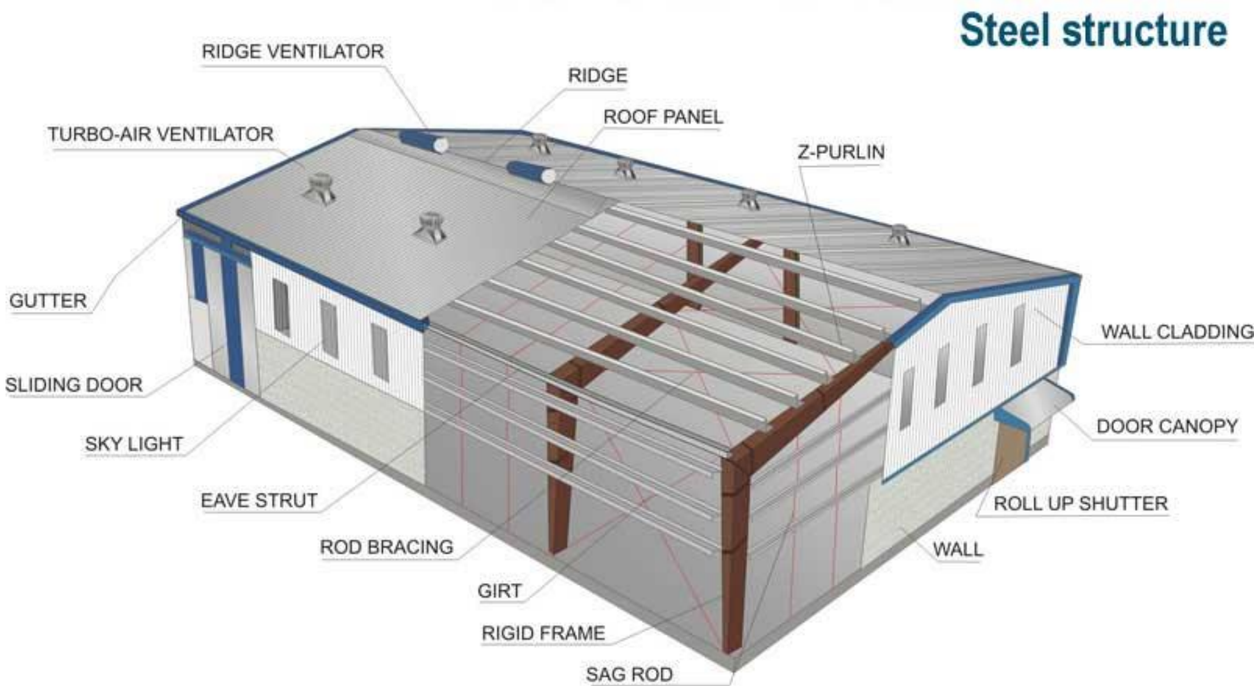
Best Practice in Steel Construction

Established 2013

Steel Project Report

INDUSTRIAL BUILDINGS
Guidance for Architects, Designers & Constructors



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Steel Project Report

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Design Details

Design an Industrial structure in steel construction in steel located in Jaipur for the details given below. Consider all possible loads and load combinations including wind. Use STAAD Pro for structural analysis and Auto CAD drawings for steel detailing. The roofing may be considered galvanized iron (GI) sheets with appropriate gauge. Design typical connection details either bolted or welded (shop or site). The columns could be laced or battened. The foundation may be designed as per the load and subsoil conditions. Provide wind bracings, eaves grider. side cladding , girt angles etc. appropriately.Show all the necessary checks as per relevant IS code.

- Live Load = 0.75 kN/m2
- Dead Load = Self weight + 20% Extra
- Crane Capacity = 100 kN
- Safe Bearing Capacity (SBC) = 20 kN/m2
- Span Length (L) = 15 m
- Bay Length (10 nos.) = 5 m
- Height (H1) = 7 m
- Height (H2) = 6 m

Note: These are the details provided by the contractor for the structure

Jaipur Climatic Topography

- Jaipur has a hot semi-arid climate receiving over 650 millimetres (26 in) of rainfall annually but most rains occur in the monsoon months between June and September.
- Temperatures remain relatively high throughout the year, with the summer months of April to early July having average daily temperatures of around 30 °C (86 °F).
- During the monsoon there are frequent, heavy rains and thunderstorms, but flooding is not common.
- The winter months of November to February are mild and pleasant, with average temperatures ranging from 15–18 °C (59–64 °F) and with little or no humidity.
- There are however occasional cold waves that lead to temperatures near freezing.

From the above climatic conditions and topography we come to conclusion that Jaipur is a warm place, with flat terrain and has a occasional cold waves and normal wind speed.

Note: The climatic and topography results were taken from Wikipedia.

Selection of Roofing Material

- We have three options for roofing material
  - Steel or Aluminum Decking Sheets
    - Since Jaipur experiences vary higher temperature so, the aluminum would expand much larger than the steel. (Note. - Aluminum expand approximately twice as much steel and are easily damaged in hailstorms.). Thus, the aluminum sheets cannot be used.
  - Corrugated Asbestos (AC) Sheets

  - Asbestos sheets are now obsolete these days and also have health concerns associated with the fibers it's made of.
  - Also, because of change in temperature from high in day time to low in night, the AC sheets may experience cracking.
- Corrugated Galvanized Iron (GI) Sheets
  - GI sheets prove best for the given location as there is no differential effect of temperature on the sheets and the connections as they all are made up of iron.
  - Also, they are most commonly used and come in variable sizes.
  - To prevent it from water leakage, washer will be used.

Thus, for this particular industrial steel structure, the roofing material chosen in Corrugated Galvanized Iron Sheets.

Specifications of GI Sheets

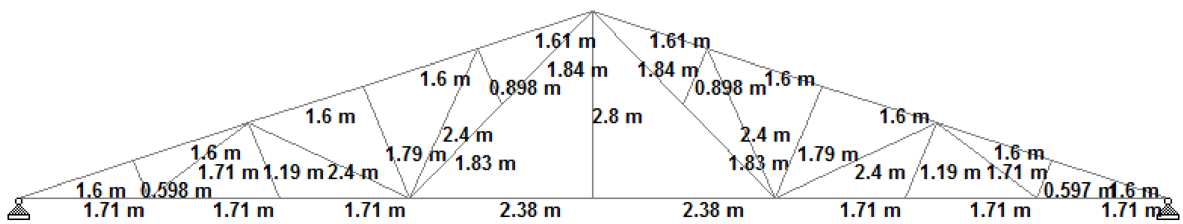
- 8 corrugations (75 mm wide and 19 mm deep) per sheet
- Size of the sheet
  - Length = 1.8m
  - Width = .75m
  - Thickness = .63mm
- Assumed weight of the sheets =85 N/m2 (50-156 N/m2) --- Design of Steel Structures,Subramanyam (1134)
- For roof
  - Side overlap = $1\frac{1}{2}$ corrugations
  - End Overlap = 150 mm
- For side cladding
  - Side Overlap = 1 corrugations
  - End Overlap = 100 mm

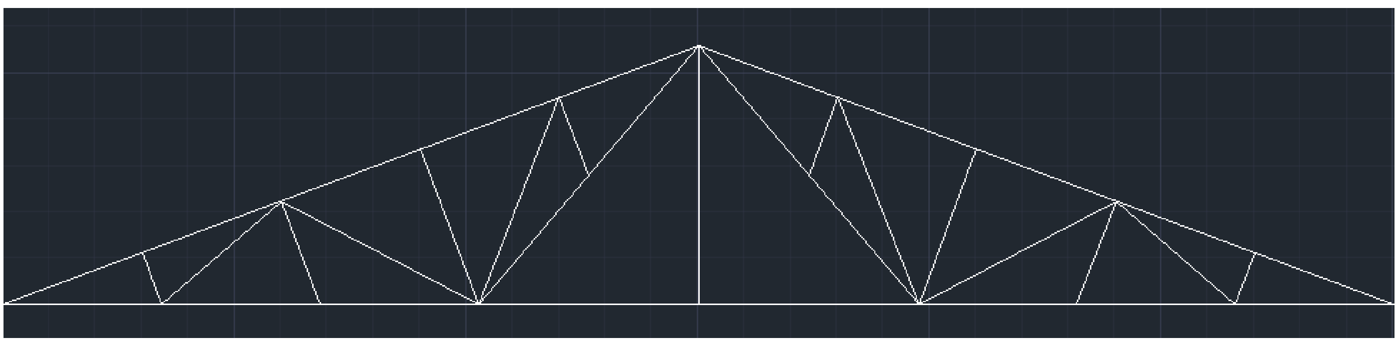
Selection of Truss

- Type of truss
  - Given Span of the Truss = 15m
  - Most Economical and appropriate truss = Fink Truss -- Design of Steel Structures,Subramanyam (1158)
- Spacing of truss
  - Bay length = 5m
  - Spacing of truss = 1/5 to 1/3 of span(15m) Design of Steel Structures,Subramanyam (1159)
    = 3m to 5m
  - Therefor Spacing is 5m
- Depth of Truss
  - Range for Fink Truss= L/6 to L/5
    = 2.5m to 3m
  - Assumed Depth of Truss within the limits= 2.8 m
- Spacing of Purlins
  - The spacing of the purlins should be between 1.5m to 1.75m
  - Assumed spacing of the purlins = 1.6m

Details of Truss

- Depth of the truss = 2.8 m
- Slope of the truss = Tan^{-1} (2.8/7.5) = 20.47228^{0}
- Span of the truss = 15 m
- Spacing of purlins = 1.6 m





Calculation of Dead Loads

- GI sheeting = 0.085
- Fixings = 0.025
- Services = 0.100
- Total Load = 0.210 kN/m2
- For 5 m bays,
  - Roof Dead Load = 0.21 x 5 x 15 = 15.75 kN
  - Weight of the purlin (assuming 70 N/m2) = 0.07 x 5 x 15
    = 5.25 kN
  - For welded sheet roof trusses , the self-weight is approximately given by
    - W = 53.7 + 0.53A
      = 53.7 + 0.53 x 5 x 15
      = 93.45 N/m2
  - Self-weight of one truss = 0.09345 x 5 x 15
    = 7.00 kN
  - Total Dead Load = 15.75 + 5.25 + 7
    = 28 kN

Calculation of Live Loads

- Live Load = 0.75 kN/m2
- Reduction due to Slope =0.75 - 0.02 x (20-10)
  =0.55 kN/m2

Calculation of Wind Loads

- As per IS 875 (Part 3) - 1987
- Basic Wind speed in Jaipur V_b = 47 m/sec
- Design Wind Speed (V_z) = V_b K_1 K_2 K_3 K_4
  - Probability factor or risk coefficient (K_1)
    - Class of Structure -> All general Buildings and Structures
    - Mean Probable Design Life -> 50 years
    - Basic Wind Speed -> 47 m/sec
    - Therefore , K_1 = 1
  - Terrain and Height Factor (K_2)
    - Category 1
    - Height = 10m
    - Therefore , K_2 = 1.05
  - Topography factor (K_3)
    - Slope < 3^0
    - Therefore , K_3 = 1
  - Importance Factor
    - For Industrial Structures
    - Therefore , K_4 = 1.15
- Design Wind Speed (V_z) = 47 x 1 x 1.05 x 1 x 1.15
  = 56.7525 m/sec
- Design Wind Pressure (P_d)= K_d x K_a x K_c x 0.6 x V_z x V_z
  = K_d x K_a x K_c x 1.932 kN/m²
  - Wind Directionality factor (K_d)
    - For circular , near circular and assymetric sections , which offer a uniform resistance , irrespective of the wind , K_d is taken as 1.0
  - Area Averaging Factor (K_a)
    - Tribute Area <= 10 m²
    - Therefore , K_a = 1.0
  - Combination Factor (K_c)
    - Kc = 1
- Design Wind Pressure (P_d)= K_d (1) x K_a (1)x K_c (1) x 1.932 kN/m²
  = 1.932 kN/m²
- Wind Pressure on Roofs
  - F = (C_{pe} - C_{pi}) A x P_d
    - h/w ratio = 7/15 = 0.4667 < ½
    - Truss Slope = 20^0 approxx

- Also , we assume medium permeability (5 – 20 % openings in wall area)
    - For industrial buildings
    - C_{pi} = +5 or -5

Wind Angle	Pressure Coefficient			C_{pe} (+/-) C_{pi}		A x P_d (kN)	Wind Load , F (kN)	
	C_{pe}		C_{pi}					
	Windward	Leeward		Windward	Leeward		Windward	Leeward
0^0	-0.4	-0.4	-0.5	-0.9	-0.9	15.456	-13.9104	-13.9104
			0.5	0.1	0.1	15.456	1.5456	1.5456
90^0	-0.7	-0.6	-0.5	-1.2	-1.1	15.456	-18.5472	-17.0016
			0.5	-0.2	-0.1	15.456	-3.0912	-1.5456

- Therefore, Design Wind Pressure = -18.5472/8 = 2.3 kN/m2

Final Load Conditions

- Dead Load = 0.37 kN/m^2
- Live Load = 0.46 kN/m^2
- Wind Load = 2.3 kN/m^2

Load Calculations from Gantry Girder

- **Derived details from Above**
    - Centre-to-center distance between columns (i.e. span of the gantry girder) = 5.0 m
    - Crane Capacity = 100 kN
    - Self-Weight of the crane girder excluding the trolley = 200 kN
    - Self- weight of the trolley, electrical motor , hook, etc. = 40 kN
    - Minimum Hook Approach = 1.2 m
    - Distance Between Wheel centers = 3.5 m
    - Centre-to-center distance between gantry rails (i.e. span of the crane) = 15.0 m
    - Self-weight of the rail section = 300 N/m
    - Yield Stress of Steel = 250 MPa

- **Loads and Bending Moments Calculation**
    - **Loads**
        - Vertical Loading
            - Maximum static wheel load due to weight of the crane = 100/4 = 25 kN
            - Maximum static wheel load due to crane load
                - $W_1 = [\ W_t\,(L_c - L_1)\ ]\ /\ (\ 2L_c)$
                  $= (100 + 40)\ (15 – 1.2)/\ (2 \text{ x } 15)$
                  $= 64.4$ kN
            - Total load due to the weight of the crane and the crane load = 50 + 64.4 = 114.4 kN
            - To allow for impact , etc. , this load should be multiplied by 25% (see table 12.3)

- Design Load = 114.4 x 1.25 = 143.0 kN
- Therefore , factored wheel load on each wheel ,
  - Wc = 143 x 1.5 = 214.5 kN

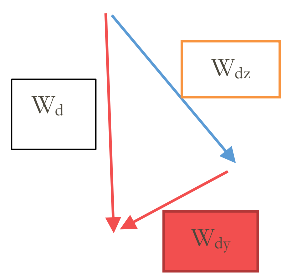
- Lateral (horizontal) surge load
  - Lateral load (per wheel) = 10% (hook + crab load) / 4
    = 0.1 x (200 + 40) / 4
    = 6 kN
  - Factored Lateral Load = 1.5 x 6 = 9 kN
- Longitudinal (horizontal) braking load
  - Horizontal force along the rails (Table 12.3)
    = 5% of wheel load
    = 0.05 x 143
    = 7.15 kN
  - Factored Load P_g = 1.5 x 7.15 = 10.725 kN

- **Maximum Bending Moment**
  - Vertical maximum bending moment
    - Without considering the self-weight ,
      - $M_1 = W_c L/4$ = 214.5 x 5/4 = 268.125 kNm
      - $M_2 = 2\ W_c (L/2 - c/4)^2 / L$
        = 2 x 214.5 $(5/2 - 3.5/4)^2 / 5$
        = 226.566 kNm
    - Hence M = 226.566 kNm
    - Assume that the self-weight of the gantry girder is 1.6kN/m
      - Total dead Load = 1600 + 300 (self-weight of rail) = 1.9 kN/m
      - Factored Dead Load = 1.9 x 1.5 = 2.85 kN/m
      - Bending Moment due to dead load = $wl^2/8$
        = $(2.85 \times 5^2/8)$
        = 8.9 kNm
  - Horizontal Bending Moment
    - Moment due to surge Load = 2 x 9 $(5/2 - 3.5/4)^2 / 5$
      M_y = 9.5 kNm
  - Bending moment due to drag
    - Assuming the rail height as 0.15 m and depth of girder as 0.6 m
      - Reaction due to drag force = $P_g\ e / L$
        = 10.725 (0.3 + 0.15)/5
        = 0.965
      - $M_3 = R(L/2 - c/4)$ = 0.965 (5/2 -3.5/4)
        1.5685 kNm
  - Total designed bending moment M_z
    - 226.566 + 8.9 + 1.57
      = 237.04 kNm
- **Shear Force**
  - Vertical Shear Force
    - Shear force due to wheel load
      - Wl (2- c/L) = 214.5 (2-3.5 /5)
        = 278.85 kN
    - Shear force due to Dead Load

- $WL/2 = 2.85 \times 5/2 = 7.125$ kN
- Maximum ultimate shear force
  - $V_z = 7.125 + 278.85$
    $= 285.975$ kN
- Lateral Shear force due to surge Load
  - $V_y = 9(2 – 3.5 / 5) = 11.7$ kN
  - Reaction due to drag Force = 0.965 kN
  - Maximum ultimate reaction
    - $R_z = 285.975$ kN + 0.965 kN
      = 286.94 or 287 kN

Design of Purlin

- **Derived Details from Above**
  - Dead load =0.36 kN/m²
  - Live load = 0.55 kN/m²
  - Wind pressure = 2.34 kN/m²
  - C-C distance between purlins = 1.6 m
  - Angle of roof (α)=$\tan^{-1}(\frac{2.8}{7.5})$
- **Load Calculation**
  - Dead Load= 0.36*1.6= 0.576 kN/m
  - Live load= 0.55*1.6= 0.88 kN/m
  - Wind Load= 2.34*1.6= 3.744 kN/m
  - W_{dz} =0.576* cos($\tan^{-1}(\frac{2.8}{7.5})$)= 0.5396 kN/m
  - W_{iz} =0.88* cos($\tan^{-1}(\frac{2.8}{7.5})$) =0.824 kN/m
  - W_{wz} =3.744 kN/m
  - W_{dy} =0.576 * sin($\tan^{-1}(\frac{2.8}{7.5})$)=0.2013 kN/m
  - W_{iy} =0.88 * sin($\tan^{-1}(\frac{2.8}{7.5})$)=0.3077 kN/m



- **Factored Load Combination: (The load factors are taken as per IS 800)**
  - Z-direction
    - WL+DL+LL=-1.2* WL+ 1.2* DL + 1.2*LL = -4.776 kN/m
    - DL+LL = 1.5 *DL + 1.5 LL= 2.0454 kN/m
  - Y- direction:
    - DL+LL= 1.5*LL +1.5*DL =0.7635 kN/m
  - Bending Moment And Shear Force Calculation:
    - M_z = Wl2/8 =2.0454*52/8
      = 6.391 kN-m
    - M_y = Wl2/8=0.7635*52/8
      = 2.385 kN-m
    - F_z =Wl/2=2.0454*5/2
      =5.113 kN
    - F_y=Wl/2=0.7635*5/2
      =1.908 kN
- **Channel Section Purlin**
  - We are assuming ISMC 150 Channel Details:
    - Area=20.88*102 mm2

- B_f=75 mm
- T_f=9 mm
- Weight=160.9 kN/m
- T_W==5.4 mm
- C_{YY}=22.2 cm
- H=150 mm
- I_{ZZ}=779.4*104 mm4
- Iyy=102.3*104 mm4
- Elastic Modulus
  - Z_{EZ}= 103.9 *10^3 mm^3
  - Z_{EY}=19.4 *10^3 mm^3
- Plastic Modulus
  - ZPZ= 119822.6 mm3
  - ZPY= 38677.5 mm3

- **Checking of the Section Classification**
  - BF/TF= 75/9 = 8.33 < 9.4 (since ϵ=1) (from IS 800)
  - D/TW=139/5.4= 25.74 < 84 (since ϵ=1) (from IS 800)
  - Hence the section is Plastic
- **Calculation of Shear Capacity of the Section**
  - Z-direction:
    - V_d= (f_y*H*T_w)/($\sqrt{3} * Ymo$)
      = 250*150*5.4/($\sqrt{3}$ *1.1)
    - =106.284 kN
    - 0.6 V_d = 63.7704 kN > 6.391 kN (as per IS 800)
- **Calculation for Moment Capacity of Section:**
  - M_{dz}= (ZPZ* fy)/ Ymo
    = (119.82*10^3*250*10^{-6})/1.1
    = 27.23 kN-m
  - (1.2*ZEZ* fy)/ Ymo
    =(1.2*103.9*103*250*10-6)/1.1
    = 28.336 kN-m
  - Therefore M_{dz} <= 28 kN m
  - Hence M_{dz}= 27.23 kN-m > 6.391 kN-m
- **Y-direction:**
  - M_{dY}= (Z_{PY}* f_y)/ Ymo
    =(38.677*250*10^{-3})/1.1
    = 8.79 kN-m
  - (1.2*Z_{EY}* f_y)/ Ymo
    = 5.29 kN-m (Not satisfying……So the factor 1.2 is replaced by 1.5 &chked)
  - Therefore M_{dY}>2.385 kN-m
- **Overall Member strength (local capacity):**
  - $\frac{Mz}{Mdz} + \frac{My}{Mdy} = \frac{6.391}{27.23} + \frac{2.385}{5.29}$
  - = 0.6855 <= 1
  - Hence it can be concluded safely that member strength is satisfactory
- **Check For Deflection:**
  - δ = (5*wl^3)/(384 EI)= 5*1.03*5000^3/(384*2*10^5*779.4*10^4)
    =5.377 mm

- Allowable deflection= l/150= 5000/150= 33.33 mm
- Therefore the section is safe

- **Load combination (DL + WL)**
  - Z-direction
    - DL + WL = 1.5 * WL + 1.5 * DL = -7.2066 kN/m
  - Y- direction:
    - DL= 1.5*DL =0.30465 kN/m
  - Bending Moment And Shear Force Calculation:
    - $M_Z = Wl^2/8 = 7.2066*5^2/8$
      $= 22.52065$ kN-m
    - MY =Wl2/8=0.30465*52/8
      =0.95203 kN-m
    - Fz =Wl/2=7.2066*5/2
      =18.0165 kN
    - FY=Wl/2=0.30465*5/2
      =0.761625 kN
- **Checking the section for this load combination**
  - M_z = 22. 52 kNm < 27.23 kN **checked**
  - My = 0.952 kNm < 5.29 kNm **checked**
  - Fz = 18.0165 kN < 63.77 kN **checked**
  - Fy = .762 kN < 63.77 kN **checked**
- **Overall Member strength (local capacity):**
  - $\frac{Mz}{Mdz} + \frac{My}{Mdy} = \frac{22.52}{27.23} + \frac{0.952}{5.29}$
    $= 0.98 <= 1$
  - Hence it can be concluded safely that member strength is satisfactory
- **Check For Deflection**
  - $\delta = (5*wl^3)/(384\ EI) = 5*7.206*5000^3/(384*2*10^5*779.4*10^4)$
    =7.524 mm
  - Allowable deflection= l/150= 5000/150= 33.33 mm
  - Therefore the section is safe **checked**

Analysis of Truss

- **Calculating Tributary area of each node:**
  - Length of each panel along sloping roof = 1.6m
  - Spacing of truss= 5m
  - Tributary area of each node of the truss = 5X1.6 = 8m^2
- **Dead Load Analysis:**
  - Given Dead Load=0.36KN/m^2
  - Projected horizontal area= 1.5X5 = 7.5m^2
  - Therefore, Dead Load on an intermediate node (W1)=0.36X7.5 =2.7KN
  - And Dead Load on an end node (W1/2) = 2.7/2 = 1.35 KN
- **Live Load Analysis:**
  - Imposed Load Calculations, From IS875(Part 2)-1987
  - Live Load = 0.55KN/m^2
  - Applying Reduction due to slope, for slope angle, α = 20.47

- Reduced Live Loading = (0.55 – (0.184)(1.3))X(2/3)= 0.206KN/m^2
- Therefore, Live load at intermediate nodes (W2)= 0.206X5X1.5 = 1.545 KN
- Load at end nodes (W2/2) = 1.545/2 = 0.7725 KN

- **Wind Load Analysis:**
  - **Based on Wind Load analysis done on Page :7**

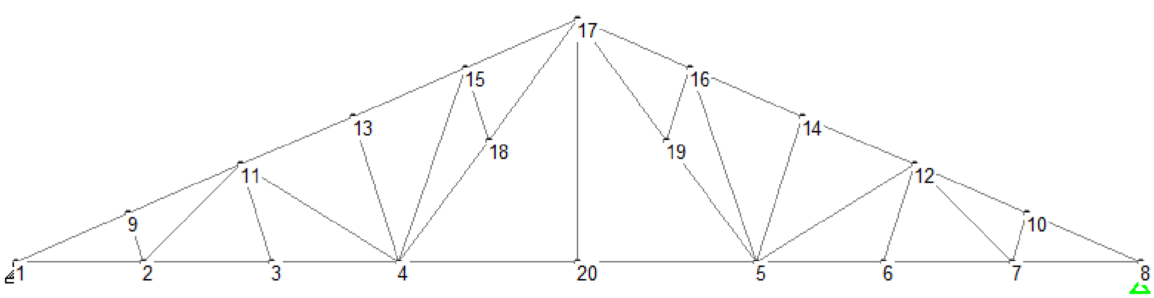
Wind Angle	Windward Side	Leeward Side
0 Degrees	-13.9104 kN	-13.9104 kN
90 Degrees	-18.5472 kN	-17.0016 kN

  - Where the values depict the wind load felt at each intermediate node.
- **Forces in the members:**
  - For the purpose of designing of members we first of all need to know the forces experienced under various loading combinations.
  - For the same the truss has been modelled as a pin jointed plane truss at one end and roller support at the other end.
- **Thus, Characteristic Load Combinations include:**
  - 1.5 x (DL+LL)
  - 1.5 x (DL+WL at 0degrees)
  - 1.5 x (DL+WL at 90degrees)

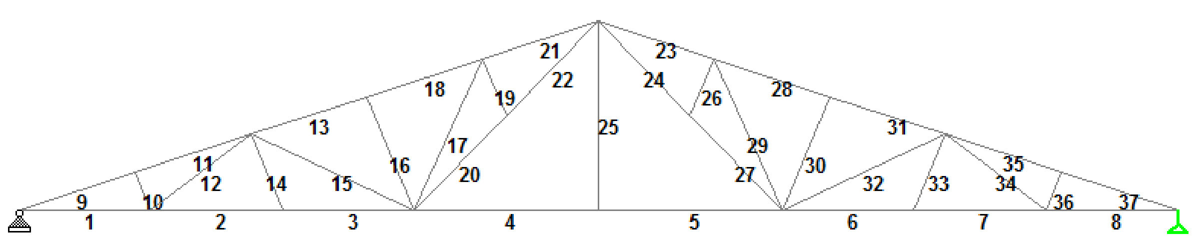
Note: Where 1.5 is the safety factor. The case of DL+LL+WL will not be critical since Wind load acts opposite to both Live Load and Dead Load.

Beam and Nodes numbering in truss

- **Nodes Numbering**



- **Beams Numbering**



Forces Generated in Truss Members

Member Types	Mem No.	DL + LL kN	DL+WL at 0 kN	DL+WL at 90 kN
Bottom Chord Members	**1**	**-76.7**	**183.186**	**256.294**
	2	-68.226	158.897	222.104
	3	-68.226	158.897	222.104
	4	-42.617	85.546	118.871
	5	-42.617	85.546	118.871
	6	-68.226	158.895	212.143
	7	-68.226	158.895	212.143
	8	-76.754	183.319	243.2
Top Chord Members	**9**	**81.877**	**-199.45**	**-274.46**
	11	79.651	-200.868	-275.879
	12	-8.474	24.292	34.193
	13	61.494	-156.639	-213.05
	18	59.266	-158.054	-214.464
	21	62.367	-174.729	-237.355
	23	62.368	-174.731	-231.228
	28	59.267	-158.055	-210.407
	31	61.495	-156.64	-208.994
	35	79.7	-200.988	-265.772
	37	81.928	-199.574	-264.358
Web Member	10	5.928	-16.992	-23.917
	14	0	0	0
	15	11.998	-34.363	-48.362
	16	5.964	-17.082	-24.041
	17	8	-22.913	-32.247
	19	0.003	-0.008	-0.011
	20	**-24.435**	**69.985**	**98.495**
	22	-24.436	69.989	98.501
	24	-24.437	69.99	89.001
	25	0	0	0
	26	0.002	-0.006	-0.007
	27	-24.436	69.987	88.997
	29	8	-22.913	-29.138

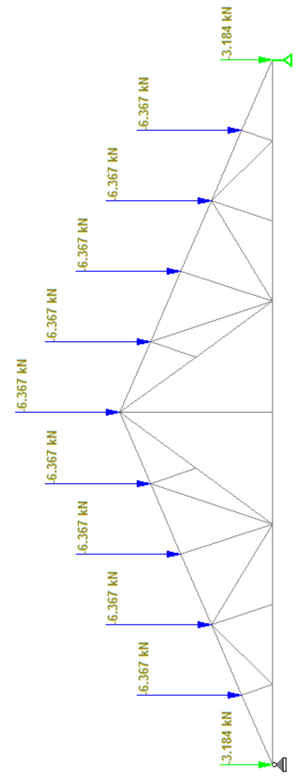


Figure 2 Load Combination: Dead Load + Live Load

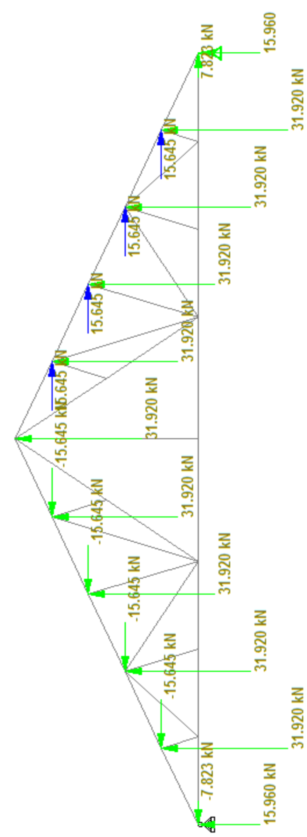


Figure 1 Load Combination: Dead Load + Wind Load

	30	5.965	-17.085	-21.726
	32	11.997	-34.36	-43.692
	33	0	0	0
	34	-8.527	24.421	31.054
	36	5.965	-17.084	-21.724

End Reactions kN					
Loading Condition	**Node 1**			**Node 8**	
	F_x	**F_y**		**F_x**	**F_y**
DL+LL	0	31.838		0	31.838
DL+WL 0	0	-77.555		0	-77.555
DL+WL 90	4.058	-107.066		0	-102.387

Deflection at nodes of Truss

Nodes	**Dead Load + Live Load**				**Dead Load + Wind Load (0 degrees)**				**Dead Load + Wind Load (90 degrees)**	
	X	**Y**	**Resultant**		**X**	**Y**	**Resultant**		**X**	**Y**	**Resultant**
1	0	0	0		0	0	0		0	0	0
2	5.706	-81.372	81.57181		-13.627	200.457	200.9196		-19.066	275.639	276.2976
3	10.781	-111.487	112.0071		-25.448	274.794	275.9698		-35.588	376.724	378.4012
4	15.856	-124.379	125.3856		-37.268	308.092	310.3379		-52.111	420.397	423.6144
5	24.677	-124.399	126.823		-54.973	308.142	313.0072		-76.713	414.181	421.2253
6	29.752	-111.522	115.4224		-66.794	274.878	282.8769		-92.494	368.105	379.5477
7	34.828	-81.426	88.56175		-78.614	200.59	215.4449		-108.276	267.571	288.6485
8	40.537	0	40.537		-92.251	0	92.251		-126.368	0	126.368
9	22.085	-75.424	78.59088		-54.523	185.666	193.5061		-75.003	255.427	266.2112
10	18.455	-75.477	77.70048		-37.735	185.797	189.5902		-53.548	247.734	253.4552
11	27.312	-105.313	108.7969		-67.038	259.261	267.7879		-91.891	355.696	367.3739
12	13.231	-105.351	106.1786		-25.226	259.354	260.5779		-36.227	347.09	348.9754
13	28.305	-120.226	123.513		-69.634	297.427	305.4697		-94.969	406.394	417.343
14	12.242	-120.252	120.8735		-22.641	297.49	298.3503		-32.251	399.385	400.685
15	27.471	-129.801	132.6761		-67.167	322.312	329.2361		-91.294	439.283	448.6693
16	13.081	-129.815	130.4724		-25.119	322.345	323.3222		-35.106	433.661	435.0796
17	20.278	-123.006	124.6662		-46.15	300.951	304.4689		-61.922	408.067	412.7384
18	25.936	-130.374	132.9288		-64.036	323.481	329.7583		-87.975	440.522	449.2207
19	14.608	-130.385	131.2008		-28.233	323.507	324.7366		-40.434	435.65	437.5224
20	20.267	-123.006	124.6645		-46.121	300.951	304.4645		-64.412	408.067	413.1193

- **Maximum Deflection at node = 448 mm**

Design of Column

- **Given details derived from above**
  - Overall height of the column = 7 m
  - Height of crane rail = 6.4 m
  - Crane rail eccentricity = 0.5 m
  - Cladding eccentricity = 0.25 m
  - Loading (unfactored)
    - Roof Truss Reactions W_A
      - Dead Load = 40 kN
      - Imposed Load = 65 kN
      - Wind Load (suction) = 70 kN
    - Gantry girder reaction (vertical) W_c
      - Dead Load (gantry self weight) = 20 kN
      - Crane Load including impact (near side) = 180 kN
      - Crane Load (far side) = 120 kN
    - Gantry Girder reaction (horizontal) W_{hc}
      - Crane surge load = 5 kN
    - Wind Load on side of building W_B
      - Wind Load =
    - Self-weight of column WD
      - Dead Load = 15kN
    - Cladding W_{DC}
      - Dead Load = 12.5 kN

- **Unfactored Load Matrix**

Loads	Dead Load Wd	Imposed Load Wl	Crane Load Vertical , Wcv	Crane Load horizontal , Wch	Wind Load Ww
WA	86.25	40	0	0	-20
WB	0	0	0	0	81.55
WC	20	0	143	0	0
WHC	0	0	0	6	0
WD	15	0	0	0	0
WDC	12.5	0	0	0	18.75
WE	2.48	0	11.1	0	15.29

- **Load Factor and Combinations**
  - **1.5 Wd + 1.5 Wl**
  - **1.5 Wd + 1.5 Ww**
  - **1.2 Wd + 1.2 Wl + 1.05Wcv + 0.6 Ww + 1.05 Wch …… Case (i)**

- $1.5\ Wd + 1.5\ Wl + 1.05Wcv + 1.05\ Wch$ ……. **Case (ii)**
- $1.2\ Wd + 1.2\ Wl + 0.53Wcv + 1.2\ Ww + 0.53\ Wch$ ……. **Case (iii)**

	Case (i)	Case (ii)	Case (iii)
Wd	1.2	Do Not Govern	1.2
Wl	1.2		1.2
Wcv	1.05		0.53
Wch	1.05		0.53
Ww	0.6		1.2

- **Factored Load Matrix**

	Case (i)	Case (ii)	Case (iii)
WA	135.3	Do Not Govern	119.1
WB	48.93		97.86
WC	174.15		99.79
WHC	6.3		3.18
WD	18		18
WDC	15		18
WC	23.753		27.144

- **Final Load values**
  - W_A = 119.1 kN
  - W_B = 97.86 kN
  - W_c = 99.79 kN
  - W_{HC} = 3.18 kN
  - W_D -= 18 kN
  - WDC = 15 kN
  - WE = 27.144 kN
- **Maximum Value of Bending Moment**
  - 99.79 x 0.4 + 3.18 x 6.4 + 97.86 x3.5 – 27.14 x 7 – 15 x 0.65 – 119.1 x0.4
  - = 155.408 kN
- **Axial Force**
  - 119.1 + 99.79 + 18 + 15
  - = 251.89 kN
- **Let us first design assuming the column as a single integral compound column , which will result in more savings**
  - **Assuming two ISHB 150 section (w= 300.2 N)**
  - **Properties of the crossection**
    - Area (A) = 3898 mm²
    - H = 150 mm
    - B = 150 mm
    - T_f = 9 mm
    - R_z = 62.9 mm
    - T_w = 8.4 mm
    - R = 8 mm

- $R_y = 34.4$ mm
- $Z_z = 205.3\ cm^3$
- $Z_y = 60.2\ cm^3$
- $I_y = 460.3\ cm^4$
- $I_z = 1540\ cm^4$

- **Combined Moment of Inertia**
  - $\mathbf{I_a = 2 \times (460.3 \times 10^4 + 38.48 \times 400^2)}$
    - $\mathbf{= 1240.56 \times 10^6\ mm^4}$
  - $\mathbf{R_a = \sqrt{\frac{1240.56 \times 10^6}{2 \times 3898}} = 398.909\ mm}$
- **Section Classification**
  - $B_F/T_F = 150/9 = 8.33 < 9.4$ (since ϵ=1) (from IS 800)
  - $D/T_W = (150 - 2\ (9+8))/8.4 = 13.8 < 40$ (since ϵ=1) (from IS 800)
  - Hence the section is Plastic
- **Checking For Local Capacity**
  - $N_d = Ag\ f_y / \gamma_{m0}$

    $= 2 \times 3898 \times 250 / (1.1 * 1000)$

    $= 1771.81$ kN
  - $n = N$ (Axial Force)/ N_d
  - $= 251.89 / 1771.81$
  - $= 0.142$
  - $M_{da} = \beta_b Z_p f_y / \gamma_{m0}$
    - $= 1 \times (2 \times 3898 \times 400) \times 250 / (1.1 \times 10^6)$
    - $= 708.727$ kN-m
  - Reduced Plastic Moment
    - Mnda = 1.1 Mda (1-n)
    - = 668.896 kN-m < 708.727 kN-m
  - Interaction equation for cross-section strength is
    - $(M_z / M_{nda})^{\alpha} \leq 1.0$ ----------- $\alpha = 2$ (Table 9.1 of code)
    - 0.0539 < 1 => Hence , **Safe**
- **Compressive Strength**
  - For compressive strength the overall slenderness ratio is based on an effective length (from fig 9.38 of the book)
    - KLa = 1.5 x 7 = 10.5 m
    - KLb = 0.85 x 8 = 5.98m
    - Ra = 398.909 mm
    - KLa / Ra = 10500/396.909 = 26.32
    - KLb/Rb = 5950 / 62.9 = 94.594
  - Local Slenderness
    - L/Ry = 1000/34.4 = 29.069 < 50 (clause 7.6.6) **checked**
    - Hence , the slenderness is safe
  - Compound Slenderness
    - L/r = 1.05 x 94.594 = 99.323
  - Check : Local Slenderness < 0.7 x Overall Slenderness ---------- (clause 7.6.6)
    - 0.7 x 99.323 > 26.32

      69.526 > 26.32 **checked**
  - From table 10 and Table 9c of the code , for L/r = 99.323 & fy = 250 N/mm

- F_{cd} = 107.947 N/mm^2

- Check for Overall Buckling
  - Bending about the A-A axis may be assumed to produce axial forces in the two I –sections of the compound column
    - Axial Force = moment / centroidal distance between the two I-sections
      155.408 / 0.8 = 194.26 kN
    - Maximum compression in one I- section
      - 194.26 + 251.89/2
        = 320.205 kN
    - Compressive resistance of the section
      - Ag fy /γ_{m0}
        =107.947 x 3898 /(1.1 x 1000)
        =382.52 kN > 320.205 kN ………. **Section is Safe**
- Design of Lacings
  - Maximum Force in the lowest diagonal
    - (97.86 + 3.18 – 27.14)/cos(51.3)
    - = 118.194 kN Compression
  - The lacing should also carry a transverse shear force equal to 2.5 % of axial Load Column
    - 0.025 x 320.205
      = 8.005 kN
  - Force in Lacing = 8.005 / cos (51.3)
    = 12.803 kN
  - Total force Resisted By Lacing
    - = 118.194 + 12.803
      = 130.997 kN i.e. 65 kN on each side of the column
  - Using ISA 8080m
    - T = 10mm
    - Area = 15.05 cm^2
    - Rv = 15.5 mm
  - Effective Length of the lacing
    - $\sqrt{1^2 + 0.8^2}$= 1.28 m
    - KL/r = 1280 / 12.8 = 100 < 145 -------- **checked**
  - Compressive Stress from table 9c of the code
    - Fcd = 107 N/mm^2
    - P_d= $A_e F_{cd}$ = 107 x 1505 mm^2
      = 161.035 kN > 130.99 kN ------------ **checked and safe**
  - Hence Provide tie plates at top & bottom of the column
    - Effective depth > 2 b_f
      - = 2 x 150 = 300 mm 0r 800 mm (distance between C.G. of section)
    - Also Provide 16mm bolts

- Lacing Design details
  - Overall depth = 800 + 2x25 = 850 mm
  - Length of plate = 800 mm
  - Thickness of plate = 1/50 (800) = 16mm
  - Plates Req. is (800 x 850 x 16)
- Column Details
  - 2 ISHB 150 used

Design of Truss Members

- Design of Principal Rafter:
  - Length of principal rafter is 8m and is divided into 5 parts members-9,11,13,18,21.
  - Maximum Factored load experienced by these members is:
    - Factored Tensile Load = 83.916KN
    - Factored Compressive Load= 275.879KN
  - Assuming a compressive strength fcd=120N/mm2 and that the gross area is 20% higher than the net are of the section.
    - Therefore, Ag required= 1.2* (275.879*1000)/120 = 2758.79mm2
  - Using, 2X ISA 75X75X5 @ 11.4 Kg/m,
    - Ag provided= 2X1454 =2908 mm2 such that back to back angles are connected to gusset plate of 8mm thickness.
    - Minimum moment of inertia is @ z-z axis= 2 Izz
    - And $r_{zz}=r_{min}$.= 23.1mm
  - Checking for slenderness, KL/rzz = (0.85X1600)/23.1 = 58.87 < 180 SAFE
  - From buckling class 'c', compressive strength we get as fcd =168 KN/ mm2 (at KL/r = 60)
  - Therefore, Capacity in compression = 168x2908 = 488.544KN
  - Hence our design is complete in compression now checking for capacity in tension.
  - **Checking for Tension:**
    - Yielding on gross area of section = Tdg = (Agxfy)/γm0 = (2908x250)/1.1 =660.9KN
    - Rupture on net section, this depends on length of connection. Estimating length of weld connection.
    - Using a welded connection for size of weld = 4mm = s
    - Weld strength per mm = 0.707 x s x (fmw/$\sqrt{3}$) x(1/γm1)

      = 0.707x 4 x(410/1.732)x(1/1.25) = 535.2 N/ mm
    - Therefore, Required weld length for each ISA = (81.928x1000)/(2x535.2) =76.54mm
    - Dividing the length on the two sides of the connected leg as L1 and L2.
    - Based on ISA Properties, L1/L2 = 54.8/20.2 based on Cxx =2.02cm
    - Now, L1= 58.45mm which is nearly 60mm
    - And L2 = 21.54mm which is nearly 22 mm
    - Now, for rupture strength for the designed welding pattern:
    - Ago=Anc= 2x (75-2.5)x5 = 725 mm2
    - Also, shear lag width= bs=75mm
    - Width of outstanding leg = w= 75mm , thickness= t=5mm
    - and Lc = Length of end connection =75-2.5 =72.5mm
    - calculating β = 1.4 – 0.0076 (w/t)(fy/fu)(bs/Lc)

      = 1.4 – 0.0076(75/5)(250/410)(75/72.5)

      =1.328 < (fuγm0)/(fyγm1) =1.44 and 1.328> 0.7 OK
    - Calculating Tdn = (0.9)(725)(410/1.25) + (1.328)(725)(250/1.1)

      = 432.83KN
  - **Block Shear Strength :**
    - As at the end, connected leg is only welded along the length of the member, block shear failure will be only along weld line by way of shear.
    - Total weld length for each ISA = 82mm each.
    - Calculating Tdb = (Avg x fy)/($\sqrt{3}$ x γm0) =(2x82x5x250)/(1.1x1.732) =107.6KN
    - Similarly Tdb2 =139.75
    - Therefore, Td = 107.6KN is governing strength in Tension among (a),(b) and (c) and T=81.928<107.6KN hence SAFE.
- **Design of Horizontal Tie member:**

- For the horizontal tie member(Bottom Chord members- 1,2,3,4,5,6,7,8)
  - Factored Compressive Load = 76.754KN
  - Factored Tensile Load = 256.294KN
- Using the same section as in the principal rafter 2X ISA 75x75x5 @ 11.4Kg/m
  - The design compressive strength = 488.54 > 76.754 KN SAFE
  - While designing the weld connection, using a weld size of 4mm=S
    - And throat thickness = t_t =KS = 0.7x4 = 2.8mm
    - Therefore, New weld length required for each ISA =(256.294x1000)/(2x535.2) = 239.437mm
    - Also according to the long joint criteria in IS800 -10.5.7.3 since
    - L=240mm < 150x t_t = 360mm, long joint correction β_{lj} =1.
  - Again length distribution along the member is L1 = 174.948mm nearly = 175mm
  - And L2 = 64.488mm nearly = 65mm
- Same as before the section is safe in Yielding and Rupture and the deciding factor restes with Block Shear Strength.
  - Total weld length for each ISA = 240mm
  - And Block Shear strength T_{db} = 314.92KN and 409.053KN
  - Hence T_d= 314.92KN > 256.294KN implies the section is safe in tension.

- **Design for Other Web members:**
  - Consider members with nominal or Zero loads, this includes members like 10,14,16,19,25,26,30,33,36.
    - Factored Compressive Load= 24.041KN and Factored Tensile Load = 5.965 KN
    - Design of these members with nominal forces are governed by minimum slenderness ratio, as design forces are very small.
  - Therefore, $r_{min.}$ for limit of slenderness ratio 180 = (0.85x1790)/180 = 8.45mm
  - (Choosing max length 1.79m)
    - As, ISA 75x75x5 has r_{zz} =$r_{min.}$=23.1mm, providing the same single section would suffice as the nominal design.

    - For other members with considerable forces, including members 12,15,17,20,22,24,27,29,32,34
    - Factored Tensile Load = 98.501KN and Factored Compressive Load =48.362KN
    - Using a single ISA 75x75x5, Slenderness KL/r = (0.85x2400)/23.1 =88.31<180
    - Hence for buckling class 'c' and f_{cd} =121N/ mm^2 (at KL/r =90)
    - Therefore, Total Compressive Strength = 175.9KN > 48.362 SAFE
  - Also, based on earlier analysis for a single section with weld length of 82mm , Tensile Strength = 107KN > 98.501 SAFE