

## UNIT II SHALLOW FOUNDATION

### Introduction

A foundation is an integral part of the structure which transfers the load of the superstructure to the soil. A foundation is that member which provides support for the structure and its loads. It includes the soil and rock of earth's crust and any special part of structure that serves to transmit the load into the rock or soil. The different types of the foundations are given in fig. 4.1

### Different types of footings

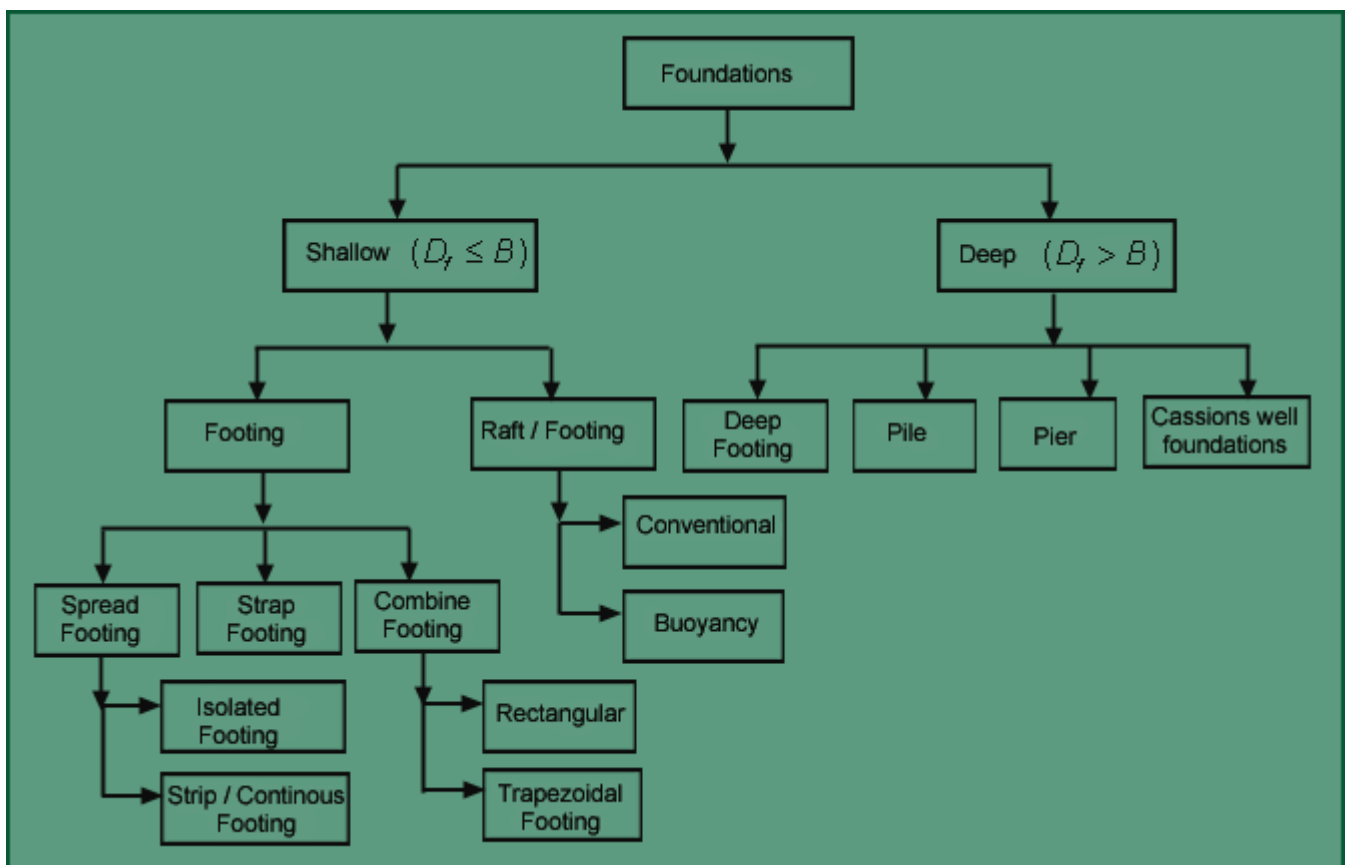


Fig. 4.1 Different types of footings

**Methods of determining bearing capacity** The various methods of computing the bearing capacity can be listed as follows:

- Presumptive Analysis
- Analytical Methods
- Plate Bearing Test
- Penetration Test

■ Modern Testing Methods

■ Centrifuge Test **Prandtl's Analysis**

Prandtl (1920) has shown that if the continuous smooth footing rests on the surface of a weightless soil possessing cohesion and friction, the loaded soil fails as shown in figure by plastic flow along the composite surface. The analysis is based on the assumption that a strip footing placed on the ground surface sinks vertically downwards into the soil at failure like a punch.

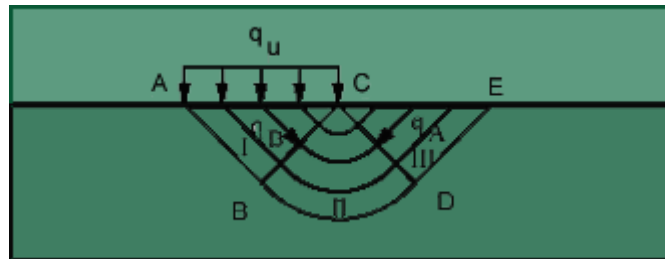


Fig 4.8 Prandtl's Analysis

Prandtl analysed the problem of the penetration of a punch into a weightless material. The punch was assumed rigid with a frictionless base. Three failure zones were considered.

- Zone I is an active failure zone
- Zone II is a radial shear zone
- Zone III is a passive failure zone identical for  $\phi = 0$

Zone I consist of a triangular zone and its boundaries rise at an angle  $45 + \phi/2$  with the horizontal two zones on either side represent passive Rankine zones. The boundaries of the passive Rankine zone rise at angle of  $45 - \phi/2$  with the horizontal. Zones 2 located between 1 and 3 are the radial shear zones. The bearing capacity is given by (Prandtl 1921) as

$$q_d = cN_c$$

where c is the cohesion and  $N_c$  is the bearing capacity factor given by the expression

$$N_c = \cot \phi e^{\pi \tan \phi} \tan^2 [(45 + \phi/2) - 1]$$

Reissner (1924) extended Prandtl's analysis for uniform load q per unit area acting on the ground surface. He assumed that the shear pattern is unaltered and gave the bearing capacity expression as follows.

$$q_d = cN_c + qN_q$$

$$N_q = e^{\pi \tan \phi} \tan^2 (45 + \phi/2)$$

$$N_c = \cot \phi e^{\pi \tan \phi} \tan^2 [(45 + \phi/2) - 1]$$

if  $\phi = 0$ , the logspiral becomes a circle and  $N_c$  is equal to  $(\pi + 2)$ , also  $N_q$  becomes 1. Hence the bearing capacity of such footings becomes

$$q_d = (\pi + 2)c + q$$

$$= 5.14c + q$$

if  $q=0$ ,

$$\text{we get } q_d = 5.14c = 2.57q_u$$

where  $q_u$  is the unconfined compressive strength.

**Terzaghi's Bearing Capacity Theory** Assumptions in Terzaghi's Bearing Capacity Theory

- Depth of foundation is less than or equal to its width.
- Base of the footing is rough.
- Soil above bottom of foundation has no shear strength; is only a surcharge load against the overturning load
- Surcharge upto the base of footing is considered.
- Load applied is vertical and non-eccentric.
- The soil is homogenous and isotropic.
- L/B ratio is infinite.

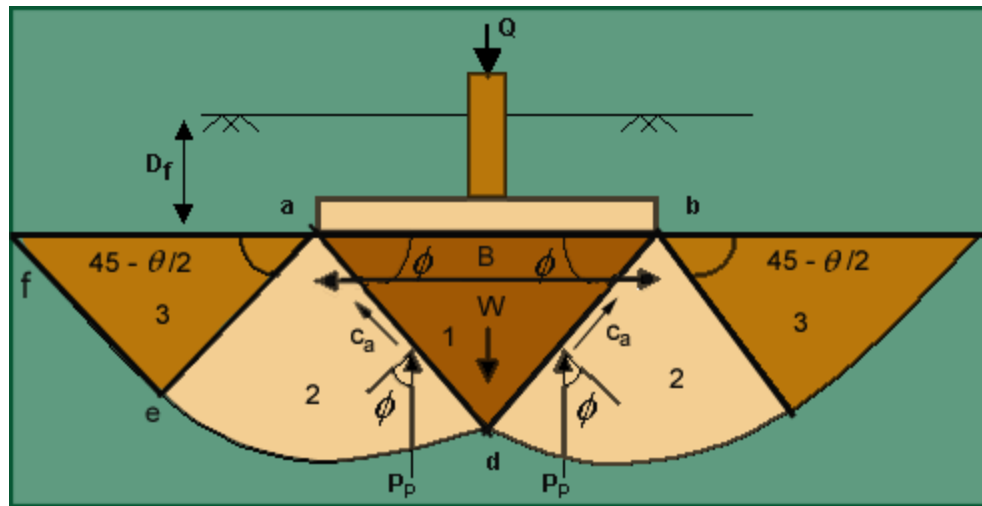


Fig. 4.9 Terzaghi's Bearing Capacity Theory

Consider a footing of width  $B$  and depth  $D_f$  loaded with  $Q$  and resting on a soil of unit weight  $\gamma$ . The failure of the zones is divided into three zones as shown below. The zone 1 represents an active Rankine zone, and the zones 3 are passive zones. The boundaries of the active Rankine zone rise at an angle of  $45 + \phi/2$ , and those of the passive zones at  $45 - \phi/2$  with the horizontal. The zones 2 are known as zones of radial shear, because the lines that constitute one set in the shear pattern in these zones radiate from the outer edge of the base of the footing. Since the base of the footings is rough, the soil located between it and the two surfaces of sliding remains in a state of equilibrium and acts as if it formed part of the footing. The surfaces  $ad$  and  $bd$  rise at  $\phi$  to the horizontal. At the instant of failure, the pressure on each of the surfaces  $ad$  and  $bd$  is equal to the resultant of the passive earth pressure  $P_p$  and the cohesion force  $C_a$ . since slip occurs along these faces, the resultant earth pressure acts at angle  $\phi$  to the

normal on each face and as a consequence in a vertical direction. If the weight of the soil  $adb$  is disregarded, the equilibrium of the footing requires that

$$Q_d = 2P_p + 2C_a \sin \phi = 2P_p + Bc \tan \phi \text{----- (1)}$$

The passive pressure required to produce a slip on  $def$  can be divided into two parts,  $P'_p$  and  $P''_p$ . The force  $P'_p$  represents the resistance due to weight of the mass  $adef$ . The point of application of  $P'_p$  is located at the lower third point of  $ad$ . The force  $P''_p$  acts at the midpoint of contact surface  $ad$ .

The value of the bearing capacity may be calculated as :

$$Q_d = 2(P_p + P_c + P_q + \frac{1}{2} Bc \tan \phi) \text{----- (2)}$$

by introducing into eqn(2) the following values:  $N_c = \frac{2P_c}{Bc} + \tan \phi$

$$N_q = \frac{2P_q}{B \cdot D_f}$$

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.

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

Design of the Column.

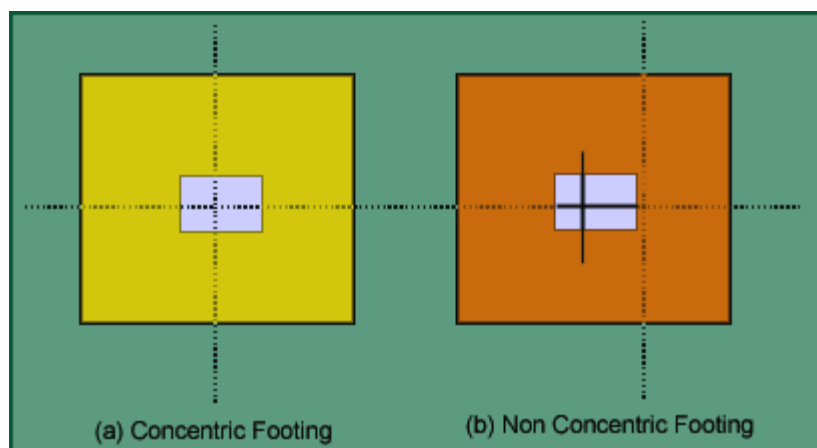


Fig. 4.26 Concentric & Non Concentric Footing

$$P_U = 40 \times 1.5 = 60 \text{t} = 60 \times 10^4 \text{N}$$

$$M_U = 1.5 \times 1000 \text{Kg} \cdot \text{m} = 1500 \times 10^4 \text{N} \cdot \text{mm} \quad \text{Trial 1 Let assume } b = 300 \text{ mm} \text{ \& } D (L) = 400 \text{ mm}$$

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

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

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

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.

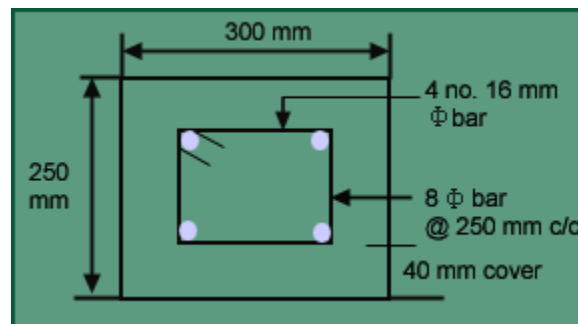


Fig -4.27 Column Section

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

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

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

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

therefore

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

$$A_{st_{required}} = \frac{0.9}{100} \times 250 \times 350 = 675 \text{ mm}^2$$

Design of footing

Size of the footing

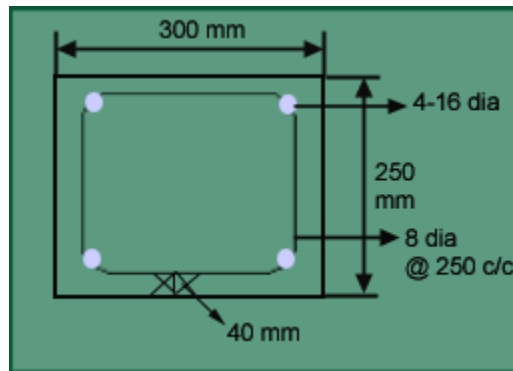


Fig 4.28 Details of the coulumn

Let D=500mm

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

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

$$V=40 \text{ t} = 40 \times 10^4 \text{ N}, e=M/V=1000 \times 10^4 / 40 \times 10^4 = 25 \text{ mm}$$

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

$$L=6e=150 \text{ mm}$$

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

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

Let provide footing size is 2.2 m\*1.0 m.

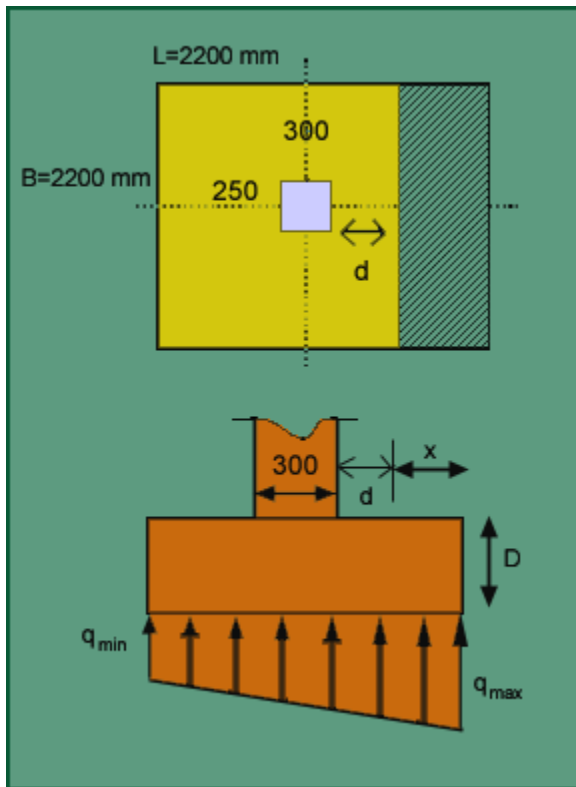
Check:

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

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

**iii Thickness of footing a. Wide beam shear**

Factored intensity of soil pressure,



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

$$= \left( 19.42 - \frac{19.42 - 16.94}{2200} x \right) 1.5 \text{ now, } V = \int_0^x q_{us} dx$$

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

$$\tau = 0.32 M_{ps} = 32t / m^2$$



2

$$V * B = \pi * B * d$$

$$0.0265d^2 + 0.86 - 0.841 = 0$$

By trial and error method,  $d = 0.45$  m

Fig 4.29 Section for wide beam shear and upward earth pressure diagram Punching shear (two way shear)

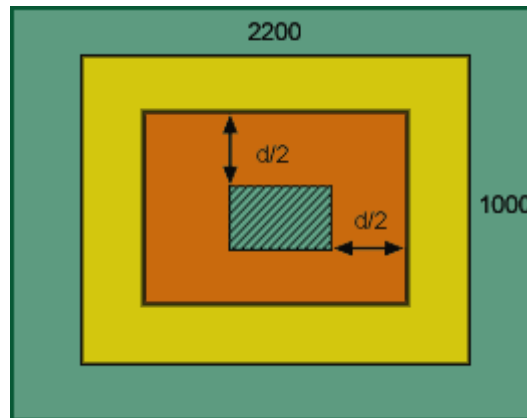


Fig 4.30 Section for two way at a distance of  $d/2$  from face of the column round

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

$$\text{Critical area} = (1.1 + 4d) d \text{ m}^2$$

$$\text{IS: 456-1978, } \beta_c = 250/300 = 0.83$$

$$K_s = (0.5 + \beta_c) = 1.33 > 1.0$$

Therefore  $K_s = 1.0$

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

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

$$P_{ult} = 40.0 \times 1.5 = 60 \text{ t/m}^2$$

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

by trial and error,  $d = 0.255$  m

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

**Flexural reinforcement**

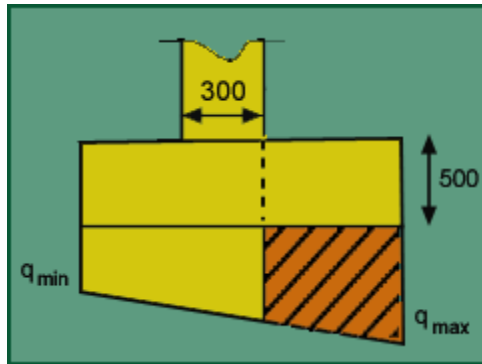


Fig 4.31 Section for bending moment

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

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

$$= 18.35 \text{ t/m}^2$$

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

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

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

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

Table I of SP-16,  $P_{treq} = 0.193\%$

For wide beam shear  $P_t = 0.2\%$

$$A_{streq} = 0.2 * 1000 * 450 / 100$$

Provide 16mm diameter torq bars @200 mm c/c in both directions.

According to clause 33.3.1 of IS: 456

$$L_d = 2.2/1 = 2.2$$

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

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

**v Check for development length**

$$\text{Clause 35.2.1 } L_d = \frac{16 \cdot \sigma_s}{4 \tau_{bd}} = \frac{16 \cdot 415}{4 \cdot 1.6} = 1037.5 \text{ mm}$$

$L_d$

Now length of bars provided,  $(2200-300)/2 = 950 \text{ mm} <$   
Provide extra development length of  $1037.5-950=87.5 \text{ mm}$  say 90 mm on side of the footing.

## VI. Transfer of load at base of column

Clause 34.4

Permissible bearing pressure,  $q_b = 0.45 \cdot 15 = 6.75 \text{ } M_{pa} = 675 \text{ t/m}^2$

$$A_1 = 1 \cdot 2.2 = 2.2 \text{ m}^2$$

$$A_2 = 0.3 \cdot 0.25 = 0.075 \text{ m}^2$$

$$\sqrt{\frac{A_1}{A_2}} = 5.42 > 2.0$$

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

$$q_{max \text{ at col base}} = \left( \frac{V}{BL} + \frac{6V_e}{BL^2} \right) \times 1.5$$

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

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

ok

## Footing subjected to eccentric loading Problem 2

Design a non-concentric footing with vertical load = 40t and moment = 2tm. Allowable bearing

capacity = 20t/m<sup>2</sup>.  $f_{ck} = 15 \text{ N/mm}^2$ .  $f_y = 415 \text{ N/mm}^2$ .

■Determination of size of column:

$$P = 40 \text{ t.} \Rightarrow P_u = 40 \cdot 1.5 = 60 \text{ t.}$$

$$M = 2 \text{ tm.} \Rightarrow M_u = 2 \cdot 1.5 = 3 \text{ tm.}$$

Trial I

Let us assume footing size  $b = 250 \text{ mm}$ ,  $D = 350 \text{ mm}$ .

$$\frac{P_u}{f_{ck} b d} = \frac{60 \times 10^4}{15 \times 250 \times 350} = 0.46 \text{ N/mm}^2$$

$$\frac{M_u}{f_{ck} b d^2} = \frac{3 \times 10^7}{15 \times 250 \times 350^2} = 0.065 \text{ N/mm}^2$$

$$\frac{d'}{D} = \frac{40 + 20/2}{350} = 0.14$$

(see chart for 0.15)

$$\frac{p}{f_{ck}} = 0.06$$

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

$$A_{st} = \frac{0.9}{100} \times (250 \times 350) = 787.5 \text{ mm}^2$$

Provide 4 nos. 16  $\phi$  bars as longitudinal reinforcement and 8  $\phi$  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 ,

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

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

Eccentricity of footing = M/P= 50mm.

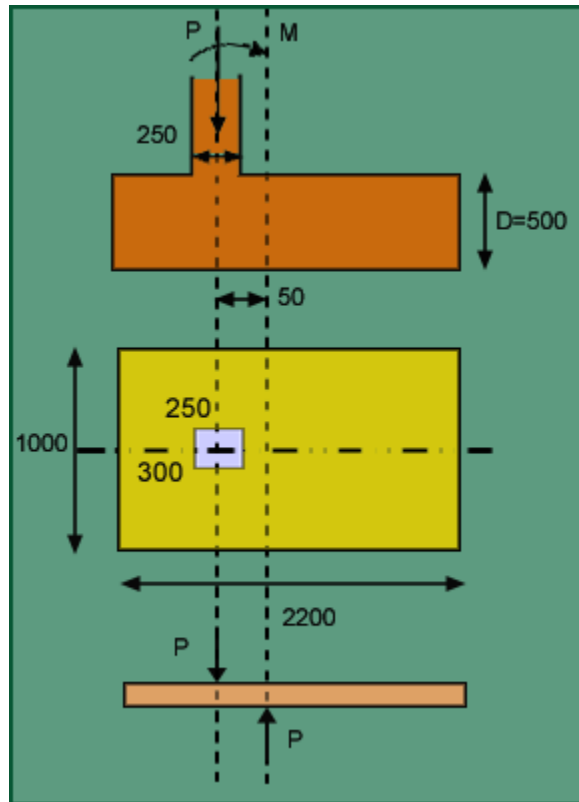


Fig. 4.32 Elevation and Plan of a non-concentric footing

■ **Determination of design soil pressure**

Soil reaction =  $P = 40t$ .

$$= 40 / (2 * 1.1) = 18.2 \text{ t/m}^2 < 20 \text{ t/m}^2$$

Therefore,  $q_{us} = 18.2 * 1.5 = 27.3 \text{ t/m}^2 = 0.273 \text{ N/mm}^2$ .

■ **Determination of depth of footing:**

a. Wide beam shear:

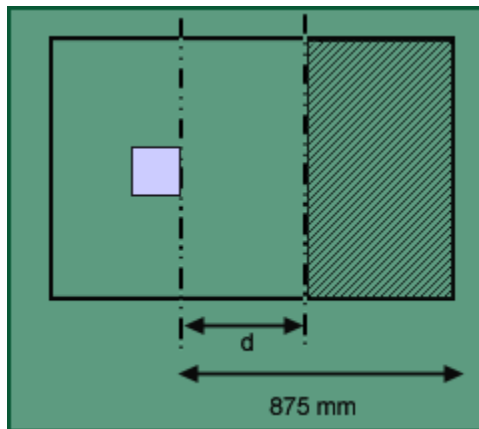
Consider a section at a distance 'd' from the column face in the longer direction.

Assuming  $P_t = 0.2\%$  for  $f_{ck} = 15 \text{ N/mm}^2$ ,  $\tau_c = 0.32 \text{ N/mm}^2$ .

$$\tau_c \cdot B \cdot d = q_{us} \cdot B \cdot (L_1 - d)$$

$$0.32 * d = 0.273 * (0.875 - d)$$

Therefore,  $d = 0.403 \text{ m}$



b. Punching shear:

Fig. 4.33 Section for wide beam shear

Critical area for punching shear:

$$= 2 * (350 + d + 250 + d) * d$$

$$= 4d(300 + d)$$

Clause :31.6.3.1 (IS 456:2000)

$$A_c = 0.25 / 0.35 = 0.71$$

$$K_s = 0.5 + A_c = 1.21 > 1.0$$

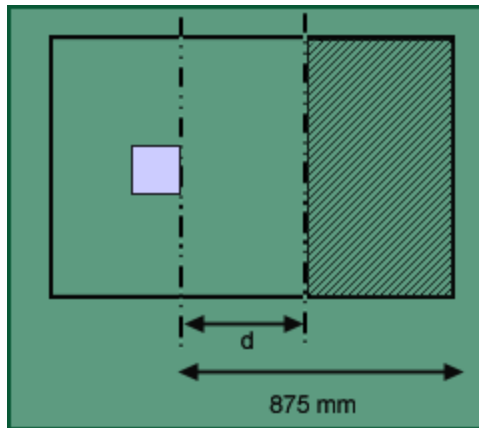
Therefore, take,  $K_s = 1.0$ .

$$\tau_c = 0.25 * (15) / 0.5 = 0.968 \text{ N/mm}^2$$

$$\tau' = \tau \cdot K_s = 0.968 \text{ N/mm}^2$$

$$96.8 * 4d * (0.3 + d) = 60 - 27.3 * (0.35 + d) * 8(0.25 + d)$$

$$d = 0.246 \text{ m.}$$



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

Fig. 4.34 Section for wide beam shear

403 mm. D required is  $(403 + 40 + 20/2) = 453 \text{ mm} < 500 \text{ mm}$  (D provided). OK.

#### Flexural reinforcement:

Design soil pressure  $(q) = 27.3 \text{ t/m}^2$

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

$$M_u = 27.3 * 0.875^2 / 2 = 10.45 \text{ tm/m width.}$$

d provided = 450 mm.

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

For singly reinforced section, table 1, SP-16,  $p_t = 0.147 \text{ N/mm}^2$

$$\frac{0.147}{100} * (1000 * 450) = 661.5 \text{ mm}^2 / \text{m width.}$$

Area of steel required =

Spacing using 16  $\phi$  bars =  $201 * 1000 / 661.5 = 303 \text{ mm c/c.}$

Provide 16 F bars as longitudinal reinforcement @ 300 mm 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 =  $661.5 * 2 = 1323 \text{ mm}^2$

Area of steel in the central band =  $2 / (1.82 + 1) * 1323 = 938 \text{ mm}^2$   
Spacing = 207.6 mm.

Provide 16  $\phi$  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)

$$L_d = \frac{\phi \sigma_s}{4 \tau_{\phi d}} = \frac{16 \times 415}{4 \times 1.6} = 1037.5 \text{ mm.}$$

Now, length of bars provided =  $(2000 - 350)/2 = 825 \text{ mm.} < L_d$ .

Extra length to be provided =  $(1037.5 - 825) = 212.5 \text{ mm.}$

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 =  $\{350 + 2*500*2\} = 2350 \text{ mm} > 2000 \text{ mm.}$

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

$$A_1 = 2 * 1.1 = 2.2 \text{ m}^2.$$

$$A_2 = 0.25 * 0.35 = 0.0875 \text{ m}^2$$

$\ddot{O} (A_1 / A_2) = 5.01 > 2.0$ . Take as 2.0.

$$q_{perm.} = q_b * \ddot{O} (A_1 / A_2) = 675 * 2 = 1350 \text{ t/m}^2.$$

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

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.

### Design of strap footing: Example:

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

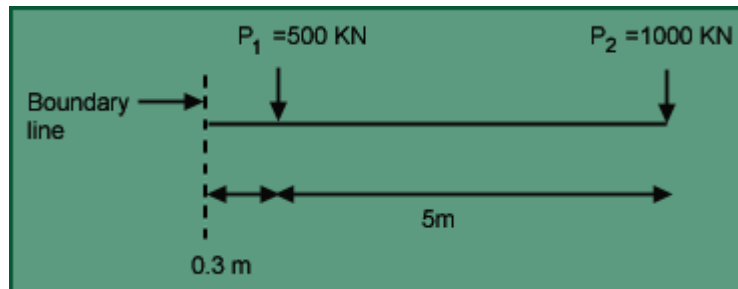


Fig. 4.35 Strap footing

#### ■ Design of the column Column A:

$$P_u = 750 \text{ KN}$$

Let  $P_t = 0.8\%$ , so,  $A_s = 0.008A$  and  $A_c = 0.992A$ ,

Where, A is the gross area of concrete.

As per clause 39.3 of IS 456-2000,

$$750 \times 103 = (0.4 \times 15 \times 0.992A) + (0.67 \times 415 \times 0.008A)$$

$$A = 91727.4 \text{ mm}^2$$

Provide column size (300 x 300) mm

$$\therefore 750 \times 103 = 0.4 \times 15 \times (1 - (p_t/100)) \times 90000 + 0.67 \times 415 \times (P_t/100) \times 90000$$

$$\therefore P_t = 0.86\%$$

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

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

Column B:

$$P_u = 1500 \text{ KN}$$

Provide column size (400 x 400) mm

$$\therefore 1500 \times 103 = 0.4 \times 15 \times (1 - (P_t/100)) \times 160000 + 0.67 \times 415 \times (P_t/100) \times 160000$$

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

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

### Footing design



Let us assume eccentricity  $e = 0.9\text{m}$ .

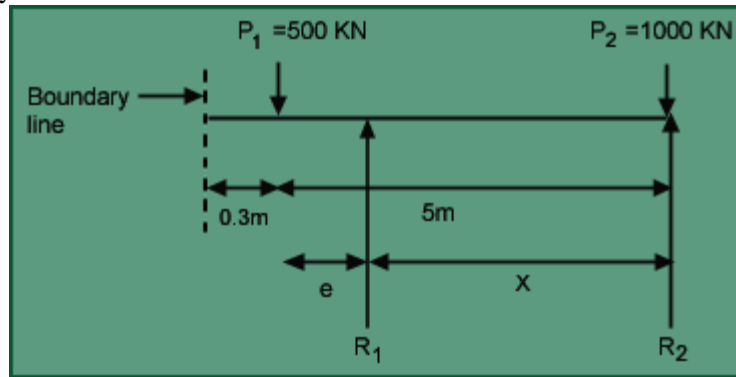


Fig. 4.36 Strap footing – soil reaction

Taking moment about line  $P_2$ ,

$$P_1 \times 5 - R_1 \times (5 - e) = 0$$

$$\therefore R_1 = \frac{5 \times 500}{5 - 0.9} = 609.8 \text{ kN}$$

$$\therefore R_2 = P_1 + P_2 - R_1 = 500 + 1000 - 609.8 = 890.2 \text{ kN}$$

Footing size:

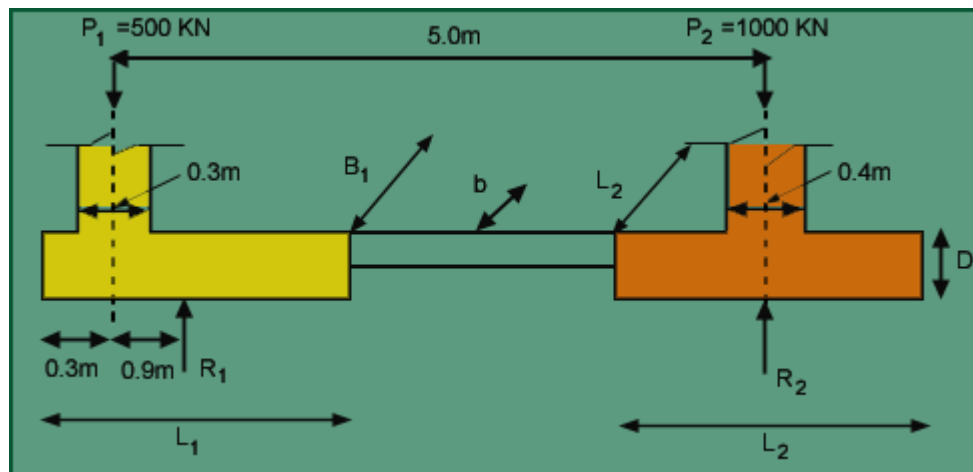


Fig. 4.37 Footing sizes

For footing A:

$$L_1 = 2(0.9 + 0.3) = 2.4\text{m}$$

Assume overall thickness of footing,  $D = 600\text{mm}$ .

$$B_1 = \frac{R_1}{(q_2 - (\gamma - \gamma_s)D)L_1} = \frac{60.98}{(20 - (2.5 - 1.8)0.6)2.4} = 1.298\text{m}$$

For footing B:

$$\text{Assume square footing of size } L_2 = \frac{89.02}{(q_a - (\gamma_c - \gamma_s)D)L_1} = \frac{89.02}{(20 - (2.5 - 1.8)0.6)L_1}$$

$$L_2 = 2.13\text{m}$$

Provide (2.2 x 2.2)m footing.

**Analysis of footing**

$$q_{u1} = \frac{R_1 \times 1.5}{L_1 \times 1\text{m}} = \frac{60.98 \times 1.5}{2.4 \times 1.0} = 38.1125 \text{ t/m}$$

$$q_{u2} = \frac{R_2 \times 1.5}{L_2 \times 1\text{m}} = \frac{89.02 \times 1.5}{2.2 \times 1.0} = 60.695 \text{ t/m}$$

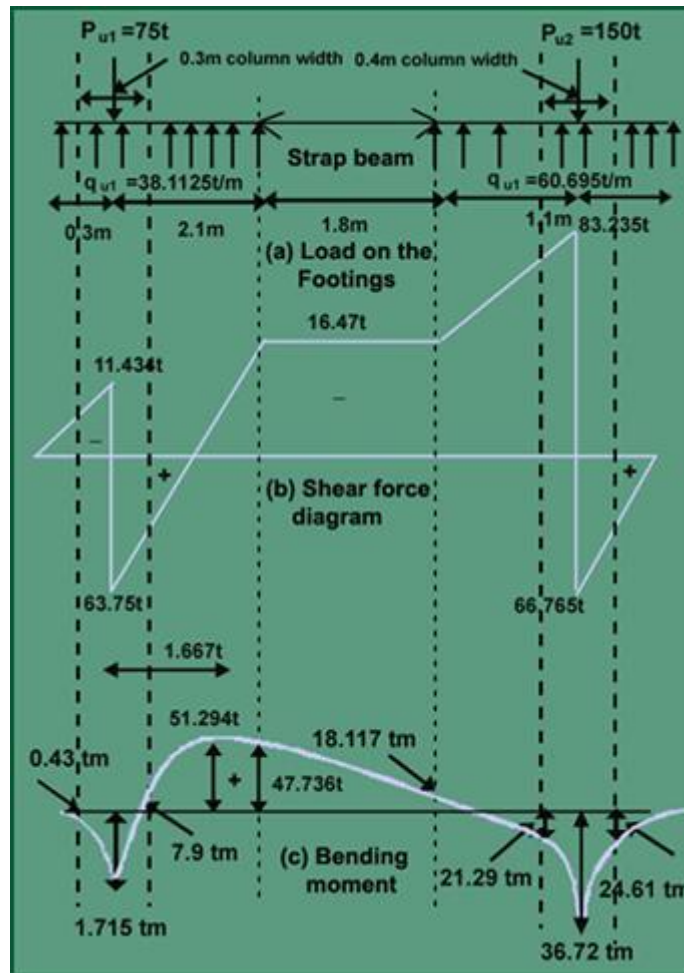


Fig. 4.38 Analysis of footing

**Thickness of footing** i) Wide beam shear: For footing A:

Let us assume  $F_t = 0.2\% B_1$  so from table 16 of IS456,  $\tau_c = 0.32 \text{ N/mm}^2 = 32 \text{ t/m}^2$

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

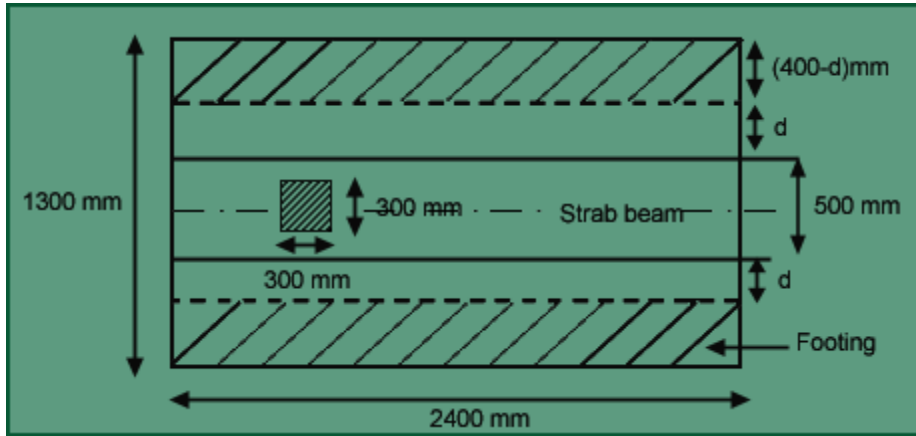


Fig. 4.39 Wide beam shear for footing A

$$\text{Shear} = b d \tau_c = q_u (0.4 - d)$$

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

$$\therefore d = 0.282 \text{ m}$$

For footing B:

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

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

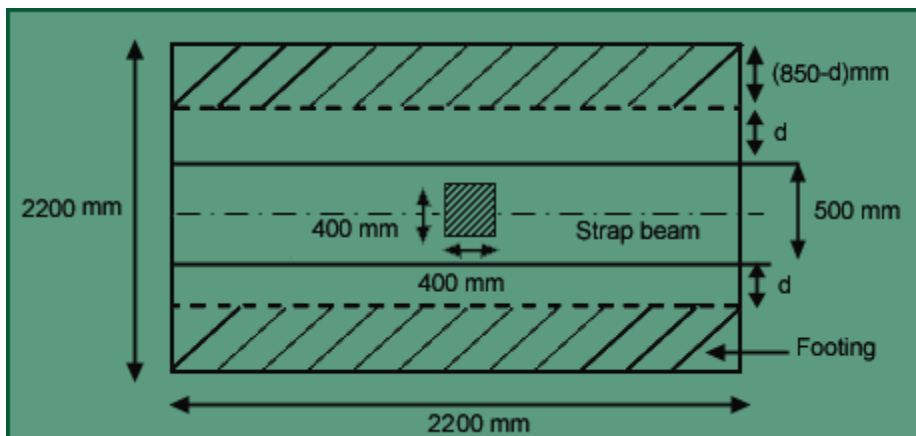


Fig. 4.40 Wide beam shear for footing B

$$\text{Shear} = b d \tau_c = q_u (0.4 - d)$$

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

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

∴ Increasing the width of the beam to 700 mm

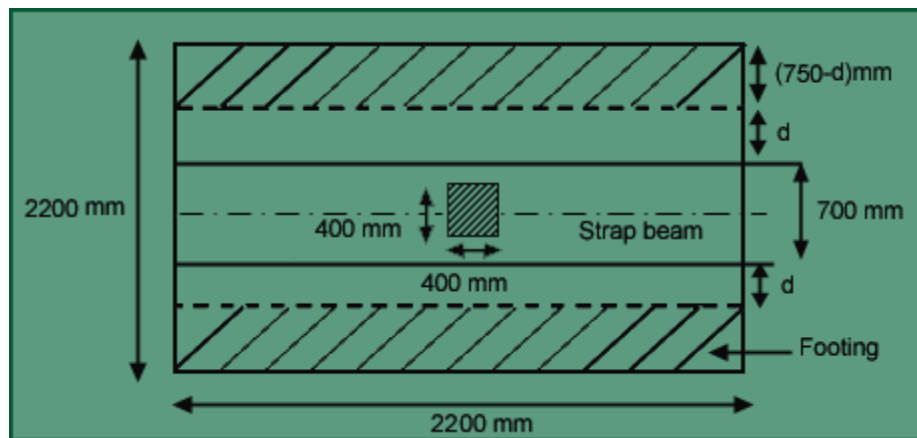


Fig. 4.41 Wide beam shear for footing B

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

$$\text{Shear} = b d \tau_c = q_u (0.75 - d)$$

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

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

∴ Safe

ii) Two way shear: For column A:

From clause 31.6.3.1 of IS456-2000.

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

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

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

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

Critical perimeter  $\times d \times \tau_c = P_u - q_u \times$  (critical area – dotted area in fig. 4.42)

So, shear equation becomes,

Critical perimeter  $\times d \times \tau_c = P_u - q_u \times$  (critical area – dotted area in fig. 4.42)

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

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

$$d = 0.246 \text{ mm} < 600 \text{ mm.}$$

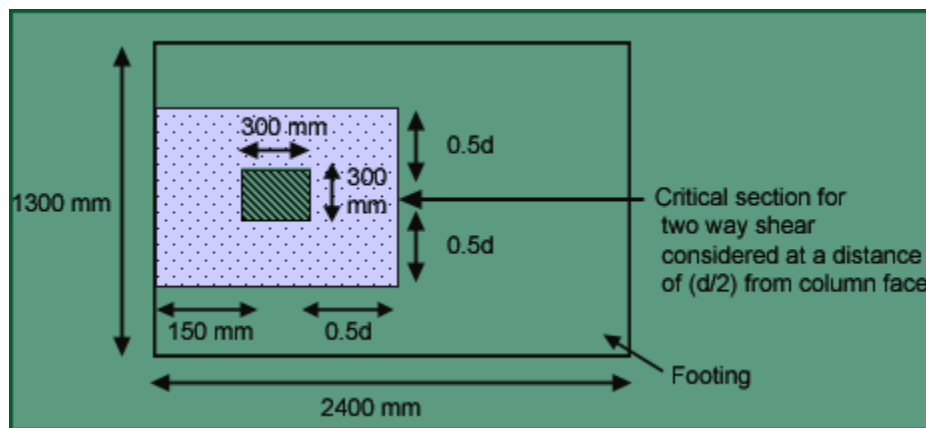


Fig. 4.42 Wide beam shear for footing A

For column B: From clause 31.6.3.1 of IS456-2000.

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

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

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

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

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

So, shear equation becomes,

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

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

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

$d = 0.355 \text{ mm} < 600 \text{ mm}$ . Among all the required  $d$  values (for wide beam shear and two way shear criteria),

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

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

So, provide  $D = 600 \text{ mm}$

$$\therefore d_{provided} = 550 \text{ mm}$$

**Reinforcement for flexure for footings (i) Design along the length direction:** Comparing the moments at the column faces in both the footings (A & B),

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

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

From table 1 of SP-16,  $P_t = 0.242 \%$  (ii) Design along the width direction:

$$q_{u1} (=38.1125 \text{ t/m}) < q_{u2} (=60.695 \text{ t/m})$$

So, for design along width direction footing B ( $q_{u2}$ ) is considered.

$$M_u = \frac{60.6955 \times (0.75)^2}{2} = 17.1 \text{ tm} < M_u \text{ in longer direction (24.61 tm)}$$

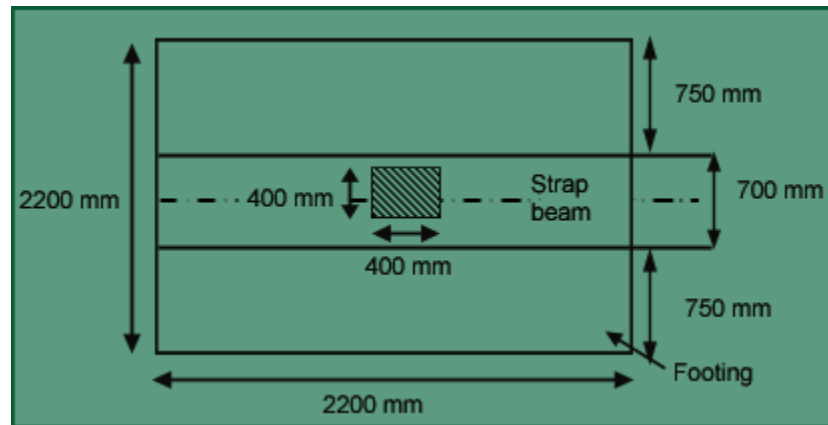


Fig. 4.44 Bending along the width of footing B

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

But, From wide beam criteria  $P_t = 0.3 \%$ ,

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

$\therefore$  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 flexure:**

$$\frac{M_u^{\max}}{bd^2} = \frac{51.294 \text{ tm}}{700 \times (550)^2} = 2.43 \text{ N/mm}^2 \quad (\text{Refer fig. 4.45})$$

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

$$\therefore A_{st} = 0.83 \% \text{ and } P_c = 0.12 \%$$

$$\therefore A_x \text{ (required on tension face)} = (0.83/100) \times 700 \times 550 = 3195.5 \text{ mm}^2,$$

$$\therefore A_x \text{ (required on compression face)} = (0.12/100) \times 700 \times 550 = 462 \text{ mm}^2,$$

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:

$$V_{\max} = 83.235 \text{ t}$$

$$\tau_{\text{acting}} = \frac{V_{\max}}{bd} = \frac{83.235 \times 10^4}{700 \times 550} = 2.162 \text{ N/mm}^2 < \tau_{\text{max}} = 2.5 \text{ N/mm}^2 \text{ (for M15)}$$

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

From table 61 of SP-16,  $\tau_c = 0.57 \text{ N/mm}^2$

But, provide shear reinforcement for shear =  $(\tau_{\text{acting}} - \tau_c) = 1.592 \text{ N/mm}^2 = V_{us}$

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

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

∴ From table 62 of SP-16, provide 4L-stirrups 10 Tor @ 100 c/c near the column (upto distance of  $d=550\text{mm}$  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,

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

For column A:

$$\text{Length of the bar provided} = 150-40 = 110 \text{ mm} < L_d$$

∴ By providing 2 no.s 90° bend the extra length to be provided =  $(1297-110-3(8 \times 20)) = 707 \text{ mm}$ .

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

∴ Providing two 90° bend, the extra length to be provided =  $(1297-460-2(8 \times 20)) = 517 \text{ mm}$ .

(ii) Check for shear:

$$V_{max} = 83.235 \text{ t} = \frac{83.235 \times 10^4}{700 \times 550} = 2.162 \text{ N/mm}^2$$

$$P_t = \frac{11 \times 314}{700 \times 550} \times 100 = 0.897\% < \text{max} = 2.5 \text{ N/mm}^2 \text{ (for M15)}$$

(provided) =  $\pi$

From table 61 of SP-16,  $\sigma_s = 0.57 \text{ N/mm}^2$

$$\frac{V_{us}}{d} = (\sigma_s - \sigma_c) b = (1.592)(700) = 1114.4 \text{ N/mm} = 11.144 \text{ KN/cm}$$

$$= 11.144 \text{ KN/cm}$$

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

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,

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

For column A:

$$\text{Length of the bar provided} = 150 - 40 = 110 \text{ mm} < L_d$$

∴ By providing 2 nos 90° bend the extra length to be provided =  $(1297 - 110 - 3(8 \times 20)) = 707 \text{ mm}$ .

$$\frac{1300 - 300}{2} - 40 = 460 \text{ mm} < L_d$$

In B direction length of the bar provided =

∴ Providing two 90° bend, the extra length to be provided =  $(1297 - 460 - 2(8 \times 20)) = 517 \text{ mm}$ .

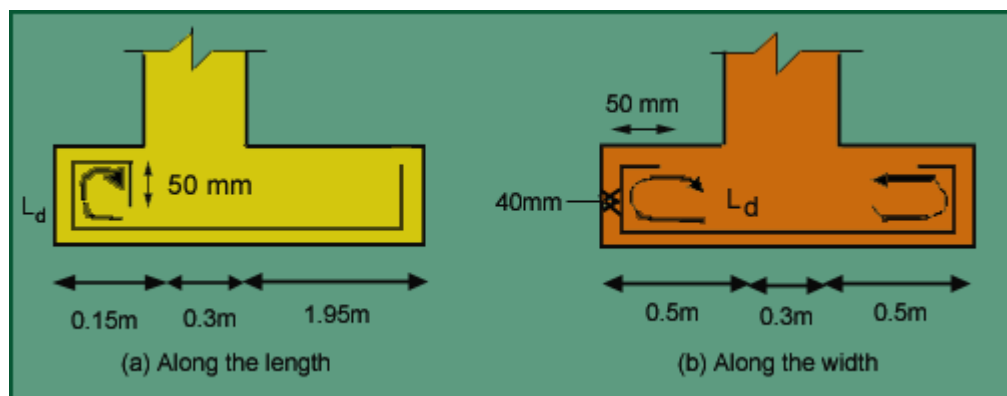


Fig. 4.45 Development length for footing A

For column B:



Length of the bar provided =  $\frac{2200 - 400}{2} - 40 = 860 \text{ mm} < L_d$   
 Providing one  $90^\circ$  bend, the extra length to be provided =  $(1297 - 860 - (8 \times 20)) = 277 \text{ mm}$ .

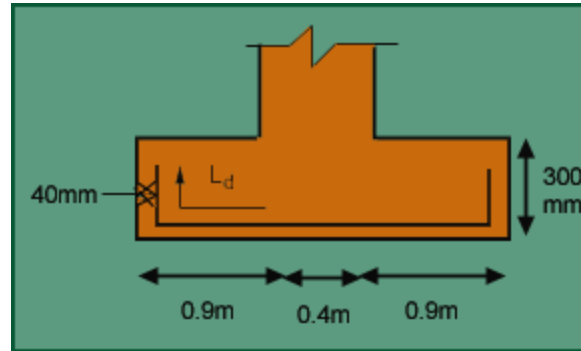


Fig. 4.46 Development length for footing B (Along the length and width)

**Transfer of load at base of the column:** For footing A:

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

$$A_1 = (150 + 300 + 1200)(1300) = 2145000 \text{ mm}^2$$

$$A_2 = (300 \times 300) = 90000 \text{ mm}^2$$

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

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

$$q_{per} = 2 \times 0.45 \times 1500 = 1161 \text{ t/m}^2$$

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

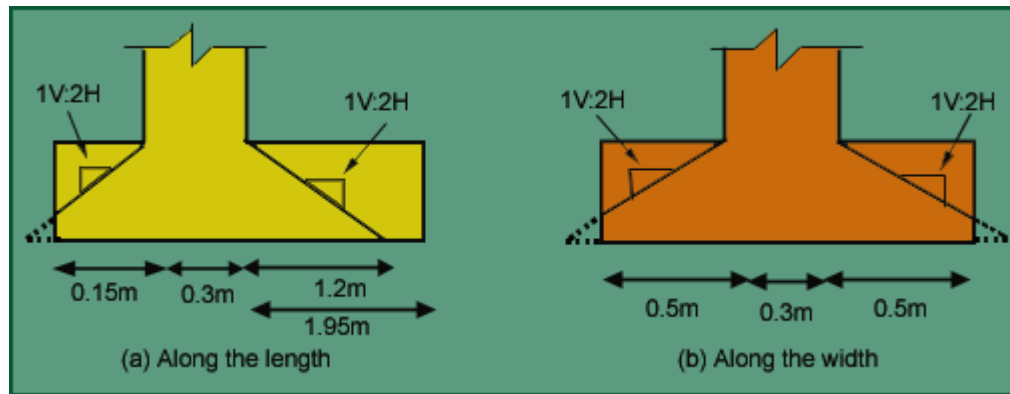


Fig. 4.47 Area of footing A considered for check of transfer of load at column base

For Footing B: From clause 34.4 of IS456-2000, permissible bearing stress ( $q_{per}$ ) =  $\sqrt{\frac{A_1}{A_2}} (0.45 f_{ck})$

$$A_1 = (2200)^2 = 4840000 \text{ mm}^2$$

$$A_2 = (400 \times 400) = 160000 \text{ mm}^2$$

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

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

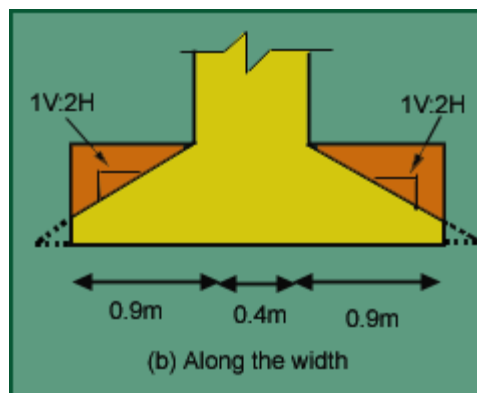


Fig. 4.48 Area of footing B considered for check of transfer of load at column base

$$q_{per} = 2 \times 0.45 \times 1500 = 1161 \text{ t/m}^2$$

$$q_{allow} = (\text{load on column}/\text{area of column})$$

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

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