USN

Internal Assessment Test 2 –Dec. 2022

Sub:	Design of RC and Steel Structure				Sub Code:	18CV72	Branch	Civil
Date:	02/12/2022	Duration:	90 min's	Max Marks:	50	Sem / Sec	7A	OBE

	Answer ANY ONE FULL Questions **Note: Use of IS 456:2000 is permitted** **Assume missing data suitably.**	**MARKS**	**CO**	**RBT**
1	Design a roof truss with tabulation of member forces as shown in Figure. Design various member of roof truss along with their end connections with a gusset plate of 10 mm thick. The truss rest on 300 x 500 mm column made of M 20 grade concrete. Design the support bearing base plate for a reaction of 120 kN and anchor bolt for an uplift force of 18 kN. Draw to suitable scale i) Elevation of truss greater than half span ii) Enlarged view of support joint iii) Elevation of upper joint of truss.	[50]	CO3	L2

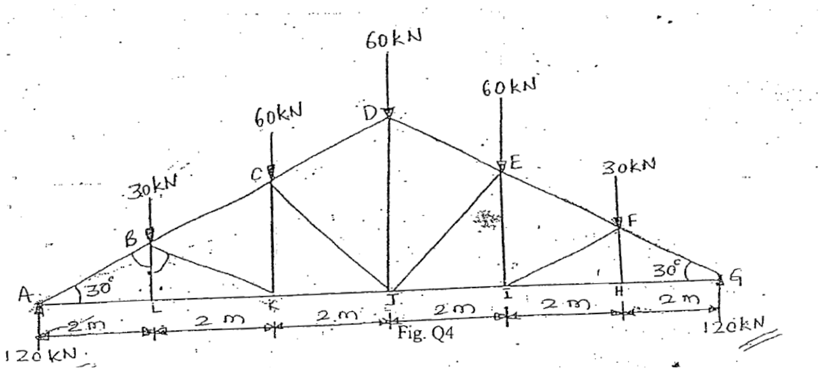


Fig. Q4

Member	Force	Nature of force (N)
AB, GF	240 kN	Compression, C
BC, FE	210 kN	C
CD, ED	160 kN	C
AL, GH	208 kN	Tension, T
LK, HI	208 kN	T
KJ, IJ	182 kN	T
BL, FH	0	-
BK, FI	30 kN	C
CK, EI	18 kN	T
CJ, EJ	66 kN	C
DJ	60 kN	C

- **Design of Top chord members (AB, BC, CD) (Compression members)**

AB, BC, CD are top chord member, maximum load is 240 kN

Factored load or $Force$ = 240 x 1.5 = 360 kN

Length of members AB/ BC/ CD

cos 30 = 2/ AB , AB = 2/ cos 30 = 2.3 m = 2300 mm

Assume design compressive stress, f_{cd} = 110 N/mm^2

(Assume f_{cd} = 40 -120 N/mm^2 based on load and experience)

Gross area, $A_g = \frac{Force}{f_{cd}} = \frac{360 \times 10^3}{110} = 3272.7$ mm^2 = 32.73 cm^2

- **Select double angle section from steel table**

From Steel Table 6, Page 18, Try 2 ISA 80 x 80 x 10 mm

Area = 30.10 cm^2 , r_{xx} = 2.41 cm Taking gusset plate of 10 mm thickness,
r_{yy} = 3.73 cm
r_{min} = 2.41 cm or 24.1 mm

- **Effective length of section - Page 48, CL7.5.2.1, IS 800-2007**

7.5.2 Double Angle Struts

7.5.2.1 For double angle discontinuous struts, connected back to back, on opposite sides of the gusset or a section, by not less than two bolts or rivets in line along the angles at each end, or by the equivalent in welding, the load may be regarded as applied axially. The effective length, *KL*, in the plane of end gusset shall be taken as between 0.7 and 0.85 times the distance between intersections, depending on the degree of the restraint provided. The effective length, *KL*, in the plane perpendicular to that of the end gusset, shall be taken as equal to the distance between centres of intersections. The calculated average compressive

48

Effective length for Top chord members (AB) , L_{eff} = 0.85 x L = 0.85 x 2.3 = 1.955 m

Slenderness ratio, $\lambda = \frac{L_{eff}}{r_{min}} = \frac{1955}{24.1} = 81.12$

Since it is a built-up member it will come under buckling class "c" (IS 800 – 2007, Page 44, Table 10). If it is class "c", Use Table 9(c)

From Table 9 (c) Page 42 – IS 800 2007, through interpolation, for λ = 81.12 we get design Compressive Stress, $f_{cd} = 134.32\ N/mm^2$

(Page 34) Design compressive strength, $P_c = f_{cd} \times A_g = 134.32 \times 3010$

$= 404.3$ kN > 360 kN

So selected section is safe.

- ## Connections

Using M 22 Property Class 5.6 bolts

- ## Shear strength of bolts - Page 75 CL10.3.3 , IS 800 2007

Assume fully threaded bolts, number of shear planes $n_n = 2$, $n_s = 0$ (no shank portion)

$A_{nb} = 0.78 \times \frac{\pi}{4} \times 22^2 = 296.5\ mm^2$, $A_{sb} = 0$, $f_{ub} = 500\ N/mm^2$, $\gamma_{mb} = 1.25$

$$= \frac{500}{\sqrt{3} \times 1.25}(2 \times 296.5) = 136.94 \text{ kN….. (1)}$$

- ## Bearing strength of bolts - Page 75 CL10.3.4 , IS 800 2007

Pitch, p = 2.5× d = 2.5 × 22 = 55 mm

Edge distance e = 1.7 × d_o= 1.7× 24 = 40.8 ≈ 45 mm (d_o *is the dia of bolt hole,* (22 + 2)

$f_{ub} = 500\ N/mm^2$, $f_u = 410\ N/mm^2$, t = 10 mm, d = 22 mm

$k_b = \frac{45}{3 \times 24} = 0.63$, $k_b = \frac{55}{3 \times 24} - 0.25 = 0.513$, $\frac{f_{ub}}{f_u} = \frac{500}{410} = 1.22$, 1.0

$$V_{dpb} = 2.5 \times 0.513 \times 22 \times 10 \times \frac{410}{1.25} = 92.54 \text{ kN....... (2)}$$

Bolt value = Minimum of (1) and (2) = 92.54 kN

No of bolts = $\frac{360}{92.54} = 3.8 \approx 4$

Hence provide 2 ISA 80 × 80 × 10 mm with 4 bolts

- ## Design of Bottom chord members (AL, LK, KJ)-Tension members

Taking Max Force = 208 kN
Factored Tensile Force T_{dg} = 208 x 1.5 = 312 kN
Tensile strength due to gross section yielding, Page 32, CL 6.2 (IS 800)

$$T_{dg} = \frac{A_g f_y}{\gamma_{m0}}$$

Gross area, $A_g = \frac{312 \times 10^3 \times 1.1}{250} = 1372.8\ mm^2$

Since it is bottom member, increase the area by 30% = 1.3×1372.8 = 1784.64 mm^2= 17.84 cm^2

From Steel table Page No 18, table 6 (Double angle)

ISA 8080	80 x 80	6.0	14.6	143.2	18.58	112.0	19.2	2.46
		8.0	19.2	188.4	24.42	145.0	25.2	2.44
		10.0	23.6	231.5	30.10	175.4	31.0	2.41
		12.0	28.0	274.7	35.62	203.8	36.6	2.39

Try 2 ISA 80× 80 × 6 mm (two angles back to back) $with\ 10\ mm\ gap$

$A_g = 18.58\ cm^2 = 1858\ mm^2$

- ## Connections

Use M 22, class 5.6 (Same bolt diameter for all connection design)

- ## Shear strength of bolts

Assume fully threaded bolts, number of shear planes n_n = 2 ,(double angle), n_s = 0 (no shank portion)

$A_{ns} = 0.78 \times \frac{\pi}{4} \times 22^2 = 296.5\ mm^2$, $A_{sb} = 0$

$$V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}} = \frac{f_{ub}}{\sqrt{3}\,\gamma_{mb}}(n_n A_{nb} + n_s A_{sb})$$

$= \frac{500}{\sqrt{3} \times 1.25}(2 \times 296.5)$ = 136.94 kN…(1)

- ## Bearing strength of bolts

$$V_{dpb} = 2.5 k_b d t \frac{f_u}{\gamma_{mb}}$$

$V_{dpb} = 2.5 \times 0.513 \times 22 \times 6 \times \frac{410}{1.25}$ = 55.53 kN…. (2)

Bolt value = 55.53 kN (least of (1) or (2)

No of bolts = $\frac{312}{55.53} = 6$

Hence provide 2 ISA 80 × 80 × 6 mm with 6 bolts

(In case of High Strength Friction Grip Bolts (HSFG) - Shear capacity only needs to be calculated by CL 10.4.3, page 76 and then calculate no of bolts based on shear capacity) No need of calculating "bearing strength of bolts".

$V_{dsf} = V_{nsf} / \gamma_{mf}$

V_{nsf} = nominal shear capacity of a bolt as governed by slip for friction type connection, calculated as follows:

$$V_{nsf} = \mu_f \, n_e \, K_h \, F_o$$

where

μ_f = coefficient of friction (slip factor) as specified in Table 20 ($\mu_f = 0.55$),

n_e = number of effective interfaces offering frictional resistance to slip,

K_h = 1.0 for fasteners in clearance holes,

= 0.85 for fasteners in oversized and short slotted holes and for fasteners in long slotted holes loaded perpendicular to the slot,

= 0.7 for fasteners in long slotted holes loaded parallel to the slot,

γ_{mf} = 1.10 (if slip resistance is designed at service load),

= 1.25 (if slip resistance is designed at ultimate load),

F_o = minimum bolt tension (proof load) at installation and may be taken as $A_{nb} f_0$,

A_{nb} = net area of the bolt at threads, and

f_o = proof stress ($= 0.70 f_{ub}$).

To find partial safety factor

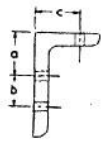
Table 5 Partial Safety Factor for Materials, γ_m

(*Clause* 5.4.1)

Sl No.	Definition	Partial Safety Factor	
i)	Resistance, governed by yielding, γ_{m0}	1.10	
ii)	Resistance of member to buckling, γ_{m0}	1.10	
iii)	Resistance, governed by ultimate stress, γ_{m1}	1.25	
iv)	Resistance of connection:	*Shop Fabrications*	*Field Fabrications*
	a) Bolts-Friction Type, γ_{mf}	1.25	1.25
	b) Bolts-Bearing Type, γ_{mb}	1.25	1.25
	c) Rivets, γ_{mr}	1.25	1.25
	d) Welds, γ_{mw}	1.25	1.50

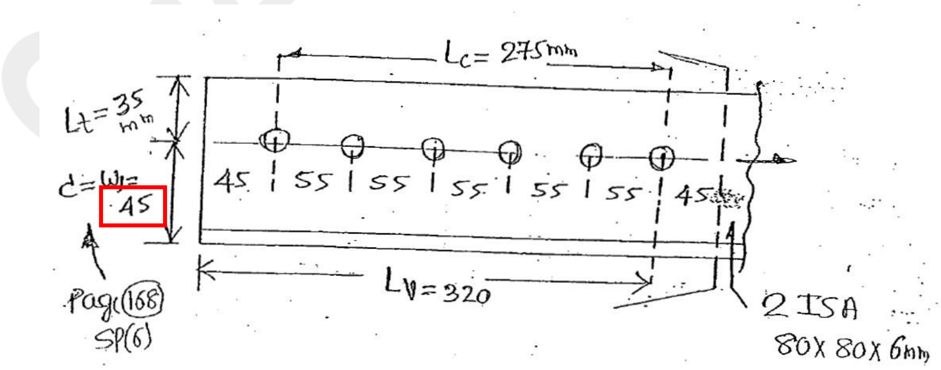
30

TABLE XXXI RIVET GAUGE DISTANCES IN LEGS OF ANGLES



Leg Size	Double Row of Rivets a	Double Row of Rivets b	Single Row of Rivets c	Maximum Rivet Size for Double Row of Rivets
mm	mm	mm	mm	mm
200	75	85	115	27
150	55	65	90	22
130	50	55	80	20
125	45	55	75	20
115	45	50	70	12
110	45	45	65	12
100	40	40	60	12
95	—	—	55	—
90	—	—	50	—
80	—	—	45	—
75	—	—	40	—
70	—	—	40	—
65	—	—	35	—
60	—	—	35	—
55	—	—	30	—
50	—	—	28	—
45	—	—	25	—
40	—	—	21	—
35	—	—	19	—
30	—	—	17	—
25	—	—	15	—
20	—	—	12	—

SP 6, Page 168

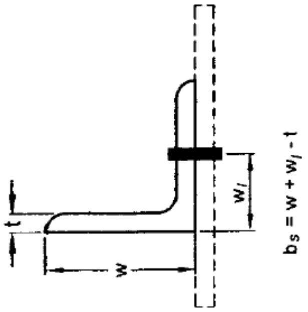


Longitudinal section of a double angle with bolts

- **Check for rupture (tension member)**Page 33 CL 6.3.3 IS 800 - 2007w = outstand leg width or **width of unconnected leg** = 80 mm

L_c = **Distance between the outermost bolts in the end joint measured along the load direction** = 275 mm

t = 6mm
$f_u = 410\ N/mm^2$, Ultimate strength of material
$f_y = 250\ N/mm^2$, Yield strength of material



$b_s = w + w_t - t = 80 + 45 - 6 = 119$ mm

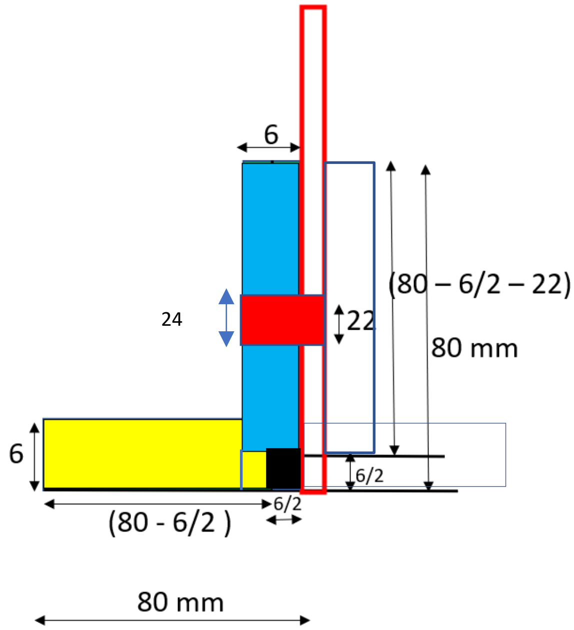
$$\beta = 1.4 - 0.076\,(w/t)\,(f_y/f_u)\,(b_s/L_c)$$

$= 1.132$

Also find $\frac{\gamma_{m0}\, f_u}{f_y\, \gamma_{m1}}$ **= 1.44**

As per IS 800 – 2007, β = 1.132 ≥ 0.7 ≤ 1.44

Hence take β = 1.132



Angle section attached to gusset plate

t =

A_{go} = Gross area of outstanding or unconnected leg (without bolt) = $\left(B - \frac{t}{2}\right) t$

$\left(80 - \frac{6}{2}\right) 6 = 462\ mm^2$

t

A_{nc}

= $(A - d_o - {}_{2}) t = ($

Net area of connected leg (subtract area of bolt hole)

=

$(80 - 24 - \frac{6}{2})\, 6 = 318\ mm^2$ where $diameter\ of\ bolt\ hole, d_o = 22 + 2 = 24\ mm$

$$T_{dn} = 0.9\, A_{nc} f_u / \gamma_{m1} + \beta A_{go}\, f_y / \gamma_{m0}$$

For double angle section multiply the value of T_{dn} by "2"

For double angle $T_{dn} = 2 \times (0.9 \times 318 \times \frac{410}{1.25} + 1.132 \times 462 \times \frac{250}{})$

$= 425.47$ kN > 312 kN , It is safe.

- ## Check for block shear

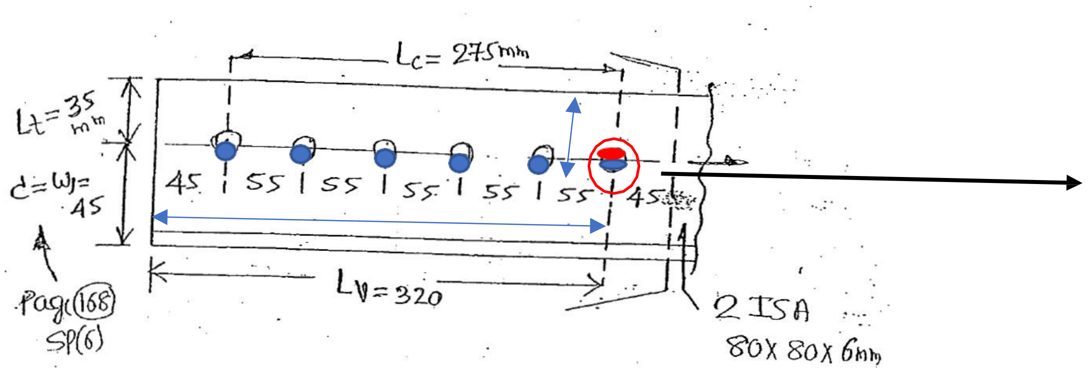
Page 33 CL 6.4.1

The block shear strength, T_{db} of connection shall be taken as the smaller of,

$$T_{db} = [A_{vg} f_y/(\sqrt{3}\gamma_{m0}) + 0.9A_{tn} f_u/\gamma_{m1}]$$

or

$$T_{db} = (0.9A_{vn} f_u/(\sqrt{3}\gamma_{m1}) + A_{tg} f_y/\gamma_{m0})$$



Length of shearing action, L_v = 45 + 5 × 55 = 320 mm
Length of tensile action, L_t = 35 mm
Gross area in shear parallel to force , $A_{vg} = L_v \times t = 320 \times 6 = 1920\ mm^2$
Net area in shear parallel to force, $A_{vn} = A_{vg} - 5.5 \times d_o \times t$
$= 1920 - 5.5 \times 24 \times 6 = 1128\ mm^2$
Gross area in tension perpendicular to force, $A_{tg} = L_t \times t = 35 \times 6 = 210\ mm^2$
Net area in tension perpendicular to force, A_{tn} = 210 – 0.5 × 24 × 6 =138 mm^2

$$T_{db} = [A_{vg} f_y/(\sqrt{3}\gamma_{m0}) + 0.9A_{tn} f_u/\gamma_{m1}]$$

For two angles, $T_{db} = 2 \times [1920 \times \frac{250}{\sqrt{3}\times 1.1} + 0.9 \times 138 \times \frac{410}{1.25}]$ = 585.3 kN > 312 kN

$$T_{db} = (0.9A_{vn} f_u/(\sqrt{3}\gamma_{m1}) + A_{tg} f_y/\gamma_{m0})$$

For two angles, $T_{db} = 2 \times [0.9 \times 1128 \times \frac{410}{1.25\times\sqrt{3}} + 210 \times \frac{250}{1.1}]$ = 479.9 kN> 312 kN

Hence 2 ISA 80 × 80 × 6 mm is safe .

- ## Design of "Inner Tension members"

Member Force CK = 18 kN
Factored Force, T_{dg}= 1.5 × 18 = 27 kN
Tensile strength due to gross section yielding, Page 32, CL 6.2 (IS 800)

$$T_{dg} = \frac{A_g f_y}{\gamma_{m0}}:$$

A_g = **118.8 mm²** = 1.18cm²
Use Steel table and Take single angle section (it is a inner member)
Table 1, Page 2 , Choose ISA 50 X 50 X 6 mm ,
Area = 5.68 cm^2 =568 **mm^2**

- Connection – Bolts - M 12 Class 5.6
  Diameter of bolt hole $d_o = 12 + 1 = 13\ mm$

- # Shear strength of bolts

  Assume fully threaded bolts, number of shear planes n_n = 1 (single angle section) , n_s = 0 (no shank portion)

$A_{nb} = 0.78 \times \frac{\pi}{4} \times 12^2 = 88.2\ mm^2$, $A_{sb} = 0$, , f_{ub} = 500 N/mm^2

$$V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}} = \frac{f_{ub}}{\sqrt{3}\,\gamma_{mb}} (n_n\, A_{nb} + n_s\, A_{sb})$$

0

$$= \frac{500}{\sqrt{3} \times 1.25} (1 \times 88.2) = 20.36 \text{ kN…..(1)}$$

- **Bearing strength of bolts**

Pitch, p = 2.5× d = 2.5× 12 = 30 mm, t = 6 mm
Edge distance e = 1.7 × d_o= 1.7× 13 = 22.1 = 25 mm

where k_b = smaller of $\frac{e}{3d_0}$, $\frac{p}{3d_0} - 0.25$, $\frac{f_{ub}}{f_u}$, and 1.0

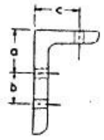
Smallest of : $k_b = \frac{25}{3 \times 13} = 0.64$, $k_b = \frac{30}{3 \times 13} - 0.25 = 0.519$, $\frac{f_{ub}}{f_u} = \frac{500}{410} = 1.22$, 1.0

$V_{dpb} = 2.5 \times 0.519 \times 12 \times 6 \times \frac{410}{1.25} =$ 30.64 kN…..(2)

Bolt value = 20.36 kN (LEAST OF (1) and (2))

No of Bolts = 27/ 20.36 = 2 (Minimum no of bolts =2)

TABLE XXXI RIVET GAUGE DISTANCES IN LEGS OF ANGLES



Leg Size	Double Row of Rivets a	Double Row of Rivets b	Single Row of Rivets c	Maximum Rivet Size for Double Row of Rivets
mm	mm	mm	mm	mm
200	75	85	115	27
150	55	65	90	22
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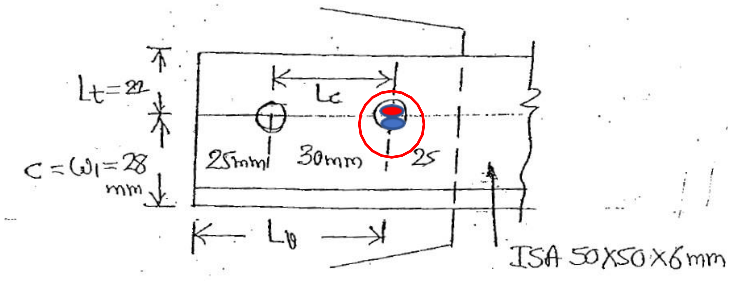
125	45	55	75	20
115	45	50	70	12
110	45	45	65	12
100	40	40	60	12
95	—	—	55	—
90	—	—	50	—
80	—	—	45	—
75	—	—	40	—
70	—	—	40	—
65	—	—	35	—
60	—	—	35	—
55	—	—	30	—
50	—	—	28	—

Use SP6, Page 168, Note down gauge distance for leg size 50 mm, which is 28 mm



- ## Check for rupture

w, width of unconnected leg = 50 mm, t = 6 mm, distance between end bolts, Lc = 30 mm,

$b_s = w + w_t - t$

bs = 50 +28 - 6 = 72 mm

A_{go} = Gross area of unconnected leg = $(B - \frac{t}{2})\,t = (50 - \frac{6}{2})\,6 = 282\ mm^2$

A_{nc} = Net area of connected leg = $(A - d_o - \frac{t}{2})\,t = (50 - 13 - \frac{6}{2})\,6 = 204\ mm^2$

Also calculate **β = 1.4 – 0.076 x (50 /6) x (250/410) x (72/30) = 0.47 ≤ 0.7**

$\frac{\gamma_{m0} f_u}{f_y \gamma_{m1}}$ **= 1.44 , The value of β = 0.47 ≤ 1.44 .**

Let us take β = 0.7, since calculated value of β is less than 0.7

For single angle $T_{dn} = 1 \times (0.9 \times 204 \times \frac{410}{1.25} + 0.7 \times 282 \times \frac{250}{1.1}) =$

The block shear strength, T_{db} of connection shall be taken as the smaller of,

= 105.08 kN >27 kN, it is safe.

$$T_{db} = [A_{vg} f_y/(\sqrt{3}\,\gamma_{m0}) + 0.9 A_{tn} f_u/\gamma_{m1}]$$

- **Check for block shear**

or

Page 33 CL 6.4.1

$$T_{db} = (0.9 A_{vn} f_u/(\sqrt{3}\,\gamma_{m1}) + A_{tg} f_y/\gamma_{m0})$$

$A_{vg} = L_v \times t = (30 + 25) \times 6 = 330\ mm^2$ $A_{vn} = 330 - 1.5 \times 13 \times 6 = 213\ mm^2$

$A_{tg} = L_t \times t = 22 \times 6 = 132\ mm^2$ $A_{tn} = 132 - 0.5 \times 13 \times 6 = 93\ mm^2$

For single angles, $T_{db} = 1 \times [\,330 \times \frac{250}{1.1 \times \sqrt{3}} + 0.9 \times 93 \times \frac{410}{1.25}\,] = 70.75 kN > 27 kN$

$$T_{db} = (0.9 A_{vn} f_u/(\sqrt{3}\,\gamma_{m1}) + A_{tg} f_y/\gamma_{m0})$$

For two angles, $T_{db} = 1 \times [\,0.9 \times 213 \times \frac{410}{1.25 \times \sqrt{3}} + 132 \times \frac{250}{1.1}\,] = 66.30 kN > 27 kN$

Hence ISA 50 × 50 × 6 mm is safe .

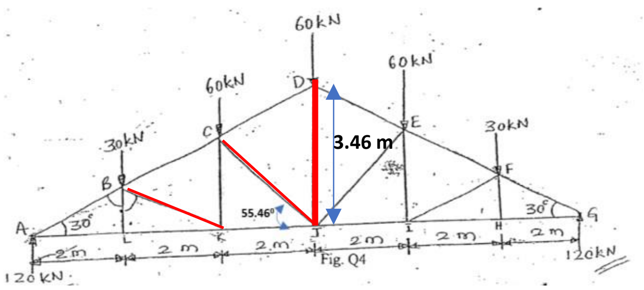


Fig. Q4

- **Design of Inner compression members (BK, CJ, DJ)**

Taking Maximum Force = 66.0 kN

Factored Force = 1.5 × 66.0 = 99. kN

Take the member which is having maximum length ie DJ = 3.46 m

Maximum Length = 3.46 m

Assume f_{cd} = 50 N/mm^2

(Assume f_{cd} = 40 -120 N/mm^2 based on load and experience)

Gross Area, $A_g = \frac{Factored\ Force}{f_{cd}} = \frac{99.07 \times 10^3}{50}$ = 1981.4 mm^2 = 19.81 cm^2

From Steel Table, Try Single ISA 100 x 100 x 10 mm (Table 1, Page 4)

Area = 19.03 cm^2
$r_{xx} = r_{yy}$ = 3.05 cm = 30.5 mm
r_{uu} = 3.85 cm, r_{vv} = 1.94 cm
So r_{min} = 1.94 cm = 19.4 mm

Here load is acting through only one leg it will be subjected to torsional buckling
Using Page 48, Table 12 and using CL 7.5.1.2 Loaded through one leg (IS 800 2007)

Assuming bolts ≥ 2 and hinged end conditions with gusset plate, $k_1 = 0.7$, $k_2 = 0.6$, $k_3 = 5$
Assuming Effective length, $L_{eff} = 0.85 \times L = 0.85 \times 3460 = 2940\ mm$
ε = 1 , E = 2 × 10^5 N/mm^2 $b_1 = b_2$ = 100 mm, t = 10 mm

$$\lambda_{vv} = \frac{\frac{Leff}{rvv}}{s\sqrt{\frac{\pi^2 \varepsilon}{250}}} = 1.705$$

λ_Φ = 0.1125, $k_1 = 0.7$, $k_2 = 0.6$, $k_3 = 5$

$$\lambda_e = \sqrt{k_1 + k_2\,\lambda_{vv}^2 + k_3\,\lambda_\varphi^2}$$

Equivalent slenderness ratio, λ_e = 1.589
From Table 10, Page No 44, Choose the buckling class based on type of section
From Page 34, CL 7.1.2.1, IS 800-2007

Put α = 0.49, λ_e = 1.589, Φ = 2.102

$$f_{cd} = \frac{f_y / \gamma_{m0}}{\phi + \left[\phi^2 - \lambda^2\right]^{0.5}}$$

(Single angle section) Design compressive stress, $f_{cd} = \frac{250}{1.1(2.102+(2.102^2-1.589^2)^{0.5})}$ = 108.47 N/mm^2

The design compressive strength of a member is given by:
Load $P_d = A \times f_{cd}$ = 1903 × 108.47 = 206.42 kN > 99.07 kN
Hence it is safe.

Design of connection using M22, class 5.6 (same diameter bolt)
Benefit- No need to do shear strength calculations, (for single angle, n_n = 1)

• Shear strength of bolts

Assume fully threaded bolts, number of shear planes $n_n = 1$ (Single angle section), $n_s = 0$ (no shank portion)

$$A_{ns} = 0.78 \times \frac{\pi}{4} \times 22^2 = 296.5\ mm^2 \quad , A_{sb} = 0$$

$$V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}} = \frac{f_{ub}}{\sqrt{3}\,\gamma_{mb}}(n_n\,A_{nb} + n_s\,A_{sb})$$

$$= \frac{500}{\sqrt{3} \times 1.25}(1 \times 296.5) = 68.47 \text{ kN….. (1)}$$

• Bearing strength of bolts

$$V_{dpb} = 2.5 k_b d t \frac{f_u}{\gamma_{mb}}$$

Pitch, p = 2.5× d = 2.5× 22 = 55 mm
Edge distance e = 1.7 × d_o= 1.7× 24 = 40.8 = 45 mm

where k_b = smaller of $\frac{e}{3d_0}$, $\frac{p}{3d_0} - 0.25$, $\frac{f_{ub}}{f_u}$, and 1.0

$k_b = \frac{45}{3 \times 24} = 0.63$, $k_b = \frac{55}{3 \times 24} - 0.25 = 0.513$, $\frac{f_{ub}}{f_u} = \frac{500}{410} = 1.22$, 1.0

$$V_{dpb} = 2.5 \times 0.513 \times 22 \times 10 \times \frac{410}{1.25} = 92.54\text{kN…..(2)}$$

Bolt value = Minimum of (1) and (2) = 68.47kN
No of bolts = $\frac{99}{68.47} = 2$

• Design of Bearing Plate

Support Reaction = 120 kN
Factored Reaction = 1.5 × 120 = 180 kN

CL 34.4, Page 56 IS 456 2000

Using M 20 concrete, Bearing pressure of concrete = 0.45 f_{ck} = $0.45 \times 20 = 9\ \mathrm{N}/mm^2$

IS 456 : 2000

For working stress method of design the permissible bearing stress on full area of concrete shall be taken as $0.25 f_{ck}$; for limit state method of design the permissible bearing stress shall be 0.45 f_{ck}.

Area of Bearing plate = $\frac{180 \times 10^3}{9}$ = 20000 mm^2

Provide square plate = $\sqrt{20000}$ = 141.4 mm

Provide 200 mm × 200 mm as base dimensions

- Calculation of thickness ‘t’ of the bearing plate

Upward pressure, q_o = Support Reaction / Size of plate = 180 x 10^3/ (200 x 200) = 4.5 N/mm^2
For 1mm, it is 4.5 N/mm
Moment at section XX (near the inner face of angle section), $M_{xx} = 4.5 \times 87 \times \frac{87}{2} = 17030.25\ Nmm$
Take b = 1mm, d = (t + 8)

Using Bending formula $M_{xx} = \sigma_b \times Z = \frac{f_y}{\gamma_{mo}} \times b \times \frac{d^2}{6}$ (Z is section modulus)

$17.03 \times 10^3 = \frac{250}{1.1} \times 1 \times \frac{(t+8)^2}{6}$
t = 13.20 mm ≈ 14 mm
Bearing plate 200 × 200 × 14 mm

- **Design of Anchor Bolts**

Given Uplift force = 18 kN at each support
Assuming two Anchor bolts at each support, Force on each anchor bolt = 18/2 =9.0 kN
Factored force on anchor bolt = 9.0 x 1.5 = 13.5 kN

From CL 26.2.1.1 Bond stress for M 20 (RCC column M 20) = **1.2** N/mm^2, increase by **60**%
(Page 43, IS 456 2000) = 1.2 x 1.6
To find length of Anchor bolt
Let us assume 18 mm diameter Anchor Bolt
Factored Uplift Force = Force developed around the anchor bolt
= (Circumference $\times$ length) $\times$ Bond stress
$13.5 \times 10^3 = \pi \times 18 \times \text{length} \times (1.2 \times \mathbf{1.6})$
$\text{length} = 124.34 \text{ mm} \approx 125 \text{ mm}$
Provide 2, 18 mm Φ anchor bolts of length 125 mm at each end.