USN | | | | | | | | | |

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

Answer any one Questions- Use of IS 456 -2000/IS 800 is permitted — MARKS

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]

SOLUTION

- **Footing base dimensions**

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

$\Delta P = (900 + 1600) \times 15/100 = 375$ kN

$P_1 + P_2 = 900 + 1600 = 2500$ kN

$q_a = 240$ kN/m^2

$$A_{reqd} = \frac{P_1 + P_2 + \Delta P}{q_a} = 11.98 \text{ m}^2$$

In order to obtain a uniform soil pressure distribution, the line of action of the resultant column load must pass through the centroid of the footing. Let the footing centroid be located at a distance x' from the centre of C1. (Fig.1)

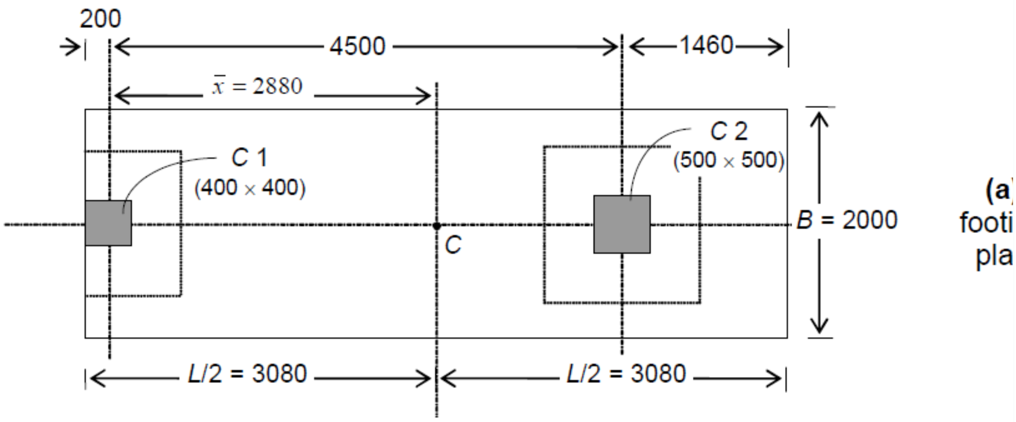


Fig. 1

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

$P_{u1} = 900 \times 1.5 = 1350$ kN; $P_{u2} = 1600 \times 1.5 = 2400$ kN $\Rightarrow P_{u1} + P_{u2} = 3750$ kN

spacing between columns s = 4500 mm

Let x' be measured from centre of C1 (x' is centroid of column loads)

$$\Rightarrow \bar{x} = \frac{P_{u2}\, s}{P_{u1+} + P_{u2}} = \frac{2400 \times 4500}{3750} = 2880 \text{ mm}$$

The total length L of the footing should be such that, centroid of footing,

L/2 = (2880 + 200), L = 2 (2880 + 200) = 6160 mm = 6.16 m

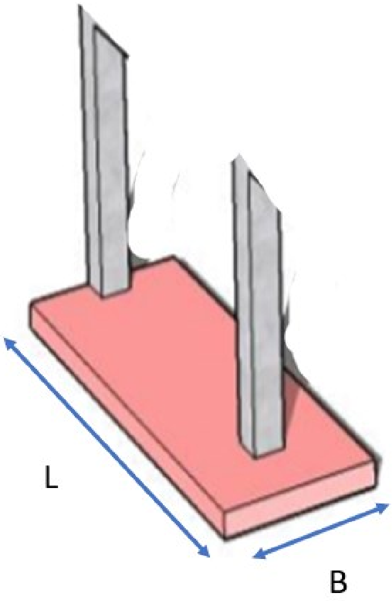
Provide L = 6.16 m

⇒ width of footing required B = A/L = 11.98/6.16 = 1.95 m rounded to 2.00 m

Provide B = 2.00 m

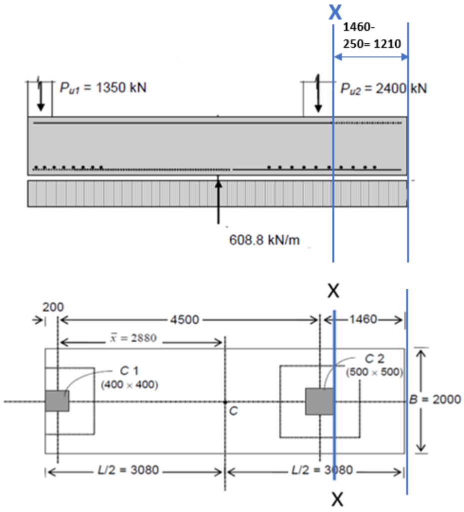
- **Uniformly distributed load acting in upward direction (soil pressure)**

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}



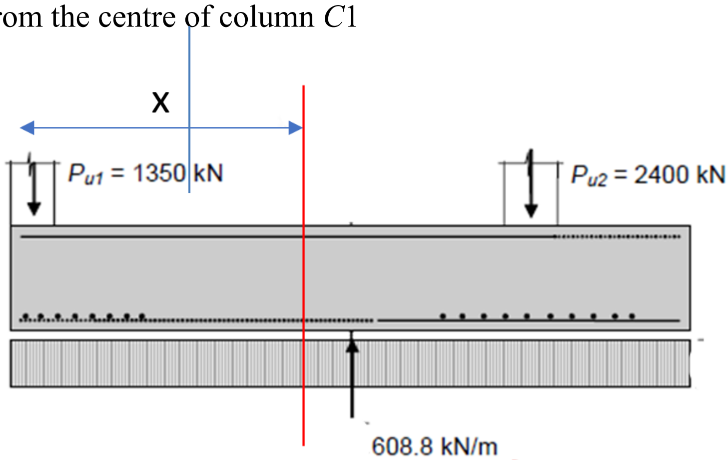
$q_{uB} = (P_{u1} + P_{u2})/L = 3750/6.16 = 608.8$ kN/m (Upward soil pressure intensity)

The maximum 'positive' bending moment (**heavier column**) at the face of column *C*2 at XX is given by



Moment at section XX , just right of XX, Mu **+ = 608.8 × (1.460 – 0.250)^2/2 = + 446 kNm**

The maximum 'negative' moment occurs at the **location of zero shear**, which is at a distance *X* from the centre of column *C*1

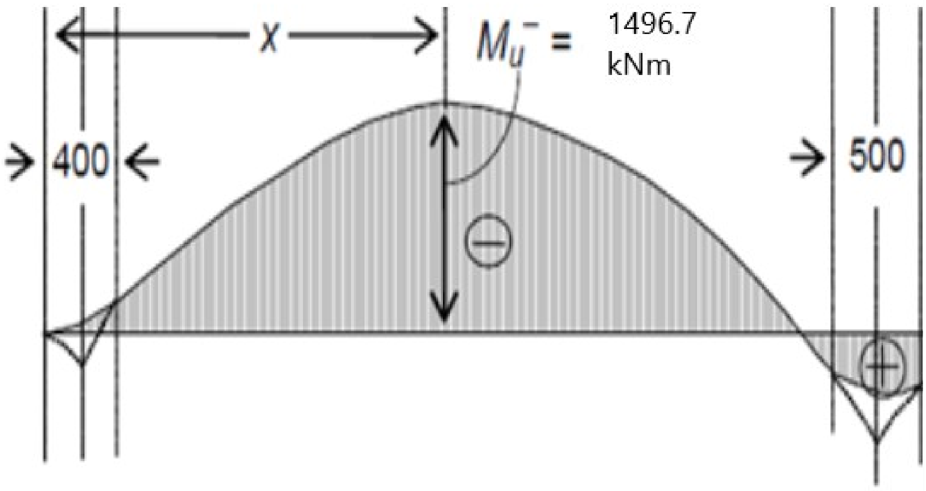


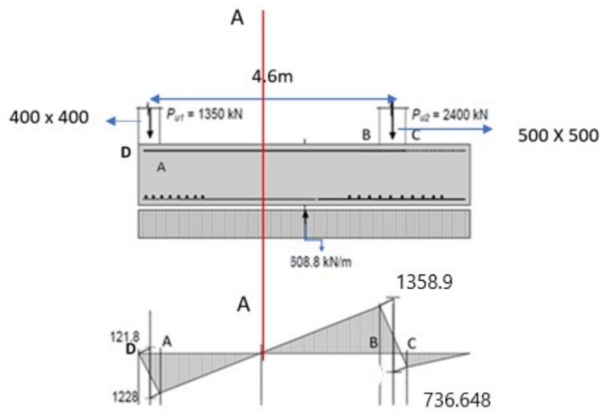
To find 'x' , Shear force at 'x' = 608.8 × (X) – 1350 = 0 (Location of zero shear)

X = 1350/608.8 = 2.2175 m

⇒ Negative bending moment M*u* - at X = 2.2175 m,

M*u*– = 608.8 × (2.2175)^2/2 – 1350 × (2.2175) = (–)1496.79 kNm





- **Shear force calculations (optional right now)**

  Shear force at D, (outer edge of column C1) just left of section AA= 608.8 x 0.2 = +121.8 kN

  Shear force at A , (inner edge of column C 1) just left of section AA = -1350 + 608.8 x 0.2 = 1228kN

  Shear force at C, (outer edge of column C 2) just right of section AA =

  – 608.8 x (1.46 – 0.25) = 736.648kN

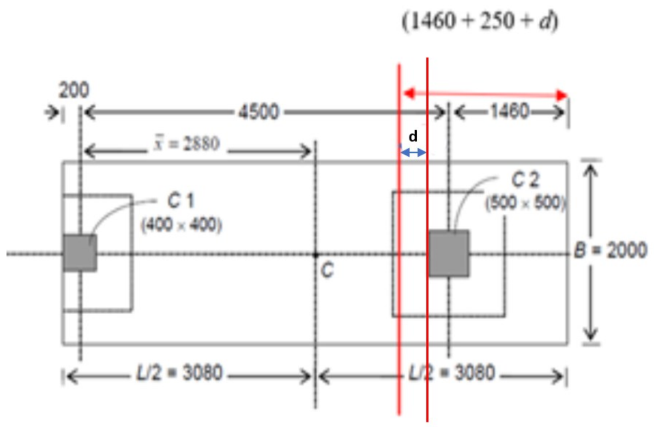
  Shear force at B, (inner edge of column C 2) just right of section AA = – 608.8 x (1.46 + 0.25) +2400

  =

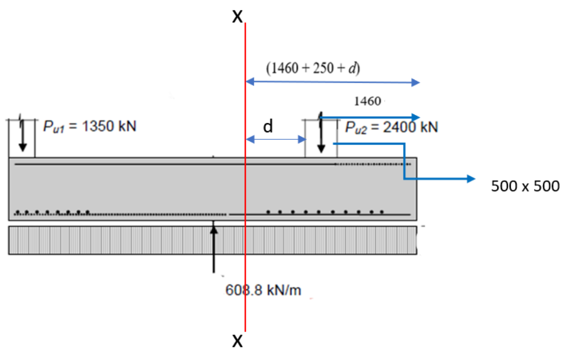
  1358.95kN

- **Thickness of footing based on shear**

One-way shear (longitudinal): $Vu1$ 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 greater load**) for one-way shear is located at a distance *d* from the (**inner)**face of *C*2, and has a value



Critical One way shear, $Vu1$ at section XX (just right of XX section) = Column load (C2) - Uniformly distributed upward load × (1460 + 250 + *d*)

= (2400 – 608.8 x (1.460 + 0. 250 + *d*)) = 2400 – 888.848 - 152.2 – 608.8 x *d*

= (1359 – 608.8 x *d*) kN …(1)

Assuming 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)

$V_{u1} = V_{uc} \Rightarrow (1359 - 608.8d) \times 10^3 = 960d$

⇒ Effective depth of footing, d = **866 mm**

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

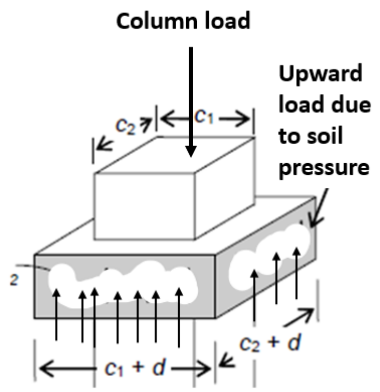
D = *d* + *75*+ *20/2* = 866 + 75 + 20/2 = 951 mm = 950 mm

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.***

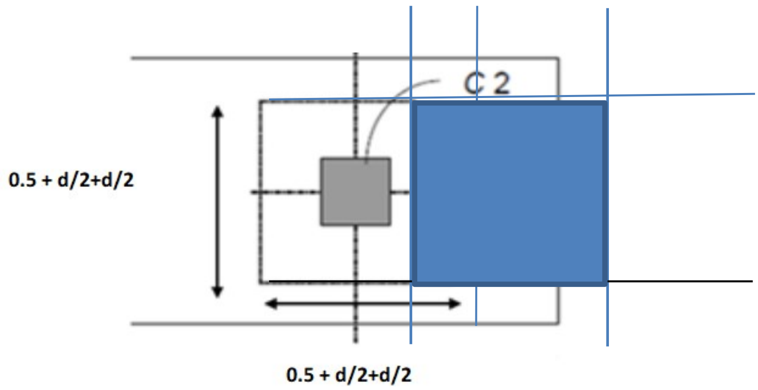
Factored soil pressure or Upward soil pressure, $q_u = (608.8) / (B \times 1) = (608.8/2) = 304.4$ kN/m^2

The critical section is located *d*/2 from the periphery of columns *C*1 and *C*2.

Two-way shear force for columns *C*1 and *C*2 (Punching shear)



Shear stresses in footing slab due to punching shear

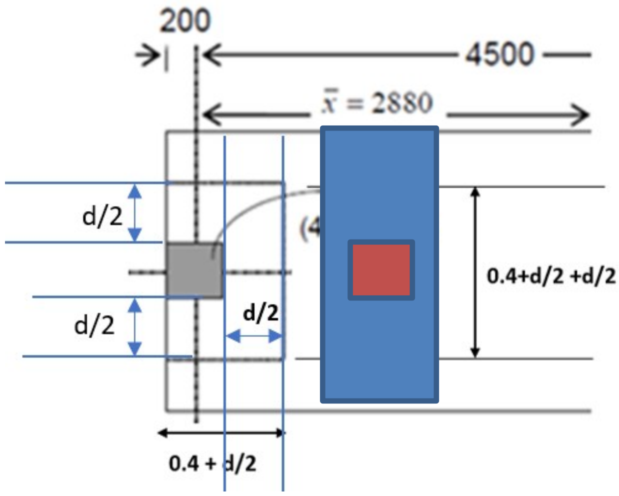


Punching shear or Two- way shear calculations for heavier column C2

V_{u2} = 2400 – 304.4 (0.5 + 0.866/2 + 0.866/2)(0.5+0.866/2 + 0.866/2)

= 1832 kN @ C2 (Heavier column)

Punching shear or Two way shear for C1



Punching shear or Two-way shear @ C1,

Two way shear V_{u2} = (Column load at C1) 1350 - 304.4 x (0.4 +0.866/2 + 0.866/2) x (0.4+0.866/2)

= 1029 kN @ C1

If no shear reinforcement is provided,

Page 58, IS 456, Clause 31.6.3.1, when no shear reinforcement is provided, 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

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

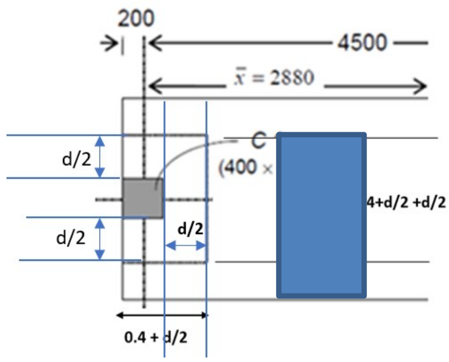
For square columns, *ks* = 500/ 500 = 400/400 = 1.0

Permissible shear stress, τ_{c2} = 1.0 × 0.25 ×√ 20 = **1.118** N/mm^2

Permissible two-way shear force for columns *C*1 and *C*2

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

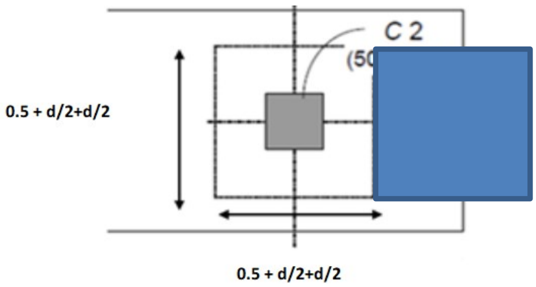
Perimeter of critical section = (400+866/2 +866/2) + 2 X (400+866/2) , depth = 866 mm



V_{uc} = 1.118 x [(400 +866/2 + 866/2) + (400 +866/2) × 2] × 866 = 2839kN @ C1

In the similar way lets calculate for C2

Permissible two way shear force , V_{uc} = 1.118 × (500 + 866/2+866/2) × 4 × 866 = 5290 kN @ C2



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

V_{uc} = 2839 kN > V_{u2} = 1029 kN @ C1

V_{uc} = 5290 kN > V_{u2} = 1832 kN @ C2

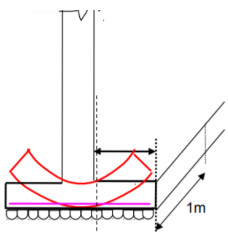
Hence safe against two way or punching shear, (if not provide shear reinforcement-stirrups or bent up bars)

- **Design of longitudinal flexural reinforcement**

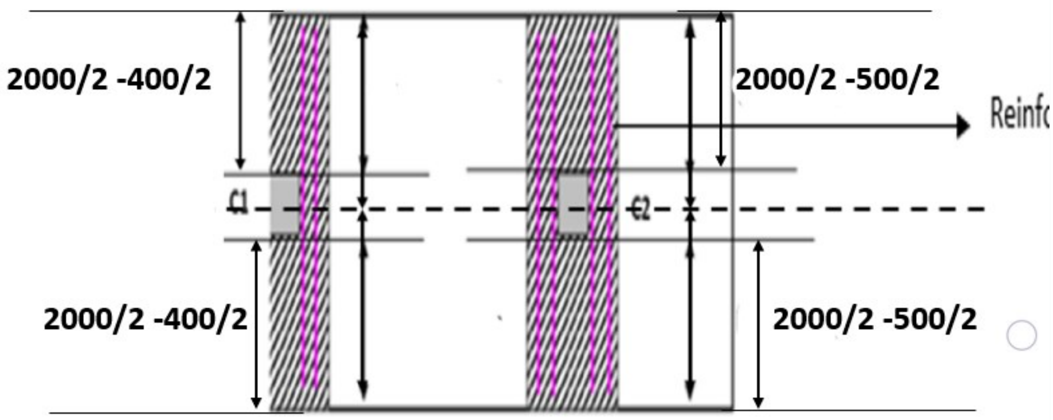
Maximum 'negative' moment: *Mu* = 1496.7 kNm at the location of zero shear	Maximum 'positive' moment: of column *C2*
$$M_u = 0.87 f_y A_{st} d\left(1 - \frac{A_{st} f_y}{bd f_{ck}}\right)$$ *Mu* = 1496.79 × 10^6 N mm B = b = 2000 mm, fck = 20 N/mm^2, fy = 415N/mm^2 d = 866 mm /865 mm Ast provided = 5098.24 mm^2 (check the value!!)	$$M_u = 0.87 f_y A_{st} d\left(1 - \frac{A_{st} f_y}{bd f_{ck}}\right)$$ *Mu* = 446 × 10^6 N mm B = b = 2000 mm d = 866 or 865 mm Ast = 1451.7 mm^2

Check for $(Ast)min = 0.0012\ BD$ = $0.0012 \times 2000 \times 950 = 2280\ mm^2$ Ast provided > $(Ast)min$ But we have assumed pt = 0.5 pt = 100 Ast, req/ (B × d) **No,of bars = Ast req/ Area of one bar** **Assume 20 mm dia bars** $\Rightarrow (A_{st})_{reqd} = 0.50 \times 2000 \times 865/100 = 8650\ mm^2$ $> (A_{st})_{min} = 0.0012BD$ Number of 20 mm ϕ bars required = 8650/314 = 28 [Corresponding spacing = (2000 – 75 × 2 – 20)/27 = 68 mm, which is low but acceptable.] ∴ **Provide 28 nos 20 mm ϕ bars at top** between the two columns as indicated in • Required development length (with M 20 concrete and Fe 415 bars) will be less than $L_d = 47.0 \times 20 = 940$ mm Adequate length is available on both sides of the peak moment section.	Check for Min Ast $(Ast)min = 0.0012\ BD = 0.001$ 0 mm^2 $1451.7 < 2280 mm^2$ Since moment is less, smaller c d • Number of 16 mm ϕ bars required = 2280/201 = 12 [Corresponding spacing = (2000 – 75 × 2 – 16)/11 = 167 mm — OK.] ∴ **Provide 12nos 16 mm ϕ bars at bottom** • Required development length = 47.0 × 16 = 752 mm, which is available on the side of the column C_2 close to the edge of the footing; by placing the bars symmetrically with respect to column C_2, the required length will be available on both sides of the section of maximum 'positive' moment.

Design of column strips as transverse beams



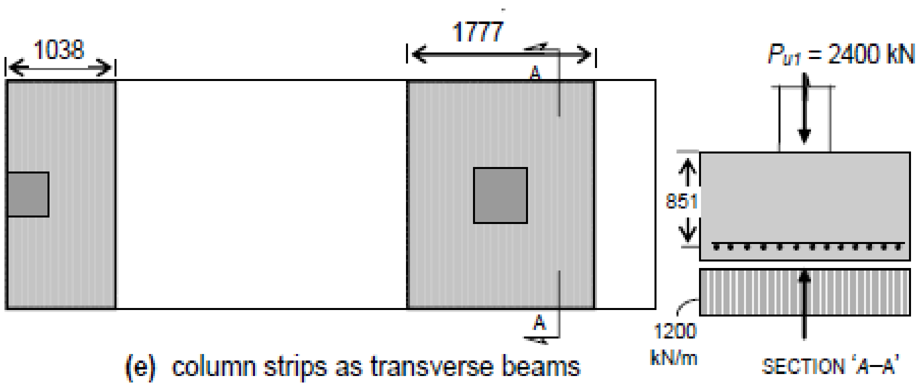
Tranverse bending of footing



Transverse beam under column C1	*Transverse beam under column* C2
• Factored Column load per width of footing = 1350/2.0 = 675 kN/m • Projection of beam beyond column face = (2000 – 400)/2 = 800 mm =0.8 m • Maximum moment at column face: *Mu* = 675 × $0.80^2/2$ = 216 kNm • Effective depth for transverse beam (16 mm φ bars placed above the 16 mm φ longitudinal bars):	• Factored Column load per width of footing kN/m • Projection beyond column face = (2000 – 5 =0.75m • Moment at column face = 1200 × $0.75^2/2$ = 338 kNm • Width of transverse beam = width of colun • 500 + 0.75× 851 + 0.75 × 851 = 1777 mm $M_u = 0.87\ f_y\ A_{st}\ d\left(1 - \frac{A_{st}\ f_y}{bd\ f_{ck}}\right)$ Mu = 338 x 10^6 N mm b = 1777 mm

d = 950 – 75 – 16 – 16/2 = 851 mm • Assume width of transverse beam, b = width of column + 0.75d b = 400 + 0.75 × 851 = 1038 mm $M_u = 0.87\ f_y\ A_{st}\ d\Big(1$ b = 1038 mm, d = 851 mm Mu = 216 x 10^6 N mm Ast =709 mm^2	d = 851 mm Ast = 1113.6mm^2
Page 48, CL No 5.2.1 • Minimum Ast = 0.0012 bD = Ast = .0012 × 1038 × 950 = 1183 mm^2 • Use 16 mm or 12mm dia bars (Your wish!!) • Number of 16 mm φ bars required = Ast/ area of one bar = 1183/201 = 6 Check for development length = 47 × 16 = • Alternatively, no. of 12 mm φ bars required = 1183/113 = 11	• Provide $(Ast)min$ = 0.0012 × 1777 × 950 = Use 16mm or 12 mm dia bars Number of 12 mm φ bars required = 2026/113 = 18 **Provide 18 nos 12 mmφ bars** • Required development length = 47.0 × 12 = beyond the column face.

Provide 11 nos 12 mm φ bars • Required development length = 47 × dia of the bar = 47.0 × 12 = 564 mm	



(e) column strips as transverse beams

Transfer of force at column base *Column* C1	Transfer of force at colun
• 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}. i) column face = $0.45fck = 0.45 \times 20 = 9.0$ MPa	• Limiting bearing st i) column face = $0.45fck$ = Permissible bearing stress = [$A1 = 2000^2$, $A2 = 500^2$ m = 0.45 × 20 × 2.0 = 18.0 M *Permissible bearing resista* 3375 kN *3375kN > 2400kN , Hence*

Permissible bearing stress = $0.45fck\sqrt{\frac{A1}{A2}}$ [As the **column is located at the edge of the footing**, Assume $A1 = A2 = 400^2$ mm^2] = 0.45 × 20 × 1.0 = 9.0 MPa =9.0 N/mm^2 Permissible bearing resistance or force *Fbr* = Permissible bearing stress × column area = 9.0 × 400^2 = 1440 × 10^3 N = 1440kN *1440> 1350 kN, hence* OK.	
Hence, full force transfer can be achieved without the need for reinforcement across the interface. However, it is desirable to provide some nominal dowels (4 nos 20 mm φ),	In this case also, full force achieved without the need across the interface. Howev provide some nominal dow

The reinforcement details are indicated in Figure 1. Some of the longitudinal bars at the bottom are shown (arbitrarily) extended across the full length of the footing in order to provide some nominal reinforcement in the large (otherwise unreinforced) area of concrete between the columns and also to tie up with the transverse bars under column C1. Nominal transverse reinforcement is also indicated at top between the columns, in order to tie up with the main longitudinal bars provided.

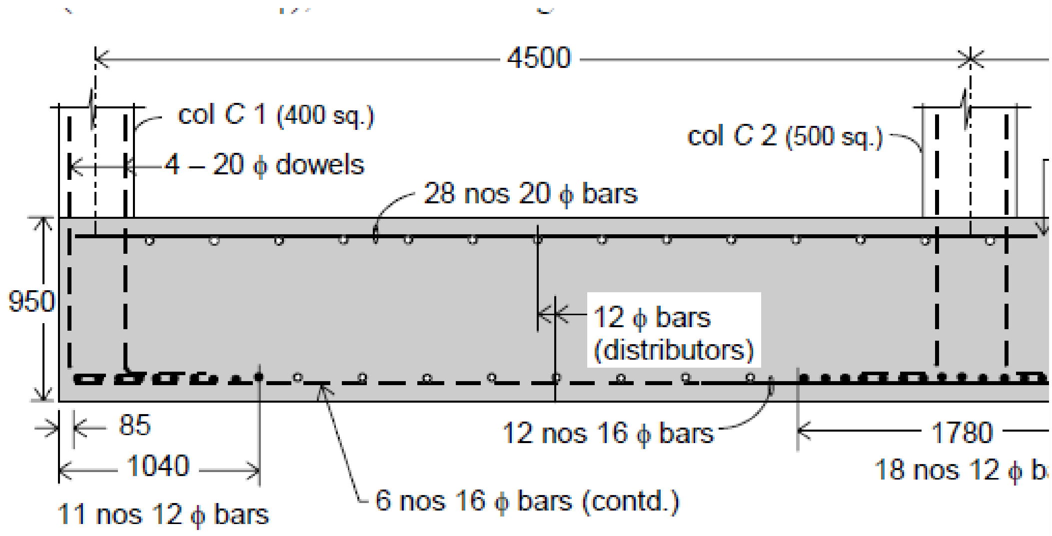
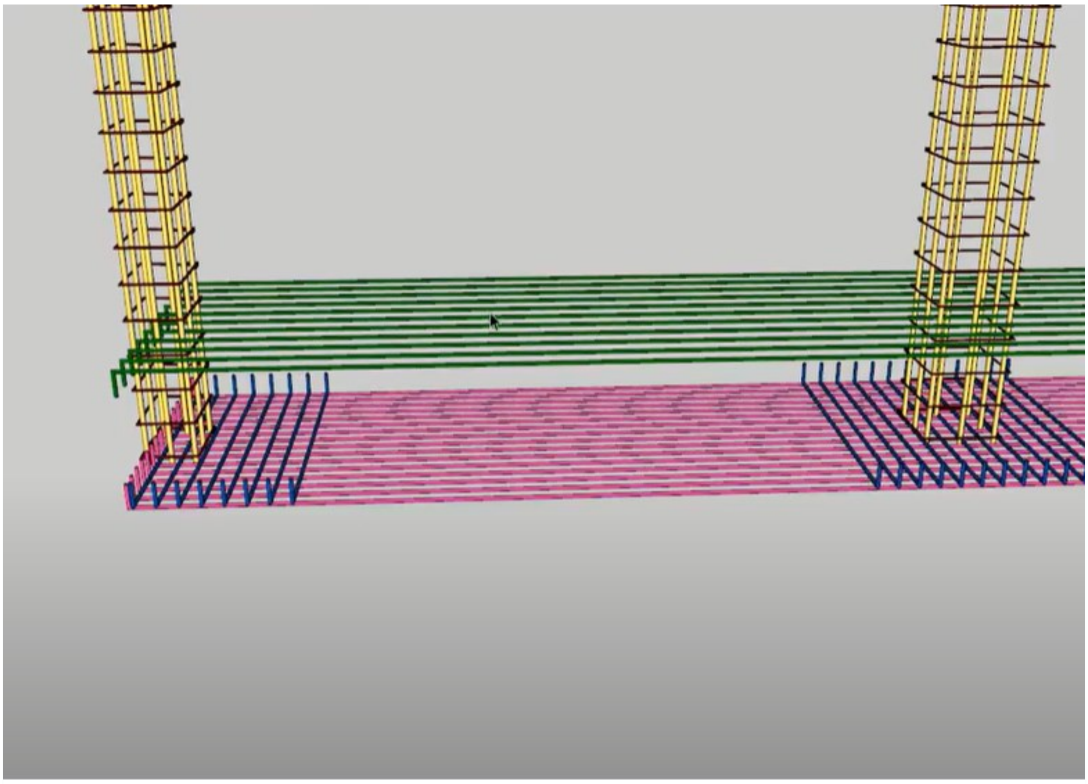


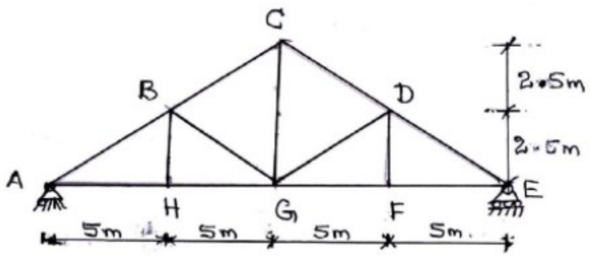
Figure 1



3D view of Reinforcement details

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]



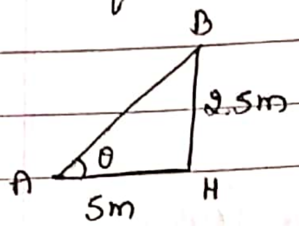
C
B
D
2.5m
2.5m
A
H
G
F
E
5m
5m
5m
5m.

Step 1: Design of Top chord member (AB)

Top chord member (Max Load) = − 120 KN (Compression)

Factored Load = 1.5 × 120 = 180 KN.

To find out Length



$\tan\theta = \frac{opp}{adj} = \frac{2.5}{5}$

$\theta = \tan^{-1}\left(\frac{2.5}{5}\right) = 26.56°$

$AB^2 = AH^2 + BH^2$

$= 5^2 + 2.5^2$

$AB = 5.59\ m. = 5590\ mm$

Assume design compressive stress as $110\ N/mm^2$ (Range $40 - 120\ N/mm^2$)

$\text{Compressive stress} = \frac{\text{Factored Load}}{\text{Gross area}}$

$\text{Gross area } A_g = \frac{\text{Factored Load}}{\text{Compressive stress}}$

$= \frac{180 \times 10^3}{110} =$

$= 1636\ mm^2$

$= 16.36\ cm^2$

Steel table No 6, page 18

Let us use double angle section

2ISA 80 × 80 × 10 mm

Area = 30.10 cm^2 r_{xx} = 2.41 cm r_{yy} = 3.73 cm

r_{min} = 2.41 cm = 24.1 mm

Effective Length calculation :

Effective length for top chord member (AB)

$L_{eff} = 0.8 \times L$

$= 0.8 \times 5590$

$= 4472$ mm

Slenderness ratio (λ) $= \frac{L_{eff}}{r_{min}} = \frac{4472}{24.1} = 185.56$

for λ = 185.56 find fcd from Table 9c pg 42 IS800-2007

180	43.6
190	39.7

f_{cd} = 41.43 N/mm² (through interpolation)

Design of Compressive Strength $= f_{cd} \times A_g$ 6

$= 41.43 \times 3010$

$= 124.70$ kN

124.70 < 180

∴ The Selected Section is not Safe. Revise the Section with more area

∴ Hence we need to [illegible] 2ISA 80×80×12 mm

Step 2: Connection desigine

Use M16, 4.6 Class bolt

[16 is the dia of bolt, grade 4.6 means 400 N/mm^2 is the strength of bolt then 0.6 is the ratio of ultimate strength of bolt divided by design strength of bolt]

1. Shear strength of bolt

from IS 800-2007 page 75

$$V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}}$$

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

$\gamma_{mb} = 1.25$

$f_{ub} = 400$ N/mm^2

$n_n = 2$

$A_{nb} = 0.78 \times \pi/4 \times d^2$

$A_{nb} = 156.82$ mm^2

no shank portion $A_{sb} = 0$

$$V_{nsb} = \frac{400}{\sqrt{3}}(2 \times 156.82)$$

$V_{dsb} = 58$ KN — (1)

Bearing strength of bolt

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

$$k = \frac{e}{3d_0},\ \frac{p}{3d_0} - 0.25,\ \frac{f_{ub}}{f_u},\ 1.0$$

e = 1.7 × dia of bolt hole

bolt hole = dia of bolt + 2mm
= 16 + 2
= 18 mm

e = 1.7 × 18 = 30.6 ≈ 40

P = 2.5 × dia of bolt
= 2.5 × 16
P = 40 ≈ 50

k_b = 0.67

V_{dpb} = 87.9 kN — (2)

Bolt value = smaller of 1 or 2
= 58 kN

No of bolts = $\frac{\text{design load}}{\text{Bolt value}} = \frac{180}{58}$ = 3 No

Hence provide 2 ISA 80mm with 3 bolts.

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IS page 32

Step 3: Design of bottom chord (AH)

Max Load = 100 kN

Factored Load = 1.5 × 100 = 150 kN.

length = 5 m = 5000m

From IS 800-2006 Page 32

Tensile Strength due to yielding of Gross section

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

$\gamma_{m0} = 1.1$

$f_y = 250 \text{ N/mm}^2$

$T_{dg} = 150 \times 10^3$ N

$$A_g = \frac{T_{dg} \times \gamma_{m0}}{f_y} = \frac{150 \times 10^3 \times 1.1}{250}$$

$A_g = 660 \text{ mm}^2$

Since it is a bottom section ↑ the area by 30%.

$A_g = 660 + 660 \times 0.3$

$A_g = 858 \text{ mm}^2$

From steel table, page 18 Table 6

For 2 ISA 80 × 80 × 6

$A_g = 18.58 \text{ cm}^2$

$= 1858 \text{ mm}^2$

Connections : Use M16, 4.6

Shear Strength of bolt

IS 800 - 2007, pg 75

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

$$V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}}$$

$A_{nb} = 0.78 \times \pi/4\, d^2$ $\quad d = 16mm$

$A_{nb} = 156.82 mm^2$

$n_n = 2$

$f_{ub} = 400 N/mm^2$

$V_{nsb} = 58 kN$ — (1)

Bearing Strength of bolt.

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

$k_b = 0.67$ $\quad f_u = 410 N/mm^2$

$d = 16$ $\quad \gamma_{mo} = 1.25$

$t = 6$

$V_{dpb} = 53 kN$ — (2)

Smaller value = 53 kN

No of bolt = $\frac{150}{53} = 2.83$

≈ 3 No

Step 4: Check for rupture

IS 800-2007, pg. 33

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

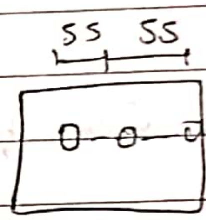
$$\beta = 1.4 - 0.076 (w/t)(f_y/f_u)(b_s/L_c) \le \left(\frac{f_u \gamma_{m0}}{f_y \gamma_{m1}}\right) \ge 0.7$$

$w = 80mm$

$t = 6mm$

$f_y = 250$, $f_u = 410 N/mm^2$

$L_{cc} = 55 + 55 = 110mm$



$b_s = w + w_t - t$

$= 119mm$

$w_t = g = 45$

$\beta = 0.73$

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

$0.73 \le 1.44 \ge 0.7$

Hence take $\beta = 0.73$.

$A_{go} = (b - t/2) \times t$

$= (80 - 6/2) \times 6$

$= 462 mm^2$

$A_{nc} = (b - t/2)\,t - d_o \times t$

$= (80 - 6/2)6 - 18 \times 6$

$= 354\,mm^2$

For double angle $T_{dn} = 2 \times \left[0.9 A_{nc} \frac{f_u}{\gamma_{m1}} + \beta A_{go} \frac{f_y}{\gamma_{m0}} \right]$

$T_{dn} = 362.301$ kN

$\therefore 362.301 > 150$ kN

Hence the rupture is safe

Step 5: Check for Block Shear Shear

page 33, IS 800, CL 6.4

$T_{db} = \left[\frac{A_{vg} f_y}{\sqrt{3}\,\gamma_{m0}} + \frac{0.9 A_{tn} f_u}{\gamma_{m1}} \right]$

or

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

$L_v = 45 + 55 + 55$

$= 155.$

$A_{vg} = L_v \times t$

$= 155 \times 6 = 930\,mm^2$

$A_{tg} = L_t \times t = 35 \times 6 = 210\,mm^2$

$A_{vn} = A_{vg} - 2.5 \times d_o \times t$

$= 930 - 2.5 \times 18 \times 6$

$= 660\,mm^2$

ed

due to

$A_{tn} = A_{tg} - 0.5 \times d_o \times t$
$= 210 - 0.5 \times 18 \times 6$
$= 156\ mm^2$

$$T_{db} = 2\left[\frac{230 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 156 \times 410}{1.25}\right]$$

$= 336.16$ kN

or

$T_{db} = 160.213$ kN.

160.213 > 150 kN

Hence safe against block shear

Step 6: Design of inner member BG.

Max load = −60 kN.
Factored load = $1.5 \times 60 = 90$ kN.

Since it is '−'ve: Design a compression member

Assume f_{cd} as $50\ N/mm^2$

$$Area = \frac{90 \times 10^3}{50} = 1800\ mm^2 = 18.00\ cm^2$$

From steel table, Table 1, page 4.

Since it is a inner member select single angle section

Choose ISA × 80×80×10

$A = 15.05\ cm^2 = 1505\ mm^2$

$r_{uu} = 3.04\ cm$ $r_{xx} = r_{yy} = 2.41\ cm$

$r_{vv} = 1.55\ cm$

Using IS 800-2007, Page 48

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

$$\lambda_{vv} = \frac{\left(\frac{L_{eff}}{r_{vv}}\right)}{\varepsilon\sqrt{\frac{\pi^2 E}{250}}} \qquad \lambda_\phi = \frac{(b_1+b_2)/2t}{\varepsilon\sqrt{\frac{\pi^2 E}{250}}}$$

use table 12, page No 48 find the value of k_1, k_2, k_3

1) Assume No of bolt ≥ 2
2) Hinged Condition with gusset plate.

$k_1 = 0.70$, $k_2 = 0.60$, $k_3 = 5$

~~λ_{vv} =~~ $\varepsilon = 1$ $E = 2 \times 10^5$ N/mm²

length of member AB = 5.59m llly for BG = 5.59m = 5590mm

Effective length = 0.85 × Length of member BG
= 0.85 × 5590
= 4751.5 mm

$b_1 = b_2 = 80$ mm $t = 10$mm

$\lambda_{vv} = 3.44$.

$\lambda_\phi = 0.09$

From table

$k_1 = 0.70$ $k_2 = 0.60$ $k_3 = 5$

$\lambda e = 2.80$

From pao IS 800-2007 Page 35, Table 7

$\alpha = 0.49$

Page 34

$\phi = 0.5 [1 + \alpha (\lambda - 0.2) + \lambda 2]$ $\lambda = 2.80$

$\phi = 5.05$

$f_{cd} = \dfrac{f_y / \gamma_{mo}}{\phi + [\phi^2 - \lambda^2]^{0.5}}$ $\gamma_{mo} = 1.1$

$= 24.56 \ N/mm^2$

Load $= 24.56 \times 1505$

$= 369.62$ kN

$369.62 > 90$ kN

Section is Safe

Design is OK

Connection

$V_{nsb} = \dfrac{f_{ub}}{\sqrt{3}} (n_n A_{nb} + n_s A_{sb})$

$A_{nb} = 156.83 mm^2$

$V_{nsb} = \frac{400}{\sqrt{3}} [1 \times 156.83] = 29 kN$

Bearing strength

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

$\gamma_{mo} = 1.25$

$= 89.22\ kN$

Bolt value = Smaller = 29 kN

No of bolts = $\frac{\text{Design Load}}{\text{Bolt value}} = \frac{90}{29} = 3$ No