4.4 Code Provisions Design Of Steel Roof Truss

Example:5

Design a steel roof truss to suit the following data,

Span of the truss = 12m

Type of truss = pan type

Roof cover = Galvanization corrugated (GC) sheeting

Materials = Rolled steel angles

Spacing of roof truss = 5 m

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

=12/4

= 3m

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

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

$= (-0.9 \times 1.863 \times \cos 26^\circ 34`)$

$= 1.5$ KN

Step:5 Design of purlin

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

M $= (1.5 \times 1.305 \times 5^2)/10$

$= 4.89$ KNm

V $= (1.5 \times 1.305 \times 5)/2$

$= 4.9$ KN

Adopt ISA 100x75x8mm having section properties given below,

Zx $= (4.38 \times 10^4) mm^3$

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 $= 100 \times 8$

$= 800 mm^2$

clause 8.4.1,

$(A_v f_{yw}/\sqrt{3}\,\gamma_{mo}) = (800\times250)/(\sqrt{3}\times1.10\times10^3)$

$= 105$ KN > 4.9 KN

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

(c) Check for moment capacity

$M_d = (\beta_b Z_x f_y)/\gamma_{mo}$

$= (1\times4.38\times10^4\times250)/(1.1\times10^6)$

$M_d = 9.95$ KNm > 4.89 KNm

Step:7 Load on truss

(a) Dead load

Slopping length of rafter,

$AD = \sqrt{(5^2+2.5^2)}$

$= 5.59$m

Spacing of trusses = 5m c/c

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

$= 5\times5\times0.18$

$= 4.5$ KN

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

$= 4\times0.1\times5$

$= 2$ 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 x5

= 2.075 KN

∴ Total load on half truss = 4.5+2+2.075

= 8.57 KN

Dead load on intermediate- panel point

= 8.57/3

= 2.85 KN

Dead load on end panel point

= 2.85/2

= 1.425 KN

(b) Live loads

Live load on half truss = 0.52 x5 x 5

= 13 KN

Live load on intermediate panel point

= 13/3

= 4.3 KN

Live load on end panel point

= 4.3/2

= 2.15KN

© Wind loads

Maximum wind load acting perpendicular to the sloping surface

=- 0.9x5x5.59

= -25.155KN

Wind load on intermediate- panel point

= -(25.155/3)

= -8.38 KN

Wind load on end panel point

= -(8.38/2)

= 4.19 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

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 / √3γmo]+[0.9 Atn fu/γmi]

= [(500x250)/(√3x1.1) + (0.9x116x410)/1.25] x 10^-3

= 99.92 KN

Tdb2 = [0.9 Avn fu/√3 γmi]+[Atg fy/γmo]

= [(0.9x473x410)/(√3x1.25) + (125x250)/1.10] x 10^3

= 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

γmin = 15.1mm

i) Design strength due to yielding of cross section ,

Tdj = Ag fy /γmo

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

= 258 KN

(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

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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

γmin = 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.