Analysis and Design of Structural Components for Industrial Steel Structure by using Software Package STAAD PRO.V8i

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ABSTRCT : Structural Steel is a common building material used throughout the construction industry. Its primary purpose is to form a skeleton for the structure, essentially the part of the structure that holds everything up and together. Steel is one of the friendliest environmental materials which are 100% recyclable. Structural design has evolved, mostly due to the necessity caused by earthquakes. The erudition of steel gives architects, and the freedom was to achieve the most ambitious visions. Steel is also one of the most sustainable construction materials, building owners naturally value the flexibility of steel buildings in addition the value of benefits they provide. Steel is ideal for modernization, reconfiguring, extending or adapting with minimal disruption. The conception of design analysis as well as modeling of steel structures is the most up-to-date edition in the civil engineering field. It is necessary to model a steel structure but if it is also analyzed during its modeling then there will not be any chances of failure. In this project work, a general building data is adopted for the study and is well analyzed and designed. The project was undertaken at Belagavi location. The analysis and designing was done according to the standard specification to the possible extend. The analysis of structure was done using the software package STAAD PRO.V8i. All the structural components were designed manually.

1. INTRODUCTION

The structural engineer must contemplate the resulting points during the design and construct of industrial buildings; Selection of roof, material, bay width, structural framing systems, Roof trusses, Purlins, girts, sag rods, Bracing system, Gantry girders, columns, base plate, and foundation. Steel frame construction offers many advantages over traditional reinforced concrete with lower costs, sustainability and flexibility being among the many gains of selecting steel framed buildings over the alternatives. Usually, the bays in industrial buildings have frames spanning the width direction. Some such frames are planned at suitable spacing out to get the requisite length. Reliant upon the requisite, several bays may be constructed adjacent each other. The choice of structural design differs upon the span between the rows of the columns, the head room or clearance required the nature of roofing material and type of lighting. The roof system is one of the most expensive parts of the entire structural system even though walls are more expensive per square meter. The roof often covers much larger area. The predominant roof system in industrial buildings involves the use of metal deck which is cold rolled from sheet steel in various gage thicknesses. The inclusion of cranes will generally not affect the basic roof covering system. Industrial operations can be carried on most efficiently when adequate illumination is provided. The requirements of good lighting are its intensity and uniformity. Since natural light is free, it is economical and wise to use daylight most satisfactory for illumination in industrial plants whenever practicable. Side windows are much value in lighting the interiors of small buildings, but they are not much effective in case of large buildings. In case of large buildings monitors are useful. Ventilation of industrial buildings is also important. Ventilation will be used for removal of heat, elimination of dust, used air and its replacement by clean fresh air. It can be done by means of natural forces such as aeration or by mechanical equipment such as fans. The large height of the roof may be used advantageously by offering low level outlets of air.

Trusses are triangular frame works in which the members are imperiled to basically axial forces due to externally applied load. They may be plane trusses [fig. 1(a)], where in the external load and the members lie in the same plane or space trusses [fig.1 (b)], in which members are oriented in three dimensions in space and loads may also act in any path. Trusses are often employed to span long lengths in the place of solid web girders and such trusses are also mentioned to as lattice girders. Steel members imperiled to axial forces are usually more effective than members in the flexure since the cross section is almost equally stressed. Trusses, comprising of basically axially loaded members, thus are very efficient in resisting external loads. They are widely used, particularly to span large gaps. Since truss systems use comparatively less material and more labor to fabricate, related to other systems, they are especially suited in the Indian context.

This results in a much better quality of the fabricated structure.

- Method of fabrication and erection to be followed, facility for shop fabrication available, transportation restrictions, field assembly facilities.
- Preferred practices and experience.
- Availability of materials and sections to be used in fabrication.
- Erection technique to be followed and erection stresses.

- Method of connection preferred by the contractor and client (bolting, welding, or riveting).
- Choice of as rolled or fabricated sections.
- Simple design with maximum repetition and minimum inventory of material.

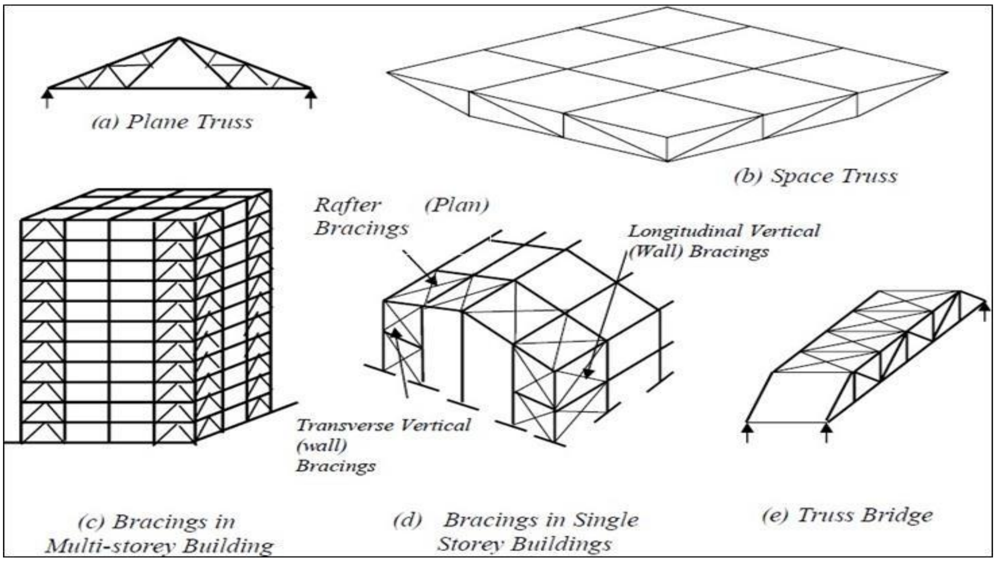

Fig.1: Types of Trusses

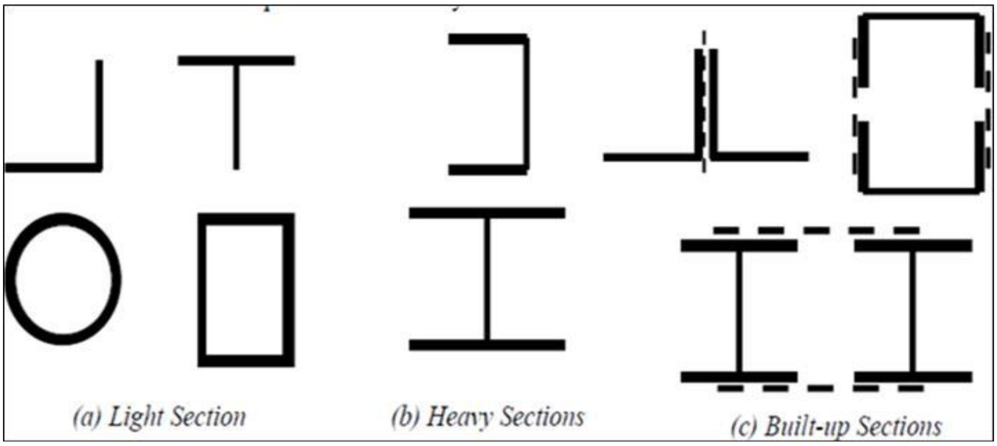

Fig. 2: Types of steel sections for Truss members

2. METHODOLGOY

2.1. Essence of the Present study

The main objective of the present study is to plan, analyze and design an industrial steel structure. For this purpose, STAAD.Pro software package v.8i has been made use of. The general features of the structure are given in Tables 1 and 2. The various views of the structure given by the STAAD.Pro software are depicted in Figures.

Table 1: General Building Data adopted for the study

Feature	Description
Location	Belagavi
Type of Structure	Industrial Steel Structure
Functionality	Godown
Total Site Area	165.6 Sq. m
Size of the Structure	16.35m * 10.13m
Total Height of Frame	8m
Height of the Truss	2m
Height of Truss Column	6m
Span of the Truss	10.13m

No. of Trusses	05
Spacing of Trusses	4.09m
Spacing of Purlins	1.36m

Table.2 Design data for Roof truss design

DESIGN DATA:						
*	Truss span (l)			=	10.13	m
*	Height of column			=	6	m
*	Type of roofing = AC/GI Sheets (1.6mm thick)					
*	Roof angle			=	21.54	degree
*	Spacing between the bay= 18x30=540cm(L)			=	4.09	m
*	Spacing between the purlins			=	1.36	m
*	Size of GI sheet			=	2.2x0.9	m
*	Weight of sheet			=	156	N/mm^2
*	Assuming Rise of truss (h) (Central span)			=	2	m
		$\sqrt{(h^{\wedge}}$	2 (1/2))			
*	Inclined length =			=	5.45	m
*	Roof angle = tanθ =	Central span x(Span / 2)		=	0.395	
				θ =	21.54	
					0.31939525	Radians
*	Spacingofpurlins=	(Inclined		=	1.361	m
*	Therefore number of purlins			=	5	no
*	Yeild stress of materials(f_y)			=	250	N/mm^2
*	Ultimate stress of materials(f_u)			=	400	N/mm^3

2.2. PURLINS

A purlin is a horizontal structural member in a roof. Purlins support the load from the roof deck or sheathing and supported by the principal rafters and building walls, steel beams etc. Purlins are beam used on trusses to support the sloping roof system between the adjacent trusses. Channels, angle sections, and cold formed C or Z sections are widely used as purlins. They are placed in an inclined position over the main rafters of the trusses. To avoid bending in the top chords of roof trusses, it is theoretically desirable to place purlins only at panel points. For larger trusses, however, it is more economical to place purlins only at closer intervals.

2.2.1 Design of Purlins

Bending moment on continuous span (Mu):

$$Mu = (1.5 \times W \times L^2)/10$$

$$\mathbf{Mu = 2.645\ kN\text{-}m}$$

Shear force on continuous span (Vu):

$$Vu = (1.5 \times W \times L)/2$$

$$\mathbf{Vu = 3.233\ KN}$$

Now, Adopting

Angle From steel table

ISA - 125x75x10 mm

Properties from steel table

b=	125	mm	
t_f=	10	mm	
l=	75	mm	
A=	19.02	cm^2	1902 mm^2
Weight=	14.9	kg	146.169 N
Z_{xx}=	36.3	cm^3	36300 mm^3
Z_{yy}=	14.2	cm^3	14200 mm^3
f_{yw}=	250	N/mm^2	

Referring to IS - 800-2007

Table -2 clause 3.7.2

(b_f/t_f)= 12.5

Therefore section is **Semi compact section**

* **Check for Shear Capacity** :

From clause 8.4.1 of IS-800-2007

$$V \le V_d$$

$$V_d = V_n / \Upsilon_{mo}$$

$$V_n = (A_v \times f_y)/(\sqrt{3})$$

V= Design shear force

V_n= Normal plastic shear resistance

Υ_{mo}= Yield Strength of web = 1.1

V_d= Design shear Strength

A_v= Shear area

$V_n = (1902 \times 250)/(\sqrt{3})$

V_n= 274530.053 N

V_n= 274.530053 kN

V_d= 274.530 / 1.1

V_d= 249.5727755 kN

Since Vd › Vu the Section adopted is safe in shear.

* **Check for Moment Capacity:**

$M = (Z_p \times f_y)/\Upsilon_{mo}$

Z_p= Plastic section modulus

f_y= Yield stress =250 N/mm^2

M= (36300x 250)/1.1

M= 8250000 N-mm

M= 8.25 kN-m

Since M › M_u the Section adopted is safe in bending.
Therefore adopt the purlin section ISA- 125x75x10 mm

2.3 Design of Truss Members

2.3.1 Top Chord Member (Principle Rafter)

* Members : AB, BC, CD
* Compression force = -47.137 kN
* Factored Compression force= 1.5x-47.137
  = -70.7055 kN
* Tension force = 91.18 kN
* Factored tension force = 1.5x91.18
  136.77 kN
* Length of top chord (L) = 1.36 m 1360 mm
* Effective length (l) = 0.85 x L
  l= 1.020 m

Since the tension force is more than the compression force , so first design the section for tension force and then check for compression force.

2.3.2 Check for Compression

Design compressive strength (P_d) = Axf_{cd}

Slenderness ratio = KL/r_{min}

Cross sectional area required:

A_{req}= (Force)/f_{cd}

Assume, **f_{cd}= 90 N/mm²**

70.705x1000/90

A_{req}= 785.617 mm^2

The Principle Rafter is an Equal angle section

Try, ISA 70x70x6 A= 806 mm^2

Therefore,

$C_{xx}=C_{yy}=$ 19.4 mm
$e_{xx}=e_{yy}$ 50.6 mm
$r_{min}=r_{xx}=r_{yy}=$ 21.4 mm

Slendernessratio(λ) =0.85x1360/21.4

λ= **54.019**

Refering to IS-800:2007 table 9(c) for fy 250

λ	f_{cd}
50	183
54.019	?
60	168

By interpolating

f_{cd}= 176.9715 N/mm^2

Design compressive strength (Pd) =142639.029 N/mm^2
142.639029 kN/mm^2

Since the design compressive strength is > the compressive force in member the section is safe.

2.4 Design of Tension Member

Connection Detail:

Assuming M16 bolts of grade 4.6 and 6mm thick gusset plate.

d=	16	mm
d_0=	18	mm
f_{ub}=	400	N/mm^2
f_u=	410	N/mm^2
fy=	250	N/mm^2
γ_{mb}=	1.25	
γ_{mo}=	1.1	

Bolt Stregth (BS) is least of following two,

* **Shearing Capacity of bolts:(V_{dsd})**

$$V_{dsd} = \frac{f_{ub}}{\sqrt{3}\,\gamma_{mb}}\,[n_n \times A_{nb} + n_s \times A_{sb}]$$

$$= 400/(\sqrt{3} \times 1.25)\,[(1 \times 0.78 \times \pi/4 \times 〖16〗^2) + (1 \times \pi/4 \times 〖16〗^2)]$$

V_{dsd}= 66121.15299 N/mm^2
66.12115299 kN/mm^2

* **Bearing capacity of bolt(V_{dpb}):**

$$V_{dpb} = (2.5 \times k_b \times d \times t \times f_u/\gamma_{mb}$$

Assume,

Pitch (P) =	2.5d	40 mm
End distance=	1.7d_0	30.6
		35 mm

k_b= $e/3d_0$, $p/(3d_0)-.25$, f_{ub}/f_u

k_b= Least of above three.

k_b= 35/3x18 , 40/3x18-0.25, 400/410

k_b= **0.648** **0.49** **0.976**

Therefore, **k_b= 0.49**

V_{dpb}= (2.5 x 0.52x 18 x 6 x410)/1.25

V_{dpb}= 38631.11111 N/mm²

V_{dpb}= 38.631 kN/mm²

Therefore the BS = 38.631 kN/mm²

Number of bolts = Force/BS

Number of bolts = 1.830

Therefore provide 3 nos of 16mm dia bolts.

2.4.1 Check for Tension Member

*** Tensile strength in tearing(T_{dg}):**

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

806x250/1.1

T_{dg}= 183181.8182 N

183.182 kN >103.314

Therefore safe in tearing

*** Tensile strength Due to rupture of critical section(T_{dn}):**

T_{dn}= $(0.9xA_{nc}xf_u/\gamma_{ml}+ \beta xA_{go}xf_y/\gamma_{mo})$x 2

β= 1.4-(0.075x w/t x f_y/f_u x b_s/L_c)x2

w= 70 mm

t= 6 mm

b_s= 83.4 mm

L_c= 80 mm

β= 1.4-(0.075x 70/10 x 250/410 x 83.4/80)

β= 0.844

A_{nc}= (90-10/2 -26) x10

A_{nc}= 490 mm²

A_{go}= (90-10/2)x10

A_{go}= 670 mm²

T_{dn}= (0.9x590x410/1.25+ 1.037x850x250/1.1)x2

T_{dn}= 546267.8334 N/mm²
546.2678334 kN/mm² >103.314 kN/mm² Therefore safe in critical rupture of section.

*** Tensile strength in block shear(T_{bd}):**

$T_{bd1} = (A_{vg} \times f_y / (\sqrt{3}\ \gamma_{mw}) + 0.9 \times A_{tn} \times f_u / \gamma_{ml}) \times 2$

$T_{bd2} = (0.9 \times A_{vn} \times f_u / (\sqrt{3}\ \gamma_{ml}) + A_{tg} \times f_y / \gamma_{mw}) \times 2$

A_{vg}= Gross shear area.
= (40+40+35)x6 **690 mm²**
A_{vn}= Net shear area.
= (40+40+35-(2.5x18)) **420 mm²**
A_{tg}= Gross area in tension.
= (50.6x6) **303.6 mm²**
A_{tn}= Net area in tension.
= (50.6-18x0.5)x6 **225.6 mm²**

Now,
T_{bd1}= 314272.3634 kN
T_{bd1}= 314.2723634 kN
T_{bd2}= 284126.8844 kN
T_{bd2}= 284.127 kN

Therefore, **T_{bd}= 284.127 kN >103.29 kN**
Therefore, safe in block shear.

Since the section is safe in both compression and tension, therefore the section provided ISA-70x70x6 for principle rafter is safe.

2.4.2 Bottom Chord (Main TIE):

* Members : AM, MN, NO
* Tension force = 43.28 kN
* Factored tension force = 1.5x43.28
64.92 KN
* Length of bottom chord (L)= 2.53 m 2530
* Effective length (l) = 0.85 x L
l= 1.898 m

Since only tensile force exists, therefore check and design for tensile force

2.4.3 Check in Tensile Force:

*** Connection Detail:**

Assuming M16 bolts of grade 4.6 and 6mm thick gusset plate.

d=	16	mm
d_0=	18	mm
f_{ub}=	400	N/mm²
f_u=	410	N/mm²
f_y=	250	N/mm²
γ_{ml}=	1.25	
γ_{mo}=	1.1	

Bolt Stregth (BS) is least of following two,
* **Shearing Capacity of bolts:(V_{dsd})**

$V_{dsd} = (f_{ub})/(\sqrt{3}\ \gamma_{mb})\ [n_n x A_b + n_s x A_{sb}]$

400/(√3 x1.25) [(1x0.78x π/4 x 〖16〗 ^2)+(1x π/4 x 〖16〗 ^2)]

V_{dsd}= 66121.15299 N/mm^2

66.121 kN/mm^2

* **Bearing capacity of bolt(V_{dpb}):**

$V_{dpb} = (2.5 \times k_b \times d \times t \times f_u)/\gamma_{mb}$

Assume,

Pitch (P) =	2.5d	40 mm
End distance=	$1.7d_0$	30.6
		35 mm

k_b= $e/3d_0$, $p/3d_0$–.25 ,f_{ub}/f_u

k_b= least of above three.

k_b= 35/3x 18 , 40/3x18–.25, 400/410

Therefore, k_b= 0.648 0.49 0.976

k_b= 0.491

V_{dpb}= 2.5x0.491x16x6x410/1.25

V_{dpb}= 38631.11111 N/mm²

38.63111111 kN/mm²

Therefore the BS = 38.63111111 kN/mm²

Number of bolts = Force/BS

= 64.92/38.631

Number of bolts = 1.681

Thererfore provide 3 nos of 16mm dia bolts.

2.5 Design of tension member:

Cross sectional area required:

T_{dg}= A_g= $A_g x f_y/\Upsilon_{mo}$

A_g= ("T_{dg}" xΥ_{mo})/f_y

64.92x10³x1.1/250

$A_g =$ 285.648 mm^2

The bottom chord is of Double Equal angle sectio Connected back to back to longer legs.

Try, ISA 45x45x4

$A=$	347	mm^2
$C_{xx}=C_{yy}=$	12.5	mm
$e_{xx}=e_{yy}$	32.5	mm
$r_{min}=r_{xx}=r_y$	13.7	mm
$y=$		

* **Tensile strength Due to rupture of critical section(T_{dn}):**

$T_{dn} =$ $(0.9 x A_{nc} x f_u/\gamma_{ml} + \beta x A_{go} x f_y/\gamma_{mo})$

$\beta =$ $1.4-(0.075 x\ w/t\ x\ f_y/f_u\ x\ b_s/L_c)$

$w=$	45	mm
$t=$	4	mm
$b_s=$	53.5	mm
$L_c=$	100	mm

$\beta =$ $1.4-(0.075 x 45/4 x 250/410 x 53.5/100)$

$\beta =$ **1.125**

$A_{nc} = (45-4/2-18)\ x4$

$A_{nc} =$ **100 mm^2**

$A_{go}(=45-4/2)\ x4$

$A_{go} =$ **172 mm^2**

$T_{dn} =$ (0.9x100x410/1.25 +
0.837x172x250/1.1)

$T_{dn} =$ 73487.58938 N/mm^2
73.488 kN/mm^2 >64.92 kN/mm^2
Therefore safe in critical rupture of section.

* **Tensile strength in block shear (T_{bd}):**

$T_{bd1} = (A_{vg} x f_y/(\sqrt{3})\ \gamma_{mw}) + 0.9 x A_{tn} x f_u/\gamma_{ml})\ x\ 2$

$T_{bd2} = (0.9 x A_{vn} x f_u/(\sqrt{3})\ \gamma_{ml}) + A_{tg} x f_y/\gamma_{mw})\ x\ 2$

$A_{vg} =$ Gross shear area.
= ((40x2)+35)x6 **1410 mm^2**
$A_{vn} =$ Net shear area.
= 40x2)+45-(2.5x18))x6 **420 mm^2**
$A_{tg} =$ Gross area in tension.
= (32.5x10) **325 mm^2**
$A_{tn} =$ Net area in tension.
= (32.5-18x0.5)x6 **141 mm^2**

Now, Therefore,

T_{bd1}= 453275.6087 N/mm²
T_{bd1}= 453.276 kN/mm²
T_{bd2}= 292686.8844 N/mm²

T_{bd2}= 292.687 kN/mm²
T_{bd}= 292.687 kN/mm² >64.92 kN/mm²
Therefore safe in blockshear.

Since the section is safe in tension .
Therefore provided ISA-45x45x6 for bottom chord is safe.

2.6 Diagonal Members:

* Members : BM, CL, CO, DO.
* Compression force = -28.06 kN
* Factored compression force= 1.5x-28.06
= -42.09 kN
* Tension force = 18.64 kN
* Factored tension force = 1.5x18.64
27.96 kN
* Length of top chord (L) = 1.36 m 1360 mm
* Effective length (l) = 0.85 x L
l= 1.020 m

Since the compression force is more than the tensile force , so first design the section for compression force and then check for tensile for

2.7 Design of Compression Member:

Design compressive strength (Pd) = $A x f_{cd}$

Slenderness ratio (λ) KL/r_{min}

Cross sectional area required:

A_{req} = (Force) / f_{cd}

= $42.09 x 10^3/150$

= 280.600 mm^2

The diagonal members may be of single angle section.
Try ISA 45x45x4

A= 347 mm^2
$C_{xx}=C_{yy}$= 12.5 mm
$e_{xx}=e_{yy}$ 32.5 mm
$r_{min}=r_{vv}$ or r_{uu}= 13.7 mm

Slenderness ratio(λ) = 0.85x1360 / 13.7

(λ)= 84.380

Refering to IS-800:2007 table 9(c) for fy 250

λ	f_{cd}
80	136
84.380	?
90	121

By interpolating

f_{cd}= 129.431 N/mm²

Design compressive strength (Pd) = 44912.43796 N/mm^2

= 44.912 kN/mm^2

Since the design compressive strength is > the compressive force in member hence the section is safe.

2.7 Check for Tension Force:

*** Connection Detail:**

Assuming M16 bolts of grade 4.6 and 6mm thick gusset plate.

d= 16 mm

d_0= 18 mm

f_{ub}= 400 N/mm^2

f_u= 410 N/mm^2

f_y= 250 N/mm^2

γ_{ml}= 1.25

γ_{mo}= 1.1

Bolt Stregth (BS) is least of following two,

* **Shearing Capacity of bolts:(V_{dsd})**

$$V_{dsd} = (f_{ub})/(\sqrt{3}\,\gamma_{mb})[n_n x A_b + n_s x A_{sb}]$$

$$= 400/(\sqrt{3}\,x1.25)\,[(1x0.78x\,\pi/4\,x\,〖16〗\,^2)]$$

V_{dsd}= 28974.438 N/mm^2

28.974 kN/mm^2

* **Bearing capacity of bolt(V_{dpb}):**

$$V_{dpb} = (2.5\,x\,k_b\,x\,d\,x\,t\,x\,f_u)/\gamma_{mb}$$

Assume,

Pitch (P)=	2.5d	40 mm
End distance=	$1.7d_0$	30.6
		35 mm

k_b= $e/3d_0$, $p/3d_0$–.25, f_{ub}/f_u

k_b= least of above three.

k_b= 35/3x22 , 40/3x18–0.25, 400/410

k_b= 0.648 0.49 0.976

Therefore, **k_b= 0.491**

V_{dpb}= 2.5x0.491x16x6 x410/1.25

V_{dpb}= 38631.11111 N/mm^2

38.63111111 kN/mm^2

Therefore the BS = 28.974 kN/mm^2

Number of bolts = Force / BS

= 42.09 / 28.97

4

Number of bolts = 1.453

Thererfore provide 3nos of 16mm dia bolts.

* **Design as tension member:** $A_g x f_y / \gamma_{mo}$

* **Tensile strength in tearing(T_{dg}):**

T_{dg}=

$4\bar{}278x250/1.1$

T_{dg}= 78863.63636 N/mm²

78.864 kN/mm² >27.96 kN/mm²

Therefore safe in tearing

* **Tensile strength Due to rupture of critical section(T_{dn}):**

T_{dn}= $(0.9 x A_{nc} x f_u / \gamma_{ml} + \beta x A_{go} x f_y / \gamma_{mo})$

β= $1.4-(0.075x\ w/t\ x\ f_y/f_u\ x\ b_s/L_c)x2$

w=	45	mm
t=	4	mm
b_s=	53.5	mm
L_c=	100	mm

β= 1.4−(0.075x 45/4 x 250/410 x 53.5/100)

β= 1.125

A_{nc}(=45−4/2 −18) x4

A_{nc}= 100 mm²

A_{go}= (45-4/2)x4

A_{go}= 172 mm²

T_{dn}= (0.9x100x410/1.25+ 1.207x172x250/1.1)

T_{dn}= 73487.58938 N/mm²
= **73.488 kN/mm² >27.96 kN/mm²**
Therefore, safe in critical rupture of section.

* **Tensile strength in block shear(T_{bd}):**

$$T_{bd1}=(A_{vg} x f_y/(\sqrt{3}\,\gamma_{mw}) + 0.9 x A_{tn} x f_u/\gamma_{ml})$$

$$T_{bd2}=(0.9 x A_{vn} x f_u/(\sqrt{3}\,\gamma_{ml}) + A_{tg} x f_y/\gamma_{mw})$$

A_{vg}= Gross shear area.
= (40x2+35)x4 **1260 mm²**
A_{vn}= Net shear area.
= ((40x2)-(2.5x18))x4 **52 mm²**
A_{tg}= Gross area in tension.
= (32.5x4)x4 **130 mm²**
A_{tn}= Net area in tension.
= (32.5-18x0.5)x4 **94 mm²**

now,

T_{bd1}= 193080.9996 KN
T_{bd1}= 193.081 KN
T_{bd2}= 36071.09285 KN
T_{bd2}= 36.071 KN

Therefore

T_{bd}= 36.071 KN >27.96KN
Therefore, safe in block shear.

Since the section is safe in both compression and tension, therefore the section provided ISA-45x45x4 for DIAGONAL MEMBER is safe.

2.8 SAG ROD:

These are round section rods and are fastened to the web of the purlins. The roof coverings in industrial buildings are not rigid and do not provide proper support. Therefore, sag rods are provided between adjacent purlins to extend lateral support for the purlins in their weaker directions. A Sag rod is designed as a tension member to resist the tangential component of the resultant of the roof load and purlin dead load. The tangential component of the roof load is considered to be acting at the top flange of the purlins, whereas the normal component and purlins dead load is assumed to act at its centroid. Therefore, the sag rod should be theoretically placed at the point where the resultant of these forces act. But this is not practicable and sag rods are placed at the minimum gauge distance below the top. The sag rod provided at the crown is termed as tie rod. This resists the tangential components from the two sides of the roof truss. The number of sag rods to support each purlin depends upon the length of the purlin and the load to be supported.

2.8.1 Design of SAG ROD:

* Desing of sag rod for both the sides

$$= (w_d x \sin\theta\ L/2)x2$$

* Total dead load (W_d) = 348 N/m²
* Roof angle (θ) = 21.5
* Spacing between the bay = 4.09 m

$$= (348 x \sin((21.54) x\ 4.09/2)x2$$

= **446.9117425 N**
= **0.447 kN**

Number of sag rods = 10

Area required for sag rod $= F/\sigma_a$

$=10x446.911/150$

$=29.794\ mm^2$

Diameter of the the bar $= \sqrt{(29.794/\pi)\ x2}$

$=$ 5.907 mm

$\approx$ 10 mm

But minimum diameter recommended= 16 mm

2.8.1 Design of Shoe Angle:

* From analysis
* Live Load = -12.82 kN
* Wind load = 55.18 kN
* Dead load = -13.44 kN
* Yeild stress of materials(f_y) = 250 N/mm^2
* Ultimate stress of materials(f_u) = 400 N/mm^2

*Tension load per bolt(V_{tb}) = ((55.18-13.44)/2)

V_{tb} = 20.87 kN

* Wind load per bolt (V_{sb}) = ((12.82+13.44)/2)x1.5

V_{sb} = 19.695 kN

Trying 2 numbers of 16mm diameter Black bolts 16

From IS 800-2007 , Clause 11.6.2

*Actual shear stress in bolt (f_{sb})= $(V_{sb})/A_{sb}$

f_{sb}=19.695/πx0.25x16^2

f_{sb}= 0.098 kN/mm^2

Nominal shear capacity of bolt(V_{nsb})= $f_{ub}x(n_a x A_{nb}+n_s x A_{sb})$

$= 400/\sqrt{3}(0.78x\pi x0.25x〚16〛^2+1x\pi x0.25x〚16〛^2)$

V_{nsb}= 82651.178 N

V_{nsb}= 82.6512 kN

* Permissible bearing stress of bolt (f_{apb}) $0.6xV_{nsb}/A_{sb}$

f_{apb}= f_{apb}= 0.6x82.6512/0.25xπx16^2

f_{apb}= 0.247 kN/mm^2

* Actual tensile stress of bolt (f_{tb}) = T_s/A_{sb}

= 20.87/πx0.25x16^2

Now, **f_{tb}= 0.1038 kN/mm^2**

T_{nb} = 0.9 x f_{ub} x A_{nb}< f_{yb} xA_{sb} x (ϒ /ϒ)

T_{nb} = 0.9 x 400 x π x 0.25 x 0.78 x 16^2 <
250 x π x 16^2 x 0.25 x (1.25/1.10)

Therefore,

T_{nb} = 56458.142 < 57119.8182

T_{nb} = 56.458 < 57.120

f_{atb}=0.6xT_{nb}/A_{sb}

$0.6x56.458/0.25x\pi x16^2$

$f_{atb}=$ **0.168 kN/mm²**

If the bolt is subjected to combined shear and tension, actual shear and axial stresses, do not exceed the respective permissible stresses , and expression below should satisfy

$$\left(\frac{f}{f}\right)^2 + \left(\frac{f_t}{f_t}\right)^2 \leq 1$$

$$\left(\frac{0.132}{0.2466}\right)^2 + \left(\frac{0.0862}{0.168}\right)^2 \leq 1$$

0.537 ≤1

Hence the stresses are safe.

Assuming angle section,

Number of bolts = 19.695/55.18

= 0.357 nos But minimum 2 bolts should be provided.

Adopting 200x200 bearing plate.

L= 200 mm

B= 200 mm

e= 40 mm

Pressure below bearing plate (P) =(Wind load)/(Area of bearing plate)

$= 19.695x10^3/200x200$

P= 0.492 N/mm²

Bending moment (M) = 0.492x40*40/2

M= 393.9 N-mm

* Permissible Bending Stress for plate:

$f_{bd}\ bt^2/6=$ BM

$(f_y/1.1)\ x\ bt^2/6$ =393.9 mm

$(250/1.1\ )\ x\ 1xt^2/6$ =393.9 mm

Therefore providing bearing plate of 200x200x4 mm

CONCLUSIONS

- ✓ On completion of the design the proposed industrial structure including all the structural elements detailed understanding of interpreted design of structure is achieved. All the structural components were designed manually and detailed using Saidpur software package v.8i. The analysis and design were done according to standard specifications to the possible extend and which proved to be premium software of great potential in analysis and design sections of construction industry.
- ✓ A detailed understanding of the loading and load combination provisions of IS-800:2007 for the design of Columns, Gantry girder, Truss members, Purlins and their Connections are understood.
- ✓ Use of MS-EXCEL for design is understood.
- ✓ Use of AutoCAD for drawings is understood.

REFERENCES

1) IS: 800-2007, "Indian Standard Code of general construction in steel" Indian Standards Institute, New Delhi

2) IS:875-1987, "Indian Standard Code of practice for Design loads for buildings and Structures (Second Revision)", Indian Standards Institute, New-Delhi

3) N. SUBRAMANIAN, "Design of Steel Structures", Oxford University Press, New Delhi

4) N. KRISHNA RAJU, "Structural Design and Drawings for Reinforced Concrete and Steel", Third Edition-2009

5) S. S. BHAVIKATTI, "Design of Steel Structures", 2007

6) S. RAMAMRUTHAM, "Steel Tables", 2010