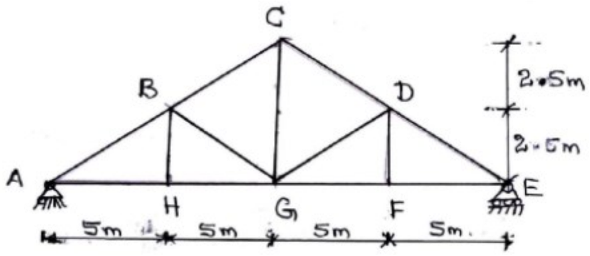
USN | | | | | | | | | |

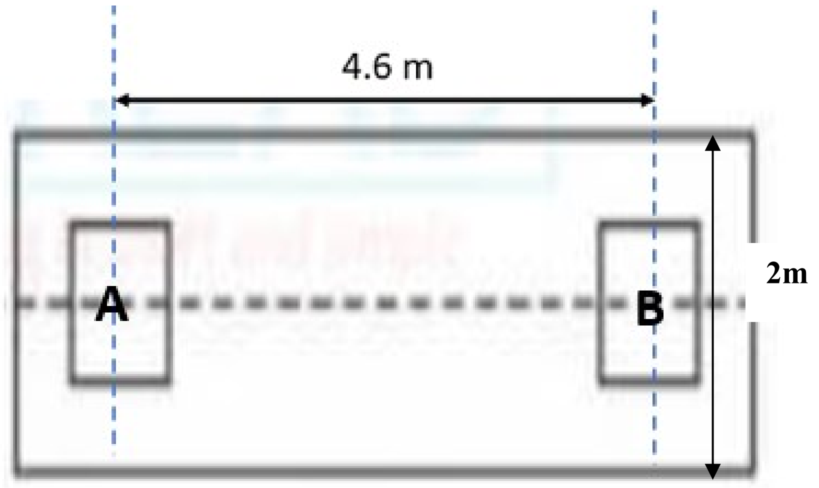
Internal Assessment Test 4 – Feb. 2022

Sub:	Design of RC and steel structural elements				Sub Code:	18CV72/17CV72	Branch:	CIVIL
Date:		Duration:	90 min's	Max Marks: 50	Sem / Sec:	ALL		OBE

	Answer any one Questions- Use of IS 456 -2000/IS 800 is permitted	MARKS	CO	RBT
1 (a)	Design a combined rectangular slab type footing for two columns A and B to carry loads of 600 kN and 900 kN. The cross section of column A is 300 x 300 mm and 400x 400 mm. The width of the footing is restricted to 1.8 m. The centre to centre spacing between the columns is 3.6 m. The safe bearing capacity of the soil is 175 kN/m^2. Use M20 concrete and Fe 415 steel. The design must include all necessary checks and draw the reinforcement details.	[50]	CO1	L2
	OR			
2 (a)	The centre line of the roof truss is shown. The magnitude of forces under service conditions are Top chord member = 120 kN Compression, Bottom Chord member = 100 kN Tension, Interior members = 60kN compression. Use M16 bolts of grade 4.6. Design all the members. 	[50]	CO1	L2

SOLUTIONS

Solutions



- **Footing base dimensions**

Assuming Δ P, the self-weight of the combined footing plus backfill to constitute 10 or 15 percent of the total column loads,

Δ P = (700 + 1200) x 15/100 = 285 kN

$P_1 + P_2$ = 700 +1200 = 1900 kN

Allowable soil pressure or safe bearing capacity, q_a = 130 kN/m^2

$Area\ of\ the\ footing, A_{req} = \frac{P_1 + P_2 + \Delta P}{q_a} = 16.8$ m^2

Width of footing, B = 2m (Given in question)

Total Length of footing, L = $\frac{A_{req}}{B} = \frac{16.8}{2} = 8.4$ m

- **Locate the point of application of the column loads**

In order to obtain a uniform soil pressure distribution, the line of action or point of application of the resultant column load must pass through the centroid of the footing.

Assuming a load factor of 1.5, the factored column loads are:

- $P_{u1} = 700 \times 1.5 = 1050$ kN; $P_{u2} = 1200 \times 1.5 = 1800$ kN ⇒ $P_{u1} + P_{u2} = 2850$ kN

  Let $\bar{x}$ be the centroid of the column loads, where s = 4.6 m

$$\Rightarrow \bar{x} = \frac{P_{u2}\ s}{P_{u1+} + P_{u2}} =$$

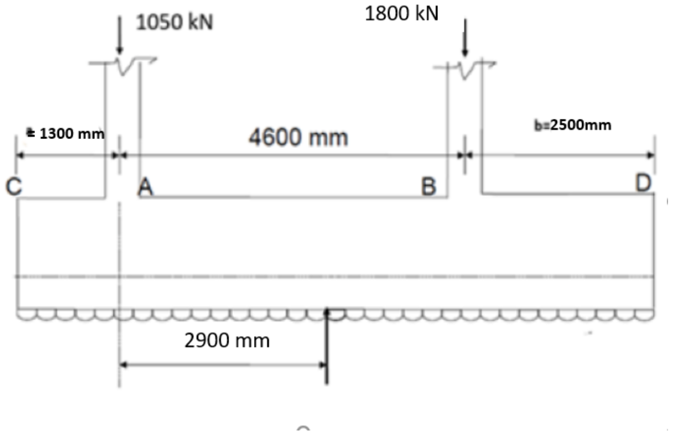
$$= \frac{1800 \times 4.6}{1050 + 1800} = 2.9 \text{ m}$$

If the cantilever projection of footing beyond column A is 'a' then,

a + 2.9 = L /2 = a = 8.4/2 – 2.9 = 1.3 m

Similarly, if the cantilever projection of footing beyond Column B is 'b' then,

b = 8.4 - 1.3 - 4.6 = 2.5 m

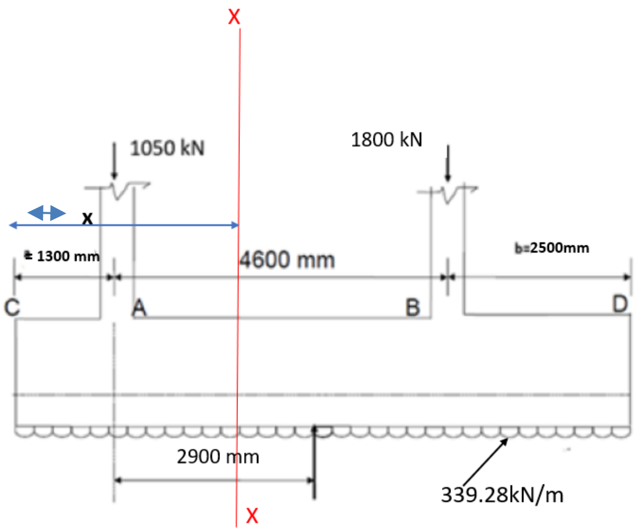


- **Uniformly distributed load acting in upward direction (soil pressure)**

Treating the footing as a wide beam (B = 2000 mm) in the longitudinal direction, the uniformly distributed load (acting upward) is given by q_{uB}

$$q_{uB} = \frac{P_{u1}+P_{u2}}{L} = \frac{1050+1800}{8.4} = 339.28 \text{ kN/m}$$

- **Shear force calculations**



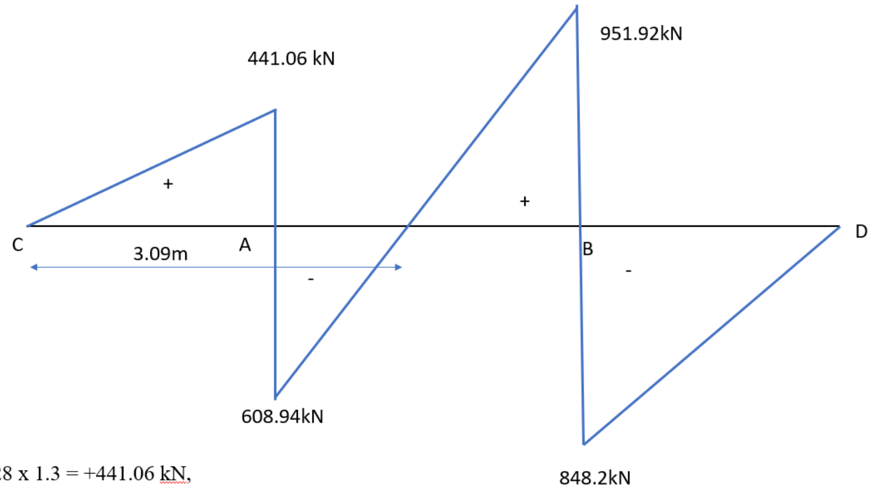
- Shear force at A, just before 1050 kN, left of section XX, V_{AC}= +339.28 x 1.3 = +441.06 kN
- Shear force at A, just after 1050 kN, left of section XX, V_{AB} = -1050+ 339.28 x 1.3 = -608.94 kN
- Shear force at B just after 1800kN, right of section XX , V_{BA} = +1800 – 339.28 x 2.5= + 951.92 kN
- Shear force at B just before 1800kN, right of section XX, V_{BD}=339.28 x 2.5= -848.2 kN

- **Location of zero shear , Left of section XX**

339.28 × X – 1050 = 0 , **location of zero shear,**

339.28 × X = 1050 , 1050/339.28 = X, X =3.09m

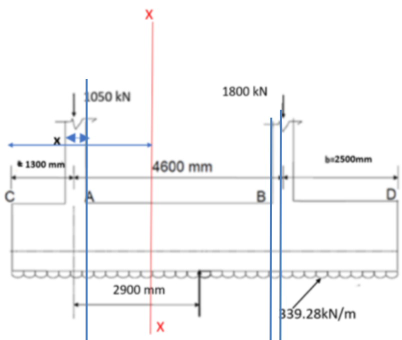
X = 3.09 m from C



- V_{AC}= 339.28 x 1.3 = +441.06 kN,
- V_{AB} = -1050+ 339.28 x 1.3= -608.94 kN
- V_{BA} = 1800 – 339.28 x 2.5= + 951.92 kN
- V_{BD}=339.28 x 2.5= -848.2 kN

Shear force diagram

- **Bending moment calculations**

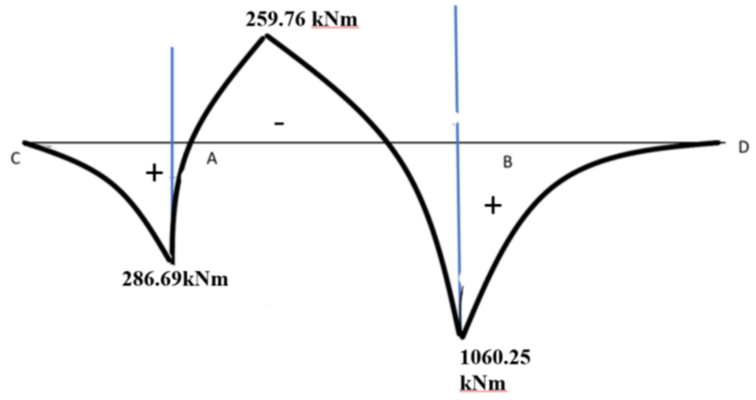


- BM at A, just before 1050kN, left of section XX, M_{AC}= 339.28 x 1.3 x1.3 /2 =+ 286.69 kNm
- BM at just at the inner face of Column A(1050kN), left of section XX,

  M_{AB} = -1050 x 0.35/2 + 339.28 x (1.3 + 0.35/2) x (1.3 + 0.35/2)/2

  = -1050 x 0.35/2 + 339.28 x (1.3 + 0.172) x (1.3 +0.172)/2 = + 185.32kNm
- Negative Bending moment at X = 3.09 m (Location of zero shear)

  Mu_{-} = 339.28 × $(3.09)^2$/2 – 1050 × (3.09 -1.3) = - 259.76 kNm
- BM at B, just before 1800 kN , right of section XX = + 339.28 × 2.5^2/2 = +1060.25 kNm
- BM at B, just after the inner face of Column B (1800 kN) , right of section XX =
- 339.28 × **(2.5 +0.4/2)**^{2}/2 – 1800 × **0.4/2** = + 876.67 kNm

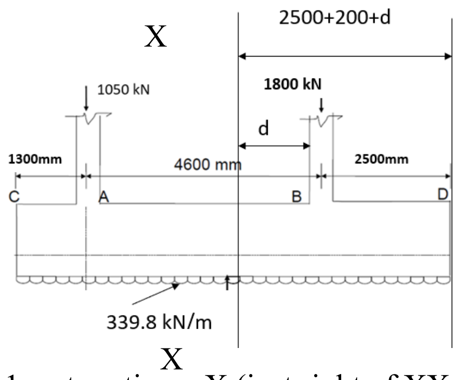


Bending moment diagram

- **Thickness of footing or effective depth of footing based on shear**

One-way shear (longitudinal): V_{u1} calculate it at a distance "d" from the edge of the heavier column, where "d" is the effective depth of the footing.

The critical section (**always for column with heavier load**) for one-way shear is located at a distance d from the (**inner)**face of column B, and has a value



Critical One-way shear force, V_{u1} at section XX (just right of XX section) =

Column load (B) - Uniformly distributed upward load intensity × (2500 + 200 + d)

= (1800 – 339.28 x (2.5 + 0. 200 + d)) = (882.54 – 339.28 x d) kN … (1)

Take τ_c = 0.48 N/mm^2 (for M 20 concrete, Assuming Percentage of steel as, pt = 0.50%) IS 456 2000, page 73, table 19

Design shear strength of concrete, $V_{uc} = \tau_c \times B \times d = 0.48 \times B \times d$

Equate V_{uc} *and* V_{u1}

B is width of footing = 2000 mm

$V_{uc} = 0.48 \times 2000 \times d = (960d)$ N…. (2)

Equating one-way shear force and design shear strength of concrete, (1) = (2)

$V_{u1} = V_{uc} \Rightarrow (882.54 \times 10^3 - 339.28 \times d) = 960d$, $882.54 \times 10^3 = 1299.8\, d$

⇒ Effective depth of footing, d = 679.25 **mm Rounded to 680 mm**

Use 20 mm φ bars with a clear cover of **75** mm, **Taking an overall depth or thickness of the footing**

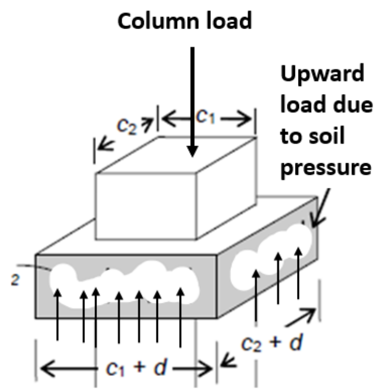
$\boldsymbol{D} = d + 75 + 20/2$ = 680 + 75 + 20/2 = 765 mm

- **Two-way shear force for columns *A* and *B* (Punching shear)**

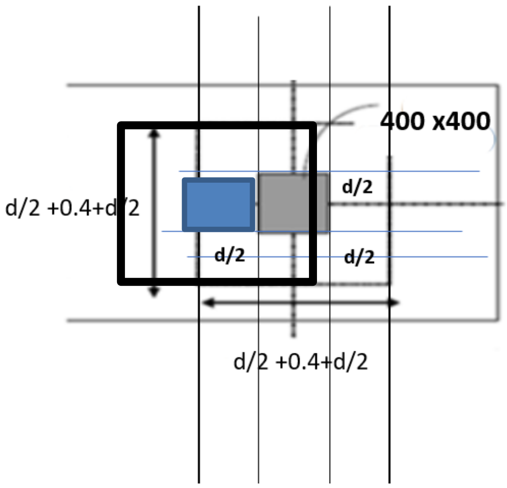
Two-way shear or punching shear ***(we need to consider the upward soil pressure not upward soil intensity) * Since it is acting on an area.***

Factored soil pressure or Upward soil pressure, q_u = (339.28) / (B ×1) = (339.8/2) = 169.64kN/m^2

The critical section is located $d/2$ from the periphery of columns A and B.



Shear stresses in footing slab due to punching shear

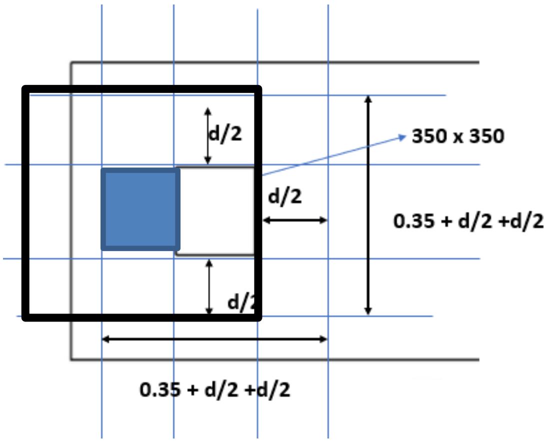


Punching shear or Two- way shear calculations for heavier Column B

V_{u2} = 1800 – 169.64 (0.4 + 0.680/2 + 0.680/2) X (0.4+0.680/2 + 0.680/2)

= 1602.13 kN @ B (Heavier column)

Punching shear or Two-way shear for Column A (350 mm x 350 mm)



Punching shear or Two-way shear @ A,

Two-way shear V_{u2} = (Column load at A) 1050 – 169.64 x (0.35 +0.680/2 + 0.680/2) x (0.35 +0.680/2 + 0.680/2)

= 870 kN @ A (Lighter column)

- If no shear reinforcement is provided, Page 58, IS 456, Clause 31.6.3.1, calculated shear stress at critical section shall not exceed $k_s\,(0.25\sqrt{f_{ck}}\,)$

where

k_s = $(0.5 + \beta_c)$ but not greater than 1, β_c being the ratio of short side to long side of the column/ capital; and

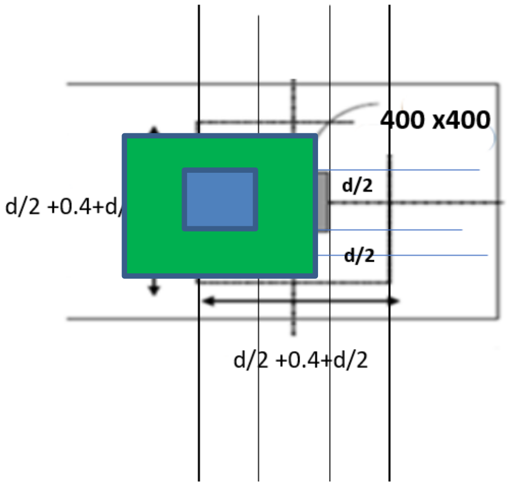
τ_c = 0.25 $\sqrt{f_{ck}}$ in limit state method of design, and 0.16 $\sqrt{f_{ck}}$ in working stress method of design.

For square columns, ks = (0.5 +βc) , βc = 350/ 350 = 400/400 = 1.0 , ks =(0.5 +1) but it should not be greater than 1, hence ks = 1

Permissible shear stress, $\tau_{c2} = k_s\,(0.25\sqrt{f_{ck}}\,)$ = 1.0 × 0.25 ×√ 20 = **1.118** N/mm^2

Permissible two-way shear force for column *B (heavier column)*

Permissible two-way shear force, V_{uc} = Permissible shear stress × (Area of the footing slab enclosed by the perimeter of the critical section)



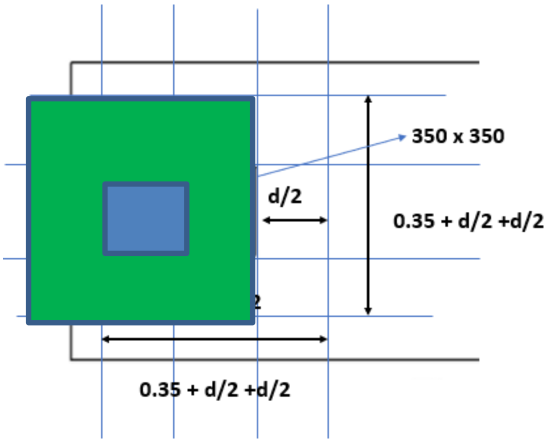
Perimeter of critical section (Green coloured area) = 4 X (400 + 680/2 + 680/2)

V_{uc} = 1.118 x [**4 X (400 + 680/2 + 680/2)**] × 680 = 3284.24 kN @ B

In the similar way lets calculate for Column A

Permissible two-way shear force for Column A

V_{uc} = 1.118 × [(350 + 680/2 + 680/2) × 4] × 680 = 3132.18kN @ A



Compare whether permissible two way shear force is greater than two shear way (Actual) force

V_{uc} = 3284.23 kN > V_{u2} = 1602.82 kN **@ B It is safe.**

V_{uc} = 3132.18**kN > V_{u2}** = 870.00kN **@ A . It is Safe.**

Hence safe against two way or punching shear, (if not provide shear reinforcement- stirrups or bent up bars)

- **Design of longitudinal flexural reinforcement**

Maximum 'negative' moment: *Mu* = - 259.76 kNm	Maximum 'positive' moment: *Mu* = + 1060.25 kNm

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$$M_u = 0.87\, f_y\, A_{st}\, d\left(1 - \frac{A_{st} f_y}{bd\, f_{ck}}\right)$$

Mu = 259.76 × 10^6 N mm
B = b = 2000 mm, fck = 20 N/mm^2,
fy = 415N/mm^2
d = 680 mm
D = 765 mm
Ast provided = 1075.67 mm^2
Check for (*Ast*)*min* = 0.0012 *BD* =
0.0012 × 2000 × 765 = 1836 mm^2
Ast provided < (*Ast*)*min, Hence provide (Ast)min*
But we have assumed pt = 0.5
pt = 100 Ast, req/ (B × d)
Choose 20 mm diameter bars, calculate no of bars
= 1836 /(π/4 x 20^2) = 6
Provide 6 # 20 mm diameter bars at top

- **Development length L_d = 47 x dia of bar**
= 47 x 20 = 940 mm

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$$M_u = 0.87\, f_y\, A_{st}\, d\left(1 - \frac{A_{st} f_y}{bd\, f_{ck}}\right)$$

Mu = 1060.25 × 10^6 N mm
B = b = 2000 mm
d = 680 mm
Ast provided = 4648.12 mm^2
Check for (*Ast*)*min* = 0.0012 *BD* =
0.0012 × 2000 × 765 = 1836 mm^2
No of 20 mm dia bars = 4648.12/(π/4 x 20^2)
= 15
Provide 15 # 20mm diameter bars at bottom

- **Development length L_d = 47 x dia of bar**
= 47 x 20 = 940 mm

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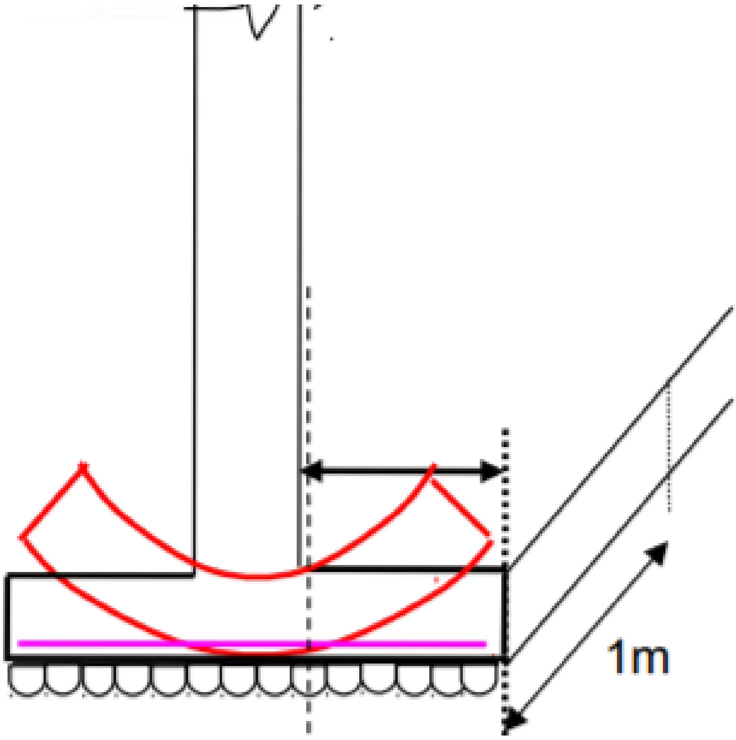
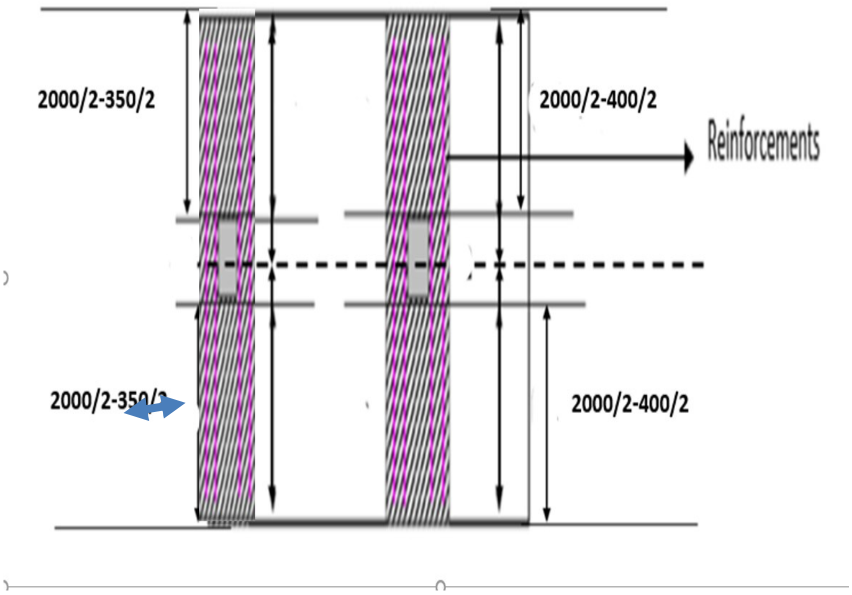
Design of column strips as transverse beams



Transverse bending of footing



Transverse beam under column A	*Transverse beam under column* B
• Factored Column load A per width of footing = 1050/2.0 = 525 kN/m • Cantilever Projection of beam beyond column face = (2000 – 350)/2 = 825 mm =0.825 m • Maximum transverse moment at column face A : $Mu = 525 \times 0.825^2/2 =$ 178.66 kNm --------- • Assume width of transverse beam, b = width of column + 2 x 0.75*d* b = 350 + 2 x 0.75 × 577 = 1215.5 mm $$M_u = 0.87\ f_y\ A_{st}\ d\left(1 - \frac{A_{st}\ f_y}{bd\ f_{ck}}\right)$$ b = 1215.5 mm, d =577 mm Mu = 178.6 x 10^6 N mm Ast = 880.23 mm^2	• Factored Column load B per width of footing = 1800/2.0 = 900 kN/m • Cantilever Projection beyond column face = (2000 – 400)/2 = 800 mm = 0.800m • Moment at column face B = $900 \times 0.80^2/2 = 288$ kNm • Width of transverse beam, b = width of column + 2 x 0.75d • 400 + 2 ×0.75 ×577 = 1265.5mm $$M_u = 0.87\ f_y\ A_{st}\ d\left(1 - \frac{A_{st}\ f_y}{bd\ f_{ck}}\right)$$ Mu = 384 x 10^6 N mm b = 1265.5mm d = 577 mm Ast = 1956.2 mm^2
Page 48, CL No 5.2.1 • Minimum Ast = 0.0012 *bD* =	• Provide $(Ast)min = 0.0012 \times 1265.5 \times 765 = 1161.73\ mm^2$

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$Ast\ min = .0012 \times 1215.5 \times 765 = 1115.83\ mm^2$

- Use 12mm dia bars (Your wish!!)
- Number of 12 mm φ bars required = Ast/ area of one bar = 1115.83/ (π/4 x 12^2) =10

Provide 10 nos 12 mmφ bars

Check for development length = 47 × 12 = 564 mm

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Use 12 mm dia bars

Number of 12 mm φ bars required

=1956.2 /(π/4 x 12^2) = 17.29 = 18

Provide 18 nos 12 mmφ bars

- Required development length = 47.0 × 12 = 564 mm is available beyond the column face.

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Transfer of force at column base -*Column* A

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Transfer of force at column base *Column* B

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- Limiting bearing stress at

IS 456 Page 65 , CL34.4

34.4 Transfer of Load at the Base of Column

The compressive stress in concrete at the base of a column or pedestal shall be considered as being transferred by bearing to the top of the supporting pedestal or footing. The bearing pressure on the loaded area shall not exceed the permissible bearing stress in direct compression multiplied by a value equal to $\sqrt{\frac{A_1}{A_2}}$ but not greater than 2;

where

A_1 = supporting area for bearing of footing, which in sloped or stepped footing may be taken as the area of the lower base of the largest frustum of a pyramid or cone contained wholly within the footing and having for its upper base, the area actually loaded and having side slope of one vertical to two horizontal; and

A_2 = loaded area at the column base.

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 $0.45\ f_{ck}$.

Permissible bearing stress = $0.45fck\sqrt{\frac{A1}{A2}}$

$A1 = 2000^2$ (2000mm is footing width)

A2 = 350 x 350 =

$= \sqrt{\frac{A1}{A2}} < 2$

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- Limiting bearing stress at

Permissible bearing stress = $0.45fck\sqrt{\frac{A1}{A2}}$

[$A1 = 2000^2$, $A2 = 400^2\ mm^2$]

$\sqrt{\frac{A1}{A2}} = 5 > 2, \ \sqrt{\frac{A1}{A2}} = 2$

= 0.45 × 20 × 2 = 18 MPa

Permissible bearing resistance = 18 × 400^2 = 2880 kN

2880kN > 1800kN , Hence safe

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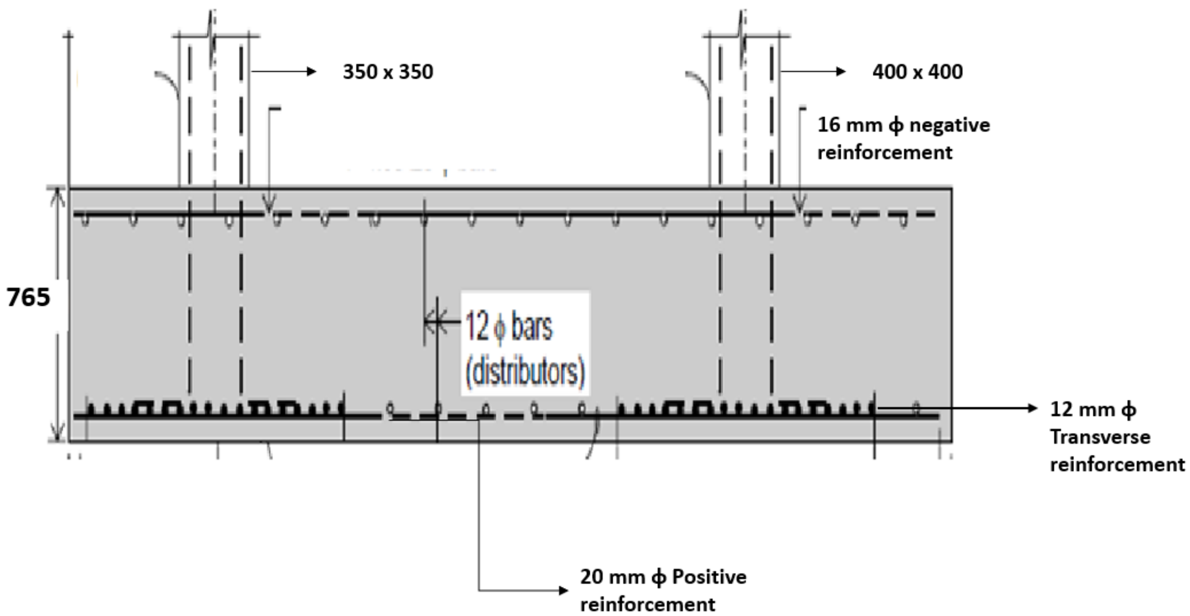
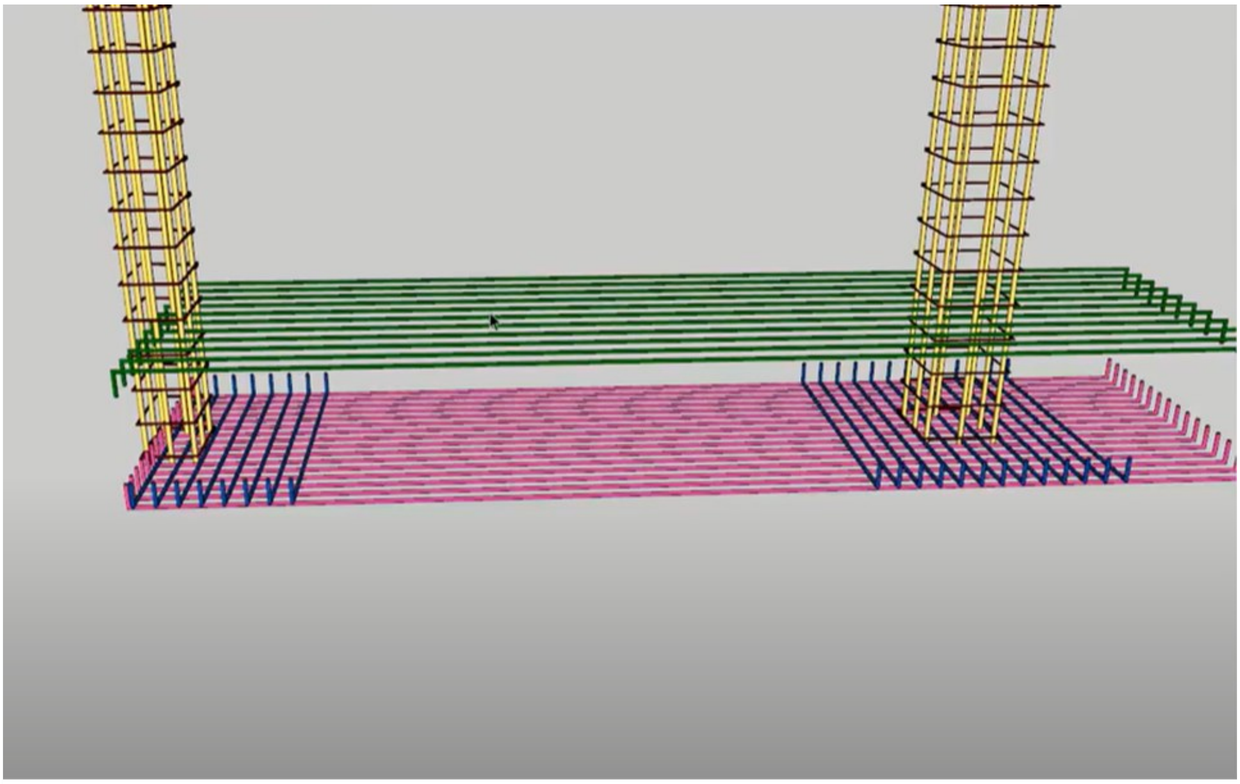
$= 5.71 > 2$, Take $\sqrt{\frac{A1}{A2}} = 2$ Permissible bearing stress = 0.45 X 20 X 2 =18 N/mm^2 Permissible bearing resistance or force F_{br} = Permissible bearing stress × column area $= 18 \times 350^2 = 2205 \times 10^3$ N = 2205 kN *2205> 1050 kN, Hence safe.*	

Reinforcement detailing





OR

Member	Length(m)	Dead Load (kN)	Live Load(kN)	Wind Load(kN)
Rafter(AB)	2.92	-58.0	-52.5	+111.6
Tie(AH)	2.5	+52.0	+47.0	-102.4
Sling(BG)	2.92	+20.3	+18.4	-63.0

Note + Tensile Force - Compressive Force

Important points to be noted

- Top and Bottom chord – Two angles or Double angles
- Inner members - Single angles
- Use at least two bolts
- Gusset plate thickness should be uniform

Various combinations of the loads on roof trusses are considered, and the critical condition is considered for the design. It may be noted that earthquake loads are not significant for roof trusses because of the small self weights. The following load combinations may be worked out:

1. Dead load + snow load
2. Dead load + partial or full live load (whichever causes the maximum stress in the member)
3. Dead load + wind load + internal positive air pressure
4. Dead load + wind load + internal suction air pressure
5. Dead load + live load + wind load

- **Load combinations**

Member	D L (kN)	L L(kN)	W L(kN)	DL+LL+WL (kN) (1)	DL+LL (kN) (2)	DL+WL (kN) (3)	Max Load (1) (2)and (3)
Rafter(AB)	-58.0	-52.5	+111.6	1.1	-110.5	53.6	-111.6
Tie(AH)	+52.0	+47.0	-102.4	-3.4	99	-50.4	99
Sling(BG)	+20.3	+18.4	-63.0	-24.3	38.7	-42.7	-42.7

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

AB, BC are top chord member, maximum load is -111.6 kN

Factored load or $Force$ = 111.6 x 1.5 = 167.4 kN

Length of members AB/ BC/ CD

cos 30.96 = 2.5/ AB , AB = 2.5/ cos 30.96 = 2.9 m = 2900 mm

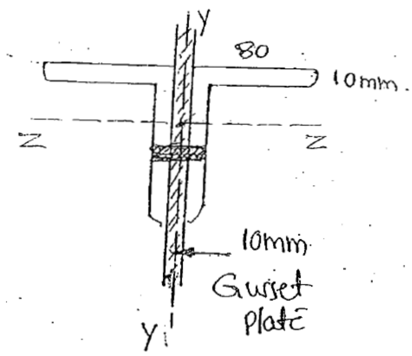
Assume design compressive stress, f_{cd} = 110 N/mm^2

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

Gross area, $A_g = \frac{Force}{f_{cd}} = \frac{111.6 \times 10^3}{110} = 1014.54$ mm^2 = 10.14 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) , $L_{eff} = K \times L =$ 0.85 x L = 0.85 x 2.9 = 2465 m

Slenderness ratio, λ $= \frac{K\,L}{r_{min}} = \frac{L_{eff}}{r_{min}} = \frac{2465}{24.1} = 102.2$

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)

(*Clause* 7.1.2.1)

KL/r ↓	Yield Stress, f_y (MPa)																	
	200	210	220	230	240	250	260	280	300	320	340	360	380	400	420	450	480	510	540
10	182	191	200	209	218	227	236	255	273	291	309	327	345	364	382	409	436	464	491
20	182	190	199	207	216	224	233	250	266	283	299	316	332	348	364	388	412	435	458
30	172	180	188	196	204	211	219	234	249	264	278	293	307	321	335	355	376	395	415
40	163	170	177	184	191	198	205	218	231	244	256	268	280	292	304	320	337	352	367
50	153	159	165	172	178	183	189	201	212	222	232	242	252	261	270	282	295	306	317
60	142	148	153	158	163	168	173	182	191	199	207	215	222	228	235	244	252	260	267
70	131	136	140	144	148	152	156	163	170	176	182	187	192	197	202	208	213	218	223
80	120	123	127	130	133	136	139	145	149	154	158	162	165	169	172	176	180	183	186
90	108	111	114	116	119	121	123	127	131	134	137	140	142	144	146	149	152	154	156
100	97.5	100	102	104	105	107	109	112	114	116	119	120	122	124	125	127	129	131	132
110	87.3	89.0	90.5	92.0	93.3	94.6	95.7	97.9	100	102	103	104	106	107	108	110	111	112	113
120	78.2	79.4	80.6	81.7	82.7	83.7	84.6	86.2	87.6	88.9	90.1	91.1	92.1	93.0	93.8	94.9	95.9	96.8	97.6

42

From Table 9 (c) Page 42 – IS 800 2007, through interpolation, for λ = 102.2 we get design Compressive Stress, $f_{cd} = 92.2\ N/mm^2$

(Page 34) Design compressive strength, $P_c = f_{cd} \times A_g = 92.2 \times 3010$

= 277.5 kN > 167.4 kN

So selected section is safe.

- **Connections**

  Using M 22 Property Class 5.6 bolts (Try to use same diameter of bolts if possible)

- **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)

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

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

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

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

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

Pitch, p = 2.5× d = 2.5 × 22 = 55 mm

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

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

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

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

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

No of bolts = $\frac{277.5}{92.54} = 2.99 \approx 3$

Hence provide 2 ISA 80 × 80 × 10 mm with 3 bolts

- **Design of Bottom chord members (AH)-Tension members**

Taking Max Force = 99 kN

Factored Tensile Force T_{dg} = 99 x 1.5 = 148.5 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, $A_g = \frac{148.5 \times 10^3 \times 1.1}{250} = 653.4\ mm^2$

Since it is bottom member, increase the area by 30% = 1.3 × 653.4 = 849.4 mm^2= 8.49 cm^2

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

ISA 8080	80 x 80	$\boxed{6.0}$	14.6	143.2	$\boxed{18.58}$	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)

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

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

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

- **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 = $\frac{148.5}{55.53} = 3$

Hence provide 2 ISA 80 × 80 × 6 mm with 3 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 $A_{nb} f_o$,

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

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

To find partial safety factor

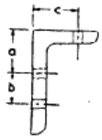
Table 5 Partial Safety Factor for Materials, γ_m

(*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

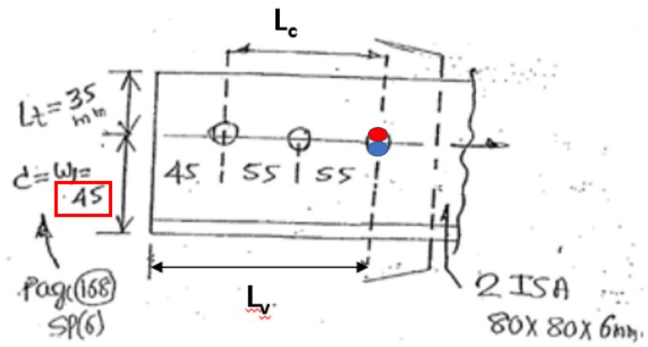
30

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
'30	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)**

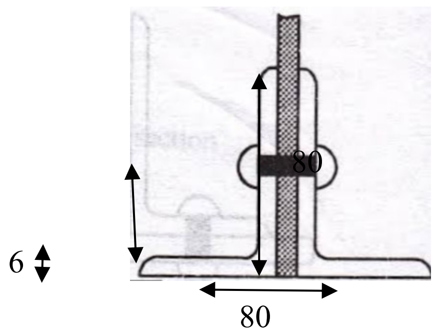
Page 33 CL 6.3.3 IS 800 - 2007

The rupture strength of an angle connected through one leg is affected by shear lag. The design strength, $T_{dn,}$ as governed by rupture at net section is given by:

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

where

$$\beta = 1.4 - 0.076\,(w/t)\,(f_y/f_u)\,(b_s/L_c) \leq (f_u \gamma_{m0} / f_y \gamma_{m1}) \geq 0.7$$



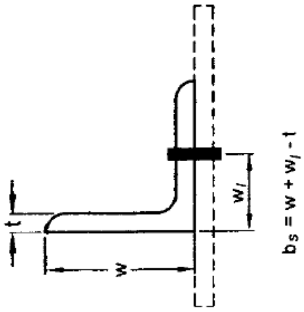
w = outstand leg width or **width of unconnected leg** = 80 mm

L_c = **Distance between the outermost bolts in the end joint measured along the load direction** = 55 +55 = 110 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 (CL 6.3.3)

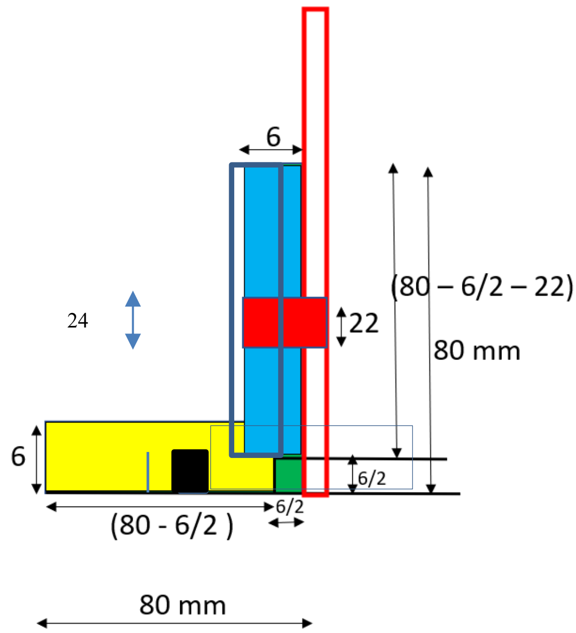
$$\beta = 1.4 - 0.076\,(w/t)\,(f_y/f_u)\,(b_s/L_c)$$

$= 1.132$

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

As per IS 800 – 2007, $\beta = 1.132 \geq 0.7 \leq 1.44$

Hence take $\beta = 1.132$



Angle section attached to gusset plate

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

A_{nc} = Net area of connected leg (<u>subtract area of bolt hole</u>) $= \left(A - d_o - \frac{t}{2}\right) t = (\left(80 - 24 - \frac{6}{2}\right) 6 = 318\ mm^2$ where $diameter\ of\ bolt\ hole, d_o = 22 + 2 = 24\ mm$

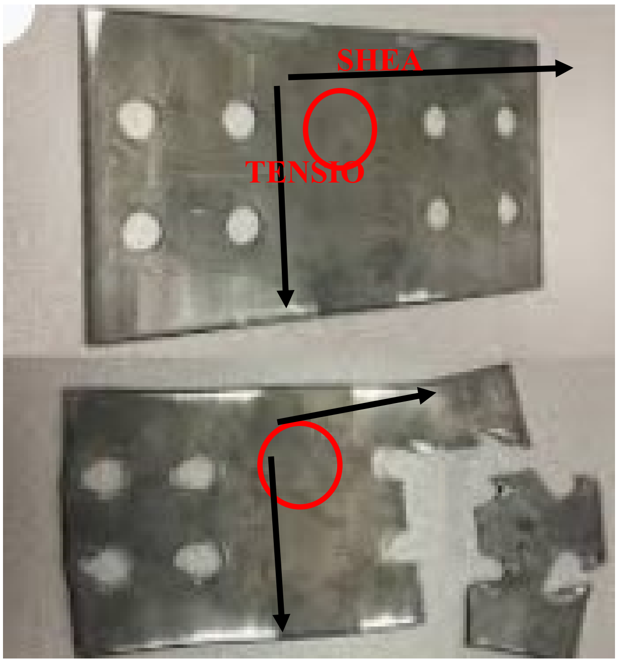
$$T_{dn} = 0.9\, A_{nc}\, f_u / \gamma_{m1} + \beta\, A_{go}\, f_y / \gamma_{m0}$$

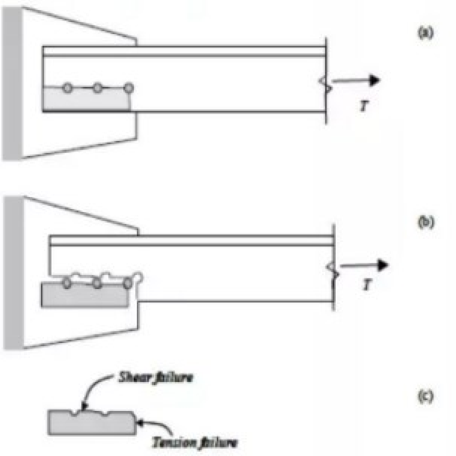
For double angle section multiply the value of T_{dn} by “2”

For double angle $T_{dn} = 2 \times \left(0.9 \times 318 \times \frac{410}{1.25} + 1.132 \times 462 \times \frac{250}{1.1}\right)$

$= 425.47$ kN > 148.5 kN , It is safe.

- **Check for block shear**





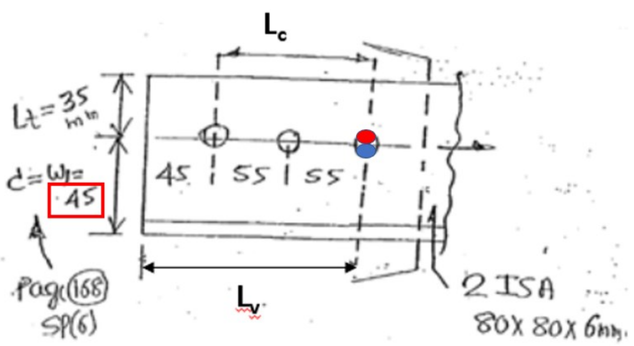
Page 33 CL 6.4.1

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

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

or

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



Length of shearing action, L_v = 45 + 2 × 55 = 155 mm

Length of tensile action, L_t = 35 mm

Gross area in shear parallel to force , $A_{vg} = L_v \times \mathrm{t} = 155\times 6 = 930\ mm^2$

Net area in **shear** parallel to force, $A_{vn} = A_{vg} - \mathbf{2.5} \times d_o \times \mathrm{t}$

$= 930 - 2.5 \times 24 \times 6 = 570\ mm^2$

Gross area in **tension** perpendicular to force, $A_{tg} = L_t \times \mathrm{t} = 35 \times 6 = 210\ mm^2$

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, $T_{db} = 2 \times [\,930 \times \frac{250}{\sqrt{3}\times 1.1} + 0.9 \times 138 \times \frac{410}{1.25}\,]$ = 337.4 kN > 148.5 kN

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

For two angles, $T_{db} = 2 \times [\,0.9 \times 570 \times \frac{410}{1.25\times\sqrt{3}} + 210 \times \frac{250}{1.1}\,]$ = 289.7 kN> 148.5 kN

Hence 2 ISA 80 × 80 × 6 mm is safe .

- **Design of Inner compression members (BG)**

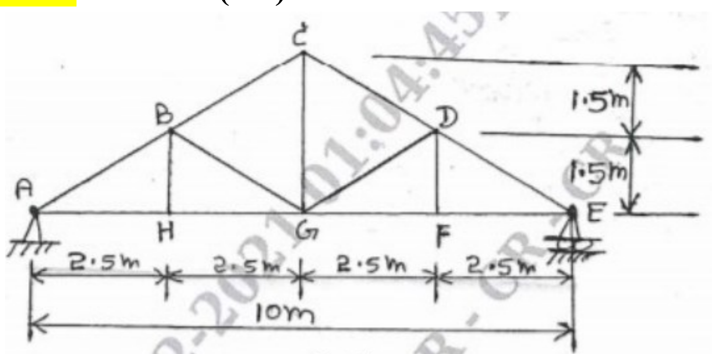


Fig.Q4

Taking Maximum Force = -42.7kN

Factored Force = 1.5 × −42.7 = −64.05kN

The length of BG is $= \sqrt{(1.5^2 + 2.5^2)}$ = 2.91 m

Maximum Length = 2.91 m

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

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

Gross Area, $A_g = \dfrac{Factored\ Force}{f_{cd}} = \dfrac{64.05 \times 10^3}{50} = 1281\ mm^2 = 12.81\ cm^2$

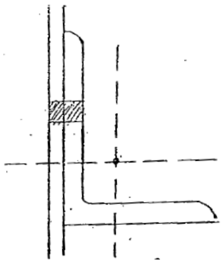
From Steel Table, Try Single ISA 100 x 100 x 10 mm (Table 1, Page 4 Steel table)

Area = 19.03 cm^2 (You can choose ISA 80 x 80 x 10 mm also!!)

$r_{xx} = r_{yy}$ = 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)

7.5.1.2 *Loaded through one leg*

The flexural torsional buckling strength of single angle loaded in compression through one of its legs may be evaluated using the equivalent slenderness ratio, λ_e, as given below:

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

where

k_1, k_2, k_3 = constants depending upon the end condition, as given in Table 12,

$$\lambda_{vv} = \frac{\left(\dfrac{l}{r_{vv}}\right)}{\varepsilon\sqrt{\dfrac{\pi^2 E}{250}}} \quad \text{and} \quad \lambda\varphi = \frac{(b_1 + b_2)/2t}{\varepsilon\sqrt{\dfrac{\pi^2 E}{250}}}$$

where

l = centre-to-centre length of the supporting member,

r_{vv} = radius of gyration about the minor axis,

b_1, b_2 = width of the two legs of the angle,

t = thickness of the leg, and

ε = yield stress ratio ($250/f_y)^{0.5}$.

Table 12 Constants k_1, k_2 and k_3

Sl No.	No. of Bolts at Each End Connection	Gusset/Connecting Member Fixity [1]	k_1	k_2	k_3
(1)	(2)	(3)	(4)	(5)	(6)
i)	≥ 2	Fixed	0.20	0.35	20
		Hinged	0.70	0.60	5
ii)	1	Fixed	0.75	0.35	20
		Hinged	1.25	0.50	60

[1] Stiffeness of in-plane rotational restraint provided by the gusset/connecting member.
For partial restraint, the λ_e can be interpolated between the λ_e results for fixed and hinged cases.

Assuming bolts ≥ 2 and hinged end conditions with gusset plate, $k_1 = 0.7$, $k_2 = 0.6$, $k_3 = 5$
Assuming Effective length, $l = 0.85 \times L = 0.85 \times 2910 = 2473.5\ mm$
$\varepsilon = 1$, $E = 2 \times 10^5\ N/mm^2$ $b_1 = b_2 = 100$ mm, t = 10 mm

$$\lambda_{vv} = \frac{\frac{L}{r_{vv}}}{\varepsilon\sqrt{\frac{\pi^2 E}{250}}} = \frac{\frac{2473.5}{30.5}}{1 \times \sqrt{\frac{\pi^2 \times 2 \times 10^5}{250}}} = \frac{81.09}{0.198} = 0.913$$

$$\lambda\varphi = \frac{(b_1 + b_2)/2t}{\varepsilon\sqrt{\frac{\pi^2 \varepsilon}{250}}}$$

$\lambda_\Phi = 0.1125$, $k_1 = 0.7$, $k_2 = 0.6$, $k_3 = 5$

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

Equivalent slenderness ratio, $\lambda_e = 1.124$
From Table 10, Page No 44, Choose the buckling class based on type of section

Table 10 Buckling Class of Cross-Sections

(*Clause* 7.1.2.2)

Cross-Section (1)	Limits (2)	Buckling About Axis (3)	Buckling Class (4)
Rolled I-Sections	$h/b_f > 1.2$: $t_f \leq 40$ mm	z-z y-y	a b
	$40 \leq$ mm $< t_f \leq 100$ mm	z-z y-y	b c
	$h/b_f \leq 1.2$: $t_f \leq 100$ mm	z-z y-y	b c
	$t_f > 100$ mm	z-z y-y	d d
Welded I-Section	$t_f \leq 40$ mm	z-z y-y	b c
	$t_f > 40$ mm	z-z y-y	c d
Hollow Section	Hot rolled	Any	a
	Cold formed	Any	b
Welded Box Section	Generally (except as below)	Any	b
	Thick welds and $b/t_f < 30$	z-z	c
	$h/t_w < 30$	y-y	c
Channel, Angle, T and Solid Sections		Any	

Since it is single angle section, choose buckling class as ‘c’. Based on Buckling class, find α from Table 7 Page No 35 as 0.49.

Table 7 Imperfection Factor, α

(*Clauses* 7.1.1 *and* 7.1.2.1)

Buckling Class	a	b	c	d
α	0.21	0.34	0.49	0.76

From Page 34, CL 7.1.2.1, IS 800-2007

$$\phi = 0.5\,[1 + \alpha\,(\lambda - 0.2) + \lambda^2]$$

Put $\alpha = 0.49$, $\lambda_e = 1.124$, $\Phi = 1.36$

To find f_{cd}

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

(Single angle section) Design compressive stress, $f_{cd} = \dfrac{250}{1.1(\,1.36+(\,1.36^2 - 1.124^2\,)^{0.5})} = 106.9$ N/mm^2

The design compressive strength of a member is given by:

Load $P_d = A \times f_{cd} = 1903 \times 106.9 = 203.42$ kN > 68.05 kN

Hence it is safe.

Design of connection using M 22, 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)

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

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

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

- **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 = $1.7 \times d_o = 1.7 \times 24 = 40.8 = 45$ mm

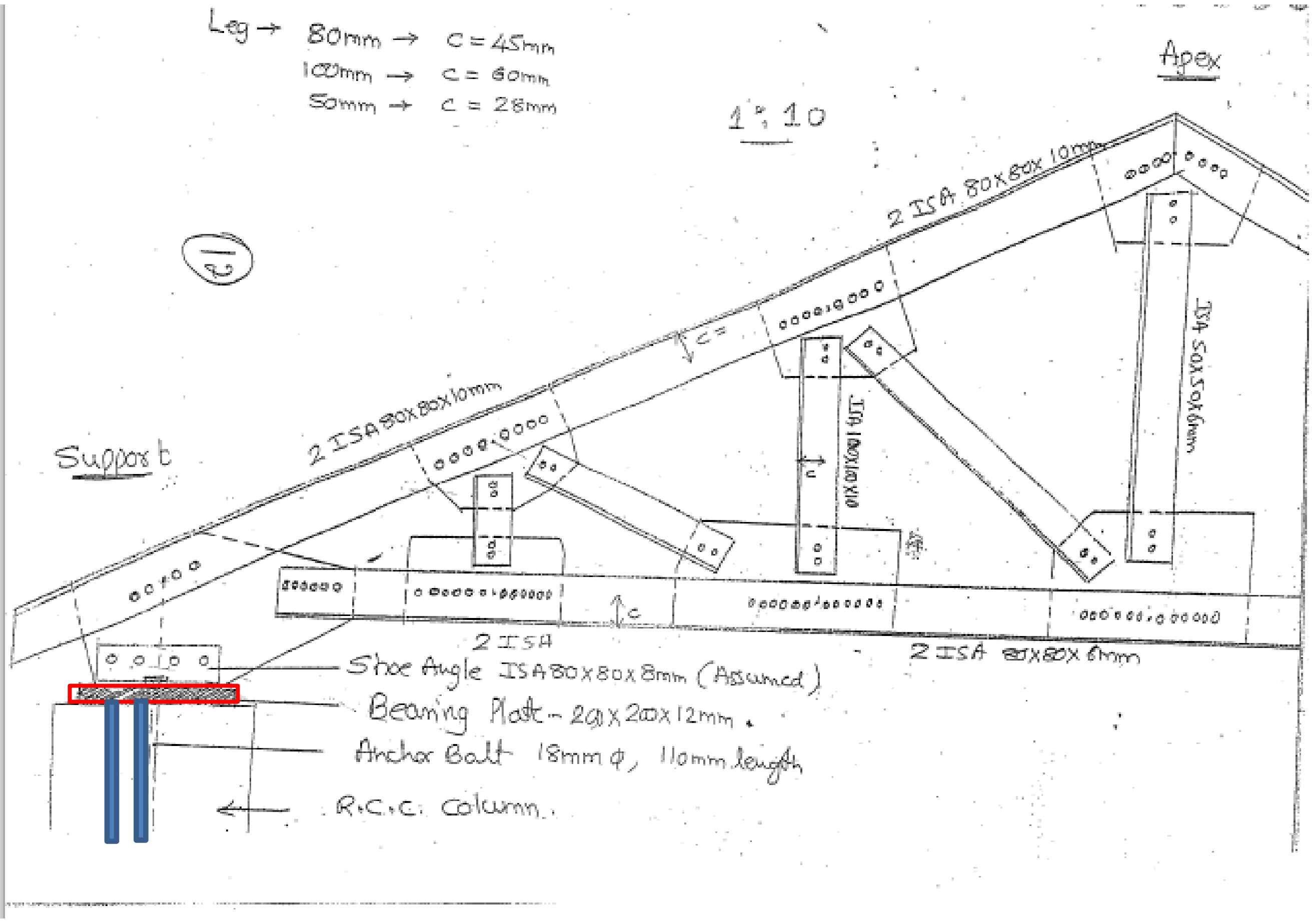
where k_b = smaller of $\frac{e}{3d_0}$, $\frac{p}{3d_0} - 0.25$, $\frac{f_{ub}}{f_u}$, and 1.0

$k_b = \frac{45}{3 \times 24} = 0.63$, $k_b = \frac{55}{3 \times 24} - 0.25 = 0.513$, $\frac{f_{ub}}{f_u} = \frac{500}{410} = 1.22$, 1.0 (try to copy value of k_b)

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

Bolt value = Minimum of (1) and (2) = 68.47kN

No of bolts = $\frac{68.05}{68.47} = 2$ (**Minimum no of bolts = 2**)


Leg → 80mm → c = 45mm
100mm → c = 60mm
50mm → c = 28mm
1 : 10
Apex
2 ISA 80x80x10mm
2 ISA 80x80x10mm
Support
ISA 100x100x10
ISA 50x50x6mm
2 ISA
2 ISA 80x80x6mm
Shoe Angle ISA 80x80x8mm (Assumed)
Bearing Plate - 200x200x12mm.
Anchor Bolt 18mm φ, 110mm length
R.C.C. column.