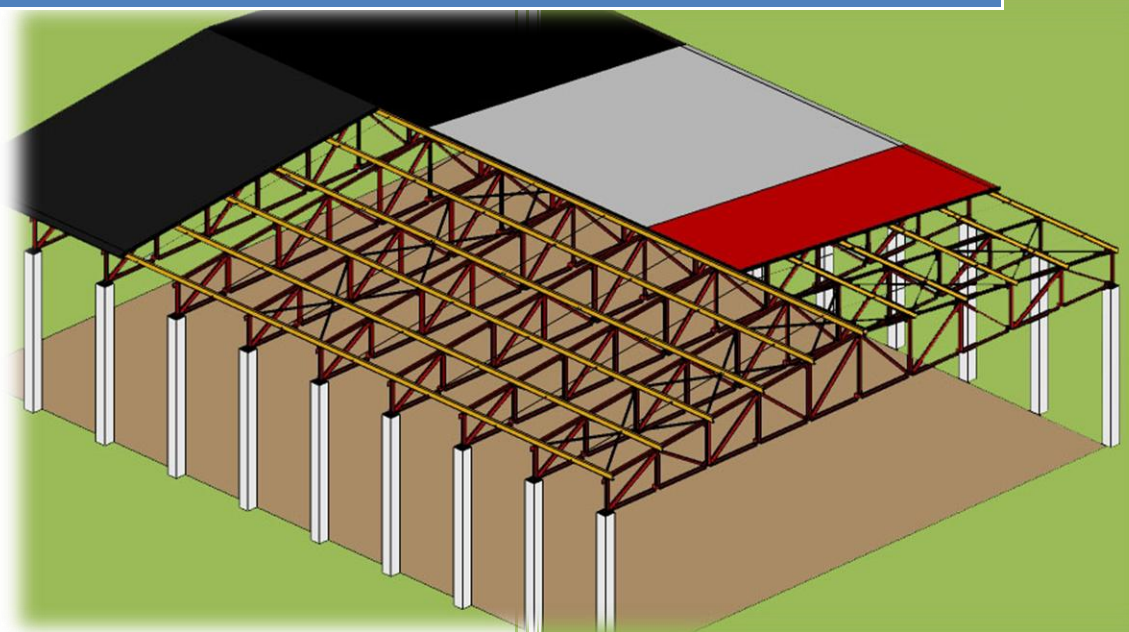
2011

STEEL STRUCTURAL PROJECT (ROOF TRUSS)



تحت اشرف/

د/ نبيل فلاح

د/ سليمان الصافي

سمير محمد محمد الصياد 2008/141

سليمان علي صالح مجيديع 2008/407

عبدالرحمن يحيى الحبابي 2008/82

عمار غانم سليمان مسعود 2008/173

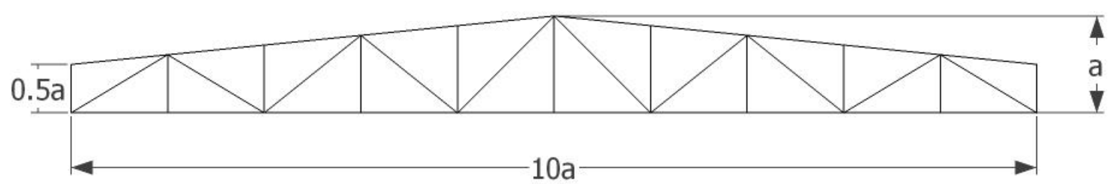
عادل علي حسن هرمس 2008/221

The project

A garage consist of (n=10) trusses, (type as has been selected for each group of students), each spaced (s) m. The roof trusses are supported by concrete columns 10 x a m apart. Estimate the dead load on the truss and use snow load of 1.2 kN/m^2 of horizontal projection and wind load of 0.5 kN/m^2 on leeward side and 0.25 kN/m^2 on the windward side and roofing of 1.0 kN/m^2.

<u>Design Data</u>

1. A36 steel.
2. Type of connections: welded for left hand bolted connections for right hand side half. (assume all connection are bolted for member design)



<u>Requirement</u>

1. Draw to scale 1:100 general arrangements showing all structural elements.
2. Calculate different load acting on an intermediate truss.
3. Determine the design force in each member (table format).
4. Design each section (Find suitable section for chord and web members).
5. Design half of the joints as welded connections and the other as bolted connections.
6. Study the system of bracing for the building.

For our project:

s = 4m, a = 2.8m

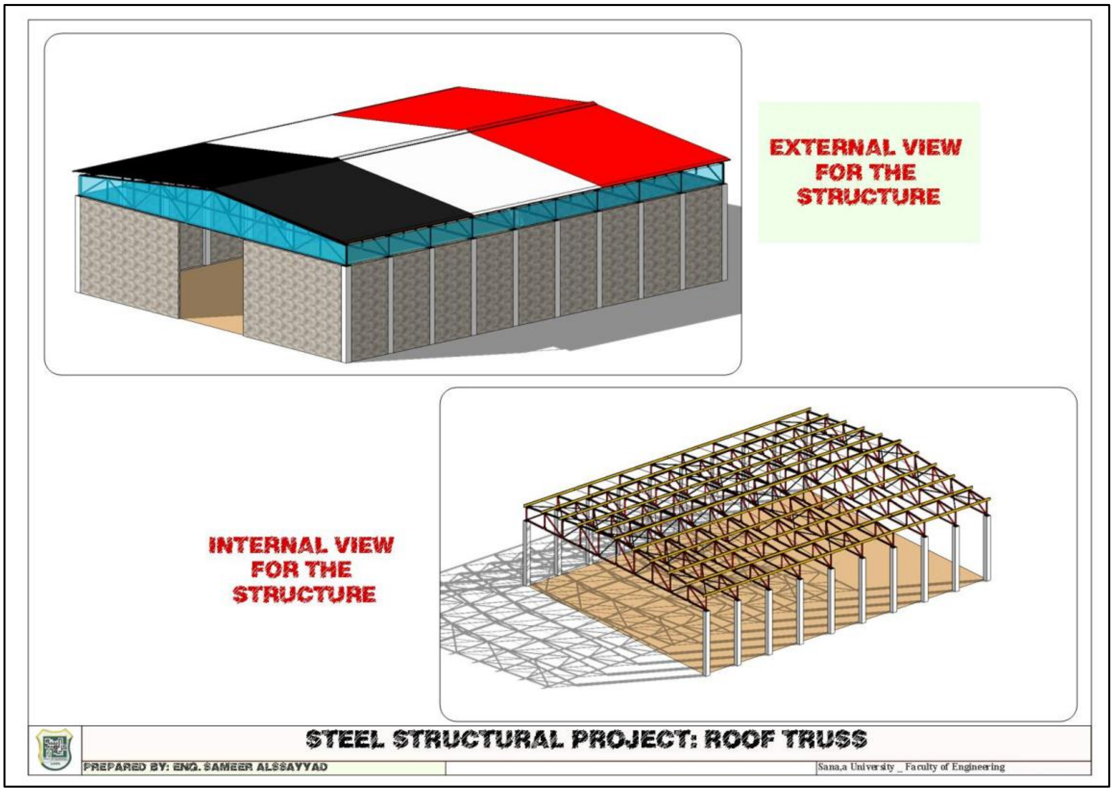

EXTERNAL VIEW FOR THE STRUCTURE
INTERNAL VIEW FOR THE STRUCTURE
STEEL STRUCTURAL PROJECT: ROOF TRUSS
PREPARED BY: ENG. SAMEER ALSSAYYAD
Sana,a University _ Faculty of Engineering

Fig. (1)

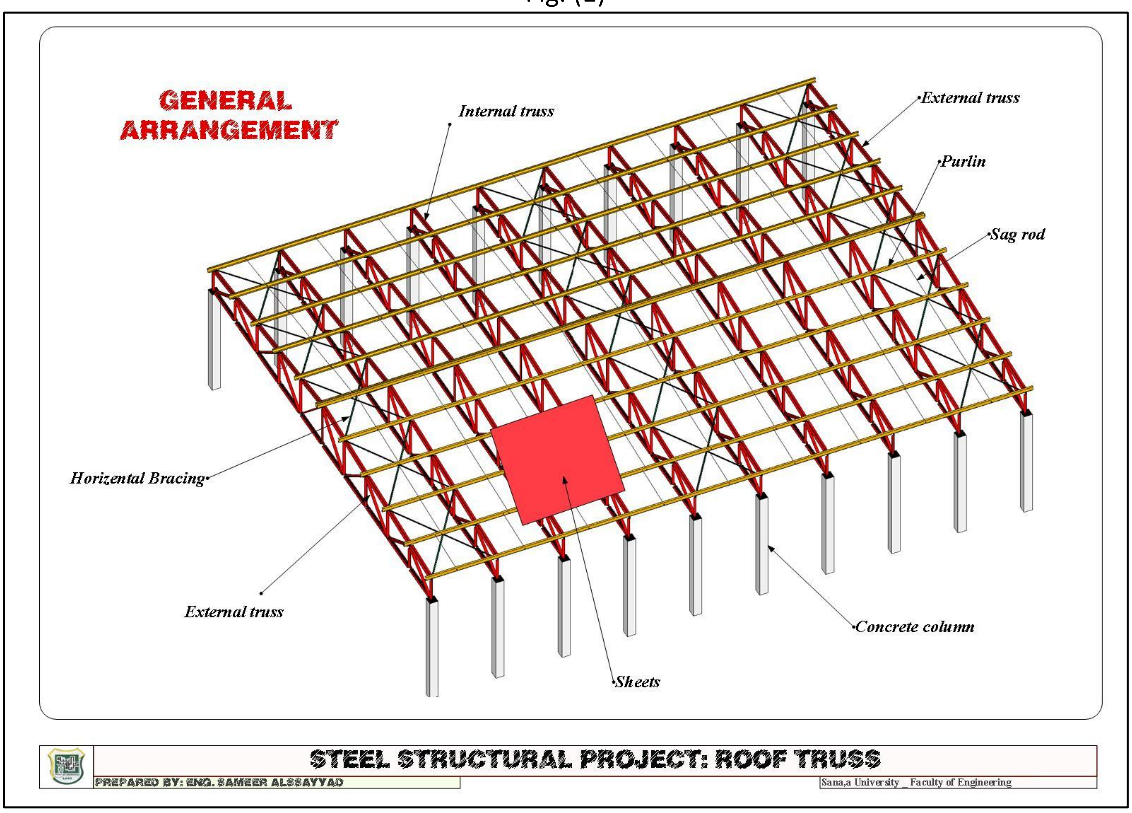

GENERAL ARRANGEMENT
Internal truss
External truss
Purlin
Sag rod
Horizental Bracing
External truss
Concrete column
Sheets
STEEL STRUCTURAL PROJECT: ROOF TRUSS
PREPARED BY: ENG. SAMEER ALSSAYYAD
Sana,a University _ Faculty of Engineering

Fig. (2)

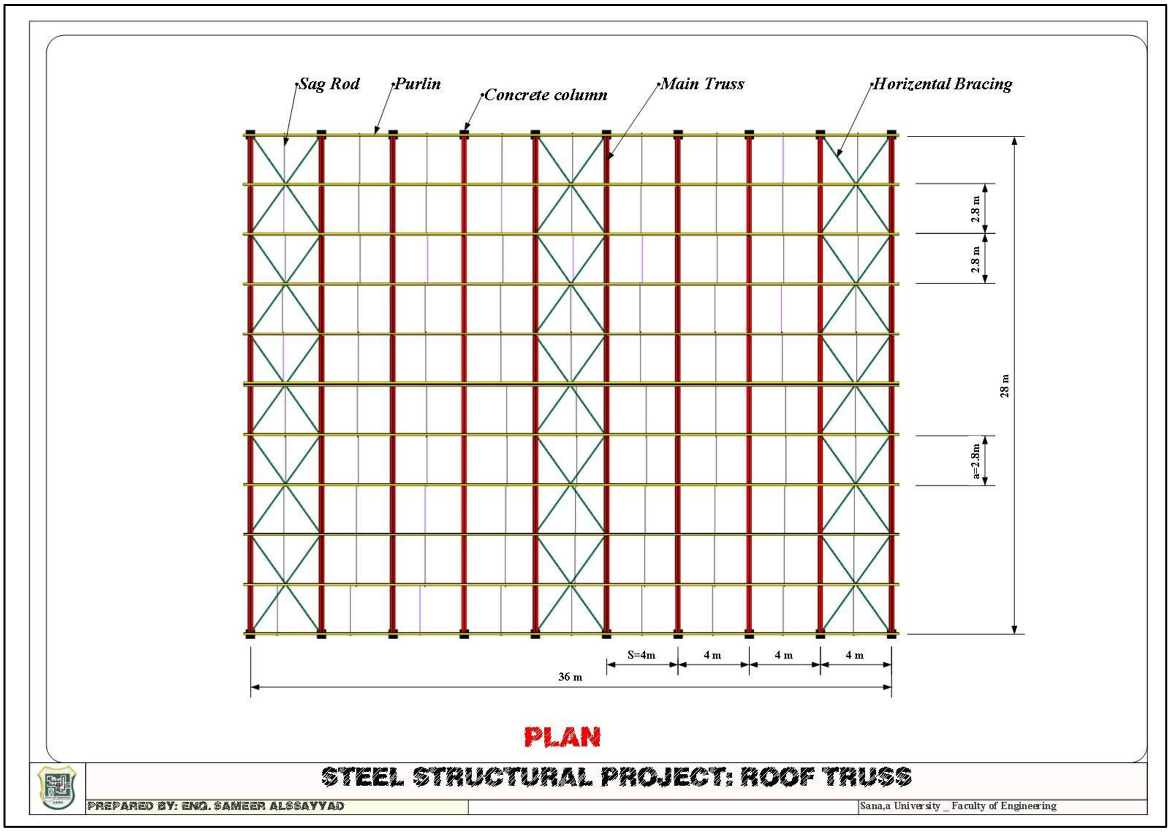

Sag Rod
Purlin
Concrete column
Main Truss
Horizental Bracing
2.8 m
2.8 m
28 m
a=2.8m
S=4m
4 m
4 m
4 m
36 m
PLAN
STEEL STRUCTURAL PROJECT: ROOF TRUSS
PREPARED BY: ENG. SAMEER ALSSAYYAD
Sana,a University _ Faculty of Engineering

Fig. (3)

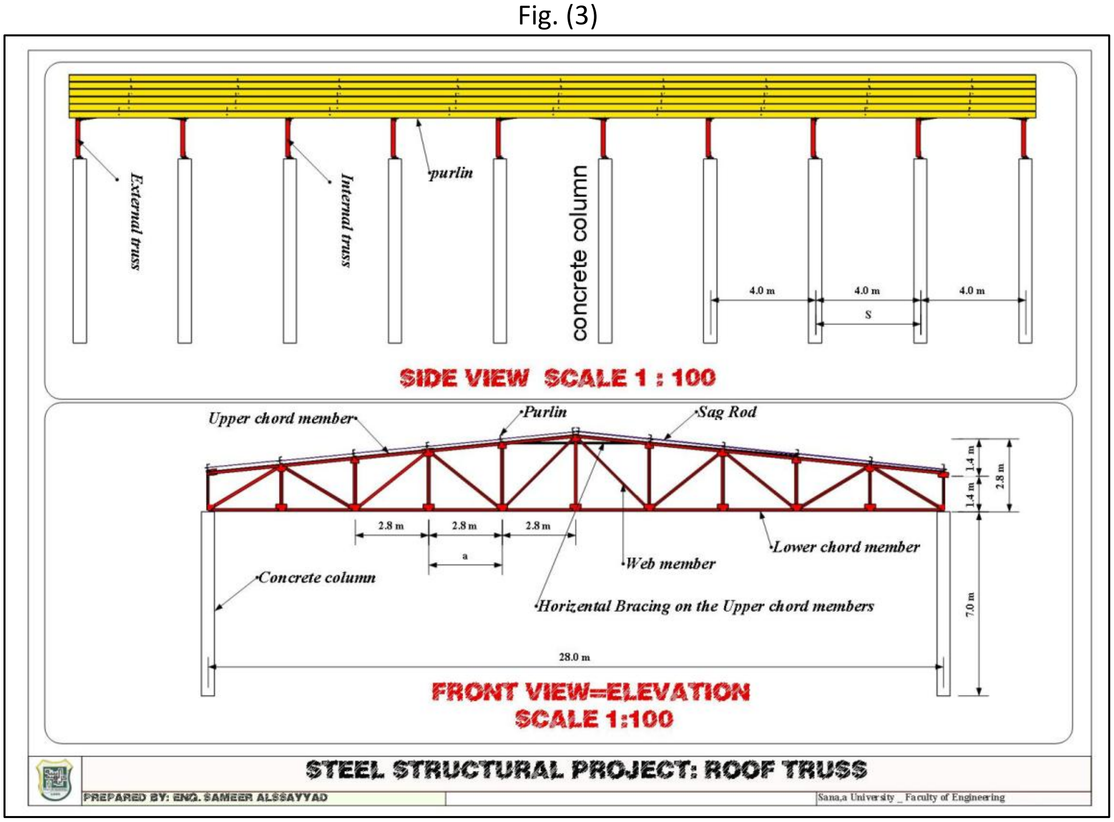

External truss
Internal truss
purlin
concrete column
4.0 m
4.0 m
4.0 m
S
SIDE VIEW SCALE 1 : 100
Upper chord member
Purlin
Sag Rod
1.4 m
1.4 m
2.8 m
2.8 m
2.8 m
2.8 m
a
Web member
Lower chord member
Concrete column
Horizental Bracing on the Upper chord members
7.0 m
28.0 m
FRONT VIEW=ELEVATION
SCALE 1:100
STEEL STRUCTURAL PROJECT: ROOF TRUSS
PREPARED BY: ENG. SAMEER ALSSAYYAD
Sana,a University _ Faculty of Engineering

Fig.(4)

From figure 1 to figure 4 are show the general arrangement of the building.

Calculation of loads acting on an internal truss

Live load

From the ASCE 7-05 table (4-1) we take LL = 1 kN\m^2 which is on the horizontal projection.

Then LL = 1 cosΘ = 0.995 kN\m^2 (on the inclined length and Θ = 5.710593137^o)

Load on internal nod in a typical internal truss equal 0.995 * 4 * 2.8 = 11.144 kN\m^2

Snow load

As it given in our project 1.2 kN\m^2 on the horizontal projection

Then SL = 1.2 cosΘ = 1.194 kN\m^2 (on the inclined length and Θ = 5.710593137^o)

Load on internal nod in a typical internal truss equal 1.194 * 4 * 2.8 = 13.373kN\m^2

Estimating Dead load

The weight of the roof truss and its bracing is taken approximately 10% of the loading it is required to support.

We will take it due the live and snow loads as shown in the table

NOTE

The unit load analysis will use for calculate the live, snow and dead loads internal forces.

Analysis of the truss using Joints method due to unit load

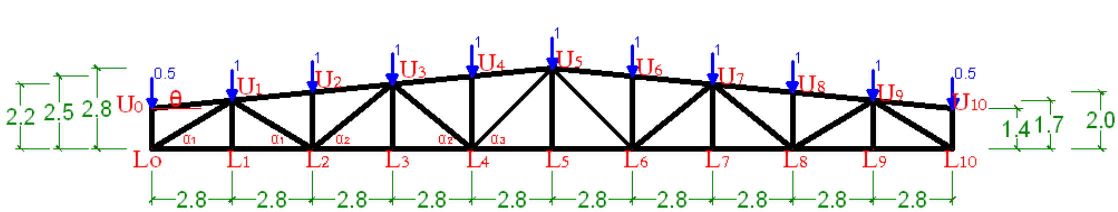
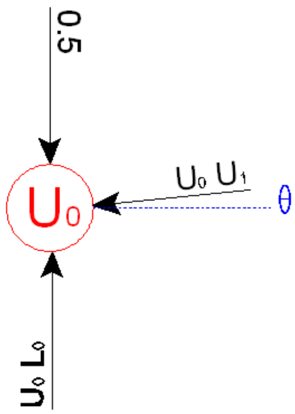


Fig. (5) Names of Joints and truss geometry



$\sum F_x = 0$

$U_0 U_1 \cos\Theta = 0$

$U_0 U_1 = 0$

$\sum F_y = 0$

$U_0 L_0 = 0.5$

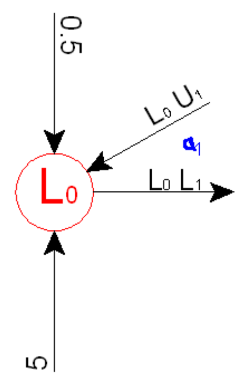
$\sum F_y = 0$

$0.5 + L_0 U_1 \sin\alpha_1 = 5$

$L_0 U_1 = 8.746427842$

$\sum F_x = 0$

$L_0 L_1 = \cos\alpha_1 = 7.5$

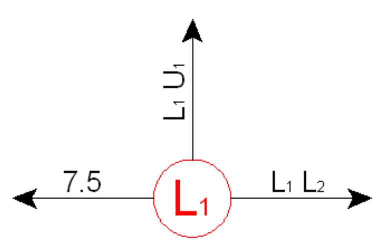


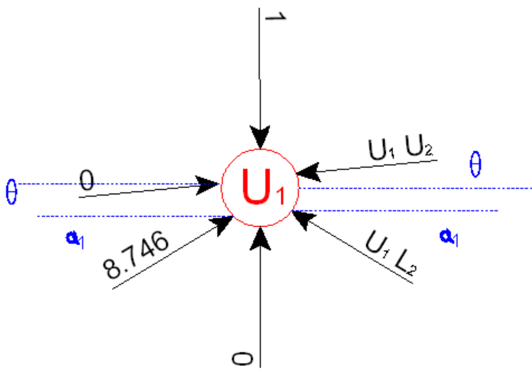
$\sum F_x = 0$

$L_1 L_2 = 7.5$

$\sum F_y = 0$

$L_1 U_1 = 0$





$\sum F_y = 0$

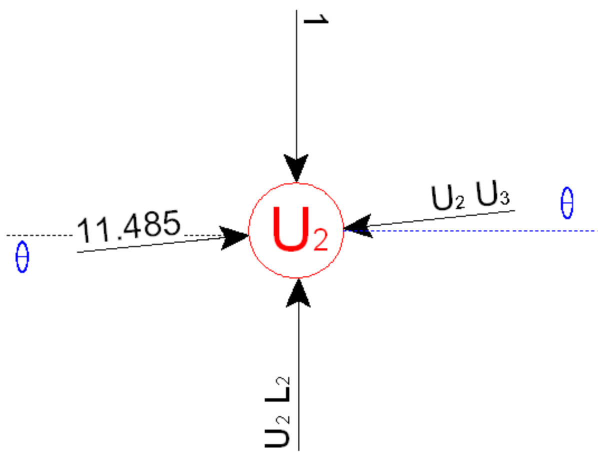
$1 + U_1 U_2 \sin\Theta = 8.746 \sin \alpha_1 + U_1 L_2 \sin \alpha_1$

$\sum F_y = 0$

$8.746 \cos \alpha_1 = U_1 L_2 \cos \alpha_1 + U_1 U_2 \cos\Theta$

$U_1 U_2 = 11.48557214$

$U_1 L_2 = -4.581462203$



$\sum F_x = 0$

$U_2 U_3 = 11.48557214$

$\sum F_y = 0$

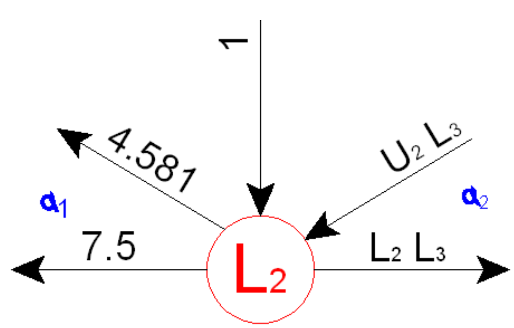
$U_2 L_2 = 1$

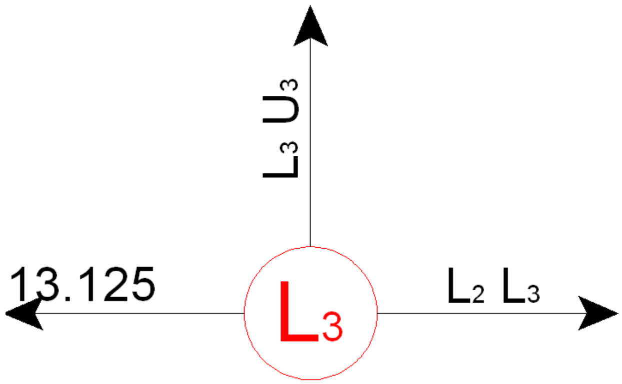
$\sum F_y = 0$ ➔ $1 + L_2 U_3 \sin\alpha_2 = 4.581 \sin\alpha_1$

$L_2 U_3 = 2.17248858$

$\sum F_x = 0$ ➔ $7.5 + 4.581 \cos \alpha_1 + L_2 U_3\ \mathrm{s}\cos\alpha_2 = L_2 L_3$

$L_2 L_3 = 13.125$





$\sum F_x = 0$

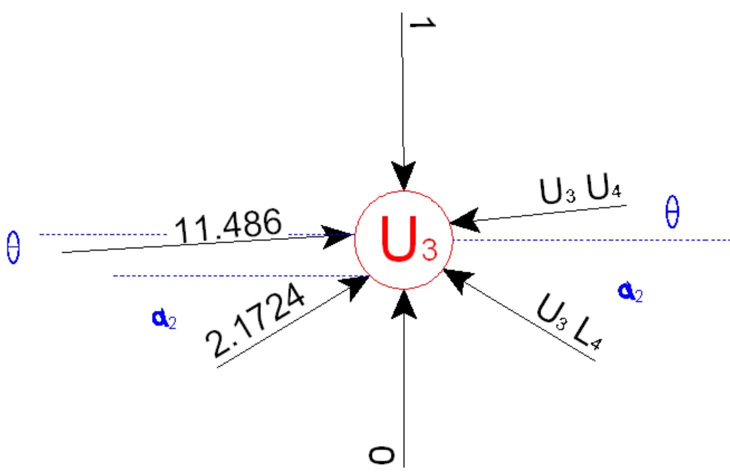
$L_3 L_4 = 13.125$

$\sum F_y = 0$

$L_3 U_3 = 0$

$\sum F_x = 0$

$U_3 U_4 \cos\Theta + U_3 L_4 \cos\alpha_2 = 11.4855 \cos\Theta + 2.1724 \cos \alpha_2$

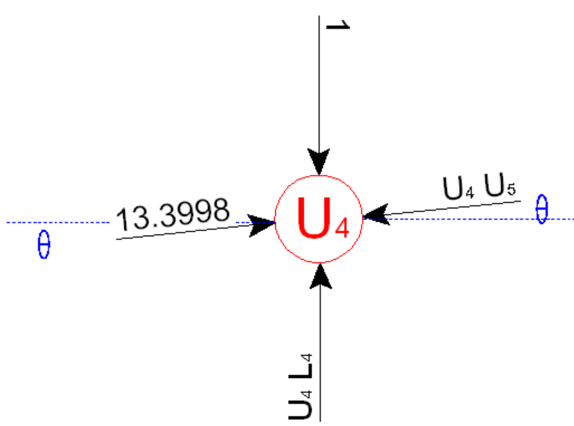


$\sum F_y = 0$

$1 + U_3 U_4 \sin\Theta = 2.175 \sin \alpha_2$
$+ U_3 L_4 \sin\alpha_2 + 11.4855 \sin\Theta$

$U_3 U_4 = 13.39983416$

$U_3 L_4 = -0.2667968432$

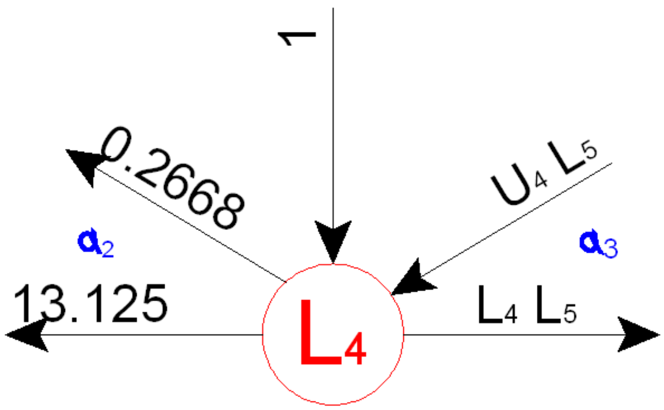


$\sum F_x = 0$

$U_4 U_5 = 13.39983416$

$\sum F_y = 0$

$U_4 L_4$



$\sum F_y = 0$

$1 + L_4 U_5 \sin \alpha_3 =$

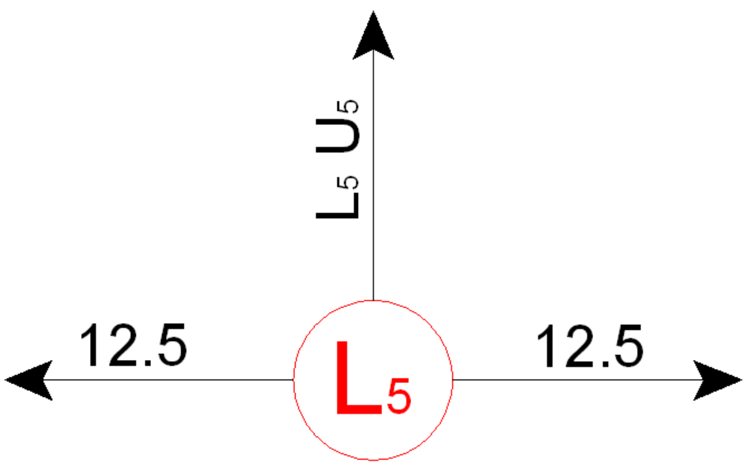
$0.266797 \sin \alpha_2$

$L_4 U_5 = -1.178511302$

$\sum F_x = 0$

$13.125 + 1.266 \cos \alpha_2 + L_4 U_5 \cos \alpha_3 = L_4 L_5$

$L_4 L_5 = 12.5$



$\sum F_y = 0$

$L_5 U_5 = 0$

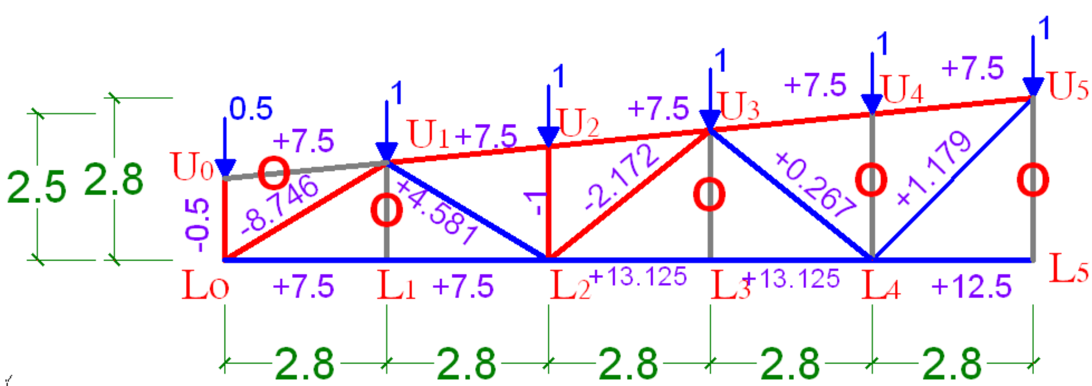
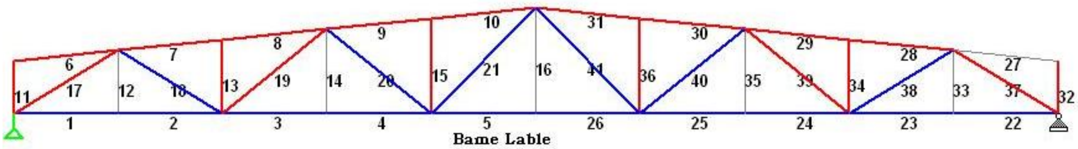


Fig. (6) Internal forces.

Fig. (7) Beams labels.



Calculation of Wind Load

For the static approach, the fluctuating pressure caused by a constantly blowing wind is approximated by a mean velocity pressure that acts on the structure. This pressure q is defined by its kinetic energy, q =P/ V2,

Where = p is the density of the air and V is its velocity. According to the ASCE 7-05 Standard, this equation is modified to account for the importance of the structure, its height, and the terrain in which it is located. It is represented as

$$\boxed{q_z = 0.613\ K_z\ K_{zt}\ K_d\ V^2\ I\ \ (N\backslash m^2}$$ (ASCE 7-05 eq. 6-15)

Where,

V = the velocity in m/s of a 3-second gust of wind measured 10 m above the ground during a 50-year recurrence period. Values are obtained from H. Althaferee wind map of Yemen.

I = the importance factor that depends upon the nature of the building occupancy; Values are obtained from ASCE 7-05 table (6-1)

K_z = the velocity pressure exposure coefficient, which is a function of height and depends upon the ground terrain. Hibbeler 7th edition Table 1-5 lists values for a structure which is located in open terrain with scattered low-lying obstructions.

K_{zt} = a factor that accounts for wind speed increases due to hills and escarpments. For fiat ground K_{zt} = 1.

K_d = a factor that accounts for the direction of the wind. It is used only when the structure is subjected to combinations of loads Values are obtained from ASCE 7-05 table (6-4)

Table of Velocity pressure Exposure Coefficient for Terrain with Low-Lying Obstructions	
z (m)	K_z
0 – 4.6	0.85
6.1	0.90
7.6	0.94
9.1	0.98
12.2	1.04
15.2	1.09

Design Wind Pressure for Enclosed Buildings. Once the value for qz is obtained, the design pressure can be determined from a list of relevant equations listed in the ASCE 7-05 Standard. The choice depends upon the flexibility and height of the structure, and whether the design is for the main wind-force resisting system, or for the building's components and cladding. For example, for a conservative design wind-pressure on nonflexible buildings of any height is determined using a two-termed equation -resulting from both external and internal pressures, namely,

$$\boxed{P = q\,G\,C_p - q_h\,(GC_{pi})} \qquad \text{(ASCE 7-05 eq. 6-19)}$$

Here

$q = q_z$ for the windward wall at height z above the ground (last Eq.), and q = qh for the leeward walls, side walls, and roof, where z = h, the mean height of the roof.

G = a wind-gust effect factor, which depends upon the exposure. For example, for a rigid structure,

G = 0.85.

C_p = a wall or roof pressure coefficient determined from ASCE 7-05 table (6-1).

(GC_{pi}) = the internal pressure coefficient which depends upon the type of openings in the building. For fully enclosed buildings $(GC_{pi}) = \pm 0.18$ ASCE 7-05 figure (6 – 5). Here the signs indicate that either positive or negative (suction) pressure can occur within the building.

Table (1)

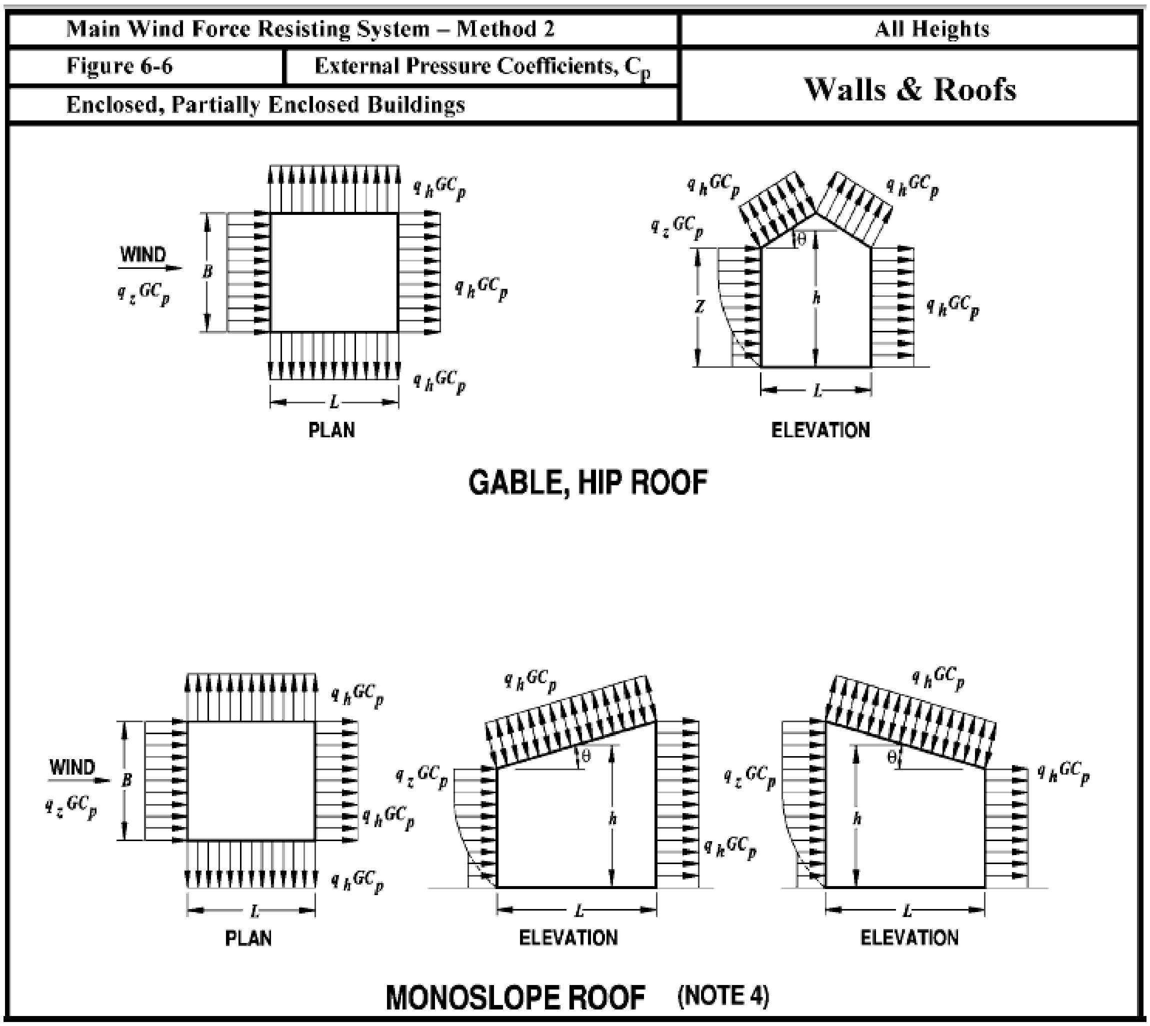


Table (2)

Main Wind Force Resisting System – Method 2		All Heights
Figure 6-6 (con't)	External Pressure Coefficients, C_p	Walls & Roofs
Enclosed, Partially Enclosed Buildings		

Wall Pressure Coefficients, C_p

Surface	L/B	C_p	Use With
Windward Wall	All values	0.8	q_z
Leeward Wall	0-1	-0.5	q_h
	2	-0.3	
	≥4	-0.2	
Side Wall	All values	-0.7	q_h

Roof Pressure Coefficients, C_p, for use with q_h

Wind Direction		Windward								Leeward		
		Angle, θ (degrees)								Angle, θ (degrees)		
	h/L	10	15	20	25	30	35	45	≥60#	10	15	≥20
Normal to ridge for θ ≥ 10°	≤0.25	-0.7 -0.18	-0.5 0.0*	-0.3 0.2	-0.2 0.3	-0.2 0.3	0.0* 0.4	0.4	0.01 θ	-0.3	-0.5	-0.6
	0.5	-0.9 -0.18	-0.7 -0.18	-0.4 0.0*	-0.3 0.2	-0.2 0.2	-0.2 0.3	0.0* 0.4	0.01 θ	-0.5	-0.5	-0.6
	≥1.0	-1.3** -0.18	-1.0 -0.18	-0.7 -0.18	-0.5 0.0*	-0.3 0.2	-0.2 0.2	0.0* 0.3	0.01 θ	-0.7	-0.6	-0.6

Wind Direction	h/L	Horiz distance from windward edge	C_p
Normal to ridge for θ < 10 and Parallel to ridge for all θ	≤ 0.5	0 to h/2	-0.9, -0.18
		h/2 to h	-0.9, -0.18
		h to 2 h	-0.5, -0.18
		> 2h	-0.3, -0.18
	≥ 1.0	0 to h/2	-1.3**, -0.18
		> h/2	-0.7, -0.18

*Value is provided for interpolation purposes.

**Value can be reduced linearly with area over which it is applicable as follows

Area (sq ft)	Reduction Factor
≤ 100 (9.3 sq m)	1.0
200 (23.2 sq m)	0.9
≥ 1000 (92.9 sq m)	0.8

Notes:

1. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
2. Linear interpolation is permitted for values of *L/B*, *h/L* and θ other than shown. Interpolation shall only be carried out between values of the same sign. Where no value of the same sign is given, assume 0.0 for interpolation purposes.
3. Where two values of C_p are listed, this indicates that the windward roof slope is subjected to either positive or negative pressures and the roof structure shall be designed for both conditions. Interpolation for intermediate ratios of h/L in this case shall only be carried out between C_p values of like sign.
4. For monoslope roofs, entire roof surface is either a windward or leeward surface.
5. For flexible buildings use appropriate G_f as determined by Section 6.5.8.
6. Refer to Figure 6-7 for domes and Figure 6-8 for arched roofs.
7. Notation:
   B: Horizontal dimension of building, in feet (meter), measured normal to wind direction.
   L: Horizontal dimension of building, in feet (meter), measured parallel to wind direction.
   h: Mean roof height in feet (meters), except that eave height shall be used for θ ≤ 10 degrees.
   z: Height above ground, in feet (meters).
   G: Gust effect factor.
   q_z, q_h: Velocity pressure, in pounds per square foot (N/m^2), evaluated at respective height.
   θ: Angle of plane of roof from horizontal, in degrees.
8. For mansard roofs, the top horizontal surface and leeward inclined surface shall be treated as leeward surfaces from the table.
9. Except for MWFRS's at the roof consisting of moment resisting frames, the total horizontal shear shall not be less than that determined by neglecting wind forces on roof surfaces.

#For roof slopes greater than 80°, use C_p = 0.8

For our project

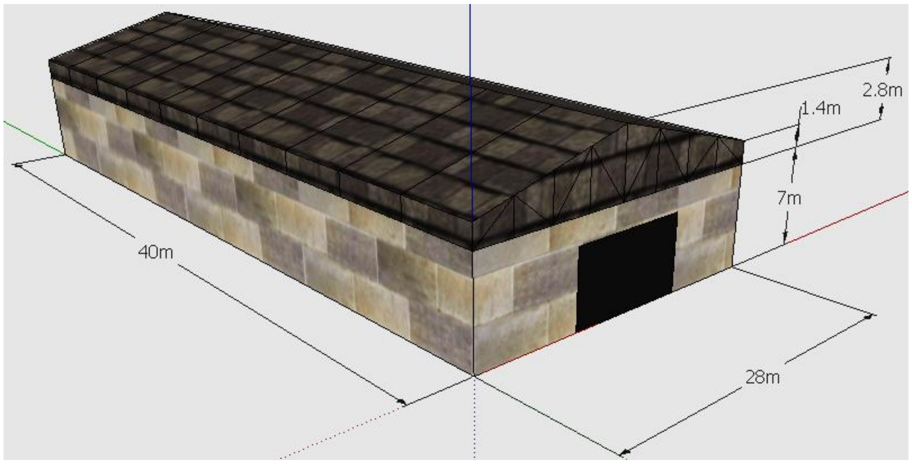


Fig. (8)

V = 95 km/hr or 26.389 m/s
I = 1.0 from ASCE 7-05 table 6-1 class 2
K_{zt} = 1.0 assuming flat ground
K_d = 0.85 from ASCE 7-05 table 6-4

$$\boxed{q_z = 0.613\ K_z\ K_{zt}\ K_d\ V^2\ I\ \ (N\backslash m^2)}$$ (ASCE 7-05 eq. 6-15)

$q_z = 0.613*K_z*1.0*0.85*26.389^2*1.0$
$= 362.845*K_z$
h = 7m + 2.8m (because Θ < 10 deg.)

Table (3)

Z (m)	K_z	q_z (N/m^2)
0-4.6	0.85	308.42
6.1	0.9	326.56
7.6	0.94	341.07
9.1	0.98	355.59
h=9.8	0.999	362.36

$$P = q\ G\ C_p - q_h\ (GC_{pi})$$
$$= q*0.85*C_p - 362.36(-+0.18)$$
$$= 0.85*q*C_p\ -\&+\ 65.22$$

Windward Wall

From z=7m to 8.4m (C_p=0.8 , q =q_z)
$q_{7\text{-}8.4}$ = 166.71 N/m^2 or 297.15 N/m^2

Leeward Wall

L/B = 28/40 = 0.7

then C_p= -0.5 , q =q_h
p = -219.22 N/m^2 or -88.78 N/m^2

Roof

according to Θ=5.7106 <10s

First value:

$C_p = -0.9$ for distance 0 to h
$C_p = -0.5$ for distance h to $2h$
$C_p = -0.3$ for distance $>2h$

Second value:

$C_p = -0.18$ for distance 0 to end.

When C_p =-0.9 then p = -342.43 N/m^2 or -211.99 N/m^2
When C_p =-0.5 then p = -219.22N/m^2 or -88.78 N/m^2
When C_p =-0.3 then p = -157.62 N/m^2 or -27.18 N/m^2
When C_p =-0.18 then p = -120.66 N/m^2 or 9.77 N/m^2

The load cases are showing in figure 9.

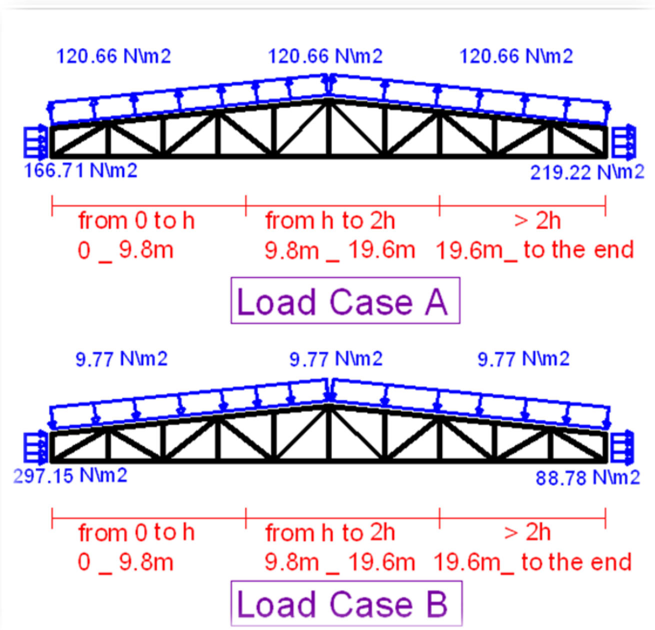


Fig. (9)

Here we will apply the loads for both cases in the two directions (from the left and from the right) as shown here.

Load case A

Applying wind load from the left as showing in figure 10

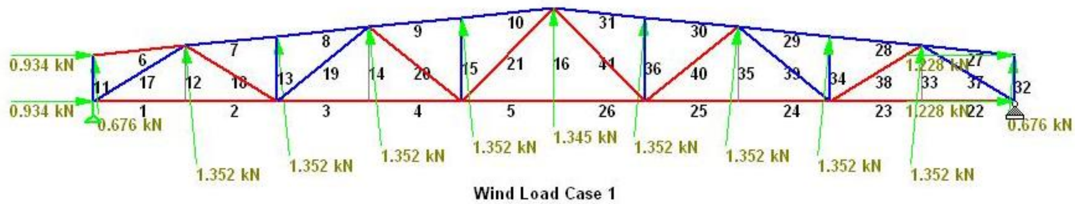


Fig. (10)

Applying wind load from the right as showing in figure 11

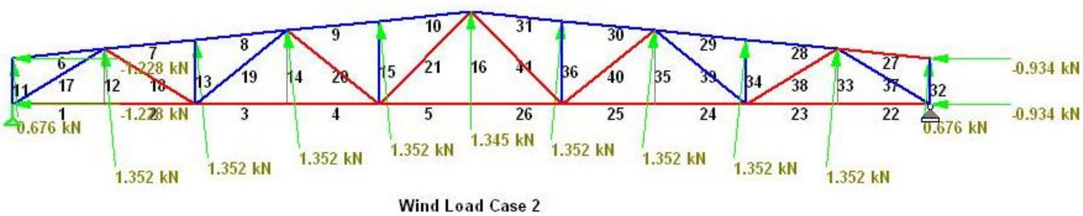


Fig. (11)

Load case B

Applying wind load from the left as showing in figure 12

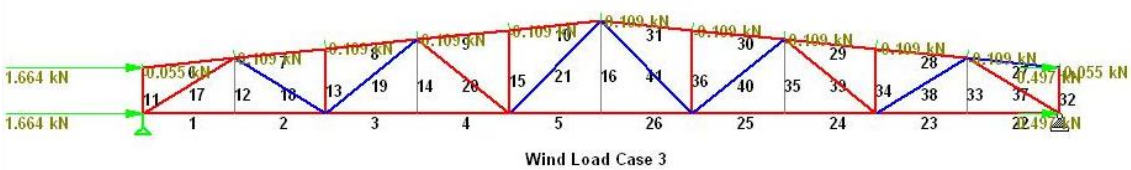


Fig. (12)

Applying wind load from the right as showing in figure 13

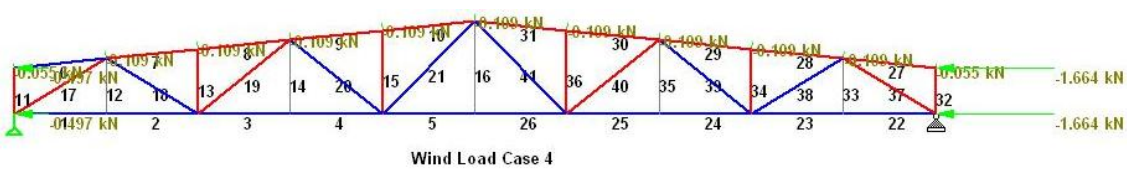


Fig. (13)

to analysis the truss due to load cases we will use the graphical method using computer program like Auto CAD as shown with load case 1 and the other we will use structural analysis program like STAAD Pro, The values of internal force writer in table 4.

Labeling of spaces between forces figure 14.

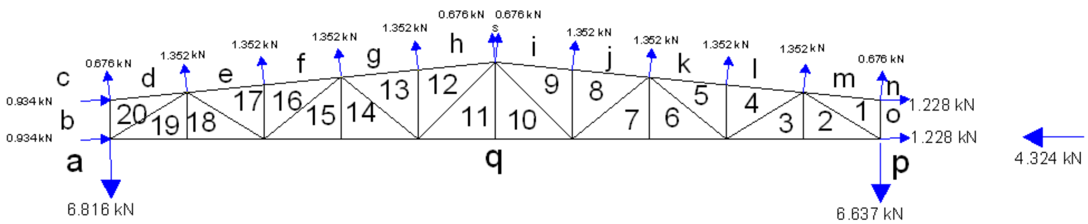


Fig. (14)

Drawing the graph figure 15.

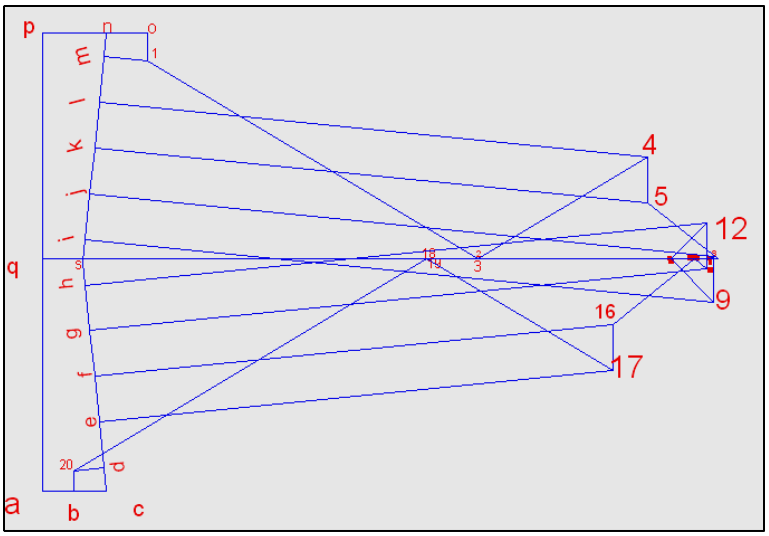


Fig. (15)

The loads due to load case 1 are shown in figure 16. (Red members for compression and blue members for tension member for all figures in this project)

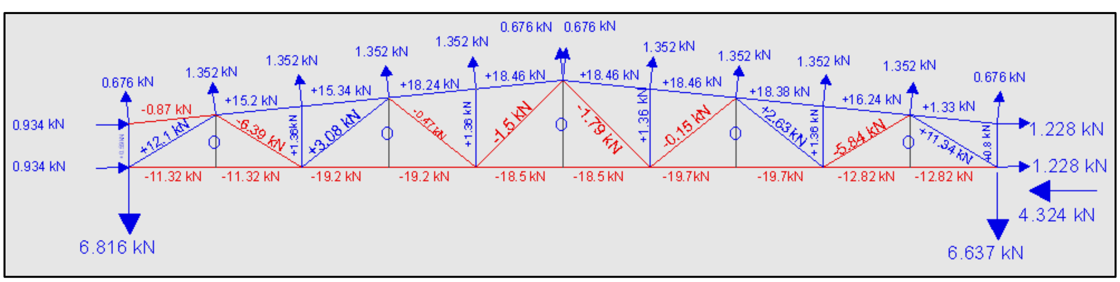


Fig. (16)

Design of Purlins

Calculation of load per 1m

Live Load

L = 1 kN/m^2 ASCE 7-05 table (4-1) on the horizontal projection.

L = 1 cosΘ = 0.995 kN/m^2 on inclined length.

Live load on internal purlin = 0.995 * 2.8 =2.786 kN/m.

Snow Load

S = 1.2 kN/m^2 on the horizontal projection.

L = 1.2 cosΘ = 1.194 kN/m^2 on inclined length.

Snow load on internal purlin = 1.194 * 2.8 =3.343 kN/m.

Roofing Load

In general roofing load taken between (0.3 – 1.0)kN/m^2 , in our project we will take it 0.575kN/m^2 ➔ 0.575*2.8=1.609 kN/m on purlin ➔ 1.609*4= 6.436 kN on nod

Roofing load per unit length on internal truss = 0.575 * 2.8 = 1.609 kN/m.

Wind Load

The critical wind load is load case B, then W = 9.77 * 10^{-3} * 2.8 =0.027 kN/m.

Load Combinations

- w_{ux} = { (1.2)* (1.609) + (1.6) * (2.786) +(0.5) *(3.343)} cosΘ = 8.020 kN/m.
- w_{ux} = { (1.2)* (1.609) + (1.6) * (3.343) +(0.5) *(2.786)} cosΘ = <u>8.630 kN/m</u>
  or = { (1.2)* (1.609) + (1.6) * (3.343) } cosΘ + (0.8) *(0.027) = 7.265 kN/m.
- w_{ux} = { (1.2)* (1.609) + (0.5) * (2.786) +(0.5) *(3.343)} cosΘ +1.6 * 0.027 = 5.014 kN/m.

$M_{ux} = \frac{w_{ux} * l^2}{8} = \frac{8.630 * 4^2}{8}$ = 17.26 kN.m

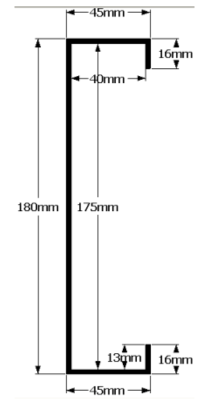
w_{uy} = { (1.2)* (1.609) + (1.6) * (3.343) +(0.5) *(2.786)} sinΘ = 0.8630 kN/m.

$M_{uy} = \frac{w_{uy} * l^2}{32} = \frac{0.863 * 4^2}{32}$ = 0.4315 kN.m one sag rod is used in mid point of the purlin

Here we will use calculation to find the suitable cold rolled section then we will create table 4 to other sections to use it in the design as follow

Try C180x250

Area = 2.5*(180 + 2*40 +2*16) =730 mm^2



Z_x = 2*2.5(90*45 +40*88.75 + 16*82) =44560 mm^3

Distance from the inner face toward the back (x)

(2*2.5*)(40+16)+180X = 180(2.5-X) ➔ X = 0.47222 mm

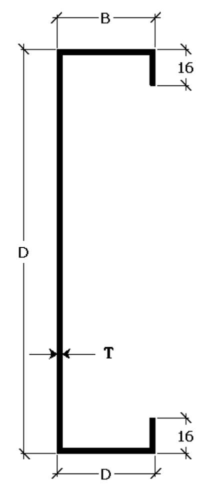
Z_y = {{2.5-0.47222)^2 *180/2} + 2*2.5{(40*20.47222) + (16*(46.25+.47222))} + {180/2 *0.47222^2} = 7822.36 mm^3

w_{ux} = { (1.2)* (1.609+5.23*9.81*10^{-3}) + (1.6) * (3.343) +(0.5) *(2.786)} cosΘ = 8.691 kN/m.

w_{uy} = { (1.2)* (1.609+5.23*9.81*10^{-3}) + (1.6) * (3.343) +(0.5) *(2.786)} sinΘ = 0.8691 kN/m.

$M_{ux} = \frac{w_{ux}*l^2}{8} = \frac{8.691*4^2}{8}$ = 17.382 kN.m

$M_{uy} = \frac{w_{uy}*l^2}{32} = \frac{0.8691*4^2}{32}$ = 0.435 kN.m



M_{px} = 0.9*400*44560*10^{-6} = 16.042 kN.m

M_{py} = 0.9*400*7822.36*10^{-6} = 2.8161 kN.m

$\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}} \leq 1.0$ ➔ $\frac{17.382}{16.042} + \frac{0.435}{2.8161}$ = 1.238 ➔ Try lager one using table

Section	Weight (kg/m)	D (mm)	T (mm)	B (mm)	Area (mm2)	Zx (mm3)	Zy (mm3)	ΦMpx (kN*m)	ΦMpy (kN*m)
C180x150	3.40	180	1.5	45	444	27331.5	4887.7	9.839	1.7596
C180x200	4.48	180	2.0	45	588	36044	6385.8	12.976	2.2989
C180x250	5.56	180	2.5	45	730	44560	7822.361	16.042	2.8161
C210x150	3.93	210	1.5	45	489	34329	4910.582	12.358	1.7678
C210x200	5.23	210	2.0	45	648	45314	6426.114	16.313	2.3134
C210x250	6.54	210	2.5	45	805	56072.5	7884.792	20.186	2.8385
C258x150	4.79	258	1.5	60	606	52700.25	7908.401	18.972	2.8470
C258x200	6.38	258	2.0	60	804	69698	10385.63	25.091	3.7388
C258x250	7.98	258	2.5	60	1000	86413.75	12787.26	31.109	4.6034

Table (4)

Try C 210x250 weight = 6.54 kg/m
w_{ux} = { (1.2)* (1.609+6.54*9.81*10^{-3}) + (1.6) * (3.343) +(0.5) *(2.786)} cosΘ = 8.706 kN/m.
w_{uy} = { (1.2)* (1.609+6.54*9.81*10^{-3}) + (1.6) * (3.343) +(0.5) *(2.786)} sinΘ = 0.8706 kN/m.
$M_{ux} = \frac{w_{ux}*l^2}{8} = \frac{8.706 *4^2}{8}$ = 17.412 kN.m
$M_{uy} = \frac{w_{uy}*l^2}{32} = \frac{0.8706*4^2}{32}$ = 0.435 kN.m
M_{px} = 20.186 kN.m

M_{py} = 2.8385 kN.m

$\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}} \leq 1.0$ ➔ $\frac{17.412}{20.186} + \frac{0.435}{2.8385}$ = 1.0158 ➔ Try lager one

Try C258x150 weight = 4.79 kg/m

w_{ux} = { (1.2)* (1.609+4.79*9.81*10^{-3}) + (1.6) * (3.343) +(0.5) *(2.786)} cosϴ = 8.686 kN/m.

w_{uy} = { (1.2)* (1.609+4.79*9.81*10^{-3}) + (1.6) * (3.343) +(0.5) *(2.786)} sinϴ = 0.8686 kN/m.

$M_{ux} = \frac{w_{ux}*l^2}{8} = \frac{8.686*4^2}{8}$ = 17.371 kN.m

$M_{uy} = \frac{w_{uy}*l^2}{32} = \frac{0.8686*4^2}{32}$ = 0.434 kN.m

M_{px} = 18.972 kN.m

M_{py} = 2.8470 kN.m

$\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}} \leq 1.0$ ➔ $\frac{17.371}{18.972} + \frac{0.434}{2.847}$ = 1.068 ➔ Try lager one

Try C258x200 weight = 6.38 kg/m

w_{ux} = { (1.2)* (1.609+ 6.38*9.81*10^{-3}) + (1.6) * (3.343) +(0.5) *(2.786)} cosϴ = 8.704 kN/m.

w_{uy} = { (1.2)* (1.609+6.38*9.81*10^{-3}) + (1.6) * (3.343) +(0.5) *(2.786)} sinϴ = 0.8704 kN/m.

$M_{ux} = \frac{w_{ux}*l^2}{8} = \frac{8.704*4^2}{8}$ = 17.409 kN.m

$M_{uy} = \frac{w_{uy}*l^2}{32} = \frac{0.8704*4^2}{32}$ = 0.4352 kN.m

M_{px} = 25.091 kN.m

M_{py} = 3.7388 kN.m

$\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}} \leq 1.0$ ➔ $\frac{17.409}{25.091} + \frac{0.4352}{3.7388}$ = 0. 810 ➔ OK use C258x200

Loading on each nod of the truss = (9.81*10^{-3}*6.38)*4 = 0.250 kN

Design of Sag Rod

Gravity load in kN/m^2 of roof surface are as follows:

Average weight in kN/m^2 of the six purlin on each side of the roof = $\frac{6*0.06259}{14.0698}$ = 0.0267 kN/m^2

Live load = 0.995 kN/m^2

Snow load =1.194 kN/m^2

Roofing = 0.575 kN/m^2

Wind load = 9.77*10^{-3} *cosϴ = 0.0097 kN/m^2

LRFD load on top inclined sag rod using controlling load factor equation

w_u = 1.2*(0.0267 +0.575) + 1.6*1.194 + 0.5*0.995 =3.01299 kN/m^2

Component of load parallel to roof surface = 3.01299 * sinϴ = 0.3114 kN/m^2

Loading on top inclined sag rod = (9/10)*(14.0698)*(6)*(0.3114) =23.66 kN = P_u

$A_D = \frac{P_u}{\Phi\, 0.75\, F_u} = \frac{23.66*1000}{0.75*0.75*400}$ = 105.17 mm^2

Try (5/8-in) or (1.5 cm) rod as a minimum practical size 4 threads per 1cm A_D = 176.7 mm^2

R_n = 0.75 F_u A_D = 0.75*400*176.7*10^{-3} = 53.01 kN

ΦR_n = 0.75 * 53.01 = 39.76 kN > P_u OK

Use 1.5cm

Check force in tie rod between ridge purlins

P_u = 14.0698 * 6 * 0.3114 * 1/cosϴ = 26.42 kN < 39.76kN

Use 1.5cm rod

Table (5) Internal Forces.

Member	Unit Load	L L	S L	WL 1	WL 2	WL 3	WL 4	Max WL	Min WL	Roofing	purlin
1	7.5	83.6	100.3	-11.3	-8.5	-1.3	1.6	1.6	-11.3	48.3	1.9
2	7.5	83.6	100.3	-11.3	-8.5	-1.3	1.6	1.6	-11.3	48.3	1.9
3	13.125	146.3	175.5	-19.3	-15.5	-1.3	2.5	2.5	-19.3	84.5	3.3
4	13.125	146.3	175.5	-19.3	-15.5	-1.3	2.5	2.5	-19.3	84.5	3.3
5	12.5	139.3	167.2	-18.6	-14.3	-1.7	2.6	2.6	-18.6	80.5	3.1
6	0	0.0	0.0	-0.9	1.3	-1.7	0.5	1.3	-1.7	0.0	0.0
7	-11.486	-128.0	-153.6	15.3	16.2	-2.1	-1.2	16.2	-2.1	-73.9	-2.9
8	-11.486	-128.0	-153.6	15.4	16.3	-2.2	-1.2	16.3	-2.2	-73.9	-2.9
9	-13.4	-149.3	-179.2	18.3	18.6	-1.9	-1.7	18.6	-1.9	-86.2	-3.4
10	-13.4	-149.3	-179.2	18.5	18.7	-1.9	-1.7	18.7	-1.9	-86.2	-3.4
11	-0.5	-5.6	-6.7	0.6	0.8	-0.2	0.0	0.8	-0.2	-3.2	-0.1
13	-1	-11.1	-13.4	1.4	1.4	-0.1	-0.1	1.4	-0.1	-6.4	-0.3
15	-1	-11.1	-13.4	1.4	1.4	-0.1	-0.1	1.4	-0.1	-6.4	-0.3
17	-8.746	-97.5	-117.0	12.1	11.3	-0.4	-1.3	12.1	-1.3	-56.3	-2.2
18	4.581	51.1	61.3	-6.4	-5.8	0.1	0.7	0.7	-6.4	29.5	1.1
19	-2.172	-24.2	-29.0	3.1	2.6	0.1	-0.4	3.1	-0.4	-14.0	-0.5
20	0.267	3.0	3.6	-0.5	-0.1	-0.2	0.2	0.2	-0.5	1.7	0.1
21	1.179	13.1	15.8	-1.5	-1.8	0.3	0.0	0.3	-1.8	7.6	0.3
22	7.5	83.6	100.3	-12.8	-7.0	-2.7	3.0	3.0	-12.8	48.3	1.9
23	7.5	83.6	100.3	-12.8	-7.0	-2.7	3.0	3.0	-12.8	48.3	1.9
24	13.125	146.3	175.5	-19.8	-14.9	-1.8	3.0	3.0	-19.8	84.5	3.3
25	13.125	146.3	175.5	-19.8	-14.9	-1.8	3.0	3.0	-19.8	84.5	3.3
26	12.5	139.3	167.2	-18.6	-14.3	-1.7	2.6	2.6	-18.6	80.5	3.1
27	0	0.0	0.0	1.3	-0.9	0.5	-1.7	1.3	-1.7	0.0	0.0
28	-11.486	-128.0	-153.6	16.2	15.3	-1.2	-2.1	16.2	-2.1	-73.9	-2.9
29	-11.486	-128.0	-153.6	16.3	15.4	-1.2	-2.2	16.3	-2.2	-73.9	-2.9
30	-13.4	-149.3	-179.2	18.6	18.3	-1.7	-1.9	18.6	-1.9	-86.2	-3.4
31	-13.4	-149.3	-179.2	18.7	18.5	-1.7	-1.9	18.7	-1.9	-86.2	-3.4
32	-0.5	-5.6	-6.7	0.8	0.6	0.0	-0.2	0.8	-0.2	-3.2	-0.1
34	-1	-11.1	-13.4	1.4	1.4	-0.1	-0.1	1.4	-0.1	-6.4	-0.3
36	-1	-11.1	-13.4	1.4	1.4	-0.1	-0.1	1.4	-0.1	-6.4	-0.3
37	-8.746	-97.5	-117.0	11.3	12.1	-1.3	-0.4	12.1	-1.3	-56.3	-2.2
38	4.581	51.1	61.3	-5.8	-6.4	0.7	0.1	0.7	-6.4	29.5	1.1
39	-2.172	-24.2	-29.0	2.6	3.1	-0.4	0.1	3.1	-0.4	-14.0	-0.5
40	0.267	3.0	3.6	-0.1	-0.5	0.2	-0.2	0.2	-0.5	1.7	0.1
41	1.179	13.1	15.8	-1.8	-1.5	0.0	0.3	0.3	-1.8	7.6	0.3

Table (6) Load combinations.

1.4D	1.2D+1.6L+.5(Lr/S/R)	1.2D+1.6(Lr/S/R)+(.5Lor. 8W)			1.2D+1.6W+.5L+.5(Lr/S/R)	1.2±1E+.5L+.2S	0.9D±(1.6Wor1E)		+ve Max Force (kN)	-ve Max Force (kN)	Check for stress reversal	Member
		A	B	C			A	B	Design	Design		
96.2	266.3	284.7	244.2	233.8	176.9	144.3	64.3	43.7	284.7	43.7	NA	1
96.2	266.3	284.7	244.2	233.8	176.9	144.3	64.3	43.7	284.7	43.7	NA	2
168.3	466.0	498.2	427.1	409.6	309.1	252.5	112.2	77.3	498.2	77.3	NA	3
168.3	466.0	498.2	427.1	409.6	309.1	252.5	112.2	77.3	498.2	77.3	NA	4
160.3	443.8	474.5	406.9	390.0	294.8	240.5	107.2	73.3	474.5	73.3	NA	5
0.1	0.1	0.1	1.1	-1.3	2.1	0.1	2.1	-2.6	2.1	-2.6	*Reversal*	6
-147.1	-407.7	-435.9	-358.9	-373.6	-241.0	-220.8	-68.7	-98.0	-68.7	-435.9	NA	7
-147.1	-407.7	-435.9	-358.8	-373.6	-240.8	-220.8	-68.5	-98.0	-68.5	-435.9	NA	8
-171.7	-475.7	-508.5	-419.0	-435.4	-281.7	-257.6	-80.6	-113.4	-80.6	-508.5	NA	9
-171.7	-475.7	-508.5	-418.9	-435.4	-281.5	-257.6	-80.4	-113.5	-80.4	-508.5	NA	10
-6.4	-17.7	-19.0	-15.5	-16.4	-10.3	-9.6	-2.8	-4.5	-2.8	-19.0	NA	11
-12.8	-35.5	-37.9	-31.3	-32.5	-21.1	-19.2	-6.1	-8.4	-6.1	-37.9	NA	13
-12.8	-35.5	-37.9	-31.3	-32.5	-21.1	-19.2	-6.1	-8.4	-6.1	-37.9	NA	15
-112.1	-310.5	-331.9	-273.5	-284.2	-183.8	-168.2	-52.6	-74.0	-52.6	-331.9	NA	17
58.7	162.6	173.9	148.9	143.2	107.6	88.1	38.9	27.5	173.9	27.5	NA	18
-27.8	-77.1	-82.4	-67.9	-70.7	-45.5	-41.8	-12.9	-18.6	-12.9	-82.4	NA	19
3.4	9.5	10.1	8.8	8.3	6.5	5.1	2.5	1.4	10.1	1.4	NA	20
15.1	41.8	44.7	38.4	36.7	27.9	22.7	10.3	6.8	44.7	6.8	NA	21
96.4	266.5	284.9	245.5	232.8	179.4	144.5	66.8	41.5	284.9	41.5	NA	22
96.4	266.5	284.9	245.5	232.8	179.4	144.5	66.8	41.5	284.9	41.5	NA	23
168.3	466.1	498.2	427.6	409.3	310.1	252.5	113.1	76.5	498.2	76.5	NA	24
168.3	466.1	498.2	427.6	409.3	310.1	252.5	113.1	76.5	498.2	76.5	NA	25
160.3	443.8	474.5	406.9	390.0	294.8	240.5	107.2	73.3	474.5	73.3	NA	26
-0.2	-0.2	-0.2	0.8	-1.5	1.9	-0.2	1.9	-2.8	1.9	-2.8	*Reversal*	27
-147.2	-407.8	-436.0	-359.0	-373.7	-241.1	-220.9	-68.8	-98.1	-68.8	-436.0	NA	28
-147.2	-407.8	-436.0	-358.9	-373.7	-240.9	-220.9	-68.5	-98.1	-68.5	-436.0	NA	29
-171.7	-475.7	-508.5	-419.0	-435.4	-281.7	-257.7	-80.6	-113.5	-80.6	-508.5	NA	30
-171.7	-475.7	-508.5	-418.9	-435.4	-281.5	-257.7	-80.4	-113.5	-80.4	-508.5	NA	31
-6.4	-17.8	-19.0	-15.6	-16.4	-10.4	-9.6	-2.8	-4.5	-2.8	-19.0	NA	32
-12.8	-35.5	-37.9	-31.3	-32.5	-21.1	-19.2	-6.1	-8.4	-6.1	-37.9	NA	34
-12.8	-35.5	-37.9	-31.3	-32.5	-21.1	-19.2	-6.1	-8.4	-6.1	-37.9	NA	36
-111.9	-310.4	-331.8	-273.4	-284.1	-183.7	-168.1	-52.5	-74.0	-52.5	-331.8	NA	37

58.6	162.6	173.8	148.8	143.1	107.5	88.0	38.8	27.4	173.8	27.4	NA	38
-27.8	-77.1	-82.4	-67.8	-70.6	-45.5	-41.7	-12.9	-18.5	-12.9	-82.4	NA	39
3.4	9.4	10.1	8.7	8.2	6.4	5.1	2.4	1.4	10.1	1.4	NA	40
15.1	41.9	44.8	38.5	36.7	28.0	22.7	10.3	6.8	44.8	6.8	NA	41

Table (7) Final Internal forces.

	+ve Max Force (kN)	**-ve Max Force (kN)**	**Type of force**	**Member**	**Length**	**Final design load (kN)**
	Design	Design				
Lower Chord Members 2LBB	284.6941	43.66425	Tension	1	2.8	284.6941
	284.6941	43.66425	Tension	2	2.8	284.6941
	498.1851	77.33111	Tension	3	2.8	498.1851
	498.1851	77.33111	Tension	4	2.8	498.1851
	474.491	73.29334	Tension	5	2.8	474.491
	284.867	41.48674	Tension	22	2.8	284.867
	284.867	41.48674	Tension	23	2.8	284.867
	498.2499	76.51571	Tension	24	2.8	498.2499
	498.2499	76.51571	Tension	25	2.8	498.2499
	474.491	73.29334	Tension	26	2.8	474.491
Upper Chord Members 2LBB	2.14248	-2.64034	Reversal	6	2.814	Reversal
	-68.67	-435.856	Comp.	7	2.814	435.8559
	-68.455	-435.857	Comp.	8	2.814	435.8573
	-80.6214	-508.519	Comp.	9	2.814	508.5187
	-80.4048	-508.52	Comp.	10	2.814	508.52
	1.93218	-2.83582	Reversal	27	2.814	Reversal
	-68.7538	-435.968	Comp.	28	2.814	435.9677
	-68.5388	-435.969	Comp.	29	2.814	435.969
	-80.6431	-508.548	Comp.	30	2.814	508.5476
	-80.4265	-508.549	Comp.	31	2.814	508.5489
Web Members (One Angle)	-2.82931	-18.9677	Comp.	11	1.4	18.96774
	0	0	Zero	12	1.68	0
	-6.05943	-37.9472	Comp.	13	1.96	37.94724
	0	0	Zero	14	2.24	0
	-6.05943	-37.9472	Comp.	15	2.52	37.94724
	0	0	Zero	16	2.8	0
	-52.6074	-331.922	Comp.	17	3.265	331.9217
	173.8615	27.48048	Tension	18	3.265	173.8615
	-12.9315	-82.4422	Comp.	19	3.586	82.44217
	10.14819	1.439439	Tension	20	3.586	10.14819

	44.72531	6.759223	Tension	21	3.96	44.72531
	-2.84875	-18.9937	Comp.	32	1.4	18.99366
	0	0	Zero	33	1.68	0
	-6.05943	-37.9472	Comp.	34	1.96	37.94724
	0	0	Zero	35	2.24	0
	-6.05943	-37.9472	Comp.	36	2.52	37.94724
	-52.5318	-331.821	Comp.	37	3.265	331.8209
	173.7895	27.42648	Tension	38	3.265	173.7895
	-12.887	-82.3829	Comp.	39	3.586	82.38289
	10.10211	1.404879	Tension	40	3.586	10.10211
	44.76611	6.789823	Tension	41	3.96	44.76611

Design of members

Zero members

A36 (F_y= 248 MPs, F_u= 400MPs)

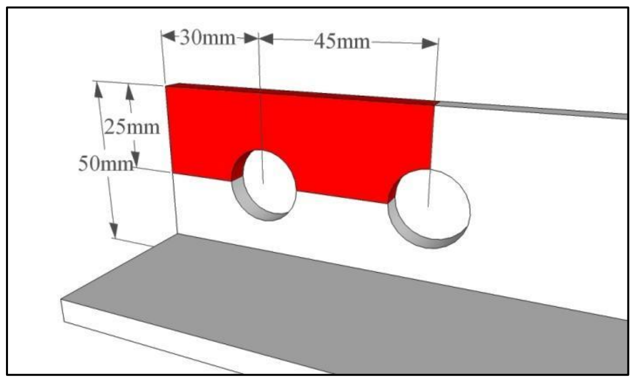
We will use L50x50x6
(A=569mm^2, $\bar{x}$=$\bar{y}$=14.5mm, r_z=9.7mm, r_x=r_y=15mm)

a) Gross section yielding for angle

ΦP_n= 0.9$A_g F_y$
= 0.9*569*248*10^{-3}
= 127kN

b) Tensile rupture strength for angle

ΦP_n= 0.75$A_e F_u$
A_n= 569 – 6(16+4) = 449mm^2
Minimum edge distance equal (1.5 ~ 2) d
(24mm ~ 32mm) take it 30mm
Center to center distance equal (2 $\frac{2}{3}$ ~ 3) d
(43mm ~ 48mm) take it 45mm
U= 1 - $\frac{\bar{x}}{L}$ = 1 - $\frac{14.5}{45}$ = 0.678 ←
U= 0.6 AISC-05 table D3-1 case 8
A_e= UA_n = 0.678*449 = 304.32mm^2
ΦP_n= 0.75*304.32*400*10^{-3}= 91.32kN



c) Block shear strength

R_n = 0.6 A_{nv} F_u + U_s A_{nt} F_u ≤ 0.6 A_{gv} F_y + U_s A_{nt} F_u
A_{gv}= 6*(45+30) = 450mm^2, A_{nv} = 450 – 1.5*6*20 =270mm^2, A_{nt}= 6(25-.5*20) = 90mm^2
R_n =10^{-3}(0.6*270*400 + 1.0*90*400) ≤ 10^{-3}(0.6*450*248 + 1.0*90*400)
= 100.8 < 102.96
ΦR_n = 0.75* R_n = 75.6kN

d) slenderness ratio

$\frac{Kl}{r}$ = $\frac{2800}{9.7}$ =288.66 < 300 OK

e) Bearing strength of bolts

$R_{n\ of\ 1bolt}$ =1.2 L_c t F_u ≤ 2.4 d t F_u
L_c = 30 - 0.5*20 = 20mm or L_c = 45 -1*20 = 25mm
R_n = 2(1.2*20*6*400)*10^{-3} ≤ 2*10^{-3}(2.4*16*6*400)
= 115.2 < 184.32
ΦR_n = 0.75 R_n = 86.4kN

f) Shearing strength of bolts

$R_{n\ of\ 1bolt} = F_{nv}\ A_b = 2(330 * \frac{\pi 16^2}{4})*10^{-3} = 132.7kN$
$\Phi R_n = 0.75 * R_n = \underline{99.526kN}$

∴ Use L50x50x6 for members No. (12 , 14 , 16 , 33 , 35)

Checking compression strength of L50x50x6 with 1.96m length

$\frac{L}{r_x} = \frac{1960}{15} = 130.66 > 80$
$\therefore \frac{Kl}{r_z} = 32 + 1.25\frac{L}{r_x} \leq 200 \rightarrow \frac{Kl}{r_z} = 32 + 1.25*130.66 = 195.325$
$, 4.71\sqrt{\frac{E}{F_y}} = 4.71\sqrt{\frac{200000}{248}} = 133.755$
$\frac{Kl}{r_z} > 4.71\sqrt{\frac{E}{F_y}} \rightarrow F_{cr} = 0.877F_e$
$F_e = \frac{\pi^2 E}{\left(\frac{Kl}{r}\right)^2} = \frac{\pi^2 * 200000}{195.325^2} = 51.735MPs \rightarrow F_{cr} = 0.877*51.735 = 45.372\ MPs$
$P_n = A_g\ F_{cr} = 569*45.372*10^{-3} = 25.81kN$
$\Phi P_n = 0.9P_n = 23.235kN$

∴ Use L50x50x6 for members No. (11 , 32)

For members 20 , 40

$r_{reqd} = \frac{3586}{300} = 11.953mm$

∴ Use L65x65x7 for members No. (20 , 40)

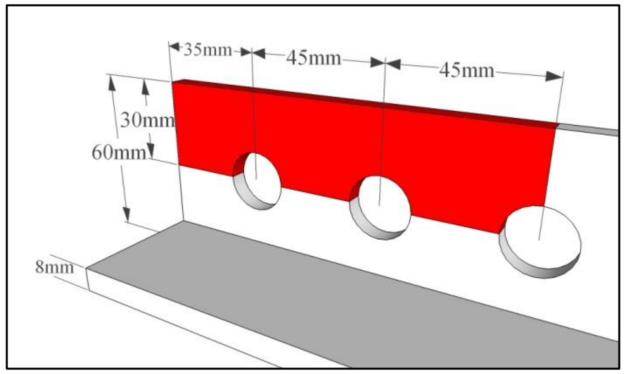
For members 21, 41

$r_{reqd} = \frac{3960}{300} = 13.2mm$

∴ Use L70x70x6 for members No. (21 , 41)

For members 18, 38

$P_u = 173.8615kN3$



1) Min $A_g = \frac{p_u}{\emptyset F_y} = \frac{173.86\times10^3}{0.9\times248}$ =779mm^2
2) Assume U=0.85

Min $A_{g\ reqd} = \frac{p_u}{\emptyset F_u U} + area\ of\ bolts\ hole =$
$\frac{173.86\times10^3}{0.75\times400\times0.85} + (16+4)t$
$= 682 + 20t$

3) Min $r_{reqd} = \frac{3265}{300}$
$= 10.88mm$

Try L60x60x8 (A=903mm^2, $\bar{x}$=$\bar{y}$=17.7mm, r_z=11.6mm, r_x=r_y=18.0mm)

a) Bearing strength of bolts

$R_{n\ of\ 1bolt}$ = 1.2 Lc t Fu ≤ 2.4 d t F_u
L_c = 35 - 0.5*20 = 25mm or L_c = 45 -1*20 = 25mm
R_n = 3(1.2*25*8*400)*10^{-3} ≤ 2*10^{-3}(2.4*16*8*400)
= 288 < 368.64
ΦR_n = 0.75 R_n = 218kN OK

b) Shearing strength of bolts

$R_{n\ of\ 1bolt}$ = F_{nv} A_b = 3(414 * $\frac{\pi 16^2}{4}$)*10^{-3} = 249.72kN
ΦR_n = 0.75 * R_n =187.29kN (A325M threads are excluded from shear plane)

c) Block shear strength

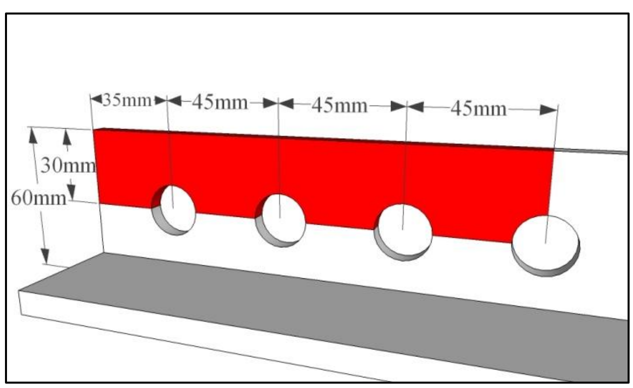
$R_n = 0.6\ A_{nv}\ F_u + U_s\ A_{nt}\ F_u \le 0.6\ A_{gv}\ F_y + U_s\ A_{nt}\ F_u$
A_{gv}= 8*(2*45+35) = 1000mm^2, A_{nv} = 1000 – 2.5*8*20 =600mm^2, A_{nt}= 6(30-.5*20) = 160mm^2
R_n =10^{-3}(0.6*600*400 + 1.0*160*400) ≤ 10^{-3}(0.6*1000*248 + 1.0*160*400)
= 208 < 212.8
ΦR_n = 0.75* R_n = 156kN < P_u (Not OK try with 4 bolts)

A_{gv}= 8*(3*45+35) = 1360mm^2, A_{nv} = 1360 – 3.5*8*20 =800mm^2, A_{nt}= 6(30-.5*20) = 160mm^2
R_n =10^{-3}(0.6*800*400 + 1.0*160*400) ≤ 10^{-3}(0.6*1360*248 + 1.0*160*400)
= 250 < 266.4
ΦR_n = 0.75* R_n = 192kN OK

(Note)

No need to recalculate shearing and bearing strength of bolts because it should be safe.

d) Gross section yielding of angle



ΦP_n= 0.9 $A_g F_y$
= 0.9*903*248*10^{-3}
= 201.5 kN OK

e) tensile rupture strength of angle

A_n= 903 – 8(16+4) = 743mm^2

U= 1 - $\frac{\bar{x}}{L}$ = 1 - $\frac{17.7}{3\times45}$ = 0.869 ←
U= 0.8 AISC-05 table D3-1 case 8
A_e= UA_n = 0.869*743 = 645.7mm^2
ΦP_n= 0.75*645.7*400*10^{-3}= 193.7kN OK

∴ Use L60x60x8 for members No. (18 , 38)

Design of compression web members

For members 13 , 34

P_u = 37.9472 kN
Assume $\frac{Kl}{r}$ = 130 (near from member 11)
$4.71\sqrt{\frac{E}{F_y}} = 4.71\sqrt{\frac{200000}{248}} = 133.755$
$\frac{Kl}{r_z} < 4.71\sqrt{\frac{E}{F_y}}$ → $F_{cr} = \left(0.658^{\frac{F_y}{F_e}}\right)F_y$
$F_e = \frac{\pi^2 E}{\left(\frac{Kl}{r}\right)^2} = \frac{\pi^2 * 200000}{130^2}$ = 116.8MPs → $F_{cr} = \left(0.658^{\frac{248}{116.8}}\right)248$ = 101.97MPs
$P_u = F_{cr} A_g$ → $A_g = \frac{P_u}{\phi F_{cr}} = \frac{37.9472\times10^3}{0.9\times101.97}$ = 413.47mm^2
Try L60x60x5 (A=582mm^2, r_z=11.7mm, r_x=r_y=18.2mm)
$\frac{L}{r_x} = \frac{1960}{18.2}$ = 107.69 > 80
∴ $\frac{Kl}{r_z} = 32 + 1.25\frac{L}{r_x} \le 200$ → $\frac{Kl}{r_z}$ = 32 +1.25*107.69 =166.615 < 200
, $4.71\sqrt{\frac{E}{F_y}} = 4.71\sqrt{\frac{200000}{248}} = 133.755$
$\frac{Kl}{r_z} > 4.71\sqrt{\frac{E}{F_y}}$ → $F_{cr} = 0.877F_e$
$F_e = \frac{\pi^2 E}{\left(\frac{Kl}{r}\right)^2} = \frac{\pi^2 * 200000}{166.615^2}$ = 71.10 MPs → F_{cr} = 0.877*71.10 = 62.359 MPs
$P_n = A_g F_{cr}$ = 582*62.359*10^{-3} = 36.293kN
$\Phi P_n = 0.9P_n$ = 32.66kN not OK try lager section
Try L60x60x6 (A=691mm^2, r_z=11.7mm, r_x=r_y=18.2mm)
ΦP_n = 0.9*691*62.359*10^{-3} = 38.78kN > P_u

∴ Use L60x60x6 for members No. (13 , 34)

For members 15 , 36

We can begin with L65x65x7 (A=870mm^2, r_z=12.6mm, r_x=r_y=19.6mm)
$\frac{L}{r_x} = \frac{2520}{19.6}$ = 128.571 > 80
∴ $\frac{Kl}{r_z} = 32 + 1.25\frac{L}{r_x} \le 200$ → $\frac{Kl}{r_z}$ = 32 +1.25*128.571 = 192.71 < 200

, $4.71\sqrt{\frac{E}{F_y}}$ = $4.71\sqrt{\frac{200000}{248}}$ = 133.755

$\frac{Kl}{r_z}$ > $4.71\sqrt{\frac{E}{F_y}}$ ➔ F_{cr} = 0.877F_e

$F_e = \frac{\pi^2 E}{\left(\frac{Kl}{r}\right)^2} = \frac{\pi^2 * 200000}{128.571^2}$ = 53.1498 MPs ➔ F_{cr} = 0.877*53.1498= 46.612 MPs

ΦP_n = 0.9*813*46.61*10^{-3} = 36.5 kN < P_u not OK try larger section

Try L70x70x6 (A=813mm^2, r_z=13.7mm, r_x=r_y=21.3mm)

$\frac{L}{r_x} = \frac{2520}{21.3}$ = 118.31 > 80

∴ $\frac{Kl}{r_z}$ = 32 +1.25 $\frac{L}{r_x}$ ≤ 200 ➔ $\frac{Kl}{r_z}$ = 32 +1.25*118.31= 179.887 < 200

, $4.71\sqrt{\frac{E}{F_y}}$ = $4.71\sqrt{\frac{200000}{248}}$ = 133.755

$\frac{Kl}{r_z}$ > $4.71\sqrt{\frac{E}{F_y}}$ ➔ F_{cr} = 0.877F_e

$F_e = \frac{\pi^2 E}{\left(\frac{Kl}{r}\right)^2} = \frac{\pi^2 * 200000}{179.887^2}$ = 60.9999 MPs ➔ F_{cr} = 0.877*60.9999= 53.497 MPs

ΦP_n = 0.9*813*53.497*10^{-3} = 39.14 kN > P_u OK

∴ Use L70x70x6 for members No. (15 , 36)

for members 19 , 39

P_u = 82.4422kN

Assume $\frac{Kl}{r} = 134$

$4.71\sqrt{\frac{E}{F_y}}$ = $4.71\sqrt{\frac{200000}{248}}$ = 133.755

$F_e = \frac{\pi^2 E}{\left(\frac{Kl}{r}\right)^2} = \frac{\pi^2 * 200000}{134^2}$ = 109.93 MPs ➔ F_{cr} = 0.877*109.93= 96.409 MPs

$P_u = F_{cr} A_g$ ➔ $A_g = \frac{P_u}{\emptyset F_{cr}} = \frac{82.4422\times 10^3}{0.9\times 96.409}$ = 950.1mm^2

Try L75x75x8 (A=1140mm^2, r_z=14.5mm, r_x=r_y=22.7mm)

$\frac{L}{r_x} = \frac{3586}{22.7}$ = 157.974 > 80

∴ $\frac{Kl}{r_z}$ = 32 +1.25 $\frac{L}{r_x}$ ≤ 200 ➔ $\frac{Kl}{r_z}$ = 32 +1.25*157.974 =229.5 > 200 not OK try larger section

Finding minimum r_x from $\frac{Kl}{r_z}$ = 32 +1.25 $\frac{L}{r_x}$ ≤ 200

$\frac{1}{r_x} = \frac{1}{1.25\,L} \times (200 - 32)$ ➔ $r_{x\,min.} = \frac{1.25L}{168}$ = 26.68mm

Try L90x90x7 (A=1220mm^2, r_x= r_y= 27.5mm)

$\frac{L}{r_x} = \frac{3586}{27.5}$ = 130.4 > 80

∴ $\frac{Kl}{r_z}$ = 32 +1.25 $\frac{L}{r_x}$ ≤ 200 ➔ $\frac{Kl}{r_z}$ = 32 +1.25*130.4 = 195 < 200

$\frac{Kl}{r_z}$ > $4.71\sqrt{\frac{E}{F_y}}$ ➔ F_{cr} = 0.877F_e

$F_e = \frac{\pi^2 E}{\left(\frac{Kl}{r}\right)^2} = \frac{\pi^2 * 200000}{195^2}$ = 51.911 MPs ➔ F_{cr} = 0.877*51.911 = 45.526 MPs

ΦP_n = 0.9*1220*45.526*10^{-3} = 49.99 kN < P_u not OK try larger section

Try L100x100x8 (A=1550mm^2, r_x= r_y= 30.6mm)

$\frac{L}{r_x} = \frac{3586}{30.6}$ = 117.1895 > 80

∴ $\frac{Kl}{r_z}$ = 32 +1.25 $\frac{L}{r_x}$ ≤ 200 ➔ $\frac{Kl}{r_z}$ = 32 +1.25*117.1895 = 178.487 < 200

$\frac{Kl}{r_z} > 4.71\sqrt{\frac{E}{F_y}}$ ➔ $F_{cr} = 0.877F_e$

$F_e = \frac{\pi^2 E}{\left(\frac{Kl}{r}\right)^2} = \frac{\pi^2 * 200000}{178.487^2}$ = 61.961 MPs ➔ F_{cr} = 0.877*61.961 = 54.34 MPs

ΦP_n = 0.9*1550*54.34*10^{-3} = 75.8 kN < P_u not OK try larger section

Try L100x100x10 (A=1920mm^2, r_x= r_y= 30.4mm)

$\frac{L}{r_x} = \frac{3586}{30.4}$ = 117. 96 > 80

∴ $\frac{Kl}{r_z}$ = 32 +1.25 $\frac{L}{r_x}$ ≤ 200 ➔ $\frac{Kl}{r_z}$ = 32 +1.25*117. 96 = 179.45 < 200

$\frac{Kl}{r_z} > 4.71\sqrt{\frac{E}{F_y}}$ ➔ $F_{cr} = 0.877F_e$

$F_e = \frac{\pi^2 E}{\left(\frac{Kl}{r}\right)^2} = \frac{\pi^2 * 200000}{179.45^2}$ = 61.297 MPs ➔ F_{cr} = 0.877*61.297 = 53.758 MPs

ΦP_n = 0.9*1920*53.758*10^{-3} = 92.89 kN > P_u OK

∴ Use L100x100x10 for members No. (19, 39)

For members 17 , 37

P_u = 331.821 kN

Assume $\frac{Kl}{r} = 100 < 4.71\sqrt{\frac{E}{F_y}}$

$4.71\sqrt{\frac{E}{F_y}} = 4.71\sqrt{\frac{200000}{248}}$ = 133.755

$F_e = \frac{\pi^2 E}{\left(\frac{Kl}{r}\right)^2} = \frac{\pi^2 * 200000}{100^2}$ = 197.39 MPs ➔ $F_{cr} = \left(0.658^{\frac{F_y}{F_e}}\right)F_y$

$F_{cr} = \left(0.658^{\frac{248}{133.755}}\right)248$ = 146.58 MPs

$P_u = F_{cr} A_g$ ➔ $A_g = \frac{P_u}{\emptyset F_{cr}} = \frac{331.821\times10^3}{0.9\times146.58}$ = 2515.3mm^2

Try L120x120x11 (A=2540mm^2, r_z=23.5mm, r_x=r_y=36.3mm)

$\frac{L}{r_x} = \frac{3265}{36.3}$ = 89.945 > 80

∴ $\frac{Kl}{r_z}$ = 32 +1.25 $\frac{L}{r_x}$ ≤ 200 ➔ $\frac{Kl}{r_z}$ = 32 +1.25*89.945 = 144.43 < 200

$\frac{Kl}{r_z} > 4.71\sqrt{\frac{E}{F_y}}$ ➔ $F_{cr} = 0.877F_e$

$F_e = \frac{\pi^2 E}{\left(\frac{Kl}{r}\right)^2} = \frac{\pi^2 * 200000}{144.43^{\ 2}}$ = 94.625 MPs ➔ F_{cr} = 0.877*94.625 = 82.987 MPs

ΦP_n = 0.9*2540*94.625*10^{-3} = 189.707 kN < P_u not OK try larger section

Try L140x140x10 (A=2720mm^2, r_z=27.6mm, r_x=r_y=43mm)

$\frac{L}{r_x} = \frac{3265}{43}$ = 75.93 < 80

$\frac{Kl}{r_z}$ =72 + 0.75*75.93 =128.948

$4.71\sqrt{\frac{E}{F_y}} = 4.71\sqrt{\frac{200000}{248}}$ = 133.755

$F_e = \frac{\pi^2 E}{\left(\frac{Kl}{r}\right)^2} = \frac{\pi^2 * 200000}{128.948^2}$ = 118.714 MPs ➔ $F_{cr} = \left(0.658^{\frac{F_y}{F_e}}\right) F_y$

$F_{cr} = \left(0.658^{\frac{248}{118.714}}\right) 248$ = 103.45 MPs

ΦP_n = 0.9*2720*103.45 *10^{-3} = 253.24 kN < P_u not OK try larger section

Try L150x150x10 (A=2930mm^2, r_z=29.6mm, r_x=r_y=46.2mm)

$\frac{L}{r_x} = \frac{3265}{46.2}$ = 70.671 < 80

$\frac{Kl}{r_z}$ =72 + 0.75*70.671=125.003

$4.71\sqrt{\frac{E}{F_y}} = 4.71\sqrt{\frac{200000}{248}}$ = 133.755

$F_e = \frac{\pi^2 E}{\left(\frac{Kl}{r}\right)^2} = \frac{\pi^2 * 200000}{125.003^2}$ = 126.324 MPs ➔ $F_{cr} = \left(0.658^{\frac{F_y}{F_e}}\right) F_y$

$F_{cr} = \left(0.658^{\frac{248}{126.324}}\right) 248$ = 109.042 MPs

ΦP_n = 0.9*2930*109.042*10^{-3} = 287.54 kN < P_u not OK try larger section

Try L150x150x12 (A=3480mm^2, r_x=r_y=46.0mm)

$\frac{L}{r_x} = \frac{3265}{46.0}$ = 79.783 < 80

$\frac{Kl}{r_z}$ =72 + 0.75*79.783=125.234

$4.71\sqrt{\frac{E}{F_y}} = 4.71\sqrt{\frac{200000}{248}}$ = 133.755

$F_e = \frac{\pi^2 E}{\left(\frac{Kl}{r}\right)^2} = \frac{\pi^2 * 200000}{125.234^2}$ = 125.8599 MPs ➔ $F_{cr} = \left(0.658^{\frac{F_y}{F_e}}\right) F_y$

$F_{cr} = \left(0.658^{\frac{248}{125.8599}}\right) 248$ = 108.7116 MPs

ΦP_n = 0.9*3480*108.7116*10^{-3} = 340.485 kN

∴ Use L150x150x12 for members No. (17, 37)

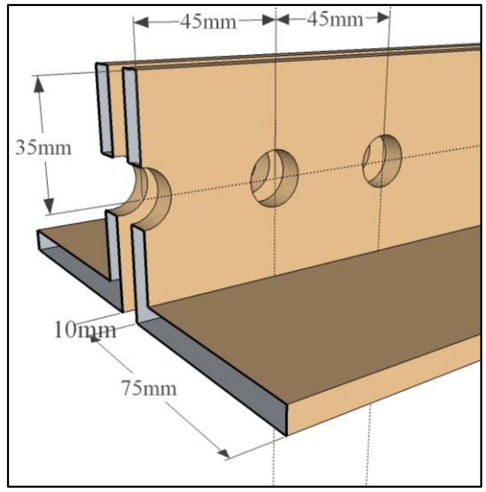
Design of Chord members

Design of lower chord

For members 1, 2, 3, 4, 22, 23, 24, 25

P_u = 498.3 kN

Designing 2Ls back to back with 10mm thickness of slices between the angles



1) min $A_{g\ reqd} = \frac{P_u}{\emptyset F_y} = \frac{498.3\times10^3}{0.9\times248}$ =2232.5 mm^2
2) min $A_{g\ reqd} = \frac{P_u}{\emptyset F_u U}$ + estimated holes of area = $\frac{498.3\times10^3}{0.75\times400\times0.85}$ + 2(16+4) t = 1954.1 + 40 t
3) min r = $\frac{L}{300} = \frac{2800}{300}$ = 9.33mm

Try 2Ls 75x75x8 (A=1140mm^2, r_x=r_y= 22.7mm, I_x=I_y=588700mm^4, $\bar{x} = \bar{y}$ =21.3mm)

Total area = 2*1140 = 2280 mm^2

I_x = 2*588700 = 1177000 mm^4, I_y = 2(588700+2*1140*(5+21.3)^2) = 4331506.4 mm^4

$r_{min} = r_y = \sqrt{\frac{I_y}{A}} = \sqrt{\frac{4331506.4}{2280}}$ = 43.587 mm

a) Gross section yielding

ΦP_n = 0.9*2280*248*10^{-3} = 508.9 kN > P_u OK

b) Tensile rupture strength

A_n = 2280 – 2(16+4)*8 = 2200 mm^2

U = 1 - $\frac{\bar{x}}{L}$ = 1 - $\frac{21.3}{3\times45}$ = <u>0.8422</u> or U = 0.6 table D-3.1 AISC Manual

A_e = 0.8422*2200 = 1852.89 mm^2

ΦP_n = 0.75*1852.89*400*10^{-3} = 555.87 kN > P_u OK

c) Slenderness ratio

$\frac{K_l}{r} = \frac{2800}{43.587}$ = 64.239 < 300 OK

Check if tie plate is required $r_{min} = \frac{2800}{300}$ = 9.333 mm, r_z =14.5 mm > r_{min} ➔ no need for tie plate

d) Bearing strength of bolts

$R_{n\ of\ one\ bolt}$ = 1.2 Lc t Fu ≤ 2.4 d t F_u

L_c = 45 - 1*20 = 25mm

R_n = (1.2*25*8*400)*10^{-3} ≤ 10^{-3}(2.4*16*8*400) = 96 < 122.88

No. of bolts reqd. = $\frac{P_u}{\emptyset R_n} = \frac{498.3}{0.75\times96}$ = 6.92 say 7 bolts

c) Shearing strength of bolts

R_n= m F_{nv} A_b = 2(330 * $\frac{\pi 16^2}{4}$)*10^{-3} = 132.7 kN

No. of bolts reqd. = $\frac{498.3}{0.75\times132.7}$ = 5.006 say 6 bolts

Due to 7 bolts are required we will put them in two lines with change in the section

Try L100x75x8 (A=1336mm^2, r_x= 31.8mm^4, r_y= 22.2mm, I_x= 1348673mm^4, I_y=656123 mm^4, $\bar{x} = 19.0mm,\ \bar{y}$ =31.5 mm)

d) gross section yielding

ΦP_n = 0.9*(2*1336)*248*10^{-3} = 596.39 kN > P_u OK

e) Tensile rupture strength

A_n = (2*1336) – 4*(16+4)*8 = 2032 mm^2

U = 1 - $\frac{\bar{x}}{L}$ = 1 - $\frac{31.5}{3\times45}$ = <u>0.767</u> or U = 0.6 table D-3.1 AISC Manual

A_e = 0.767*2032 = 1558.544 mm^2

ΦP_n = 0.75*1558.544*400*10^{-3} = 467.563 kN < P_u not OK try larger section

Try 2Ls120x80x8 (A=1550mm^2, r_x= 38.2mm^4, r_y= 22.8mm, I_x= 2257000mm^4, I_y=807600 mm^4, $\bar{x} = 18.7mm$, $\bar{y}$ =38.3 mm)

a) Tensile strength rupture

A_n = (2*1550) –4 (16+4)*8 = 2460 mm^2

U = 1 - $\frac{\bar{x}}{L}$ = 1 - $\frac{38.3}{3\times45}$ = <u>0.716</u> or U = 0.6 table D-3.1 AISC Manual

A_e = 0.716*2460 = 1762.09 mm^2

ΦP_n = 0.75*1762.09*400*10^{-3} = 528.63 kN > P_u OK

b) Block shear strength

A_{gv} = 4*8(2*45+25) = 3840 mm^2

A_{nv} = 4*8(120-2.5*20) =2240 mm^2

A_{nt} =2*8(45-20) = 400 mm^2

$R_n = 0.6 A_{nv} F_u + U_{bs} A_{nt} F_u \le 0.6 A_{gv} F_y + U_s A_{nt} F_u$

= 0.6*2240*400*10^{-3} + 1.0*400*400*10^{-3} ≤ 0.6*3840*248*10^{-3} + 1.0*400*400*10^{-3}

= 697.6 < 731.392

$ØR_n$ = 0.75*697.6 = 523.3 kN > P_u OK

c) Bearing strength of bolts

R_n = 1.2 Lc t Fu ≤ 2.4 d t F_u = 6*1.2*25*2*8*400*10^{-3} ≤ 6*2.4*16*2*8*400*10^{-3} = 1152<1474.6

$ØR_n$ = 864 kN > P_u OK

d) Shearing strength of bolts

R_n= m F_{nv} A_b =6* 2(330 * $\frac{\pi 16^2}{4}$)*10^{-3} = 796.205 kN

$ØR_n$ = 0.75*796.205 = 597.15 kN > P_u OK

∴ Use 2Ls 120x80x8 for members No. (1 , 2 , 3 , 4 , 22 , 23 , 24 , 25)

For members 5, 26

Pu = 474.491 kN

Try 2Ls 100x75 8 (A=1336mm^2, r_z=16.2, $\bar{x} = 19.0$, $\bar{y}$ =31.5mm)

a) Gross section yielding

$ØP_n$ = 0.9*(2*1336)*248*10^{-3} = 596.39 kN > P_u OK

b) Tensile rupture strength

A_n = 2*1336-4*8*(16+4) = 2032 mm^2

U= 1 - $\frac{\bar{x}}{L}$ = 1 - $\frac{31.5}{2*45}$ = 0.65 ←, U= 0.6 AISC-05 table D3-1 case 8

A_e= UA_n = 0.65*2032 = 1320.8mm^2

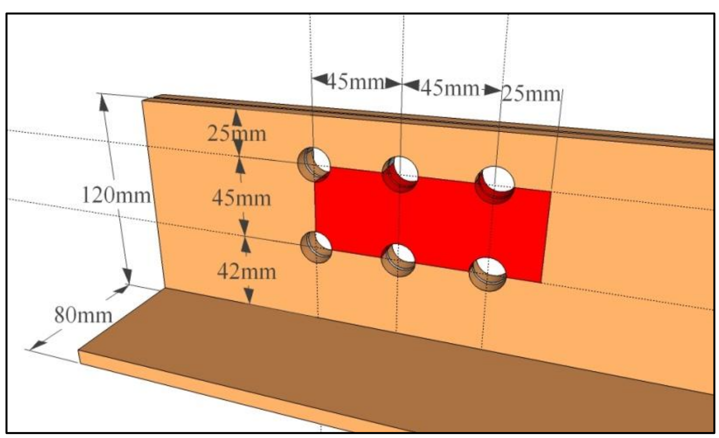
$ØP_n$= 0.75*1320.8*400*10^{-3}= <u>396.24</u> < P_u try with larger section

Try 2Ls 100x75x10 (A=1650mm^2, $\bar{x} = 19.8$, $\bar{y}$ =32.3mm)

A_n = 2*1650-4*10*(16+4) = 2500 mm^2

U= 1 - $\frac{\bar{x}}{L}$ = 1 - $\frac{32.3}{2*45}$ = 0.6422←, U= 0.6 AISC-05 table D3-1 case 8

A_e= UA_n = 0.6422*2500 = 1605.56mm^2



$\emptyset P_n = 0.75*1605.56*400*10^{-3} = 481.67 > P_u$ OK

(Note)

Block shear, shearing strength of bolts and bearing strength of bolts are adequate compering with previous section.

∴ Use 2Ls 100x75x10 for members No. (5, 26)

Design of the upper chord members

P_u = 508.6 kN, L = 2.814 m

Assume $\frac{Kl}{r} = 50 < 4.71\sqrt{\frac{E}{F_y}}$

$4.71\sqrt{\frac{E}{F_y}} = 4.71\sqrt{\frac{200000}{248}} = 133.755$

$F_e = \frac{\pi^2 E}{\left(\frac{Kl}{r}\right)^2} = \frac{\pi^2 * 200000}{50^2}$ = 789.568 MPs ➔ $F_{cr} = \left(0.658^{\frac{F_y}{F_e}}\right) F_y$

$F_{cr} = \left(0.658^{\frac{248}{789.568}}\right) 248$ = 217.45 MPs

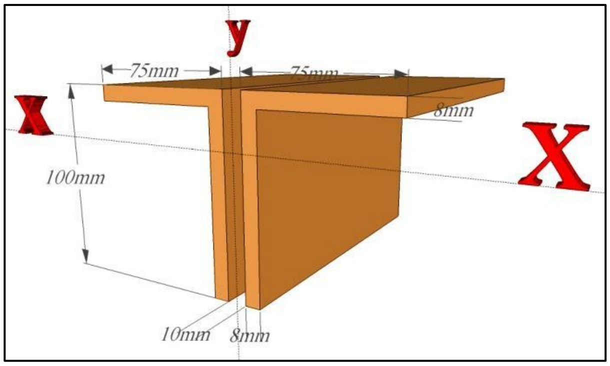
$P_u = F_{cr} A_g$ ➔ $A_g = \frac{P_u}{\emptyset F_{cr}} = \frac{508.6\times10^3}{0.9\times217.45}$ = 2599mm^2

Try 2Ls100x75x8 BBLL for each angle (A=1336mm^2, r_z=16.2mm, r_x= 31.8mm, r_y=22.2mm, I_x =1348673mm^4, I_y=656123mm^4, $\bar{x} = 19.0$)

I_x = 2*1348673 = 2697346 mm^4, I_y = 2(656123+1336(19.0+5)^2=2851318mm^4

$r_y = \sqrt{\frac{I_x}{A}} = \sqrt{\frac{2697346}{2\times1336}}$ = 31.77 mm, $r_y = \sqrt{\frac{I_y}{A}} = \sqrt{\frac{2851318}{2\times1336}}$ = 32.67 mm

$\left(\frac{K_l}{r_x}\right) = \frac{2814}{31.77} = \underline{88.574}$, $\left(\frac{K_l}{r_y}\right) = \frac{2814}{32.67} = 86.134$



$4.71\sqrt{\frac{E}{F_y}} = 4.71\sqrt{\frac{200000}{248}} = 133.755$

$\left(\frac{K_l}{r_x}\right) < 4.71\sqrt{\frac{E}{F_y}}$ ➔ $F_{cr} = \left(0.658^{\frac{F_y}{F_e}}\right) F_y$

$F_e = \frac{\pi^2 E}{\left(\frac{Kl}{r}\right)^2} = \frac{\pi^2 * 200000}{88.574^2}$ = 251.6038 MPs

➔ $F_{cr} = \left(0.658^{\frac{248}{251.6038}}\right) 248$ = 164.165 MPs

$\emptyset P_n = 0.9*164.165*(2*1336)*10^{-3}$ =394.78 kN < P_u not OK try larger section

Try 2Ls120x80x8 BBLL for each angle (A=1550mm^2, r_z=17.3mm, $\bar{x} = 18.7$, r_x= 38.2mm, r_y=22.8mm, I_x =2257000mm^4, I_y=807600mm^4)

I_x = 2*2257000 = 4514000 mm^4, I_y = 2(807600+1550(18.7+5)^2=3356435mm^4

$r_y = \sqrt{\frac{I_x}{A}} = \sqrt{\frac{4514000}{2\times1550}}$ = 38.159 mm, $r_y = \sqrt{\frac{I_y}{A}} = \sqrt{\frac{3356435}{2\times1550}}$ = 32.905 mm

$\left(\frac{K_l}{r_x}\right) = \frac{2814}{38.159} = 73.744$, $\left(\frac{K_l}{r_y}\right) = \frac{2814}{32.905} = \underline{85.52}$

$4.71\sqrt{\frac{E}{F_y}}$ = $4.71\sqrt{\frac{200000}{248}}$ = 133.755

$\left(\frac{K_l}{r_x}\right)$ < $4.71\sqrt{\frac{E}{F_y}}$ ➔ F_{cr} = $\left(0.658^{\frac{F_y}{F_e}}\right)F_y$

F_e = $\frac{\pi^2 E}{\left(\frac{Kl}{r}\right)^2}$ = $\frac{\pi^2 * 200000}{85.52^2}$ = 269.894 MPs ➔ F_{cr} = $\left(0.658^{\frac{248}{269.894}}\right)248$ = 168.82 MPs

$ØP_n$ = 0.9*168.82*(2*1550)*10^{-3} =471.01 kN < P_u not OK try larger section

Try 2Ls120x80x10 BBLL for each angle (A=1910mm^2, $\bar{x} = 20.3$, I_x =3228000mm^4, I_y=1143000mm^4)

I_x = 2*3228000 = 6456000 mm^4, I_y = 2(1143000+1910(20.3+5)^2=4731143.8mm^4

r_y = $\sqrt{\frac{I_x}{A}}$ = $\sqrt{\frac{6456000}{2\times1910}}$ = 41.11 mm, r_y = $\sqrt{\frac{I_y}{A}}$ = $\sqrt{\frac{4731143.8}{2\times1910}}$ = 35.193 mm

$\left(\frac{K_l}{r_x}\right)$ = $\frac{2814}{41.11}$ = 68.45, $\left(\frac{K_l}{r_y}\right)$ = $\frac{2814}{32.905}$ = $\underline{79.960}$

$4.71\sqrt{\frac{E}{F_y}}$ = $4.71\sqrt{\frac{200000}{248}}$ = 133.755

$\left(\frac{K_l}{r_x}\right)$ < $4.71\sqrt{\frac{E}{F_y}}$ ➔ F_{cr} = $\left(0.658^{\frac{F_y}{F_e}}\right)F_y$

F_e = $\frac{\pi^2 E}{\left(\frac{Kl}{r}\right)^2}$ = $\frac{\pi^2 * 200000}{79.960^2}$ = 308.7339 MPs ➔ F_{cr} = $\left(0.658^{\frac{248}{308.7339}}\right)248$ = 177.189 MPs

$ØP_n$ = 0.9*177.189*(2*1910)*10^{-3} =609.17 kN > P_u OK

∴ Use 2Ls 120x80x10 for members No. (6 , 7 , 8 , 9 , 10 , 27 , 28 , 29 , 30 , 31)

For reversal members 6, 27

Tension force in the reversal members is too small therefore 2Ls 120x80x10 are adequate.

Checking estimated weight of roof truss

Hear we will calculate actual internal force due to the weight of the roof truss using STAAD Pro. Then we will compare it with the internal force due to the estimated load as shown hear in table 8.

Table (8)

member	Actual values		Used values	
	+ve Max Force (kN)	-ve Max Force (kN)	+ve Max Force (kN)	-ve Max Force (kN)
1	280.31	40.38	284.69	43.66
2	280.31	40.38	284.69	43.66
3	490.18	71.32	498.19	77.33

4	490.18	71.32	498.19	77.33
5	466.72	67.46	474.49	73.29
6	2.05	-2.72	2.14	-2.64
7	-63.61	-429.11	-68.67	-435.86
8	-63.40	-429.11	-68.46	-435.86
9	-74.53	-500.40	-80.62	-508.52
10	-74.31	-500.40	-80.40	-508.52
11	-2.15	-18.06	-2.83	-18.97
12	1.04	0.67	0.00	0.00
13	-4.68	-36.11	-6.06	-37.95
14	1.07	0.69	0.00	0.00
15	-4.73	-36.17	-6.06	-37.95
16	1.17	0.75	0.00	0.00
17	-49.03	-327.15	-52.61	-331.92
18	171.03	25.35	173.86	27.48
19	-11.99	-81.19	-12.93	-82.44
20	9.94	1.28	10.15	1.44
21	44.17	6.34	44.73	6.76
22	280.31	38.07	284.87	41.49
23	280.31	38.07	284.87	41.49
24	490.18	70.46	498.25	76.52
25	490.18	70.46	498.25	76.52
26	466.72	67.46	474.49	73.29
27	2.05	-2.72	1.93	-2.84
28	-63.61	-429.11	-68.75	-435.97
29	-63.40	-429.11	-68.54	-435.97
30	-74.53	-500.40	-80.64	-508.55
31	-74.31	-500.40	-80.43	-508.55
32	-2.15	-18.06	-2.85	-18.99
33	1.04	0.67	0.00	0.00
34	-4.68	-36.11	-6.06	-37.95
35	1.07	0.69	0.00	0.00
36	-4.73	-36.17	-6.06	-37.95
37	-49.03	-327.15	-52.53	-331.82
38	171.03	25.35	173.79	27.43
39	-11.99	-81.19	-12.89	-82.38
40	9.94	1.28	10.10	1.40
41	44.17	6.34	44.77	6.79

The table show that our estimate of the roof truss weight is good with no problems.

DESIGN OF THE CONNECTIONS

Bolted connections

$\boxed{L_{10}}$

Member 32, compression -19 kN, L50x50x6

a) bearing strength

L_c =45-(16+4) = 25mm

R_n = 1.2 L_c t F_u ≤ 2.4 d t F_u ➔1.2*25*6*400*10^{-3} ≤ 2.4*16*6*400*10^{-3} ➔ 72<92.16

No. of bolts = $\frac{P_u}{\emptyset R_n}$ = $\frac{19}{0.75\times 72}$ = 0.35 ➔ say 2 bolts as a minimum fastener

b) shearing strength of 2 bolts

$\emptyset R_n$ = 0.75*F_{nv} A_b = 2*0.75*330*$\frac{\pi 16^2}{4}$ * 10^{-3} = 99.5 kN > P_u OK

∴ Use tow bolts 16 mm in diameter.

$\boxed{L_{10}}$

Member 37, compression -331.8 kN, L150x150x12

a) bearing strength of one bolt

L_c =45-(16+4) = 25mm

R_n = 1.2 L_c t F_u ≤ 2.4 d t F_u ➔1.2*25*12*400*10^{-3} ≤ 2.4*16*12*400*10^{-3} ➔ 144<184.32

No. of bolts = $\frac{P_u}{\emptyset R_n}$ = $\frac{331.8}{0.75\times 144}$ = 3.07 ➔ say 4 bolts.

b) shearing strength of 4 bolts

$\emptyset R_n$ = 0.75*F_{nv} A_b = 4*0.75*330*$\frac{\pi 16^2}{4}$ * 10^{-3} = 199.05 kN < P_u ➔ increase the bolts.

No. of bolts reqd. = $\frac{331.8\times 10^3}{0.75\times 330\times\frac{\pi\times 16^2}{4}}$ = 6.67 ➔ say 7 bolts in two lines.

$\boxed{L_{10}}$

Member 22, tension 284.9 kN, 2Ls120x80x8

a) bearing strength of one bolt

L_c =45-(16+4) = 25mm

R_n = 1.2 L_c t F_u ≤ 2.4 d t F_u ➔1.2*25*8*400*10^{-3} ≤ 2.4*16*8*400*10^{-3} ➔ 96<122.88

No. of bolts = $\frac{P_u}{\emptyset R_n}$ = $\frac{284.9}{0.75\times 96}$ = 3.96 ➔ say 4 bolts

b) shearing strength of 4 bolts

$\emptyset R_n$ = 0.75*F_{nv} A_b = 2*4*0.75*330*$\frac{\pi 16^2}{4}$ * 10^{-3} = 398.1 kN > P_u

∴ Use four bolts 16 mm in diameter.

$\boxed{U_{10}}$

Member 27, reversal +1.93 kN and -2.835 kN, 2Ls120x80x10

The same with member 32

∴ Use two bolts 16 mm in diameter.

$\boxed{L_9}$

Member 23, tension +285 kN, 2Ls120x80x8

The same with member 22

∴ Use four bolts 16 mm in diameter.

$\boxed{L_9}$

Member 33, zero member 0 kN, L50x50x6

The same with member 32
∴ Use two bolts 16 mm in diameter.

$\boxed{U_9}$
Member 38, tension +173.8 kN, L60x60x8
∴ Use three bolts 16 mm in diameter (page ##).

$\boxed{U_9}$
Member 28, comp. -436 kN, 2Ls120x80x10

a) bearing strength of one bolt
L_c =45-(16+4) = 25mm
$R_n = 1.2\ L_c\ t\ F_u \leq 2.4\ d\ t\ F_u$ ➔ $1.2*25*10*400*10^{-3} \leq 2.4*16*10*400*10^{-3}$ ➔ 120<153.6
No. of bolts = $\frac{P_u}{\emptyset R_n} = \frac{436}{0.75\times120}$ = 4.84 ➔ say 5 bolts
b) shearing strength of 5 bolts
$\emptyset R_n = 0.75*F_{nv}\ A_b = 2*5*0.75*330*\frac{\pi 16^2}{4} * 10^{-3}$ = 497.6 kN > P_u OK
∴ Use five bolts 16 mm in diameter.

$\boxed{L_8}$
Member 34, comp. -38.9 kN, L60x60x6

a) bearing strength of one bolt
L_c =45-(16+4) = 25mm
$R_n = 1.2\ L_c\ t\ F_u \leq 2.4\ d\ t\ F_u$ ➔ $1.2*25*6*400*10^{-3} \leq 2.4*16*6*400*10^{-3}$ ➔ 72<92.16
No. of bolts = $\frac{P_u}{\emptyset R_n} = \frac{38.9}{0.75\times72}$ = 0.72 ➔ use 2 bolts
b) shearing strength of 2 bolts
$\emptyset R_n$ = 99.5 kN > P_u OK
∴ Use two bolts 16 mm in diameter.

$\boxed{L_8}$
Member 39, comp. -82.4 kN, L100x100x10
∴ Use 2 bolts 16 mm in diameter which adequate comparing with member 34.

$\boxed{U_9}$
Member 24, tension +498.3 kN, 2Ls120x80x8
∴ Use 6 bolts 16 mm in diameter (page ##).

$\boxed{U_8}$
Member 29, comp. -436kN, 2Ls120x80x10

a) bearing strength of one bolt
No. of bolts = $\frac{P_u}{\emptyset R_n} = \frac{436}{0.75\times120}$ = 4.84 ➔ say 5 bolts
b) shearing strength of 5 bolts
$\emptyset R_n$ = 497.6 kN > P_u OK
∴ Use 5 bolts 16 mm in diameter.

$\boxed{L_7}$
Member 35, zero member 0 kN, L50x50x6

The same with member 32
∴ Use 2 bolts 16 mm in diameter.

$\boxed{L_7}$
Member 25, tension +498.3 kN, 2Ls120x80x8
∴ Use 6 bolts 16 mm in diameter (page ##).

$\boxed{U_7}$
Member 40, tension +10.102 kN, L65x65x7
∴ Use 2 bolts 16 mm in diameter (page ##).

$\boxed{U_7}$
Member 30, comp. -508.55kN, 2Ls120x80x10

a) bearing strength of one bolt
No. of bolts = $\frac{P_u}{\emptyset R_n} = \frac{508.55}{0.75\times120}$ = 5.65 ➔ say 6 bolts

b) shearing strength of 6 bolts
$\emptyset R_n = 0.75*2*6*330*\frac{\pi 16^2}{4} * 10^{-3} = 597.1$ kN > P_u OK
∴ Use 6 bolts 16 mm in diameter.

$\boxed{L_6}$
Member 36, comp. -37.95 kN, L65x65x7
∴ Use 2 bolts 16 mm in diameter which adequate comparing with member 34.

$\boxed{L_6}$
Member 41, tension 44.8 kN, L70x70x6
∴ Use 2 bolts 16 mm in diameter (page ##).

$\boxed{L_6}$
Member 26, tension 474.5 kN, 2Ls100x75x10

a) bearing strength of one bolt
No. of bolts = $\frac{P_u}{\emptyset R_n} = \frac{474.5}{0.75\times120}$ = 5.65 ➔ say 6 bolts

b) shearing strength of 6 bolts
$\emptyset R_n = 0.75*2*6*330*\frac{\pi 16^2}{4} * 10^{-3} = 597.1$ kN > P_u OK
∴ Use 6 bolts 16 mm in diameter.

$\boxed{U_6}$
Member 31, comp. -508.55 kN, 2Ls120x80x10
∴ Use 6 bolts 16 mm in diameter the same with member 30.

Gusset plate for bolted connections

As we can see all thicknesses of the members are equal or less than 10 mm (gusset plate thickness) except member 37 has 12mm thickness but shearing strength of bolts is controls and 7 bolts are adequate. Then we will use 10 mm thickness of the gusset plate.

Table (9) summary to bolted connections

Joint	member	section	type	force	No. of bolts
L_{10}	32	L50x50x6	comp.	-19	2
	37	L150x150x12	comp.	-331.8	7
	22	2Ls120x80x8	Tens.	285	4
U_{10}	27	2Ls120x80x10	reversal	+1.9 -2.8	2
L_9	23	2Ls120x80x8	Tens.	285	4
	33	L50x50x6	zero	0	2
U_9	38	L60x60x8	Tens.	173.8	3
	28	2Ls120x80x10	comp.	436	5
L_8	34	L60x60x6	comp.	-38	2
	39	L100x100x10	comp.	-82.4	2
	24	2Ls120x80x8	Tens.	498.3	6
U_8	29	2Ls120x80x10	comp.	-463	5
L_7	35	L50x50x6	zero	0	2
	25	2Ls120x80x8	Tens.	498.3	6
U_7	40	L65x65x7	Tens.	10.102	2
	30	2Ls120x80x10	comp.	-508.3	6
L_6	36	L65x65x7	comp.	-37.95	2
	41	L70x70x6	Tens.	44.8	2
	26	2Ls100x75x10	Tens.	474.5	6
U_6	31	2Ls120x80x10	comp.	-508.55	6

Welded connections

We will use 70EXX (it's strength 483MPs)

$\boxed{L_0}$

Member 11, compression -19 kN, L50x50x6, $\bar{x}$ = 14.5mm

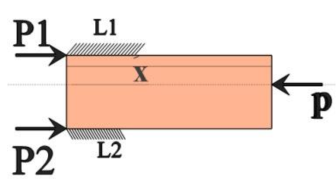
Maximum weld size = 6 mm, minimum weld size = 3mm (AISC manual table J 2.4)

Try with 4mm

$\emptyset R_n = \emptyset F_w A_w$

$= 0.75*0.6*483*(4*0.707)*1*10^{-3} = 0.6147$ kN/mm

Total length of weld required = $\frac{18.97}{0.6147}$ = 30.9 mm, min. length = 4t = 4*6 = 24mm



$\Sigma M_{P1} = 0 \rightarrow 18.97*14.4 = 50\ P_2 \rightarrow P_2 = 5.5$ kN $\rightarrow P_1 = 18.97 - 5.5 = 13.47$ kN

$L_1 = \frac{13.47}{0.6147}$ = 21.9mm say 30mm ($\frac{l}{w} = \frac{21.9}{4} = 5.475 < 100 \rightarrow \beta = 1.0$)

$L_2 = \frac{5.5}{0.6147}$ = 8.9mm say 10mm ($\frac{l}{w} = \frac{8.9}{4} = 2.225 < 100 \rightarrow \beta = 1.0$)

$\boxed{L_0}$

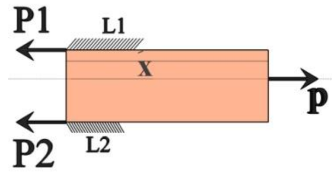
Member 1, tension +284.7 kN, 2Ls120x80x8, $\bar{x}$ = 38.3mm

Maximum weld size = (8–2) =6 mm, minimum weld size = 5mm (AISC manual table J 2.4)

Try with 6mm

$ØR_n = Ø\ F_w\ A_w$

= 0.75*0.6*483*(6*0.707)*1*10^{-3} =0.9220 kN/mm



$ΣM_{P1} = 0$ ➔ 284.7*38.3 = 120 P_2 ➔ P_2 = 90.9 kN ➔ P_1 = 284.7 – 90.9 = 193.8 kN

$L_1 = \frac{193.8}{0.922}$ = 210.1mm say 210mm ($\frac{l}{w} = \frac{210}{6}$ = 35 < 100 ➔ β =1.0)

$L_2 = \frac{90.9}{0.922}$ = 98.6mm say 100mm ($\frac{l}{w} = \frac{100}{6}$ = 16.67 < 100 ➔ β =1.0)

$\boxed{L_0}$

Member 17, compression -331.9 kN, L150x150x12, $\bar{x}$ = 41.2mm

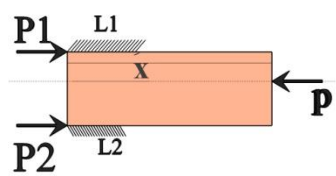
Maximum weld size = (12-2) =10 mm, minimum weld size = 5mm (AISC manual table J 2.4)

Try with 10mm

$ØR_n = Ø\ F_w\ A_w$

= 0.75*0.6*483*(10*0.707)*1*10^{-3} =1.537 kN/mm

$ΣM_{P1} = 0$ ➔ 331.9*41.2 = 150 P_2 ➔ P_2 = 91.16 kN ➔ P_1 = 331.9 – 91.16 = 240.7 kN



$L_1 = \frac{240.7}{1.537}$ = 156.7mm say 160mm ($\frac{l}{w} = \frac{160}{10}$ = 16 < 100 ➔ β =1.0)

$L_2 = \frac{91.16}{1.537}$ = 8.9mm say 60mm ($\frac{l}{w} = \frac{60}{10}$ = 6 < 100 ➔ β =1.0)

$\boxed{U_0}$

Member 6, reversal +2.142 kN and -2.640 kN, 2Ls120x80x10, $\bar{x}$ = 39.2mm

Maximum weld size = (10–2) =8 mm, minimum weld size = 5mm (AISC manual table J 2.4)

Try with 5mm

$ØR_n = Ø\ F_w\ A_w$

= 0.75*0.6*483*(5*0.707)*1*10^{-3} =0.7683 kN/mm

Total length of weld required = $\frac{2.64}{0.7683}$ = 3.4 mm, min. length = 4t = 4*5 = 20mm

L_1 = 10mm, L_2 = 10mm

$\boxed{L_1}$

Member 12, zero 0 kN, L50x50x6, $\bar{x}$ = 14.5mm

Use minimum weld size, minimum weld size = 3mm (AISC manual table J 2.4)

Min. length = 4t = 4*3 = 12mm

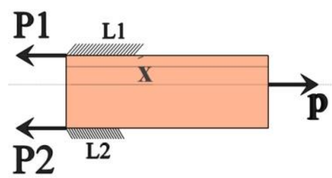
L_1 = 10mm, L_2 = 10mm

$\boxed{L_1}$

Member 2, tension +284.7 kN, 2Ls120x80x8, $\bar{x}$ = 38.3mm
Use L_1 = 10mm, L_2 = 10mm the same with member 1

$\boxed{U_1}$

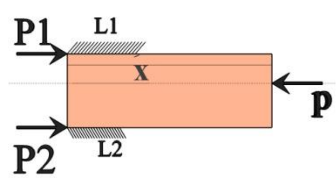
Member 18, tension +173.9 kN, L60x60x8, $\bar{x}$ = 16.9mm
Maximum weld size =6 mm, minimum weld size = 5mm (AISC manual table J 2.4)
Try with 6mm
$ØR_n$ = 0.9220 kN/mm



$\Sigma M_{P1} = 0$ ➔ 173.9*16.9 = 60 P_2 ➔ P_2 = 48.98 kN ➔ P_1 = 173.9 - 48.98 = 124.92 kN
$L_1 = \frac{124.92}{0.922}$ = 135.5mm say 140mm ($\frac{l}{w} = \frac{140}{6}$ = 23.33 < 100 ➔ β =1.0)
$L_2 = \frac{48.98}{0.922}$ = 53.1mm say 60mm ($\frac{l}{w} = \frac{60}{6}$ = 10 < 100 ➔ β =1.0)

$\boxed{U_1}$

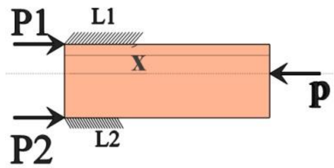
Member 7, compression -435.66 kN, 2Ls120x80x10, $\bar{x}$ = 39.2mm
Maximum weld size = (10–2) =8 mm, minimum weld size = 5mm (AISC manual table J 2.4)
Try with 8mm
$ØR_n = Ø F_w A_w$
$= 0.75*0.6*483*(8*0.707)*1*10^{-3}$ =1.229 kN/mm
$\Sigma M_{P1} = 0$ ➔ 39.2*435.86 = 120 P_2 ➔ P_2 = 142.38 kN ➔ P_1 = 435.86-142.38 = 293.48 kN



$L_1 = \frac{293.48}{1.229}$ = 238.8mm say 240mm ($\frac{l}{w} = \frac{240}{8}$ = 30 < 100 ➔ β =1.0)
$L_2 = \frac{142.38}{1.229}$ = 115.9mm say 120mm ($\frac{l}{w} = \frac{120}{8}$ = 15 < 100 ➔ β =1.0)

$\boxed{L_2}$

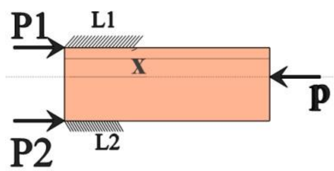
Member 19, compression -82.44 kN, L100x100x10, $\bar{x}$ = 39.2mm
Maximum weld size = (10–2) =8 mm, minimum weld size = 5mm (AISC manual table J 2.4)
Try with 6mm
$ØR_n = Ø F_w A_w$ = 0.9220 kN/mm
$\Sigma M_{P1} = 0$ ➔ 82.44*28.2 = 100 P_2 ➔ P_2 = 23.25 kN ➔ P_1 = 59.19 kN



$L_1 = \frac{59.19}{0.922}$ = 64.2mm say 70mm, $L_2 = \frac{23.25}{0.922}$ = 25.2mm say 30mm

$\boxed{L_2}$

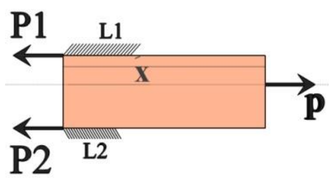
Member 13, compression -37.947 kN, L60x60x6, $\bar{x}$ = 16.9mm
Try with 4mm
$ØR_n = Ø\ F_w\ A_w$ = 0.6147 kN/mm
$\Sigma M_{P1} = 0$ ➔ 37.947*16.9 = 60 P_2 ➔ P_2 = 10.69 kN ➔ P_1 = 27.26 kN



$L_1 = \frac{27.26}{0.6147}$ = 44.35mm say 50mm, $L_2 = \frac{10.69}{0.6147}$ = 17.8mm say 20mm

$\boxed{L_2}$

Member 3, tension +498.19 kN, 2Ls120x80x8, $\bar{x}$ = 38.3mm
Maximum weld size =6 mm, minimum weld size = 5mm (AISC manual table J 2.4)
Try with 6mm
$ØR_n$ = 0.9220 kN/mm



$\Sigma M_{P1} = 0$ ➔ 498.19*38.3 = 120 P_2 ➔ P_2 = 159 kN ➔ P_1 = 339.18 kN
$L_1 = \frac{339.18}{0.922}$ = 367.9mm say 370mm ($\frac{l}{w} = \frac{370}{6}$ = 61.67 < 100 ➔ β =1.0)
$L_2 = \frac{159}{0.922}$ = 172.5mm say 180mm ($\frac{l}{w} = \frac{180}{6}$ = 30 < 100 ➔ β =1.0)

$\boxed{U_2}$

Member 8, L_1 = 120mm, L_2 = 240mm the same with member 7

$\boxed{L_3}$

Member 4, L_1 = 50mm, L_2 = 20mm the same with member 3

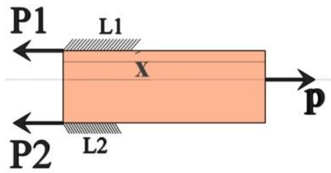
$\boxed{L_3}$

Member 14, L_1 = 10mm, L_2 = 10mm the same with member 12

$\boxed{U_3}$

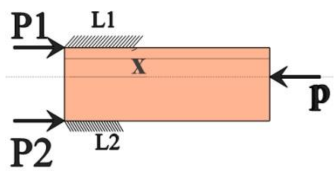
Member 20, tension +10.15 kN, L65x65x7, $\bar{x}$ = 18.5mm

Using minimum weld size = 5mm (AISC manual table J 2.4)
$\text{Ø}R_n = 0.7683$ kN/mm



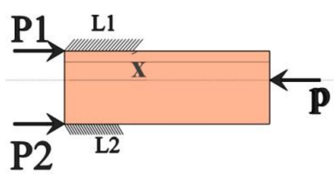
Total length = $\frac{10.15}{0.7683}$ = 13.21mm, minimum length of weld = 4w = 4*5 = 20mm 🡨
$\Sigma M_{P1} = 0$ ➔ 20*18.5 = 65 L_2 ➔ L_2 =5.7mm say 10mm
L_1 =20 – 5.7 = 14.3mm, say 20mm

$\boxed{U_3}$
Member 9, compression -508.5 kN, 2Ls120x80x10, $\bar{y}$ = 39.2mm
Use maximum weld size 10 -2 = 8mm
$\text{Ø}R_n = \text{Ø}\, F_w\, A_w = 1.229$ kN/mm



$\Sigma M_{P1} = 0$ ➔ 508.5*39.2 = 120 P_2 ➔ P_2 = 166.11 kN ➔ P_1 = 342.39 kN
$L_1 = \frac{342.39}{1.229}$ = 278.6mm say 280mm, $L_2 = \frac{166.11}{1.229}$ = 135.2mm say 140mm

$\boxed{L_4}$
Member 15, compression -37.95 kN, L70x70x6, $\bar{x}$ = 19.3mm
Try with 3mm (AISC manual table J 2.4)
$\text{Ø}R_n = \text{Ø}\, F_w\, A_w = 0.75{*}0.6{*}483{*}3{*}0.707{*}10^{-3} = 0.4610$ kN/mm
$\Sigma M_{P1} = 0$ ➔ 37.95*19.3 = 70 P_2 ➔ P_2 = 10.46 kN ➔ P_1 = 27.49 kN



$L_1 = \frac{27.49}{0.4610}$ = 59.6mm say 60mm, $L_2 = \frac{10.46}{0.4610}$ = 22.7mm say 30mm

Note

Members 5, 21 and 10 prefer to use bolted connection for erection purpose the same with members 26, 41, and 31 respectively.

Table (10) summery to welded connections

Joint	Member	Section	w (mm)	L_1 (mm)	L_2 (mm)
L_0	11	L50x50x6	4	30	10
	1	2Ls120x80x8	6	210	100
	17	L150x150x12	10	160	60
U_0	6	2Ls120x80x10	5	10	10
L_1	12	L50x50x6	3	10	10
	2	2Ls120x80x8	6	210	100
U_1	18	L60x60x8	6	140	60
	7	2Ls120x80x10	8	240	120
L_2	19	L100x100x10	6	70	30
	13	L60x60x6	4	50	20
	3	2Ls120x80x8	6	370	180
U_2	8	2Ls120x80x10	8	240	120
L_3	4	2Ls120x80x8	6	370	180
	14	L50x50x6	3	10	10
U_3	20	L65x65x7	5	20	10
	9	2Ls120x80x10	8	280	140
L_4	15	L70x70x6	3	60	30

As we were drawing we noted that, some members can't install like members 17 and 19. Then we will recalculate their lengths as shown hear.

$\boxed{L_0}$

Member 17, w = 10 mm

160mm length provide 0.75*0.6*483*10*0.707*160*10^{-3} = 245.87 kN

⇨ P_2 = 331.9 – 245.87 = 86.03 kN

The resistance provided by 1mm length in the transverse direction equal

= 0.75*0.6*483*(1.0+0.5 sin$^{1.5}$90^o)*(10*0.707)*1*10^{-3}

= 2.305 kN/mm

Reqd. length = $\frac{86.03}{2.305}$ = 37.32 mm say 40 mm

Check $R_u = R_{wl} + R_{wt}$ or $R_u = 0.85\ R_{wl} + 1.5\ R_{wt}$

= 1.537*160+2.305*40 = 338.12 kN >P_u OK

= 0.85(1.537*160) + 1.5(2.305*40) = 347.33 kN > P_u OK

Use 40mm transverse weld

$\boxed{L_2}$

Member 19

L = 70mm ➔ ØR_n = 0.922*70 = 64.54 kN

The resistance provided by 1mm length in the transverse direction equal

= 0.75*0.6*483*(1.0+0.5 sin$^{1.5}$90^o)*(10*0.707)*1*10^{-3}

= 1.383 kN/mm

Reqd. length = $\frac{82.44-64.54}{1.383}$ = 12.9 mm say 20 mm

Check $R_u = R_{wl} + R_{wt}$ or $R_u = 0.85\ R_{wl} + 1.5\ R_{wt}$

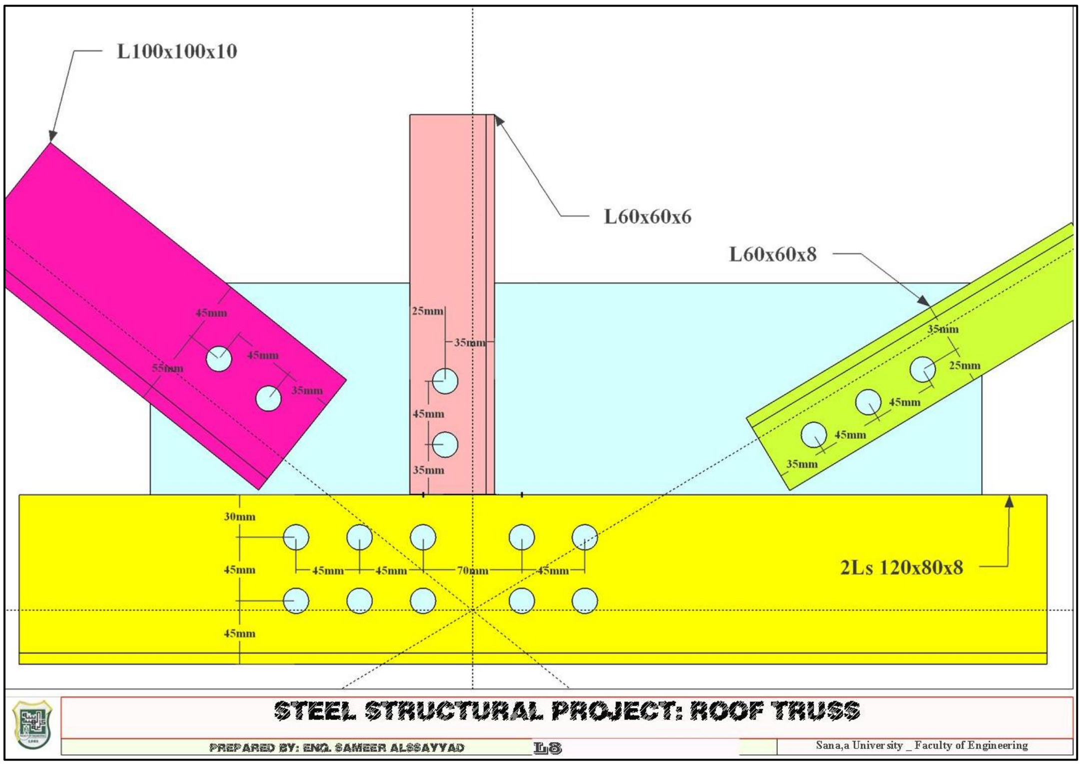
= 0.922*70+1.383*20 = 92.2 kN >P_u OK

= 0.85(0.922*70) + 1.5(1.383*20) = 96.35kN > P_u OK
Use 20mm transverse weld

Drawing of the connections

The drawings are show in figure 17 as a sample for bolted connections, a sample for gusset plates in figure 18 and a sample for welded connections in figure 19. All the connections and their gusset plates are presented in PDF file also in JPEG and PNG format with the project.

Fig. (17) bolted connection L_8



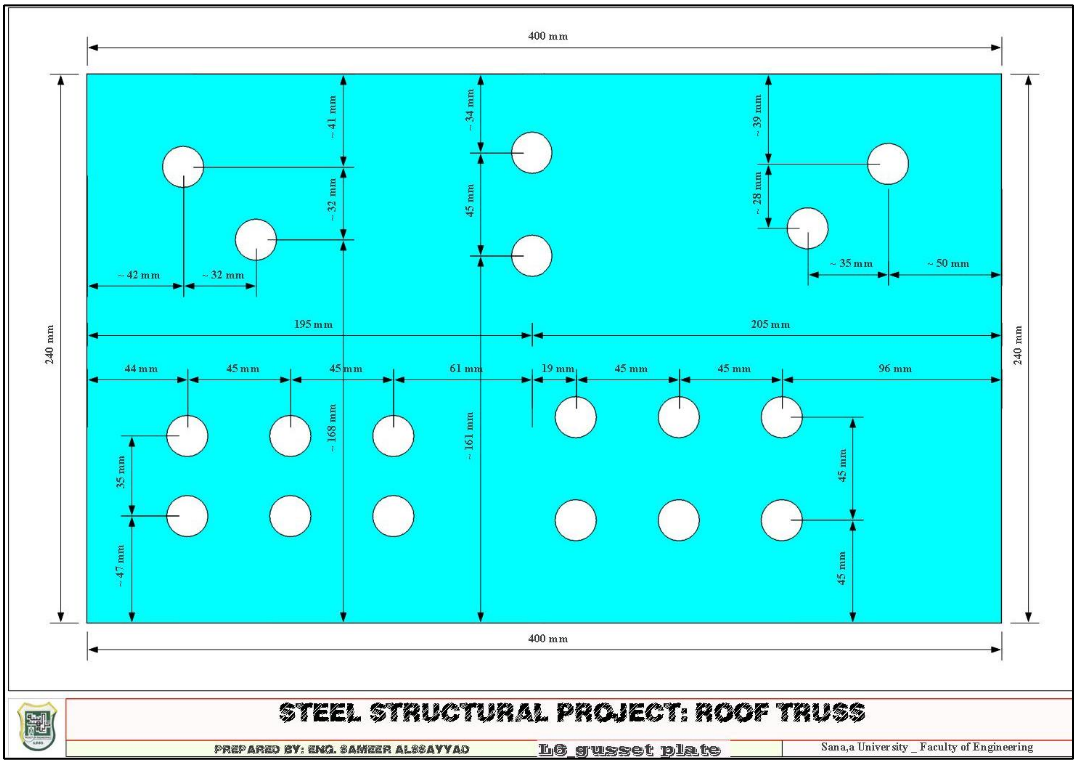
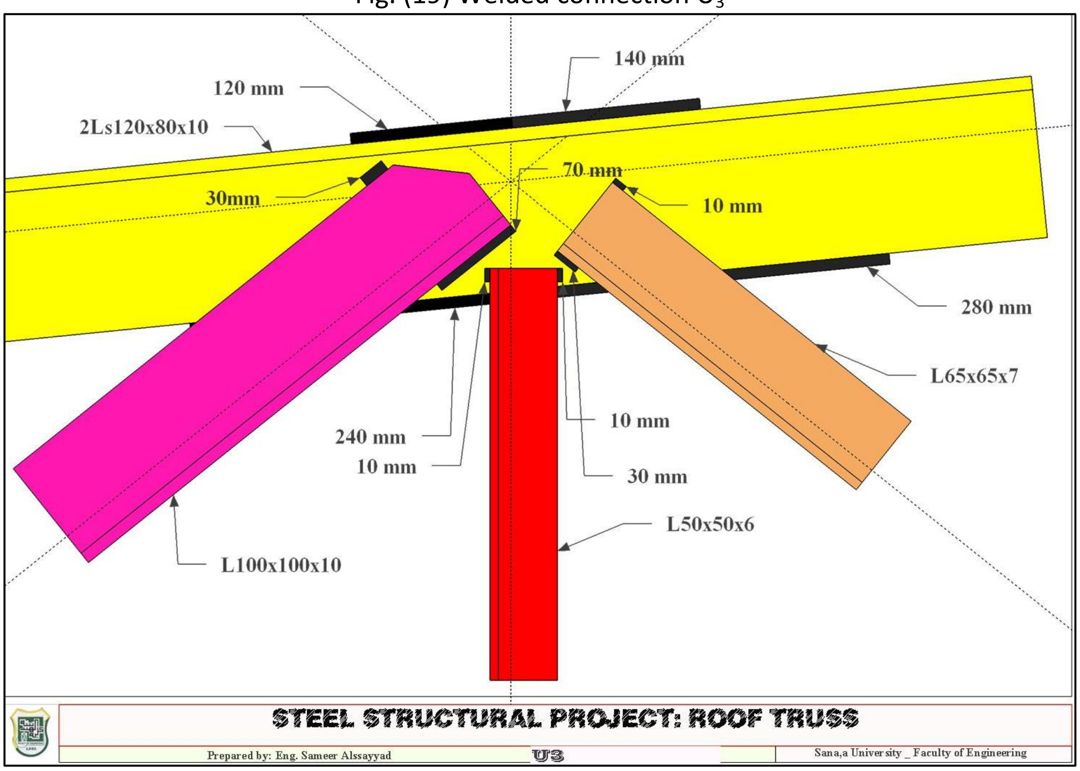


Fig. (18) Gusset plate for L_6

Fig. (19) Welded connection U_3



The bracing system of the building

As we assumed the building doesn't use with vibration loading then we can use bracing in the upper chord only every four pays as shown in figures 2 and 3.

References

1. STRUCTURAL STEEL DESIGN, Fourth Edition, Jack C. mccormac.
2. STEEL CONSTRUCTION MANUAL Thirteenth edition.
3. STRUCTURAL STEEL DESIGN, Second Edition, Jack C. mccormac chapter 17.
4. ASCE 7-05 Minimum Design Loads for buildings and other Structures.
5. Guide to the Use of the Wind Load Provisions of ASCE 7-02.
6. Structural steelwork design to limit state theory Second Edition.
7. AISC design examples.

Contents