STEEL ROOF TRUSSES - ROOFING ELEMENTS

4.2 Design of steel roof truss

Example 2

Design a steel roof truss to suit the following data,

Span of the truss	= 10m
Type of truss	= pan type
Roof cover	= Galvanization corrugated (GC) sheeting
Materials	= Rolled steel angles
Spacing of roof truss	= 4.5m
Wind pressure	=1KN/m^2

Draw the elevation of the roof truss and the details of joints.

Solution:

Step:1 Dimension of truss

Central rise = span/4

=10/4

= 2.5m

Purlins are provided at intervals of 1.863m on the principal rafter

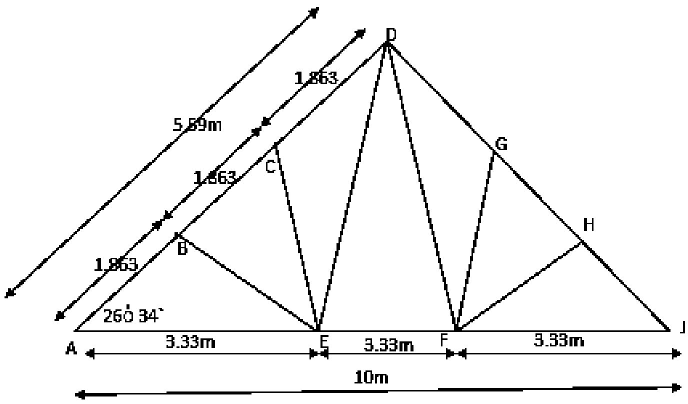

Fig.4.1 Roof truss

Step:2 Dead loads

Self weight of GC sheeting per purlin at 0.18KN/m^2

= 0.18 x 1.863

= 0.335 KNm

Self weight of purlin at 0.1KN/m

= 0.10 KN/m

Total dead load = 0.435KN/m

Step:3 Live loads

Slope of the truss = 26̊ 34`

Live load of the truss

= 0.75 – (10x0.01 + 6.5x0.02)

= 0.52 KN/m^2

Live load per purlin per metre

= 0.52x1.836xcos 26̊ 34`

= 0.87KN

Step:4 Wind loads

F = (Cpe – Cpi)A pd

Cpe = external pressure coefficient

Cpi = internal pressure coefficient

A = Surface area of structural element or cladding unit

pd = design wind pressure

Slopping angle ,

Ɵ = 26̊ 34`

Cpe = -0.7

Cpi = 0.2

F = (-0.7 – 0.2)pd

= -0.9 pd

=-0.9x1

= -0.9 Kn/m^2

Maximum wind load per purlin per metre

= (-0.9x1.863xcos26̊ 34`)

= 1.5 KN

Step:5 Design of purlin

For continuous purlin, the max factored bending moment and shear force are computed as follow,

M = (1.5x1.305x4.5^2)/10

= 3.96 KNm

V = (1.5x1.305x4.5)/2

= 4.4 KN

Adopt ISA 100x75x8mm having section properties given below,

Zx = (4.38x10^4)mm^3

D = 100mm

b = 75mm

t = 8mm

IS 800:2007 clause 3.7,

(a) Check for section classification is done by computed the rations,

(b/t) =75/8

= 9.37<9.4

Hence the section considered as plastic .

(b) Check for shear capacity

Av = 100X8

= 800mm^2

clause 8.4.1,

(Av fy w/√3 γmo) = (800x250)/(√3x1.10x10^3)

= 105 KN >4.40KN

The shear capacity of the section is very large compared to the applied shear force.

(c) Check for moment capacity

Md = (ßb Zx fy)/γmo

= (1x4.38x10^4x250)/(1.1x10^6)

Md = 9.95 KNm > 3.96 KNm

Step:7 Load on truss

(a) Dead load

Slopping length of rafter,

AD = √(5^2+2.5^2)

= 5.59m

Spacing of trusses = 4.5m c/c

Weight of GC sheeting on half truss (plan area) at 0.18 KN/m^2

= 4.5x5x0.18

= 4.05 KN

Weight of purlins (4nos) at 0.10 KN/m

= 4x0.1x4.5

= 1.8 KN

Self weight of roof truss = (span/300)+0.05

= (10/300)+0.05

= 0.083 KN/m^2

Weight of half roof truss = 0.083 x5 x4.5

= 1.86 KN

Total load on half truss = 4.05+1.8+1.86

= 7.71 KN

Dead load on intermediate- panel point

= 7.71/3

= 2.57 KN

Dead load on end panel point = 2.57/2

= 1.285 KN

(b) Live loads

Live load on half truss = 0.52 x5 x 4.5

= 11.7 KN

Live load on intermediate panel point

= 11.7/3

= 3.9 KN

Live load on end panel point = 3.9/2

=1.95 KN

(c) Wind loads

Maximum wind load acting perpendicular to the sloping surface

= 0.9x4.5x5.59

= -22.63KN

Wind load on intermediate- panel point

= -(22.68/3)

= -7.5 KN

Wind load on end panel point = -(7.5/2)

= 3.75 KN

Step:8 Design of truss members

(a) Members AB, BC, CD

Maximum service load compressive force

= 36.17 KN

Maximum factored compressive force

= 1.5x36.17

= 54.25 KN

Maximum service load tensile force

= 22.95 KN

Maximum factored tensile force

= 1.5x22.95

= 34.42 KN

Length (L) = 1.863m

Effective length (KL) = 1.304m

Try two angle ISA 50x50x6mm placed back to back

Area (A) = 1136mm^2

Minimum radius of gyration (γmin) = 15.1mm

Slenderness ratio = (KL/γmin)

= 1304/15.1

= 86.3 < 180

Stress reduction factor x for column buckling class (c) corresponding to the slenderness

ratio 86.3 and fy = 250 N/mm^2

x = 0.56

∴Design compressive stress is computed as,

Fcd = x fy/γmo

=(0.56x250)/1.25

= 112 N/mm^2

Design compressive force is given by,

Pd =[A fcd]

= (1136x112)/1000

= 127 KN > 54.25 KN

(b) Member DE

Maximum service load tension

= 12.83 KN

Maximum factored load tension

= 1.5x12.83

= 19.24 KN

Maximum service load compression

= 9.57 KN

Maximum factored load compression

= 1.5x9.57

= 14.35 KN

Effective length = 3m

Try a single angle ISA 50x50x5mm connected by 6mm thick gusset plate the junction with

two bolts of 16mm at 50mm.

Gross area (A) = 479mm^2

γmin = 15.2mm

Using 16mm dia bolts,

Anc = [50-18]5

=160mm^2

Ago = [50-5]5

= 225mm^2

Ag = 479mm^2

(a) Strength governed by rupture of critical section

Tdn = [0.9 Anc fy/γmi] + [ß Ago fy/γmo]

where, ß = 1.4 – 0.076(w/t)(fy/fu)(bs/Lc)

$= 1.4 – 0.076(50/5)(250/410)(50+25/50)$

ß $=0.70$

Tdn $= [0.9x160x410/1.25]+[0.7x225x250/1.10]x10^3$

$= 83.02$ KN $= T_0$

(b) Strength governed by yielding of gross section

Tdg = Ag fy / γmo

$= (470x250x10^3)/1.10$

= 108.8KN

(c) Strength governed by block shear

Avg $= 5[50+50]$

$= 500mm^2$

Avn $= 5[50+50]- [1.5x18]$

$= 473mm^2$

Atg $= [5x25]$

$= 125mm^2$

Atn $= [(5x25)-(0.5x18)]$

$= 116mm^2$

The block shear strength is the smaller of the value of Tdb1 and Tdb2 as computed using

the equation given below,

Tdb1 = [Avg fy / V3 γmo]+[0.9 Atn fu/γmi]

= [(500x250)/(V3x1.1)+(0.9x116x410)/1.25]

= 99.92 KN

Tdb2 $= [0.9\ Avn\ fu/\sqrt{3}\ \gamma mi]+[Atg\ fy/\gamma mo]$

$= [(0.9x473x410)/(\sqrt{3}x1.25) + (125x250)/1.10]$

= 109.12 KN

Hence, Tdb = 109.12 KN

The design shear strength is the least of the three value computed under (a)(b)(c) , which are 108.8 KN, 83.02KN, 109.12KN.

The design tensile strength of angle = 83.02KN > 19.24 KN

(C) Member BC ,EB

Service load compressive force = 6.95KN

Factored compressive force = 1.5 X 6.95

= 10.42 KN

Service load tensile force = 6.38 KN

Factored tensile force = 1.5x6.38

= 9.57KN

Effective length (kL) = 0.7x1.6

= 1.12m

Use minimum size angle ISA 50x50x5mm,

Area (A) = 479mm^2

γmin = 9.7mm

Slenderness ratio (λ) = 1120/9.7

= 115

The stress reduction factor x corresponding to fy =250N/mm^2 and λ = 115

x = 0.39

Design compressive stress is computed as,

fcd = x fy/γmo

= (0.39x250)/1.25

= 78 N/mm^2

Design compressive force is given by ,

Pd = A fcd

= [479x78]/1000

= 37.36KN > 10.42KN

(d) Member EA and EF

Max service load tension = 32.21 KN

Factored tension = 1.5 x32.21

= 748.31 KN

Max service load compression

= 18.84 KN

Factored compression = 1.5x18.84

= 28.26 KN

Length of member = 3.33m

Effective length (kL) = 0.7x3.33

=2.331m

Try minimum two angle ISA 50x50x6mm connect by guesst plate 6mm thick with two 16mm dia bolts spaced at 50mm

Area (A) = 2x598

= 113.6mm^2

$\gamma min = 15.1mm$

i) Design strength due to yielding of cross section ,

Tdj = Ag fy /γmo

=[(1136+250)/1.10]x10^-3

= 258 KN

ii) Design strength governed by tearing at net section,

Tdn = α An fu/γmi

Assume a single line of 16mm dia bolts of two number spaced 50mm apart x=0.6

An = [(50-18)(6x2)]

= 384mm^2

Tdn = [(0.6x384x410)/1.25] x 10^-3

= 75.5KN > 48.31KN

Hence, the angle section designed for the truss can safely resist the factored loads.

Example 2

A beam column is to be designed to support a factored axial load of 500KN(tension). Factored moment Mx Acting at top and bottom of the column are 30 KNm and 50KNm respectively. Effective length of column may be Taken as 3.2m. Assuming fy=250N/mm^2, design the beam column section and check the same to conform the Specification of the Indian standard code IS 800:2007.

Solution:

Given data:

Factored axial load = 600KN (tension)

Bending moment at top = 30KNm

Bending moment at bottom = 50KNm

Yield stress of steel = 250N/mm^2

Step 1 Selection of beam column section

Mdx $= Z_0$ fy/ γmo

= (62x10^4x250)/(1.1x10^6)

= 140.7KNm

Tdg = fy Ag/γmo

= 250x6500/1.10x1000

= 1477.3KN

Desgin strength due to rupture of critical section,

Tdn =0.9 fy An/γmi

= (0.9x415x6500)/ (1.25x1000)

= 1942.2 KN

The design strength Td = 1477.3 KN

Step 2 Check for resistance of cross section to combined effects

Using the interaction equation ,

[N/Nd + Mx/Mdx + My/Mdy] ≤ 1.0

Nd = Ag fy /γmo

= 6500x250 / 1.1x1000

= 1477.3 KN

Mx = 50KNm and

Mdx = 140.7 KNm

∴ [600/1477.3 + 50/140.7] = 0.756 <1

Hence safe

Step 3 : Check for lateral torsional buckling resistance

Reduced effective moment is computed as,

$$M_{eff} = [\, M - \Psi t\, Z_{ec} / A \,] \leq M_d$$

$$= [(50\times10^6)-(0.8\times600\times10^3\times619\times10^3)/6500]$$

$$= 4.3\times10^6 \text{ Nmm}$$

$$= 4.3 \text{ KNm} < 127.3 \text{KNm}$$

Step 4 Check for overall buckling strength

$$[\, P/P_{dx} + M_{eff}/M_{dx} \,] \leq 1.0$$

$$[\, 600/1477.3 + 4.3/127.3 \,] = 0.439 < 1.0$$

Hence safe