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ANALYSIS AND DESIGN OF STEEL STRUCTURE

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Abstract: Steel buildings are buildings made from metal in order to go and provide support and strength to the structure on which building stands on. Steel structure proves to be strong, durable and stable. Steel is also environment friendly and also fire resistant. Steel structure requires low cost for maintenance and the speed of construction is fast when compared with other material. The paper describes the analysis of steel building considering dead load, live load, and wind loads. For the analysis graphical method has been used. The industrial structures shall be mainly designed and constructed in order to resist the effect of wind loads in accordance to the provisions given in IS:875(Part 3):1987. This paper work is focused on the analysis of steel truss and supporting columns. Relevant calculations are done and appropriate figures are drawn. Both during analysis and design manual calculations are done.

Keywords: Analysis, Design of Steel

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INTRODUCTION

Steel is one of the most sustainable construction materials. Its strength and durability coupled with its ability to be recycled, again and again without ever losing quality make it truly compatible with long term sustainable development. Steel structures are ease of fabrication and mass production. This construction is fast and has easy erection and installation. This construction also has economy in transportation and handling.

EXPERIMENTAL WORK

Manual Calculation

Design a fink type roof truss for an industrial building for the following data.

Overall length of the building = 40m

Overall width of the building = 16m

Spacing of the truss = 8m

Rise of truss = ¼ 0f span

Self - weight of purlin = 314N/m

Height of column = 15m

Roofing and Side covering = Asbestos cement sheets = 171N/m

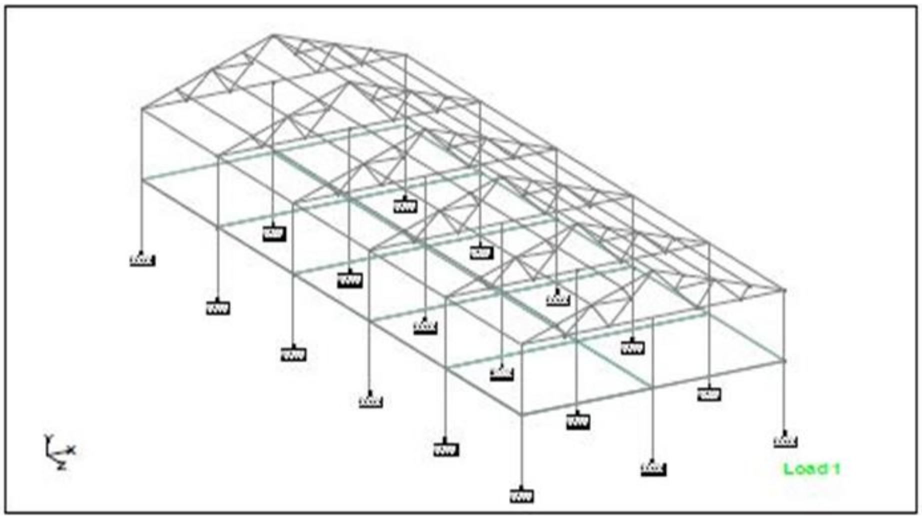
The building is located in Mumbai.

Use steel of grade Fe415 .Both the ends of the trusses are hinge.

Step 1:-Use above data as given.

Step:-2 Find design of roof truss

Step:-3 Design of steel building



Step:-4Calculation of θ

tanθ = 4/8 = 26.56˚

Step:-5Calculation of length of rafter

Length of principal rafter = √ [(16/2) ^2 +8^2]= 8.94m

Length of each panel = 8.94/4=2.235m

Step:-6Calculation of loads

1) Dead load

2) Assume weight of bracings = 12N/m

3) Dead load on AC sheets = 171N/m

4) Self-weight of roof truss = (span/3 +5) ×10= (16/3+5) ×10= 103.33 N/m2

5) Self -weight of Purlins = 314N/m

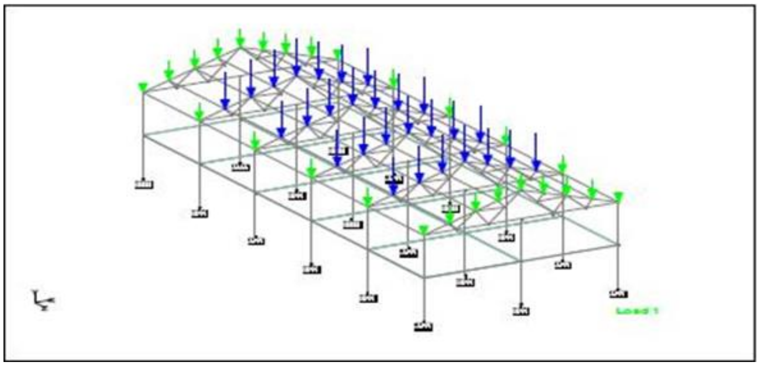
6) Self -weight of Purlins = for inner frame= 314× spacing= 314× 8= 2512N = for outer frame= 314×4= 1256N

7) Dead load on inner frame intermediate panel = (12+171+110)×(8×2.235)+2512=7.75KN

8) Dead load on inner frame end panel point = 7.75/2=3.87KN

9) Dead load on outer frame intermediate panel = (12+171+110)×(4×2.235)+1256=3.87KN

10) Dead load on outer frame end panel point = 3.87/2= 1.93KN



2) Live load

1) θ = 26.56˚

2) Live load

Let assume that no access is provided to the roof. The live load is reduced by 20N/m2 for each one degree above 10 ˚ slope.

Live load = (750-20 × (26.566-10))= 418.68N/m2

3) Live load on inner frame each intermediate panel = live load× spacing of truss on both side

= 418.68×8×2.235= 7485.99 N= 7.5KN

4) Live load on inner frame end panel point = 7.5/2= 3.75KN

5) Live load on outer frame each intermediate panel = live load × spacing of truss on both side

= 418.68×4×2.235= 3750N= 3.75KN

6) Live load on outer frame end panel point = 3.75/2= 1.875KN

3) Wind load

1) Basic wind velocity Vb = 44m/s (Appendix A clause5.2 page no 53 of IS code)

2) Let assume the life of the industrial building to be 50 years and the land to be plain and surrounded by small building

$K1$= (risk coefficient) = 1(From IS 875 (part3):1987 page no 11)

$K2$ = (terrain factor) = 1.09(From IS 875(part3):1987 page no12)

$K3$= (topography factor) = 1(For plain land)

1) Calculation of wind speed $VZ = K1 \times K2 \times K3 \times Vb = 1 \times 1.09 \times 1 \times 44 = 47.96$m/s

2) Calculation of design wind pressure $Pd = 0.6 \times (Vz)^2 = 0.6 \times (47.96)^2 = 1380.0$N/m2= 1.3KN/m2

3) Calculation of external and internal air pressure coefficients

1) External air pressure coefficient

Cpe for the condition$1/2<h/w<3/2$ and for $\theta = 26.56$ from IS code

$X1 = -0.37$ for windward side

$X2 = -0.8$ for leeward side

4) Let assume the building to have normal permeability. The internal pressure coefficient Cpi are ±0.2 for both the windward and leeward side.

1) Wind load= $(Cpe - Cpi) \times Pd \times A = (-0.8-(\pm0.2)) \times 1.3 \times 8 \times 2.235 = -23.24$KN (uplift)

2) Wind load per unit length of purlin = 23.24/8= 2.90KN/m

3) Wind load on panel points

Windward side

$F = (Cpe-Cpi) \times Pd \times A = (-0.37(\pm0.2)) \times 1.3 \times (8 \times 2.235) = -13.24$KN

Therefore wind load on intermediate panel point = -13.24KN

Wind load on each panel = (13.24/2)= -6.62KN

Leeward side

$F = (Cpe-Cpi)\ Pd \times A = (-0.8(\pm0.2)) \times 1.3 \times (8 \times 2.235) = -23.24$KN

Wind load on each intermediate panel point = -23.24KN

Wind load on each panel = -23.24/2= -11.62KN

4) Assumed

1) Dead load on the roof covering asbestos cement sheets = 171N/m2

2) Dead load on purlins = 300N/m

3) Dead lad on AC sheets/m = 171×2.235= 382.185N/m

4) Total dead load = 382.185+300= 682.185N/m

5) Design load normal to roof surface = net uplift force= 2900-682.185= 2217.81N/m

6) Maximum moment = wl/10= (2217.8×8) ×8×10^/10= 1419×10^4N/mm

7) Required section modulus= 1419× 10^4/1.33×165= 64661mm3

8) Approximate depth of angle section purlin = L/45= 8000/45= 200mm

9) Approximate width of the angle section purlin = L/60= 8000/60= 150mm

Let us provide I.S.A 200×150×12mm angle section as purlins

Self-weight of the section = 312N/m

Section modulus provided = 117400mm3>6466mm3 OK

Bending stress = 1419×10^4/117400=120.8N/mm2<219.45N/mm2

5) From the truss geometry the line of action 43.2KN resultant cuts the horizontal member and the line of action of 75.2 KN resultant cuts the vertical member

Components of resultant

1) Force43.2KN

Vertical component= 43.2Cos26.56˚ = 38.64KN

Horizontal component = 43.2Sin26.56˚ = 19.32KN

2) Force75.2KN

Vertical component = 75.2Cos26.56˚ = 67.26KN

Horizontal component = 75.2Sin26.56˚= 33.63KN

Net horizontal component = 33.63-19.32= 14.31KN

Horizontal force at each shoe = 14.31/2= 7.16KN

To find vertical component take moment about either shoe

Take moment about L5

Rlo×16= 43.2× (4.472×5.367) +752×4.472 ,Rl0 = 47.58KN

Take moment about L0

Rl5×16 = 43.2×4.472+75.2 × (4.472+5.36), Rl5 = 58.32KN

A] Considering dead load in member

Consider joint L0

Apply $\sum V = 0$

29.6-3.7-FL0U1Sin26.56=0

-FL0U1 (0.44)+25.6=0

L0U1=58.0(Compression)

Apply $\sum H = 0$

FL0L`-FL0U1Cos25.56=0

FL0L1=52.0(Tension)

Consider joint U1, $\sum H = 0$

FL0U1Cos226.56-FU1U2Cos26.56=0

58.0Cos26.56-FU1U2Cos26.56=0

U1U2=55.6(Compression)

Apply $\sum V = 0$

FU1L1-7.4+FLOU1Sin26.56-FU1U2Sin26.56=0

FU1L1+18.89-0.44×55.6=0

FU1L1=6.2(Compression)

B] Live Load in the member

While considering live load in the member consider symmetry in the loading and truss configuration.

FL0L1 = $\frac{52.0}{3.7} \times 3.35 = 47.0$(Tension)

FL0U1 = $\frac{58.0}{3.7} \times 3.35 = 52.5(C$ompression)

FU1L1 = $\frac{6.2}{7.4} \times 6.7 = 5.6$(Compression)

Same analysis we have done for wind load

Stress Member	Dead Load stress(KN)	Live Load stress(KN)	Wind Load stress(KN)		Maximum stress			Design force	Member
			Wind left	Wind right	DL+LL	DL+LL+WL	DL+WL		Rafter
b-1	-58.0	-52.5	+95.6	+111.6	-110.5	-14.9	+53.6		L0U1
c-2	-55.6	-50.3	+95.6	+111.6	-105.9	-10.3	+56.0		U1U2
d-5	-51.4	-46.5	+95.6	+111.6	-97.9	+13.7	+60.2	110.5KN	U2U3
e-6	-48.0	-43.5	+95.6	+111.6	-91.5	+20.1	+63.6	Compression	U3U4
f-8	-48.0	-43.5	+111.16	+95.6	-91.5	+20.1	+63.6	63.6KN	U4U5
g-9	-51.4	-46.5	+111.16	+95.6	-97.9	+13.7	+60.2	Tension	U5U6
h-12	-55.6	-50.3	+111.16	+95.6	-105.9	-10.3	+56.0		U6U7
i-13	-58.0	-52.5	+111.16	+95.6	-110.5	-14.9	+53.6		U7U5
									Main Tie

1-k	+52.0	+47.0	-76.0	-102.4	+99.0	+23.0	-50.4		L0L1
3-k	+45.0	+40.7	-64.0	-51.8	+85.7	+21.7	-36.8	99.0KN	L1L2
7-k	+31.1	+28.2	-39.8	-39.8	+59.3	+19.5	-8.7	Tension	L2L3
11-k	+45.0	+40.7	-81.8	-64.0	+85.7	+21.7	-36.8	50.4KN	L3L4
13-k	+52.0	+47.0	-102.4	-76.0	+99.0	+23.0	-50.4	Compression	L4L5
									Struts
1-2	-6.2	-5.6	+108.8	+18.8	-11.8	+7.0	+12.6		U1L1
3-4	-12.4	-11.2	+21.6	+37.6	-23.6	+14.0	+25.2		U2L2
5-6	-6.6	-6.0	+10.8	+18.8	-12.6	+6.2	+12.2	23.6KN	U3M1
8-9	-6.6	-6.0	+18.8	+10.8	-12.6	+6.2	+12.2	Compression	U5M2
10-11	-12.4	-11.2	+37.6	+21.6	-23.6	+14.0	+25.2	25.2KN	U6L3
12-13	-6.2	-5.6	+18.8	+10.8	-11.8	+7.0	+12.6	Tension	U7L4
									Minor Sling
2-3	+7.0	+6.3	-12.1	-21.0	+13.3	-7.7	-14.0	13.3KN	U2L1
4-5	+6.0	+5.4	-12.1	-21.0	+11.4	-9.6	-15.0	Tension	U2M1
9-10	+6.0	+5.4	-21.0	-12.1	+11.4	-9.6	-15.0	15.0KN	U6M2
11-12	+7.0	+6.3	-21.0	-12.1	+13.3	-7.7	-14.0	Compression	U6L4
									Main Sling
6-7	+20.3	+18.4	-36.3	-63.0	+38.7	-24.3	-42.7	38.7KN	U4M1
4-7	+13.8	+12.5	-24.2	-42.0	+26.3	-15.7	-28.2	Tension	L2M1
7-8	+20.3	+18.4	-63.0	-36.3	+38.7	-24.3	-42.7	42.7KN	U4M2
7-10	+13.8	+12.5	-42.0	-24.2	+26.3	-15.7	-28.2	Compression	L3M2

Therefore, primarily it is design as a tension member and the section is checked for maximum compression in it. While checking it as a compression member it is treated as a strut. Design tensile force = 99.0KN

Design compressive force = 50.4 KN

Net Area required = $\frac{99.0\times10^3}{150}$ = 660mm^2

Increase it by 40%

Gross area required = 1.4$\times 660$ =924mm^2

From I.S Handbook no 1 let us provide two unequal angles 65$\times 45 \times 8mm$ back to back on the opposite side of the gusset plate.Area provided = 1634mm^2

Provide 14mm rivets for connection

Net area provided = 1634-2$\times 8 \times 15.5$ = 1386mm^2

Safe tensile load = $\sigma_{at} \times net\ area\ provide$1.33×150×1386= 276.507N = 276.5KN >99.0KN

Due to reversal of stresses the main tie is checked as a compression member.

Length of tie between nodal points, L= 6.0m

Effective Length, $L_{xx} = 0.85 \times 6.0 = 5.10m = 5100mm$

Two angle sections are provided for bracing, one at L_2 and the other at L_3 node in the truss. Assuming longitudinal tie at the main nodal points,

Effective length, $L_{yy} = 1.0 \times 6.0 = 6.0m = 6000mm$

The relevant properties of the built up section are

A=1634mm^2, $r_{xx} = 20.2mm, r_{yy} = 21.1mm$

$$\frac{l_{xx}}{r_{xx}} = \frac{5100}{20.2} = 252..47$$

$$\frac{l_{yy}}{r_y} = \frac{6000}{21.1} = 284.36$$

From IS: 800-1984, for $f_y = 250N/mm2\frac{l}{r}$ =284.36

$$\sigma_{ac} = \frac{14.26n}{mm2}$$

Sae compressive load carrying capacity= $\sigma_{ac} \times area\ provied$

= 1.33×14.26×1634×10^-3= 30.9<50.4KN

Hence the section is not safe.

Revise the section and provide two I.S.A.100×65×8mm

The relevant properties are A=2514mm^2, $r_{xx} = 31.6mm, r_{yy} = 27.5mm$

As we have chosen the higher section than 65×45×8mm there is no need to check for tension member. (as the section smaller than this was found to be safe)

Let us check for compression under reversal due to wind.

$$\frac{l_{xx}}{r_{xx}} = \frac{5100}{31.6} = 161.39$$

$$\frac{l_{yy}}{r_{yy}} = \frac{6000}{27.5} = 218.18$$

Therefore the critical slenderness ratio is 250.

From .I.S. 800-1984 for $\frac{l}{r} = 218.18, f_y = 250Mpa$

$\sigma_{ac} =$ 23.36KN/mm2

Safe compressive load=1.33×18×2514×10^-3=78.10KN >50.4KN

which is sufficient

RATER

Panel length, L=2.235m

Effective length about X-X axis =0.85L=0.85×2.235=1.8997m=1899.7mm

The purlins are placed at the panel points. Hence the length of the rafter between purlins is equal to the panel length.

Effective length about Y-Y axis=1.0×L=2.235m=2235mm

Design compressive load=110.5KN

Let us try some section and check its safety. From .I.S handbook no 1 select two unequal angle section I.S.A 65×45×8mm. The relevant properties of section are

A=1634mm^2, $r_{xx} = 20.2mm, r_{yy} = 21.1mm$

$\frac{l_{xx}}{r_{xx}} = \frac{1899.7}{20.2} = 94.04$

$\frac{l_{yy}}{r_{yy}} = \frac{223.5}{21.1} = 105.924$

The slenderness ratio $\frac{l_{yy}}{r_{yy}}$ *is* critical

From .I.S. 800-1984, slenderness ratio=105.92and $f_y = 250Mpa$

$\sigma_{ac} =$ 75.26N/mm2

Safe load carrying capacity= $\sigma_{ac} \times$ area provided

= 1.33 $\times 75.26 \times 1634 \times 10^{-3}$ =163.55KN>110.5KN

which is sufficient

Design compressive force=23.6KN

Design tensile force=25.2KN

Length of member=2.235m

Let us assume allowable compressive stress=60Mpa

Area required= $\frac{23.6\times10^3}{60}$ =393.33mm2

Let us try an I.S.A 65×65×5mm section from handbook no 1.The relevant properties of section are - A=625mm^2, $r_{xx} = r_{yy} = 16.6mm$, r_{vv}=12.6

$\frac{l}{r} = \frac{223.5}{12.6}$ =177.38<180

From I.S.800-1984, for $\frac{l}{r}$=177.38and $f_y = 250Mpa$

$\sigma_{ac} =$ 34.05N/mm2

Safe compressive load= σ_{ac} × area provided

=1.3×34.05×625×10^-3=228.20kN >23.6KN

Check as a tension member. Let us provide 14mm $\emptyset$ power driven rivets for connection

A_1= area of connected leg= (65-15.5-$\frac{5}{2}$) $\times 5$ = 260mm2

$A_2 =$ Area of outstanding leg= (65-$\frac{5}{2}$) $\times 5 = 312.5mm2$

K=$\frac{3A_1}{3A_1+A_2}$= $\frac{3\times260}{3\times260+312.5}$=0.741

Net area required=$A_1 + kA_2$=260+0.741×312.5=483.125

Safe tensile load=$\sigma_{ac} \times$ net area required

= 1.33 $\times 150 \times 483.125 \times 10^{-3}$ = 96.38KN >25.2KN

This is sufficient

MINOR SLING

Design tensile force=13.3KN

Design compressive force=15.0KN

Net area required= $\frac{13.3\times10\text{^}3}{150}$ =88.67mm2

Gross area required=1.4 $\times$ 88.67 = 124mm2,Length=2.48m

Let us try I.S.A 50$\times 50 \times 5mm$ section from handbook no 1. The relevant properties of the section are

A=479mm^2, $r_{xx} = r_{yy} = 15.2mm$, $r_{vv} = 9.7mm$

Let us provide 14mm diameter rivets for connection

$A_1 =$ Area of connected leg= (50-15.5-$\frac{5}{2}$$) \times 5$=160mm^2

$A_2 =$ Area of outstanding leg= (50-$\frac{5}{2}$$) \times 5 = 237.5mm2$

K=$\frac{3A_1}{3A_1+A_2}$ =$\frac{3\times160}{3\times160+237.5}$ = 0.669

Net area required= $A_1 + kA_2$=160+0.669$\times 237.5 = 318.88mm^2$

Safe tensile load=$\sigma_{ac} \times$ net area required

= 1.33$\times 150 \times 318.88 \times 10^{-3} = 63.62KN$ >13.3KN

This is sufficient

Check for compression

Length of member=2.48m=2480mm

Effective length= 0.85$\times 2480 = 2108mm$

$\frac{l}{r} = \frac{2108}{9.7} = 217.3$

From .I.S 800-1984, for $\frac{l}{r} = 217.3 and f_y = 250Mpa$

$\sigma_{ac} = 23.54$N/mm2

Safe compressive load=$\sigma_{ac} \times area\ provided$

=1.33$\times 23.54 \times 479 \times 10^{-3} = 15KN \geq 15.0KN$

This is sufficient

MAIN SLING

Design tensile force=38.7KN

Design compressive force= 42.7KN

The length of the member about xx and yy axis is

$$U_4M_1 = 2.48m, U_4L_2 = 4.96m, l_{xx} = 0.85 \times 2.48 = 2108mm l_{yy} = 4960m$$

There is no need to design the member because observation shows that the same section will have to be provided for the main sling as that of the main tie. Hence provide two .I.S.A 90$\times 60 \times 8mm$ sections for the main sling,

JOINTS

Let us provide 14mm diameter power driven rivets

Gross diameter of rivet=15.5mm

Strength of rivet in single shear=$\frac{\pi}{4} \times d^2 \times \tau_{vf}$ =$\frac{\pi}{4} \times 15.5^2 \times 1.25 \times 100 \times 10^{-3} = 23.58KN$

Strength of rivet in double shear=2×23.58=47.16KN

Strength of rivet in bearing over 6mm= dtσ_{pf} =15.5$\times 6 \times 1.25 \times 300 \times 10^{-3} = 34.8KN$

Strength of rivet in bearing over 8mm= 15.5$\times 8 \times 1.25 \times 300 \times 10^{-3} = 46.5KN$

RAFTER

Joint U_4 Maximum force= 91.5KN,Rivet value=46.5KN

Number of rivets $= \frac{91.5}{46.5} 1.96 = 3$

JointL_0L_5 Maximum force= 110.5KNRivet value=46.5KN

Number of rivets=$\frac{110.5}{46.5} = 2.37 = 3$

Joint U_1, U_7 Force in the gusset plate parallel to rafter= 110.5-105.9=4.6KN

Number of rivets=$\frac{4.6}{23.58} = 2$

Provide two rivets to connect the angle with the rafter

Maximum force in the gusset plate normal to the rafter=

=Maximum force in member U_1L_1=18.8KN

Therefore, number of rivets required=$\frac{18.8}{23.58} = 0.79 = 3$

Provide three rivets to connect the rafter with the gusset plate

Joint U_2, U_6

Force in the gusset plate parallel to rafter=105.9-97.9=8KN

Number of rivets=$\frac{8}{23.58} = 0.34 = 2$

Provide two rivets to connect the angle with the rafter

Joint U_3, U_5

Force in the gusset plate parallel to the rafter= difference in the member forces

= 97.9-91.5=6.4KN, Number of rivets=$\frac{6.4}{23.58}$=0.27=2

Provide two rivets to connect angle with the rafter

Maximum force in the gusset plate normal to the rafter=18.8KN

Number of rivets=$\frac{18.8}{23.58}$=0.79=2

Provide minimum two rivets

Main Tie

Joint L_0, L_5 Maximum force=99.0KNRivet value=37.2KN

Number of rivets=$\frac{99}{37.2} = 2.13 = 3$

Joint $L_{1,}L_4$ Force in the gusset plate parallel to main tie= difference in member forces

=99-85.7=13.3KN, Number of rivets=$\frac{13.3}{46.5} = 0.286$

Provide minimum two rivets

Joint L_2L_3 Force I gusser plate parallel to main tie=difference in main forces

=85.7-59.3=26.4KN, Number of rivets=$\frac{26.4}{46.5}$=0.56

Provide minimum two rivets

Minor Sling

Maximum force=15.0KN, Rivet value=18.869KN

Number of rivets= $\frac{15.0}{23.58}$ =0.63=2

Provide minimum two rivets

3.CONCLUSION

1] In this type of the building ,as the building is symmetrical therefore the live load and dead load reactions are same on both the supports but the reaction due to wind load will be different on two supports.

2] In this type of the steel building strut is designed as a tension member and then the section is checked for the maximum compression in it.

3] This analysis is based on the different combinations of load as per IS: 800: 2007, IS: 1893:2007.

4] The members on rafters are subjected to compressive forces.

5] It is seen that the design is done for maximum values of dead load, live load and wind load.

6] As the magnitude of forces on inner frame is twice that of outer frame we can conclude that inner frame needs to be stronger as that of outer frame.

7] Purlins are designed for normal component of forces. Purlins are also design to support the roof covering.

8] Strut used in this building is a compression member which is continuous over a number of joints.

9] It is also seen that the structure is design by considering dead load, live load, and wind load will be safe against earthquake forces also.

10] We can also conclude from analysis that dead load stress is more as compare to live load stress. And wind load is greater than live load stress and dead load stress.

11] This type of truss is most commonly used in residential building and also used in design of bridges for railroads and the strength of this type of truss allows home owners to store up to 20 pounds per sq. foot on structure.

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4. IS: 1893(part1):2002-Crieteria for Earthquake resistant design of structures.

5. IS: 875(part1):1987-Code of practice for design loads for building structure-Dead load.

6. IS: 875(part2):1987-Code of practice for design loads for building structure-Imposed load.

7. IS: 875(part3):1987-Code of practice for design for building structure-Wind load.

8. Design of Steel Structure by Ramachandra Vol.1 and 2 standard warehouses.

9. Pre-engineered building design of standard warehouse.

10. Aznar IBagadia (2012) Comparison of response of industrial structures as per IS: 1893:1984 with IS: 1893(part4):2005.