UNIT 3 FOOTINGS

Structure

3.1 INTRODUCTION

This unit seeks to introduce you to structural design of different types of footings and is fundamental practice of foundation design. The selection of different types of footings and the need for reinforcing the same are discussed in this unit. Methods of designing plain concrete wall footings are described together with detailed structural design of reinforced concrete wall, square and rectangular footings. Proportioning of plan dimensions of rectangular, trapezoidal and strap footings and eccentrically loaded footings are described with examples. Procedure of structural design of combined footings and rafts have also been indicated.

Indian Standards 456 (1978) describes the **Limit State Design** of reinforced concrete. Practicing engineers adopt this method of design and it has become necessary for educational institutions to switch over to teaching the Limit State Design. Hence Limit State Design is adopted in this unit. The following publications of the Bureau of Standards shall be read along with this unit.

1) IS 456 (1978): *Code of Practice for Plain and Reinforced Concrete*
2) SP 16 (1980): *Design Aids* to IS 456 (1978)

The theoretical treatment of Limit State Design is specialised field and is beyond the scope of this unit. The relevant formulae and tables are given so as to obtain the desired design values.

Objectives

After studying this unit, you should be able to:

- identify different types of footings and the situation where each can be used,
- design plain and reinforced concrete wall, square and rectangular footings, and
- calculate the shear forces and moments for the design in the case of eccentrically loaded footings and combined footings.

3.2 TYPE OF FOOTINGS

Footings are generally the lowermost supporting part of the structure known as sub-structure and are the last structural elements through which load is transferred to foundation comprising soil/rock. Structural elements transfer the applied loads from one part of the building to the other. These are in turn transmitted to the foundation which transfers it to the underlying soil/rock. The footing transmit the superimposed loads on the foundation so that the safe bearing capacity of the underlying foundation material, is not exceeded. Our discussions in this Unit will be limited to different type of footings and mat foundation shown in Figure 3.1

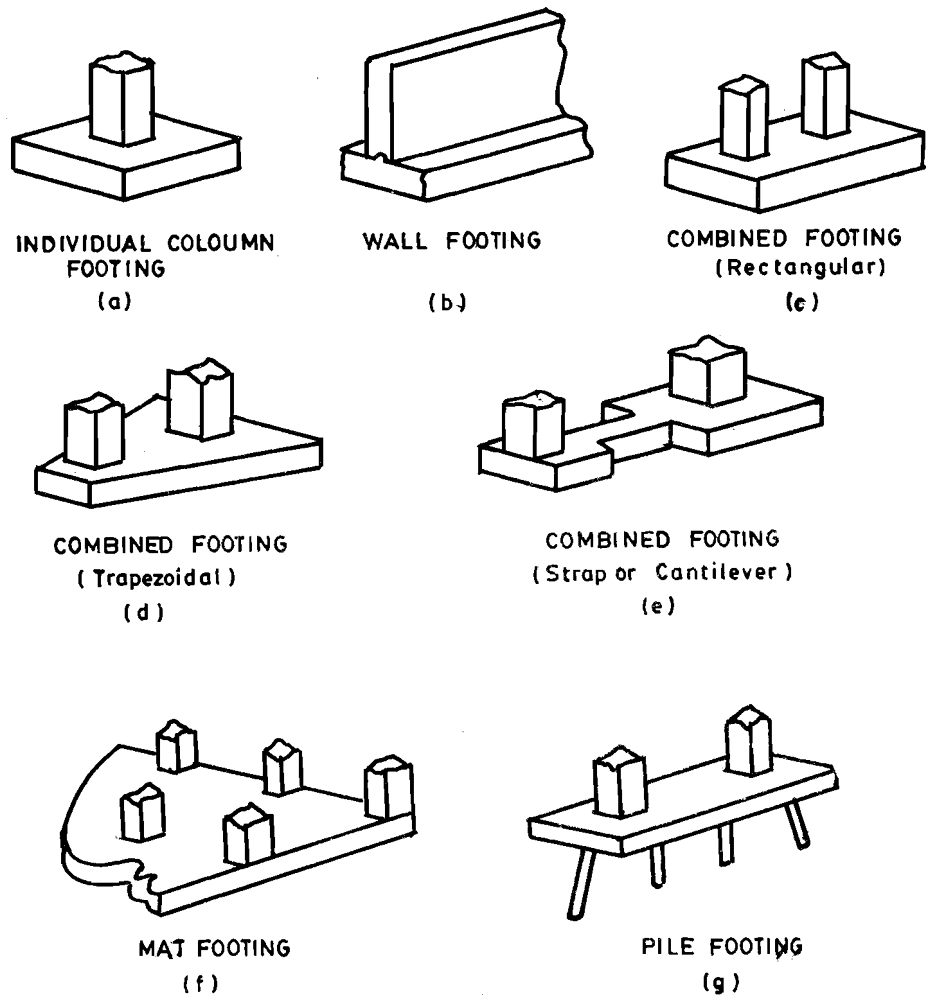


Figure 3.1: Types of Footings

The type of footings commonly used in design can be categorized as under:

Individual Column Footings: This type of footing shown in Figure 3.1(a) is usually square in shape and is also termed as isolated spread footing. They also may be rectangular due to space limitations in some cases.

Wall Footings: This type of footings is shown in Figure 3.1(b). Wall footing is also termed as strip footing to support walls. The walls may be in load bearing or non-load bearing.

Combined Footings: This type of footings are shown in Figures 3.1(c) and (d) and usually support two or more columns. They may be either rectangular or trapezoidal in shape. If two isolated footings are joined by a strap beam, the foundation structure is called a strap footing or cantilever footing as shown in Figure 3.1(e).

Mat or Raft Foundations: Mat or raft foundation as shown in Figure 3.1(f) is large continuous footings which support all columns and walls of a structure. They are usually preferred when spacing of individual footings workout to be very close and size of footing exceed 2 m to 3 m.

Pile Foundations: Pile foundation as shown in Figure 3.1(g) is used to transmit heavy column loads to a group of piles joined at top by a pile cap. The piles transmit the structural loads to the underlying soil through friction and bearing.

This type of foundation system is usually adopted when the material below footing is too weak to support the structure and it becomes essential to transfer loads to better strata underlying weaker strata. This type of foundation is very expensive.

3.3 SELECTION OF TYPE OF FOUNDATION

How do you decide on the type of foundation to be used for a structure? Two important factors to be considered in decision making are the safe bearing capacity of the soil and the imposed loads. Generally, if the plan area of all the footings exceed 50% of the plan area of the building, footing foundations will not be economical and raft foundation is preferred.

An example will be helpful in illustrating the process of selection of the type of foundation footing, raft or pile.

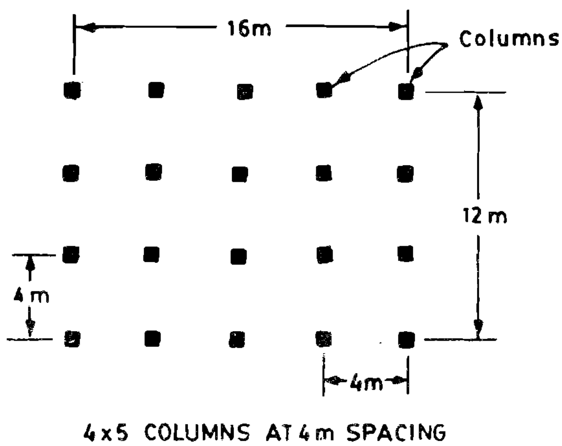


Figure 3.2: Column Arrangement

Example 3.1

A building of plan area 12 m × 16 m is to be constructed. The load is supported by 20 columns arranged in a 4 × 5 array with columns spacing as 4 m as shown in Figure 3.2. The safe bearing capacity of soil is 100 kPa.

Solution

Case 1

The building has two storeys and load coming on each column is 100 kN.

If footing foundations are to be used,
Area of footing under each column = 100/100 = 1 m^2

So a square footing of 1.0 m × 1.0 m will be adequate.

Total area occupied by 20 footings = 20 × 1 = 20 m^2

Percentage of area occupied by the footings = $\frac{20}{16 \times 12} \times 100 = 10.4$ less than 50%.

So footing foundations will be economical.

Case 2

The building has ten storeys and the load coming on each column is 500 kN.

Area of footing under each column = 500/100 = 5 m^2

A square footing 2.25 m × 2.25 m will be adequate.

Total area occupied by footings = 20 × 5 = 100 m^2

$$\text{Percentage of area occupied by footings} = \frac{100}{192} \times 100 = 52\%$$

This is slightly more than 50% of the plan area. So raft foundation may be selected.

Case 3

The building has 20 storeys and the load coming on each column is 1000 kN statements. Area of footing under each column = 1000/100 = 10 m^2. Total area occupied by footings = 20 $\times$ 10 = 200 m

The area is greater than the plan area of the building itself. So raft foundation is not feasible. Pile foundation will be a possible solution.

You can see that the above example is very much simplified. You know that the selection of foundation type depends on a number of factors such as site and soil conditions, type of structure and loading, permissible total and differential settlements etc. Experience plays very important role in the final choice. You may have to prepare a few alternate designs for comparison and select the most suitable type among them.

SAQ 1

i) Tall buildings of fifty storeys are now being constructed. What is the type of foundation used for such structures ?

ii) You want to build a bridge across a river. What is the type of foundation generally used for bridges in India?

3.4 NEED FOR REINFORCEMENT

Concrete is economical and durable and is now universally used for footings. Plain concrete is adequate to carry relatively light loads. The minimum plan dimensions are decided by the safe bearing capacity. Usually, the load distribution through the footing is assumed to be independent of the material and to occur at 45° as shown in Figure 3.3.

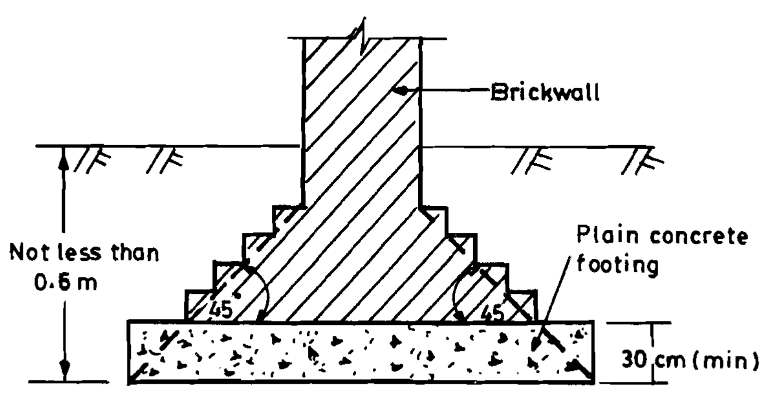


Figure 3.3 : Plain Concrete Footing for a Brick Wall

The thickness of the footing must be sufficient so that column loads can be spread through them at 45° dispersion to give uniform bearing pressure less than allowable bearing capacity. Stepped or sloped footings are most commonly used to reduce the concrete away from the column where the bending moment and shear forces are small.

As the load increases, the thickness of the footing increases. Quantity of concrete increases and deeper excavations have to be made. Beyond a certain thickness, you may find it more economical to use reinforcement in the footing than plain concrete. The thickness of the footing will then be reduced. Also excavation will be less.

Let us work out an example to illustrate this.

Example 3.2

For the two-storey building in Example 3.1, the size of the footing was 100 cm× 100 cm.

Let the column size be 30 × 30 cm.

Assuming a 45° dispersion, the thickness of plain concrete footing as shown in Figure 3.4

$$= (50 - 15) = 35 \text{ cm.}$$

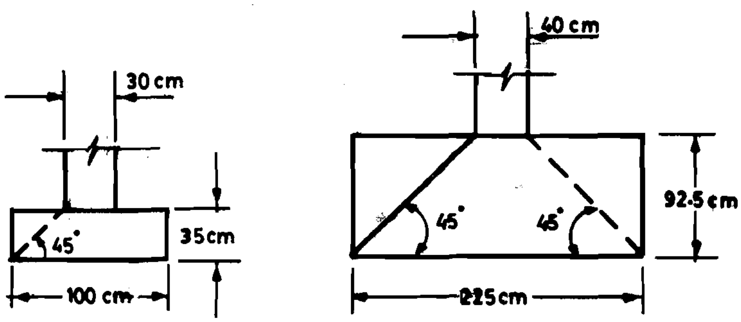


Figure 3.4 : Thickness of Plain Concrete Footings

For the ten-storey building in example the footing size is 225 cm × 225 cm.

Let the column size is 40 cm × 40 cm.

Thickness of plain concrete footing = 112.5 – 20 = 92.5 cm.

You can use stepped footings to reduce the amount of concrete. But by using reinforcement in the footings the thickness of concrete footing can be considerably reduced. This will be illustrated in subsequent examples.

3.5 WALL FOOTINGS

Generally wall footings are required to support direct concentric loads. However, an exception is the wall footings for retaining walls when they have to resist eccentric inclined loads. A wall footing may be of either plain or reinforced concrete.

3.5.1 Plain Concrete Footing

Footings carrying relatively light loads are constructed with plain concrete. You have already seen a plain concrete footing for a brick wall in Figure 3.3. The required width of the footing in the field is normaly achieved by assuming a load dispersion of 45 ° as indicated in Figure 3.3.

Angle of Dispersion

Indian Standards (IS) has specified relation for determining the angle of dispersion. According to IS the angle of dispersion of load α within the footing is taken as a function of the ratio between strength of concrete f_{ck} and the imposed stress on the soil, q.

$$\tan \alpha \nless 0.9 \left(\frac{100q}{f_{ck}} + 1 \right)^{1/2} \qquad \ldots (3.1)$$

Under this condition there will not be any tension at the base. Steel reinforcement is not necessary. It is to be noted that tension is not permitted at the base.

Example 3.3

A plain concrete wall footing is to support a 300 mm thick concrete block masonry wall. Dead load including the self weight of the wall is 150 kN/m and the live load is 300 kN/m. The allowable bearing capacity of the soil is 250 kPa. The weight of earth is 16.8 kN/m^3. The footing is to be placed 1.20 m below the ground level. Assume M20 grade of concrete (Figure 3.5).

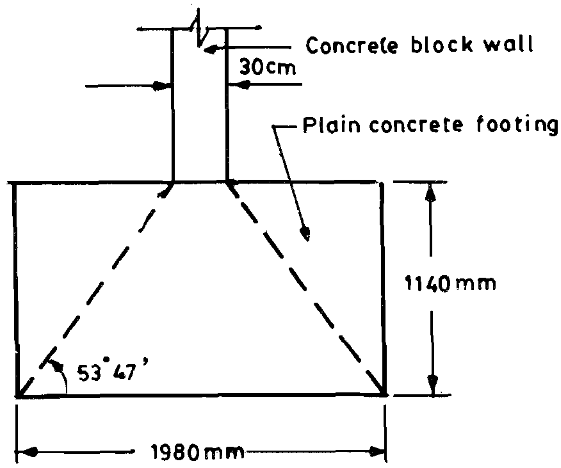


Figure 3.5 : Plain Concrete Footing

Solution

The self weight of the footing and the weight of earth above footing is generally assumed to be 5 to 10% of the applied load.

Assuming self weight of the footing and weight of earth, above footing as 10% of the imposed load.

$$W = 450 \times 0.1 = 45 \text{ kN}$$

Total Load = 150 + 300 + 45 = 495

Required plan area of the footing $= \dfrac{450 + 45}{250} = 1.98 \text{ m} \times 1.0 \text{ m}$

Width of footing = 1.98 m.

Imposed stress, $q = \dfrac{495}{1.98} = 250 \text{ kPa} = 0.25 \text{ N/mm}$

The ratio $f_{ck}/q = \dfrac{20}{0.25} = 80$

$$\tan \alpha = 0.9 \left(\frac{100q}{f_{ck}} + 1 \right)^{1/2}$$

So angle of dispersion a = 53o 47′

Length of projection = 1/2 (1980 – 300) = 840 mm^2

Thickness of footing = 840 × 1.35 = 1134 mm (Figure 3.5)

$$\text{Transfer stress} = \frac{450 \times 10^3}{1000 \times 300} = 1.5 \text{ N/mm}^2$$

Allowable transfer stress = $0.45 f_{ck} = 0.45 \times 20 = 9 \text{ N/mm}^2$.
So Satisfactory.

SAQ 2

i) In the Example 3.3 what will be the thickness of the footing, if the angle of dispersion of load is assumed as 45° ?

ii) Calculate the ratio $\dfrac{q}{f_{ck}}$ for angle of dispersion to be 45° using IS formula. f_{ck}

iii) Calculate the magnitude of q in kPa for M 20 grade concrete for angle of dispersion to be 45° by IS formula.

Bending Stresses

You can also design the plain concrete footing based on bending moment and shear considerations. In wall footings, bending is only in one direction. So it is designed as a cantilever beam fixed at the wall and the loading is in an upward direction due to soil pressure. The allowable flexural stress is assumed to be 1.25 N/mm^2 for M 20 grade concrete. Generally, shear stresses need not be considered due to relatively large thickness of the footing.

Let us now redesign the footing taking into account flexural stresses.

Example 3.4

The self weight of the footing and the weight of earth can also be computed by assuming a footing thickness.

Solution

Let us assume a footing thickness of 90 cm.

The imposed stress due to footing, assuming a unit weight of concrete as

$$24 \text{ kN/m}^3 = 24 \times 0.90 = 21.6 \text{ kPa}$$

The bottom of the footing is 1.2 m below ground level. So there is 0.3 m of thickness of soil layer above the foundation.

Imposed stress due to weight of soil on top of the footing = 0.3 × 16.8 = 5.04 kPa

Net allowable soil pressure for service loads = 250 – 21.6 – 5.04 = 223.4 kPa.

Total load = 150 + 300 = 450 kN/m

$$\text{Width of footing} = \frac{450}{223.4} = 2.01 \text{ m}$$

For calculating the bending moment factored loads have to be used. However the weight of footing and soil above the footing need not to be considered.

$$\text{Design load} = 1.5 \text{ DL} + 1.5 \text{ LL} = 1.5\,(150 + 300) = 675 \text{ kN/m}$$

$$\text{Design soil pressure} = \frac{675}{2.01 \times 1} = 335.8 \text{ kPa}$$

According to IS, the critical section for bending moment for concrete block masonry should be taken at the quarter point of wall thickness.

$$\text{Design moment, '}M_u\text{', at the critical section} = 335.8 \times \frac{0.930^2}{2} = 145.2 \text{ kNm/m}$$

The tensile stress, f_t, at the bottom of the footing is $f_t = \dfrac{M_u}{S}$

$$S = \frac{bh^2}{6} \qquad \text{.... (3.2)}$$

where, S = the section modulus,

b = width (1000 mm), and

h = total thickness in mm.

The maximum allowable stress = 1.25 N/mm^2

$$1000 \times \frac{h^2}{6} = \frac{145.2 \times 10^6}{1.25}$$

$$h^2 = 696960$$

$$h = 835 \text{ mm (adopt 840 mm)}$$

It is generally assumed that the bottom 30 mm to 50 mm of concrete placed against the ground may be of poor quality and can be ignored for strength purposes. So total required thickness

$$840 + 40 = 880 \text{ mm.}$$

This is nearly the same as the assumed thickness. Hence reworking is not needed.

IS specifies that the critical section for shear should be taken at a distance equal to the effective depth of the footing from the face of the wall. It is observed that the critical section is very close to the edge of the wall. Shear force acting on the section will be negligible (Figure 3.6).

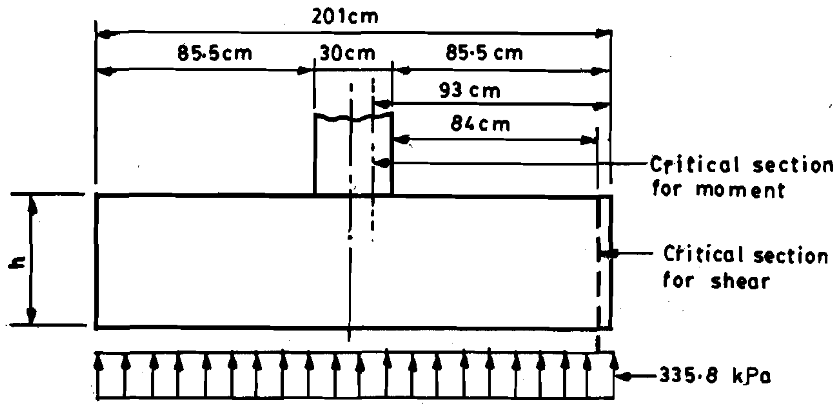


Figure 3.6 : Plain Concrete Footing

Hence no computation for shear is needed for the footing.

Sometimes longitudinal steel is provided to improve the structural integrity of the footing if there are differential movements between different points of the footing. It will lend flexural strength in the longitudinal direction.

Assuming a minimum steel required is equal to shrinkage and temperature steel for F_e415, i.e. 0.15%of cross-sectional area.

$A_s = 0.0015\, bh = 0.0015 \times 1000 \times 880 = 1320 \text{ mm}^2/\text{m}$

However, provision of this steel requirement will make the plain concrete footings uneconomical.

3.5.2 Reinforced Concrete Footing

Now let us design the footing as a reinforced concrete footing. Limit State Design method is adopted.

Example 3.5

The only additional information needed is that steel used is F_e415. The allowable tensile strength is 415 N/mm^{-2}.

Solution

Width of Footing

Let the footing thickness = 450 mm

Assume unit weight of concrete = 24 kN/m^3

Then the imposed stress to weight of footing = $0.45 \times 24 = 10.80$ kPa.

Since the bottom of the footing is at a depth of 1.2 m, the thickness of soil above the footing is 0.75 m. Imposed load due to the weight of earth

$$0.75 \times 16.80 = 12.60 \text{ kPa.}$$

So allowable soil pressure for superimposed service loads is = 250 – 10.80 – 12.60
= 226.60 kPa

Total service loads/m = 450 kPa

$$\text{Width of the footing} = \frac{450}{226.60} = 1.99 \text{ m}$$

Ultimate Soil Reaction

Factored load is to be used. Weight of footing and soil need not be considered.

$$\text{Design Load} = 1.5\ DL + 1.5\ LL = 1.5 \times 450 = 675\ \text{kPa}$$

$$\text{Soil Reaction} = \frac{675}{1.99 \times 1} = 339.2\ \text{kPa}$$

One Way Shear

Shear considerations normally control the depth of reinforced concrete wall footing. The critical section for shear is the vertical plane at a distance equal to the effective depth of the footing. The allowable shear is taken as 0.35 N/mm^2.

The actual effective depth = Total depth – Concrete cover – one half the bar diameter.

Assuming 75 mm cover and 20 mm bars effective depth = 450 – 75 – 10 = 365 mm

Shear at critical section = 339.2 × 0.48 × 1.0 = 162.8 kN/m

$$\text{Shear stress} = \frac{162.8 \times 10^3}{1000 \times 365} = 0.45\ \text{N/mm}^2$$

This is greater than the allowable stress.

Let us try a thickness of 510 mm effective depth = 510 – 75 – 10 = 425 mm

Allowable soil pressure = 250 – 0.51 × 24 – 0.69 × 16.8 = 226.17 kN/m^2

$$\text{Width} = \frac{450}{226.17} = 1.99\ \text{m}$$

Soil Reaction = 339.2 kPa

Shear force at the critical section as shown in Figure 3.7

= 339.2 × 0.420 = 142.5 kN/m

$$\text{Shear Stress} = \frac{142.5 \times 1000}{425 \times 1000} = 0.335\ \text{N/mm}^2$$

Less than 0.35 N/mm^2, So Satisfactory

Please note that allowable shear will be greater than the minimum value of 0.35 N/mm^2, if the tensile reinforcement is also considered.

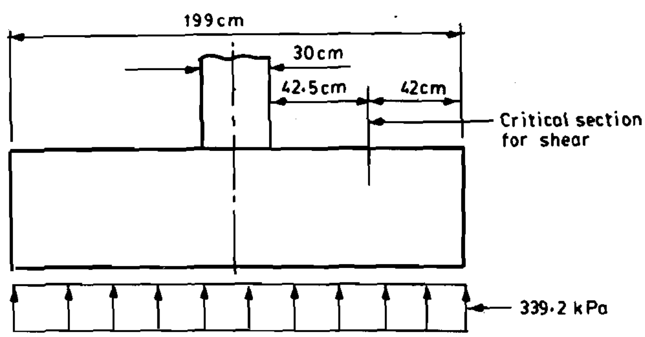


Figure 3.7: Reinforced Concrete Wall Footing

Bending Moment

The critical section for footing supporting concrete masonry walls is at a section half way between the centre line and edge of the wall, as seen in Example 3.4.

Bending moment at the critical section

$$M = 339.2 \times 1.0 \times 0.920 \times \ = 287.1\ \text{kNm/m}$$
$$= 287.1 \times 10^6\ \text{N mm/m}$$

The required effective depth d can be obtained from the relation

$$d = \left[\frac{M_u}{K_b}\right]^{1/2} \qquad \text{... (3.3)}$$

From Table 3.1, $K = 0.138 f_{ck} = 2.76$

$$d = \left(\frac{287.1 \times 10^6}{2.76 \times 1000}\right)^{1/2} = 322.5 \text{ mm}$$

So effective depth of 425 mm based on shear considerations is adequate.

Reinforcement Required

$$\frac{M}{bd^2} = \frac{287.1 \times 10^6}{1000 \times 425^2} = 1.59$$

From Table 3.2, steel percentage = 0.494

$$\text{Area of steel} = \frac{0.494 \times 1000 \times 425}{100} = 2100 \text{ mm}^2$$

Take 20 mm ϕ bars,

$$\text{Area of each bar} = \frac{\pi\, 20^2}{4} = 314.16 \text{ mm}^2$$

No. of bars required = 2100/314.16 = 6.6

Use of 7 bars of 20 mm dia.

Area provided = 7 × 314.16 = 2199 mm^2

Steel is uniformly distributed, let the cover be 75 mm

$$\text{Spacing} = \frac{1000-2\,(75)-20}{6} = \frac{830}{6} = 138 \text{ mm}$$

Longitudinal steel is provided as the minimum required, i.e. 0.15%

$$A_S = 0.0015\, bh$$

$$= 0.0015 \times 1990 \times 510 = 1522 \text{ mm}^2$$

Provided 8 bars of 16 mm dia

Area provided = 1608 mm

$$\text{Spacing} = \frac{1990 - 2\,(75) - 16}{7} = 260 \text{ mm}$$

Development Length

Length from the critical section = 995 – 75 = 920 mm

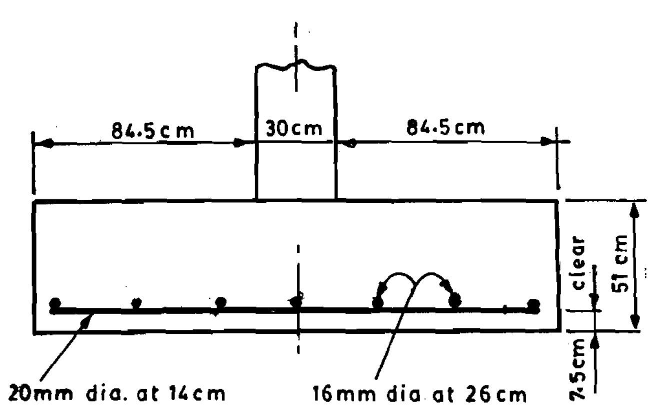


Figure 3.8: Reinforcement Details of Footing

Development length required for 20 mm bar = 40× 20 = 800 mm

So satisfactory

Figure 3.8 shows the reinforcement details.

SAQ 3

i) In India temples 50m high have been built. They impose a heavy load on the foundation. If we asume the unit of weight of masonry as 22 kN/m , the imposed load will be about 1000 kPa. There were no reinforcement in those days. Can you state the type of foundations used for them ?

ii) A reinforced concrete wall 40 cm wide carries a load of 200 kN/m. It is supported by a reinforced concrete footing having 2.0 m width and 50 cm effective depth. Calculate the one way shear force and moment at the critical sections.

3.6 SQUARE FOOTINGS

This type of footing, also termed an isolated spread footing is probably the most common, simplest and most economical of the various types of footings used for structures. Individual column footings are square in plan though rectangular footings have also to be used when dimensional limitations exist. Basically the footing is a slab that directly supports a column.

The footing behaviour under concentric load is that of two way cantilever action extending out from the column.The footing is loaded in an upward direction by the soil pressure and tensile stresses are induced in each direction at the bottom of the footing. Hence two layers of steel reinforcement perpendicular to each other and parallel to edges are provided.

Example 3.6

A square column 410 × 410 mm (with 4 No. 16 mm bars) carries a dead load of 1020 kN and an imposed load of 410 kN. The foundation soil has a safe bearing capacity of 200 kPa. F_e 415 is to be used for reinforcement. You have to design a square footing. The column and the foundation are constructed of M20 grade concrete.

Solution

Plan Area

No details are given regarding the depth of foundation and unit weight of soil. Hence we will assume that the weight of footing and soil above it is 10% of applied load.

Total load = 1020 + 410 + 0.1 × 1430 = 1573 kN

$$\text{Area required} = \frac{1573}{200} = 7.865 \text{ m}^2$$

Adopt 2800 mm × 2800 mm square footing.

Ultimate Soil Reaction

Factored loads have to be taken

Weight of footing is not to be considered

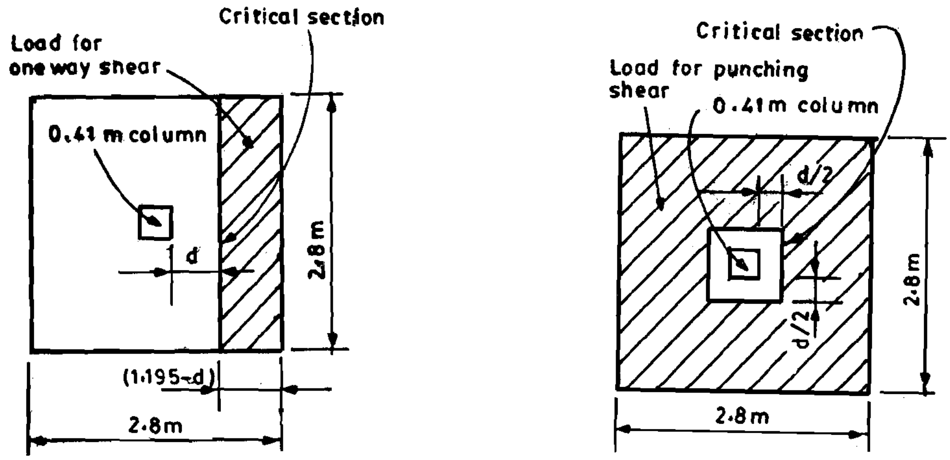
Design load = 1.5 DL + 1.5 LL = 1.5 × 1430 = 2145 kN

$$\text{Soil Reaction} = \frac{2145}{2.80 \times 2.80} = 273.6 \text{ kPa}$$

One Way Shear

You have already worked out a similar case in the case of wall footings in section 3.5.2. For square footings also the critical section is considered along a vertical plane extending to full width of the base located at a distance equal to the effective depth of the footing. The lowest allowable shear of 0.35 N/mm^2 or 350 kN/m^2 is used. Please note that the allowable shear stress can be increased based on tension reinforcement provided according to some codes.

The upward shear force acting along the critical section assuming effective depth = d, as shown in Figure 3.9 (a).



(a) One Way Shear (b) Punching Shear

Figure 3.9: Critical Section for Shear

$$V = 273.6 \times (1.195 - d)\,(2.80)\text{ kN}$$

$$\text{Area resisting shear} = b \times d = 2.80\ d\text{ m}^2$$

$$\text{Resisting shear force} = 350 \times 2.80 \times d\text{ kN}$$

$$\text{So, } 350 \times 2.80 \times d = 273.6\,(1.195 - d)\,(2.80)$$

$$d = \frac{328.32}{623.6} = 0.521\text{ m},\quad \text{Adopt, } d = 530\text{ mm}$$

Two Way Punching Shear

In columns, generally the tendency of the column is to punch through the footing. This also is to be considered in design. In wall footings this will not be the case. According to IS, the two way punching shear has to be checked along the surface of a truncated cone around the column. The critical perimeter is specified at a distance $d/2$ from the column face (Figure 3.9(b)).

The maximum value of perimeter shear stress is specified as $0.25\sqrt{f_{ck}}$ (0.5 + ratio of short side to long side of column) and should not be greater than $0.25 f_{ck}$.

For a square column of M20 grade concrete, the maximum value of perimeter shear stress $= 0.25 \times \sqrt{20} = 1.12$ N/mm^2 = 1120 kPa.

Total punching shear force = $273.6\,[(2.80 \times 2.80) - (0.41 + d)^2]$

Area over which punching shear acts $= 4\,(0.41 + d)\,d$

Punching resistance = $1120 \times 4\,(0.41 + d)\,d$ kN

So, $273.6\,(2.80^2 - (0.41 + d)^2] = 1120 \times 4\,(0.41 \times d)\,d$

Solving for d, $d = 0.48$ m = 480 mm.

Bending Moment

You have already seen that the critical section for bending moment for masonry or concrete block walls is half-way between the centre line and edge of the wall in section 3.5.2. For gusseted bases the critical section is half-way between the face of the column and edge of the gusseted base.

IS specifies that the critical section for reinforced concrete footings supporting reinforced concrete columns is at the face of the column (Figure 3.10).

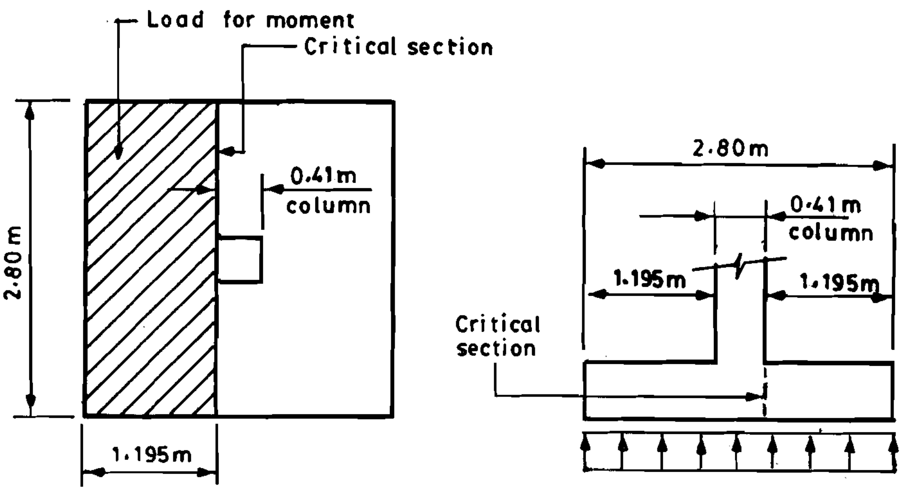


Figure 3.10: Footing Moment Analysis

Assuming effective depth as d, Bending moment M at the face of column (Figure 3.10(c))

$$M = \frac{273.60 \times 2.80 \times (1.40 - 0.205)^2}{2} \text{ kN-m}$$

$$= 547 \text{ kN-m} = 547 \times 10^6 \text{ N.mm}$$

$$d = \left(\frac{M}{k_b}\right)^{1/2}$$

where K is obtained from Table 3.1, L = 2800 mm

$$d = \frac{547 \times 10^6}{2.76 \times 2800} = 266 \text{ mm}$$

You can see that in this particular example, one way shear controls the thickness of the footings.

Effective depth of 530 mm is selected.

Table 3.1 : Values of Resistance Moments for Limiting Values of *x*/*d*

(Refer Table E of SP 16)

Steel	*x*/*d*	Expression for M
Fe 250	0.53	$M = 0.149 f_{ck} bd^2$
Fe 415	0.48	$M = 0.138 f_{ck} bd^2$
Fe 500	0.46	$M = 0.136 f_{ck} bd^2$

Reinforcement Required

$$\frac{M}{bd^2} = \frac{546 \times 10^6}{2800 \times 530^2} = 0.695$$

From Table 3.2, steel percentage = 0.206

This is greater than minimum percentage of 0.15

$$\text{Area of steel} = \frac{0.206 \times 2800 \times 5302}{100} = 3057 \text{ mm}^2$$

Use 10 bars of 20 mm diameter

$$\text{Area provided} = 10 \times 314.2 = 3142 \text{ mm}^2$$

The steel requirement in other direction may be assumed identical.

Steel is uniformly distributed in both directions. Let cover be 75 mm

$$\text{Spacing} = \frac{2800 - 2 \times 75 - 20}{9} = 292 \text{ mm}$$

Spacing is less than 3 d.

Total depth = 530 + 75 + 20 = 625 mm^2

Development Length for Bond

Length from face of column = 1/2 (2800 – 410) = 1195 mm

Required bond for 20 mm bar = 40 × 20 = 800 mm

So bond length provided is sufficient.

Transfer of Stress to Footing

The column transfers the load to the top of the footing by bearing. The magnitude of the pressure allowed under direct compression on an unreinforced loaded area of the same size is limited to $0.45 f_{ck}$. When the supporting area is larger than the loading area on all sides, the stress may be increased by a factor $\sqrt{A_1/A_2}$ where A_1 is the area of footing and A_2 area of column. However, this factor should not be greater than 2. If the strength of footing is less than that of column, this also should be considered. If the allowable stress is exceeded, steel reinforcement is extended into the footing to transfer the stress. Dowel bars can also be provided to transfer the stresses and they should extend into the column to a distance equal to the development length of column base. For M20 grade concrete and F_e 415, the depth required for anchorage is 37.6 ϕ where ϕ is the diameter of the bar. Usually it is rounded off to 40 ϕ.

The external longitudinal bars or dowels should be atleast 0.5 percent of the cross-sectional area of the supported columns. A minimum of 4 bars are to be provided and the diameter of the bars should not exceed the diameter of the column bars by 3 mm.

Maximum design load at base of column = 2145 kN

Permissible bearing stress = $0.45 \sqrt{f_{ck}} \; \sqrt{A_1/A_2}$

$$= \sqrt{\frac{A_1}{A_2}} = \sqrt{\frac{2800 \times 2800}{410 \times 410}} = \sqrt{2.61} > 2.0$$

So limiting value of permissible bearing stress $= 2 \times 0.45 \times 20 = 18 \text{ N/mm}^2$

Load taken up by concrete $= 18 \times 410 \times 410 = 3025$ kN

The entire column load can be transferred by concrete alone. However code and field practice require a minimum dowel area of 0.5% of area of column.

$$A_s = \frac{0.5 \times 410^2}{100} = 840.5 \text{ mm}^2$$

Provide 4 No. of 16 mm bars

Area provided = 804.2 mm^2

The general practice is to provide dowel bars of the same diameter as column bars.

Figure 3.11 shows the arrangement of reinforcement.

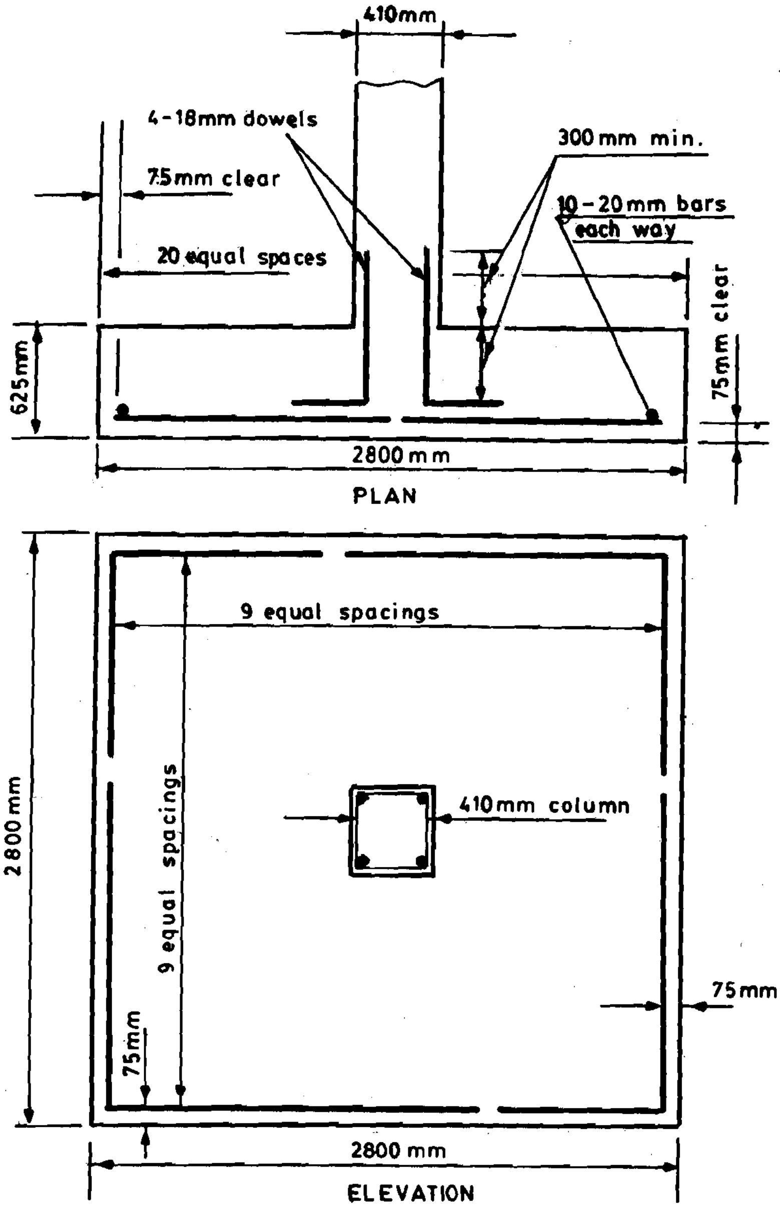


Figure 3.11:Design Sketch

SAQ 4

i) If you have to design a square footing for a circular column, how will you select the critical sections?

ii) A RCC footing 3.0 m × 3.0 m supports a column 50 cm × 50 cm and carries a factored load of 1800 kN. The effective depth is 60 cm. Calculate the one way shear force, punching shear force and the moment at the critical sections.

3.7 RECTANGULAR FOOTINGS

The design of rectangular footings is similar to that of the square footings. The major difference is that each direction should be investigated independently. Shear is checked for two way action in the normal manner; but one way action is checked across the shorter side only. Bending moments must be considered separately for each direction and each direction will have a different area of steel required. The reinforcing steel running in the long direction should be placed below the short direction steel so that it may have the larger effective depth to carry the larger bending moment in that direction.

Please note that the distribution of reinforcement in the rectangular footings is different than that of square footings. The reinforcement in the large direction should uniformly be distributed over the shorter footing width. A part of the required reinforcement in the short direction is placed in a band equal to the short side of the footing. The portion of the total steel that should go into in this band is $2/(\beta + 1)$ where, β is the ratio of the long side of the footing to the short side of the footing. The remainder of the reinforcement is uniformly distributed in the outer portions of the footing. The distribution is shown in Figure 3.12.

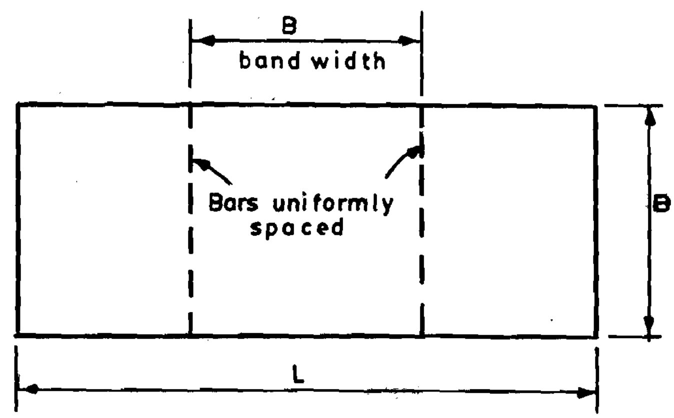


Figure 3.12: Rectangular Footing Plan

Example 3.7

Design a rectangular reinforced concrete footing for the following design data.

Loads: Dead Load = 340 kN; Live Load = 260 kN

Column size = 400 mm × 400 mm

Column reinforcement is four 20 mm bars

M20 grade concrete and F_e 415 are used in column and footing.

Allowable bearing capacity = 100 kPa

Assume B = 2.00 m

Solution

Plan Area

Rectangular footings are necessary

i) where square footings cannot be used because of space limitations,

ii) where an overturning moment is present, and

iii) for rectangular columns.

Width B is given. So length L is to be determined.

Assume weight of footing + weight of soil above the footing = 10% of load.

The load = (340 + 260) + 0.1 × 600 = 660 kN

$$\text{Area required} = \frac{660}{100} = 6.60 \text{ m}^2$$

$$L = \frac{6.60}{2.00} = 3.30 \text{ m}$$

Provide 3.30 × 2.00 m

Ultimate Soil Reaction

Design load = 1.5 × 600 = 900 kN

Soil Reaction = $\frac{900}{6.60}$ = 136.4 kPa

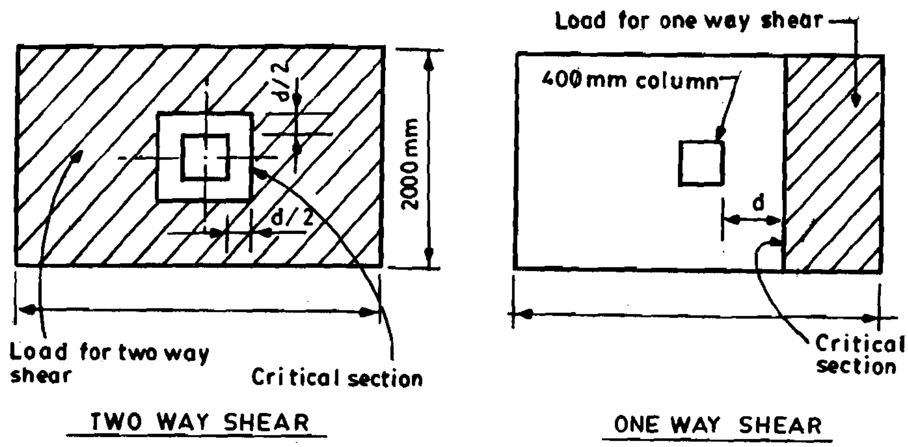


Figure 3.13: Footing Shear Analysis

Two Way Shear

Allowable shear stress = $0.25\sqrt{f_{ck}}$

$= 0.25\sqrt{20}$

$= 1.12$ N/mm^2 = 1120 kPa

as $b > 1/2\,L$, no correction is needed.

Taking a section at $d/2$ from the face of the column, punching shear force V (Figure 3.13 a).

$V = 136.4\,[6.60 - (0.40 \times d)^2\,]$

The shear resistance = $1120 \times 4 \times d\,(0.40 + d)$

So, $4480\,d + 1792\,d = 878.4 - 109.1d - 136.4\,d$

$d = 0.28$ m or 280 mm.

One Way Shear

For maximum shear, the section along the breadth at a distance d from the face of the column is taken

Assume allowable shear = 0.35 N/mm^2 = 350 kN/m^2

Shear force acting = 136.4 (2.00) (1.45 – d) (Figure 3.13)

Shear Resistance = 350 (2.00) (d)

$d = 0.407$ m = 410 mm.

Checking in the other direction, assuming $d = 410$ mm

Shear force 136.4 (3.3) (0.39) = 175.6 kN

Shear resistance 350 × 3.3 × 0.41 = 473.6 kN

Hence satisfactory.

Bending Moment

Section yy (longitudinal direction)

$$M_y = 136.4\,(1.45)\,(2.00)\,\frac{1.45}{2} = 286.8 \text{ kN m (Figure 3.14)}$$

Section xx (short direction)

$$M = 136.4\ (3.3)\frac{(0.80)^2}{2} = 144.0 \text{ kN m (Figure 3.14)}$$

$$d = \frac{286.8 \times 10^6}{2.76 \times 3300} = 177 \text{ mm}$$

Adopt 410 mm based on one way shear.

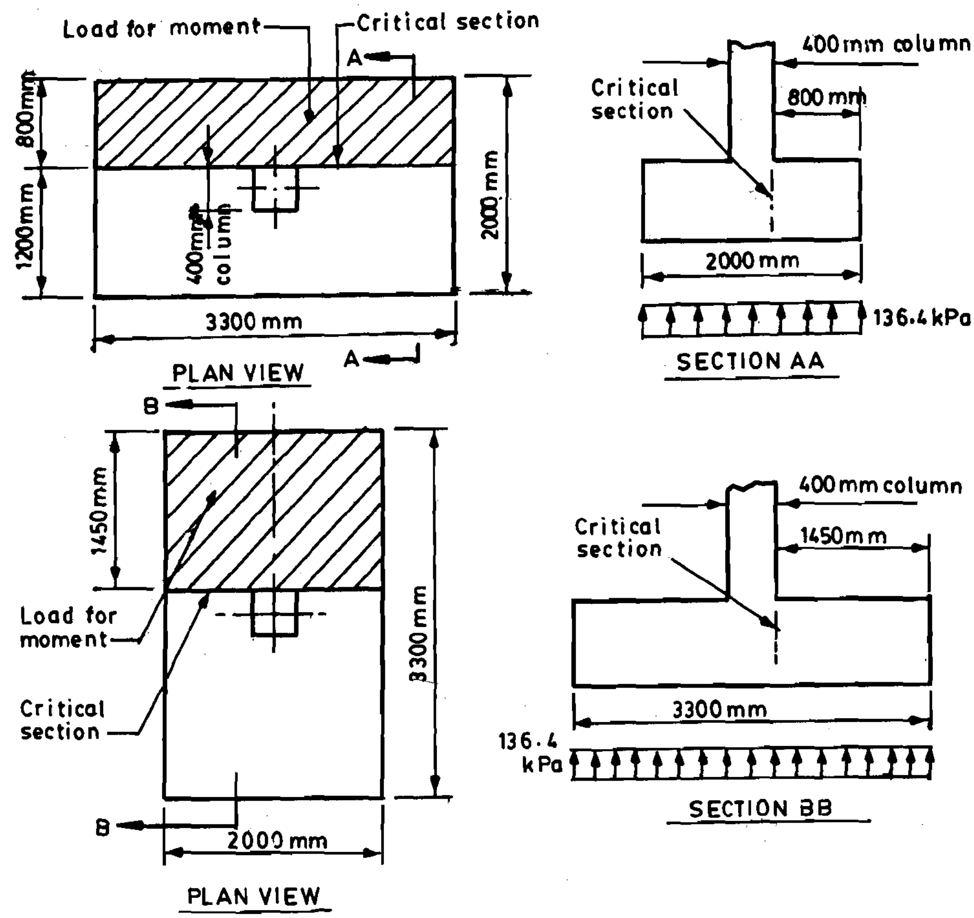


Figure 3.14: Footing Moment Analysis

Reinforcement Required

Longitudinal direction (YY) $= \frac{M}{bd^2} = \frac{286.8 \times 10^6}{2000 \times 410 \times 410} = 0.853$

Percentage of steel = 0.257

$$A_s = \frac{0.257 \times 2000 \times 410}{100} = 2107.4 \text{ mm}^2$$

Use 11 bars 16 mm dia, Area provided = 2211 mm^2.

Short Direction (XX)

$$= \frac{M}{bd} = \frac{144 \times 10^6}{3300 \times 410 \times 410} = 0.26$$

Percentage of steel = 0.085

$$A_s = \frac{0.085 \times 3300 \times 410}{1000} = 1150.0 \text{ mm}^2$$

Use 11 bars of 12 mm dia, Area provided = 1244.1 mm^2.

Development Length in Short Direction

For 12 mm roads length required is 564 mm

Assume cover 75 mm

Development length available = 1000 – 200 – 150 = 650 mm, So satisfactory

Placing of Steel

Reinforcement in the long direction is placed at uniform spacing.

$$\frac{2000 - 2\,(75) - 16}{18} = 102 \text{ mm spacing}$$

For reinforcement in the short direction

$$\frac{3.30}{2.0} = 1.65$$

$$\frac{2}{1.65 + 1} = \frac{2}{2.65} = 0.75 \text{ i.e. } 0.75 \times 11 = 8.2 = 8 \text{ bars}$$

This percentage is placed in a band width of 2000 mm. 8 bars are placed on a spacing of 285 mm. 4 bars are spaced at

$$\frac{3300 - 2000 - 150 - 12}{2 \times 2} = 285 \text{ mm spacing}$$

It is decided to place the reinforcement at uniform spacing of

$$= \frac{3300 - 150 - 12}{10} = 314 \text{ mm.}$$

Transfer of Load to Base of Column

Capacity = $0.45 f_{ck} \times 2 \times \text{area}$

$$= 0.45 \times 20 \times 2 \times 400 \times 450 = 2880 \text{ kN}$$

This is greater than 900 kN. Hence dowels are not theoretically needed. But atleast 4 bars equal to 0.5 percent area of column are extended to the footing.

Assume 20 mm ϕ bars are used. The depth of footing should be equal to the development length. For 20 mm bar, development length 752 mm.

Provide a stepped footing to reduce the amount of concrete. Use a pedestal 350 mm thick around the column with an offset of 200 mm around the column (Plan area 800 mm × 800 mm). The rest of the footing will be stepped and will have a constant depth of 400 mm.

Please note that in many codes it is not specified that the depth of footing should satisfy the development requirements.

SAQ 5

A rectangular R.C.C. footing 3.0 m × 2.0 m supports a R.C.C column 45 cm × 30 cm carrying a factored load of 1200 kN. The effective depth is 60 cm. Calculate the one way shear force, punching shear force and the moment at the critical sections.

3.8 PEDESTALS

A pedestal is used to transfer the loads from the metal columns through the floor and soil to the footing when it is at some depth within the ground. It is a short compression member whose height is usually less than three times its least lateral dimension (Figure 3.15).

The provision of a pedestal avoids possible corrosion of the metal from soil and wet floors. In industrial buildings where floors are washed regularly, the column bases should be 50 mm to 100 mm above the floor level.

Introduction of pedestals enables the construction engineers to make up levels to supply larger bearing areas to the foundations and to provide enough development length for reinforcements (Example 3.7).

The pedestal can be designed either as reinforced or a plain pedestal. When the stress on top of the pedestal is less than $0.45 f_{ck}$ (A_2/A_1), no steel is theoretically required. However it is advisable to have atleast 0.4 percent of the area of the pedestal as nominal longitudinal steel with 12mm laterals binding them together as in columns. When the stress on the base is greater than $0.45 f_{ck}$ (A_1/A_2) reinforcements have to be provided either by extending the longitudinal bars from the column into the base or by means of dowels from base to column.

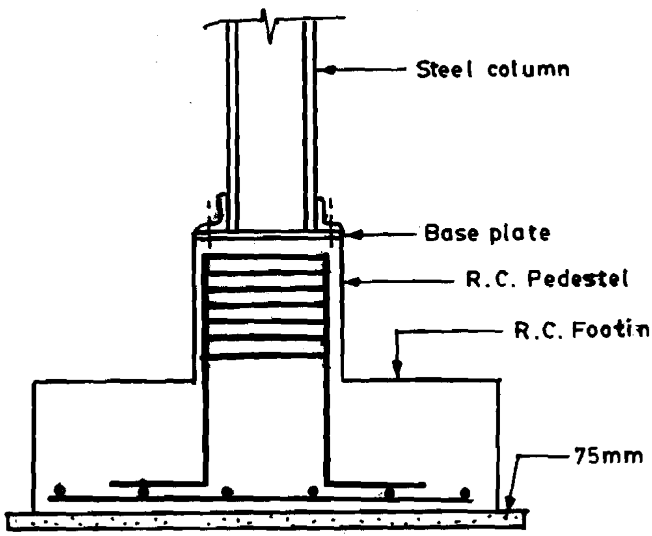


Figure 3.15: R.C.C. Pedestal

Steel should be liberally added at the top to avoid spalls and keep the edges from cracking. Space must be provided to place the anchor bolts to hold the bearing plate and column in correct position. The steel should surround the anchor bolt to increase its pull out resistance.

Example 3.8

Design a concrete pedestal for a steel column carrying a dead load of 600 kN and an imposed load of 600 kN. Assume the base plate is 350 mm × 350 mm M20 grade concrete and F_e 415 are used.

Factored load = 1.5 (600 + 600) = 1800 kN.

Solution

Design of Unreinforced Pedestal

Bearing strength = $0.45 f_{ck} = 0.45 \times 20 = 9$ N/mm^2

Maximum allowable strength = $2 \times 9 = 18$ N/mm^2

$$\text{Pressure on base plate} = \frac{1800 \times 10^3}{350 \times 350} = 14.7 \text{ N/mm}^2$$

$$\text{Area of concrete required} = \frac{1800 \times 1000}{9} = 200000 \text{ mm}^2$$

$$= 447 \times 447 \text{ mm.}$$

Provide a pedestal of 450 mm × 450 mm. Theoretically the above pedestal need not be reinforced. Usually 0.4 percent of the area of pedestal is provided.

$$A_s = \frac{0.4 \times 450 \times 450}{100} = 810 \text{ mm}^2$$

Provide 4 bars of 16 mm dia.

Area provided = 804.2 mm^2. Provide usual laterals with closer spacing of 8 to 15 cm at the top.

Design of Reinforced Pedestal

Adopt a minimum size for the pedestal, adopting 10 mm clearance
= 370 mm× 370 mm.

Load carried by the pedestal = 370 × 370 × 9 = 1232 kN

Balance load = 1800 – 1232 = 568 kN

$$\text{Area of steel required} = \frac{568 \times 10^3}{0.87 \times 415} = 1573 \text{ mm}^2$$

$$\text{Percentage of steel} = \frac{1573}{370 \times 370} = 1.14\%$$

Provide 4 bars of 20 mm dia.

Area provided = 1576 mm^2

Provide usual laterals with closer spacing near the top.

3.9 ECCENTRICALLY LOADED FOOTINGS

When footings have overturning moments and axial loads, the base pressure under the footing will be non-uniform. In simplified rigid body approach, the base pressure will vary from a maximum value of q_2 and a minimum value of q_1 as shown in Figure 3.16 (a). You can also see that the eccentric load on a column as in Figure 3.16 (b) also produces a similar effect. The linear non-uniform soil pressure diagram is obtained from a superposition of compression and moment stresses.

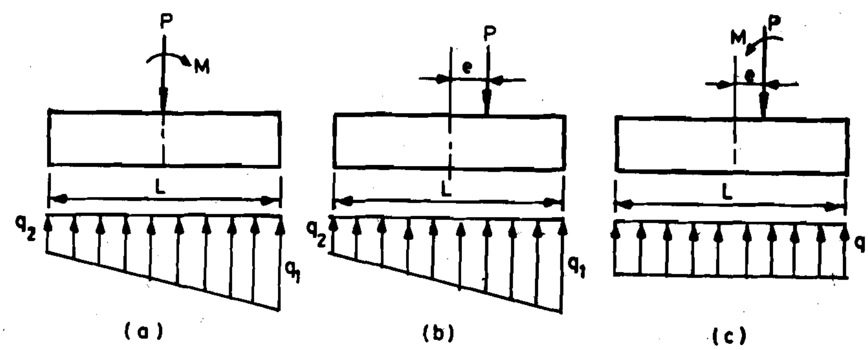


Figure 3.16: Non-uniform Pressure under Footings

$$q_1 = \frac{P}{BL} + \frac{6M}{BL} \qquad ....(3.4)$$

$$q_2 = \frac{P}{BL} - \frac{6M}{BL} \qquad ... (3.5)$$

$$q_1 = \frac{P}{BL}\left(1 + \frac{6e}{L}\right) \qquad ... (3.6)$$

$$q^2 = \frac{P}{BL}\left(1 - \frac{6e}{BL}\right) \qquad ... (3.7)$$

Figure 3.16(c) is a method of deliberately placing a column away from the centre of the footing so that resulting soil pressure is uniform. However this method may not be economical. Also the solution is valid only for moments which always act in the direction shown for that footing configuration and it is not valid for wind moments wherein reversals are possible.

It should be evident that a column transmits moment to the footing only shall have to if it is rigidly attached. All concrete columns satisfy this criterion.

When eccentricity e is sufficiently large, the computed minimum soil pressure becomes negative, indicating a tensile state of stress between the soil and the footing. For $q_2 = 0$, the eccentricity is $L/6$ and the footing will be fully effective in bearing. This implies that as long as the resultant force falls within the middle third the entire footing is effective.

Normally, the ratio q_1 / q_2 should not exceed 2 to 4. If e is in the range of 1/10 to $L/12$ eccentricity e can be considered small. In footings subjected to biaxial bending, the effect of variation of base pressure in both planes should be considered and combined to get the final values.

Example 3.9

Proportion a footing for a concentric column load and overturning moment for Dead load = 400 kN; live load = 400 kN; moment = 400 kN m. Allowable bearing capacity of soil 100 kPa as shown in (Figure 3.17).

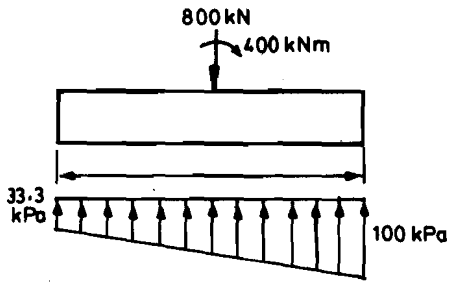


Figure 3.17 : Pressure Diagram

Solution

$$e = \frac{M}{V} = \frac{400}{800} = 0.5 \text{ m}$$

q_1 should not exceed 100 kPa

$$q_1 = 100 \text{ kPa} = \frac{800}{BL} \left(1 + \frac{6 \times 0.5}{L}\right)$$

$$\text{or } B = \frac{8}{L} (1 + 0.5) = 2 \text{ m}$$

$$\text{Let, } L = 6 \text{ m}, B = \frac{B}{6} (1 + 0.5) = 2 \text{ m}$$

$$q_1 = \frac{800}{12} (1 + 0.5) = 100 \text{ kPa}$$

$$q_2 = \frac{800}{12} (1 - 0.5) = 33.3 \text{ kPa}$$

Example 3.10

Design a footing such that base pressure is approximately uniform under the footing for the data given below:

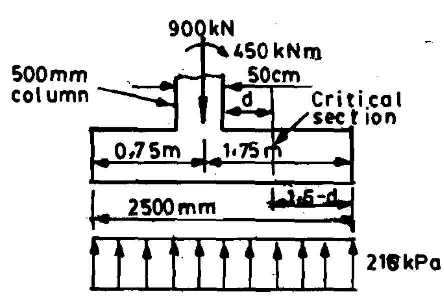


Figure 3.18: One Way Shear

Dead load = 400 kN; Live load = 500 kN

Moment = 450 kN-m

Allowable bearing capacity of soil = 150 kPa

Column size 500 mm × 500 mm M 20 grade concrete and F_e 415 are used (Figure 3.18)

Solution

Footing Size

$$\text{Eccentricity} = \frac{450}{400 + 450} = 0.5 \text{ m}$$

The centre line of column should be 0.5 m from the footing centre line.

$$\text{Required footing area} = \frac{900}{150} = 6 \text{ m}^2 \text{ (neglecting moment)}$$

For a square footing B = 2.45 m

Try B = 2.5 m

Factored Load = 1.5 (900) = 1350 kN

$$\text{Soil Reaction} = \frac{1350}{2.5 \times 2.5} = 216 \text{ kPa}$$

One Way Shear

Let the allowable shear stress = 350 kPa as in previous examples.

Critical section is at a distance of d from the face of the column as shown in Figure 3.18.

Shear force acting on the section = 216 × 2.5 (1.5 – d)

Shear resistance = 350 (2.5) (d)

$$d = \frac{344}{566} = 0.608 \text{ m}, \qquad \text{Adopt, } d = 610 \text{ mm}$$

Two Way Shear

Assume allowable shear stress in punching = 1120 kPa

Punching shear force = 216 [2.5 × 2.5 – (0.5 + d)] (as shown in Figure 3.19)

$$= 1296 - 1350\,d - 1350d$$

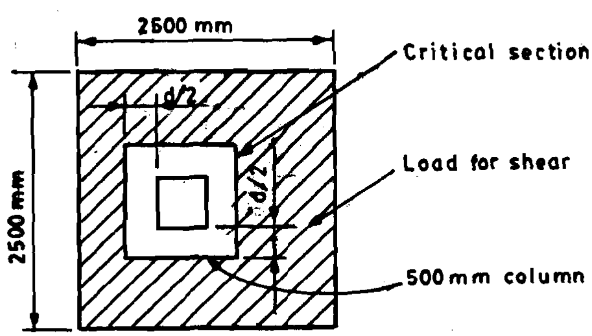


Figure 3.19: Two Way Shear

Punching shear resistance = 4 (0.5 + d) d × 1120 = 4480d + 2240d

So, 1296 – 1350 d – 1350 d = 4480 d + 2240 d

$$d = 260 \text{ mm.}$$

Bending Moment

Bending moment at the face of the column

$$= 216 \times 2.5 \times 1.5 \times \frac{1.5}{2} = 1518.75 \text{ kNm}$$

$$d = \frac{M}{276 \times 2500} = \frac{1518.75 \times 10^6}{276 \times 2500} = 469 \text{ mm}$$

It is evident that the wide beam shear or one way shear controls the thickness.

Adopt $d = 610$ mm.

The rest of the design is similar to what has been done in previous examples. The reinforcement obtained for the heavily loaded section can be used for other sections.

Example 3.11

Design a square footing for the following data.

Dead Load = 320 kN; Live load = 570 kN

Moment = 37.5 kNm

M20 grade concrete and F_e 415 are used.

Allowable bearing capacity = 180 kPa

Solution

Footing Size

$$\text{Width } B = \frac{570 + 320}{180} = 4.94 = 2.22 \text{ m}$$

Adopt $B = 2.50$ m, since moments are present.

$$\text{Eccentricity, } e = \frac{37.5}{890} = 0.042 \text{ m}$$

$$\text{The maximum pressure } q_1 = \frac{P}{A}\left(1 + \frac{6e}{B}\right)$$

$$= \frac{890}{6.25}\left(1 + 6 \times \frac{0.042}{2.50}\right) = 156.8 \text{ kN/m}^2$$

$$\text{The minimum pressure } q_2 = \frac{890}{6.25}(1 - 0.101) = 128.0 \text{ kPa}$$

These are satisfactory.

Soil Reaction

Factored load = $1.5 \times 890 = 1335$ kN
The ultimate soil reactions q_1 and q_2 are then

$$q_1 = \frac{1335}{6.25}(1 + 0.101) = 235.2 \text{ kPa}$$

$$q_2 = \frac{1335}{6.25}(1 - 0.101) = 192.0 \text{ kPa}$$

The soil pressure diagram and critical sections are shown in Figure 3.20.

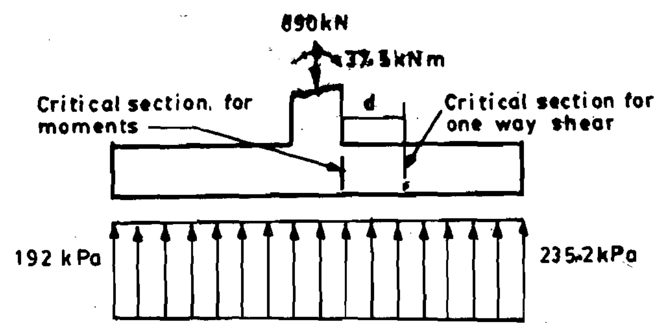


Figure 3.20: Critical Section

The thickness and reinforcement required are calculated for the heavily loaded portions. Same dimensions are adopted for other sections.

SAQ 6

i) Sketch the pressure distribution under a footing 2.0 m × 2.0 m carrying a load 1000 kN when the eccentricity is 0.50 m.

ii) Calculate the eccentricity of a vertical load acting on a footing $B \times L$ for $\frac{q_1}{q_2} = 2$.

3.10 COMBINED FOOTINGS

When a footing supports two or more columns in the same line, it is called a combined footing. The combined footing which supports two columns is relatively common. Two conditions in which it is used are

i) When an exterior column is immediately adjacent to a property or mechanical equipment where it is not possible to utilise an individual column footing, and

ii) When two columns are closely spaced resulting in the individual footings also to be closely spaced. In such situations a rectangular or trapezoidal shaped combined footing would generally be used.

The choice of shape is decided by the difference in column loads as well as physical limitations. Trapezoidal footings are used only when rectangular footings are not possible.

The footing geometry is made such that the resultant of several columns passes through the centroid of the footing area. This enables the designer to assume uniform soil pressure distribution.

3.10.1 Rectangular Combined Footings

In the design of rectangular combined footings it is assumed that the footing is rigid. Soil pressure will then be linear. If the line of action of the resultant of the forces acting on the columns passes through the centroid of the footing area, the soil reaction will be uniform. Though the assumption of a rigid foundation is questionable, the method has been successfully used for the design of combined footings. The combined footing can also be analysed on the assumption that the footing is flexible by beam on elastic foundation analysis approach. This analysis produces smaller design moments than those obtained by the rigid method and is realistic.

In this section the analysis is limited to the determination of plan dimensions of the footing. The physical dimensions of the combined footing are determined by allowable soil pressure. The centroid of the footing area should coincide with the line of action of the resultant of two column loads. These dimensions are generally determined by using service loads in combination with an allowable soil pressure.

Example 3.12

Two columns are loaded as shown in the Figure 3.21. Determine the dimensions of the combined footing to carry the column loads.

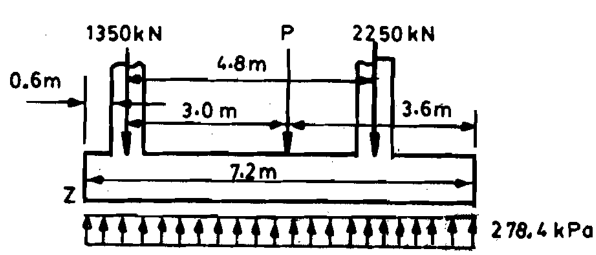


Figure 3.21: Combined Footing

Solution

Service load on column A = 1350 kN; Column B = 2250 kN

Column A is 45 cm × 45 cm; Column B is 50 cm × 50 cm

The centre to centre spacing between columns is 4.8 m.

Allowable soil pressure is 300 kPa

Let the resultant R acts at a distance x from the edge Z.

$R = 1350 + 2250 = 3600$ kN

$3600 \times x = 1350 \times 0.6 + 2250 \times 5.4, \; x = 3.6$ m

Assuming a rectangular shape, length L should be established so that the line of action of the resultant passes through the centroid of the footing.

Required $L = 3.6 \times 2 = 7.2$ m.

Let us assume a footing thickness of 0.9 m.

The weight of the footing = $0.9 \times 24 = 21.6$ kPa

Ignoring the weight of soil above the footing, allowable soil pressure = 300 – 21.6
= 278.4 kPa.

The footing area required $= \dfrac{3600}{278.4} = 12.93 \text{ m}^2$

Width of the footing $= \dfrac{12.93}{7.2} = 1.80$ m

SAQ 7

i) A combined footing 6 × 2 m carries two columns loads of 1000 kN and 800 kN. The centre to centre spacing of columns is 4.0 m and the column sizes are 50 cm × 50 cm. The columns are placed concentrically over the footing. Sketch the pressure distribution.

ii) For soil pressure under the footing in the above case to be uniform, determine the dimensions of the footing and the position of the columns over the column. Assume the property line is at a distance of 1.0 m from the centre of column carrying a load of 1000 kN. Allowable bearing capacity of soil is 150 kPa. Ignore the weight of footing.

3.10.2 Trapezoidal Combined Footings

A trapezoidal footing have to be used if there is a space limitation. The resultant of column loads will be closer to the larger load and due to constraints in physical dimensions it may not be possible to double the centroid distance as was done for the rectangular footings. The footing geometry necessary for a trapezoid shaped footing is shown in Figure 3.22.

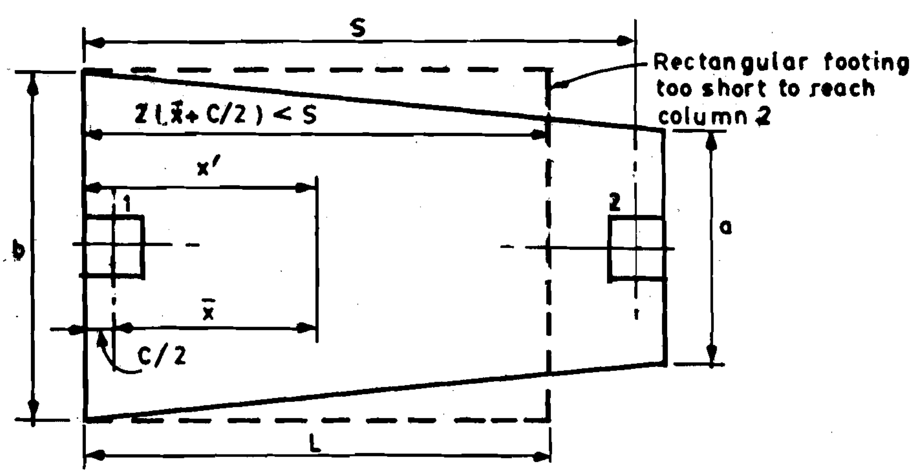


Figure 3.22: Trapezoidal Footing Geometry

From plane geometry considerations

$$A = \frac{a+b}{2} \times L \tag{3.8}$$

$$x' = \frac{L}{3}\ \frac{2a+b}{a+b} \tag{3.9}$$

where, A = area of the trapezium,

a,b = length of two parallel sides of the trapezium,

L = distance between the outer edges of the columns, and

x' = the distance of centroid of trapezium from the larger side b.

The value of L is normally known and the area A can be determined based on allowable soil pressure. By solving the above equations we can obtain the values of a and b.

Example 3.13

Two columns A and B are spaced 5.5 m apart. Column A as shown in Figure 4.23 carries a load of 1600 kN and column B, a load of 2000 kN. It is specified that the footing should not project beyond the edges of the column. The size of the column is 50 cm × 50 cm. The allowable bearing capacity is 200 kPa.

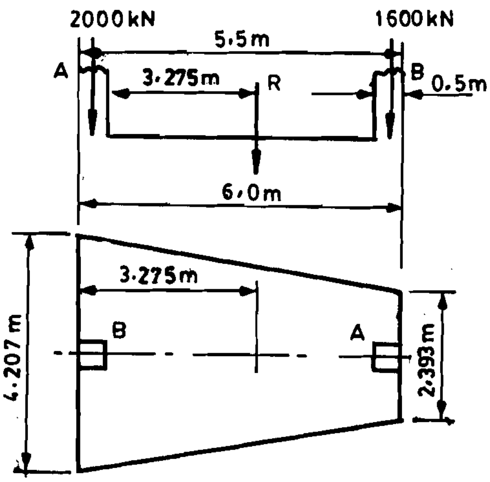


Figure 3.23: Trapezoidal Footing

Solution

Assume weight of footing = 10% of the load.

$$\text{Required area} = \frac{1600 + 2000 + 360}{200} = 19.8\ \text{m}^2$$

$$x' = \frac{2000 \times 5.75 + 1600 \times 0.25 + 360 \times 3}{3960} = 3.275\ m$$

$L = 5.50 + 0.50 = 6.0$ m

$$\text{So, } 19.80 = \frac{a+b}{2} \times 6.0$$

$$3.275 = \frac{6}{3}\ \frac{2a+b}{a+b}$$

$$\text{So, } a+b = \frac{2 \times 19.80}{6} = 6.60$$

$$2a+b = \frac{3.275 \times 3 \times 6.60}{6} = 10.807$$

$a = 4.207$ m

$b = 2.393$ m

3.10.3 Structural Design of Combined Footing

The rectangular and trapezoidal combined footings are designed assuming uniform soil pressure. You can observe that the footings are positioned relatively close to the edges of the footings. Hence the columns are assumed as supports and the footing is subjected to an upward uniformly distributed load caused by the soil reaction. Moments which create tension at the top of the footing will predominate in the longitudinal direction. So the principal longitudinal reinforcement will be placed at the top of the footing equally distributed across the footing width. Smaller moments in the transverse direction will cause compression at the top of the footing. So transverse steel will be provided under each column at the bottom of the footing to distribute the column loads. The combined footing can be considered as a wide rectangular beam in the longitudinal direction.

The relevant design considerations may be summarized as follows:

i) Main reinforcement (uniformly distributed) is placed in a longitudinal direction assuming that the footing is a longitudinal beam.

ii) Shear should be checked for both one way shear and two way shear for the columns.

iii) Stirrups or bent bars may be needed to maintain an economical footing thickness. This assumes that the shear effect is uniform across the width of the footing.

iv) Transverse reinforcement is uniformly placed at the bottom of the footing within a band having a width not greater than the column width plus twice the effective depth of the footing. The design procedure is similar to that of individual column footings.

v) Longitudinal steel is also placed at the bottom of the footing in order to tie together and position the stirrups and transverse steel. Although the required steel area may be small, the effect of cantilever moments in the vicinity of columns should be checked.

SAQ 8

i) Two columns 50 cm × 50 cm in size support service loads of 1200 kN and 800 kN. The centre to centre spacing between columns is 5.5m. It is specified that the sides of the footing should not project beyond the edges of the column. Explain why rectangular footing is not possible if the pressure distribution is to be uniform under the footing.

ii) In the above case, safe bearing capacity of the soil is 200 kPa. The self weight of the footing is 200 kN. Determine the dimensions of the footing.

3.10.4 Strap Footing

You have to use cantilever or strap footings when the proximity of a property line precludes the use of other types for example an isolated column footing may be too large

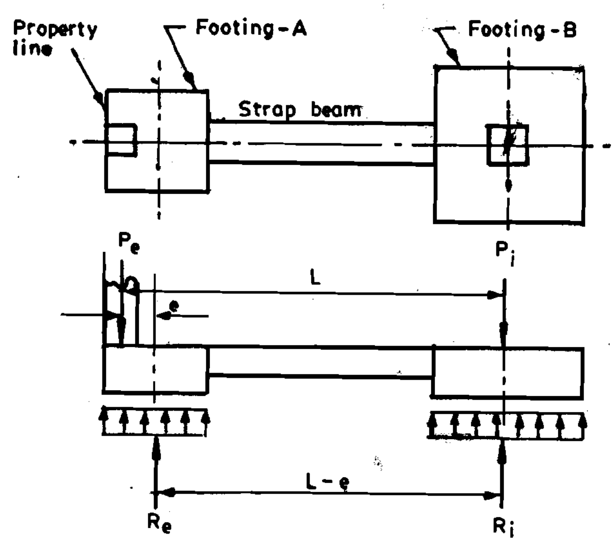


Figure 3.24: Strap Footing

for the area available and the nearest column may be too distant to allow a rectangular or trapezoidal combined footing to be economical. The strap footing may be considered as two individual column footings connected by a strap beam.

Figure 3.24 shows the exterior footing placed eccentrically under the exterior column so that property line limitations are not violated. Non-uniform pressure distribution is produced under the footing. This may lead to footing rotation. To balance this rotational or over turning effect, the exterior footing is connected by a stiff beam or strap to the nearest interior footing and uniform soil pressures under the footings are assumed.

The strap may be considered as a flexural member. It is subjected to both bending moment and shear resulting from the forces P_e and R_e acting on the exterior footing. The applied moment is counterclockwise and R_e will be greater than P_e. There is no eccentricity at the interior column between the column load P_e and the resultant soil pressure force R_i. Hence it is assumed that there will be no induced moment in the strap at the interior column.

We will define V as the vertical shear force necessary to keep the strap in equilibrium as shown in Figure 3.25.

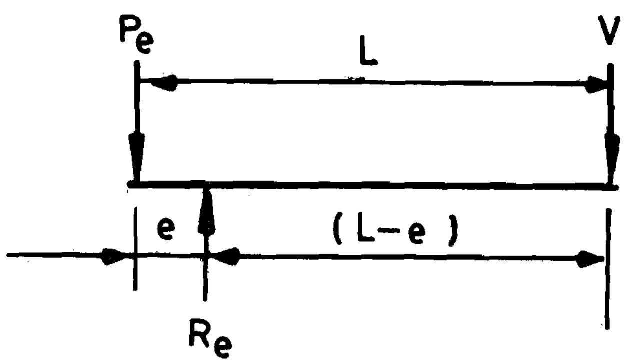


Figure 3.25: Strap Beam

You can calculate the magnitude of V and R_e since P_e is known. Taking moments about R_e.

$$P_e e = V\left(L - \frac{2.05}{2} e\right)$$

$$V = \frac{P_e e}{(L-e)}$$

From a summation of vertical forces

$$R_e = P_e + V$$

Note that V acting downward on the strap beam also means that V is an uplift force on the interior footing. So

$$R_i = P_i - V$$

You can observe that R_e becomes greater than P_e by a magnitude of V while R_i becomes less than P_i by a magnitude equal to V.

You can obtain the footing areas by dividing the reactions R_i and R_e by net allowable soil pressures. These values are based on an assumed value of e and have to be recomputed until the trial e and actual e are the same.

The interior footing is designed as an isolated column footing subjected to a load R_i. The exterior footing is considered as under one way transverse bending similar to a wall footing. The longitudinal steel is furnished by extending the strap steel into the footing. The selection of footing thickness and reinforcement should be based on factored loads consistent with IS code and Limit State Design Approach.

The strap beam is assumed to be a flexural member with no bearing on the soil underneath. The strap is designed as a rectangular beam subjected to a constant shearing force and a linearly varying negative bending moment.

Example 3.14

A strap footing is to be constructed as shown in Figure 3.26. Determine the sizes of external and internal footings.

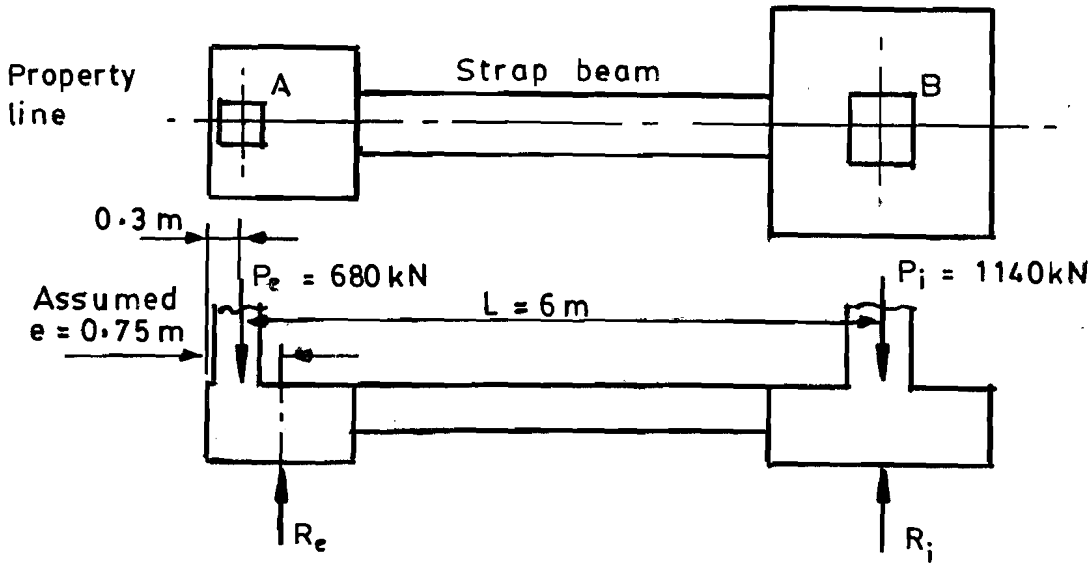


Figure 3.26: Strap Footing Design

Solution

Service load column A = 680 kN; Column B = 1140 kN

Column spacing = 6.0 m

Alllowable bearing capacity of soil = 200 kPa

Assume footing thickness = 0.6 m

Weight of footing = 0.6 × 24 = 14.4 kPa

Allowable soil pressure for superimposed loads = 200 – 14.4

= 185.6 kPa

Assume $e = 0.75$ m

$$V = \frac{680 \times 0.75}{5.25} = 97.1 \text{ kN}$$

Footing Reaction $R_e = 680 + 97.1 = 777.1$ kN

$R_i = 1140 - 97.1 = 1042.9$ kN

For the exterior footing required area $= \frac{777.1}{185.6} = 4.187 \text{ m}^2$

$$B = 2.05 \text{ m}$$

$$e = \frac{2.05}{2} - 0.3 = 0.725 \text{ m}$$

This is different from assumed $e = 0.75$ m, Try $e = 0.72$ m

$$V .= \frac{680 \times 0.72}{5.28} = 92.7 \text{ kN}$$

$R_e = 680 + 92.7 = 772.7$kN

$R_i = 1140 - 92.7 = 1047.3$ kN

Required Area $= \frac{772.7}{185.6} = 4.16 \text{ m}^2$, B = 2.04 m

$$e = \frac{2.04}{2} - 0.30 = 0.72 \text{ m}$$

For the interior footing, area required

$$\frac{1047.3}{1856} = 5.643 \text{ m}^2$$

Adopt a footing 2.40 m× 2.40 m.

SAQ 9

In the Example 3.19, explain why rectangular or trapezoidal footing are not feasible solutions.

3.11 RAFT FOUNDATION

A raft or mat can be considered as a combined footing which covers the entire building area and supports all columns and walls. Such a foundation may consist of a slab of uniform thickness or a slab stiffened by beams either above or below the slab as shown in Figure 3.27

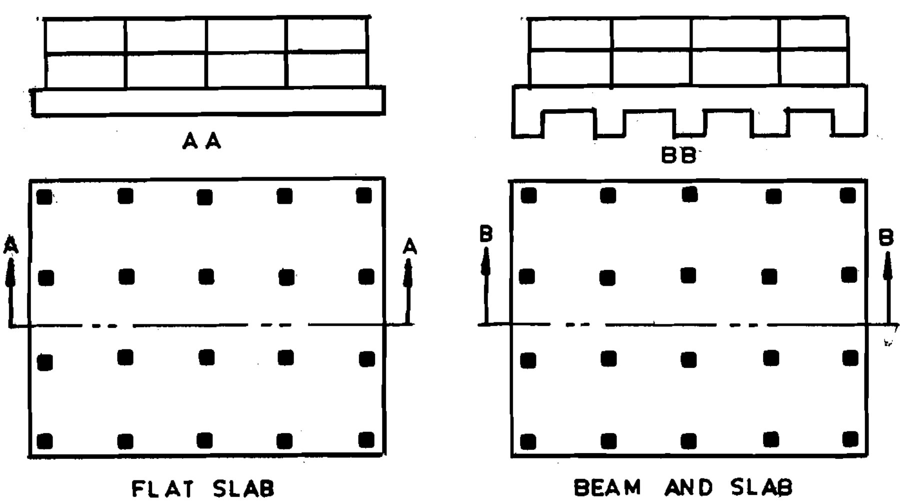


Figure 3.27: Common Types of Mat Foundations

You have already seen that when the allowable soil pressure is low or the building loads are heavy, the use of individual spread footings would cover a major portion of the plan area. When the total area of the footings cover more than fifty percent of the area, it is more economical to use a raft for the entire structure.

Raft or mat is also recommended when the soil mass contains compressible layers or the soil stratum is variable. In such cases use of raft tends to eliminate differential settlements.

You should also note that contact stresses are transferred to deeper depths since rafts are larger in dimension than footings. This may increase the total settlement in highly compressible soils. However this can be taken care of by making the weight of the structure and the raft approximately equal to the weight of soil excavated. Hence the net increase in pressure causing settlement will be reduced. This type of foundation is called **Floating Foundation.**

One of the common types of raft foundation is the flat concrete slab 0.75 to 2.00 m thick with a two way reinforcing both at the bottom and at the top. Conventional or rigid method can be used where columns are regularly spaced. The method assumes a rigid mat so that the pressure diagram is linear. If the line of action of the resultant of the vertical loads passes through the centroid of the raft, the upward pressure can be taken as uniform. The weight of the raft is not considered in the structural design. The mat is divided into convenient strips as shown in Figure 3.28 in both x and y directions.

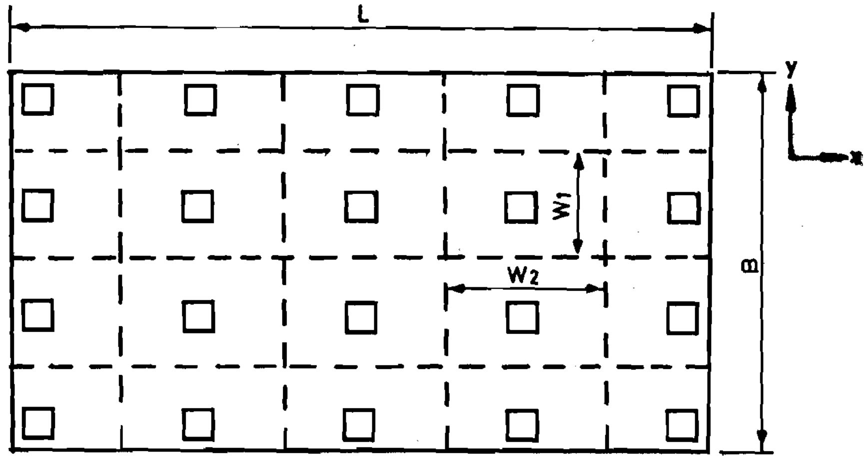


Figure 3.28 : Rigid Mat Design

The strips will have several columns and each strip is considered as a combined footing. The depth of the raft foundation is determined on the basis of punching shear as for a spread foundation and columns adjacent to the edges are likely to control the design. The raft will have a uniform depth based on the most critical section. Pattern of reinforcement are very conservative in rafts to allow for uncertainties in the design. A grid of both top and bottom reinforcement bars xx and yy directions may be placed throughout the mat.

The overdesign of raft foundations by rigid method can be attributed to three major reasons:

i) there is uncertainty in conventional analysis; even more costly sophisticated analyses have their short comings,

ii) the extra cost of overdesign of the raft foundation will generally be small relative to the total project cost, and

iii) additional safety factor is provided for the additional cost.

The raft can also be designed as a plate or beams on elastic foundation using either finite difference or finite element method. However these methods require the use of a computer.

3.12 SUMMARY

We conclude by summarizing what we have covered in this unit. We have

i) described different types of footings and the process of selecting each type,

ii) shown the need for reinforcement in footings,

iii) worked out detailed structural designs for plain concrete wall footings and reinforced concrete wall, square and rectangular footings,

iv) described the procedures for proportioning the plan dimensions of eccentrically loaded footings and combined footings, and

v) indicated the procedures for calculating the shear forces and moments in combined footings and rafts for their structural design.

3.13 ANSWERS TO SAQs

SAQ 1

i) Caisson foundation

ii) Open Caisson or well foundations

SAQ 2

i) 840 mm

ii) $\dfrac{q}{f_{ck}} = 23 \times 10^{-4}$

iii) $q = 46$ kPa

SAQ 3

i) Thick raft or well foundation

ii) $M = 25$ kN m; Shear Force = 30 kN

SAQ 4

i) Assume a square footing of dimensions. $D \times D$ where, D is the diameter of the circle and select the dimensions.

ii) $M = 432$ kN m; one way shear force = 390 kN

Punching Shear Force = 1558 kN

SAQ 5

Moments = 217 kN m; 325 kN m

One way Shear Forces = 270 kN; 150 kN

Punching shear Force = 1011 kN

SAQ 6

i) $q_1 = 625$ kPa; $q_2 = -125$ kPa

ii) e = L/18

SAQ 7

i) $q_1 = 183$ kPa; $q_2 = 117$ kPa

ii) $L = 5.56$ m; $B = 2.16$ m

SAQ 8

i) $x = 2.45$ m $2x = 4.90$

This is less than 5.5 m.

ii) $a = 0.75$ m; $b = 2.58$ m

SAQ 9

The dimensions of the rectangular combined footing required is 9.72×0.94 m. This is not suitable.