Module 4 : Design of Shallow Foundations

Lecture 18 : Structural designs of column and footing [Section18.1: Footing subjected to Concentric loading]

Objectives

In this section you will learn the following

- Design of the Column.
- Design of footing
- Thickness of footing
- Flexural reinforcement
- Check for development length
- Transfer of load at base of column

Footing subjected to Concentric loading

Problem 1

Shallow footing subjected to vertical load along with moment. Design a column footing to carry a vertical load of 40 t (DL+LL) and moment of 1000 Kg-m.

$q_{allow} = 20t/m^2,\ f_y = 415N/mm^2,\ f_{ck} = 15N/mm^2$

i **Design of the Column.**

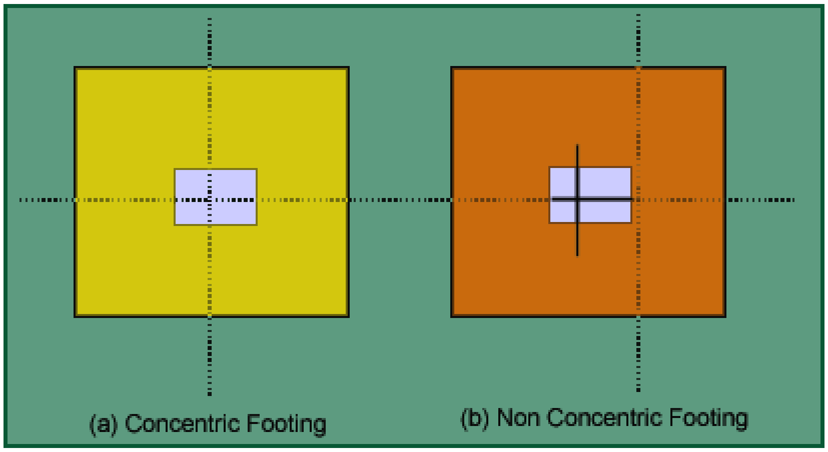
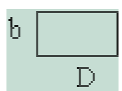


Fig. 4.26 Concentric & Non Concentric Footing

$$P_U = 40\times1.5 = 60t = 60\times10^4 N$$

$$M_U = 1.5\times1000 Kg-m = 1500\times10^4 N-mm$$

Trial 1 Let assume b = 300 mm & D (L) = 400 mm



$$\frac{P_U}{f_{CK}.b.D} = 0.33$$

$$\frac{M_U}{f_{CK}.b.D^2} = 0.021$$

$$\frac{d'}{D} = \frac{40+\frac{20}{2}}{400} = 0.125$$

See chart 33 of SP-16. Assume Diameter of bar 20 mm.

It shows for this trial No Reinforcement required, but practically we have to provide reinforcement.

Trial 2

b = 250 mm, D = 300 mm.

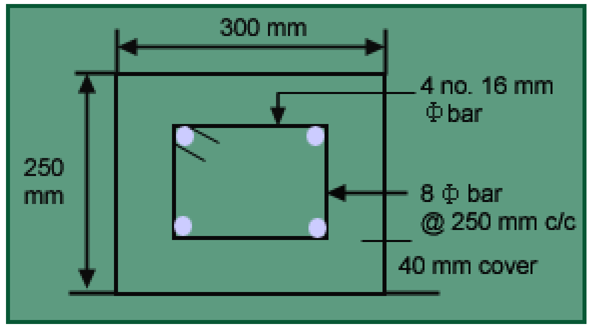


Fig -4.27 Column Section

$$\frac{P_U}{f_{CK}.b.D} = 0.53$$

$$\frac{M_U}{f_{CK}.b.D^2} = 0.044$$

$$\frac{d'}{D} = 0.167$$

$$\frac{p_t}{f_{CK}} = 0.06$$

therefore

$$p_t = 0.06 \times 15 = 0.9\%$$

$$A_{st_{required}} = \frac{0.9}{100} \times 250 \times 350 = 675 mm$$

ii Design of footing

Size of the footing

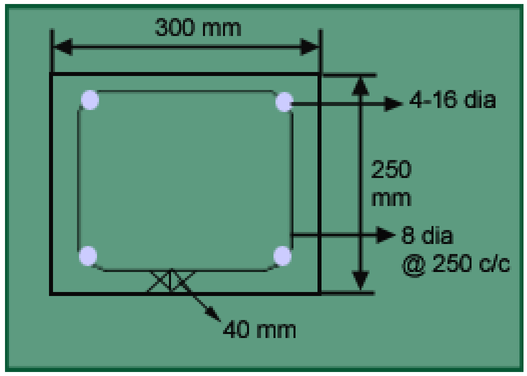


Fig 4.28 Details of the coulmn

Let D=500mm

For concentric footing; $\{q_a - (\gamma_c - \gamma_s)D\} = \frac{V}{BL} + \frac{6Ve}{BL^2}$

$Q_a = 20t/m^2, \gamma_c = 2.5, \gamma_s = 1.8$

V=40 t =40*104 N, e=M/V=1000*104/40*104 =25 mm

For no tension case: $\frac{V}{BL} = \frac{6Ve}{BL^2} = 0$ Determination of L & B for different values of L & B.

L in m	B in m
1.0	2.34
2.0	1.1
2.2	0.988

L=6e=150mm

$$19.65 = \frac{40}{BL} + \frac{6}{BL^2}$$

$$or \qquad B = \frac{6+40L}{19.65L^2}$$

Let provide footing size is 2.2 m*1.0 m.
Check:

$$Q_{min} = \frac{40}{1\times 2.2} - \frac{6\times 1}{1\times 2.2^2} = 16.94\ t/m^2$$

$$Q_{min} = \frac{40}{1\times 2.2} + \frac{6\times 1}{1\times 2.2^2} = 19.92\ t/m^2$$

iii Thickness of footing

a. Wide beam shear

Factored intensity of soil pressure,

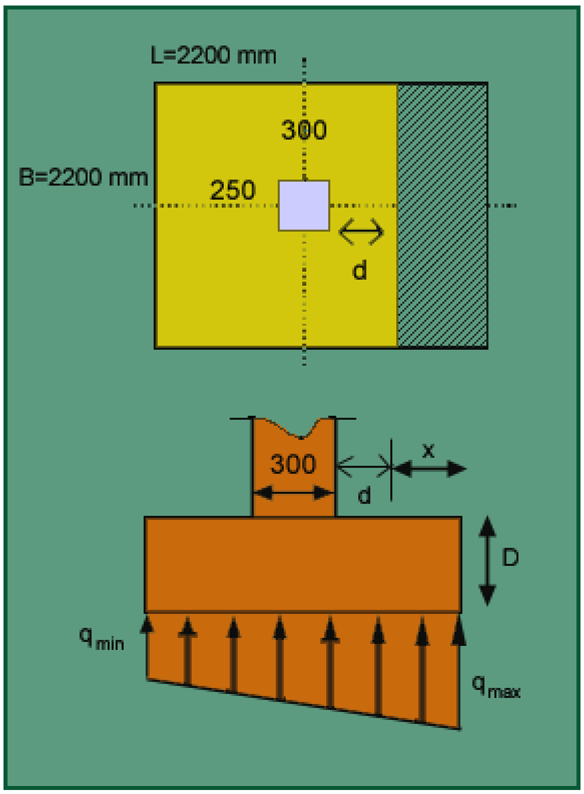


Fig 4.29 Section for wide beam shear and upward earth pressure diagram

$$q_{us} = \left\{q_{max} - \frac{q_{max} - q_{min}}{L}x\right\}1.5$$

$$= \left(19.42 - \frac{19.42 - 16.94}{2200}x\right)1.5$$

$$now, \qquad V = \int_0^x q_{us}\,dx$$

For critical section of wide beam shear: x=(2.2/2)-(0.3/2)-d=0.95-d

$$V = \int_0^{0.95-d}(29.13 - 1.691x)dx = -0.85d^2 - 28.52d + 26.91$$

Assuming P_t=0.2%, and from table 16 of SP-16

$$\tau_c = 0.32\ M_{pa} = 32t/m^2$$

$$V * B = \tau_c * B * d$$

0.0265d2+0.86-0.841=0

By trial and error method, d=0.45 m

b. Punching shear (two way shear)

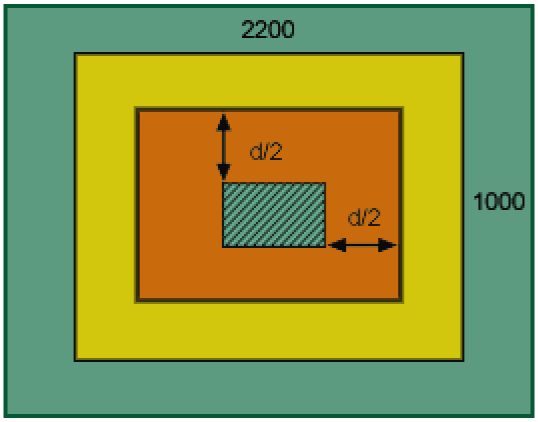


Fig 4.30 Section for two way at a distance of d/2 from face of the column round

$$q_{avg} = \frac{q_{max} + q_{min}}{2} \times 1.5 = 27.27 t/m^2$$

Critical area= (1.1+4d) d m^2

IS: 456-1978, βc =250/300=0.83

Ks=(0.5+ βc)=1.33>1.0

Therefore Ks=1.0

$$\tau_c = 0.25\sqrt{f_{ck}} = 96.8 t/m^2$$

$$\tau'_c = k_s \tau_c = 96.8 t/m^2$$

P_{ult} =40.0*1.5=60 t/m^2

(1.1+4d)*96.8=60-27.27(0.3+d) (0.25+d)

by trial and error, d=0.255 m

d_{req} =450 mm, D=450+40+20/2=500 mm

iv Flexural reinforcement

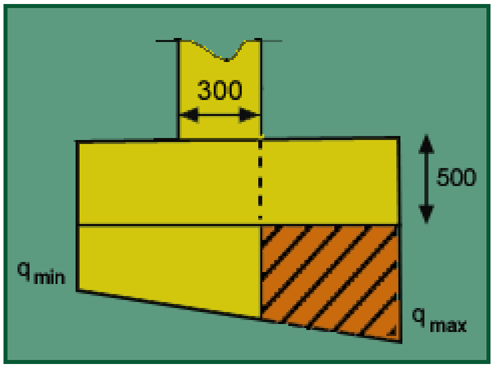


Fig 4.31 Section for bending moment

$$q = \left\{ q_{max} - \frac{q_{max} - q_{min}}{L} x \right\}$$

$$= \left(19.42 - \frac{19.42 - 16.94}{2200} x \left(\frac{2.2}{2} - \frac{0.3}{2} \right) \right)$$

$$= 18.35 t / m^2$$

q_{us} =18.35*1.5=27.53 t/m^2

q_{umax} =19.42*1.5=29.13 t/m^2

BM= {27.53*0.5*0.952} + {(29.13-27.53)*0.95*2/3*0.95}=13.386 t.m

$$\frac{M_u}{bd^2} = \frac{13.386 \times 10^7}{1000 \times 450^2} = 0.66 N / mm^2$$

Table I of SP-16, Pt_{req} =0.193%

For wide beam shear P_t=0.2%

$A_{st\,req}$ =0.2*1000*450/100

Provide 16mm diameter torq bars @200 mm c/c in both directions.
According to clause 33.3.1 of IS: 456

β=2.2/1=2.2

A_{st} in central band width=2/(β+1)* A_{st} total in short direction=2/(2.2+1)*1980=1237.5 mm^2

Hence 16 mm dia @200c/c in longer direction satisfied all criteria & 16 dia @150c/c for central band.

v Check for development length

Clause 25.2.1

$$L_d = \frac{\varphi \sigma_{st}}{4\tau_{bd}} = \frac{16*415}{4*1.6} = 1037.5mm$$

Now length of bars provided, (2200-300)/2= 950 mm< L_d

Provide extra development length of 1037.5-950=87.5 mm say 90 mm on side of the footing.

vi Transfer of load at base of column

Clause 34.4

Permissible bearing pressure, qb=0.45*15=6.75 M_{pa}=675 t/m^2

A_1=1*2.2=2.2 m^2

A_2 =0.3*0.25=0.075 m^2

$$\sqrt{\frac{A_1}{A_2}} = 5.42 \not> 2.0$$

$$q_{pem} = q_p \times \sqrt{\frac{A_1}{A_2}} = 675*2.0=1350 \ t/m^2$$

$$q_{\max\ atcolbase} = (\frac{V}{BL} + \frac{6Ve}{BL^2}) \times 1.5$$

$$= \left(\frac{40}{0.3\times 0.25} + \frac{6\times 1}{0.3^2 \times 0.25}\right)\times 1.5$$

$$= 1200 t/m^2 < q_{perm}$$

$$ok$$

Recap

In this section you have learnt the following

- Design of the Column.
- Design of footing
- Thickness of footing
- Flexural reinforcement
- Check for development length
- Transfer of load at base of column

Objectives

In this section you will learn the following

- Determination of size of column
- Determination of the size of the footing
- Determination of design soil pressure
- Determination of depth of footing
- Flexural reinforcement
- Check for development length
- Transfer of load at the column footing junction

Footing subjected to eccentric loading

Problem 2

Design a non-concentric footing with vertical load =40t and moment = 2tm. Allowable bearing capacity=20t/m 2 . f_{ck} = 15 N/mm^2. f_y =415N/mm^2 .

- **Determination of size of column:**

P = 40t. => P_u = 40 * 1.5 = 60t.

M = 2tm. => M_u = 2 *1.5 = 3tm.

Trial I
Let us assume footing size b= 250mm, D=350mm.

$$\frac{P_u}{f_{ck}\,bd} = \frac{60\times10^4}{15\times250\times350} = 0.46N/mm^2$$

$$\frac{M_u}{f_{ck}bd^2} = \frac{3\times10^7}{15\times250\times350^2} = 0.065\,N/mm^2$$

$$\frac{d'}{D} = \frac{40+20/2}{350} = 0.14$$ (see chart for 0.15)

Ref. Chart 33, SP-16 => $\frac{p}{f_{ck}} = 0.06$ or, p =0.9%

$$A_{st} = \frac{0.9}{100}\times(250\times350) = 787.5mm^2$$

Provide 4 nos. 16 ϕ bars as longitudinal reinforcement and 8 ϕ stirrups @250mm c/c as transverse reinforcement.

- **Determination of the size of the footing**

  Depth of the footing assumed as D= 500mm. For non-concentric footing ,

  Area required = $\dfrac{P}{[q_a-(\gamma_c-\gamma_s)D]} = \dfrac{40}{[20-(2.5-1.8)\times.5]} = 2.036m^2$

  Adopt a rectangular footing of size 2m * 1.1m and depth 0.5m.

  Eccentricity of footing = M/P= 50mm.

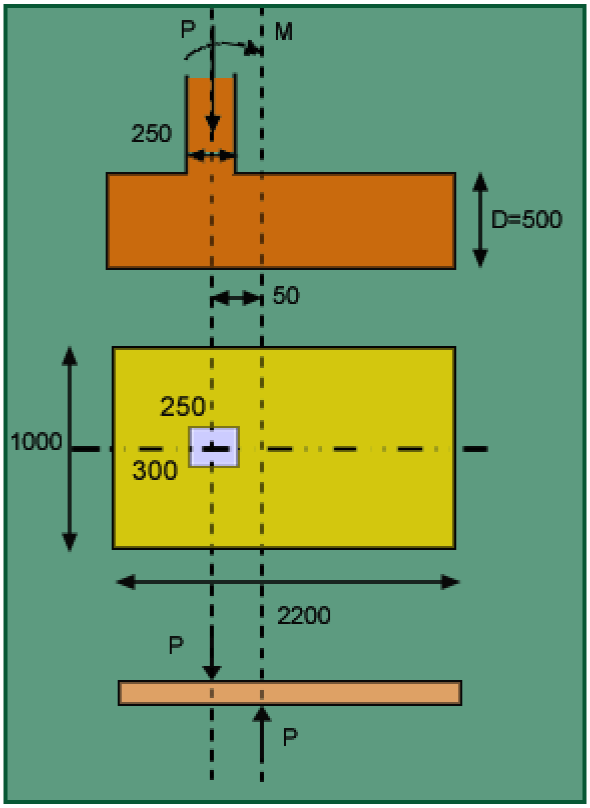


Fig. 4.32 Elevation and Plan of a non-concentric footing

- **Determination of design soil pressure**

R= soil reaction =P =40t.

Q_{acting} =40 / (2 * 1.1) = 18.2 t/m^2 < 20 t/m^2

Therefore, q_{us} = 18.2*1.5 =27.3 t/m^2 .=.273 N/mm^2.

- **Determination of depth of footing:**

a. Wide beam shear:

Consider a section at a distance 'd' from the column face in the longer direction.

Assuming P_t =0.2% for f_{ck} =15N/mm^2, τ_c =0.32N/mm^2.

τ_c .B.d. = q_{us} .B.(L_1 –d)

0.32 * d = 0.273 * (0.875 – d)

Therefore, d = 0.403 m

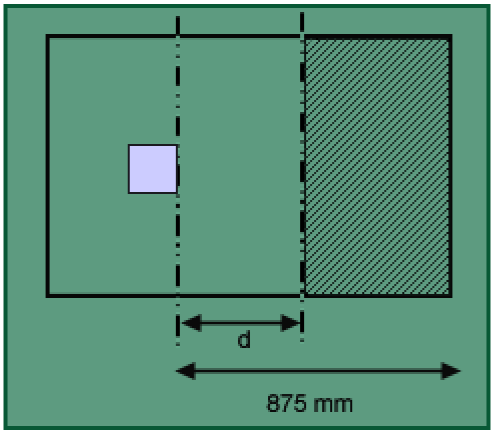


Fig. 4.33 Section for wide beam shear

b. Punching shear:

Critical area for punching shear:
= 2* (350+d+250+d)*d
= 4d(300 + d).
Clause :31.6.3.1 (IS 456:2000)

β_c= 0.25/0.35 =0.71

K_s = 0.5 + β_c=1.21 >1.0

Therefore, take, K_s =1.0.

τ_c = 0.25* (15) 0.5 =0.968 N/mm^2

τ_c ' = τ_c . K_s =0.968 N/mm^2

96.8 * 4d* (0.3 +d) = 60 – 27.3 *(0.35+d)8(0.25+d)
d = 0.246m.

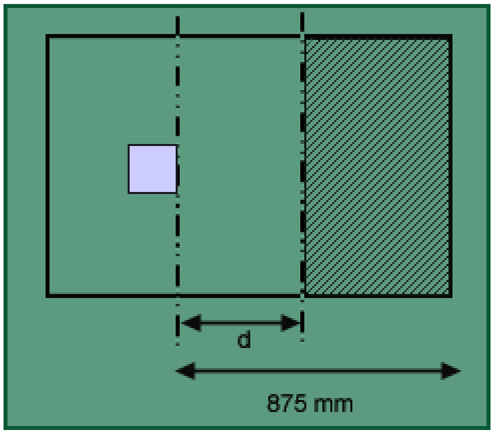


Fig. 4.34 Section for wide beam shear

Therefore, from the punching and wide beam shear criteria we get, 'd" required is

403 mm. D required is (403+40+20/2)=453mm <500mm (D provided). OK.

- **Flexural reinforcement:**

Design soil pressure (q) = 27.3 t/m^2

Bending moment at the face of the column in the longer direction

M_u =27.3 * 0.87 5^2 / 2 =10.45 tm/m width.

d provided = 450mm.

$$\frac{M_u}{bd^2} = \frac{10.45}{1\times 0.45^2} = 51.6t/m^2 = 0.516N/mm^2$$

For singly reinforced section, table 1, SP-16, p t =0.147 N/mm^2

Area of steel required = $\frac{0.147}{100}\times(1000\times 450) = 661.5mm^2/mwidth.$

Spacing using 16 ϕ bars = 201*1000 / 661.5 = 303 mm c/c.

Provide 16 F bars as longitudinal reinforcement @ 300mm c/c in longer direction.

Cl. 33.4.1. (IS-456:2000)

B = 2.0 / 1.1 =1.82

Area of steel in the longer direction = 661.5 * 2 =1323 mm^2

Area of steel in the central band =2 / (1.82 +1)* 1323 =938 mm^2

Spacing = 207.6 mm.

Provide 16 ϕ bars as longitudinal reinforcement @ 200mm c/c in shorter direction in the central band. For remaining portion provide spacing @330mm c/c.

The central band width = width of the foundation =1100mm.

- **Check for development length:**

Cl. 26.2.1 (IS 456 :2000)

$$L_d = \frac{\phi\sigma_s}{4\tau_{bd}} = \frac{16\times415}{4\times1.6} = 1037.5mm.$$

Now, length of bars provided =(2000 – 350)/2 = 825 mm.< L_d .

Extra length to be provided = (1037.5 – 825) = 212.5mm.
Provide development length equal to 225mm at the ends.

- **Transfer of load at the column footing junction :**

Cl. 33.4 (IS 456:2000)

Assuming 2:1 load dispersion,

Required L = {350 + 2*500*2} =2350mm >2000mm.

Required B = {250 + 2*500*2} =2250mm >1100mm.

A_1= 2 * 1.1 =2.2 m^2.

A_2 = 0.25 * 0.35 = 0.0875 m^2

Ö (A_1 / A_2) = 5.01 > 2.0. Take as 2.0.

q_{prem}. = q b * Ö (A 1 / A 2) = 675 * 2 = 1350 t/m^2 .

q_{max} = 40*1.5/(0.25* 0.35) * { 1 + 6 *0.05 / 0.35 } = 1273 t/m^2 . < 1350 t/m^2 .

Therefore, the junction is safe.

Actually there is no need to extend column bars inside the footing, but as a standard practice the column bars are extended upto a certain distance inside the footing.

Recap

In this section you have learnt the following

- Determination of size of column
- Determination of the size of the footing
- Determination of design soil pressure
- Determination of depth of footing
- Flexural reinforcement
- Check for development length
- Transfer of load at the column footing junction

Module 4 : Design of Shallow Foundations

Lecture 18 : Structural designs of column and footing [Section18.3 : Design of Strap Footing]

Objectives

In this section you will learn the following

- Design of the column
- Footing design
- Analysis of footing
- Thickness of footing
- Reinforcement for flexure for footings
- Design of strap beam
- Check for development length
- Transfer of load at base of the column:

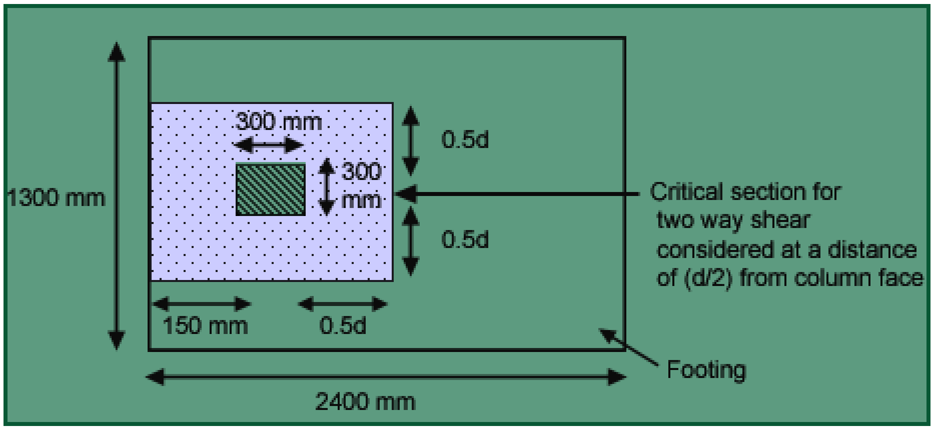


Fig. 4.42 Wide beam shear for footing A

For column B:

From clause 31.6.3.1 of IS456-2000.

$$\beta_c = \frac{width\ of\ column}{length\ of\ column} = \frac{400}{400} = 1.0$$

$$k_s = \beta_c + 0.5 = 1.5 \leq 1.0$$

$$\tau_c = k_s\,(0.25)\sqrt{fck}\ \ (N/mm^2)$$

$$\tau_c = 1.0\,(0.25)\sqrt{15} = 0.968\ \ N/mm^2 = 96.8\ t/m^2$$

Critical perimeter = 2 (0.4+d+0.4+d) = 4 (0.4+d)

So, shear equation becomes,

Critical perimeter x d x $\tau_c = P_u - q_u$ x (critical area – dotted area in fig. 4.43)

$\therefore$ 2 (0.4+d) d (96.8) = 150 – 60.6955 (0.4 + d)

$$\therefore 387.2\,d^2 + 215.58\,d - 125.72 = 0$$

d = 0.355 mm < 600 mm.

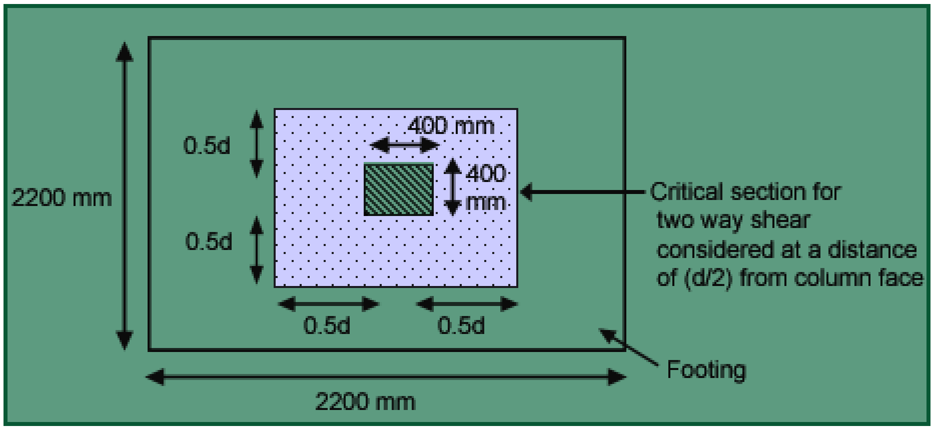


Fig. 4.43 Wide beam shear for footing B

Among all the required d values (for wide beam shear and two way shear criteria),

Max. $d_{required}$ = 521 mm.

$\therefore D_{required}$ = 521 + (20/2) + 40 = 571 mm

So, provide D = 600 mm

$\therefore d_{provided}$ = 550 mm

- **Reinforcement for flexure for footings**

(i) Design along the length direction:

Comparing the moments at the column faces in both the footings (A & B),

M_{max} = 24.61 tm (for Footing B)

$$\frac{M_u}{bd^2} = \frac{24.61 \ x \ 10^7}{10^3 \ x \ (550)^2} = 0.813 \ N/mm^2$$

From table 1 of SP-16, P_t = 0.242 %

(ii) Design along the width direction:

q_{u_1} (=38.1125 t/m) < q_{u_2} (=60.695 t/m)

So, for design along width direction footing B (q_{u_2}) is considered.

$$M_u = \frac{60.6955 \ x \ (0.75)^2}{2} = 17.1 \ tm < M_u \ in \ longer \ direction \ (24.61 \ tm)$$ As shown in fig. 4.44,

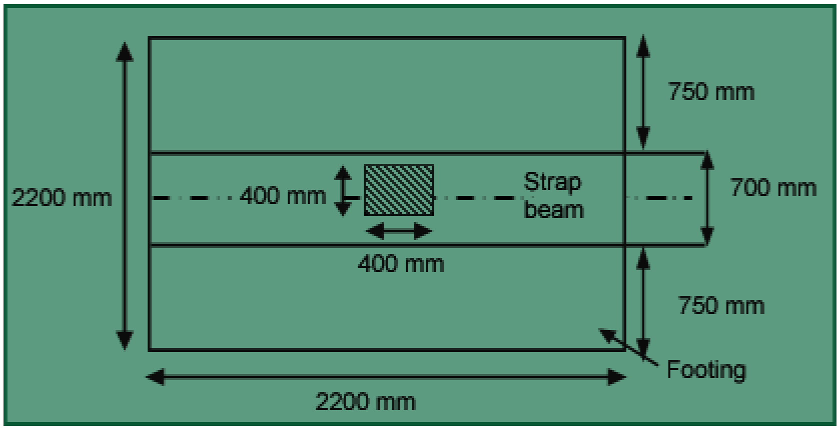


Fig. 4.44 Bending along the width of footing B

So, P_t = 0.242 % i. e. same as reinforcement along longer direction.

But. From wide beam criteria P_t = 0.3 %,

$\therefore$ A_{st}(required) = (0.3/100) x (103) x (550) = 1650 mm^2.

$\therefore$ Provide 20 Tor @ 175 c/c along both directions at bottom face of the footing A and B.

- **Design of strap beam**

(i) Reinforcement for flexture:

M_{max} = 51.294 tm (Refer fig. 4.45)

$$\frac{M_u}{bd^2} = \frac{51.294 \ x \ 10^7}{700 \ x \ (550)^2} = 2.43 \ N/mm^2$$

From table 49 of SP-16, d'/d = 50/550 = 0.1,

$\therefore$ P_t = 0.83 % and P_c = 0.12 %

$\therefore$ A_{st}(required on tension face) = (0.83/100) x 700 x 550 = 3195.5 mm^2,

$\therefore$ A_{sc} (required on compression face) = (0.12/100) x 700 x 550 = 462 mm^2,

$\therefore$ Provide (6+5=) 11 no.s Tor 20 at top of the strap beam and 4 no.s Tor 20 at bottom of the strap beam.

(ii) Check for shear:

V_{max} = 83.235 t

$\tau_{acting} = \dfrac{V_{max}}{b\,d} = \dfrac{83.235 \times 10^4}{700 \times 550} = 2.162\ N/mm^2 < \tau_c$ max = 2.5 N/mm^2 (for M15)

P_t (provided) = $\dfrac{11 \times 314}{700 \times 550} \times 100 = 0.897\%$

From table 61 of SP-16, τ_c = 0.57 N/mm^2

But, provide shear reinforcement for shear = (τ_{acting} – τ_c) = 1.592 N/mm^2= Vus

$\dfrac{V_{us}}{d} = (\tau_{acting} - \tau_c)b = (1.592)(700) = 1114.4\ N/mm$ = 11.144 KN/cm

From table 16 of SP-16, using 4L stirrups, (V_{us}/d) = (11.144/2) = 5.572 KN/cm

∴From table 62 of SP-16, provide 4L-stirrups 10 Tor @ 100 c/c near the column (upto distance of d=550mm from column face) and 4L-stirrups 10 Tor @ 250 c/c for other portions.

- **Check for development length**

From clause 25.2.1 of IS456-2000,

Development length = $L_d = \dfrac{\phi\,\sigma_s}{4\,\tau_{bd}} = \dfrac{20 \times 415}{4 \times 1 \times 1.6} = 1297\,mm$

For column A:

Length of the bar provided = 150-40 = 110mm < L_d

∴By providing 2 no.s 90^o bend the extra length to be provided = (1297-110-3(8 x 20)) = 707 mm.

In B direction length of the bar provided = $\dfrac{1300-300}{2} - 40 = 460\ mm < L_d$

∴Providing two 90^o bend, the extra length to be provided = (1297-460-2(8 x 20)) = 517 mm.

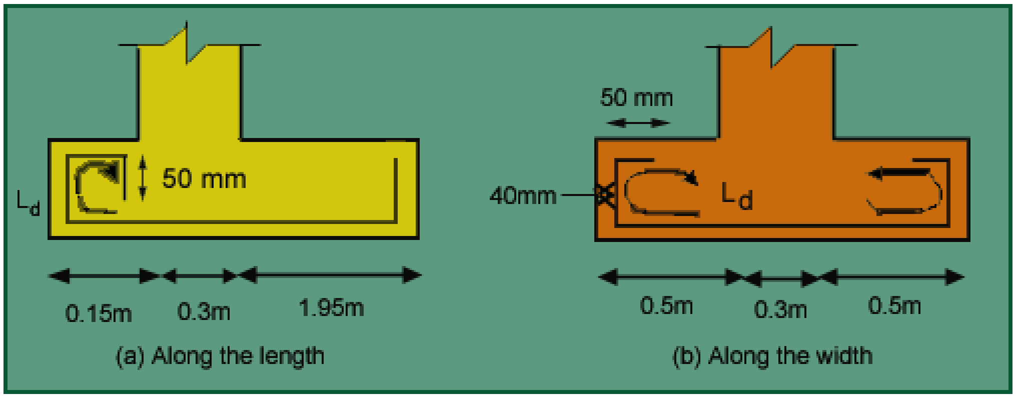


Fig. 4.45 Development length for footing A

For column B:

Length of the bar provided = $\dfrac{2200-400}{2} - 40 = 860\ mm < L_d$

$\therefore$ Providing one 90° bend, the extra length to be provided = (1297-860- (8 x 20)) = 277 mm.

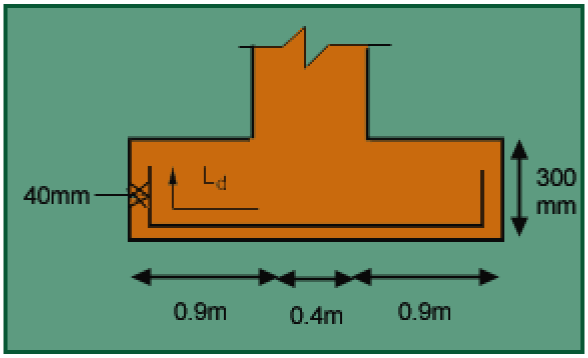


Fig. 4.46 Development length for footing B (Along the length and width)

- **Transfer of load at base of the column:**

For footing A:

From clause 34.4 of IS456-2000, permissible bearing stress (q_{per})= $\sqrt{\frac{A_1}{A_2}}(0.45 fck)$

A_1 = (150+300+1200)(1300)= 2145000 mm^2

A_2 = (300 x 300) = 90000 mm^2

$\therefore \sqrt{\frac{A_1}{A_2}} = 4.88 \leq 2$

$\therefore \sqrt{\frac{A_1}{A_2}} = 2$

q_{per} = 2 x 0.45 x x1500 = 1161 t//m^2

q_{acting} = (load on column/area of column) = (1.5 x 50)/(0.3)2 = 833.3 t//m^2< q_{per} $\because$ Safe.

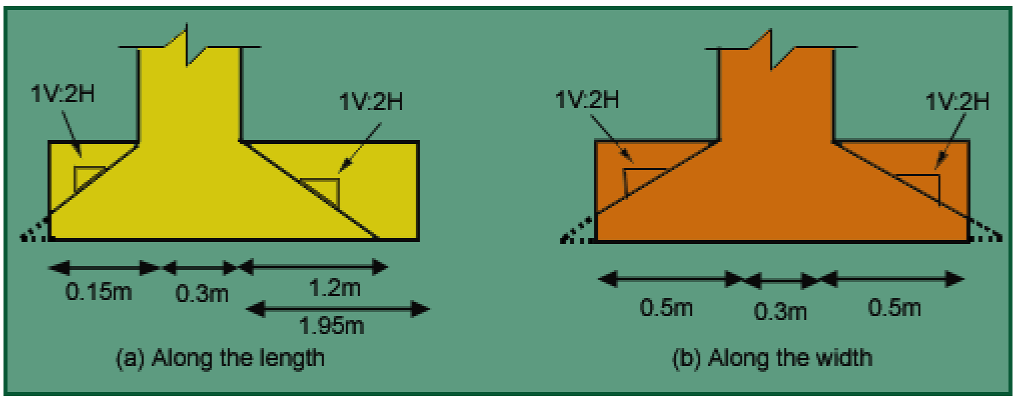


Fig. 4.47 Area of footing A considered for check of transfer of load at column base

For Footing B:

From clause 34.4 of IS456-2000, permissible bearing stress (q_{per})= $\sqrt{\frac{A_1}{A_2}}\,(0.45 fck)$

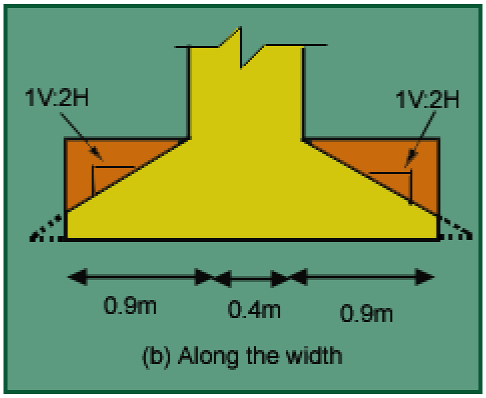


Fig. 4.48 Area of footing B considered for check of transfer of load at column base

A_1 = $(2200)^2$ = 4840000 mm^2

A_2 = (400 x 400) = 160000 mm^2

$\therefore \sqrt{\frac{A_1}{A_2}} = 5.5 \leq 2$

$\therefore \sqrt{\frac{A_1}{A_2}} = 2$

q_{per} = 2 x 0.45 x x1500 = 1161 t/m^2

q_{acting} = (load on column/area of column)

= $(1.5 \times 100)/(0.4)^2$

=937.5 $t/m^2 < q_{per}$

$\therefore$ Safe

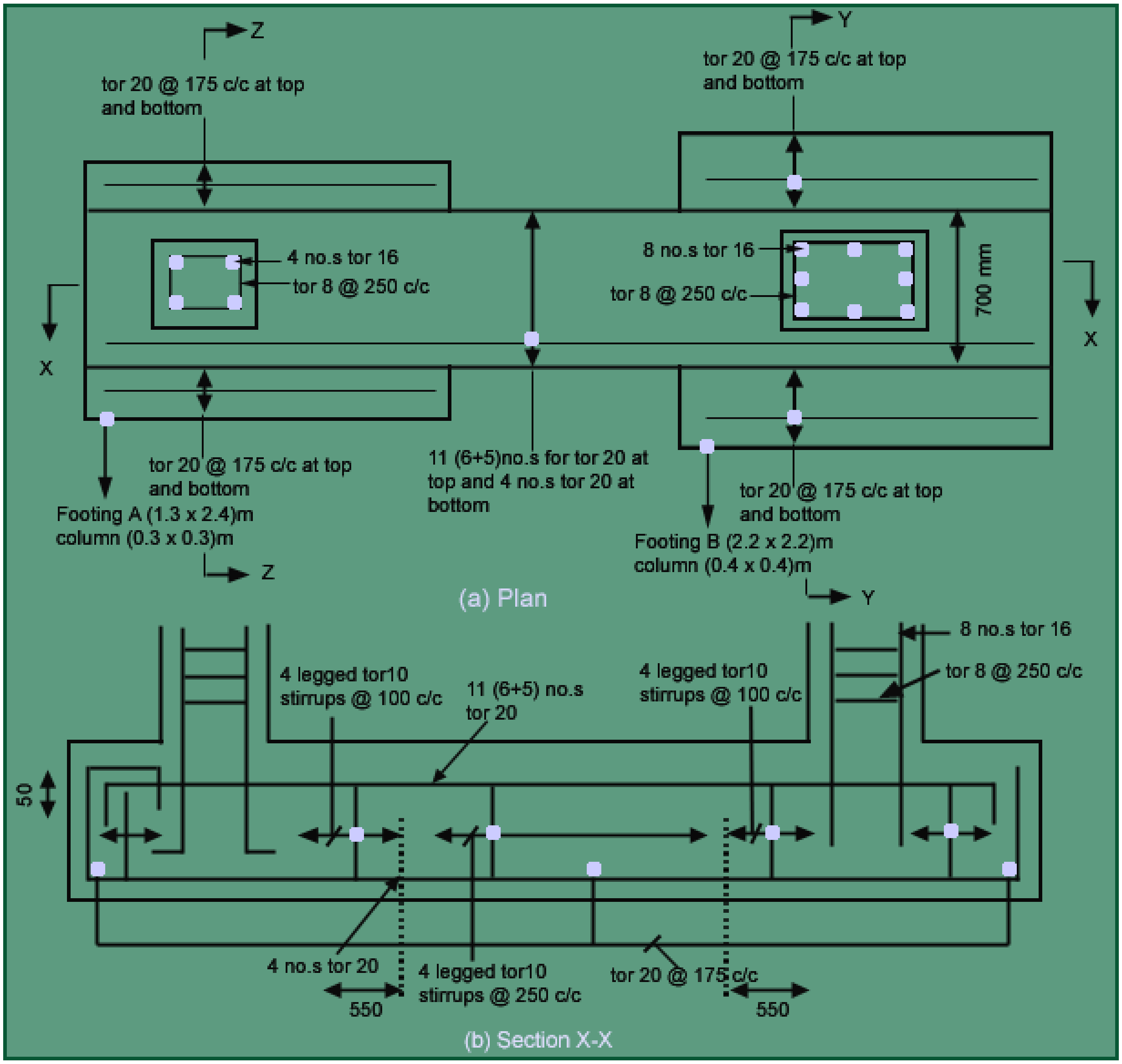


Fig. 4.49 Reinforcement in footing

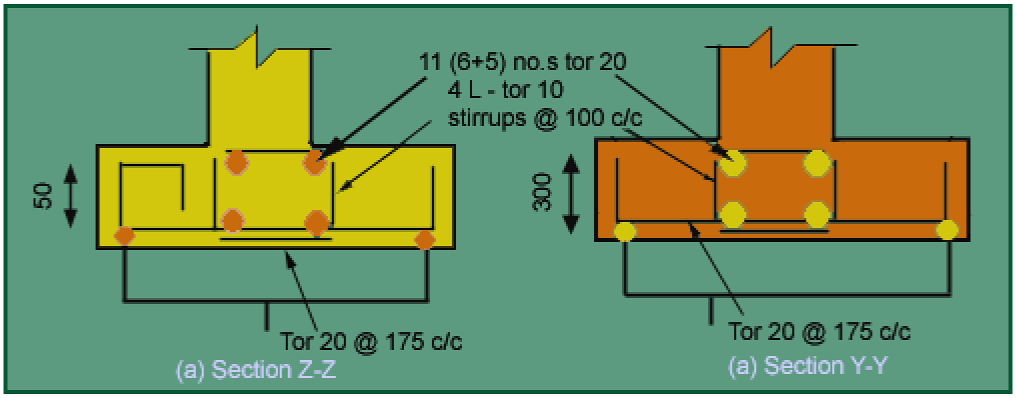


Fig. 4.50 Reinforcement of footing

Design of strap beam

Recap

In this section you have learnt the following

- Design of the column
- Footing design
- Analysis of footing
- Thickness of footing
- Reinforcement for flexure for footings
- Design of strap beam
- Check for development length
- Transfer of load at base of the column:

Design of strap footing:

Example:

The column positions are is as shown in fig. 4.35. As column one is very close to the boundary line, we have to provide a strip footing for both footings.

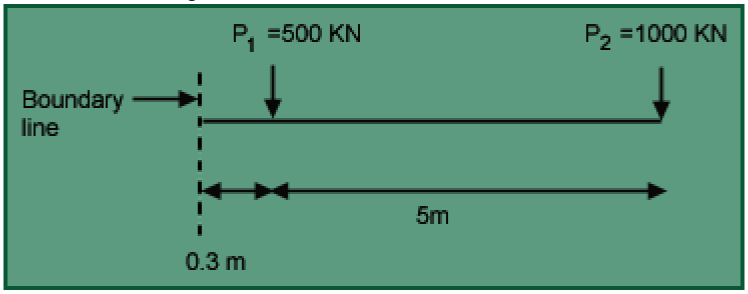


Fig. 4.35 Strap footing

- **Design of the column**

Column A:

P_u =750 KN

Let P_t = 0.8%, so, A_x= 0.008A and A_c = 0.992A,

Where, A is the gross area of concrete.
As per clause 39.3 of IS 456-2000,
750 x 103 = (0.4 x 15 x 0.992A) + (0.67 x 415 x 0.008A)
A = 91727.4 mm^2
Provide column size (300 x 300) mm
$\therefore$ 750 x 103 = 0.4 x 15 x (1- (pt/100)) x 90000 + 0.67 x 415x (P_t /100) x 90000

$\therefore$ P_t = 0.86% ,

$A_{st\ required}$ = (0.86/100) x (300)2 = 774 mm^2

Provide 4 no's tor 16 as longitudinal reinforcement with tor 8 @ 250 c/c lateral ties.

Column B:

P_u =1500 KN

Provide column size (400 x 400) mm

$\because$ 1500 x 103 = 0.4 x 15 x (1- (P_t/100)) x 160000 + 0.67 x 415x (pt/100) x 160000

$\therefore$ P_t = 1.24% , $A_{st\,required}$ = (1.24/100) x (300)2 = 1985 mm^2

Provide 8 no.s tor 16 as longitudinal reinforcement with tor 8 @ 250 c/c lateral ties.

- **Footing design**

Let us assume eccentricity e = 0.9m.

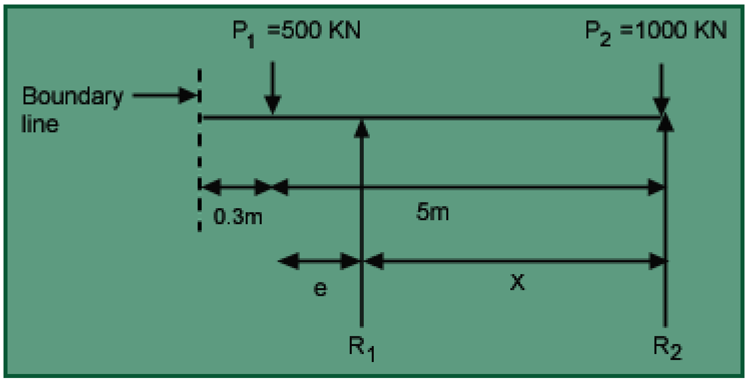


Fig. 4.36 Strap footing – soil reaction

Taking moment about line P_2,

P_1x 5 – R_1x (5-e) = 0

$$\therefore R_1 = \frac{5x500}{5-0.9} = 609.8\,KN$$

$$\therefore R_2 = P_1 + P_2 - R_1 = 500+1000-609.8 = 890.2\,KN$$

Footing size:

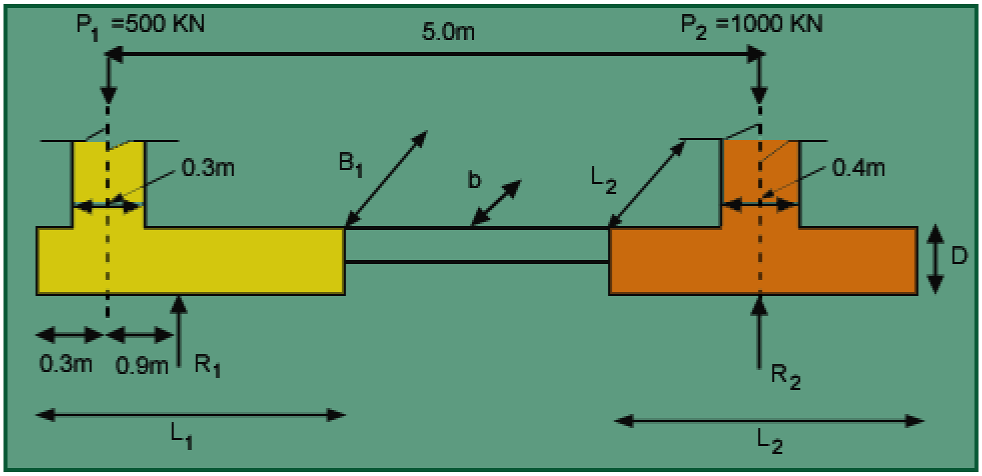


Fig. 4.37 Footing sizes

For footing A:

L_1 = 2(0.9+0.3) =2.4m.

Assume overall thickness of footing, D = 600mm.

$$B_1 = \frac{R_1}{(q_a - (\gamma_c - \gamma_s)D)L_1} = \frac{60.98}{(20 - (2.5 - 1.8)\,0.6)\,2.4} = 1.298m$$

For footing B:

Assume square footing of size L_2 ,

$$L_{22} = \frac{R_2}{(q_a - (\gamma_c - \gamma_s)D)L_1} = \frac{89.02}{(20 - (2.5 - 1.8)\,0.6)}$$

L_2 = 2.13m

Provide (2.2 x 2.2)m footing.

- **Analysis of footing**

$$q_{u1} = \frac{R_1 \; x\, 1.5}{L_1 \; x \, 1m} = \frac{60.98 \; x \, 1.5}{2.4 \; x \, 1.0} = 38.1125 \; t/m$$

$$q_{u2} = \frac{R_2 \; x\, 1.5}{L_2 \; x \, 1m} = \frac{89.02 \; x \, 1.5}{2.2 \; x \, 1.0} = 60.695 \; t/m$$

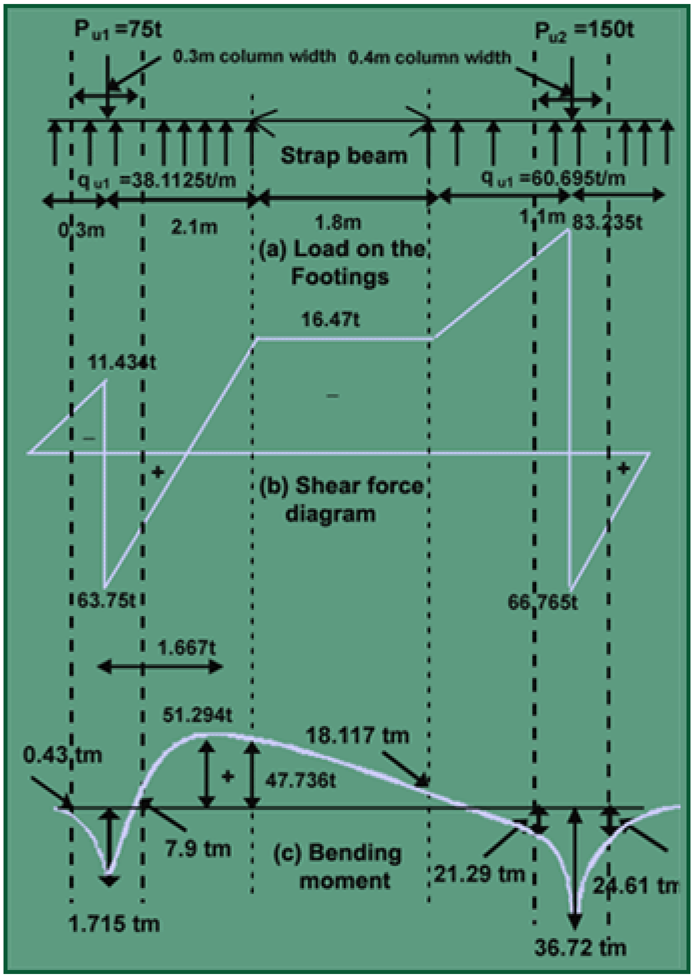


Fig. 4.38 Analysis of footing

- **Thickness of footing**

i) Wide beam shear:

For footing A:

Let us assume P_t = 0.2%, so from table 16 of IS456, $\tau_c = 0.32\ N/mm^2 = 32\ t/m^2$

Assume in direction of B_1 , width of strap beam (b) is 500 mm.

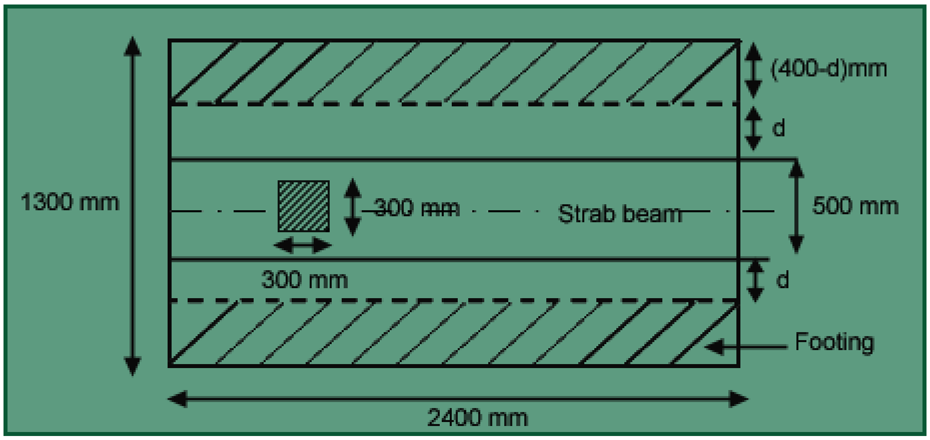


Fig. 4.39 Wide beam shear for footing A

Shear = b d τ_c = q_u (0.4 - d)

$\therefore (0.5)\ d\ (32) = (38.1125)\ (0.4 - d)$

$\therefore d = 0.282\ m$

For footing B:

Let us assume P_t (%) = 0.2%, so from table 16 of IS456, $\tau_c = 0.32\ N/mm^2 = 32\ t/m^2$

Assume in direction of B_1, width of strap beam (b) is 500 mm.

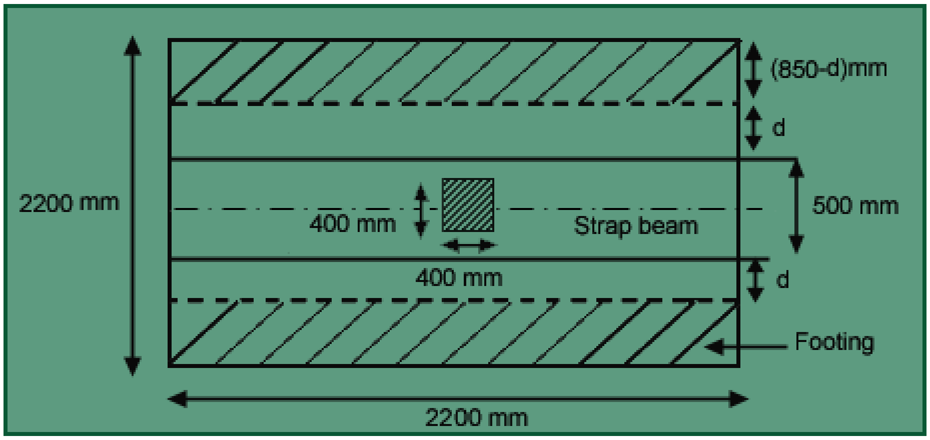


Fig. 4.40 Wide beam shear for footing B

Shear = b d τ_c = q_u (0.4 - d)

$\therefore (0.5)\ d\ (32) = (60.6955)\ (0.85 - d)$

$\therefore d = 0.673\ m$ > 600 mm depth earlier assumed.

$\because$ Increasing the width of the beam to 700 mm

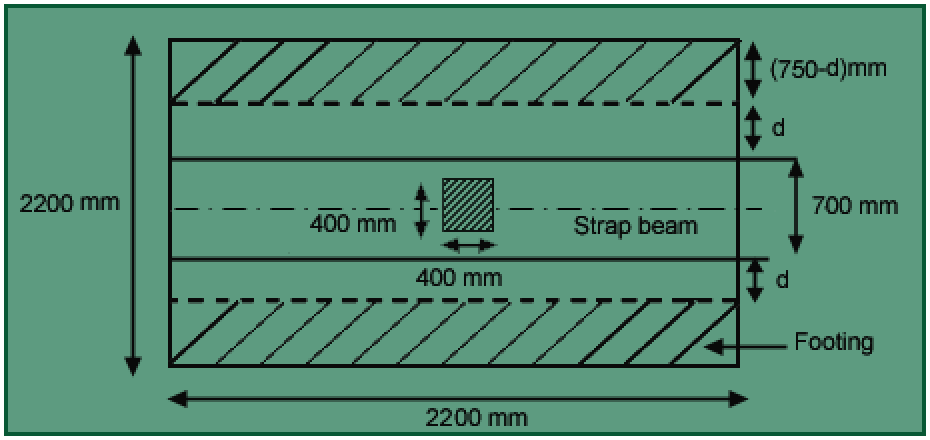


Fig. 4.41 Wide beam shear for footing B

Let us assume P_t (%) = 0.3%, so from table 16 of IS456, $\tau_c = 0.38\ N/mm^2 = 38\ t/m^2$

Shear = b d τ_c = q_u (0.75 - d)

$\therefore (0.7)\ d\ (38) = (60.6955)\ (0.75 - d)$

$\therefore d = 0.521\ m$ < 600 mm depth earlier assumed.

$\because$ Safe

ii) Two way shear:

For column A:

From clause 31.6.3.1 of IS456-2000.

$$\beta_c = \frac{width\ of\ column}{length\ of\ column} = \frac{300}{300} = 1.0$$

$$k_s = \beta_c + 0.5 = 1.5 \leq 1.0$$

$$\tau_c = k_s\ (0.25)\sqrt{fck}\ \ (N/mm^2)$$

$$\tau_c = 1.0\ (0.25)\sqrt{15} = 0.968\ \ N/mm^2 = 96.8\ t/m^2$$

Critical perimeter x d x τ_c = $P_u - q_u$ x (critical area – dotted area in fig. 4.42)

So, shear equation becomes,

Critical perimeter x d x τ_c = $P_u - q_u$ x (critical area – dotted area in fig. 4.42)

$\because$ 2 (0.75+1.5d) d (96.8) = 75 – 38.1125 (0.3 + 0.15 + 0.5d)

$\therefore 290.4\, d^2 + 164.25\, d - 57.85 = 0$

d = 0.246 mm < 600 mm.

Objectives

In this section you will learn the following

- Column design
- Thickness of Footing
- Flexural reinforcement

Design of Combined Footing

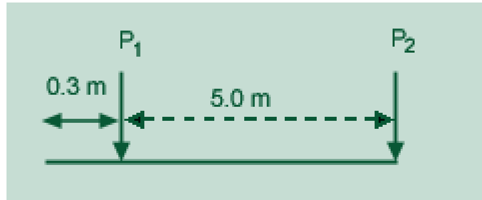


Fig. 4.51 Loading on combined footing

P_1 =800kN

P_2 =1000kN

q_a =20 t/m^2,M15, f_y =415kN/m^2

Column size: 400x400mm.

See Fig 4.54 for details of footing.

- **Column design**

Let pt=0.8%

A_{sc} =.008A; A_c =0.992A

Clause.39.3 of IS 456-2000

A=146763.8mm^2

A_{sc} =1174.11 mm^2, A_c =145589.746mm^2

Provide footing of 400x400size for both columns.

Using 8-16 ϕ as main reinforcement and 8 ϕ @250c/c as lateral tie

- **Design of Footing**

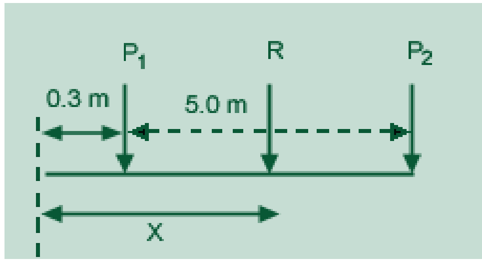


Fig. 4.52 Forces acting on the footing

Resultant of Column Load

R =1800 kN acting 3.08m from the boundary.

Area of the footing :

Taking length L=6m, Depth of footing D_f =0.9m, $\gamma_c = 2.5 t/m^2$, $\gamma_s = 1.8 t/m^2$

Width of footing, $B = \dfrac{P_1 + P_2}{L[q_a - (\gamma_c - \gamma_s)D_f}$ =1.549m.

Therefore, provide footing of dimension 6m x 1.6m

Soil Pressure q = $\dfrac{180}{6\text{x}1.6}$ =18.75 t/m^2< 20 t/m^2 OK.

q_u =28.125 t/m^2

Soil pressure intensity acting along the length =B x q_u =1.6x28.125 =45t/m.

R_B =119.88kN, R_C =150.12kN.

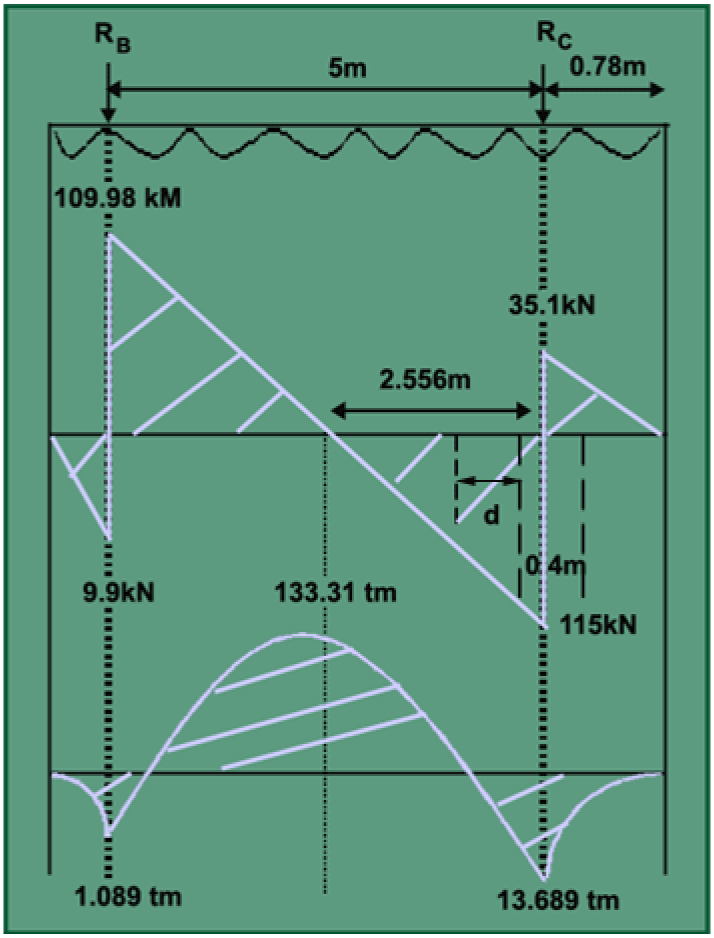


Fig. 4.53 Shear Force and Bending Moment Diagrams

- **Thickness of Footing**

i. Wide beam shear:

Maximum shear force is on footing C,SF=115.02KN

$\tau_c \times B \times d = qu[2.556 - 0.2 - d]$

$\tau_c = 0.32 N/mm^2$ for percentage reinforcement P_t=0.2%

0.32 x d x 1.6=45 [2.556-0.2-d]

d=1.1m

$\tau_c = 0.5 N/mm^2$ for percentage reinforcement P_t=0.6%

0.6 x d x 1.6=45 [2.556-0.2-d]

d=0.847m.D=900mm.OK.

ii.Two way Shear

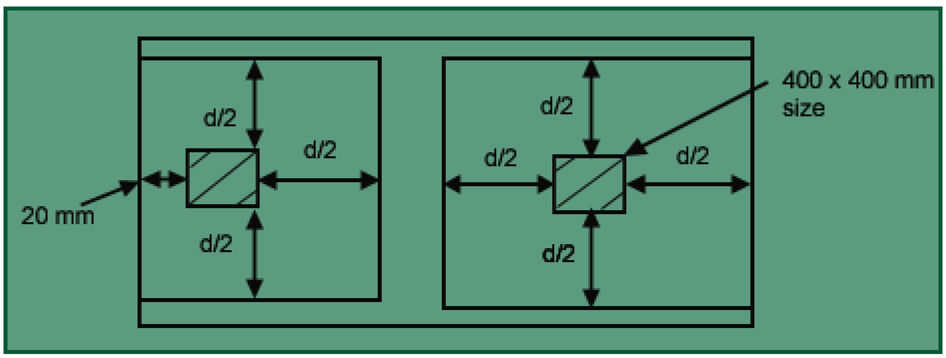


Fig 4.54 Section for two way shear

$\beta_c = 1, k_s = 1.5 \Rightarrow 1,$

$\tau_c' = k_s \tau_c = 96.8 t/m^2$

Column B

$4(0.4+d)d \times 96.8 = 150 - 28.125(0.4+d)^2$

d=0.415m.

Column A

2d[(0.4+d)+(0.42+d/2)] x 96.8=120-28.125[(0.4+d)(0.42+d/2)]

d=0.3906m

d_{reqd} =0.85mm

$D_{provided}$ =900mm, d_{reqd} =850mm.OK.

- **Flexural reinforcement**

Along Length Direction

$\frac{M_u}{bd^2} = \frac{133.31 \times 10^4}{1.6 \times 850^2}$ =1.15N/mm^2

Table 1of SP16

P_t =0.354%

P_t provided=0.6%

A_{st} required=5100 mm^2/mm

Provide 28 ϕ@120mmc/c at top and bottom of the footing

Along width direction

$M_u = \frac{28.125 \times 1.6^2}{2} = 36tm$

$M_u = 0.073 N/mm^2$

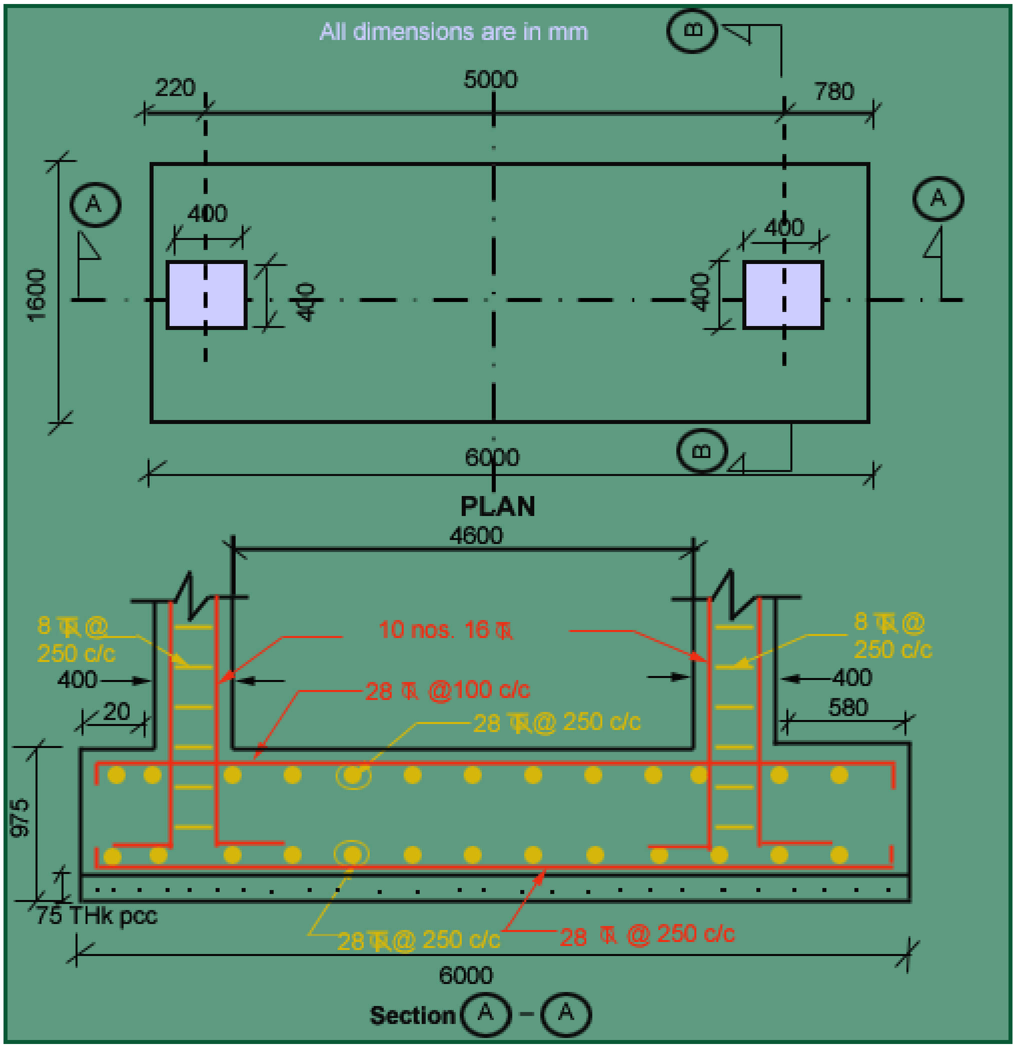


Fig. 4.55 Combined Footing

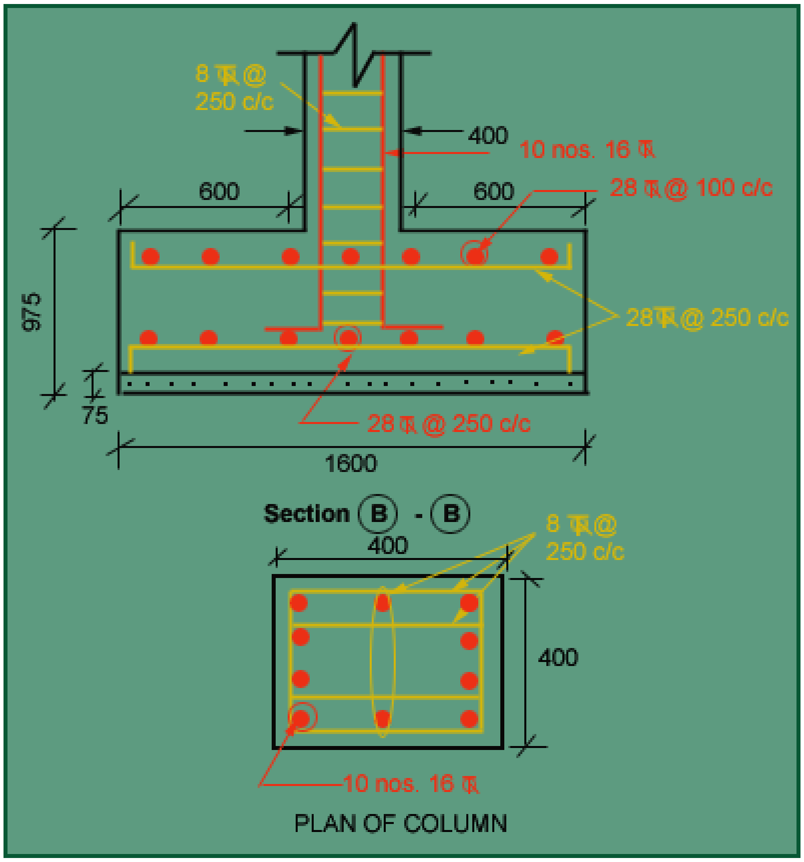


Figure 4.56 Detailed Plan of Combined Footing

Recap

In this section you have learnt the following

- Column design
- Thickness of Footing
- Flexural reinforcement

Module 4 : Design of Shallow Foundations

Lecture 18 : Structural designs of column and footing [Section18.5 : Raft Footing]

Objectives

In this section you will learn the following

- Column sizes
- Thickness of raft
- To calculate k & λ-Stiffness factors
- Reinforcement in width direction
- Reinforcement in length direction
- Shear (wide beam shear criterion)
- Along the width direction
- Shear check
- Development Length
- Transfer of load at the base of the column

- **Shear (wide beam shear criterion)**

In width direction

$$\frac{V_u}{b\,d_x} = \frac{471\times10^4}{23.2\times10^3 * 1015} 0.2 \text{ N/mm}^2 < \tau_v$$

Pt_{prov} =0.123%,

τ_c =0.27 N/mm 2 > τ_v (from table 61 of SP – 16 by extrapolation)

Therefore no shear reinforcement is required.

$$\frac{V_u}{b\,d_y} = \frac{229.7\times10^4}{12.8\times10^3 * (1013-20)} = 0.235 \text{ N/mm}^2 < \tau_c (0.27 \text{ N/mm}^2)$$

Therefore no shear reinforcement is required.

Along the width direction

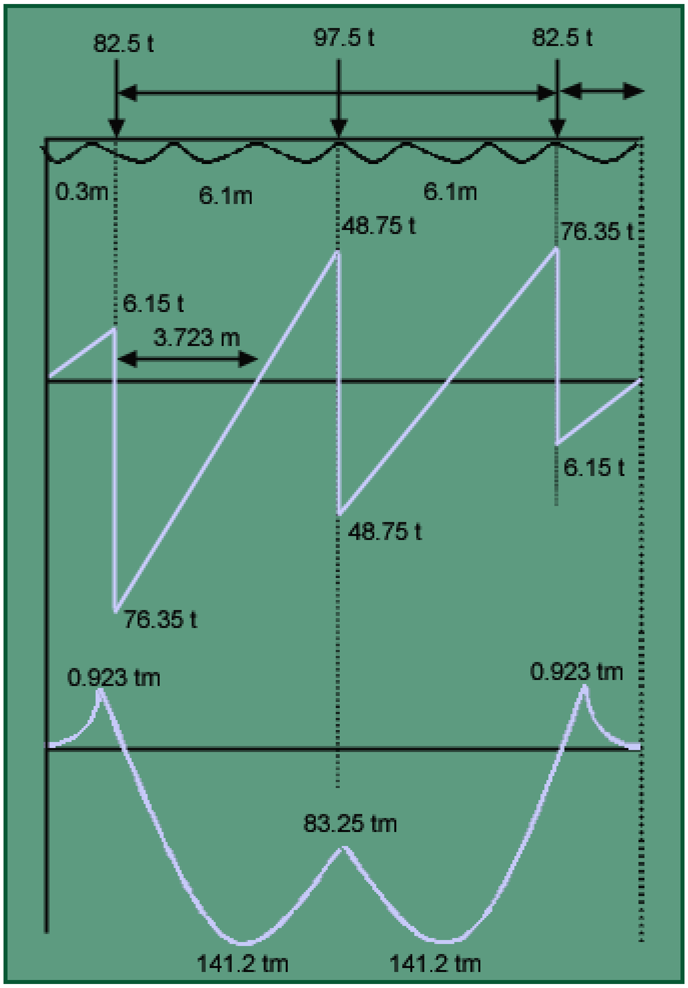


Fig. 4.63 Shear Force and Bending Moment Diagrams of strips 1 and 4

In width direction: Strip1/4:-

M_u =141.2tm

$$\frac{M_u}{bd^2} = \frac{141.2\times10^7}{4.067\times10^3\times(1015)^2} = 0.337\text{N/mm}^2$$

Strip2 / 3

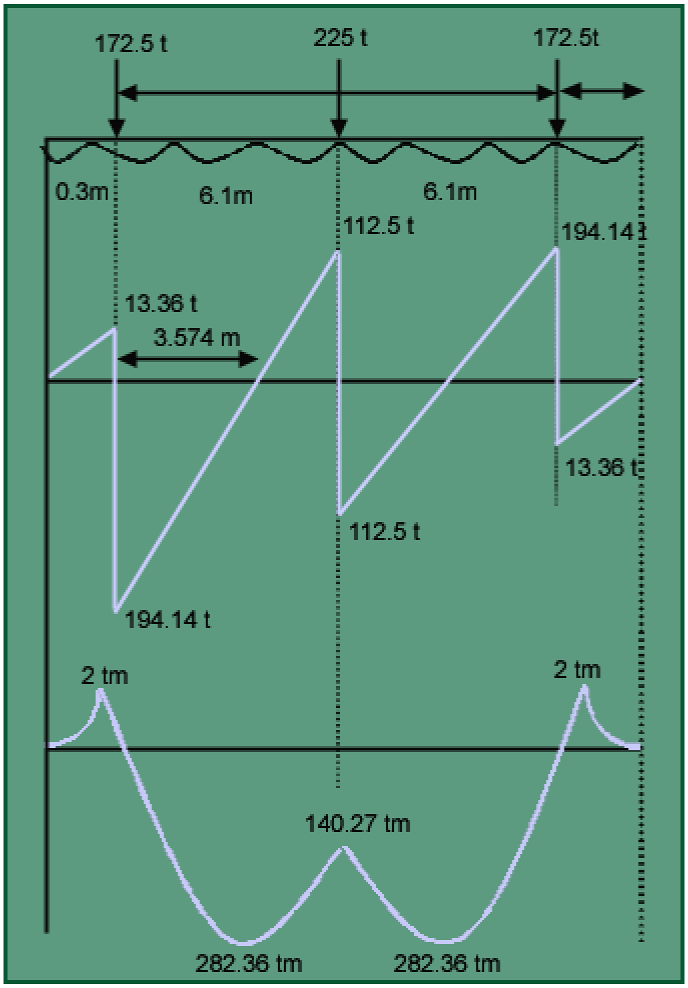


Fig. 4.64 Shear Force and Bending Moment Diagrams of strips 2 and 3

Strip 2/3

M_u =282.36tm

$$\frac{M_u}{bd^2} = \frac{282.36\times10^7}{7.533\times10^3\times(1015)^2} = 0.364 N/mm^2$$

Minimum P_t =0.12%has to be provided.

Provide 20 ϕ@200c/c in centre band and 20 ϕ@300c/c at other parts along the shorter direction.

1. **Shear check**

Along width direction:-

For strip1/4:

V_u =76.35t

$$\frac{V_u}{bd_x} = \frac{76.35\times10^4}{4.067\times10^3\times1015} = 0.185 N/mm^2 < \tau_c\text{, OK.}$$

For strip 2/3:

V_u =159.14 t

$$\frac{V_u}{bd_x} = \frac{159.14\times10^4}{7.533\times10^3\times1015} = 0.208 N/mm^2 < \tau_c\text{, OK.}$$

Hence no shear reinforcement is required.

2. Development Length

$L_d = \frac{\phi \sigma_s}{4 \tau_{bd}} = \frac{20 \times 0.87 \times 415}{4 \times 1 \times 1.6}$ =1128.3mm

At the ends, length of bar provided=150mm.
Extra length to be provided=1128.3-150-8x20=818.3mm.
Provide a Development length of 850mm

3. Transfer of load at the base of the column:-

For end column;

A_1=2650X2725=7.22125x106mm^2

A_2 =300x450=135000mm^2

$\sqrt{\frac{A1}{A2}}$ = 7.31 But not greater than 2.0

q_{prem} = $\sqrt{\frac{A1}{A2}} \times 0.45 f_{ck}$ =13.5N/mm^2

q_{acting} = $\frac{55 \times 10^4}{300 \times 450}$ =4.07N/mm^2< q_{prem} .OK.

For 150t columns

q_{acting} = $\frac{150 \times 10^4}{450^2}$ =7.41N/mm^2< q_{prem} .OK.

For 115t columns

$\sqrt{\frac{A1}{A2}}$ = 2, q_{acting} = $\frac{115 \times 10^4}{300 \times 450}$ =8.52N/mm^2< q_{prem} .OK.

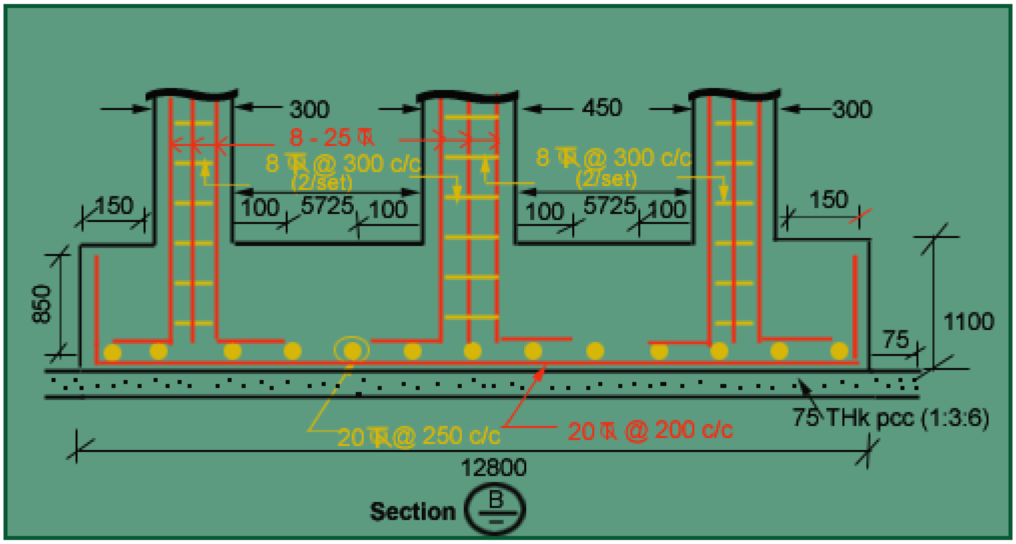


Fig 4.65 Details of reinforcement

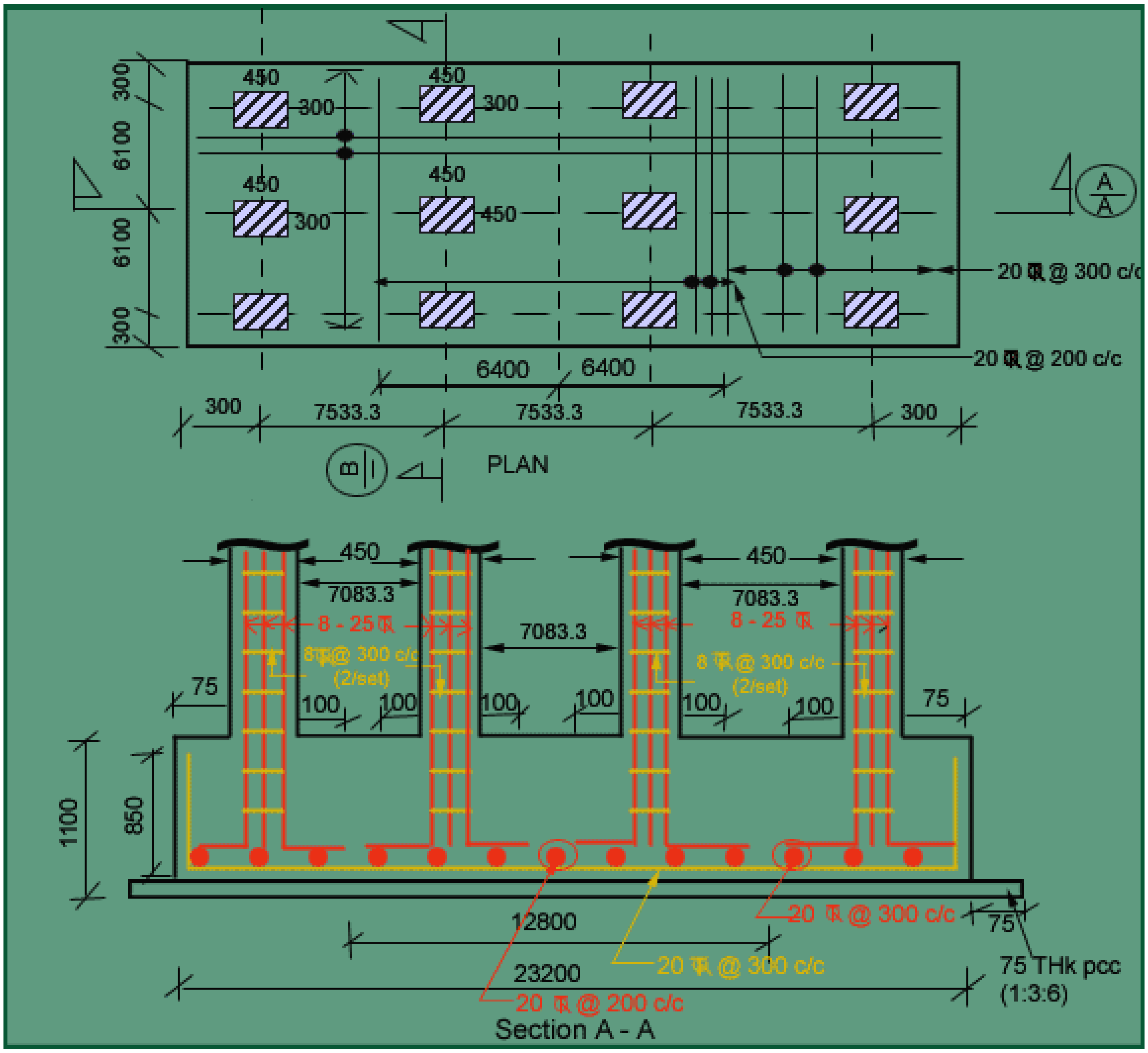

PLAN
Section A - A
20 ϕ @ 300 c/c
20 ϕ @ 200 c/c
8 - 25 ϕ
8 ϕ @ 300 c/c (2/set)
75 THk pcc (1:3:6)
12800
23200
7083.3
7533.3
6400
6100

Fig 4.66

Recap

In this section you have learnt the following

- Column sizes
- Thickness of raft
- To calculate k & β-Stiffness factors
- Reinforcement in width direction
- Reinforcement in length direction
- Shear (wide beam shear criterion)
- Along the width direction
- Shear check
- Development Length
- Transfer of load at the base of the column

Raft Footing

Design the raft footing for the given loads on the columns and spacing between the columns as shown below.

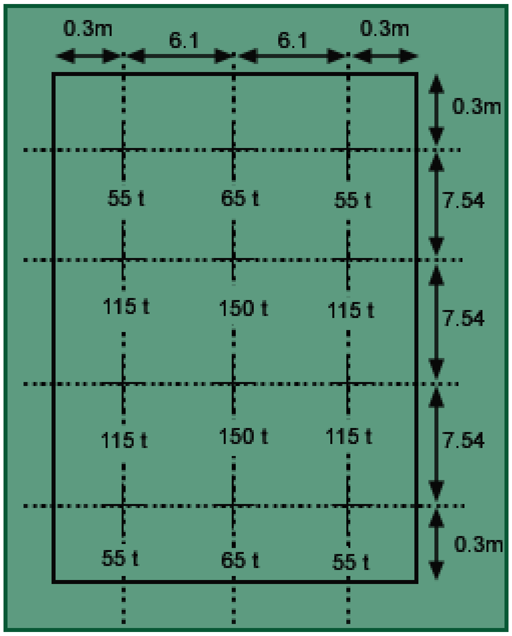


Fig 4.57 column locations and intensity of loads acting on the raft

- **a) Column sizes**

Take size of the columns are as: 300*450 mm for load of less than 115 ton

450*450 mm for a load of greater than 115 ton

- **Thickness of raft**

$$q_{us} = \frac{1110}{12.8 \times 23.22} * 1.5 = 5.607\, t/m^2$$

Two way shear
The shear should be checked for every column, but in this case because of symmetry property checking for 115 t, 150 t, and 55 t is enough.

For 150 t column

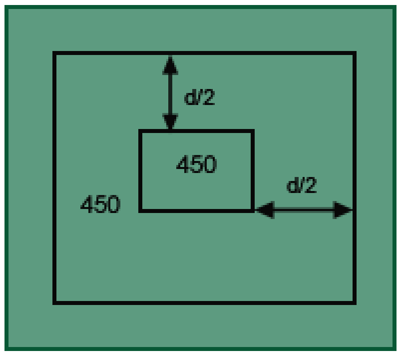


Fig 4.58 section for two way shear for 150 t column

IS: 456-1978, β_c =450/450=1.0

K_s =(0.5+ β_c)=1.0=1.0

Therefore K_s =1.0

$$\tau_c = 0.25\sqrt{f_{ck}} = 96.8\, t/m^2$$

$$\tau_c' = k_s\, \tau_c = 96.8\, t/m^2$$

$$\tau_c' = 96.8\, t/m^2$$

4(0.45+d)*d*96.8=150*1.5-5.607(0.45+d)2
Therefore d=0.562 m

For 115 t column

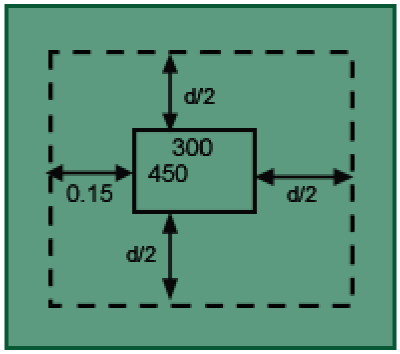


Fig 4.59 section for two way shear for 115 t column

2(0.45+d+0.15+0.3+d/2) d*96.8=115*1.5-5.607(0.45+d)(0.3+0.15+0.5d)

Therefore d=0.519 m

For 55 t column

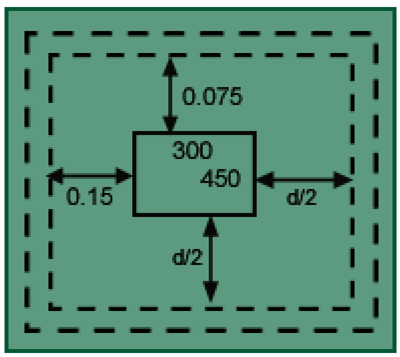


Fig 4.60 section for two way shear for 55 t column

2(0.45+0.075+0.5d+0.15+0.3+0.5d) d*96.8=55*1.5-5.607(0.45+0.5d+0.075)(0.3+0.5d+0.15)

Therefore d=0.32 m

The guiding thickness is 0.562m and code says that the minimum thickness should not be less than 1.0m.

let provide a overall depth of 1.1m=D

d_{prdv} =1100-75-20/2=1015mm.

- **To calculate k & λ-Stiffness factors**

There are two criterions for checking the rigidity of the footing:
Plate size used is 300*300 mm.
For clays: μs =0.5,

$$k = B\frac{E_s}{1-\mu^2{}_s}$$

Take k=0.7 and B=30 cm
Es=15.75 kg/cm^2=1.575 N/mm^2

$$K = \frac{EI}{E_s b^3 a}, \qquad where \;\; I = \frac{ad^3}{12}$$

$$E = 5000\sqrt{f_{ck}}$$

$$= 5000\sqrt{15} = 19364.92\, N/mm^2$$

b=23.2*103 mm, a=12.8*103 mm 4 , d=1015 mm

$$K = \frac{19364.92\times1015^3\times12.8*10^3}{12\times1.575\times(23.2*10^3)^3\times12.8*10^3}$$

=0.085<0.5

Therefore it is acting as a flexible footing.

$$\lambda = \left(\frac{kB}{4E_cI}\right)^{\frac{1}{4}} = \frac{0.7\times 12.8*10^2\times 12}{4\times 19364.92\times 12.8*10^2\times 101.5^3}$$

$=0.00179*10^{-3}$

$1.75/\lambda = 975.184 = 9.75\text{m}$

If column spacing is less than $1.75/\lambda$, then the footing is said to be rigid.

Therefore the given footing is rigid.

One criterion showing the footing is flexible and another showing that the given footing is rigid. Both are contradicting each other, so design the footing for both criterions.

$$q_{act} = \frac{1100}{12.8*23.2} = 3.738t/m^2$$

$q_{us}=5.607\ t/m^2$

$$q = \frac{2*510+645}{6.1+6.1+0.6} = 130.08t/m^2$$

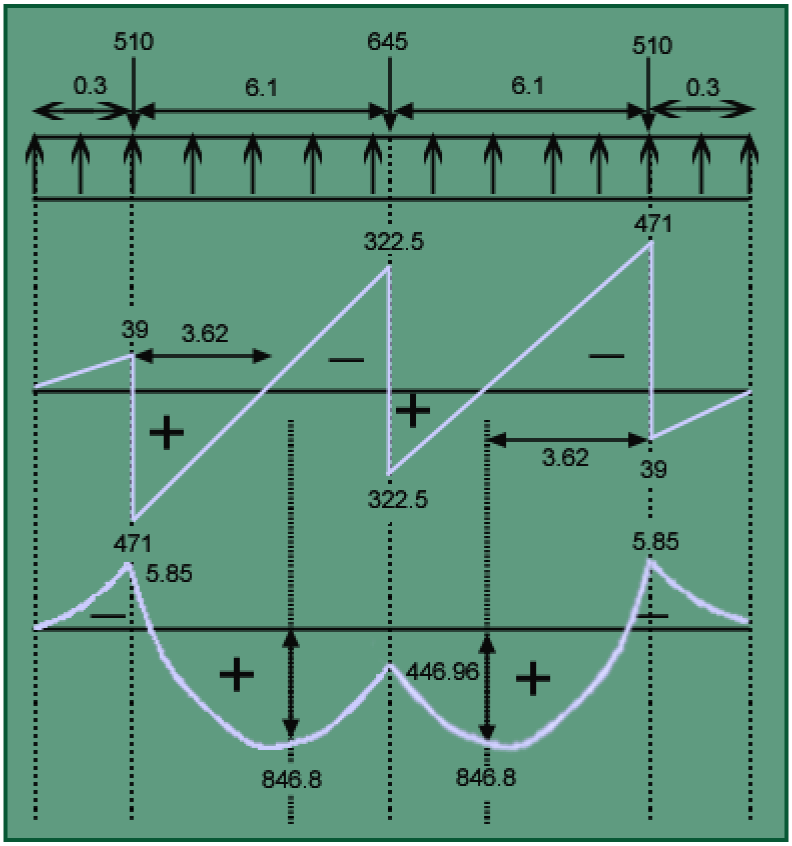


Fig 4.61 shear force and bending moment diagrams along width direction

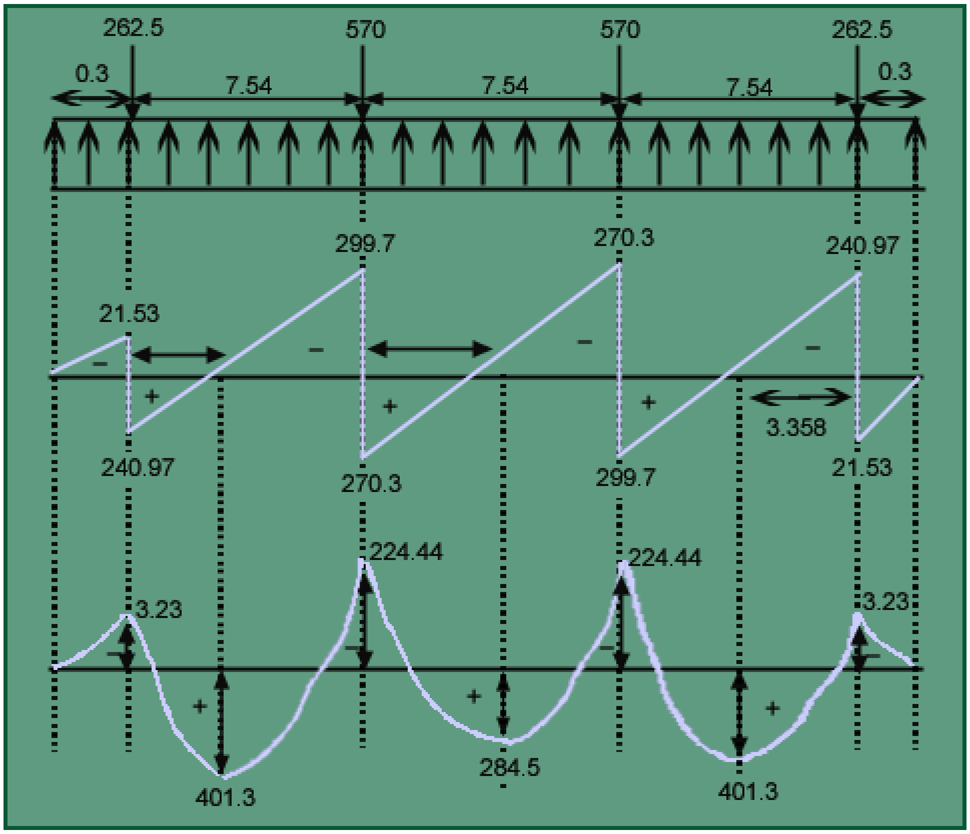


Fig 4.62 shear force and bending moment diagrams along length direction

- **Reinforcement in width direction**

$$\frac{M_u}{b\,d^2{}_x} = \frac{846.8\times10^7}{23.2\times10^3 * 1015^2} = 0.354$$

From SP-16 graphs

P_t =0.102%, but minimum is 0.12%.

A_{stx} =(0.12*1000*1015)/100=1218 mm^2

Provide 20 mm diameter bars @250 c/c along shorter direction in bottom.

- **Reinforcement in length direction**

$$\frac{M_u}{b\,d^2{}_y} = \frac{401.3\times10^7}{12.8\times10^3 * (1015-20)^2} = 0.316$$

Provide 20 mm diameter bars @250 c/c in longer direction.

Clause 33.3.1

$$A_{stx_{central band}} = \frac{2}{\frac{23.2}{12.8}+1}(1218 * 23.2)\,mm^2$$

Provide 20 mm diameter bars @ 200 c/c in central band and 20 mm diameter bars @300 c/c at other parts along shorter direction at bottom.

Objectives

In this section you will learn the following

- Introduction

Module 4 : Design of Shallow Foundations

Lecture 18 : Structural designs of column and footing [Section18.6 : Design by Finite Difference Method]

- **Finite Difference Method**

Find the deformation and draw bending moment diagram for the given footing as loads are acting as shown below using finite difference method.

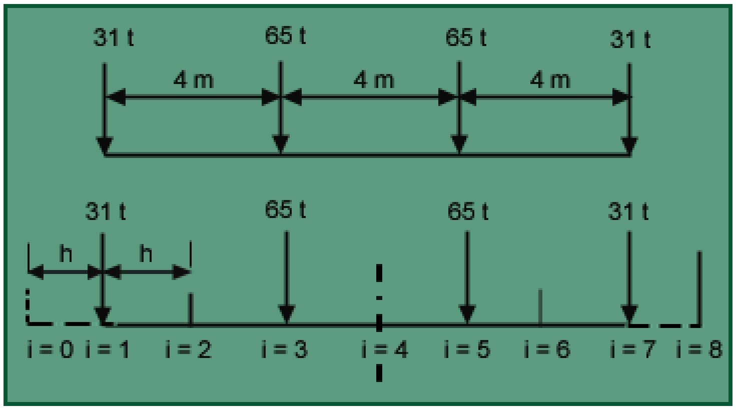


Fig 4.67 Beam divided into number of nodes

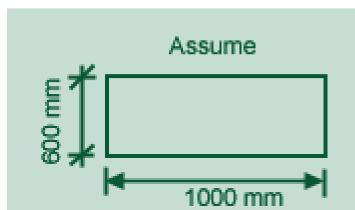
K_s = 1 Kg/cm^2 , $K_s^{'} = K_s.B = K_s \times 1 = 1 \times 100$ Kg/cm^2 = 1000 t/m^2,

h = 2 meter.

$R_1 = K_s^{'} \times y_1 \times \frac{h}{2} = 1000 \times y_1$

$R_2 = K_s^{'} \times y_2 \times h = 2000y_2$

$R_3 = K_s^{'} \times y_3 \times h = 2000y_3$



E = Econc = 5000 $\sqrt{15}$ = 1936492 t/m^2.

I = bd3/ 12 = 0.018 m^4. EI = 348656.85 t/m^2.

At Node 1

Bending moment will be zero.(condition)

$$(EI\frac{d^2y}{dx^2})_{i-1} = 0$$

$$-\frac{EI}{h^2}(y_2 - 2y_1 + y_0) = 0 \qquad \text{---------- (1)}$$

$$y_0 = 0\ (condition)$$

$$y_2 = 2y_1$$

At Node 2

$$(\mathrm{EI}\frac{d^2y}{dx^2})_{i-2} = \tfrac{3}{4}.h.R_1 - 31\times 2$$

$$\text{therefore} \quad \frac{\mathrm{EI}}{h^2}(y_3 - 2y_2 + y_1) = \tfrac{3}{4}.2.R_1 - 31\times 2$$

Substitute the value of Y_2 then we get

$$y_3 = 3.172y_1 - 7.11\times 10^{-3} \qquad \text{---------- (2)}$$

At Node 3

$$(\mathrm{EI}\frac{d^2y}{dx^2})_{i-3} = (1.75\times h)R_1 + R_2.h - (31\times 2h)$$

$$\frac{\mathrm{EI}}{h^2}(y_4 - 2y_3 + y_2) = 1.75\times 2\times 1000y_1 + 2000y_2\times 2 - 124$$

Substitute the value of Y_3 & Y_2 Then we get.....

$$y_4 = 5.664y_1 - 0.02846 \qquad \text{--------- (3)}$$

From symmetry

$$R_1 + R_2 + R_3 + \frac{R_4}{2} = (31+65)t$$

$$1000y_1 + 2000y_2 + 2000y_3 + \frac{2000y_4}{2} = 96$$

$$y_1 + 4.y_1 + 2\times(3.172y_1 - 7.11\times10^{-3}) + (5.644y_1 - 0.02846) = 0.096$$

$$y_1 = 8.14\times10^{-3}\text{meter}$$

$$y_2 = 16.28\times10^{-3}\text{meter}$$

$$y_3 = 18.7\times10^{-3}$$

$$y_4 = 17.83\times10^{-3}$$

Now Calculate the Moment at the nodal point

At Node 1

$$M = -\frac{EI}{h^2}(y_2 - 2y_1 + y_1) = 0(\text{check})$$

At Node 2

$$M = -\frac{EI}{2^2}(y_3 - 2y_2 + y_1) = +49.84 \quad t-m$$

At Node 3

$$M = -\frac{EI}{2^2}(y_4 - 2y_3 + y_2) = +28.14 \quad t-m$$

At Node 4

$$M = -\frac{EI}{h^2}(y_5 - 2y_1 + y_3)$$

$$\text{since} \quad y_5 = y_3$$

$$M = +14.12 \quad t-m$$

(Advantage due to symmetry)

Note:

Here we are not considering the portion of the raft after the load.

Difference in reinforcement is more because Finite difference method gives higher value of moment than conventional method.

Recap

In this section you have learnt the following

- Introduction

Congratulations, you have finished Lecture 18. To view the next lecture select it from the left hand side menu of the page