k4	41.223	m/s
Wind Pressure, Pz	Pz = 0.6 * Vz^2	1019.601437	N/m^2
Area averaging factor, Ka	0.8		
Wind directionality factor, Kd	0.9		(For rectangle form)
Combination factor, Kc	1		
Design wind Pressure, Pd	Pd = Pz * Ka * Kd * Kc	734.1130349	N/m^2
Cpe		-0.804	
Cpi		0.5	
Area, A		103.46064	m^2
Wind Load, F	F = (Cpe-Cpi) * Pd * A	-99.04115297	kN

3.4 LOAD AND LOAD COMBINATION

1	1.5 DL + 1.5 LL
2	1.5 DL + 1.5 WL
3	1.5 DL - 1.5 WL
4	1.2 DL + 1.5 LL + 1.2 WL
5	1.2 DL + 1.5 LL - 1.2 WL
6	0.9 DL + 0.9WL
7	0.9 DL - 0.9WL
8	Envelope

4.0 PROPERTIES OF SECTION

	Section	Sectional area (cm^2)	Unit weight (kg/m)	Dimensions				Radius of Gyration		Elastic Modulus	
				a (mm)	b (mm)	t (mm)	T (mm)	R1 (mm)	ry (mm)	Zpz (10^3* mm^3)	Zzz (mm^3)
Angle	200 x 200 x 25	94.1	73.9	200	200	25					
ISMB	450	92.2	72.38	450	150	9.4	17.4	15	30	1550	1350
ISLB	350	63	49.44	350	165	7.4	11.4	16	31.6	851	752
ISMB	400	78.4	61.55	400	140	8.9	16	14	28.1	1170	1020
ISLB	75	7.71	6.05	75	50	3.7	5	6.5	11.3	22.3	19.3

4.1 3-D RENDERED VIEW OF STRUCTURE

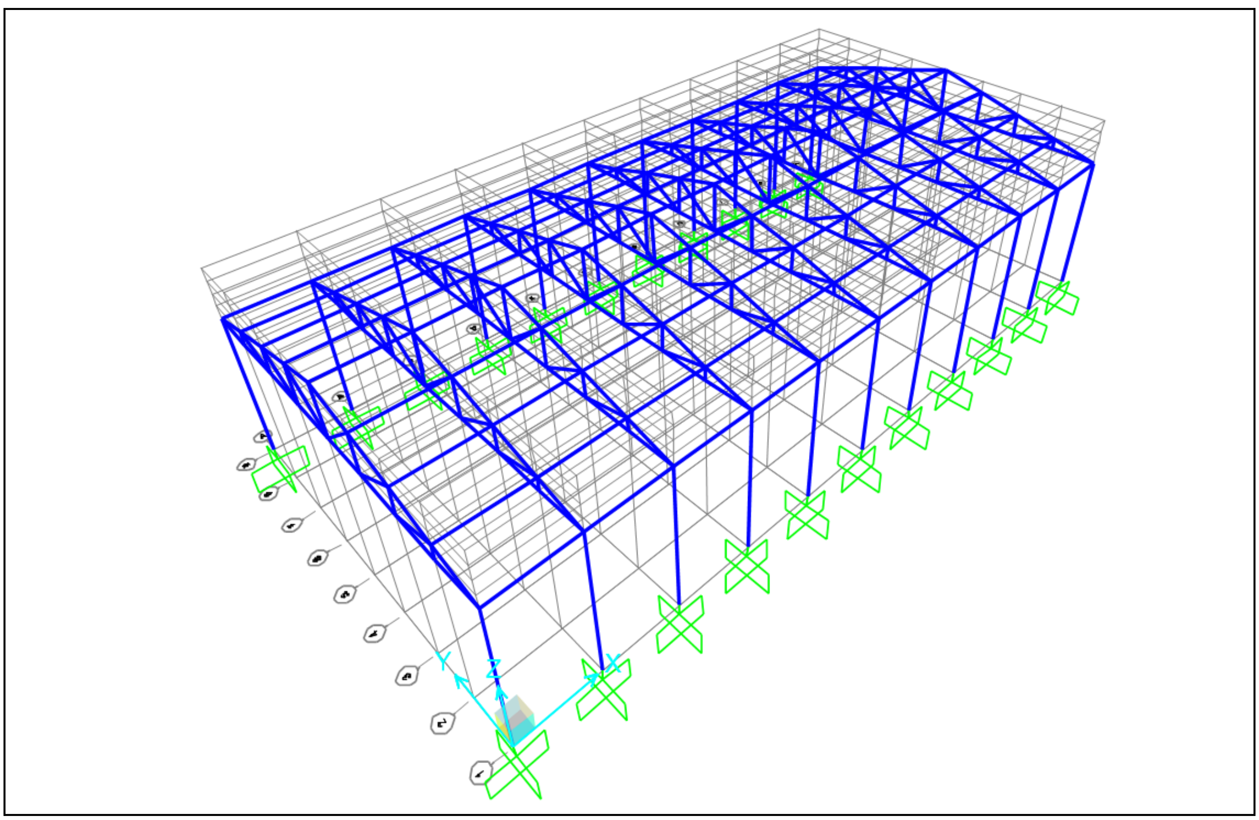


Fig 4: *Finite Element Model*

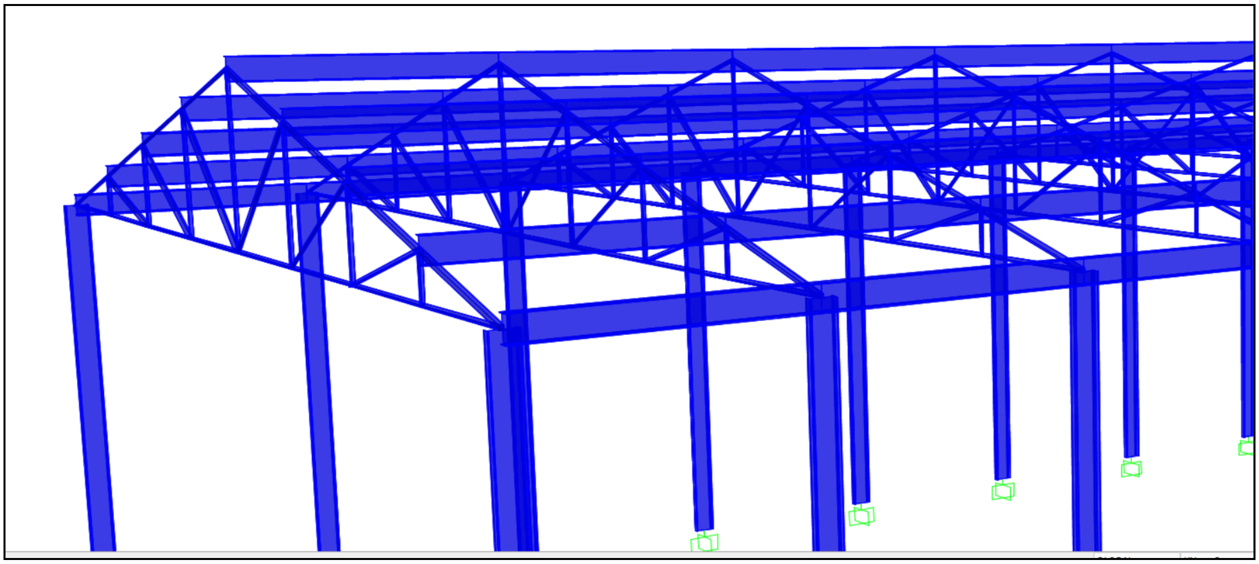


Fig 5: *3-D Sectional View*

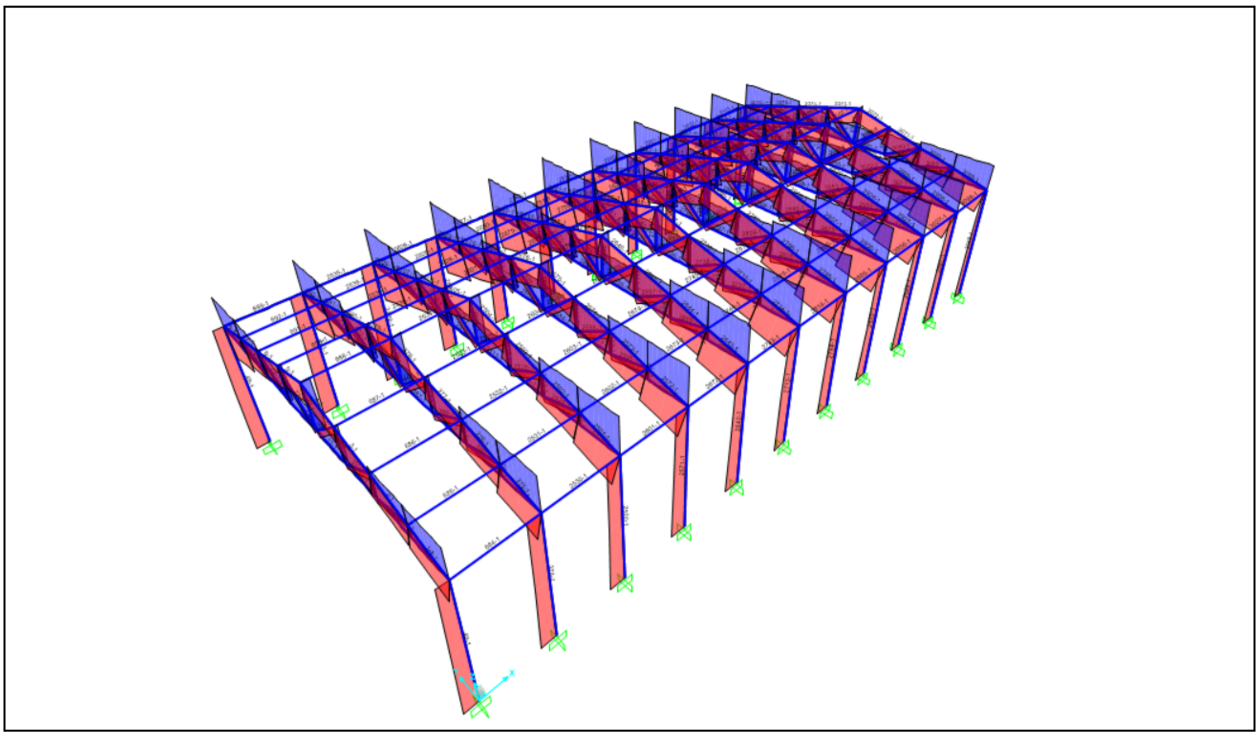


Fig 6: Axial *Force Model*

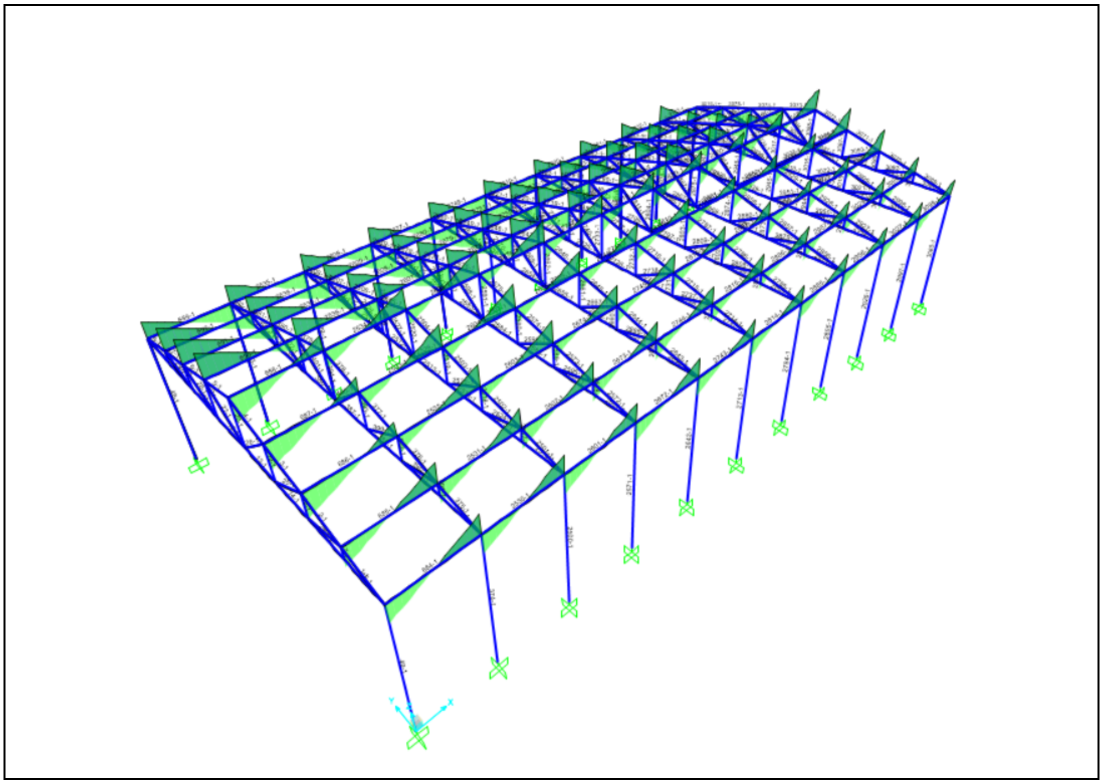


Fig 7: *Shear Force Model in y-z direction*

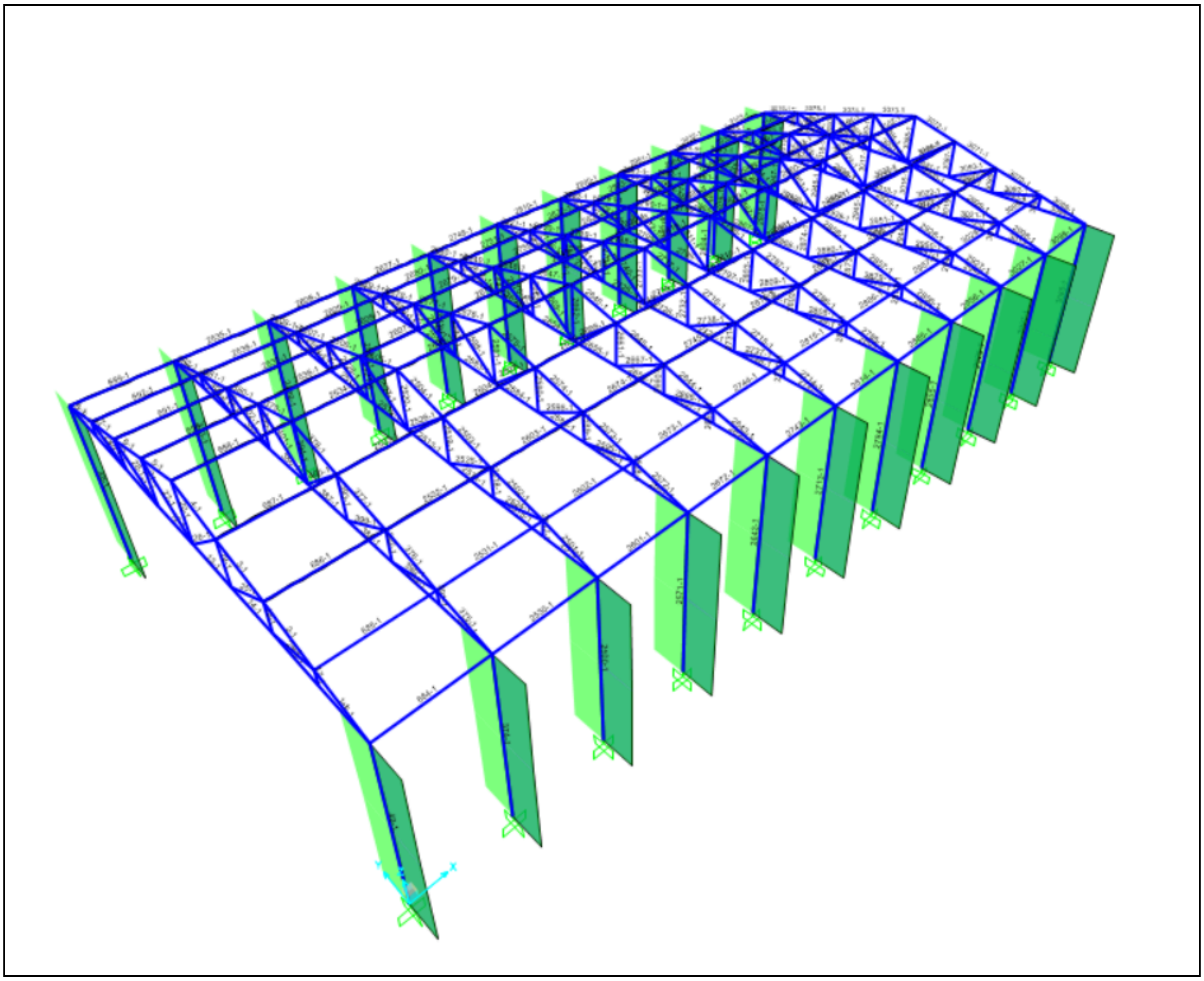


Fig 8: *Shear Force Model in x-z direction*

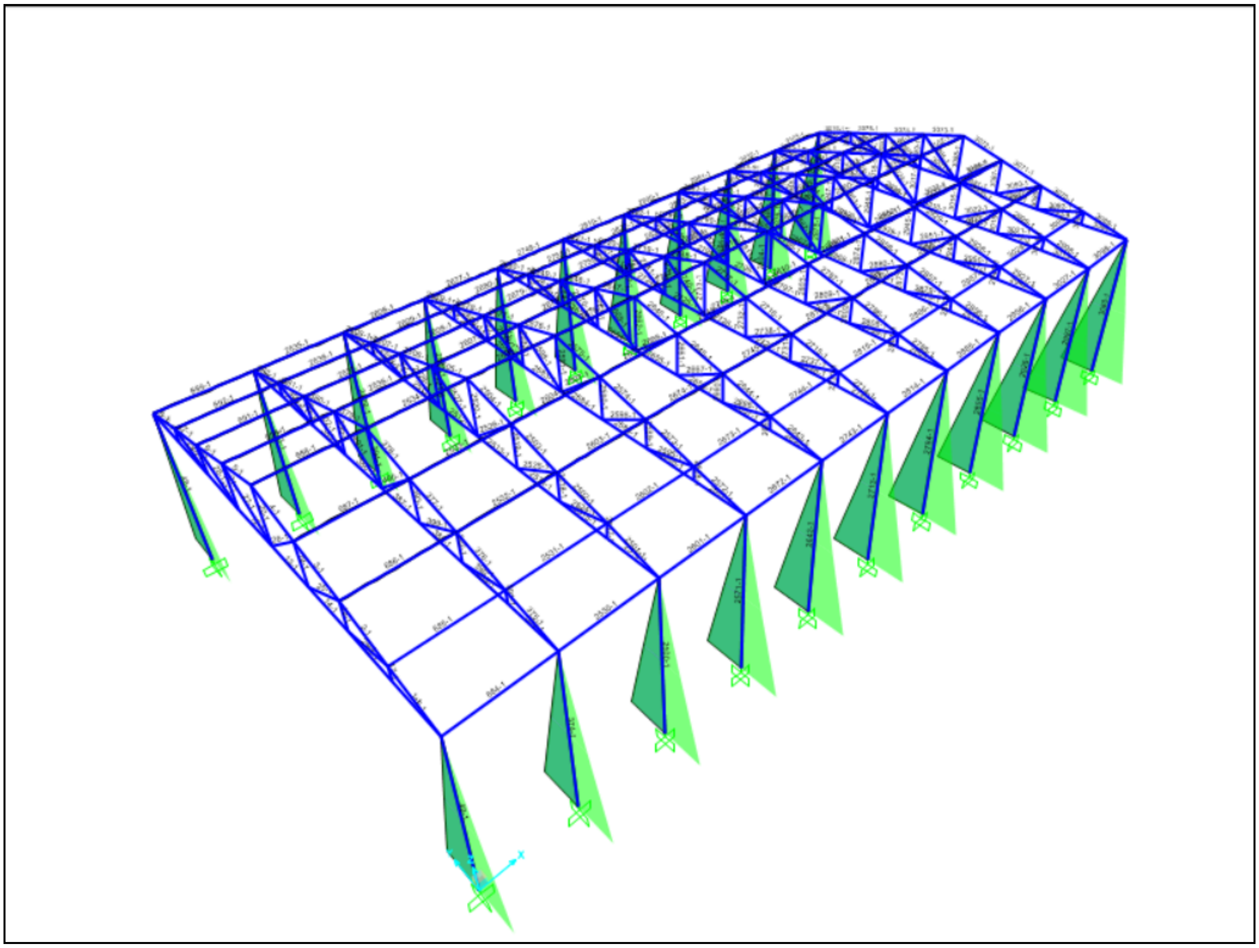


Fig 9: *Bending Moment Model in x-z direction*

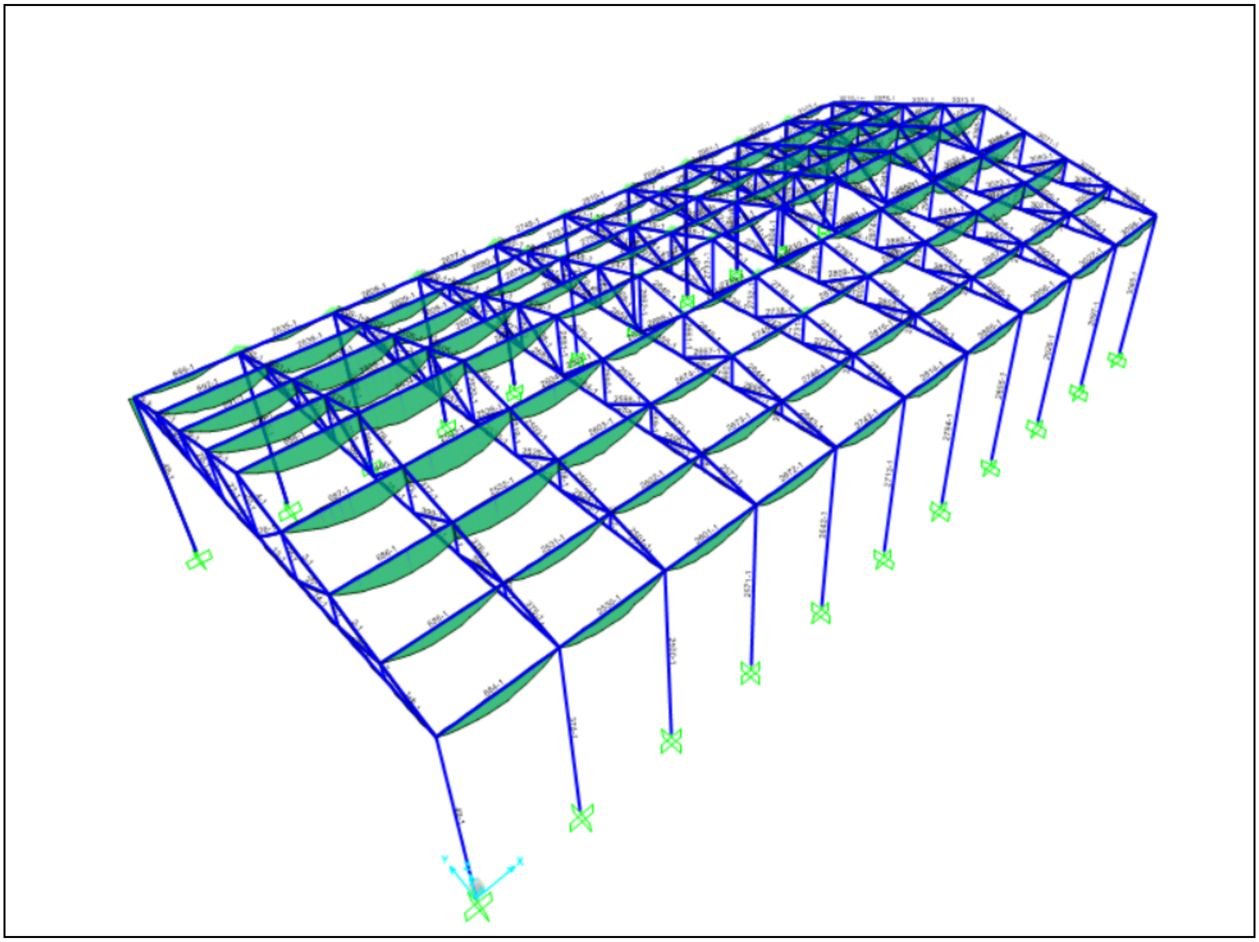


Fig 10: *Shear Force Model in y-z direction*

5.0 DESIGN SPECIFICATIONS

Analysis of the critical section is shown below based on inputs from SAP 2000

5.1 DESIGN OF TENSION MEMBER FOR CANTILEVER SHED

Tension member:-

critical tension member:

tension Load = 1648.18 kN

Let's take M24 bolts of grade 8.8

250 MPa and 410 MPa - yield and ultimate strength

Bolt calculation:-

→ Nominal dia (d) = 24 mm

Hole dia (d_0) = 26 mm

Edge distance (e) = 1.5 × 26 = 39 mm

Let's take e = 50 mm

pitch (p) = 2.5 × 24 = 60 mm

take p = 75 mm

Strength calculation:-

Shear capacity:

$$A_g = \pi \times \frac{24^2}{4} = 452.39\ mm^2$$

$$A_{nb} = 0.78 \times 452.39 = 352.86\ mm^2$$

$$V_{dsb} = \frac{f_u}{\sqrt{3}} A_{nb} \times \frac{1}{\gamma_{mb}} = \frac{(8 \times 100)}{\sqrt{3}} \times 352.86 \times \frac{1}{1.25}$$

$$\boxed{V_{dsb} = 130.38\ kN}$$

Bearing capacity:

$$k_b = \min\left\{\frac{e}{3d_0}, \frac{p}{3d_0} - 0.25, \frac{f_u}{f_{ub}}, 1\right\}$$

$$k_b = \min\left\{\frac{50}{3 \times 26}, \frac{75}{3 \times 26} - 0.25, \frac{400}{800}, 1\right\}$$

$$k_b = \min\{$$

$k_b = \min\{0.641, 0.647, 0.5, 1\}$

$k_b = 0.5$

$V_{dpb} = 2.5 \times k_b \times d\,t \times f_u \times \frac{1}{\gamma_{mb}}$

$= 2.5 \times 0.5 \times 24 \times 25 \times 410 \times \frac{1}{1.25}$

$V_{dpb} = \cancel{118.08\,kN}\ 246\,kN$

→ Bolt value $= \min\{130.38, \overset{246\,kN}{\cancel{118.08}}\}$

Bolt value $\simeq \cancel{118\,kN}\ \underline{130\,kN}$

→ Number of bolts: $\frac{1648.18}{130} \simeq 13$ bolts

Section Selection:

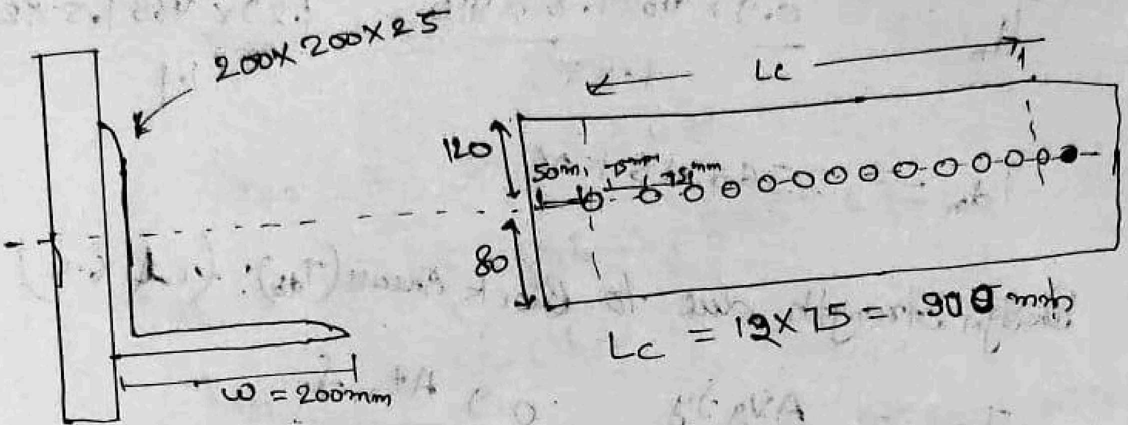
required area: $A_g = \frac{T_{dg} \cdot \gamma_{m0}}{f_y}$

$= \frac{1648.18 \times 1.1}{250}$

$A_{g_{req}} = 7251\,mm^2$

Let's take 200 × 200 × 25 angle section

$A_g = 9410\,mm^2$



$L_c = 12 \times 75 = 900$ mm

Design strength due to yielding of cross section (T_{dg}): (Cl 6.2)

$$T_{dg} = \frac{A_g f_y}{\gamma_{m0}} = \frac{9410 \times 250 \times 10^{-3}}{1.1}$$

$$T_{dg} = 2138.63 \text{ KN}$$

Design strength due to Rupture of critical section (T_{dn}): (Cl 6.3)

$$T_{dn} = \frac{0.9 A_{nc} f_u}{\gamma_{m1}} + \frac{\beta A_{go} f_y}{\gamma_{m0}}$$

$$\beta = 1.4 - 0.076\left(\frac{\omega}{t}\right)\left(\frac{f_y}{f_u}\right)\left(\frac{b_s}{L_c}\right)$$

$$= 1.4 - 0.076\left(\frac{200}{25}\right)\left(\frac{250}{410}\right)\left(\frac{200+80-25}{900}\right)$$

$$\beta = 1.29 \le \left\{\frac{f_u \, \gamma_{m0}}{f_y \, \gamma_{m1}} = 1.44\right\}$$

$$\ge 0.7 \quad \text{Ok}$$

$$A_{go} = \left(200 - \frac{25}{2}\right) \times 25 = 4687.5 \text{ mm}^2$$

$$A_{nc} = \left(200 - 26 - \frac{25}{2}\right) \times 25 = 4037.5 \text{ mm}^2$$

$$T_{d_n} = \frac{0.9 \times 4037.5 \times 410}{1.25} + \frac{1.29 \times 4687.5 \times 250}{1.1}$$

$$T_{d_n} = 2566.16 \text{ kN}$$

Design Strength due to Block Shear (T_{db}): (cl 6.4)

$$T_{db_1} = \frac{A_{vg} f_y}{\sqrt{3}\,\gamma_{m0}} + \frac{0.9\,A_{tn} f_u}{\gamma_{m1}}$$

or

$$T_{db_2} = \frac{0.9\,A_{vn} f_u}{\sqrt{3}\,\gamma_{m1}} + \frac{A_{tg} f_y}{\gamma_{m0}}$$

$$A_{vg} = (50 + 12 \times 75) \times 25 = 23750 \text{ mm}^2$$

$$A_{vn} = \left[(50 + 12 \times 75) - (12.5 \times 26)\right] \times 25$$

$$= 16275 \text{ mm}^2$$

$$A_{tg} = (120 \times 25) = 3000 \text{ mm}^2$$

$$A_{tn} = (120 \times 25) - (0.5 \times 26) \times 25$$

$$A_{tn} = 2675 \text{ mm}^2$$

$$T_{db_1} = \frac{23750 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 2675 \times 410}{1.25}$$

$$T_{db_1} = \cancel{3116}\ 3906.03 \text{ kN}$$

$$T_{db_2} = \frac{0.9 \times 16275 \times 410}{\sqrt{3} \times 1.25} + \frac{3000 \times 250}{1.1}$$

$$T_{db_2} = 3455.6 \text{ kN}$$

$$T_{db} = \min \{3906.03,\ 3455.6\}$$

$$T_{db} = 3455.6 \text{ kN}$$

$$\text{tensile strength} = \min \{2138.63,\ 2566.16,\ 3455.6\}$$

$$\text{tensile strength} = 2138 \text{ kN} > 1648.18 \text{ kN}$$

Hence the section is safe.

5.2 DESIGN OF COMPRESSION MEMBER FOR CANTILEVER SHED

Compression Member.

Factored Axial load = -1706.57 kN,
Effective length in y-axis = 4.09 m } Max length for Buckling.
Let design compressive strength,

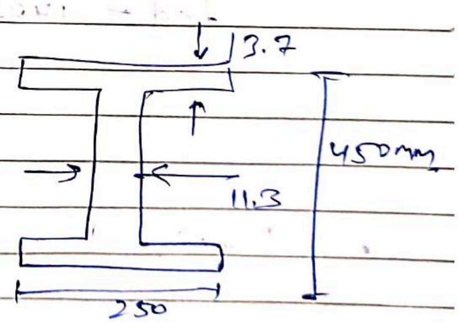
$f_{cd} = 0.6 f_y$.
$f_{cd} = 0.6 \times 250$
$f_{cd} = 150$ MPa

effective sectional Area

$A_e = \frac{1706.57 \times 1000}{150}$ clause 7.3.2

$A_e = 11378$ mm^2

Assume ISHB 450
Area = 11700 mm^2
D = 450 mm
B = 250
t = 11.3
T = 13.7.



We need to check only in y-axis because we always do in critical load condition. and take maximum length member.

for this, $r_y = 50.4$ mm

- Buckling Class

$$\frac{h}{b_f} = \frac{450}{250} = 1.8 > 1.2$$

$$t_f = 13.7 \text{ mm} < 40 \text{ mm}$$

class 'a' z-z axis Table 10
class 'b' y-y axis. Clause 7.1.2.2

- Slenderness Ratio.

$$\frac{KL}{r_y} = \frac{4.09 \times 10^3}{50.4} = 81.15$$

- Design compressive Strength

$f_{cd} = 148.16$ {By linear interpolation}
{Table 9 (b)}
Clause 7.1.2.1

- Design compressive strength P_d

$$P < P_d.$$

$$P_d = A_e f_{cd}. \quad \text{Clause 7.1.2}$$

$$P_d = \frac{11700 \times 148.16}{1000}$$

$$P_d = 1733.472 > 1706.57$$

Hence, section is adequate

Page No:

5.3 DESIGN OF BEAM

Beam

$M = 152.8\ kN\text{-}m$
$V = 73.722\ kN$
$L = 6.324\ m$
$f_y = 250\ MPa$
$f_u = 410\ MPa$

Laterally Supported Beam

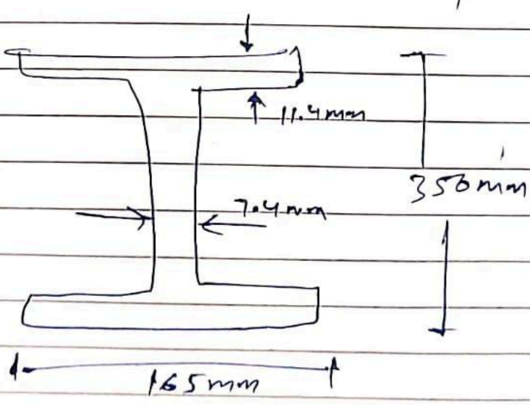
- Section modulus calculation

$$\therefore Z_p = \frac{M \gamma_{m0}}{f_y} = \frac{152.8 \times 10^6 \times 1.1}{250}$$

$Z_p = 672.32\ mm^3$
$Z_p \simeq 673\ mm^3$

$Z_p + 20\%\ Z_p \simeq 808\ mm^3$

→ let choose ISLB 350 from IS808



$D - (T+R).2$

Web depth $(d) = 350 - 2(11.4 + 16)$
$= 295.2\ mm$

$I_{zz} = 13100 \times 10^4\ mm^4$
$Z_e = 752 \times 10^3\ mm^3$
$Z_p = 851 \times 10^3\ mm^3$

Page No.

- Section classification.

$$\epsilon = \sqrt{\frac{250}{f_y}} = \sqrt{\frac{250}{250}} = 1$$

$$\frac{b}{t_f} = \frac{0.5 \times 165}{11.4} = 7.24 < 9.4\epsilon \quad \{\text{Plastic}\}$$

{clauses 3.7.2 and 3.7.4}

$$\frac{d}{t_w} = \frac{295.2}{7.4} = 39.93 < 84$$

{clause 3.7.2 and 3.7.4}

↳ Plastic section

- Shear Design.

Nominal plastic shear under pure shear is given by

$$V_n = V_p$$

where,

$$V_p = \frac{A_v f_{yw}}{\sqrt{3}}$$

A_v = shear area

f_{yw} = yield strength of web.

and the design strength

$$V_d = \frac{V_n}{\gamma_{mo}}$$

$$\Rightarrow V_d = \frac{A_v f_{yw}}{\gamma_{mo} \sqrt{3}}$$

{Clause 8.4}

Page No.

$\therefore$ ~~V_d~~ $V_d = \frac{350 \times 7.4 \times 250}{\sqrt{3} \times 1.1}$

$V_d = 339.849$ KN

$V_d \simeq 340$ KN

$\hookrightarrow$ $V_d > V$

i.e $V_d > 150$

and $0.6\, V_d \doteq 204$ kN > 150 kN

- Flexure capacity check

$M_d = \frac{\beta_b Z_p f_y}{\gamma_{mo}}$ clause 8.2.1.2

where, $\beta_b = 1$ {Plastic section}

$M_d = \frac{1 \times 851.11 \times 10^3 \times 250}{1.1}$

$M_d = 193.43$ kN-m

This beam is a simply supported beam

so, $M_d < \frac{1.5 \cdot Z_e \cdot f_y}{\gamma_{mo}} = 257$ KN.m

clause 8.2.1.2

$M_d < 257$ kNm

$\hookrightarrow$ Hence. the section is adequate.

5.4 DESIGN OF COLUMN

Column design

Factored Axial Load = 573.342 kN $\approx$ 576 kN

Factored Moment (M_z) = 145.41 kN $\approx$ 146 kN/m

250 MPa & 410 MPa - Yield & Ultimate strength

Effective Length $= \frac{0.8L}{3}$; L (Height) = 12.65 m

Effective length $= \frac{0.8 \times 12650}{3} = \frac{10120}{3} = 3373.33$ mm

Trial Section

- Assume ISHB 250

Effective Length = 3373.33 mm

$$\frac{kL}{3r_z} = \frac{3373.33}{109} = 30.94 \approx 31$$

$$P_{eff} = P + \frac{2M_z}{d} = 1744 \text{ kN}$$

- Buckling Classification
  - $\rightarrow \frac{h}{b_f} = \frac{250}{250} = 1 < 1.2$ $\rightarrow t_f = 9.7\text{mm} \nless 40\text{mm}$
  - $\rightarrow$ Buckling class - 'c'
- Table 9c f_{cd} = 211 MPa
- Capacity of section $= \frac{211 \times 6500}{1000} = 1371.5$ kN < 1744 kN ✗

Reverse Section ISMB 400

→ Effective Length:

$L_{eff} \div \frac{KL}{3} = \frac{10120mm}{3} = 3373.33$ mm

$\frac{KL}{3\times r} = \frac{10120}{3\times 161} = \frac{62.85}{3} \approx 20$

$P_{eff} = P + \frac{2M_z}{d}$

$= 576 + \frac{2\times 146000}{400}$

$P_{eff} = 1306$ kN

- Buckling Classification

→ $\frac{h}{b_f} = \frac{400}{140} = 2.85 > 1.2$

→ $t_f = 16$ mm < 40 mm

→ Buckling Class - 'a'

- Table 9(a) $f_{cd} = 226$ MPa

- Capacity of Section $= \frac{226 \times 7840}{1000}$

$= 1771.84 \approx 1772$ kN > 1306 kN

- $Z_{pz} = 1170 \times 10^3\ mm^3$
- $Z_{py} = 149 \times 10^3\ mm^3$

- Section Classification

→ $\epsilon = \sqrt{\frac{250}{f_y}} = 1$

- $\frac{b}{t_f} = \frac{400/2}{16} = 12.5 < 15.7$

- $\frac{d}{t_w} = \frac{D - 2t_f - 2R}{t_w} = \frac{400 - (2\times16) - (2\times14)}{8.9}$

- $\frac{d}{t_w} = 38.2 < 42.$

- Semi- Compact section.

- Resistance of Cross - section (cl 9.3.1.3)

- $N_d = \frac{A_g f_y}{\gamma_{m0}}$

$$= \frac{7840 \times 250}{1.1}$$

$$\underline{N_d = 1781.81 \text{ kN}}$$

- $M_d = \frac{\beta_b Z_p f_y}{\gamma_{m0}}$

for semi- compact section = $\beta_b = \frac{Z_c}{Z_p}$

$$= \frac{Z_c \cancel{Z_p} f_y}{\cancel{Z_p} \gamma_{m0}} = \frac{Z_c f_y}{\gamma_{m0}}$$

$$M_d = \frac{1020 \times 10^3 \times 250}{1.1}$$

$$\underline{M_d = 231.8 \text{ kN/m}}$$

- Interaction :

$$\frac{N}{N_d} + \frac{M}{M_d} \leq 1.0.$$

$$\frac{576}{1781.81} + \frac{146}{231.8} = 0.953 < 1.0$$

- Hence section is safe.

- <u>Buckling Resistance in compression (cl 7.1.2)</u>

  - $\frac{KL}{r_z} = \frac{10120}{16\cancel{k}3} = \frac{62.85}{3} \approx 20$
  - $\frac{KL}{r_y} = \frac{10120}{3 \times 28.1} = \frac{360.14}{3} = 120.04 \approx 120$

  - Buckling classification (Table 10)

    - $\frac{h}{b_f} = \frac{400}{140} = 2.85 > 1.2$

    - $t_f = 16\,mm < 40\,mm$

    - Major Axis Buckling class -'a'
    - Minor Axis Buckling class - 'b'

  - Table 9a : $f_{cd} = 226$ MPa

$$P_d = \frac{226 \times 7840}{1000} = 1771.84 \text{ kN} > 576 \text{ kN}$$

  - Table 9b : $f_{cd} = 91.7$ MPa

$$P_d = \frac{91.7 \times 7840}{1000} = 718.928 \text{ kN} > 576 \text{ kN}$$

  - Hence Section is safe.

- Buckling resistance in Bending (Cl 8.2.2)

  - $\frac{KL}{3 \times r_2} = \frac{10120}{3 \times 161} \approx 20$

  - $\frac{h}{t_f} = \frac{400}{16} = 25$

  - Table 14

    $f_{cr,b} = 5515.8$ MPa

  - $\lambda_{LT} = \sqrt{\frac{f_y}{f_{cr,b}}} = \sqrt{\frac{250}{5515.8}} = \underline{0.22}$

  - $\phi_{LT} = 0.5\left[1 + \alpha_{LT}\left(\lambda_{LT} - 0.2\right) + \lambda_{LT}^2\right]$

    for rolled steel section

    $\alpha_{LT} = 0.21 \Rightarrow \phi_{LT} = 0.5\left[1 + 0.21\left(0.22 - 0.2\right) + 0.22^2\right]$

    for welded steel section $\qquad \underline{\phi_{LT} = 0.5263 \approx 0.5}$

    $\alpha_{LT} = 0.49$

  - $\chi_{LT} = \frac{1}{\left\{\phi_{LT} + \left[\phi_{LT}^2 - \lambda_{LT}^2\right]^{0.5}\right\}} \leq 1.0$

    $= \frac{1}{\left\{0.5 + \left[0.5^2 - 0.22^2\right]^{0.5}\right\}}$

    $\chi_{LT} = \underline{0.957 \leq 1.0}$

$$f_{bd} = \frac{\chi_{LT} f_y}{\gamma_{m0}}$$

$$= \frac{0.957 \times 250}{1.1}$$

$$f_{bd} = 217.5 \text{ MPa}.$$

- $M_d = \beta_b Z_p f_{bd}$

for semi-compact $\beta_b = \frac{Z_c}{Z_p}$

$$M_d = Z_c \times f_{bd}$$

$$= 1020 \times 10^3 \times 217.5$$

$$M_d = 221.85 \text{ kN/m} > 146 \text{ kN/m}$$

- Hence section is safe

- Overall Member strength (cl 9.3.2)

- $$\lambda_z = \sqrt{\frac{f_y \left(\frac{KL}{r_z}\right)^2}{\pi^2 E}} \qquad \boxed{E = 2\times10^5}$$

$$= \sqrt{\frac{250 \times (70)^2}{\pi^2 \times 2\times10^5}} \Rightarrow \underline{\lambda_z = 0.22}$$

- $K_z = 1 + (\lambda_z - 0.2)\eta_z \leq 1 + 0.8\eta_z$

$$\eta_z = \frac{P}{P_z} = \frac{576}{1772} = 0.325$$

$$= 1 + (\lambda_z - 0.2)\eta_z \leq 1 + 0.8\eta_z$$

$$= 1 + (0.22 - 0.2)0.325 \leq 1 + 0.8(0.325)$$

$$= 1.0065 \leq 0.26$$

- $\psi_z = \frac{M_2}{M_1} = \frac{0}{146} = 0.$

$$C_{mz} = 0.6 + 0.4\psi = 0.6$$

- Interaction.

$$\frac{P}{P_{dz}} + k_z \frac{C_{mz} M_z}{M_d} \leq 1.0$$

$$\frac{576}{1772} + 1.0065 \times \frac{0.6 \times 146}{221.85} \leq 1.0$$

$$0.722 \leq 1.0$$

- Hence the section is safe against axial force and bending moment.

5.5 DESIGN OF BASE PLATE

Design of Base Plate

- ISMB 400 Column
- Factored Load = 576 kN
- Factored Moment = 146 kN/m

- Bearing strength of concrete
  - $0.45 f_{ck}$ =
- Eccentricity

$$e = \frac{M}{P} = \frac{146}{576} = 0.2534 m = 253.4 mm$$

- Plate Dimensions
  - For compressive pressure over whole base

$$\frac{P}{A} - \frac{My}{I} = 0$$

$$\Rightarrow D = 6e = 1520.4 mm$$

  - for safe bearing pressure

$$\frac{P}{A} + \frac{My}{I} = 0.45 f_{ck}$$

$$\Rightarrow B = \frac{2P}{L \times 0.45 f_{ck}} = 67.35 mm$$

- Provide rectangular plate of 1520.4mm × 317.35mm

  - $A = 1520.4 \times 317.35$

    $A = 482 \times 10^3\ mm^2$

  - $Z_e = \dfrac{317.35 \times 1520.4^2}{6}$

    $Z_e = 122.265 \times 10^6\ mm^3$

- Pressure Calculation

  - $P_{max} = \dfrac{P}{A} + \dfrac{My}{I} = \dfrac{576}{482 \times 10^3} + \dfrac{146 \times \left(\frac{1520.4}{2}\right)}{\dfrac{317.35 \times (1520.4)^3}{12}} = 2.39\ MPa$

  - $P_{min} = \dfrac{P}{A} - \dfrac{My}{I} = 0$

- Thickness of base plate

  - Pressure at section XX :

    $$P = \left(\frac{\text{distance from edge}}{\text{total length}}\right) \cdot P_{max}$$

    $$= \left(\frac{1520.4 - 460.2}{1520.4}\right) \times 2.39$$

    $P = 1.67\ MPa$

- Moment at section XX

$$M_x = \frac{1.67 \times (460.2)^2}{2} + \frac{(2.39 - 1.67) \times 460.2}{2} \times \frac{2}{3} \times 460.2$$

$$M_x = 227.67 \times 10^3 \text{ N mm}$$

- Moment capacity of plate

$$\frac{1.2 f_y Z_e}{\gamma_{mo}} = \frac{1.2 \times 250 \times t^2/6}{1.1}$$

$$= 45.45 t^2$$

- Equating the two equations :

- $227.67 \times 10^3 = 45.45 t^2$

$$t^2 = \frac{227.67 \times 10^3}{45.45}$$

$$t = \sqrt{\frac{227.67 \times 10^3}{45.45}}$$

$$t = 70.77 \approx 71 \text{mm}$$

- Provide 71 mm thick plate

5.6 DESIGN OF PURLIN

Purlin:

Length of member $L = 6.55\ m$

Shear force $V = 0.289\ kN$

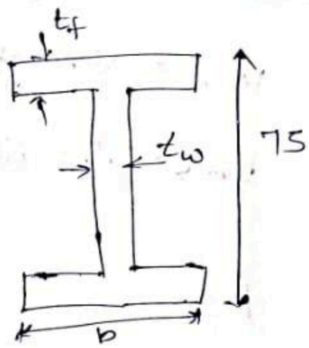
moment $M = 0.2956\ kN\text{-}m$

Section modulus calculation:

required $Z_p = \dfrac{M\,\gamma_{m0}}{f_y} = \dfrac{0.2956 \times 10^6 \times 1.1}{250}$

$Z_p = 1.3 \times 10^3\ mm^3$

Let's choose ISLB 75



$b = 50\ mm$

$t_f = 5\ mm$

web depth $d = 75 - 2(5 + 6.5)$

$d = 52\ mm$

$t_w = 3.7\ mm$

$Z_e = 19.3 \times 10^3\ mm^3$

$Z_p = 22.3 \times 10^3\ mm^3$

Section classification: according to table 2 IS800 (cl 3.7.2 and 3.7.4)

$\varepsilon = \sqrt{\dfrac{250}{f_y}} = 1$

$\dfrac{b}{t_f} = \dfrac{0.5 \times 50}{5} = 0.5 < 9.4$

$\dfrac{d}{t_w} = \dfrac{52}{3.7} = 14.05 < 84$

thus Section is plastic

Shear design: Cl 8.4

$$V_d = \frac{V_n}{\gamma_{m0}} = \frac{A_v f_y}{\sqrt{3}\,\gamma_{m0}}$$

$$V_d = \frac{75 \times 3.7 \times 250 \times 10^{-3}}{\sqrt{3} \times 1.1}$$

$$V_d = 36.41\ kN > 0.3\ kN$$

$$0.6 V_d = 21.84\ kN > 0.3\ kN$$

flexure capacity check:

$$M_d = \frac{B_0 Z_p f_y}{\gamma_{m0}}$$

[$B_0 = 1$ for plastic section]

$$M_d = \frac{1 \times 22.3 \times 10^3 \times 250 \times 10^{-6}}{1.1}$$

$$M_d = 5.068\ kN\text{-}m$$

$$M_d < \left[\frac{1.2\, Z_e f_y}{\gamma_{m0}} = \frac{1.2 \times 19.30 \times 10^3 \times 250 \times 10^{-6}}{1.1}\right]$$

$$M_d < 5.263\ kN\text{-}m$$

6.0 CONNECTION

6.1 CONNECTION OF BEAM WITH COLUMN

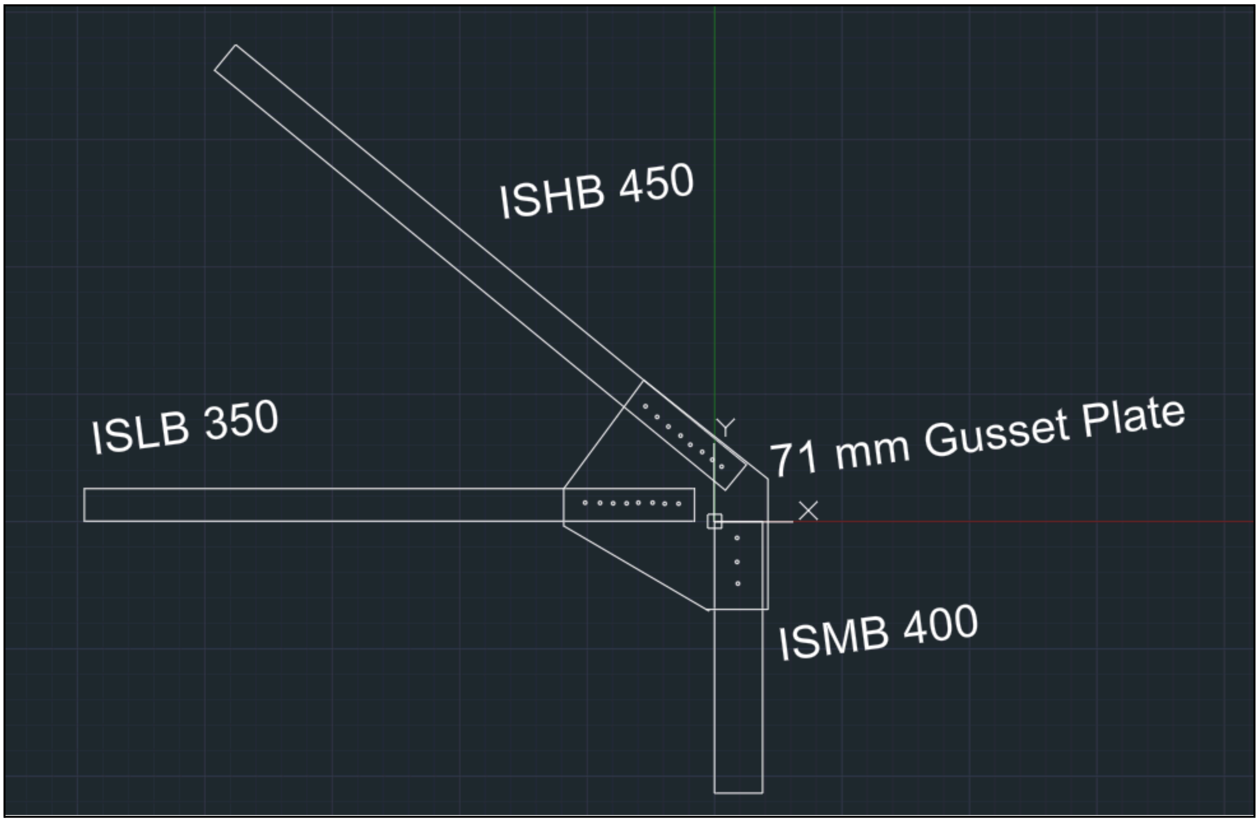


Fig 11: Column-Beam Connection

6.2 CONNECTION OF TRUSS MEMBER

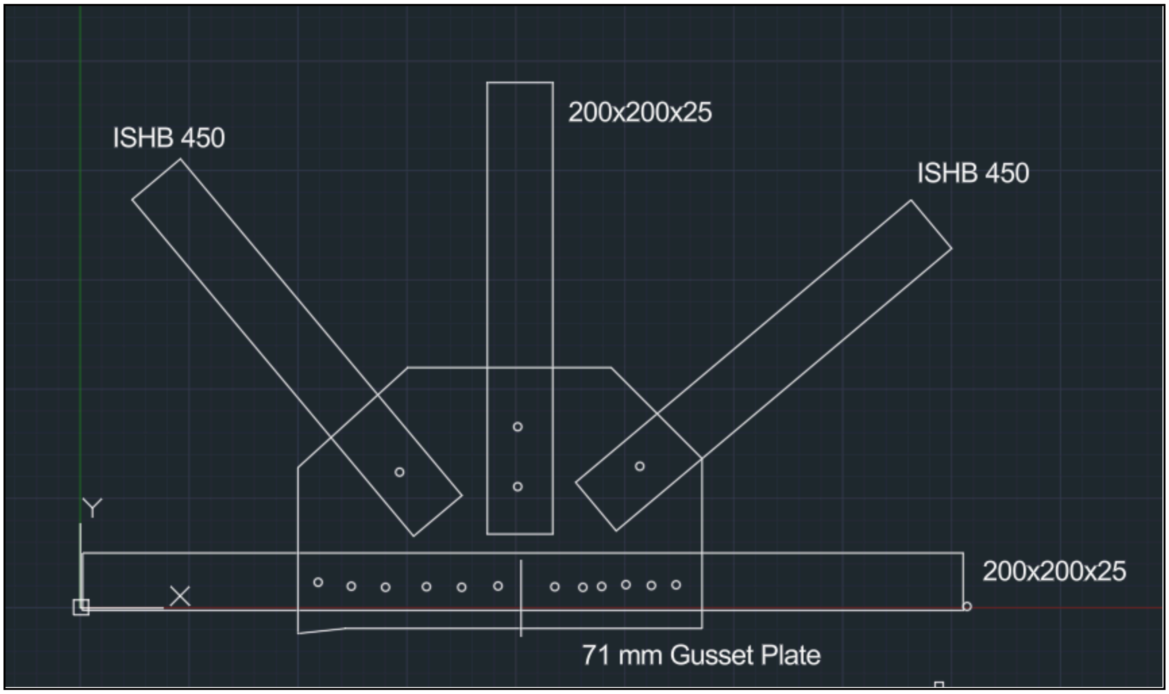


Fig 12: connection of truss member

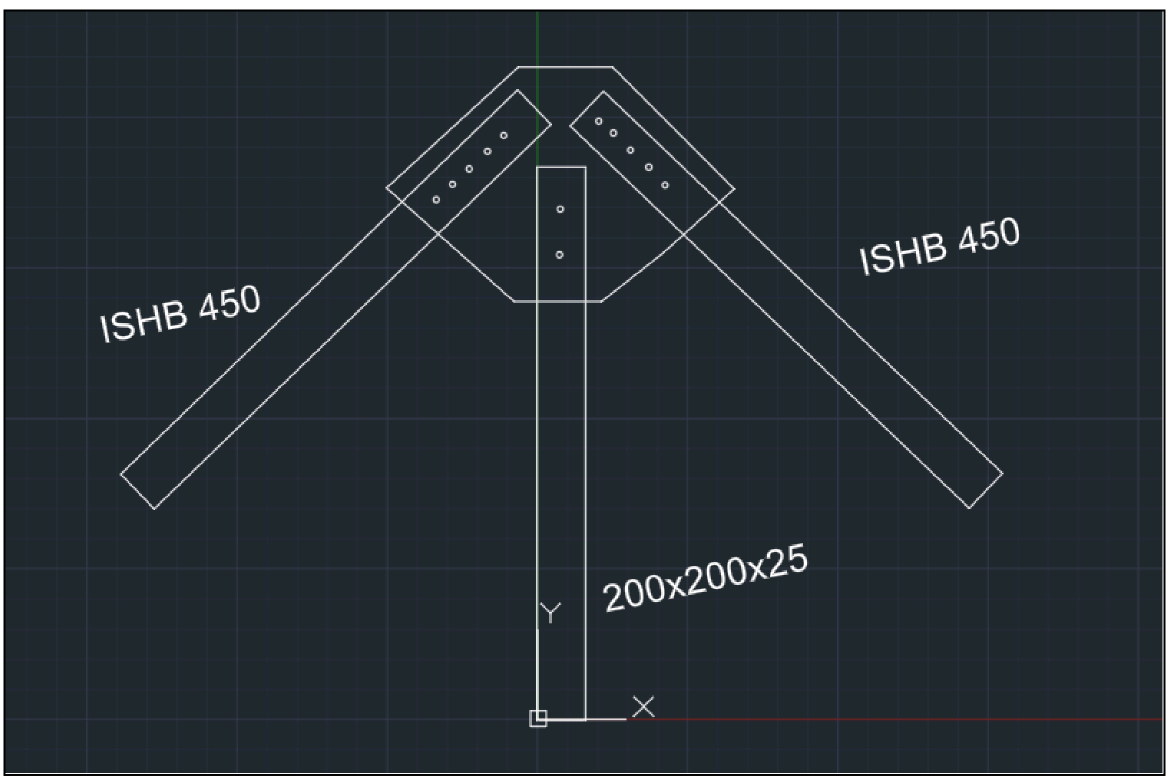


Fig 13: connection of truss member

7.0 SUMMARY AND CONCLUSIONS:

This project focuses on the design of an industrial shed spanning a 2000 m² plot in Ahmedabad, emphasizing the practical application of principles from the Design of Steel Structures course. The shed features a rectangular layout with a 2:1 aspect ratio and incorporates structural elements such as trusses, purlins, columns, and baseplates. Loads considered include dead, live, and wind loads, designed per the IS 875 standards.

1. **Design Integrity**: The structural elements are robust and satisfy all load combination requirements, ensuring safety against operational and environmental forces.
2. **Code Compliance**: Adherence to IS 800, IS 808, and IS 875 enhances reliability and aligns the design with industry standards.
3. **Practical Application**: The project brings theoretical knowledge with real-world implementation, reinforcing the significance of structural analysis and design optimization.
4. **Analytical Accuracy**: Using the SAP2000 tool for finite element analysis ensured precise modeling of structural behavior, enabling an optimized design.
5. **Future Implementation**: The design principles applied can serve as a framework for similar industrial projects, offering scalability and adaptability.

The project demonstrates a comprehensive approach to industrial shed design, showcasing technical competence and practical feasibility.

8.0 Acknowledgement

We are grateful to Professor Dhiman Basu, sir of the CE 312 (Design of Steel Structures) course, for giving us this fantastic opportunity to work on a group project. We want to thank Prof. for their invaluable guidance and unwavering support throughout this project. The professor was the one who guided us and inspired us to take on this project with various complexities and motivated us to reach our objectives as far as possible. We are also grateful to teaching assistants (TAs) for helping us during the completion of the project.