Structural Design Calculation of an Industrial Factory Shed at Kolkata

A Project Report Submitted in Partial Fulfilment

of the Requirements for the Degree of

BACHELOR OF TECHNOLOGY

IN

CIVIL ENGINEERING

By

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ABSTRACT

A shed is typically a simple single storied structure that is used for storage, hobbies, or as a workshop. Sheds vary considerably in the complexity of their construction and their sizes. Sheds used in industries are very large structures. Industrial Shed constructions are metal sheathing over a metal frame, plastic sheathing and frame. Steel offers numerous possibilities to achieve both pleasant and flexible functional use. For buildings of large enclosure, the economy of the structure plays an important role. For longer spans, the design is optimized in order to minimize the use of materials, cost and installations effort. Increasingly, buildings are designed to reduce energy costs and to achieve a high degree of sustainability. Large open spaces can be created that are efficient, easy to maintain, and are adaptable as demand changes. Steel is chosen on economic grounds as well as for other aspects such as fire, architectural quality and sustainability.

The present scope of this project is to select a representative truss and analyse the truss for different load conditions possible at the project construction location. This project design was assumed to be Industrial structure at Kolkata. The aim of this project is to design an industrial factory shed economically using manual design techniques. The Project Summary Report emphasizes the structural analysis and design findings of the industrial factory shed.

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APPROVAL SHEET

This project report entitled **"Design of Industrial Factory Shed"** by ------------------ Ms. ATASI ROY MALAKAR ------------------is approved for the partial fulfillment of the requirement for the degree of **Bachelor of Technology in Civil Engineering**.

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CERTIFICATE

It is certified that the work contained in the project titled **" Design of Industrial Factory Shed"** by Ms. ATASI ROY MALAKAR has been carried out under our supervision and that this work has not been submitted elsewhere for a degree.

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ACKNOWLEDGEMENT

I am using this opportunity to express my gratitude to everyone who supported me throughout the course of this project. I am thankful for their aspiring guidance, invaluably constructive criticism and friendy advice during the project work. I am sincerely grateful to them for sharing their practical and illuminating views on a number of issues related to the project.

I express my warm thanks to Ms. T Banerjee, Mr. A K Ghosh and Mr. K K Ghosh for their support and guidance. I would also like to thank Mr. S Khan who provided me with the aiding materials for my project.

Thank you,

Atasi Roy Malakar.

CONTENTS

PROBLEM AND DATA

CE- 783: Project Part- 1

Q1. Design an industrial factory shed at Kolkata which includes:

a) Truss Bracing
b) Gantry Girder
c) Column with connection
d) Foundation (Base Plate only)

Q2. Draw a general arrangement drawing showing:

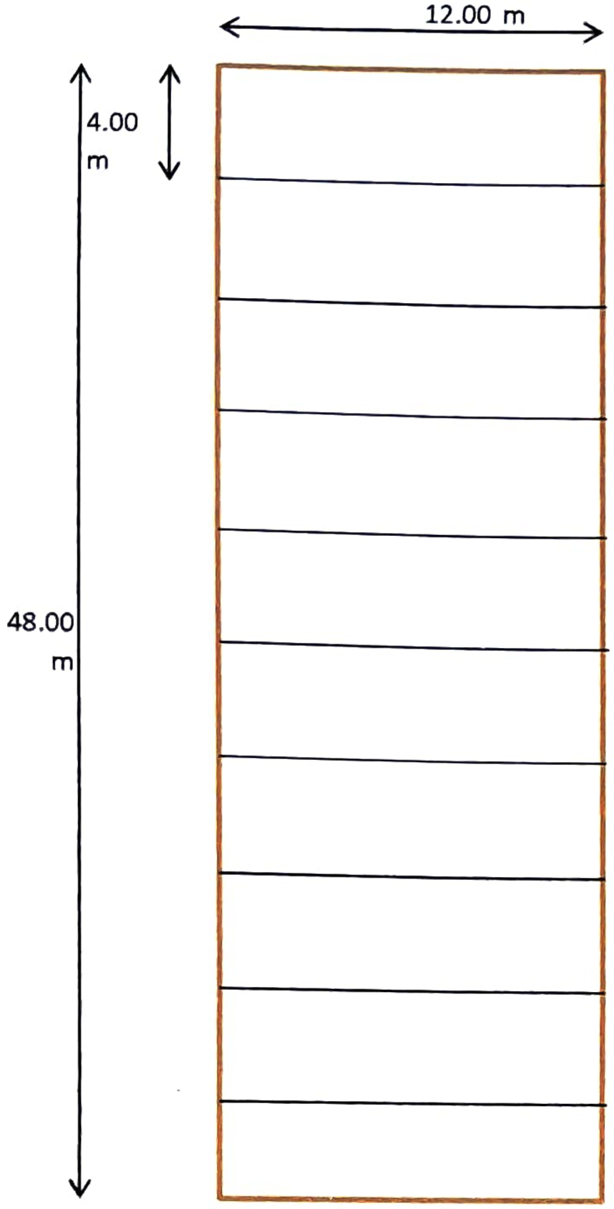
a) Grid Plan
b) Plan at Eaves level
c) Plan at rafter level
d) Plan at Bracing level
e) Roof truss arrangement showing connection details

DATA SHEET

Span of Truss (a)=	12.00 m
Spacing of Columns (b)=	4.00 m
Ratio of height & span of truss, ie. Pitch=	0.16
Length of Factory Shed=	48.00 m
Spacing of Gantry Columns=	16.00 m
Crane Capacity=	500.00 kN
Corrugated Asbestos cement used @=	171.00 N/m²
Self Weight of Purlins=	90.00 N/m
Self Weight of Bracing=	13.00 N/m²
Self Weight of Crane=	350.00 kN
Self Weight of Crab=	140.00 kN
Wheel Base Distance of Crane=	4.75 m
Minimum Approach Distance=	1.20 m
Height of Rail=	150.00 mm
Nature of Crane, EOT or Hand operated=	EOT
Rail Level=	7.60 m
Truss Level=	10.75 m

Grade of Steel used=	Fe-	410
fu=		410.00 Mpa
fy=		250.00 MPa
Grade of bolt used=		4.6
Partial safety factors=	γmo=	1.10
	γmw=	1.50
Load Factor=	$\gamma m1$=	1.25
	ε=	1.00

GRID PLAN OF FACTORY SHED:



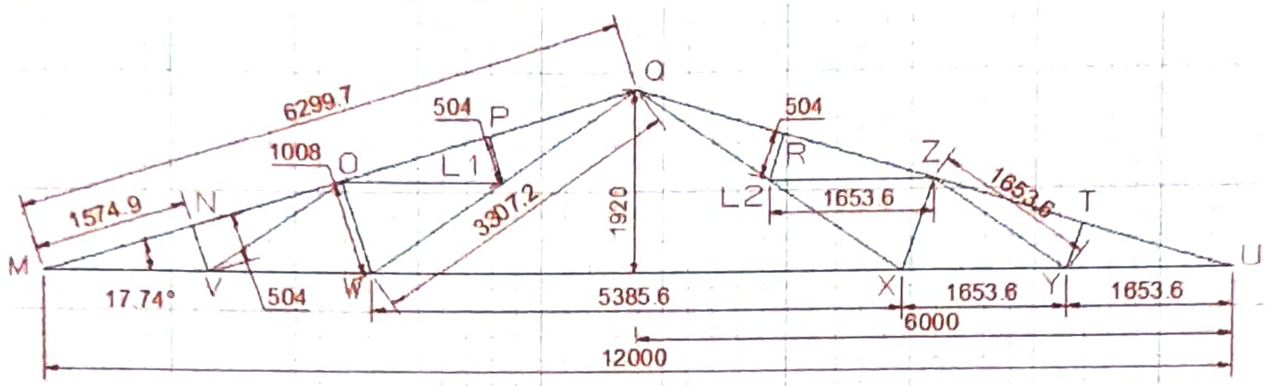
(NOT TO SCALE)

TRUSS LOAD ANALYSIS

TRUSS CONFIGURATION:

Span of truss= 12.00 m and pitch= 0.16
hence, height of truss=pitch*span= 1.92 m
Let θ be the inclination of the roof with the horizontal.
So, tan θ= height/half base= 0.32 ie., θ= 0.31 radians
or θ= 17.74467 °= 17 ° (degs) 45 ' (mins)

So, length of Rafter= $\sqrt{(span/2)^2+(height)^2}$= 6.299714 m
Length of each panel= 1.574929 m= 1.57 m



SPACE DIAGRAM OF TRUSS
(ALL DIMENSIONS IN MM)

LOADS ON PANEL POINTS:

i) Dead Load:

Given, self-weight of bracing= 13.00 N/m²
Dead Weight of AC sheets= 171.00 N/m²
Self weight of Roof truss by empirical formula= (Span/3 + 5)*10= 90 N/m²
Self weight of Purlin= 90.00 N/m= 720.00 N
Panel length= 1.57 m
Panel length in plan= 1.495306 m= 1.5 m
So, Load on each intermediate panel due to dead load= 2184 N= **2.184** kN(approx)
Load on end panel joints(0.5 X intermediate panel load)= **1.092** kN

ii)Live Load:

Here, θ= 17.74467 °
Let the roof be inaccessible, then live load (750 N/m²) is reduced by 20 N/m²
for each one degree increase above 10° slope.
So live load= 595.1066 N/m²

The load on each intermediate panel= 3570.639 N= **3.571** kN (approx)
And load on each half node/end panel point= **1.79** kN

iii) Wind Load

Location: Kolkata, WB, India

(Reference- IS 875-part III Appendix A)

According to 5.3.2.1 of the said code, let us assume the structure will be constructed in a terrain with numerous closely spaced obstructions having size of building-structures up to 10 m. With or a few isolated tall structures.

Total height of the structure= 10.75 m(upto eaves level)

Plan dimension of structure=length= 48.00 m and its width= 12.00 m

Therefore, maximum dimension of the structure is in the range= 20-50 m

So the structure belongs to the class B (As per cl. 5.3.2.2 of IS 875)

From table 2 of IS-875 part III 1987, for height of structure= 10.75 m & class B

interpolated value of k_2= 0.889

That is, k_1= 1 (Risk coefficient for 50 Years)

k_2= 0.889

k_3= 1 (Topography factor for plain land)

V_b= 50 m/s (For Zone 2)

So, design wind speed, $V_z = k_1k_2k_3V_b$ = 44.45 m/s (As per cl. 5.3.3.1)

P_d= Design wind pressure= $0.6V_z^2$= 1185.48 N/m²= **1.185** kN/m²

Wind force as per cl. 6.2.1, $F=(C_{pe}-C_{pi}).P_d.A$

<u>*<> Determination of External and Internal Pressure coefficient of roof:*</u>

From table 5 of reference code, height,h= 10.75 m and width, w= 12.00 m

So, h/w= 0.895833 ,so h/w lies in the range of (3/2>h/w>1/2) and θ= 17.744672

Let us assume the building has normal permeability.

In the case of buildings with openings consisting of not more than about 5% of wall area but without any large openings, it is necessary to consider the possibility of the internal pressure being both positive and negative.

Two design conditions of internal pressure coefficient= +0.2 and -0.2 will be considered. Where, positive sign indicates pressure and negative sign indicates suction.

Slope(°)	Wind angle 0°		Wind angle 90°	
	EF	GH	EF	GH
10	-1.1	-0.6	-0.8	-0.6
17.74	**-0.790213**	**-0.522553**	**-0.8**	**-0.6**
20	-0.7	-0.5	-0.8	-0.6

For EF, interpolating value corresponding to slope of 17.74 ° and wind angle 0°

= (-0.7 - (- 1.1)) / (20-10) x (17.74 -10.00)+ (-1.1)

= -0.790213

For GH, interpolating value corresponding to slope of 17.74 ° and wind angle 0°

= (-0.5 - (- 0.6)) / (20 - 10) x (17.74 -10.00)+ (-0.6)

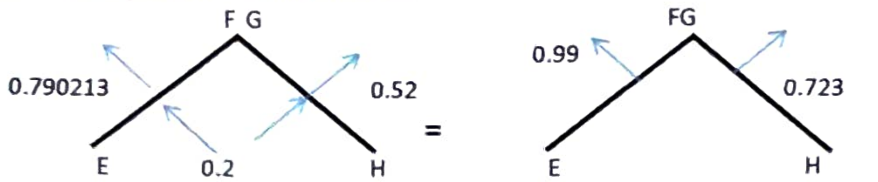
= -0.522553

Assuming that opening of the factory shed is < 5%, as per cl. 6.2.3.1, C_{pi}= ± 0.2

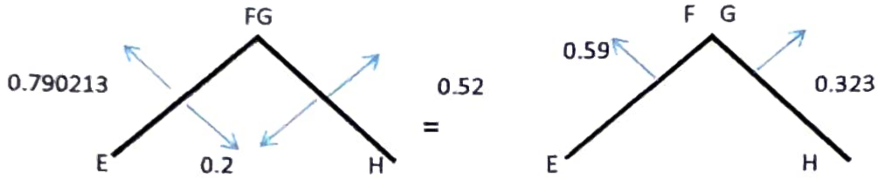
4 cases arise:

a) ***Algebraic sum of C_{pe} and C_{pi} when wind angle=0°***

Case i) :

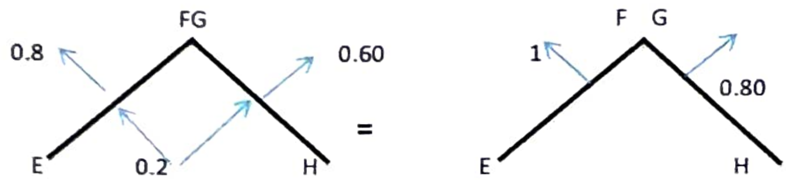


Case ii) :

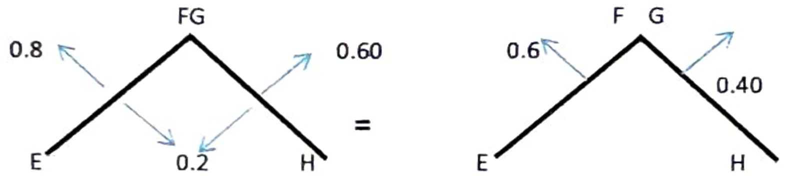


b) ***Algebraic sum of C_{pe} and C_{pi} when wind angle=90°***

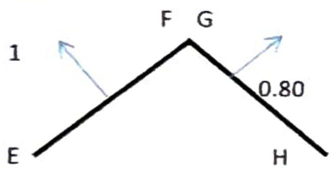
Case i) :



Case ii) :



From all the above mentioned conditions, the most critical condition is chosen, which is:



<> Determination of Total Nodal forces:

Here, C_{pe}-C_{pi}= 1 (Considering symmetrical loading, we take the max value for both sides)

P_d= 1185.482 N/m²

Area, A= 1.58 X 4.00

= 6.3 m²

Total wind force on windward and leeward side, as per cl. 6.2.1, F=(C_{pe}-C_{pi}).P_d.A

that is, F= 7468.533 N= **7500** N(suction in nature)

So, load on each full node on each side= **7.5** kN(suction in nature)

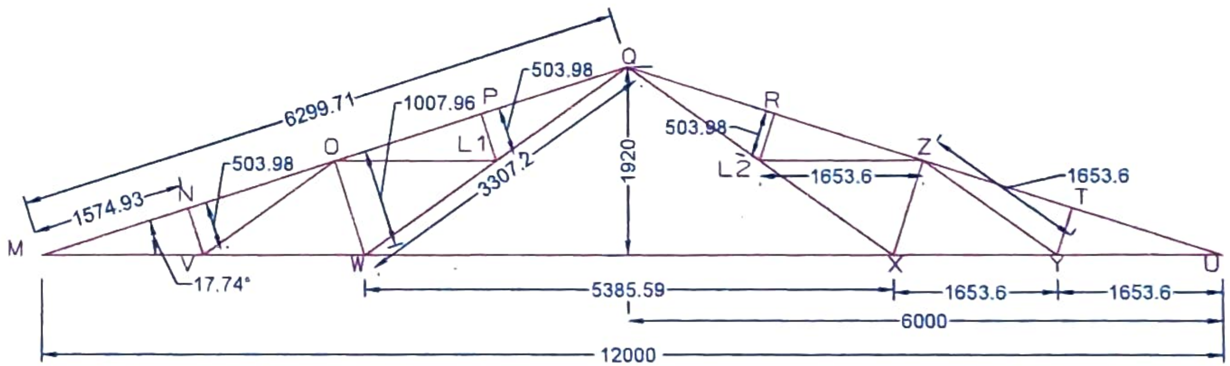
And, load on each half node on each side= **3.75** kN(suction in nature)

Truss Load

iii) Member forces: (Reference: Enclosed AutoCAD drawing)

Compression: (+); Tension: (-)

Member Name	Stress Member	Calculation of the truss member force for DL, LL & WL										Load Combination					
		Type of Member	Length (m)	DL & LL			DL (kN)	LL (kN)	Wind Load		WL (kN)	1.5(DL+LL) (kN)	1.5(DL+wL) (kN)	1.2(DL+LL+WL) (kN)	Maximum Tension	Maximum Compressn	Maximum Length(m)
				Unit Force	Pd	PL			Unit Force	WL							
MN & UT	b-1, i-13	Rafter	1.57	11.484	2.184	3.571	25.08106	41.009364	10.937	7.5	-82.0275	99.13563	-85.41967	-19.1245	88.41393 kN	99.13563 kN	1.57
NO & TZ	c-2, h-12		1.57	11.179	2.184	3.571	24.41494	39.920209	10.937	7.5	-82.0275	96.50272	-86.41885	-21.23083			
OP & ZR	d-5, g-9		1.57	10.874	2.184	3.571	23.74882	38.831054	10.937	7.5	-82.0275	93.86981	-87.41803	-23.33716			
PQ & RQ	e-6, f-8		1.57	10.57	2.184	3.571	23.08488	37.74547	10.937	7.5	-82.0275	91.24553	-88.41393	-25.43658			
MV	1-k	Bottom Tie	1.654	10.938	2.184	3.571	-23.88859	-39.059598	10.265	7.5	76.9875	-94.42229	79.64836	16.84717	94.42229 kN	79.64836 kN	5.386
VW	3-k		1.654	9.375	2.184	3.571	-20.475	-33.478125	8.624	7.5	64.68	-80.92969	66.3075	12.87225			
WX	7-k		5.386	6.25	2.184	3.571	-13.65	-22.31875	5.343	7.5	40.0725	-53.95313	39.63375	4.9245			
XY	11-k		1.654	9.375	2.184	3.571	-20.475	-33.478125	8.624	7.5	64.68	-80.92969	66.3075	12.87225			
YU	13-k		1.654	10.938	2.184	3.571	-23.88859	-39.059598	10.265	7.5	76.9875	-94.42229	79.64836	16.84717			
NV	1-2	Strut	0.504	0.952	2.184	3.571	2.079168	3.399592	1	7.5	-7.5	8.21814	-8.131248	-2.425488	16.25922 kN	16.44491 kN	1.008
OW	3-4		1.008	1.905	2.184	3.571	4.16052	6.802755	2	7.5	-15	16.44491	-16.25922	-4.84407			
PL1	5-6		0.504	0.952	2.184	3.571	2.079168	3.399592	1	7.5	-7.5	8.21814	-8.131248	-2.425488			
RL2	8-9		0.504	0.952	2.184	3.571	2.079168	3.399592	1	7.5	-7.5	8.21814	-8.131248	-2.425488			
ZX	10-11		1.008	1.905	2.184	3.571	4.16052	6.802755	2	7.5	-15	16.44491	-16.25922	-4.84407			
TY	12-13		0.504	0.952	2.184	3.571	2.079168	3.399592	1	7.5	-7.5	8.21814	-8.131248	-2.425488			
OV	2-3	Sling	1.654	1.562	2.184	3.571	-3.411408	-5.577902	1.641	7.5	12.3075	-13.48397	13.34414	3.981828	13.48397 kN	13.34414 kN	1.654
OL1	4-5		1.654	1.562	2.184	3.571	-3.411408	-5.577902	1.641	7.5	12.3075	-13.48397	13.34414	3.981828			
ZL2	9-10		1.654	1.562	2.184	3.571	-3.411408	-5.577902	1.641	7.5	12.3075	-13.48397	13.34414	3.981828			
ZY	11-12		1.654	1.562	2.184	3.571	-3.411408	-5.577902	1.641	7.5	12.3075	-13.48397	13.34414	3.981828			
QL1	6-7	Sling	1.654	4.688	2.184	3.571	-10.23859	-16.740848	4.922	7.5	36.915	-40.46916	40.01461	11.92267	40.46916 kN	40.01461 kN	1.654
WL1	4-7		1.654	3.125	2.184	3.571	-6.825	-11.159375	3.281	7.5	24.6075	-26.97656	26.67375	7.94775			
QL2	7-8		1.654	4.688	2.184	3.571	-10.23859	-16.740848	4.922	7.5	36.915	-40.46916	40.01461	11.92267			
XL2	7-10		1.654	3.125	2.184	3.571	-6.825	-11.159375	3.281	7.5	24.6075	-26.97656	26.67375	7.94775			



SPACE DIAGRAM OF TRUSS

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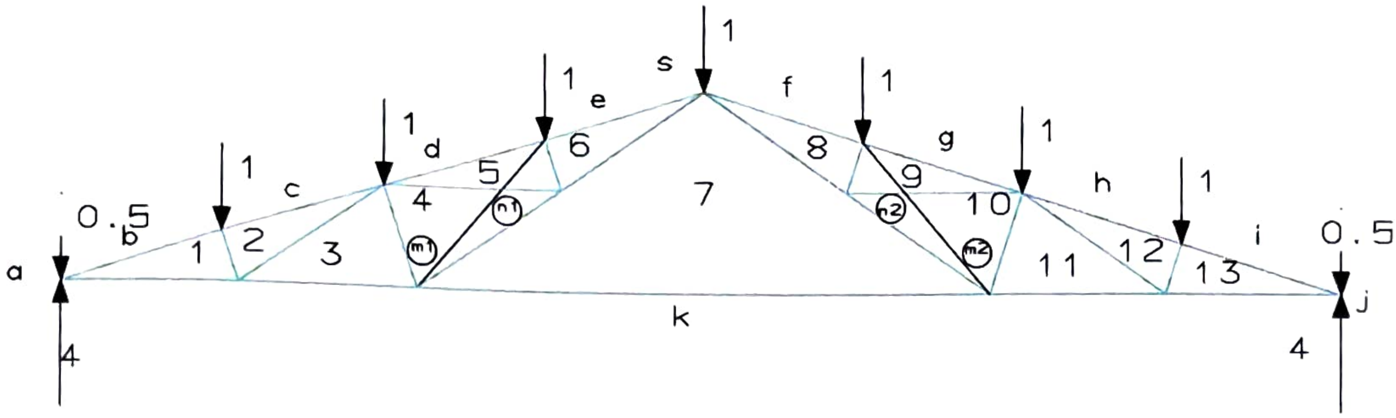
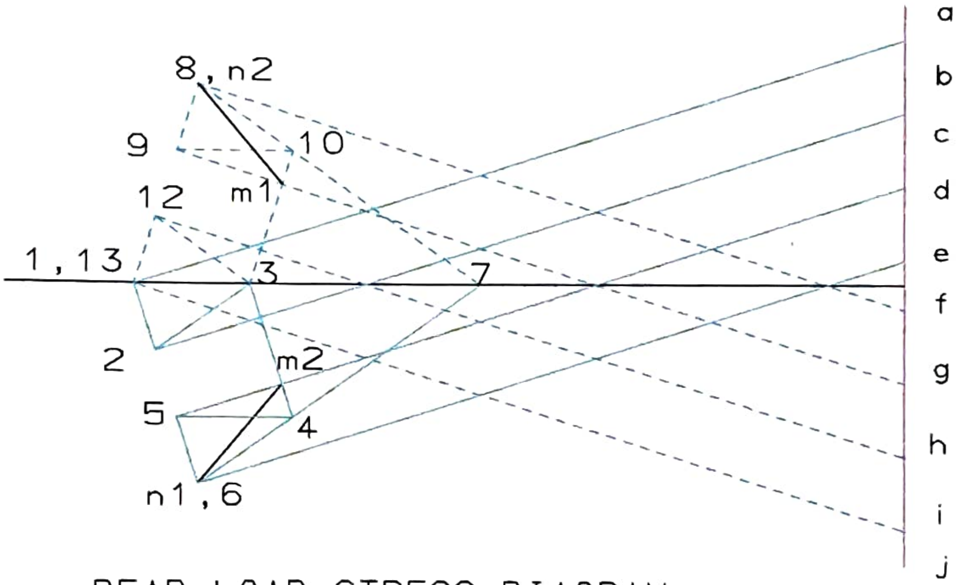
CIVIL ENGINEERING DEPARTMENT
DRAWING: SPACE DIAGRAM OF TRUSS
ATASI ROY MALAKAR
UNIV. ROLL NO.: 16501311014
GROUP-1 (A) SEVENTH SEMESTER



DEAD LOADS AT PANEL POINTS



DEAD LOAD STRESS DIAGRAM

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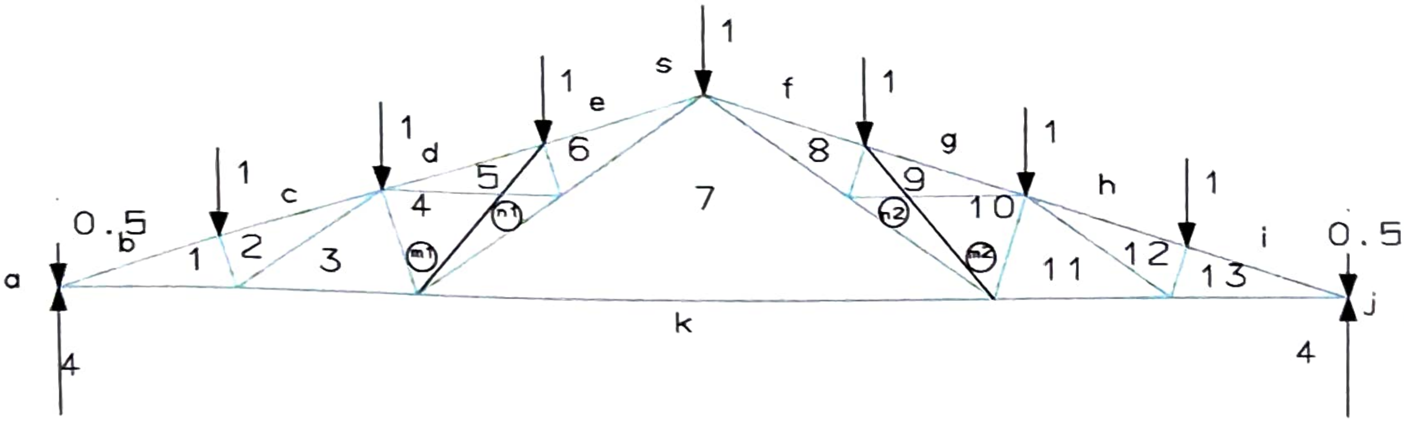
DRAWING: DEAD LOAD STRESS DIAGRAM

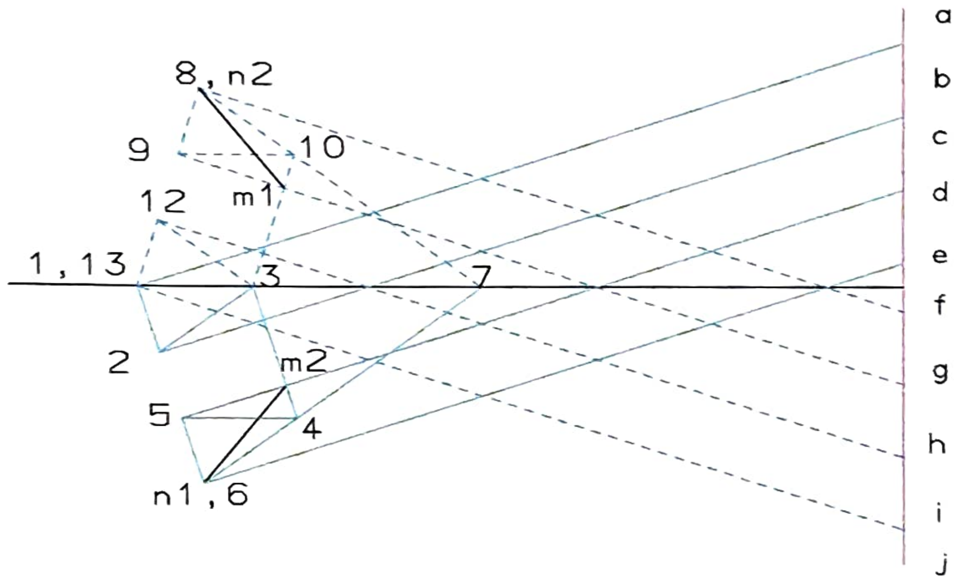
ATASI ROY MALAKAR

UNIV. ROLL NO.: 16501311014

GROUP-1 (A) SEVENTH SEMESTER



LIVE LOADS AT PANEL POINTS



LIVE LOAD STRESS DIAGRAM

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DRAWING: LIVE LOAD STRESS DIAGRAM
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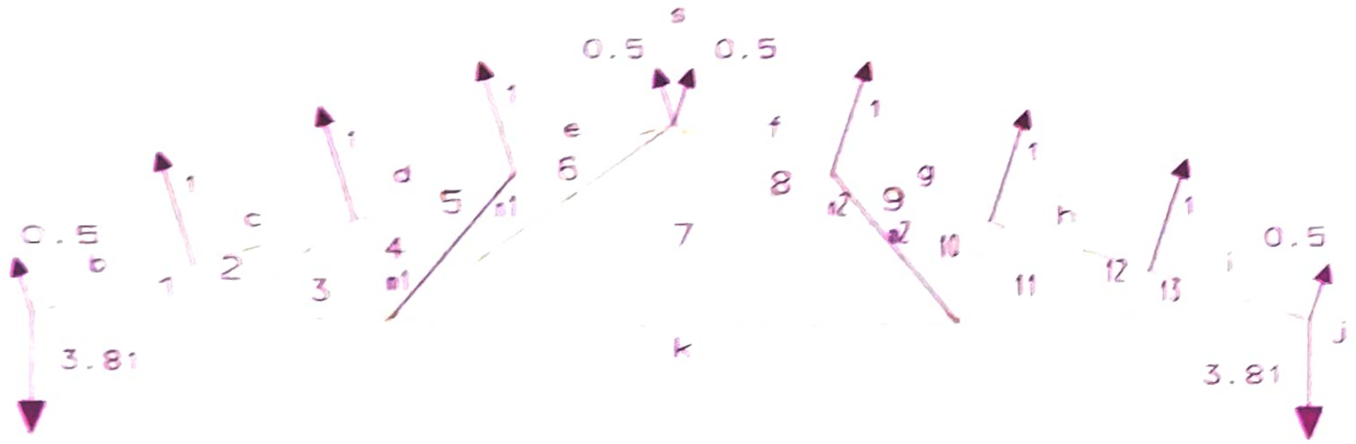
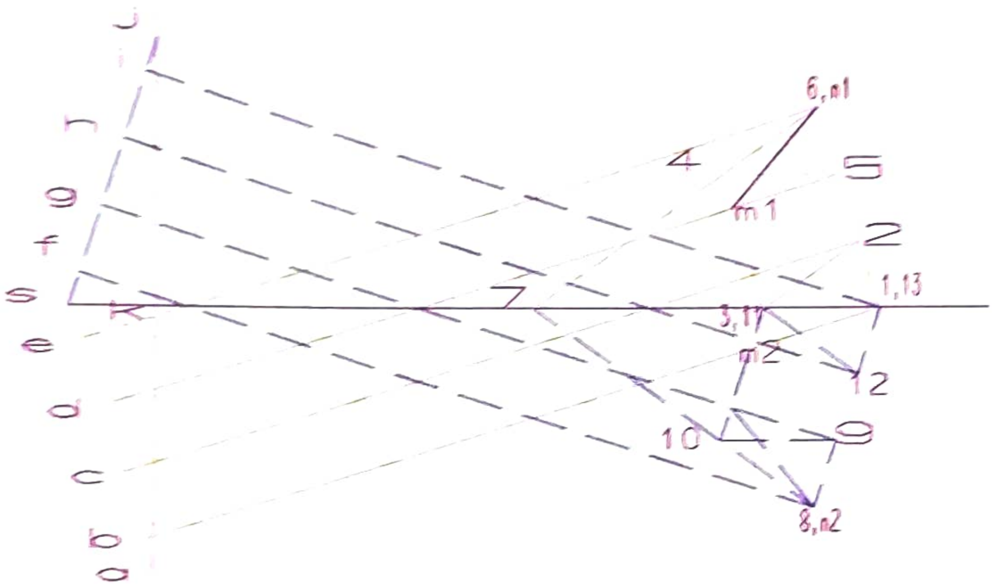
GROUP-1 (A) SEVENTH SEMESTER



WIND LOADS AT PANEL POINTS



WIND LOAD STRESS DIAGRAM

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DRAWING: WIND LOAD STRESS DIAGRAM
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UNIV ROLL No.: 16501311014
GROUP-1 (A) SEVENTH SEMESTER

PURLIN AND TRUSS MEMBER DESIGN

(Reference codes and handbook- IS:800-1997 and SP:6(1)-1964)

Design Connection using 20m dia bolt.

Grade of steel= Fe- 410 Grade of bolt= 4.6 γ_{nb}= 1.25

f_{ub}= 400 N/mm² Stress area of 20mm bolt= 245 mm²

In all cases, we will design for compression and check for tension, since the members are subjected to reversal of stress.

DESIGN OF PURLIN:

Truss spacing= 4.00 m Span of truss= 12.00 m

Slope= 17.74467 Degrees. Spacing of purlin= Y_z= 1.57 m (rafter panel length)

Design wind pressure [with WL factor= 7.5 and A= 6.3 m²] is= 1.185482 kN/mm²

Wind load= 1861.206 N/mm² (acts normal to the roof)

Wt. Of galvanised sheet= 171.00 N/m²= 268.47 N/m

Dead load of purlin= 90.00 N/m

Total dead load= 358.47 N/m

Component of dead load parallel to roof= 109.253 N/m

Component of dead load normal to roof= 341.4155 N/m

So total load on purlin normal to the roof= 2202.621 N/m

Factored load normal to roof,P= 3303.932 N/m

Factored load parallel to rooh, H= 163.8794 N/m

Max moment, M_{uu}=M_z= PL/10= 5.286291 kN-m

Max moment, M_{vv}=M_y= HL/10= 0.262207 kN-m

Let us try a section with flange width b_f= 50 mm and depth, d= 150 mm

Plastic section modulus required, $Z_{pz,\,reqd}$= $M_z.\gamma_{mo}/f_y+2.5(d/b)(M_y.\gamma_{mo}/f_y)$= 31912.52 mm³

Let us try section ISMC 125 with the following sectional properties:

Area, A= 1619 mm²

Depth of section,h= 125 mm

Width of flange, b_f= 65 mm

Thickness of flange, t_f= 8.1 mm

Thickness of web, t_w= 5 mm Radius at root, R_1= 9.5 mm

Depth of web,$d=h-2(t_f+R_1)$= 89.8 mm

Moments of inertia,I_z= 4164000 mm⁴ I_y= 599000 mm⁴

Plastic section modulus, Z_{pz}= 77150 mm³

Elastic section modulus, Z_{ez}= 66600 mm³ Z_{ey}= 13100 mm³

Classification of Section:

Outstand of flange, b= $b_f/2$= 32.5 mm

b/t_f= 4.012346 < 9.4ε

d/t_w= 17.96 < 42ε **Both of which imply that the section is plastic.**

Check for design capacity of Section:

$M_{dz}=Z_{pz}.f_y/\gamma_{mo}=$ 17.53409 kN-m < $1.2\,Z_{ez}.f_y/\gamma_{mo}=$ 18.16364 kN-m

Which is all right.

Also, $M_{dz}=Z_{pz}.f_y/\gamma_{mo}=$ 17.53409 kN-m > 5.29 kN-m Hence ok.

$M_{dy}=$ minimum of $Z_{py}.f_y/\gamma_{mo}$ and $\gamma_f.Z_{ey}.f_y/\gamma_{mo}$ where, $Z_{py}=$ 28864.34

= 6.560076 kN-m > 4.465909 kN-m with equal area axis at y (mm)= 15.70988

Hence $M_{dy}=$ 4.465909 kN-m

Check for overall member strength (local capacity):

$M_z/M_{dz}+M_y/M_{dy}$ must be ≤ 1

= 0.3602 < 1 Which is all right.

Check for Deflection:

Deflection will be checked at service loads.

$\delta_{all}=L/180=$ 22.22222 mm

$\delta=5/384\,x(\,wL^4/EI)=$ 8.816128 mm < 22.22222 mm

Which is all right. Hence section is safe.

Provide 2 nos. 20 mm dia bolt attached with both the ends of the rafter.

DESIGN OF PRINCIPAL RAFTER:

Design compressive load= **99.13563** kN

Design Tensile load= **88.41393** kN

Length of rafter panel= 1.57 m

Let us provide double angle section placed on opposite side of gusset plate of thickness= 12 mm. Let us assume the slenderness ratio= 100

Corresponding value of design compressive stress from table 9 (c) (buckling class- C) of IS:800-2007, is= 107 N/mm²

As the purlins have been placed at panel points, effective length of the rafter panel about z-axis= =0.85L= 1.3345 m= 1334.5 mm (L_z)

And effective length of rafter panel about y-axis=1.0xL= 1570 mm (L_y)

C/s area required= Design compressive load/ design compressive stress= 926.5012 mm²

Lets try 2-unequal angle sec ISA | 65 x 45 x 8 | mm back-to-back.

Following are the properties from IS Handbook SP: 6(1)-1964 for the abovementioned section:

Area, A= 1634 mm^2, r_z= 20.2 mm,

For 12 mm gusset plate, r_y= 21.95 mm, (Table XXII)

L_z/r_z= 66.06436 < 180

L_y/r_y= 71.5262 < 180 Both of which is alright.

The slenderness ratio 71.5262 is critical.

For slenderness ratio= 71.5262 ,f_y= 250.00 N/mm^2, (buckling class- C) the design compressive stress, from table 9 (c) f_{cd}= **149.558** N/mm^2(interpolated)

Classification of Section:

b/t= 8.125 < 15.78

So, the section is semi-compact. Since the section is not slender, the full section will be effective.

Design Compressive strength, $P_d=A_e.f_{cd}$= **244.3778** kN > 99.13563 kN which is alright.

Tensile strength due to gross section yielding, $T_{dg}=f_yA_g/\gamma_{mo}$= **371.3636** kN > 88.41393 kN

which is alright.

Connection design:

Let us use 20 mm dia bolts, with dia of hole= 22 mm

Section to be connected with the 12 mm thick gusset plate is the back to back double angle ISA 65 x 45 x 8 mm

Design force= 99.13563 kN

Strength of bolt in double shear, $V_{dsb}=2A_{nb}.f_{ub}/(\sqrt{3}\gamma_{mb})$= **90.52852** kN

Strength of bolt in bearing, $V_{dpb}=2.5k_b.d.t.f_u/\gamma_{mb}$ = **99.89091** kN

where, k_b is the least among:

(let, end distance, e(mm)= 40

pitch of bolts, p(mm)= 50

$e/3d_o$= 0.606061

$p/3d_o-0.25$= 0.507576

f_{ub}/f_u= 0.97561 and 1

that is, k_b= 0.507576

Hence strength of bolt= **90.52852** kN

Number of bolts required= 1.095076 = 3

Check for block shear failure:

The member is of double angle section

Let us provide end distance= 40 mm and edge distance= 40 mm

and pitch= 50 mm

A_{vg}= 2240 mm²

A_{vn}= 1712 mm²

A_{tg}= 640 mm²

A_{tn}= 464 mm²

$T_{db1}= A_{vg}.f_y/(\sqrt{3}.\gamma_{mo})+(0.9 x A_{tn}.f_u/\gamma_{mb})$= **430.8966** kN

$T_{db2}= 0.9 x A_{vn}.f_u/(\sqrt{3}.\gamma_{mb})+(A_{tg}.f_y/\gamma_{mo})$= **437.2372** kN

Hence, the block shear strength is **437.2372** kN which is > 99.13563 kN

Hence the section is safe.

Provide 3 nos. 20 mm dia bolt of grade 4.6

DESIGN OF PRINCIPAL TIE :

Design tensile load= **94.42229** kN

Design compressive load= **79.64836** kN

Max unrestrained length= 5.386 m (L)

Let us provide longitudinal tie runner at each bottom node of truss.

Effective length=1xL= 5386 mm (L_y)

Assuming the member to be connected by more than one bolt at the ends,

Effective length= 0.85 x L= 4578.1 mm (L_z)

Let us provide double angle section placed on opposite side of a gusset plate of thickness 12 mm (assumed) in back-to back mode.

Let the slenderness ratio=(say) 100

Corresponding value of design compressive stress from table 9 (c) (buckling class- C) is= **107** N/mm²

C/s area required= 744.3772 mm²

Let us try 2 equal sections ISA 90 x 60 x 8 mm back to back with 12 mm thk gusset plate in between.

From IS Handbook SP:6(1), table XXII, the properties of the section has been taken:

A= 2274 mm², r_z= 28.4 mm

For gusset plate thk= 12 mm, r_y (interpolated)= 26.8 mm

L_z/r_z= 161.2007 < 250

L_y/r_y= 200.9701 < 250 Both of which is alright.

The slenderness ratio 200.9701 is critical.

For this slenderness ratio and f_y= 250.00 N/mm² and for buckling curve C, from table 9 (C)

Design compressive stress, f_{cd}= 36.009 N/mm²

Classification of Section

b/t= 11.25 < 15.7ε

So, the section is semi-compact. Since the section is not slender, the full section will be effective.

Design Compressive strength, $P_d = A_e.f_{cd}$= **81.88447 kN** > 79.64836 kN
which is alright.

Tensile strength due to gross section yielding, $T_{dg} = f_y A_g/\gamma_{mo}$= **516.8182** kN > 94.42229 kN

which is alright.

Connection design.

Let us use 20 mm dia bolts, with dia of hole= 22 mm

Section to be connected with the 12 mm thick gusset plate is the back to back double angle
ISA 90 x 60 x 8 mm

Design force= 94.42229 kN

Strength of bolt in double shear, $V_{dsb} = 2A_{nb}.f_{ub}/(\sqrt{3}\gamma_{mb})$= **90.52852** kN

Strength of bolt in bearing, $V_{dpb} = 2.5k_b.d.t.f_u/\gamma_{mb}$
= **99.89091** kN

where, k_b is the least among:
(let, end distance, e(mm)= 40
pitch of bolts, p(mm)= 50
$e/3d_o$= 0.606061
$p/3d_o - 0.25$= 0.507576
f_{ub}/f_u= 0.97561 and 1
that is, k_b= 0.507576

Hence strength of bolt= **90.52852** kN
No. of bolts required= 1.043011 = 3

Check for block shear failure:

The member is of double angle section
Let us provide end distance= 40 mm and edge distance= 40 mm
and pitch= 50 mm

A_{vg}= 2240 mm²
A_{vn}= 1712 mm²
A_{tg}= 640 mm²
A_{tn}= 464 mm²

$T_{db1} = A_{vg}.f_y/(\sqrt{3}.\gamma_{mo}) + (0.9 x A_{tn}.f_u/\gamma_{mb})$
= **430.8966** kN

$T_{db2} = 0.9 x A_{vn}.f_u/(\sqrt{3}.\gamma_{mb}) + (A_{tg}.f_y/\gamma_{mo})$
= **437.2372** kN

Hence, the block shear strength is **430.8966** kN which is > 94.42229 kN
Hence the section is safe.

Provide 3 nos. 20 mm dia bolt of grade 4.6

DESIGN OF STRUT:

Design tensile load= **16.25922** kN
Design compressive load= **16.44491** kN
Length= 1008 mm (L)

Let us assume slenderness ratio= 100
Corresponding design compressive stress from table 9(c) of IS:800= **107** N/mm^2
As the purlins have been placed at panel points, effective length of the member about z-axis=
=0.85L= 856.8 mm (L_z)
Effective length of the member about y-axis= 1.0x L= 1008 mm (L_y)
C/s area required= 153.6908 mm^2

Let us try an equal angle section ISA 45 x 45 x 5
mm back-to-back. Properties from IS handbook SP:6(1) are as follows:
A= 327 mm^2, r_z= 6.7 mm

Let us assume the strut to be connected by more than 1 bolt at its each end.
So, effective length, l= k.L= 1008 mm (k= 1)
Equivalent slenderness ratio=$\lambda_e=\sqrt{(k_1+k_2.\lambda_{vv}^2+k_3.\lambda_\varphi^2)}$ where,
= 1.186401 $\lambda_{vv}=L/r_{vv}/(\varepsilon\sqrt{(\pi^2.E/250)})$= 1.692451
$\lambda_\varphi=(b_1+b_2)/(\varepsilon\sqrt{(\pi^2.E/250)}*2t)$= 0.101245
k_1= 0.2 k_2= 0.35
k_3= 20
$\varphi=0.5\ [1+\alpha(\lambda_e-0.2)+\lambda_e^2]$= 1.445442 where, α= 0.49
$f_{cd}=f_y/\gamma_{mo}/[\varphi\sqrt{(\varphi^2-\lambda_e^2)}]$= 190.4288 N/mm^2

Classification of Section:

b/t= 9 < 15.7ε

So, the section is semi-compact.Since the section is not slender, the full section will be effective.
Design Compressive strength, $P_d=A_e.f_{cd}$= **62.27023** kN > 16.44491 kN
which is alright.
Tensile strength due to gross section yielding, $T_{dg}=f_yA_g/\gamma_{mo}$= **74.31818** kN > 16.25922 kN

which is alright.

Connection design:

Let us use 12 mm dia bolts, with dia of hole= 14 mm

Section to be connected with the 12 mm thk gusset plate is the back to back double angle

ISA 35 x 35 x 5 mm

Design force= 16.44491 kN

Strength of bolt in single shear, $V_{dsb}=A_{nb}.f_{ub}/(\sqrt{3}\gamma_{mb})=$ where, $A_{nb}=$ 84.3

= 15.5746 kN

Strength of bolt in bearing, $V_{dpb}=2.5k_b.d.t.f_u/\gamma_{mb}$ where, k_b is the least among:

= 22.84286 kN

(let, end distance, e(mm)= 25

pitch of bolts, p(mm)= 30

$e/3d_o=$ 0.595238

$p/3d_o-0.25=$ 0.464286

$f_{ub}/f_u=$ 0.97561 and 1

that is, $k_b=$ 0.464286

Hence strength of bolt= **15.5746** kN

No. of bolts required= 1.05588 = 2

Check for block shear failure:

The member is of single angle section

Let us provide end distance= 25 mm and edge distance= 20 mm

and pitch= 30 mm

$A_{vg}=$ 275 mm²

$A_{vn}=$ 170 mm²

$A_{tg}=$ 100 mm²

$A_{tn}=$ 65 mm²

$T_{db1}= A_{vg}.f_y/(\sqrt{3}.\gamma_{mo})+(0.9xA_{tn}.f_u/\gamma_{mb})=$ 55.27239 kN

$T_{db2}= 0.9xA_{vn}.f_u/(\sqrt{3}.\gamma_{mb})+(A_{tg}.f_y/\gamma_{mo})=$ 51.70102 kN

Hence, the block shear strength is **51.70102** kN which is > 16.44491 kN

Hence the section is safe.

Provide 2 nos. 12 mm dia bolt of grade 4.6

DESIGN OF SLING -I (SECONDARY TIE) :

Design tensile load= **13.48397** kN
Design compressive load= **13.34414** kN
Length= 1654 mm (L)

Let us provide double angle section placed on opposite side of gusset plate of 12 mm thk.
Let us assume slenderness ratio= 100
Corresponding design compressive stress from table 9(c) of IS:800= **107** N/mm²
As the purlins have been placed at panel points, effective length of the rafter panel about z-axis=
=0.85L= 1405.9 mm (L_z)
Effective length of the rafter panel about y-axis= 1.0x L= 1654 mm (L_y)
C/s area required= 126.0184 mm²

Let us try a 2 equal angle section ISA | 50 x 50 x 4 |
mm back-to-back. Properties from IS handbook SP:6(1) are as follows:

A= 776 mm², r_z= 15.2 mm
For 12 mm gusset plate, r_y= 24.95 mm
So, L_z/r_z= 92.49342 < 180
And, L_y/r_y= 66.29259 < 180 Both of which is alright.
The slenderness ratio 92.49342 is critical.
For this slenderness ratio, f_y= 250.00 N/mm² and buckling curve C, the design compressive stress, f_{cd}= **117.514** kN

Classification of Section:

b/t= 12.5 < 15.7ε

So, the section is semi-compact.Since the section is not slender, the full section will be effective.

Design Compressive strength, P_d=A_e.f_{cd}= **91.19086** kN > 13.34414 kN
which is alright.
Tensile strength due to gross section yielding, T_{dg}=f_yA_g/γ_{mo}= **176.3636** kN > 13.48397 kN
which is alright.

Connection design:

Let us use 20 mm dia bolts, with dia of hole= 22 mm
Section to be connected with the 12 mm thk gusset plate is the back to back double angle
ISA 50 x 50 x 4 mm
Design force= 13.48397 kN
Strength of bolt in double shear, V_{dsb}=$2A_{nb}.f_{ub}/(\sqrt{3}\gamma_{mb})$= **90.52852** kN
Strength of bolt in bearing, V_{dpb}=$2.5k_b.d.t.f_u/\gamma_{mb}$ where, k_b is the least among:
= **99.89091** kN

(let, end distance, e(mm)= 40
pitch of bolts, p(mm)= 50
$e/3d_o$= 0.606061
$p/3d_o$-0.25= 0.507576
f_{ub}/f_u= 0.97561 and 1
that is, k_b= 0.507576

Hence strength of bolt= **90.52852** kN
No. of bolts required= 0.148947 = 3
Provide 3 nos. 20 mm dia bolt of grade 4.6

DESIGN OF SLING -II :

Design tensile load= **40.46916** kN
Design compressive load= **40.01461** kN
Length= 1654 mm (L)

Let us provide double angle section placed on opposite side of gusset plate of 12 mm thk.
Let us assume slenderness ratio= 100
Corresponding design compressive stress from table 9(c) of IS:800= **107** N/mm²
As the purlins have been placed at panel points, effective length of the rafter panel about z-axis=
=0.85L= 1405.9 mm (L_z)
Effective length of the rafter panel about y-axis= 1.0x L= 1654 mm (L_y)
C/s area required= 378.2164 mm²

Let us try a 2 equal angle section ISA 50 x 50 x 4
mm back-to-back. Properties from IS handbook SP:6(1) are as follows:
A= 776 mm², r_z= 15.2 mm
For 12 mm gusset plate, r_y= 24.95 mm
So, L_z/r_z= 92.49342 < 180
And, L_y/r_y= 66.29259 < 180 Both of which is alright.
The slenderness ratio 92.49342 is critical.
For this slenderness ratio, f_y= 250.00 N/mm² and buckling curve C, the design
compressive stress, f_{cd}= **117.514** kN

Classification of Section:
b/t= 12.5 < 15.7ε
So, the section is semi-compact. Since the section is not slender, the full section will be effective.
Design Compressive strength, $P_d=A_e.f_{cd}$= **91.19086** kN > 40.01461 kN
which is alright.
Tensile strength due to gross section yielding, $T_{dg}=f_yA_g/\gamma_{mo}$= **176.3636** kN > 40.46916 kN
which is alright.

Connection design:
Let us use 20 mm dia bolts, with dia of hole= 22 mm
Section to be connected with the 12 mm thk gusset plate is the back to back double angle
ISA 50 x 50 x 4 mm
Design force= 40.46916 kN
Strength of bolt in double shear, $V_{dsb}=2A_{nb}.f_{ub}/(\sqrt{3}\gamma_{mb})$= **90.52852** kN
Strength of bolt in bearing, $V_{dpb}=2.5k_b.d.t.f_u/\gamma_{mb}$ = **99.89091** kN
where, k_b is the least among:
(let, end distance, e(mm)= 40
pitch of bolts, p(mm)= 50
$e/3d_o$= 0.606061
$p/3d_o-0.25$= 0.507576
f_{ub}/f_u= 0.97561 and 1
that is, k_b= 0.507576

Hence strength of bolt= **90.52852** kN
No. of bolts required= 0.447032 = 3
Provide 3 nos. 20 mm dia bolt of grade 4.6

BRACING DESIGN:

RAFTER & EAVES LEVEL BRACING

(Reference- IS:800-1997 and IS: 875-part III)

DESIGN OF RAFTER BRACING :

Height of building= 10.75 m
Span of truss= 12.00 m
Length of truss= 48.00 m (=d)
Design wind pressure= 1.185 kN/m²
Let θ be the inclination of the roof with the horizontal= 17.74467 °
Let the width of the bracing be= 3 m
Factored max compressive force in rafter bracing, as per IS:800= 79.64836 kN
Shear in rafter bracing, as per IS:800= =(2.5+1.25)/100*Max compressive force
= **2.986814** Kn...(i)

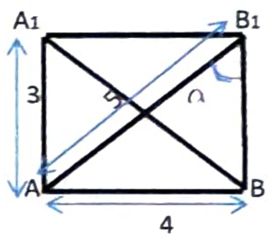
As per IS 875 (part 3) 1987,for h < b and d/h > 4
Frictional drag force, P'= $C_f'(d-4h) x b x P_d + C_f'(d-4h) x 2h x P_d$
= 3.96975 kN

Where, C_f' = 0.02 for surfaces with corrugations across the wind direction(cl. 6.3.1, IS 875) (part-3)

So, factored drag force= 1.5x frictional drag force= 5.954625 kN
Since the bracing has been done in 4 bays and total number of braced panels is 4 in each bay, ie, 16 the drag force in each panel=
= Drag force/braced panels no.
= **0.372164** kN...(ii)

Since out of two bracings one will be in tension and the other will buckle out, so 0.372164 kN wil be shared by it.
Total shear in bracing= (i) +(ii)= **3.358978** kN

Bracing length=√[(Spacing of truss)²+(Panel length projection in plan)²+(1/2 Height of truss)²]
= 5.091326 m
Design force in bracing= Total shear in bracing*bracing length/spacing of truss
= **4.275413** kN



Inclined length of portion AA_1= panel length projection in plan x sec θ
= 3.149857 m
cos α = inclined length of portion AA_1/Bracing length= 0.618671
So, α = 51.78083 °
Factored wind load on the triangular portion of the end frame=
= 1.5x 1.05 x maximum wind force coefficient x (1/2 x span of truss x height)
= **18.144** kN

This force will be equally distributed amongst those bays which have cross bracings.

Factored wind load in each bay= 18.144 / 4 = **4.536** kN

Factored wind load on restraint point A_1= 4.536 / 4 = **1.134** kN

So, Design force in diagonal member A_1B= factored wind load on restraint A_1 x sec α= **1.83296** kN

Using 2 nos. 16 mm dia 4.6 grade bolts α_{bolt} = 0.6

Design tensile force= 1.83296 kN

Limiting slenderness ratio from IS:800, table 3= 400

kL/r= 5091.326 /r ≤ 400 From table 11, k= 1

So, r ≥ 12.72831 mm

Let us try ISA 90 x 90 x 8 mm

From IS handbook no. 1, the properties are:

A= 1379 mm², r_{vv}= 17.5 mm

L/r= 290.9329 < 400 which is sufficient.

Design strength of member due to gross section yielding, $T_{dg}=A_g x f_y/\gamma_{mo}$= **313.4091** kN

> 1.83296 kN

Hence section is safe.

Design tensile strength, T_{dn}= 0.9 xA_{nc} x $f_u/\gamma_{m1}+\beta$xA_{go}xf_y/γ_{mo} where,

= 270.0433 kN β=1.4-0.076 (w/t)(fy/fu)(bs/Lc)≥0.7

> 1.83296 kN = max. of -0.071341 and 0.7

= 0.7

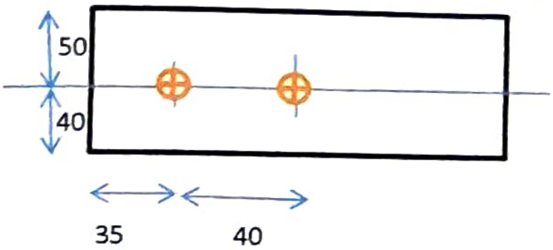
Hence section is safe. For d= 16 mm, d_o= 18

A_{nc}= 544 mm²,

A_{go}= 688 mm²,

Block Shear Strength

For ISA 90 x 90 x 8 mm, with 2 nos. 16 mm dia bolts for pitch 40 mm and at edge dist 40 mm and end distance 35 mm



A_{vg}= 600 mm²,

A_{vn}= 384 mm²,

A_{tg}= 280 mm²,

A_{tn}= 208 mm²,

$T_{db1}= f_y x A_{vg}/(\sqrt{3}\gamma_{mo})+0.3 f_u x A_{tn}/\gamma_{m1}$= 99.19678 kN

$T_{db2}= 0.9 f_y x A_{vn}/(\sqrt{3}\gamma_{m1})+f_u x A_{tg}/\gamma_{m0}$= 144.2701 kN

Minimum of T_{db1} and T_{db2} must be > 1.83296 kN

Here, 99.19678 kN > 1.83296 kN

Hence section is safe.

Hence we use ISA | 90 x 90 x 8 | mm with above mentioned connections.

DESIGN OF EAVES LEVEL BRACING :

Wind load on wall and roof:

Internal wind pressure coefficient, c_{pi}= ± 0.2 (for opening of factory shed <5%)

External wind pressure coefficient, c_{pe}, on walls=

On windward side= 0.7 (for (3/2>h/w>1/2)

On leeward side= -0.3 and for (3/2<l/w<4)

Internal wind pressure coefficient on roof, c_{pi}= ± 0.2

External wind pressure coefficient, c_{pe}, on roof=

On windward side= -0.790213

On leeward side= -0.522553

Design wind pressure, P_d= 1.185 kN/m²

Wind load on windward wall= (c_{pe}+c_{pi}) x area x wind pressure= **550.314** kN...(i)

Wind load on windward roof= (c_{pe}+c_{pi}) x area x wind pressure= **-30.69517** kN...(ii)

Wind load on leeward wall= (c_{pe}+c_{pi}) x area x wind pressure= **-61.146** kN...(iii)

Wind load on leeward roof= (c_{pe}+c_{pi}) x area x wind pressure= **-16.77501** kN...(iv)

Total horizontal load=(i) +(ii) + (iii) +(iv) = **441.6978** kN

Providing Eaves bracing,

End brace gets maximum shear, V= 6 F-0.5xF= 5.5 F

where, F= 36.80815 kN per unit length of truss span

Tension in brace, T= V/ cos α= **161.9559** kN

Tension/compression in the parallel chords of eaves bracing= T sinα= **129.5647** kN

Let us try 2 ISA | 150 x 150 x 10 | mm

r_{min}= 46.3 mm, Area= 2903 mm²

r_w= 29.3 mm, L= 5 m

So λ= 170.6485 < 350 Hence Ok.

Design strength of member due to gross yielding:

T_{dg}=A_g.f_y/γ_{mo}= 659.7727 kN > 129.5647 kN **Hence section is safe.**

Design for net section rupture:

Design tensile strength, T_{dn}= 0.9 xA_{nc} x f_u/γ_{m1}+βxA_{go}xf_y/γ_{mo} where,

= 599.6818 kN β=1.4-0.076 (w/t)(fy/fu)(bs/Lc)≥0.7

> 129.5647 kN = max. of -2.870035 and 0.7

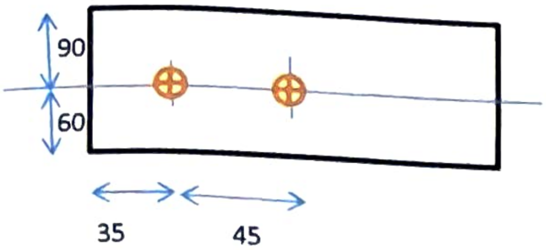
Hence section is safe. For d= 18 mm, d_o= 20

A_{nc}= 1250 mm²,

A_{go}= 1450 mm²,

Design for block shear:

For ISA 150 x 150 x 10 mm, with 2 nos. 18 mm dia bolts for pitch 45 mm and at edge dist 60 mm and end distance 35 mm



A_{vg}= 800 mm²,
A_{vn}= 500 mm²,
A_{tg}= 350 mm²,
A_{tn}= 250 mm²,

$T_{db1}= f_y x A_{vg}/(\sqrt{3}\gamma_{mo})+0.3 f_u x A_{tn}/\gamma_{m1}=$ 129.5728 kN

$T_{db2}= 0.9 f_y x A_{vn}/(\sqrt{3}\gamma_{m1})+f_u x A_{tg}/\gamma_{m0}=$ 182.4161 kN

Minimum of T_{db1} and T_{db2} must be > 129.5647 kN

Here, 129.5728 kN > 129.5647 kN

Hence section is safe.

Design compressive load:

L/r_{min}= 73.43413 < 250 Hence Ok.

For λ= 73.43413 and, f_y= 250.00 N/mm²,& buckling curve C, f_{cd}= 146.5056 N/mm²

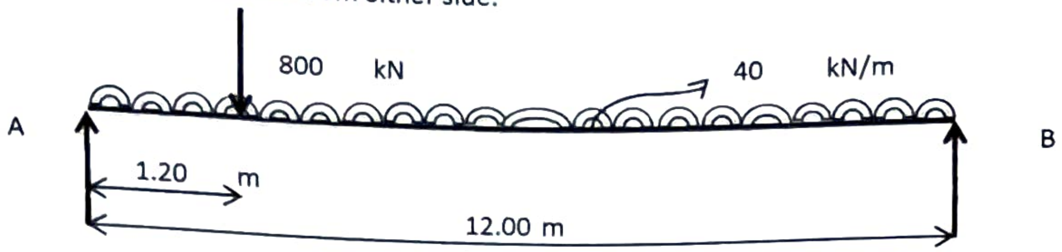
So, design compressive load, $P_d= A_e.f_{cd}$= 425.3058 kN > 129.5647 kN

Hence provide 2 ISA 150 x 150 x 10 mm as eaves girder.

GANTRY SECTION DESIGN

DESIGN FORCES:

For an EOT crane,
Maximum concentrated load on crane= 640.00 kN
Maximum factored concentrated load on crane= 800 kN
Self weight of crane as an UDL= 29.1666667 kN/m= 30 kN/m
Factored UDL= 36.45833 kN/m= 40 kN/m (approx)
For maximum reaction on the gantry girder, the crane load must be placed at a distance equal to the approach distance from either side.



Taking moment about B, we get, Ra= **960** kN,so Rb= **320** kN
Now taking the maximum of the reaction values, ie, Ra, we observe that this load is distributed equally on the two wheels at thte crane girder end.
Hence, max. wheel load on each wheel of crane=Ra/2= **480** kN

MAXIMUM BENDING MOMENT:

i)BM due to Dead Load:

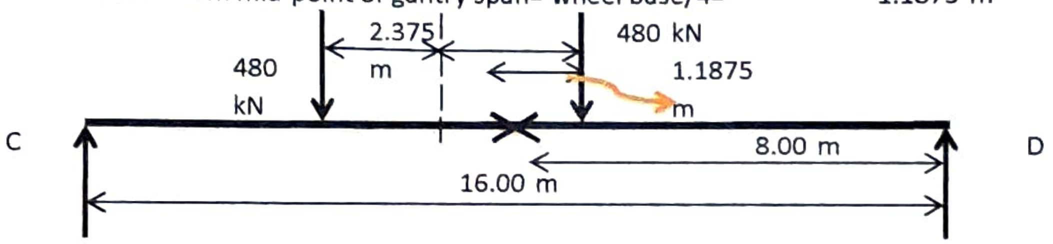
Self weight of Gantry girder= 3.43 kN/m

Self weight of Rail= 0.30 kN/m (Assumed)
Therefore, total dead load= 3.73 kN/m
Factored dead load, w= **4.660714** kN/m

Hence, BM due to dead load= $wl^2/8$= 149.142857 kN-m....(i)

ii)BM due to Live Loads:

BM due to Wheel load will be maximum when the CG of the wheel loads' CG and one of the adjacent wheel load coincides with the CG of the girder span.
ie, position of one wheel load from mid-point of gantry span= wheel base/4= 1.1875 m



Taking moment about C, we get,
Rd= **408.75** kN and Rc= **551.25** kN
Maximum Bending moment due to live load=BM about critical wheel load= 2784.6094 kN-m..(ii)

iii)BM due to Impact Load:

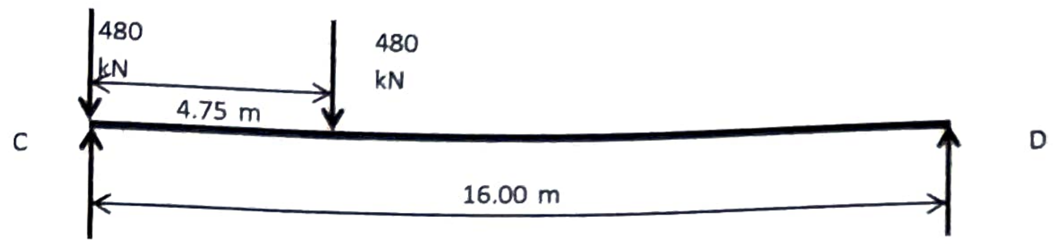
For an EOT Crane, as per IS: 875-1964, BM due to Impact loads=
0.25 of max BM due to static wheel / live loads= 696.15234 kN-m..(iii)

TOTAL BENDING MOMENT=(i)+(ii)=(iii)= 3629.90458 kN-m= 3630 kN-m Approx.

MAXIMUM SHEAR FORCE:

i) SF due to Live Loads:

Maximum shear due to live loads happens only when one of the wheel loads is at one of the supports, that is:



Taking moment about D, Rc= 817.5 kN Rd= 142.5 kN
Hence Max SF due to live load= 817.5 kN.....(iv)

ii)SF due to dead load:

Factored dead load, w, was calculated out to be= 4.660714 kN/m
Hence Max SF due to dead load=wl/2= 37.2857143 kN....(v)

iii) SF due to Impact load:

For an EOT crane, as per IS: 875-1964, SF due to impact loads=
= 0.25 of SF due to wheel load= 204.375 kN....(vi)

Lateral Load

For an EOT Crane, as per IS: 875-1964, Lateral loads transverse to rails=
= 0.1 of (weight of crab + weight lifted by crane)= 64 kN

Factored lateral load= 80 kN
Lateral force on each wheel= 40 kN
By proportionality,

Max horizontal reaction due to lateral force at C= 45.9375 kN
Horizontal reaction at D= 34.0625 kN
Max BM due to lateral load= 232.050781 kN-m
Max SF due to lateral load= 68.125 kN....(vii)

TOTAL SHEAR FORCE= (iv)+(v)+(vi)= 1059.16071 kN= 1060 kN approx.

PRELIMINARY TRIAL SECTION:

(Reference codes and handbook- IS:800-1997 and SP:6(1)-1964)

Approx. depth of gantry girder section=L/12= 1333.33333 mm= 1340 mm approx
Approx. width of gantry girder flange=L/30= 533.333333 mm= 540 mm approx
But depth= 1340 mm is not available for rolled sections, hence a built-up section will be considered.

Design of Web:

Let us assume $k=d/t_w=180$ ($<200\varepsilon$ (serviceability criteria) and $< 345\varepsilon_f^2$ (flange buckling criteria))

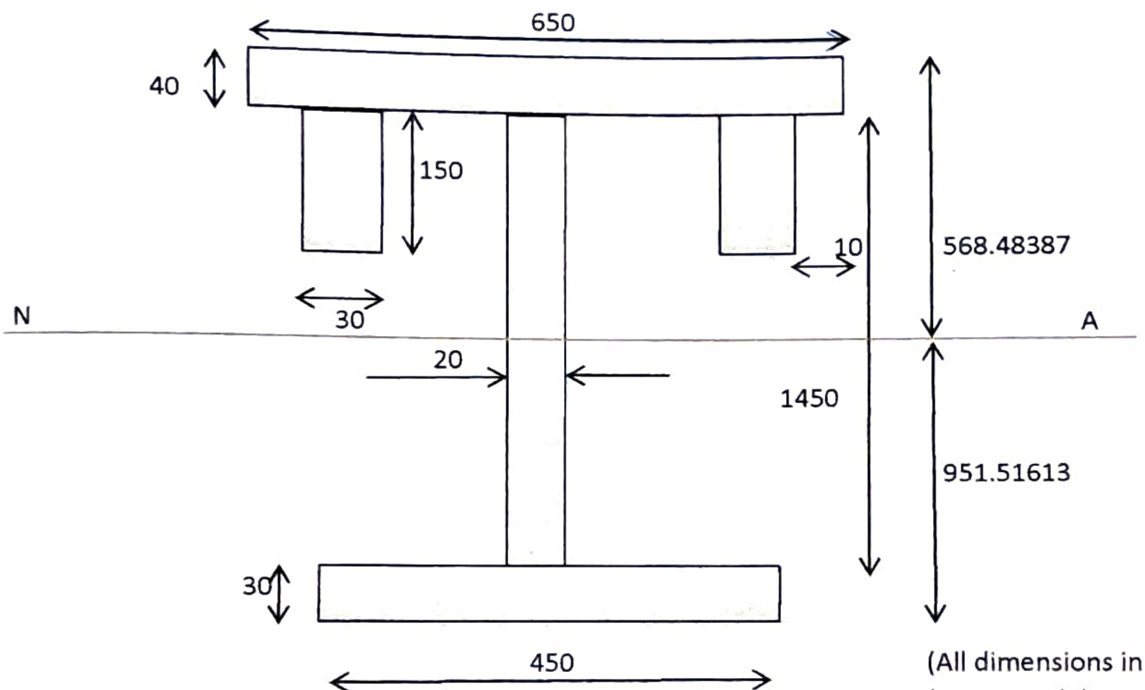
Hence, optimum depth of the plate girder, $d=\sqrt[3]{(M_z k/f_y)}=$ 1377.4502 mm= 1400 mm (approx

Optimum web thickness, $t_w=\sqrt[3]{(M_z/f_y k^2)}=$ 7.65250103 mm= 8 mm(approx.)

Let us try web plate | 1450 x 20 | mm in size

Design of Flanges:

Let us assume that bending moment will be resisted by the flanges and shear by the web.
Let us try a top flange plate | 650 X 40 | mm size and a bottom flange plate of | 450 X 30 | mm size.
Let us take 2 additional plates of 150 X 30 mm size attached to the top flange each at a distance of 10mm from the ends.



(All dimensions in mm)
(Not to scale)

Section Properties:

Total area= 77500 mm^2

Distance of **Neutral axis** from bottom, $\bar{Y}=(\Sigma ay)/\Sigma A=$ 951.5161 mm from bottom

or, 568.48387 mm from top.

I_x wrt CG= $\Sigma[I_{cg}+Ax^2]=\Sigma[bd^3/12+Ax^2]=$ 2.77E+10 mm^4 (27735.21 X $10^6 mm^4$)

I_y wrt CG= $\Sigma[I_{cg}+Ay^2]=\Sigma[db^3/12+Ay^2]=$ 1954870833 mm^4 (1954.87 X $10^6 mm^4$)

I_y (Compression flange)= $\Sigma[I_{cg}+Ay^2]=\Sigma[db^3/12+Ay^2]=$ 1.73E+09 mm^4 = (1726.0917 X$10^6 mm^4$)

$Z_{ebottom}=I_x/Y_{bottom}=$ 2.91E+07 mm^3=(29.1484341 X 10^6 mm^3)

$Z_{etop}=I_x/Y_{top}=$ 4.88E+07 mm^3=(48.7880248 X 10^6 mm^3)

$Z_{ey}=I_y/X_{max}=$ 6014987 mm^3=(6.01498718 X 10^6 mm^3)

Let the **Equal Area Axis** be at a depth Y from the top of the web.

So, area of the section above this axis must be equal to the same below the axis.

Hence, 35000 +Y x 20 = 13500 + 20 x (1450 -Y)

Or, 1075 = 1450 -2Y or, Y= 187.5 mm from top of web, or

1262.5 mm from bottom of web

Also, Y= 227.5 mm from top of section, or 1292.5 mm from bottom of section

Plastic section modulus above equal area axis:

Z_{pxtop}= Area x distance of CG of area from equal area axis

= 6759063 mm^3

Plastic section modulus below equal area axis:

$Z_{pxbottom}$= Area x distance of CG of area from equal area axis

= 33185313 mm^3

Total Z_{pz}= 39944375 mm^3

Plastic section modulus of compression flange about Y-Y axis:

Z_{py}= 6925000 mm^3

Elastic Section modulus of compression flange about Y-Y axis:

Z_{ey}= ($I_{ycompression\ flange}$)/(Y_{max}/2)

= 5311051 mm^3

Radius of Gyration= √ (I_y/A)= 158.8211 mm

Classification of Section:

Outstand of flange= 325 mm

b/t_f= 8.125 <8.4ε (Clause 3.7.2 and 3.7.4 and table 2 of IS-800)

b/t_f of extra plates= 5 <8.4ε (Clause 3.7.2 and 3.7.4 and table 2 of IS-800)

d/t_w of web= 72.5 <84ε (Clause 3.7.2 and 3.7.4 and table 2 of IS-800)

Hence the entire section is plastic. So,βb = 1.00

Check for moment capacity:

a) Local moment capacity-

M_{dz}= lesser value between $\beta_b . Z_{pz} . f_y/\gamma_{mo}$ and $1.2\ Z_e . F_y/\gamma_{mo}$ which is =

ie,lesser of 9.08E+09 N-mm = 9078.267 kN-m and 7.95E+09 N-mm= 7949.573 kN-m

So, design moment capacity= 7949.573 kN-m which is > 3630 kN-m

Hence section is safe.

b) Moment capacity of compression flange about Y-Y axis

M_{dyf}= lesser value between $\beta_b . Z_{py} . f_y/\gamma_{mo}$ and $1.2\ Z_{ey} . F_y/\gamma_{mo}$ which is =

lesser of 1.57E+09 N-mm = 1573.864 kN-m and 1.448E+09 N-mm= 1448.469 kN-m

So, design moment capacity= 1448.469 kN-m which is > 232.05078 kN-m

Hence section is safe.

c) Combined moment check for the local moment capacity-

= $M_z/M_{dz} + M_{yf}/M_{dyf}$ (must be ≤ 1.0)

= 0.616833 < 1

Hence section is safe.

Check for lateral torsional buckling: (buckling resistance in bending)

$M_{cr} = C_1 . \pi^2 E . I_y . h_f/(2L_{LT}^2) \times \sqrt{[1 + 1/20 \times \{(L_{LT}/r_y)/(h_f/t_f)\}^2]}$ (Clause 8.2.2.1)

= 1.37E+10 N-mm

= 13693.29 x 10^6 N-mm

Where, h_f= c/c flange distance = 1485 mm

Effective length, L_{LT}= 16000 mm

Thickness of (top)flange== 40 mm

Radius of gyration, r_y= 158.82109 mm

Coefficient, C_1= 1.046 (Annex E, IS 800-2007)

Now, from clause 8.2.2 of IS 800-2007, non-dimensional slenderness ratio,

$\lambda_{LT} = \sqrt{((\beta_B . Z_{pz} . f_y)/M_{cr})}$= (Clause 8.2.2)

= 0.853973

Now, $\varphi_{LT} = 0.5 [1 + \alpha_{LT}(\lambda_{LT} - 0.2) + \lambda^2_{LT}]$= 1.02485799

And, $\chi_{LT} = 1/[\varphi_{LT} + \sqrt{(\varphi^2_{LT} - \lambda^2_{LT})}]$= 0.628345 which is < 1, hence OK.

So, $f_{bd} = \chi_{LT} . f_y/\gamma_{mo}$= **142.8056** N/mm²= 145 N/mm²

Design bending strength, $M_{dz} = \beta_b . Z_{pz} . f_{bd}$= 5791934375 N-mm= 5791.9344 kN-m

> 3630 kN-m

Since lateral forces are also acting, the beam must be checked for biaxial bending. The bending strength about Y axis will be provided by the top flange only, as the lateral loads are applied here.

$M_{dy} = Z_{ey(compression\ flange)} . f_y/\gamma_{mo}$= 1.21E+09 N-mm= 1207.057 kN-m

Now, combined check= $(M_z/M_{dz}) + (M_y/M_{dy})$ which should be ≤ 1.0

= 0.818979 < 1.00

Hence section is safe.

Check for Shear capacity:

Total shear force= **1060** kN

V_d= Shear Capacity= $A_v.f_{yw}/(\sqrt{3}.\gamma_{mo})$= 3805263.14 N= **3805.2631** kN

0.6 V_d= 2283.158 kN > 1060 kN

Since V≤0.6Vd, the case is of low shear. No reduction, therefore, will be in the moment capacity.

Web buckling check:

Using Simple Post Critical Method (cl. 8.4.2.2)-

Let c/d= 1.4

So,stiffener Spacing, c= 2030 mm

d/tw= 72.5 < 200

< 345

From Serviceability condition, 3d≥c≥d,

d/tw= 72.5 < 200ε

Hence only transverse stiffeners are provided.

Number of panels formed= 7.881773 = 8

Actual spacing of stiffener= 2000 mm

So, actual c/d= 1.37931

For c/d > 1.00 k_v= 5.35+ 4.0/(c/d)²= 7.4525

From clause 8.4.2.1, resistance to shear buckilng will be verified when, d/tw>67$\varepsilon\sqrt{(k_v/5.35)}$

But, 72.5 not > 79.0767668

Hence check for resistance to shear buckling in web is not required.

Determination of Shear force corresponding to web buckling-

Elastic critical stress: $\tau_{cre}= k_v\pi^2E/[12(1-\mu^2)(d/t_w)^2]$= **256.4972** N/mm²

The non-dimensional web slenderness ratio is, $\lambda_w= \sqrt{(f_{yw}/(\sqrt{3}\tau_{cre}))}$= 0.7501505

As λ_w <0.8 τ_b= 144.337567 N/mm²

So, V_{cr}= 4185789 N= 4185.789 kN

> 1060 kN

Hence nominal stiffeners of 10 mm thick & 150 mm wide provided over the span.

Deflection check:

$\delta= wL^3.(3a/4L-a^3/L^3)/6EI_x$ where, w= max static wheel load= 480 kN

= 13.00905 mm

L= c/c distance between gantry columns= 16000.00 mm

c= wheel base distance= 4750.00 mm

a= (L-c)/2= 5625 mm

Permissible max deflection=L/500

= 32 mm > 13.009047 mm

Hence section is safe.

Connection Design:

Flange to web connection

Required shear capacity of the weld, q= VAŸ/I_x
= 706.9265 N/mm

Where, V= 1060 kN
A= area of compression flange=
= 35000 mm²
Ÿ= 528.48387 mm

Providing 8mm fillet weld, strength of weld= **883.730812** N/mm > 706.9265 N/mm

Hence the section is safe.

Connection for stiffeners

Since stiffeners are not carrying any load, 5mm fillet weld will suffice.

END PANEL DESIGN :

Local capacity of web, F_w= (b_1+n_2). t_w.f_{yw}/γ_{mo}
= 704.5455 kN
< 1060 Kn

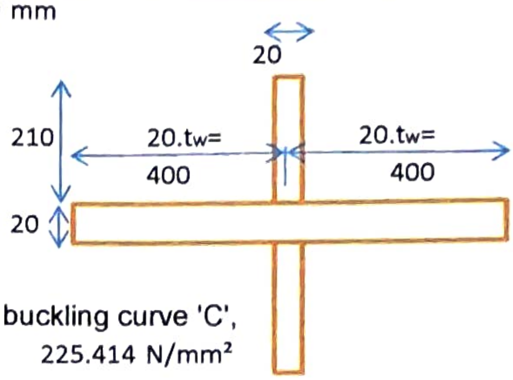
Where, b_1=stiff bearing length(mm)= 80
n_2= 2.5 x flange depth(mm)= 75

Hence stiffener will be required.

Let us try 2 flat section on each side of web 210 x 20 mm
Max Permissible outstand= 20 t_q. ε= 400 mm
Max Effective outstand= 14 t_q. ε= 280 mm

Check for buckling of stiffener:

Area, A= 24400 mm² (Stiffener's properties)
I_x = Σ[bd³/12]= 1.42E+08 mm⁴
Radius of gyration, r=√(I_x/A)= 76.27603 mm
Slenderness ratio, λ= 13.30693



For λ= 13.30693 , f_y= 250.00 N/mm², and buckling curve 'C',
f_{cd}, design compressive stress from table 9(C) of IS:800 is= 225.414 N/mm²

Buckling resistance,P_d= A_e.f_{cd}= **5500.102** kN > 1060 kN

Which is safe. Hence the stiffener is safe in compression.

Check for bearing capacity of stiffener:

Since the stiffener will be coped to accommodate the fillet weld of flnge plate to the web, the available effective width of stiffener flat for bearing will be lesser than the actual width. Let the stiffener be coped by 15 mm.
So, width available for bearing= 195 mm
Bearing strength of stiffener, F_p=A_q.f_{yq}/γ_{mo} which should be ≥ F_c-F_w

Fpsd=
= **1772.727** kN
> F_c-F_w= 355.4545 kN

Hence Safe.

Where,
A_q= area of stiffener in contact with flange
= 7800 mm²

Check for torsional resistance provided by end bearing stiffener:

The ends of the plate girder must have sufficient torsional resistance from transportation and erection point of view.

The moment of inertia of the end bearing stiffener at support, $I_s \geq 0.34\ \alpha_s.D^3.T_{cf}$...(i)

For the gantry girder section, $\lambda = L_{LT}/r_y =$ 100.742286 > 100

For $\lambda > 100, \alpha_s = 30/\lambda^2 =$ 0.002956

Now, $I_{s,\ provided} \geq$ 1.06E+08 mm^4 = 105.883573 x 10^6 mm^4

But, $I_{s,\ provided} =$ 1.23E+08 mm^4 = 123.48 x 10^6 mm^4 > 105.88357 x 10^6 mm^4

Which is safe.

End stiffener Connection:

There will be 2 weld lengths along the depth of web on each side of stiffener plate

$b_s =$ 195 mm

Tension capacity of one flat, $T_{dn} = 0.9\ A_n.f_u/\gamma_{m1} =$ 1151.28 kN

Shear per unit length $q_1 =$ 0.40538 kN/mm

Let us provide weld of size, S= 6 mm $K_s = 0.7xS =$ 4.2 mm

Strength of shop weld per unit length, $f_{wd1} =$ **0.48497423** kN/mm > 0.4053803 kN/mm

Hence provide above fillet weld to connect the end bearing stiffener to web plate.

STEPPED COLUMN DESIGN

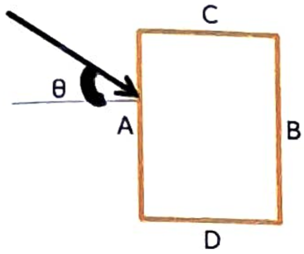
DESIGN OF GANTRY COLUMN :

Height of gantry crane from GL= 7.60 m
Eaves level from GL= 10.75 m
The gantry column has the following parts:
i) Roof leg/Upper part (Truss portion)
ii) Crane leg/ lower part (Gantry portion)

Calculation of Vertical load:
P_1= Max SF from Gantry girder= **1060** kN
P_2= Max truss reaction from DL and LL= **51.795** kN

Calculation of Horizontal load:
H_1= transverse load on gantry girder= **16** kN

Wind analysis along vertical wall of column:
Design wind pressure= **1185.482** N/m² (P_z)
For h= 10.75 m, w= 12.00 m, and L= 48.00 m,
h/w= 0.895833 (3/2>h/w>1/2) and L/w= 4 (3/2<l/w<4)
From table-4 (IS:875 (part-3) 1987), we get:

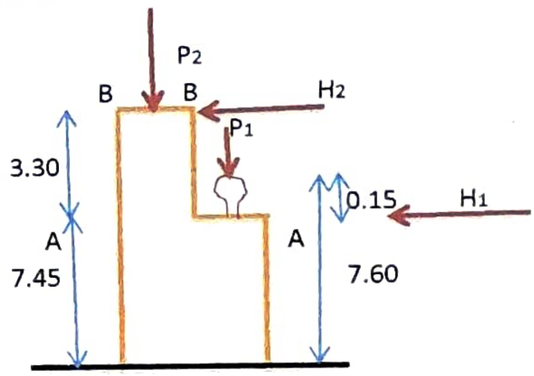


Angle (°)	C_{pe} for surface			
	A	B	C	D
0	0.7	-0.3	-0.7	-0.7
90	-0.5	-0.5	0.7	-0.1

And internal pressure coefficient for opening less than 5%,
C_{pi}= ±0.2

After combination of external and internal pressure coefficient, it is seen that the max wind will be thrust, and coefficient is= 0.9
Total wind Force, W= $(C_{pe}-C_{pi}).P_d.A$= **6.721375** kN/m (?)
H_2= Horizontal thrust from roof truss due to wind load= **8.194048** kN (=no. of sheet reqd. X W sinθ)

Calculation of trial force and BM:
At B-B level,
P_2= **51.795** kN, M_1= 36.59789 kN-m
At A-A level, P_1= **1111.795** kN
M_2= 648.4555 kN-m

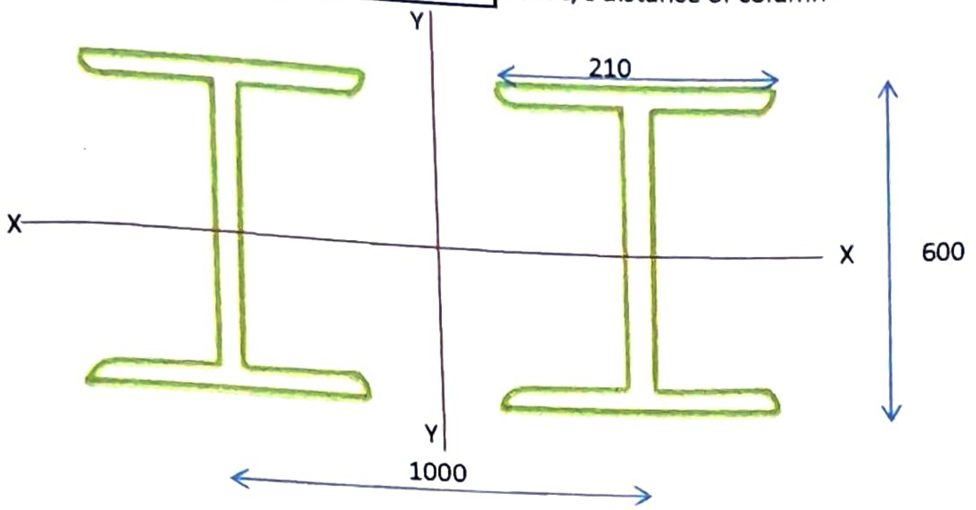


CHOICE OF SECTION :

Force per joist= $P_1/2+M_2/1.0=$ 1204.353 kN

$\sigma_{ac}=$ 100 N/mm² So, c/s area required= 12043.53 mm²

Let us choose 2 nos. ISMB 600 with c/c distance of column= 3 m



Section Properties:

A=	15621 mm²	I_{xx}=	9.18E+08 mm⁴= 91813 x10⁴ mm⁴
h=	600 mm	I_{yy}=	26510000 mm⁴
b_f=	210 mm	c/c distance of section=	1000 mm
t_f=	20.8 mm	r_{min}=	41.2 mm
t_w=	12 mm		

Effective length calculation:

Above A-A level only single secn. ISMB 600 has been provided.

Hence, $I_2=I_y=$ 26510000 mm⁴

From base level to level A-A, 2 nos. of ISMB 600 have been provided.

So, $I_1= 2(I_y+Ah^2)=$ 7.86E+09 mm⁴ = 7863.52 x10⁶mm⁴

Now $I_2/I_1=$ 0.003371 and $L_2/L_1=$ 0.442953

From table 38, IS:800-2007, $K_{12}=$ 2.43 and $K_{11}=$ 1.16

Again from table 36 of the said code, $\alpha=(P_1+P_2)/P_2=$ 22.4653

$C_1= L_2/L_1\sqrt{[I_1/(I_2.\alpha)]}=$ 1.609553

$K_1=\sqrt{[K_{12}^2+K_{11}^2.(\alpha-1)/\alpha]}=$ 1.244407

$K_2=K_1/C_1=$ 0.773138 < 3

Hence Ok.

So, effective length= $L_{e1}=K_1.L_1=$ 9.270832 m

$L_{e2}=K_2.L_2=$ 2.551356 m

Design of roof leg:

a) Check against axial compression-

λ_{eff}= L_{e2}/r_{min}= 61.92613

As per clause 7.1.2.1 of IS 800:2007,

f_{cc}= Euler buckling stress=$\pi^2E/(kL/r)^2$= **515.1473** N/mm²

$\lambda=\sqrt{(f_y/f_{cc})}$= 0.696633 =non dimensional effective slenderness ratio

As per table 10 of IS 800:2007, h/b_f= 2.857143 > 1.2

and t_f= 20.8 < 40 mm

For Y-Y axis, buckling class is 'B' and for Z-Z axis, buckling class is 'A'.

Imperfection factor, α = 0.34 for buckling class 'B',

φ = $0.5[1+\alpha(\lambda-0.2)+\lambda^2]$= 0.827077

χ=stress reduction factor= $\frac{1}{\sqrt{[\varphi+(\varphi^2-\lambda^2)]}}$ = 0.987328

So, f_{cd}= $\chi.f_y/\gamma_{mo}$= **224.3927** N/mm² < f_y/γ_{mo}= **227.2727** N/mm²

Hence Ok.

Designed compressive strength, P_d=A.f_{cd}= **3505.238** kN

P_u=1.5xP_2= **1667.693** kN < 3505.238 kN

Hence safe in axial compression.

b) Check against bending-

Section classification:- (As per table 2 of IS 800)

b/t_f= 10.09615 > 9.4ε

< 10.5ε

The section is compact in nature. So resistance in lateral torsional buckling need not be considered.

By clause 8.2.1.2 of IS 800, design bending strength, $M_d=\beta_b.Z_p.f_y/\gamma_{mo}$ (βb for compact sectn= = 1

= 122.5113 kN-m

Z_p(mm³)= 539049.6

M_u=1.5xM_1= 54.89683 kN-m < 122.5113 kN-m

Hence Ok.

c) Check against shear-

Clause 8.4.1 of IS:800 states, V_d= $A_v.f_{yw}/(\sqrt{3}\gamma_{mo})$= **1146.303** kN

V_u=1.5x(H_z+3*W)= **42.53726** kN < 1146.303 kN

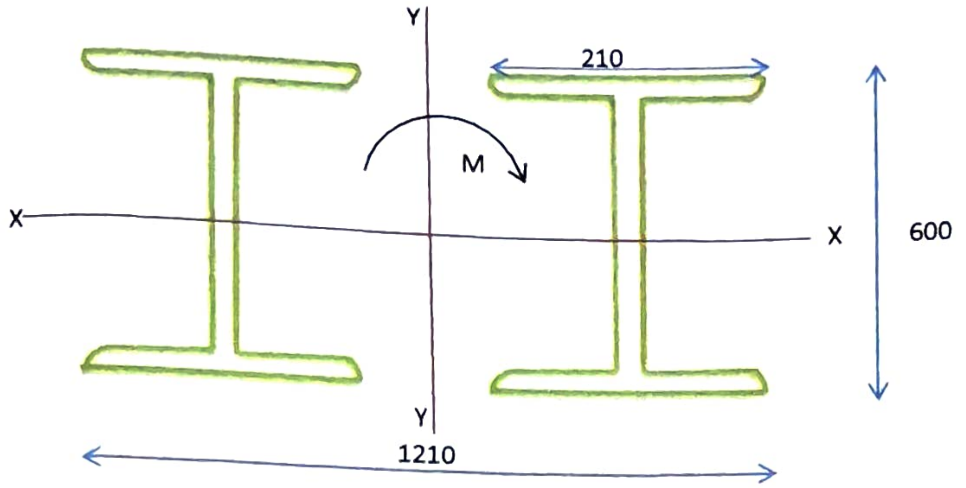
Hence Ok.

d) Combined Check-

By cl. 9.3.1, $V_d/D_u+M_y/M_{dy}+M_z/M_{dz}$= 0.485204 < 1

Hence Ok.

Design of crane leg:



P= 1111.795 kN M= 648.4555 kN-m

a) Check against axial compression-

Bending about Y-Y axis and buckling about X-axis.

$r_{x\text{-}x}$= 242.4 mm

$\lambda = L_{e1}/r_{x\text{-}x}$= 38.246

For buckling curve "C" (table 10 of IS 800) and table 9(c), f_{cd} for λ= 38.246 is=

= **201.31** N/mm²

P_d=2x Area x f_{cd}= 6289.327 kN

P_u=1.5x P_1= 1667.693 kN < 6289.327 kN

Hence Ok.

b) Check against bending-

h/t_f= 58.17308

Section classification:- (As per table 2 of IS 800)

From table 14 of IS 800, for kL/r= 30 and h/t_f= 58.17308

Critical stress, $f_{cr,b}$= 2429.506 N/mm²

From table 14 of IS 800, for kL/r= 40 and h/t_f= 58.17308

Critical stress, $f_{cr,b}$= 1373.687 N/mm²

So, for kL/r= 38.246 Critical stress, $f_{cr,b}$= 1558.877 N/mm²

α_{LT}= 0.49 (for welded omposite section)

From table 13(b) of IS:800, f_{bd}= 214.29 N/mm² (for $f_{cr,b}$= 1558.877 N/mm² and f_y= 250.00 N/mm²)

b/t_f= 58.17308 > 13.6ε

So the section is semi compact.

M_d= $\beta_b.Z_p.f_{bd}$= Z_e/Z_p x Z_p x F_{bd}= 2785.246 kN-m > 1.5 x M_2= 972.6832 kN-m

Hence Ok.

c) Interaction check-

By cl. 9.3.1, $V_d/D_u+M_y/M_{dy}+M_z/M_{dz}$= 0.614389 < 1

Hence Ok.

Provide 2 numbers ISMB 600 sections placed at a c/c distance of 1000 mm from each other. Let us provide gusset plate of size 410 x 200 x 20 mm.

DESIGN OF LACING :

Assuming the inclination of lacing bars (°)= 45 (must be 40-70)& gauge dist(mm)= 80

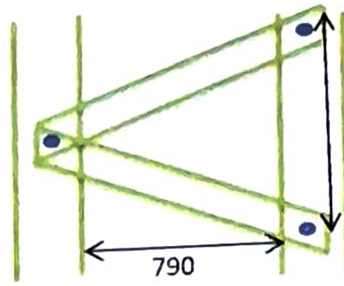
Spacing of lacing bacrs, a_1= 1900 mm

r_y of ISMB 600 is 41.2 mm

So, a_1/r_y= 46.1165 < 50

Also, a_1/r_y=should be < $0.7kL/r_z$

ie, a_1/r_y < 62.08746



Maximum shear:

V_t=2.5*(P_1+P_2)*1000/100= 27794.88 N

Transverse shear in each panel= V_t/N= 13897.44 N

Compression force in lacing bars= V_t.cosecθ/N= **19.65394** kN

Selection of lacing flats:

Let us provide 16 mm dia bolts of grade 4.6

For bolt of 16 mm dia, as per IS-800, the minimum width of lacing flat is (mm) 48

Let us provide 50 mm thk lacing flats.

Minimumm thickness(in mm) of lacing flat= 1/40 x length of flat between inner bolts= 33.58757

= 35 mm

Minimum radius of gyration, r= t/√12= 10.10363 mm

L/r= 132.9723 < 145

For, L/r= 132.9723 and f_y= 250.00 N/mm²

f_{cd}= **71.1734** N/mm²

Design compressive strength, P_d=A_e.f_{cd}= **124.5535** kN > 19.65394 kN

which is sufficient. Hence Ok.

Tensile check:

1) 0.9 (B-d_o)x t x f_u/γ_{m1}= 330.624 kN

2) A_g x f_y/γ_{mo}= 397.7273 kN

Hence 330.624 kN > 19.65394 kN

Hence Ok.

Connection:

Assuming the bolts of 16 mm dia are placed as shown, the bolts will be in double shear.

Strength of bolt in double shear:-

=2 x A_{nb} x $f_{ub}/\sqrt{3}\gamma_{mb}$= 58.01216 kN

Where, for bolts with 16 mm dia,

A_{nb}= 157 mm²

Strength of bolt in bearing:-

=2.5 K_b d t f_u/γ_{m1b}= 459.2 kN

Hence strength of bolt= 58.01216 kN

Force on bolt from lacing flat=2 x V_t.cotθ/N= 27.79488 kN

No. Of bolts required= 0.479122 = 1

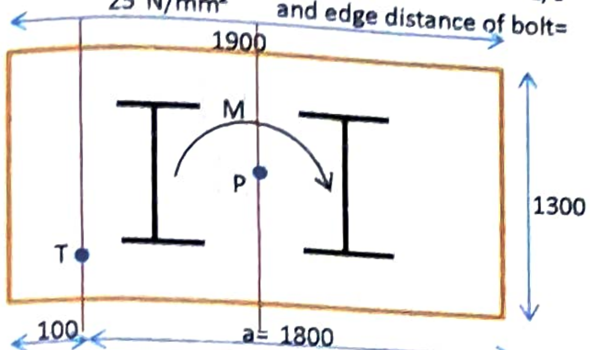
Hence provide 1 no. Of bolt of dia= 16 mm on each end of lacing flats

of size= 1345 x 50 x 35 mm

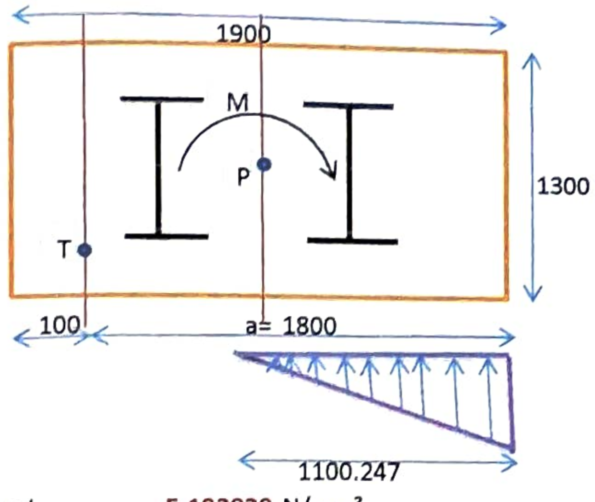
BASE PLATE DESIGN

DESIGN OF BASE PLATE :

Moment at the base , M= 648.4555 kN-m (from the design of column)
And force, P= 1111.795 kN
So, M_d=1.5M= 972.6832 kN-m and P_d=1.5P= 1667.693 kN
Length of the base plate required= 1710 mm
Let us provide length of base plate= 1900 mm
Width of base plate= 2/3 of length= 1266.667 mm
Let us provide width of base= 1300 mm
Now, eccentricity, e=M/P= 583.2509 mm
Again, L/6= 316.6667 mm So, e is > L/6
Let, f_{ck}= 25 N/mm² and edge distance of bolt= 100 mm



Let T= tension in bolt. As ΣV=0,T= 7312.5 .x - P...(i)
Again as ΣM=0 about the point where the resultant of the compressive stress diagram intersect at base level, M- P(L/2-x/3)- 7312.5 x(a-x/3)+P(a-x/3)=0..(ii)
Or, x= 201.1205 mm
So, from (i), T= -196.9991 kN
L/3(mm)= 633.3333 > 583.2509 (e)mm > 316.6667 L/6(mm)
So, most part of the base plate is in compression with little or negligible tension on the remaining part.
That is from eqn (ii),x= 1100.247 mm



f_{ck} calculated= 5.182029 N/mm²
Now, 0.45 f_{ck}/x= m'/ 345 or, m'= 0.731208 N/mm²

Moment about I-I section=M_{II}= 56.63436 kN-m
Now, M_d=1.5 . $Z_e.f_y/\gamma_{mo}$= 73863.64 d² or, d= 27.69011 mm
Provide baseplate of thickness,d= 28 mm
Provide baseplate of size= 1900 x 1300 x 28 mm

DESIGN OF BOLTS :

Considering 4.6 grade bolt, f_{yb}= 240 N/mm²

And f_{ub}= 400 N/mm²

Assuming dia of bolt, d= 30 mm, dia of hole, d_o= 33 mm

A_n= 561 mm²

Tension capacity of bolt:

From cl. 10.3.5 of IS 800:2007, T_{db}= least among T_{nb}/γ_{nb} and $f_y.A_{sb}/\gamma_{mo}$

= 0.9 $f_{ub}.A_n/\gamma_{nb}$ = 122.4 kN

= 161.568 kN

Hence, T_{db}= 122.4 kN

Bearing capacity of bolt:

$V_{dpb}=2.5K_b.d.t.f_u/\gamma_{nb}$=

= 672 kN

Where, K_b is the least among:

(let, end distance, e(mm)= 100

pitch of bolts, p(mm)= 200

$e/3d_o$= 1.010101

$p/3d_o-0.25$= 1.770202

f_{ub}/f_u= 0.97561 and 1

that is, k_b= 0.97561

Shear capacity of bolt:

$V_{dsb}=f_u \times (n_n.A_{nb}+n_s.A_{sb})/\sqrt{3}\gamma_{nb}$= 103.6459 kN

Factored SF from column= 144.6732 kN

Bolts required= 1.395841 = 4 nos.

Let us provide 4 nos. of 30 mm dia bolt on each side.

Combined check:

Now, factore SF per bolt, V_{sb}= 36.16831 kN

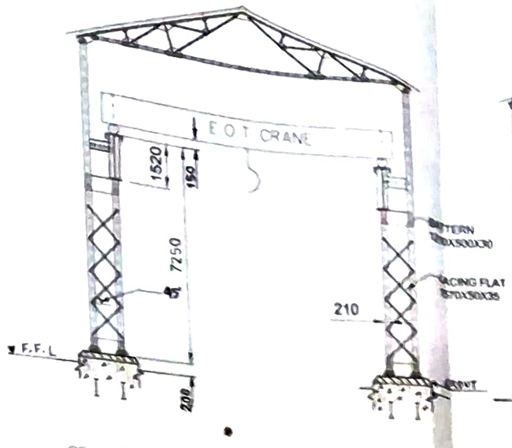
Factored tensile force per bolt, T_b= 0 kN

So, $(V_{sb}/V_{dsb})^2+(T_b/T_{db})^2$= 0.121773 $<$ 1

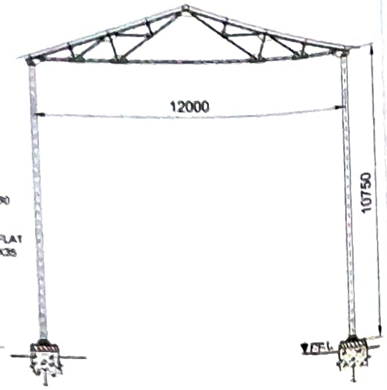
Hence Ok.

Let us provide 4 nos. of 30 mm dia bolt of grade 4.6 each side

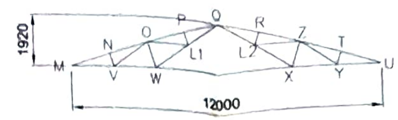
DRAWING



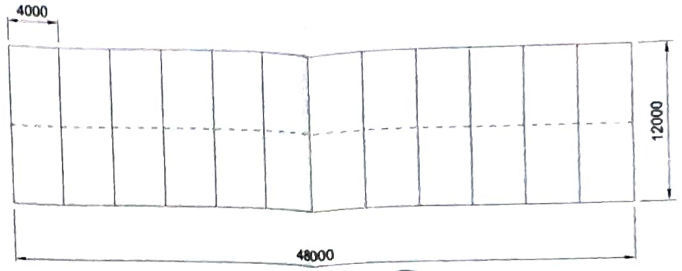
SECTION OF TRUSS WITH ROOF COLUMN AND GANTRY COLUMN
SCALE 1:100



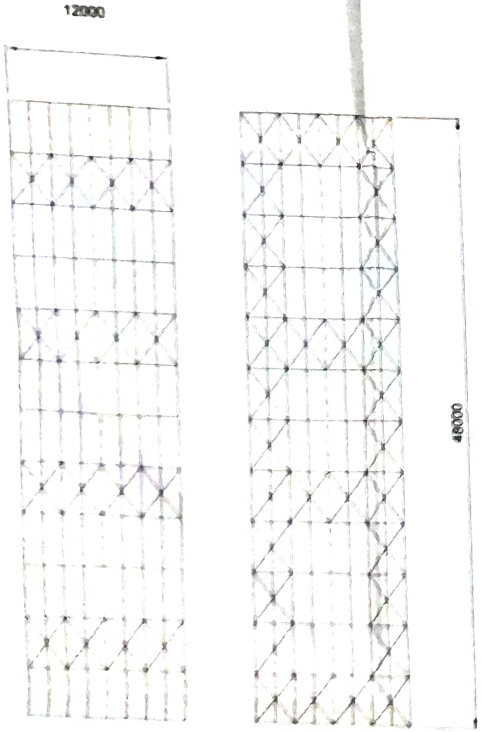
SECTION OF TRUSS WITH ROOF COLUMN
SCALE 1:100



LINE DIAGRAM OF TRUSS
SCALE 1:100

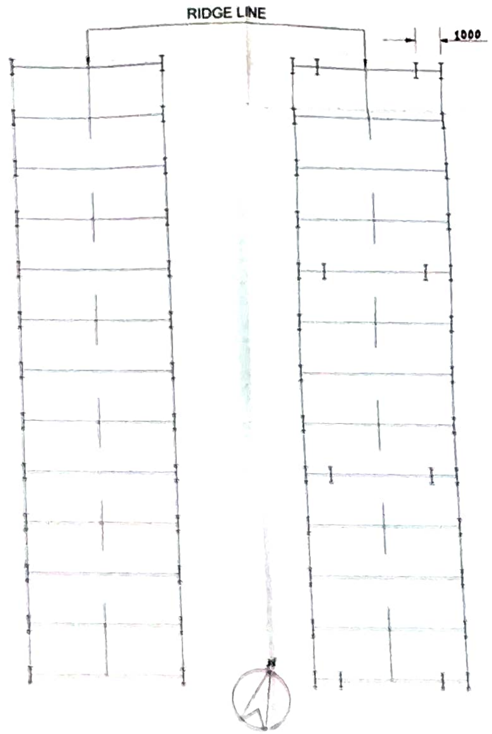


GRID PLAN
SCALE 1:200



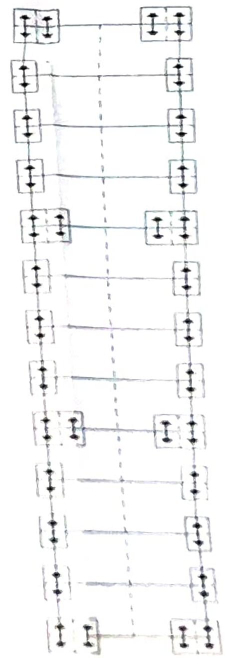
BRACING AT RAFTER LEVEL
SCALE 1:200

BRACING AT EAVES LEVEL
SCALE 1:200



COLUMN PLAN AT ROOF LEVEL
SCALE 1:200
SCALE OF COLUMN 1:100

COLUMN PLAN AT GANTRY LEVEL
SCALE 1:200
SCALE OF COLUMN 1:100

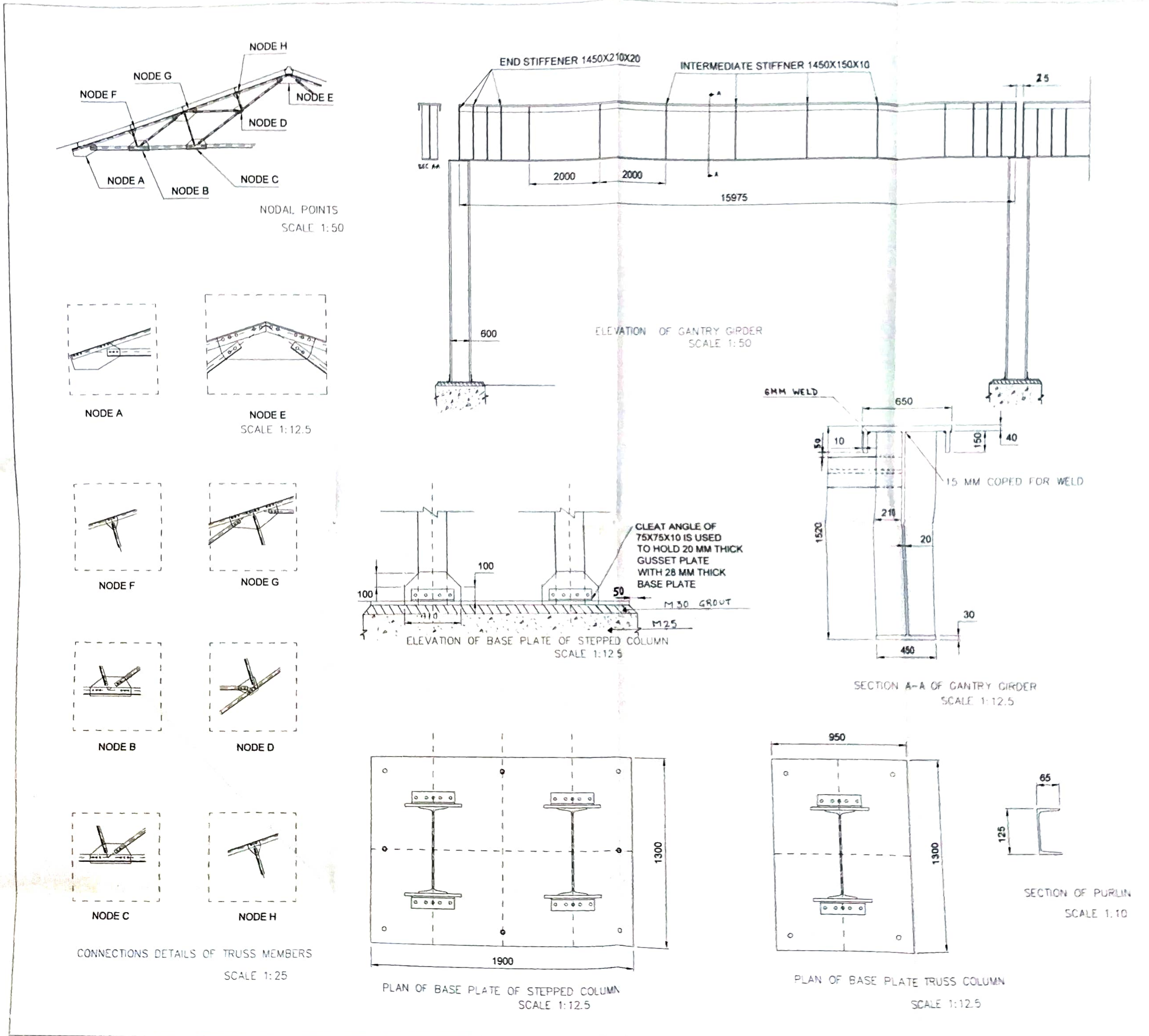


PLAN AT BASE PLATE LEVEL
SCALE 1:200
SCALE OF BASE PLATE 1:100

NOTES

1) ALL DIMENSIONS ARE IN MM
2) SPAN OF TRUSS 12000
3) LENGTH OF FACTORY SHED 48000
4) SPACING OF TRUSS 4000
5) SPACING OF GANTRY COLUMN 16000
6) HEIGHT OF COLUMN 10750
7) HEIGHT OF RAIL LEVEL 7600
8) PITCH OF TRUSS 0.16
9) CRANE CAPACITY 500KN
10) GRADE OF STEEL USED FE 410
11) FOR MORE DETAILS REFER TO SHEET NO 2

CALCUTTA INSTITUTE OF ENGINEERING AND MANAGEMENT	
DEPARTMENT OF CIVIL ENGINEERING	
WORK	LAYOUT PLAN OF FACTORY SHED AT DIFFERENT LEVELS SECTIONS OF FACTORY SHED
NAME	ATASI ROY MALAKAR
ROLL	16501311[illegible]
DATE	OF SUBMISSION [illegible]



NOTES

CONNECTION DETAILS

MEMBER	DISTANCE			
	EDGE	END	PITCH	[illegible]
RAFTER ISA 65X45X8	40	40	50	20
TIE ISA 65X45X8	40	40	50	20
STRUT ISA 45X45X5	20	25	50	12
SLING ISA 65X50X4	25	40	50	20
SECONDARY ISA 65X50X4	25	40	50	20

1) ALL DIMENSIONS ARE IN MM
2) ISMB 600 COLUMN IS USED
3) PURLIN OF ISMC 125 IS USED
4) GRADE OF BOLTS USED 4.6
5) GI SHEETS OF WT 131.7 N/M2
6) FOR DETAILS REFER TO SHEET NO 1
7) STEEL OF GRADE FE 410 IS USED
8) BASE PLATE DIM 1900X1300X28
9) DETAILS OF GANTRY

MEMBER	HEIGHT/DEPTH	THICKNESS
TOP FLANGE	650	40
BOTTOM FLANGE	450	30
HANGED PORTION	150	30
WEB	1450	20

HANGED PORTION IS FIXED 10MM FROM OUTER END

STIFFENERS ARE COPED 15 MM FOR FILLET WELD

10) 8 HOLDING DOWN BOLTS OF 30MM ARE USED

CALCUTTA INSTITUTE OF ENGINEERING AND MANAGEMENT

DEPARTMENT OF CIVIL ENGINEERING

WORK	LAYOUT PLAN OF FACTORY SHED AT DIFFERENT LEVELS, CONNECTION DETAILS SHEET 2		
NAME	ATASI ROY MALAKAR	ROLL	16501511014
DATE OF SUBMISSION	[illegible]	SIGN	[signature]

REFERENCE

1 IS:800-2007
2 SP:6(1) 1964
3 IS:875(Part3)-1987
4 SP:38
5 Limit state design of Steel Structures (S K Duggal)

THANK YOU