CHAPTER 13

Design of Reinforced Concrete Shallow Foundations

13.1 Introduction

This chapter on footings will cover the analysis and design aspects of reinforced concrete footings or foundations. The words **'footing'** and **'foundation'** are often used synonymously but there is a slight difference between these two terms. By the term **footing**, we mean the actual surface which comes in direct contact to the soil but the term **foundation** implies that sub-structure that actually transmits the superimposed load of the super-structure and its self-weight plus soil backfill to the soil underneath.

In this chapter, we will define shallow and deep foundations, identify the situations where isolated or combined footing will be beneficial, the gross and net bearing capacity of the soil and the minimum depth of foundation required. Further, we will look into the critical locations for moments and shear in foundations for analysis and design purposes, the base pressure distribution in soil just below the footing.

In the design of footings, we will consider **rigid footings**. The analysis and design of **flexible footings** involves the concept of **soil-structure interaction** which is beyond the scope of present context. The design portion will cover the design of plain concrete footings, isolated reinforced concrete footings, wall footings and combined footings.

13.2 Footings

Proportioning of footing dimension should be such that, more or less a **uniform base pressure is obtained** from the soil underneath. Highly non-uniform soil pressures (due to sustained eccentric loads) are hazardous due to possible tilting of the footing leading to partial or complete collapse of the structure.

Cl. 20.1 of IS 456: 2000 permits a reduced factor of safety of 1.2 against overturning if the over turning moment is entirely due to the dead loads. However, a uniform factor of safety of 1.4 should invariably be used in all loading cases.

13.3 Footing as a Structural Element

Any structure that is built on ground consists of two parts viz. part of the structure located above the ground which is generally referred to as the **superstructure** and the part which lies below ground which is referred to as the **substructure.** The elements of the superstructure transfer the loads and moments to its adjacent elements through a definite **load path** which finally comes down to the foundation. As per **Cl. 34.1** of **IS 456: 2000**, footings shall be designed to fulfil the following requirements:

1. Foundation structures must be able to sustain the applied loads, moments, and induced reactions without exceeding the safe bearing capacity of soil.
2. The settlement of the structure should be as uniform as possible and must be within the permissible limits.

Further, in addition to the above requirements, (as per **Cl. 20 of IS 456: 2000**) the foundation structure must provide adequate safety against the stability of the structure due to overturning and sliding. The design of foundation structure is little bit different from the design of other super structure elements in the following ways:

1. Foundation structures involve soil-structure interaction. Thus, the behavior of foundation depends on the properties of soil and the structural elements. Determination of soil properties is altogether a different field (Geotechnical Engineering). Understanding the interaction of soil with structure is quite difficult and hence many simplifying assumptions are made in the foundation design.
2. Precise estimation of all the types of loads, moments, and forces is required for the present and future needs. Foundation once designed and constructed is very difficult to strengthen in future.
3. Foundation structures remain under ground so involves very little architectural consideration. The foundation must be kept within the property line which may induce additional moments due to eccentricity of column coming to the foundation.
4. Foundation structures remain in direct contact with the soil and hence any harmful chemicals if present in soil may lead to deterioration of foundation. Fluctuations in ground water table are also to be taken into account while designing a foundation.
5. While constructing a foundation, adjacent structures may get affected leading from formation of cracks to complete collapse of the adjoining structure particularly during pile driving operations etc. This aspect needs to be taken care of in the design and construction of foundations.

13.4 Types of Footings

Footings are strictly the shallow foundations and NOT the deep foundations like piles and caissons and are used when soil of sufficient strength is available at a shallow depth below the ground surface/level. Shallow foundations consists of footings (which support columns, walls and have a smaller/limited area and width in plan), rafts (which support multiple columns on a large plan area). Shallow foundation has a large plan area as compared to cross-sectional area of the column(s) it supports because:

1. The loads on the columns (axial thrust, bending moments) are resisted by concrete under compression and reinforcing steel under tension and/or compression while these load effects are transferred by the footing/raft to a relatively weak supporting soil.
2. The 'safe bearing capacity' of the soil is very low (100 – 400 kPa) as compared to the permissible compressive stresses in concrete (5 – 15 MPa) and steel (130 – 190 MPa) under service loads.

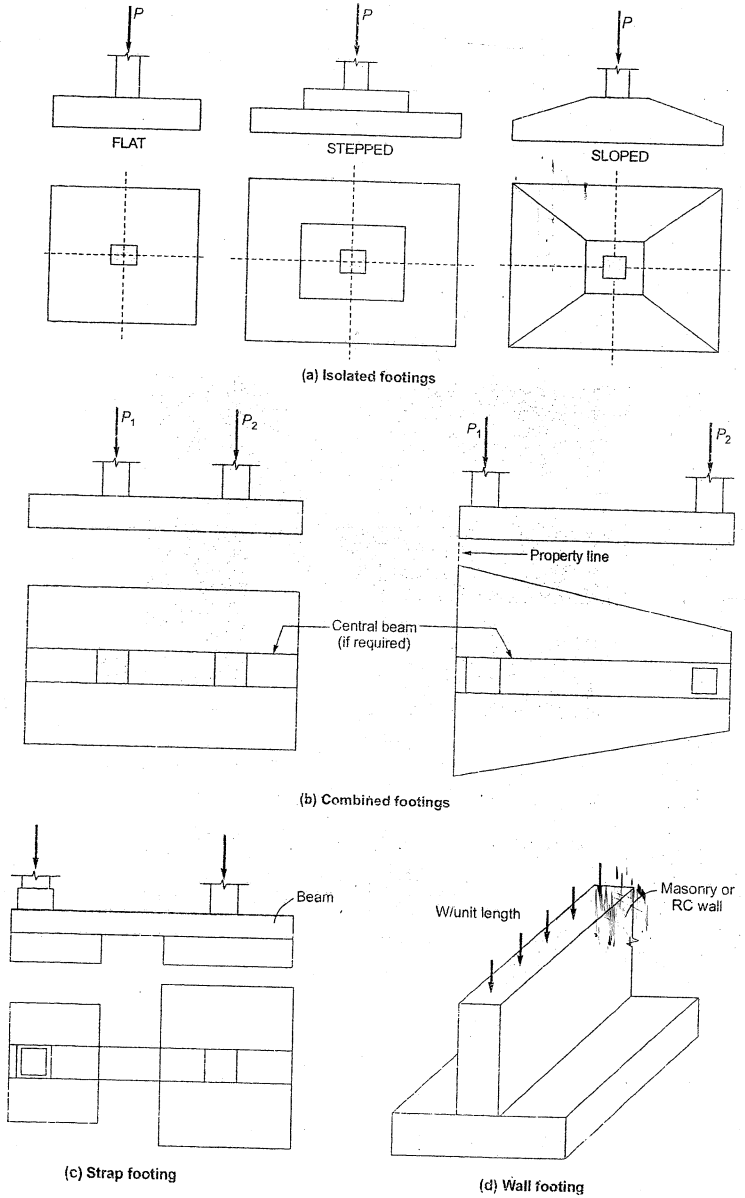


Fig.13.1 *Various types of footing*

13.4.1 Plain Concrete/Pedestal Footings

These types of footings are very much economical for columns of small loads or pedestals. (Cl. 34.12 and Cl. 34.1.3 of IS: 456-2000) (Fig. 13.2).

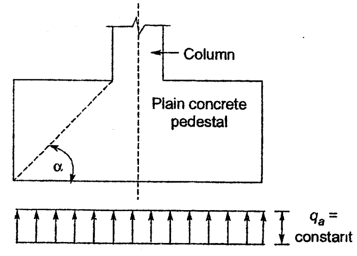


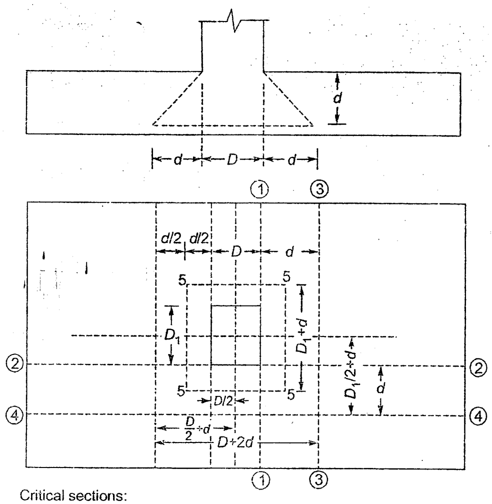
Fig.13.2 *Plain concrete pedestal*

13.4.2 Isolated Footings

For ordinary structures located on relatively firm soil, it is usually sufficient to provide individual footing for every column. Such a footing is called an **isolated footing**. Isolated footings are generally square or rectangular in plan but other shapes can also be adopted in special circumstances. The footing consists of a thick slab which may be flat (of uniform thickness), stepped or sloped (on the upper surface). Cl. 34.1.1 of IS: 456-2000 states that the sloped or stepped footings must be designed as one unit and constructed to ensure to act as a unit.

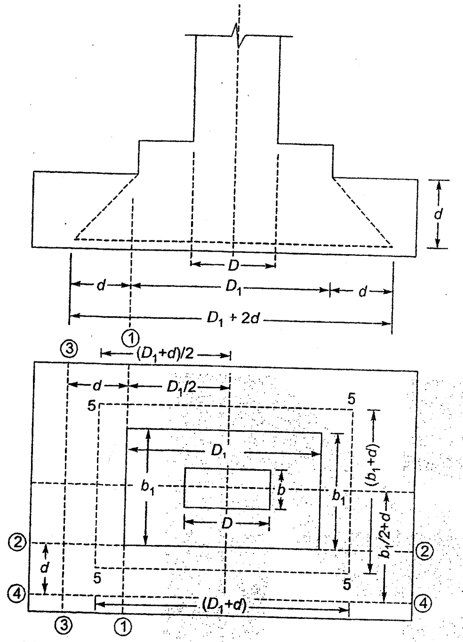
The soil bearing pressures from below the footing tend to make the base slab of the footing bend upwards and hence the footing is reinforced at the bottom of the slab.

Pedestal refers to the portion of a column below ground level where the cross-sectional dimensions are enlarged. The provision of a pedestal is not compulsory, but is often provided since it reduces the development length requirements. Pedestals are also used to support structural steel columns, the load transfer between the steel column and the concrete pedestal being achieved generally by the use of gusseted steel base plates with 'holding down' bolts.



Critical sections:
(1) For moments 1-1 and 2-2
(2) For one-way shear 3-3 and 4-4
(3) For punching shear, perimeter marked by 5-5-5-5.

Fig.13.3 *Rectangular footing*



Critical sections:
(1) For moments 1-1 and 2-2
(2) One-way shear 3-3 and 4-4
(3) Two-way punching shear marked by 5-5-5-5

Fig.13.4 *Stepped and Rectangular footing*

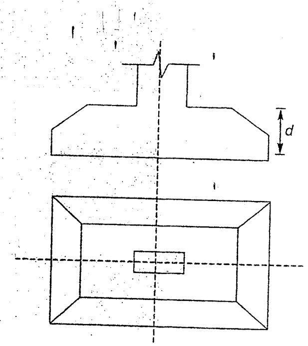


Fig.13.5 *Sloped and Rectangular footing*

13.4.3 Combined Footings

Many a times it is inconvenient to provide separate isolated footings for columns (or walls) due to insufficient areas available in plan.

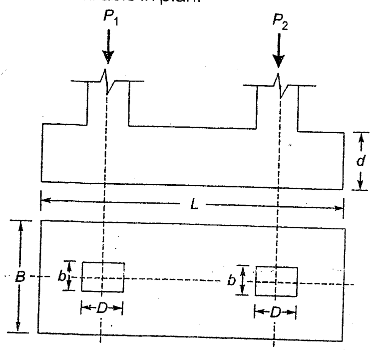


Fig13.6. *Combined footing without central beam*

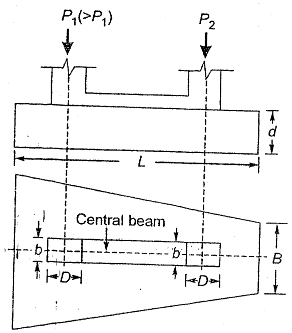


Fig.13.7 *Combined footing with central beam*

This usually occurs when two or more columns (or walls) are located close to each other or if they are relatively heavily loaded or rest on soil with low safe bearing capacity, resulting in an overlap of areas if isolated footings are provided. In such cases, it is preferred to provide a single combined footing for the columns Fig. 13.6.

At locations where there is a 'property line' which restricts the extension of the footing on the side towards the property line, in such cases, the problem of non-availability of space near the exterior column is overcome by providing combined footings Fig. 13.7.

13.4.4 Strap Footings

If the two isolated footings are joined by a strap beam then it is called strap footing. These types of footings are required where the columns carry heavy loads and individual footing areas are not overlapping. This type of footing reduces the possibility of differential settlements Fig. 13.8.

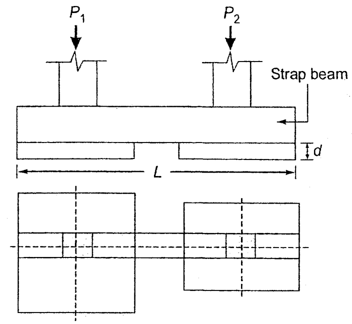


Fig.13.8 *Strap footing*

13.4.5 Wall Footings/Strip Footings

Many a times, reinforced concrete footings are required to support reinforced concrete walls and sometimes load bearing masonry walls. Wall footings distribute the load from the wall to a larger area and runs throughout the length of the wall. The footing slab bends primarily in the direction transverse to the wall (as a one-way slab) and hence it is reinforced mainly in the transverse direction, with only nominal distribution reinforcement in the longitudinal direction Fig. 13.9.

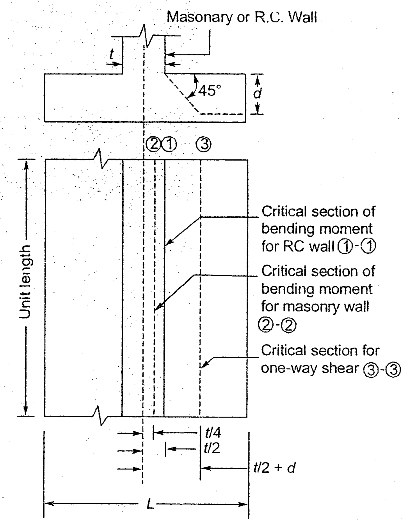


Fig.13.9 *Wall footing*

13.4.6 Raft/Mat Footings

These are special types of combined footings which support a large number of columns. This footing is provided when the column loads are heavy and the soil below is poor. Mat footing minimizes the chances of differential settlements and transfers the column loads to a very large area Fig. 13.10.

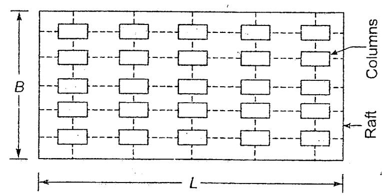


Fig.13.10 *Raft footing*

13.5 Distribution of Soil Pressure under Isolated Footings

13.5.1 Allowable Soil Pressure

The area in plan of the footing is so selected that the maximum pressure under the footing base due to dead and imposed loads of the structure (as well as weight of soil backfill and self-weight of footing) is well within the safe bearing capacity of the soil. This safe limit of soil pressure i.e. the permissible bearing capacity of the soils determined by the use of principles of soil mechanics (which is dealt in the subject of Geotechnical Engineering). Following are the main considerations in determining the safe soil pressure:

1. The loaded soil should not fail under the applied loads. Soil in fact, generally fails in shear.
2. The settlements, including both the total and differential settlements, should be well within the permissible limits.

Due to uncertainties and approximations involved in the soil mechanics problems, a safety factor in the range of 2 – 6 is generally adopted.

The safe soil bearing capacity (allowable soil pressure), q_a, provided for structural design is applicable for service load conditions since 'q_a' includes the factor of safety. Thus area of footing required must be calculated on the basis of q_a and service loads. Thus, **the partial safety factors for different load combinations will be those applicable to serviceability limit state and NOT the ultimate limit state when used along with** q_a.

The allowable soil pressure 'q_a' includes the existing overburden pressure (soil up to the foundation depth level). It is NOT the net soil pressure which is in excess of existing soil pressure. Thus total load to be calculated for footing design must include the weight of footing and that of soil backfill. Often, this value is taken as about 10-15 percent of axial column load.

Also,

Net Bearing Capacity = Gross Bearing Capacity – Pressure due to overburden soil

13.5.2 Depth of Foundation

As per **IS 1080 : 1980**, all types of foundation must have a **minimum depth of 500 mm.** This minimum depth is required to ensure the availability of soil having the safe bearing capacity assumed in the design. The foundation must be placed well below the natural or finished ground level below so that it will not be affected by swelling and shrinkage of soil. **Rankine's formula** gives a preliminary estimate of depth of foundation which is expressed as:

$$d_{min} \geq \frac{q}{\gamma}\left(\frac{1-\sin\phi}{1+\sin\phi}\right)^2$$

where, d_{min} = Minimum depth of foundation
q = Gross bearing capacity of soil
γ = Density of soil
ϕ = Angle of internal friction of soil

Rankine's formula does not take into account the loads acting on the foundation.

13.5.3 Base Pressure Distribution

The soil reaction at the base of the footing and its distribution depends on the rigidity of the footing and the soil properties i.e. it falls in the category of 'soil-structure interaction' problem. In fact the distribution of soil pressure below the footing is non-uniform, however a linear variation in soil pressure distribution below the footing is generally assumed in design.

13.5.4 Concentrically Loaded Footing

When the resultant vertical load ($P + \Delta P$) acting on the column passes through the centroid of the footing, the soil pressure is assumed to be uniformly distributed below the footing of plan area 'A' and the uniform soil pressure is given by,

$$q = \frac{P+\Delta P}{A}$$

For calculating the area of footing required,

$$A_{reqd} = \frac{P+\Delta P}{q_a}$$ where, ΔP = load due to foundation and backfill

13.5.5 Eccentrically Loaded Footing

In general, the line of action of column load does not coincide with the centroid of footing i.e. the line of action of load is often eccentric to the footing centroid, which gives rise to moments in the footing. Eccentricity in loading results due to the following reasons/causes:

1. The column transfers a moment in addition to axial load to the footing.
2. The line of action of column load does not pass through the centroid of footing.
3. When the column carries a lateral force located above the foundation level.

In practice, generally bi-axial eccentricities exist i.e. eccentricity in two principal, orthogonal, centroidal axes of the footing.

For finding the base pressure under the footing in eccentric load conditions, the footing is assumed to be rigid and linear contact pressure distribution under the footing is assumed. The magnitude of pressure distribution is determined using the principles of static equilibrium of the footing. This implies that the center of soil pressure (through which the resultant soil reaction R acts) must be collinear with the resultant line of action of eccentrically applied load, i.e. $R = P + \Delta P$. For preliminary design, ΔP, the weight of the footing plus soil backfill is taken generally as 10-15% of P.

Case 1: $|e| \leq L/6$

If the resultant eccentricity 'e' (= $M/(P+ \Delta P)$) lies within the one-third of the footing, then entire area of contact of the footing is subjected to a non-uniform base pressure distribution which varies linearly from q_{min} to q_{max}. These are obtained as:

$$q_{max,min} = \frac{P+\Delta P}{A} \pm \frac{(P+\Delta P)e}{Z}$$

where, $A = BL$ and the section modulus $Z = BL^2/6$. L is the length of the footing in the direction of eccentricity 'e'.

From the above expression, it can be deduced that:

$$q_{max,min} = \frac{P+\Delta P}{A}\left(1 \pm \frac{6e}{L}\right)$$

when $e = L/6$, then $q_{min} = 0$ and $q_{max} = 2(P + \Delta P)/A$. This results in a triangular base pressure distribution. Limiting case of $e = L/6$ is valid only for uniaxial bending. For a general case of biaxial bending, the limiting case is taken as:

$$\frac{e_x}{\frac{L_x}{6}} + \frac{e_y}{\frac{L_y}{6}} \leq 1$$

Case 2: $|e| > L/6$

When the eccentricity 'e' is greater than $L/6$, then the expression of Case 1 yields negative pressure distribution i.e. value of q_{min} is negative. This negative pressure in soil i.e., tensile capacity is not possible in soil. In this case, the soil tends to separate from the footing thereby offering no pressure to the footing. For this case, q_{max} is given by:

Here,
$$q_{max} = 2(P + \Delta P)/BL'$$
$$L' = 3c \text{ and } c = 0.5L - e$$

Thus effective length of contact gets reduced from L to $L' = 3c$ and the maximum soil pressure q_{max} gets increased to $2(P + \Delta P)/BL'$. In order that the value of q_{max} does not exceed allowable soil pressure q_a and also to maximize the effective bearing length, it is essential to design the footing with large base area.

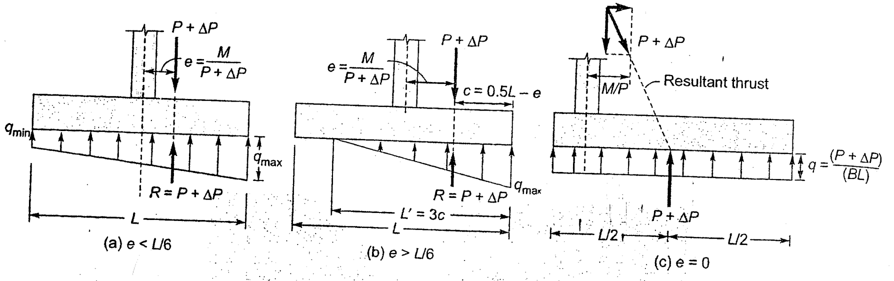


Fig.13.11 *Base pressure distribution under rectangular footing (assumed linear distribution)*

13.6 Footing Design: General Requirements and Codal (IS 456 : 2000) Provisions

13.6.1 Factored Soil Pressure at Ultimate Limit State

The base area of the footing is determined using the value of safe bearing pressure 'q_a' and applied loads and moments under the **service load conditions**. Once we are having the footing area which is in fact fixed now, the subsequent structural design of footing is done on the basis of **factored loads at ultimate limit state**. Thus for computing the values of bending moment, shear force etc. at critical locations under the factored load conditions, an **imaginary factored soil pressure** 'q_u' corresponding to factored loads is computed.

In reality, the moments and shear induced in the footing are due to net soil pressure, q_{net} (excluding the soil over burden pressure). This net pressure arises due to concentrated load carried by the column and the moments at the column base.

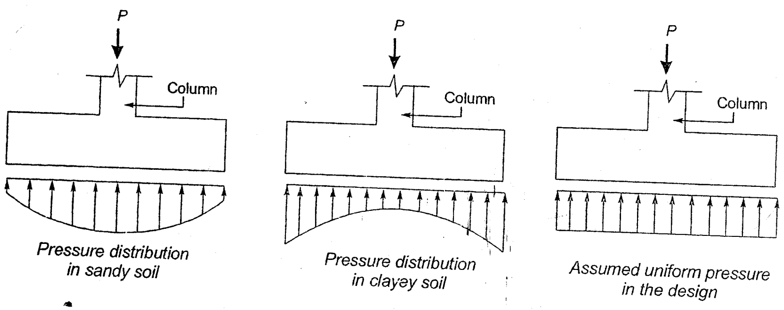


Fig.13.12 *Soil pressure distribution in sandy and clayey soils*

13.6.2 Design Considerations

In footing design, the major design considerations are as listed below:

1. Bending moments
2. Shear force (both one way and two way shear)
3. Bearing stress
4. Bond/development length
5. Other considerations include the transfer of force from column to footing and where horizontal forces are also involved, there; safety against the possible sliding and possible over turning needs to be adequately ensured. Deflection control is not considered in the design of footings which are buried under ground.

13.6.3 Thickness of Footing

The thickness of footing depends on considerations of shear force and bending moments which are critical near the column locations on the footing. Generally, shear consideration decides the thickness of footing. As per **Cl. 34.1.2 of IS: 456**, minimum thickness of footing at the edges must not be less than **150 mm** (300 mm in case of pile caps). This is done to ensure that the footing has sufficiently high rigidity to provide the calculated bearing pressures.

13.6.4 Minimum Nominal Cover

The minimum nominal cover to footings should be more than that of other structural elements of the super structure since footings remain in direct contact with the soil. As per **Cl. 26.4.2.2 of IS: 456-2000,** footings must have a minimum cover of **50 mm**. Actual cover may be more than that depending on soil conditions and presence of harmful chemicals in the soil.

13.6.5 Shear Consideration in Footing Design

The depth of footing is mostly governed by the shear requirement. In the design of footing, usually the design for shear supersedes the flexural design of footing. Both 'one-way shear' and 'two-way shear' need to be considered for footing design. The critical section for one way shear is located at a distance 'd' (effective depth) from the column face.

One way shear: (**Cl. 34.2.4 of IS: 456-2000**) One way shear has to be checked across the full width of the base slab on a vertical section located from the face of column or pedestal at a distance equal to:

1. Effective depth of footing slab in case of footing slab on soil.
2. One half of the effective depth of footing slab if footing slab is on piles.

The design shear strength of concrete without shear reinforcement is given in **Table 19 of IS: 456-2000.**

Two way shear/Punching shear: (**Cl. 31.6 and Cl. 34.2.4 of IS: 456-2000**) Two way shear should be checked around the column perimeter at a distance equal to half the effective depth of the footing slab away from the column face or pedestal.

The permissible shear stress when shear reinforcement is not provided shall not exceed $k_s\tau_c$ where, $k_s = (0.5 + \beta_c) \leq 1$, β_c being the ratio of short side to long side of the column and $\tau_c = 0.25\sqrt{f_{ck}}$.

In general, the depth of footing is governed by shear considerations and adequate depth of footing must be provided to avoid shear reinforcement.

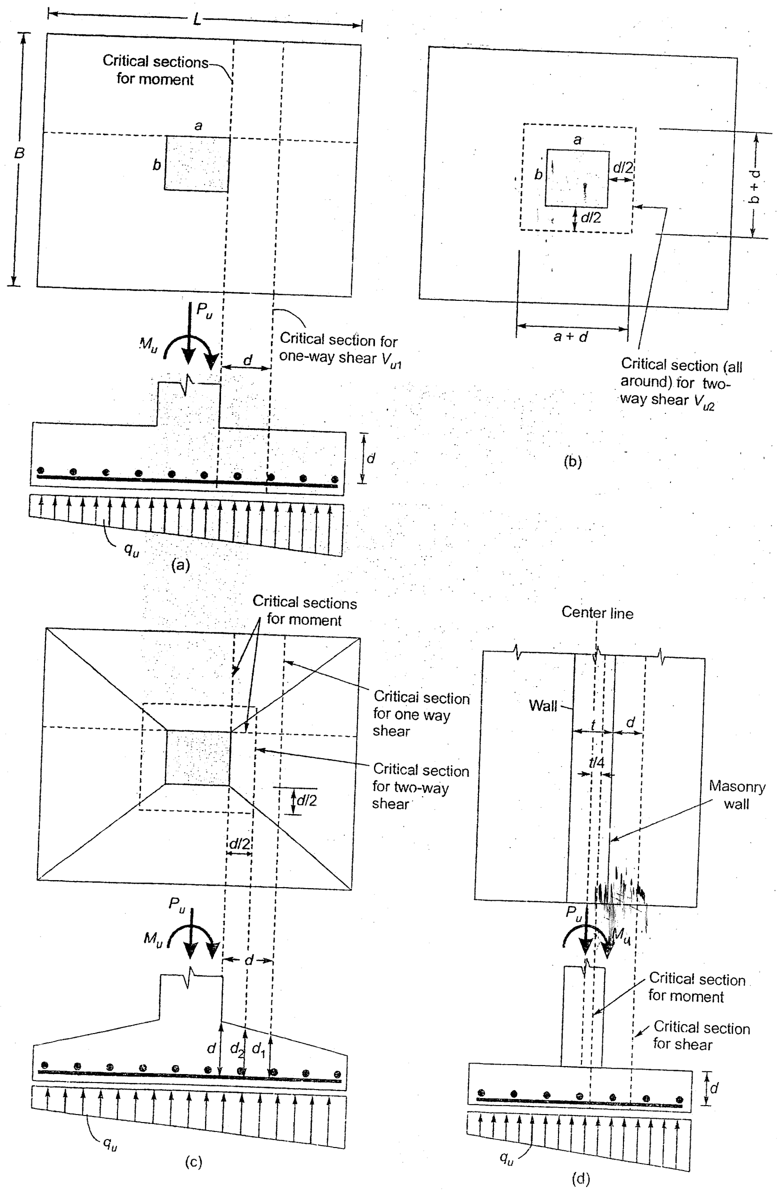


Fig.13.13 *Critical sections for shear and moment in footings*

The behavior of footings in two way shear is identical to flat two way slabs on columns. The critical section for two way shear is taken at a distance 'd/2' from the column face. Shear reinforcement is usually NOT provided in footings and factored shear force (V_u) is kept well below the factored shear resistance (V_{uc}) of concrete by providing the sufficient depth of footing.

Remember For locating the critical sections for shear and moments in columns having circular/octagonal cross section, an equivalent square section is considered. The equivalent square must be inscribed within the perimeter of the column section.

13.6.6 Flexural Moments Consideration in Footing Design

The footing base slab tends to bend upwards due to net soil pressure from below the footing base as shown in the figure below. In fact the footing base bends like a saucer shaped element. For this, as per Cl. 34.2.3.2 of IS: 456, the footing can be designed for flexure by considering the critical section for moments as a straight section passing through:

1. The face of column, pedestal or wall for footing supporting a concrete column, pedestal or wall;
2. Half-way between the wall face and wall center line for masonry walls;
3. Halfway between the face of the column or pedestal and the edge of the gusseted base for footings under gusseted bases.

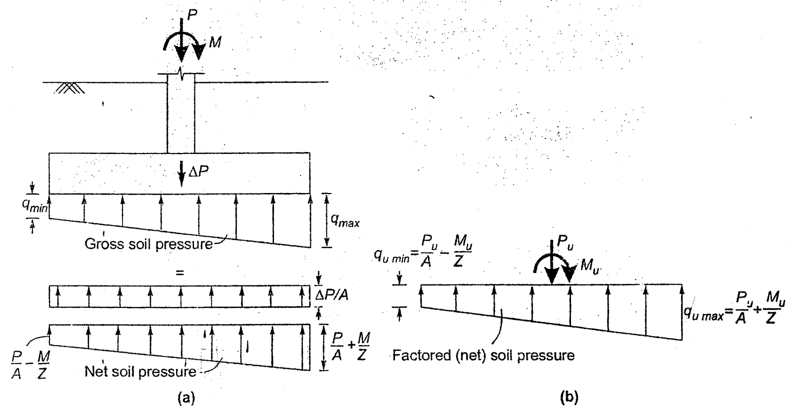


Fig.13.14 *Net soil pressure below footings*

In footings, which are mainly reinforced in one direction only i.e. in wall footings etc., the main reinforcement is placed perpendicular to the wall at a uniform spacing. In the longitudinal direction of the wall, nominal distribution reinforcement is provided generally to account for secondary moments due to Poisson's effect, shrinkage and temperature effects.

In two way reinforced concrete footings (square footing, rectangular footing etc.), flexural reinforcement is provided in both the orthogonal directions at a uniform spacing. For two-way reinforced concrete rectangular footing, the reinforcement in long direction is uniformly spaced but the reinforcement in the short direction is distributed in the central band and the end bands. Central band requires larger concentration of reinforcement than the end bands. The width of this central band is equal to width of the footing (B) as shown in **Fig. 13.15**.

Also reinforcement in the central band is given by as per Cl. 34.3.1 of IS 456 as,

$$A_{st,short} = \frac{2}{\beta+1}.$$

(Reinforcement in central band)

Where, $A_{st,\ short}$ = Total reinforcement required in the short direction

$\beta = L/B$ i.e. ratio of long to short side of the footing.

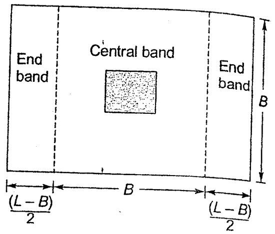


Fig.13.15 *Bands for reinforcement in a rectangular footing*

The remaining reinforcement is distributed in the end bands equally. These reinforcement detailing requirements are invariably intended for footings of uniform thickness. In sloped footings, it is usually sufficient to provide uniformly distributed reinforcement in the short direction. Due to large thickness of footing provided for shear considerations, the consequent required percentage of flexural reinforcement in footings is generally very low. But in any case, the percentage of flexural reinforcement should not be less than that prescribed for slabs.

It is beneficial to use small diameter bars at small spacing to prevent cracks in concrete and also to have sufficient development length. Development length requirement for flexural reinforcement must be satisfied at critical sections of moments. Any short fall (i.e. shortage) in the development length can be made by bending the bars up near the edges of the footing.

Further, the longitudinal reinforcement in the column or pedestal must also have the sufficient development length which is measured from the interface of the column/pedestal and the footing. When the column carries compressive load only (i.e. bars are not in tension), it is possible to have full transfer of forces from column/pedestal to footing by the action of bearing alone. Where the bars are in tension, adequate development length has to be provided at the column/pedestal – footing interface.

13.6.7 Bond Consideration in Footing Design

The critical sections for checking the development length in footings shall be the same as those for flexure/moments. However, development length must be checked at all the critical locations where the section changes abruptly. Cl. 34.2.4.3 of IS 456: 2000 specifies the critical section for checking the development lengths. This further specifies to check the anchorage requirements in case of curtailment of reinforcement. Curtailment shall be done according to Cl. 26.2.3 of IS 456: 2000.

13.6.8 Tensile Reinforcement

Tensile reinforcement should be calculated as per moments at critical locations:

1. For one way reinforced concrete footing slabs like wall footings, reinforcement shall be distributed evenly across the full width of the footing i.e. normal to the direction of wall. Nominal distribution reinforcement shall also be provided as per Cl. 34.5 of IS 456: 2000 along the length of wall in order to account for secondary moments, temperature effects, shrinkage and differential settlements.
2. In two way reinforced concrete square footings, reinforcement should extend in both the directions distributed uniformly across the full length and width of the footing.
3. In two way reinforced concrete rectangular footings, reinforcement in the long direction should be distributed evenly across the full width of the footing. In the short direction, a central band of width equal to width of the footing is marked along the length of the footing. In the central band, reinforcement is distributed as:

$$\text{Reinforcement in central band} = \frac{2}{(\beta+1)} \times \text{Total short direction reinforcement}$$

where, β = Ratio of long side to short side of the footing.

In the other two end bands, half of the remaining reinforcement should be provided uniformly distributed.

13.6.9 Force Transfer at the Column Base

All forces (axial force, lateral force and moments) acting on the column are required to be transferred to the footing either by compression of concrete or by tension/compression of reinforcing steel. But for the case of force transfer by compression of concrete, one cannot transfer a force larger than bearing resistance of concrete for either surface (i.e. supported surface or supporting surface). Under the action of factored loads, as per Cl. 34.4 of IS: 456, the maximum bearing resistance should not exceed the value as given below:

$$f_{br,\ max} = 0.45 f_{ck}\sqrt{\frac{A_1}{A_2}}$$

where, A_2 = loaded area at the column base

A_1 = maximum area of the portion of supporting surface that is geometrically similar to and concentric with the loaded area.

The significance of the factor $\sqrt{\frac{A_1}{A_2}}$ lies in the fact that it allows for increase in concrete strength in the bearing area in the footing due to confinement provided by the surrounding concrete and maximum value of this factor is limited to 2. This limitation is imposed because very high axial compressive stress gives rise to transverse tensile strain (due to Poisson's effect) which leads to spalling of concrete, lateral splitting of concrete or even bursting of concrete. Cl. 34.4 of IS 456 limits the permissible bearing stress of concrete to $0.25f_{ck}$ in case of working stress method and $0.45f_{ck}$ in limit state method of design.

The bearing resistance of concrete in column at the interface (for which $\sqrt{\frac{A_1}{A_2}} = 1$) is the governing factor rather than bearing resistance of concrete in footing at the interface (for which $1 < \sqrt{\frac{A_1}{A_2}} <= 2$). If the actual compressive stress exceeds $f_{br,\ max}$ then the excess force is transferred by the reinforcement, dowels or mechanical connectors (Cl. 34.4.1 and Cl. 34.4.2 of IS 456). For transferring moment at the column base (reinforcement is in tension), it is necessary to provide same reinforcement in footing as in the column. The reinforcement provided at the interface must consists of at least four bars with a total area not less than 0.5% of the cross sectional area of the supported column/pedestal (Cl. 34.4.3 of IS 456). Where dowels are used, the diameter of dowel bars shall not exceed the diameter of column bars by 3 mm.

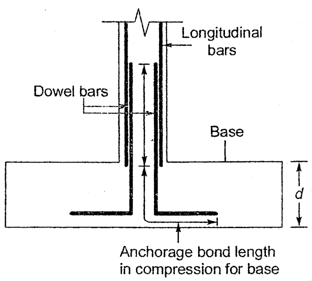


Fig.13.16 *Anchorage length of dowels*

Column bars of diameter greater than 36 mm, in compression only can be doweled at the footing with bars of smaller diameter of required area. The dowel shall extend into the column a distance equal to development length of the column bar and into the footing, a distance equal to the development length of the dowel as per Cl. 34.4.4 of IS 456: 2000.

13.6.10 Nominal Reinforcement

1. Cl. 34.5.1 of IS 456: 2000 gives minimum reinforcement and spacing of bars in footing as per the requirements of solid slab.
2. The nominal reinforcement of concrete sections of thickness greater than 1000 mm shall be 360 sq. mm per meter length of footing in each direction on each face as per Cl. 34.5.2 of IS 456: 2000.

13.7 Plain Concrete Footings

It is also called as pedestal footing and is used where the column load is very low and the footing area required is also low. If the bearing stress at the column base is less than the permissible bearing stress of concrete i.e. $f_{br,max} = 0.45 f_{ck}\sqrt{A_1/A_2}$ then the force transfer from the column base to the footing can be achieved without any need of reinforcement at the interface of column and the footing. Also, if the base area of the footing falls within a certain confined zone of load dispersion through the footing, then the entire column load can be transferred to the footing by 'strut action' (i.e. by compression) and the soil pressure does not induce any bending in the footing. These imaginary struts are inclined to the vertical whose horizontal component will call for a strut-tie like action as shown in Fig. 13.7. To carry the tie forces and to avoid the possible cracking of concrete due to tensile stresses, it is essential to provide some minimum reinforcement to serve as ties.

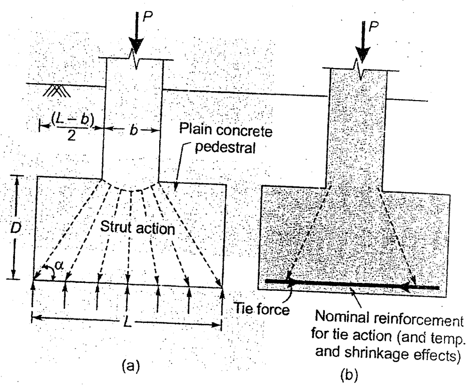


Fig.13.17 *Tie action in plain concrete footing*

For the purpose of defining this confined zone of load dispersion and thus to determine the thickness of footing required, Cl. 34.1.3 of IS: 456 defines an angle α between the plane passing through the bottom edge of the footing and the corresponding edge of the column at the interface. This angle α is defined as:

$$\tan\alpha > 0.9\sqrt{\frac{100 q_{max}}{f_{ck}} + 1}$$

where, q_{max} is the calculated maximum soil pressure at the base of the pedestal. Thus the thickness D of the footing required is: $D = (L - b)\tan\alpha/2$

Example 13.1 Design a plain concrete footing for a column of size 400 mm × 400 mm carrying an axial load of 350 kN as service loads consisting of dead and imposed loads. The allowable soil bearing pressure is 370 kN/m² at a depth of 1 m below the natural ground level (NGL). Use M 25 concrete and Fe 415 steel.

Solution:

Axial force transfer at the column base:

In case of plain concrete footing, full transfer of axial load at the column base must occur without any requirement of reinforcement at the interface of column and footing.

Factored load P_u = 1.5 × 350 kN = 525 kN

Limiting bearing stress at the column footing interface = $f_{br,max} = 0.45 f_{ck}\cdot\sqrt{(A_1/A_2)}$

Now bearing stress in this case is governed by the column face and not the footing face.

Thus $A_1 = A_2 = 400 \times 400\ \text{mm}^2 = 160000\ \text{mm}^2$

$$f_{br,max} = 0.45 \times 25 \times \sqrt{\frac{160000}{160000}} = 11.25\ \text{N/mm}^2$$

Limiting bearing resistance = F_{br} = 11.25 × 400² N = 1800 kN > Factored load P_u (= 525 kN)

Thus full transfer of load from column to footing is possible without any reinforcement.

Calculation of the size of the footing required:

Let the weight of the footing and the backfill is about 10% of the column axial load i.e. 10% of 350 kN = 35 kN.

Thus base area required = (350 + 35)/370 = 1.0405 m²

Provide a footing of size 1.25 m × 1.25 m.

Thus footing area provided = 1.25 × 1.25 m² = 1.5625 m² > 1.0405 m²

Calculation of the depth of the footing required:

Footing depth D = (1/2) × (1.25 – 0.4) × tanα

where $\tan\alpha \geq 0.9\sqrt{\left(\frac{100 q_{max}}{f_{ck}}\right) + 1} = 0.9\sqrt{\frac{100 \times 0.370}{25} + 1} = 1.41732$

(Here q_{max} = 370 kN/m² = 0.37 N/mm²)

Thus D = (1/2) × (1.25 – 0.4) × 1.41732 = 0.602361 m = 602.361 mm

Provide a footing depth of 650 mm.

Thus required footing will be of size **1250 mm × 1250 mm × 650 mm**

Minimum reinforcement requirement:

A minimum amount of reinforcement @ 0.12% of gross area is always provided to ensure tie-action in the footing and to account for temperature and shrinkage effects.

Thus $A_{st,min}$ = 0.0012 × 1250 × 650 mm² = 975 mm²

Provide 5 nos. 16 mm diameter bars ($A_{st,provided}$ = 1005.3 mm² > 975 mm²) in both the directions with a clear cover of 75 mm.

Thus spacing of bars = (1250 mm – 2 × 75 mm – 5 × 16 mm)/4

= 255 mm < 5d (= 5 × (650 – 75) = 2875 mm) < 450 mm (OK)

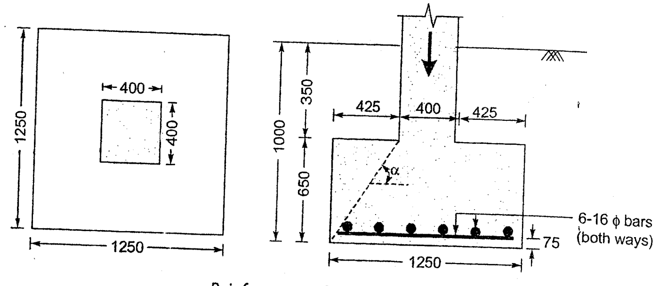
Check for soil base pressure:

Let unit weight of soil as 19 kN/m³ and that of concrete as 25 kN/m³.

Gross soil base pressure = 350/(1.25 × 1.25) + (25 × 0.65) + (19 × (1 – 0.65))

= 224 + 16.25 + 6.65

= 246.9 kN/m² < 370 kN/m² (OK)



Reinforcement detailing

13.8 Design of Rectangular Isolated Footing

Given: Column load $= P$

Safe bearing capacity of the soil $= q_o$

Grade of concrete $= f_{ck}$

Grade of steel $= f_y$

Design Steps: For both WSM and LSM

Assume weight of foundation $= P_F = 10\%$ of $P = 0.1P$

Total weight on soil $= P_T = P + 0.1\,P = 1.1P$

$$\text{Area of foundation required} = 1.1\frac{P}{q_0} \le L \times B$$

L and B should be decided suitably. (Preferably in the ratio of column size)

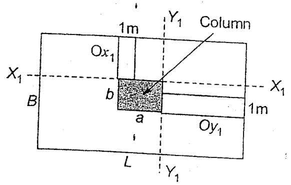
$$\text{Net pressure on soil} = w_o = q_0 - \frac{P_F}{A} = \frac{P_T}{A} - \frac{P_F}{A} = \frac{P_T - P_F}{A}$$

$$= \frac{1.1P - 0.1P}{A} = \frac{P}{A} = \text{Design soil pressure}$$

Factored soil pressure (for LSM) $= 1.5w_o = 1.5(P/A)$

Depth of footing required by bending moment requirement:

Critical section for moment is as shown above.



At critical section $X_1 - X_1$: Overhang $= OX_1 = \dfrac{B-b}{2}$

$$Mx = w_o(OX_1)1\text{m}\,\frac{OX_1}{2}$$

$$= w_o\left(\frac{B-b}{2}\right)\left(\frac{B-b}{4}\right) = \frac{w_o(B-b)^2}{8} \quad \text{(For both WSM and LSM)}$$

At critical section $Y_1 - Y_1$: Overhang $= OY_1 = \left(\dfrac{L-a}{2}\right)$

$$My = w_o(OY_1)\,1\text{m}\left(\frac{OY_1}{2}\right) = w_o\left(\frac{L-a}{2}\right)\left(\frac{L-a}{4}\right)$$

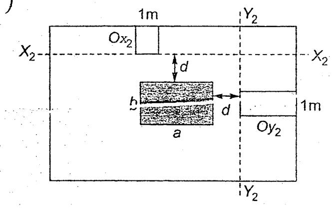
$$= w_o\frac{(L-a)^2}{8} \quad \text{(For both WSM and LSM)}$$

$$\text{Design depth of footing} = d = \sqrt{\frac{M_{max}}{QB}} \quad \text{where } B = 1000 \text{ mm}$$

Check for shear: Depth calculated as per moment criteria is checked for one way and two way shear.

$$\text{Shear at critical section } X_2 - X_2 = w_o(1\text{ m})\,OX_2 = w_o\left(\frac{B-b}{2} - d\right)$$

$$\text{Shear at critical section } Y_2 - Y_2 = w_o(1\text{ m})\,OY_2 = w_o\left(\frac{L-a}{2} - d\right)$$



Nominal shear stress for maximum shear force:

$$\tau_v = \frac{V_y}{Bd} \text{ (For WSM)}$$

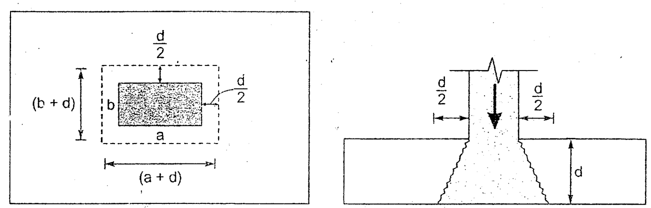
$$\tau_{uv} = \frac{V_{uy}}{Bd} \text{ (For LSM)}$$

Shear stress developed $(\tau_v) < k \cdot \tau_c$ where k is a factor depending on slab thickness

The values of k as per **Cl. 40.2.1.1 (LSM) and B.5.2.1.1 (WSM) of IS 456: 2000** are as given below:

Depth of Slab (mm)	≤ 150	175	200	225	250	275	≥ 300
k	1.3	1.25	1.2	1.15	1.1	1.05	1.0

Check for Punching Shear



- The critical section for two way shear is at a distance $\frac{d}{2}$ from the column face.
- The area around the column gets punched through the soil.

$$\text{Net punching force} = P - w_o(a+d)(b+d)$$

$$\text{Resisting cross sectional area} = 2[(a+d) + (b+d)]d$$

$$\boxed{\text{Punching shear stress} = \frac{P - w_o(a+d)(b+d)}{2[(a+d)+(b+d)]d}}$$

$$\text{Permissible shear stress } (\tau_{vp}) = \begin{cases} k_s(0.25)\sqrt{f_{ck}} & \text{(LSM)} \\ k_s(0.16)\sqrt{f_{ck}} & \text{(WSM)} \end{cases}$$

where, $$k_s = 0.5 + \frac{b}{a} \le 1.0$$

Area of steel required: Maximum depth required from the above three criteria is adopted as design footing depth (d).

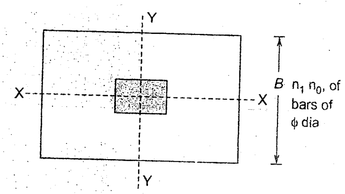
Thus overall footing depth (D) = d + effective cover = d + clear cover + $\frac{1}{2}$(bar dia.)

Area of steel along shorter side $Y - Y = A_{st}$

$$= \frac{M_x}{\sigma_{st} jd} \quad \text{(For WSM)}$$

$$A_{st} = \frac{M_x}{0.87 f_y jd} \quad \text{(For LSM)}$$

$$= \frac{0.5 f_{ck}}{f_y}\left[1 - \sqrt{1 - 4.598\frac{M_x}{Bd^2 f_{ck}}}\right]Bd$$



Total area of steel along shorter side $= LA_{st}$

$$\text{Number of bars required } (n_T) = \frac{A_{st} L}{\frac{\pi}{4}\phi^2}$$

$$\text{No. of bars to be provided in central band} = \frac{2n_T}{1 + \frac{L}{B}}$$

Remaining reinforcement shall be distributed equally in the end bands.

Area of steel along longer side $X - X = A_{st}$

$$= \frac{M_x}{\sigma_{st} jd} \quad \text{(For WSM)}$$

$$A_{st} = \frac{M_x}{0.87 f_y jd} \quad \text{(For LSM)}$$

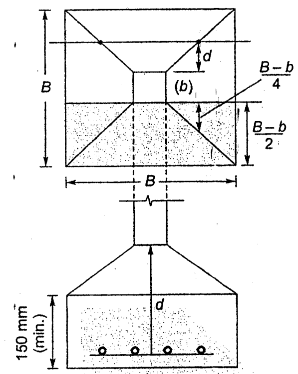
$$= \frac{0.5 f_{ck}}{f_y}\left[1 - \sqrt{1 - 4.598\frac{M_x}{Bd^2 f_{ck}}}\right]Bd$$

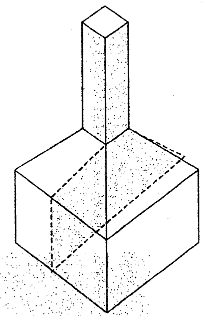
Total area of steel $= BA_{st}$

$$\text{No. of bars required} = \frac{A_{st} B}{\frac{\pi}{4}\phi^2}$$

This reinforcement is placed uniformly.

13.9 Design of Sloped Isolated Footing

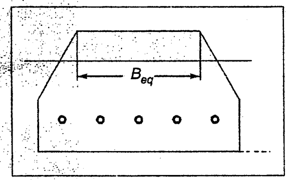




Minimum thickness of footing at the ends = 150 mm

Usually 200 mm is a sufficient value.

Size of foundation: Same as isolated foundation



Check for bending moment:

Critical section is at the face of column.

$$M_x = w_o B \cdot \left(\frac{B-b}{2}\right) \cdot \left(\frac{B-b}{4}\right)$$

$$= w_o B \frac{(B-b)^2}{8}$$

$$d = \sqrt{\frac{M_x}{QB_{eq}}}$$

Approximately, $$B_{eq} = b + \frac{B-b}{8}$$

Check for one way shear:

Shear force at critical section is given by:

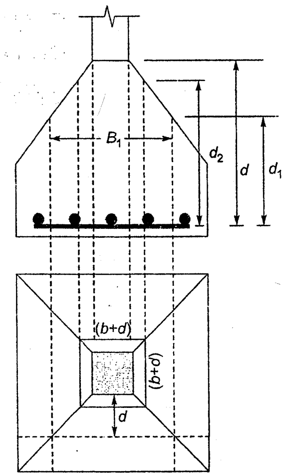
$$V_x = w_o B\left(\frac{B-b}{2} - d\right)$$

Nominal shear stress is given by:

$$\tau_v = \frac{V}{B_1 d_1} \le \tau_c$$

Here B_1 = Width of section at critical section for shear

d_1 = Depth at critical section for shear



Check for two way/punching shear

Punching shear stress developed = Net punching force / Resisting area

$$= \frac{P - w_o(b+d)^2}{4(b+d)d_2}$$

Here, d_2 = Depth of section at critical section for punching shear

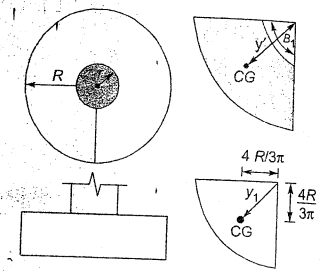
τ_v (developed) $\leq \tau_{vp}$ (permissible) = $k_s(0.16)\sqrt{f_{ck}}$ (as per WSM)

$$= k_s(0.25)\sqrt{f_{ck}} \text{ (as per LSM)}$$

Area of steel: Same as that for isolated footing.

13.10 Design of Circular Isolated Footing of Uniform Thickness

Area of footing: Same as that for isolated footing.



$$A = \frac{P_T}{q_0}$$

$$\text{Radius of footing } (R) = \sqrt{\frac{A}{\pi}}$$

$$\text{Net soil pressure } (w_o) = \frac{P}{A}$$

Check for bending moment: Consider one quadrant of circle. Bending moment is calculated at the face of column for one quadrant. Distance of CG of quadrant from centre is given by:

$$y_1 = \sqrt{2}x\frac{4R}{3\pi} = 0.6R$$

Distance of CG of shaded area from centre is given by:

$$y' = \frac{A_1y_1 - A_2y_2}{A_1 - A_2} = \frac{\frac{\pi}{4}R^2x0.6R - \frac{\pi}{4}r^2x0.6r}{\frac{\pi}{4}R^2 - \frac{\pi}{4}r^2} = 0.6\frac{R^3 - r^3}{R^2 - r^2}$$

$$\text{Distance of CG from the face of column} = y' - r = 0.6\left(\frac{R^3 - r^3}{R^2 - r^2}\right) - r$$

Bending Moment:

$$M = w_o(\text{Shaded area})(y' - r) = w_o\frac{\pi}{4}(R^2 - r^2)\left[0.6\left(\frac{R^3 - r^3}{R^2 - r^2}\right) - r\right]$$

Thus $$d = \sqrt{\frac{M}{QB_1}}$$

where, $$B_1 = \frac{2\pi r}{4}$$

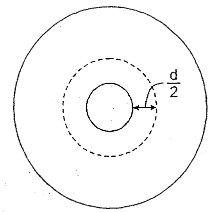
Check for one way shear

Shear force = $V = w_o \times$ Area of shaded portion

$$V = w_o x \frac{\pi}{4}[R^2 - (r+d)^2]$$

$$\text{Nominal shear stress} = \tau_v = \frac{V}{B_2 d} \quad \text{where, } B_2 = \frac{2\pi(l+d)}{4}$$

Check for punching/two way shear



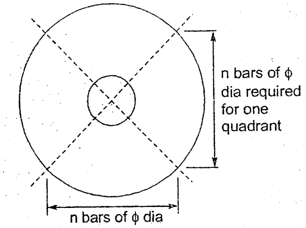
Critical section for two way shear

Net punching shear force is given by:

$$P_{net} = P - w_o\pi\left(r + \frac{d}{2}\right)^2$$

Punching shear stress is given by:

$$\tau_{vp} = \frac{P_{net}}{\text{Perimeter} \times d} = \frac{P - w_o\pi\left(r + \frac{d}{2}\right)^2}{2\pi\left(r + \frac{d}{2}\right)d}$$



Area of steel required

Area of steel for one quadrant is given by:

$$A_{set} = \frac{M_u}{\sigma_{st}jd} = \frac{M_u}{0.87f_yjd}$$

$$= \frac{0.5f_{ck}}{f_y}\left[1 - \sqrt{1 - 4.598\frac{M_u}{f_{ck}B_1d^2}}\right]B_1 d$$

13.11 Design of Wall

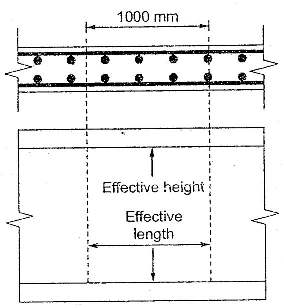
Two types of reinforcement are provided in walls viz. vertical and horizontal reinforcement.

Vertical Reinforcement: Minimum vertical reinforcement should be as given below:

(a) 0.12%of total gross area, diameter of bars ≤ 16 mm for Fe 415 and higher grades of steel.

(b) 0.15%of the total gross area for other types of steel

(c) 0.12% of total gross area for welded wire fabric



Horizontal Reinforcement: Minimum vertical reinforcement should be as given below:

(a) 0.2% of total gross area for *Fe* 415 and higher grades of steel.

(b) 0.25% of total gross area for mild steel.

(c) 0.2% of total gross area for welded wire fabric

Design is same as that of column.

Example 13.2 Design a spread footing to carry an axial load of 1200 kN through a column of size 450 mm × 450 mm having 4-25Φ bars. Bearing capacity of soil is 105 kN/m². Assume footing to be 1.25 m below ground level and concrete of grade M25 and Fe415 steel.

Solution:

$$\text{Axial load } (P) = 1200 \text{ kN}$$

Assuming 10% of column load as weight of footing and soil backfill.

$\therefore$ Weight of footing and soil backfill $= 10\%$ of $1200 \text{ kN} = 120 \text{ kN}$

$\therefore$ Total axial load $= 1200 + 120 = 1320 \text{ kN}$

$$\therefore \quad \text{Area of footing required} = \frac{1320}{105} = 12.57 \text{m}^2$$

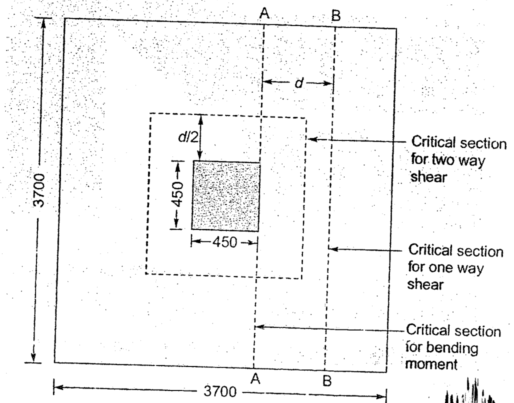
$\because$ Column section is square of size 450 mm × 450 mm

$\therefore$ Provide square footing of side $= \sqrt{12.57} = 3.55\text{m}$

i.e., provide square footing of size 3.7 m × 3.7 m.

Calculation of bending moment

$$\text{Soil pressure acting upwards} = \frac{1.5 \times 1200}{3.7 \times 3.7} = 131.48 \text{ kN/m}^2$$



Section $(A)-(A)$ is the face of column which is critical section for bending moment.

$$\therefore \quad M_u = 131.48 \times 3.7\left(\frac{3.7-0.45}{2}\right)\left(\frac{3.7-0.45}{2}\right)\frac{1}{2} = 642.3 \text{ kNm}$$

Depth of footing required

$$M_u = 0.138 f_{ck} bd^2 \quad \text{. . . (for Fe415 steel)}$$

$$\Rightarrow \quad 642.3 \times 10^6 = 0.138(25)3700 d^2$$

$$\Rightarrow \quad d = 224.315 \text{ mm}$$

Adopt overall depth of footing as 500 mm so that effective depth of footing (d),

$$d = 500 - 75 - \frac{20}{2} = 415 \text{ mm}$$

(Assuming 20 mm diameter bars and 75 mm clear cover)

Reinforcement required for footing

$$D = 500 \text{ mm}$$

$$d = 415 \text{ mm}$$

$$R = \frac{M_u}{bd^2} = \frac{642.3 \times 10^6}{3700(415)^2} = 1.00795$$

$$\frac{p_t}{100} = \frac{A_{st}}{bd} = \frac{25}{2(415)}\left[1 - \sqrt{1 - 4.598\left(\frac{1.00795}{25}\right)}\right] = 2.935 \times 10^{-3}$$

$$\therefore \quad p_t = 0.2935\%$$

$$\Rightarrow \quad A_{st} = \frac{0.2935}{100} \times 3700 \times 415 = 4506.7 \text{ mm}^2$$

Using 20 mm diameter bars,

$$\text{No. of bars required} = \frac{4506.7}{\frac{\pi}{4}(20)^2} = 14.3 \simeq 15 \text{bars}$$

$$\text{Spacing of bars} = \frac{3700 - 2 \times 75 - 15 \times 20}{(15-1)} = 232.14 \text{ mm}$$

Using 16 nos. of 20 mm dia. bars, spacing $= \dfrac{3700 - 2 \times 75 - 16 \times 20}{(16-1)} = 215.3 \text{ mm}$

Provide 16 nos. 20 Φ bars $\Rightarrow A_{st\,provided} = 16 \times \frac{\pi}{4} \times 20^2 = 5026.55 \text{mm}^2 > 4506.7 \text{mm}^2$ (OK)

Check for one way shear

Critical section for one way shear is at a distance 'd' from the face of column i.e. at section $(B)-(B)$

$$\text{SF at critical section } (B)-(B)\,(V_{u1}) = 131.48 \times 3.7\left[\left(\frac{3.7-0.45}{2}\right) - 0.415\right] \text{ kN} = 588.64 \text{ kN}$$

$$\text{Nominal shear stress } (\tau_v) = \frac{588.64 \times 10^3}{3700 \times 415} = 0.3834 \text{ N/mm}^2$$

$$\text{Shear strength of M25 concrete with } p_t = \frac{5026.55}{3700 \times 415} \times 100 = 0.3274\%$$

As per table 19 of IS:415, $(\tau_c) = 0.4 \text{ N/mm}^2 > \tau_u (= 0.3834 \text{ N/mm}^2)$ (OK)

This depth provided is safe in one way shear.

Check for two way shear

Critical section for two way shear is at a distance '$d/2$' from the column face.

$$\text{SF at critical section } (V_{u2}) = 131.48(3.7^2 - (0.45 + 0.415)^2) = 1701.6 \text{ kN}$$

$$\text{Nominal shear stress } (\tau_v) = \frac{1701.6 \times 10^3}{4(450+415)415} = 1.185 \text{N/mm}^2$$

Design shear strength of M25 concrete in two way shear

$$\tau'_c = k_s \tau_v$$

where $\quad k_s = \dfrac{1}{2} + \beta_c$

where $\quad \beta_c = \dfrac{\text{Shorter side of column}}{\text{Longer side of column}} = 1 = 0.5 + 1 \ngtr 1 = 1$

$$\tau'_c = k_s \tau_v = k_s\, 0.25\sqrt{25}$$
$$= (1)\,0.25\sqrt{25} = 1.25\ \text{N/mm}^2 > \tau_v (= 1.185\ \text{N/mm}^2)$$

Thus depth of footing is safe in two way shear.

Development length

$$\text{Development length } (L_d) = \frac{0.87 f_y \phi}{4\tau_{bd}}$$

where,

$$\tau_{bd} = 1.6 \times 1.4 = 2.24\ \text{N/mm}^2$$

$$l_d = \frac{0.87(415)10}{4(2.24)} = 403\ \text{mm} \qquad \text{(Cl. 26.2.1.1 of IS:456)}$$

Actual development length provided from the face of column

$$= \frac{3700 - 450}{2} - 75(\text{clear cover})$$
$$= 1550\ \text{mm} > 403\ \text{mm} \qquad \text{(OK)}$$

Load transfer at column-footing interface

Bearing pressure in column concrete at column footing interface.

$$= f_{br} = \frac{1.5 \times 1200}{450^2} \times 1000 = 8.89\ \text{N/mm}^2$$

$$\text{Permissible bearing stress} = 0.45 f_{ck}\sqrt{\frac{A_1}{A_2}} \quad \text{where } \frac{A_1}{A_2} = 1 \text{ here}$$

$$\text{Permissible bearing stress} = 0.45(25)$$
$$= 11.25\ \text{N/mm}^2$$
$$> 8.89\ \text{N/mm}^2 \qquad \text{(OK)}$$

Thus column load can be transferred by bearing alone. Even then a minimum 0.5% steel is provided at column footing interface.

$\therefore$

$$\text{Minimum steel } (A_{s\,min}) = 0.5\% \text{ of column area} = \frac{0.5}{100} \times 450^2 = 1012.5\ \text{mm}^2$$

Using 20 mm diameter bars,

$$\text{No. of bars required} = \frac{1012.5}{\frac{\pi}{4}(20)^2} = 3.22 = 4\ \text{bars}$$

Provide 4-20 Φ bars as dowel bars, at the interface of column and footing.

$$\text{Development length in compression } (L_d) = \frac{0.87(415)20}{4(2.24)1.25} = 644.7\ \text{mm}$$

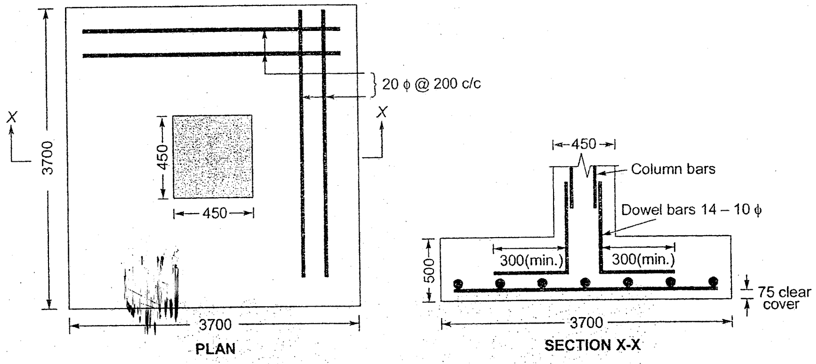
$$\text{Available length} = 500 - 75 \text{ (clear cover)} - 2 \times 20 \text{ (bar diameter)}$$
$$= 385\ \text{mm} < 644.7\ \text{mm}$$

Provide small diameter bars as dowel bars.

Using 10 mm diameter bars,

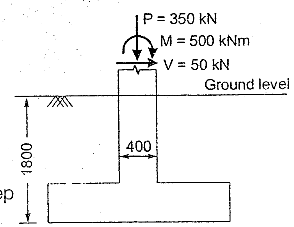
$$\text{No. of dowel bars required} = \frac{1012.5}{\frac{\pi}{4}(10)^2} = 12.89 \simeq 14\ \text{bars}$$

$$\text{Development length} = \frac{0.87(415)10}{4(2.24)1.25} = 322.37\ \text{mm} < 385\ \text{mm} \qquad \text{(OK)}$$



Example 13.3 Fix the dimensions of a footing which is required to carry an axial load of 350 kN, a lateral force of 50 kN and a uniaxial moment of 500 kNm. Safe bearing capacity of the soil is 143 kN/m^2. Consider footing to be at a depth of 1.8 m below the ground level. The water table is at 1.8 m depth from ground level which may use upto the ground level. Unit weight of soil is 16.8 kN/m^3. Take column size as 400 mm × 400 mm.

Solution:



$$\text{Moment due to lateral force} = 50 \times 1.8 = 90\ \text{kNm}$$
$$\text{Total moment at footing base } (M) = 500 + 90 = 590\ \text{kNm}$$
$$\text{Eccentricity } (e) = \frac{M}{P} = \frac{590}{350} = 1.686\ \text{m}$$
$$\text{Provide a square footing of size} = 5\ \text{m} \times 5\ \text{m} \times 1.25\ \text{m deep}$$

When water table is at 1.8 m below ground level,

$$\text{Weight of footing} = 5 \times 5 \times 1.25 \times 25$$
$$= 781.25\ \text{kN}$$
$$\text{Weight of soil above footing} = (5^2 - 0.4^2)(1.8 - 1.25)\,16.8 = 229.52\ \text{kN}$$
$$\text{Axial load} = 350\ \text{kN}$$
$$\text{Total load} = 350 + 229.52 + 781.25 = 1360.77\ \text{kN} \simeq 1361\ \text{kN}$$

When water table is at ground level,

$$\text{Weight of footing} = 5 \times 5 \times 1.25 \times (25 - 9.81) = 474.7\ \text{kN}$$
$$\text{Weight of soil above footing} = (5^2 - 0.4^2)(1.8 - 1.25)(16.8 - 9.81) = 95.5\ \text{kN}$$
$$\text{Axial load} = 350\ \text{kN}$$
$$\text{Total load} = 350 + 95.5 + 474.7 = 920.2\ \text{kN} \simeq 921\ \text{kN}$$
$$\text{Overturning moment about point x } (M_0) = 590\ \text{kNm}$$

Restoring moment about point x (M_R) $= 0.9 \times 921 \times \frac{5}{2} = 2072.25$ kNm

Factor of safety against overturning $= \frac{2072.25}{590} = 3.57 > 1.4$ (OK)

Bearing pressure check

Water table at 1.8 m below GL

$$\sigma = \frac{P}{A} \pm \frac{M}{I} y = \frac{1361}{5^2} \pm \frac{590}{5 \times \frac{5^3}{12}} \left(\frac{5}{2}\right)$$

$$= 54.44 \pm 28.32 = 82.76 \text{ kN/m}^2, \quad 26.12 \text{ kN.m}^2$$

Thus there is no uplift any where in the footing base.

Water table at GL

$$\sigma = \frac{P}{A} \pm \frac{M}{I} y = \frac{921}{5^2} \pm \frac{590}{5 \times \frac{5^3}{12}} \left(\frac{5}{2}\right)$$

$$= 36.84 \pm 28.32 = 65.16 \text{ kN/m}^2, 8.52 \text{ kN/m}^2$$

Thus footing size 5 m × 5 m × 1.25 m is adequate.

Example 13.4 **Design a spread footing for a column of size 350 mm × 500 mm which is reinforced with 4-20 Φ bars, carrying an axial load of 950 kN at service conditions and service moment of 100 kN. Assume that the moment is reversible and use M 25 concrete and Fe415 steel. The safe bearing capacity of the soil is 190 kN/m² at 1 m depth below the ground level.**

Solution:

Proportioning the size of footing

Given,

Axial load at service conditions (P) = 950 kN

Service moment (M) = 100 kNm

Factored load (P_u) = 1.5 × 950 = 1425 kN

Factored moment (M_u) = 1.5 × 100 = 150 kNm

Now since the moment is reversible and thus the fasting must be symmetric with respect to the column.

Let weight of footing and soil backfill as 10% of factored axial load (P_u).

∴ Eccentricity of load P_u at the base of the footing (e),

$$= \frac{M_u}{1.10 P_u} = \frac{150 \times 10^6 \text{ Nmm}}{1.1 \times 1425 \times 10^3 \text{ N}} = 95.694 \text{ mm}$$

For no tension any where in the footing, $e < L/6$. The stresses developed in the footing due to factored axial load and factored moment must be less than the factored bearing capacity of the soil.

i.e. $$\frac{1.1 P_u}{A} \pm \frac{M_u}{Z} \leq 1.5(190) \text{ kN/m}^2$$

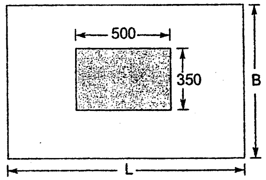
$$\Rightarrow \quad \frac{1.1(1425)}{BL} + \frac{150}{BL^2/6} \leq 285$$

$$\Rightarrow \quad \frac{1567.5}{BL} + \frac{900}{BL^2} \leq 285$$

$$\Rightarrow \quad 1567.5 L + 900 \leq 285 BL^2$$

$$\Rightarrow \quad 5.5 L + 3.1579 \leq BL^2$$

$$\Rightarrow \quad BL^2 - 5.5 L - 3.1579 \leq 0 \quad \text{... (i)}$$



Now the footing must be symmetric in order to take care of reversible moment. Thus the footing must have equal projections on either side of the column. Also for economy, the footing must have equal projections in the two orthogonal directions i.e.

$$\frac{L - 0.5}{2} = \frac{B - 0.35}{2}$$

$$\Rightarrow \quad L - B = 0.15$$

$$\Rightarrow \quad B = (L - 0.15) \quad \text{... (ii)}$$

Substitute the value of 'B' from (ii) in equation (i),

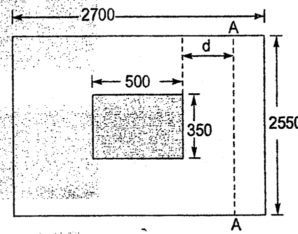
$$(L - 0.15) L^2 - 5.5 L - 3.1579 \geq 0$$

$$\Rightarrow \quad L^3 - 0.15 L^2 - 5.5 L - 3.1579 \geq 0$$

Solving by trial and error,

$$L \simeq 2.665 \text{ m} = 2.7 \text{ m (say)}$$

$$\therefore \quad B = L - 0.15 = 2.7 - 0.15 = 2.55 \text{ m}$$



Thus soil base pressure are,

$$\frac{(1.1)1425}{2.7 \times 2.55} \pm \frac{6 \times 150}{2.55 \times 2.7^2} = 227.67 \pm 48.41 \text{ kN/m}^2$$

$$= 276.08 \text{ kN/m}^2$$

$$< 1.5 \times 190 (= 285 \text{ kN/m}^2)$$

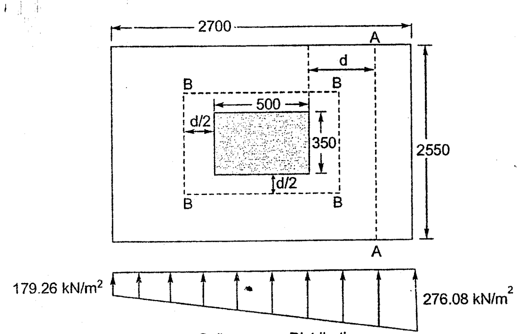
$$= 179.26 \text{ kN/m}^2$$

$$< 285 \text{ kN/m}^2 \quad \text{(OK)}$$

Determining the Depth of footing required

On the basis of one-way shear

Section (A) – (A) as shown in the figure is critical section for one way shear which is at a distance 'd' from the face of column.



Soil pressure Distribution

$$\text{Soil pressure at sec } (A)-(A) = 276.08 - \left[\left(\frac{276.08-179.26}{2700}\right)(1100-d)\right]$$

$$= 276.08 - 39.445 + 0.03586\,d$$

$$= (236.635 + 0.3586\,d)\ \text{kN.m}^2 \quad \text{(where } d \text{ is in mm)}$$

$\therefore$ One way shear force at sec $(A)-(A)$ = $(236.635 + 0.03586d)\ 2.55\ (1.1 - 0.001d)$

(where d is in mm)

Let percentage reinforcement in footing = 0.15%

$\therefore$ For M25 concrete and P_t = 0.15%, design shear strength of concrete as per table 19 of IS 456 = 0.29 N/mm^2

$\therefore$ Shear strength of M25 concrete at sec $(A)-(A) = (0.29)\ 2550d$ N $= 2.55d\ (0.29)$ kN

$\therefore$ $(236.635 + 0.03586d)\ (2.805 - 0.00255d) \le 2.55\ (0.29)d$

$\Rightarrow$ $663.791 - 0.60342d + 0.1005873d - 0.000091443d^2 \le 0.7395d$

$\Rightarrow$ $7.2587 \times 106 - 5498.86d - d^2 \le 8087.005d$

$\Rightarrow$ $d^2 + 13585.865d - 7.2587 \times 10^6 \ge 0$

$\therefore$ $d \ge 514.78$ mm

Let $d = 550$ mm

On the basis of two-way shear

Section $(B)-(B)-(B)-(B)$ around the column is critical for two way shear located at a distance of $(d/2)$ from the column face.

Shear strength of concrete in two way shear,

$$\tau'_c = k_s\tau_c \qquad \text{where } \tau_c = 0.25\sqrt{f_{ck}}$$

$$k_s = 0.5 + \beta_c \ngtr 1$$

$$\beta_c = \frac{\text{Short side of column}}{\text{Long side of column}} = \frac{350}{500} = 0.7$$

$\therefore$

$$k_s = 0.5 + \beta_c \ngtr 1$$

$$= 0.5 + 0.7 \ngtr 1 = 1.2 \ngtr 1 = 1$$

$\therefore$

$$\tau'_c = (1)\ 0.25\sqrt{f_{ck}} = 1.25\ \text{N/mm}^2$$

Average soil pressure at centre line of column

$$= \frac{276.08 + 179.26}{2}\ \text{kN/m}^2 = 227.67\ \text{kN/m}^2$$

Show force at Section $(B)-(B)-(B)-(B)$ = $(227.67)\ [2.7 \times 2.55 - (0.5 + 0.001d) \times (0.35 + 0.001d)]$

(where d is in mm)

$= 227.67\ [6.885 - (0.175 + 0.00085d + 1 \times 10^{-6}\,d^2)]$

Substituting d = 550 m in the above expression (where d = 550 mm is fixed on the basis of one-way shear),

Shear force at Section $(B)-(B)-(B)-(B)$ = $227.67\ [6.885 - (0.175 + 0.00085 \times 550 + 10^{-6} \times (550)^2)]$

$= 1352.36$ kN

Two way shear strength of soil = $1.25\ [(350 + d + 500 + d)2\ (600)]$ N

$= 1.25\ [1200\ (850 + 2 \times 600)]$ N $= 3075$ kN

> 1352.36 kN

Thus footing depth is safe in two way shear also.

Assuming 50 mm clear cover and using 20 Φ bars, overall depth of footing (D)

$$= 600 + 50 + 20 + \frac{20}{2}$$

$$= 600 + 50 + 20 + 10 = 680\ \text{mm} = 700\ \text{mm (say)}$$

$\therefore$

$$d = 700 - 50 - 20 - \frac{20}{2} = 620\ \text{mm}$$

Thus size of footing is **2700 × 2550 × 700 mm.**

Check for gross bearing pressure

$$\text{Pressure due to axial load } (P_u) = \frac{1425}{2.7 \times 2.55} = 206.97\ \text{kN/m}^2$$

$$\text{Pressure due to moment } (M_u) = \pm\frac{6 \times 150}{2.55 \times 2.7^2} = \pm 48.41\ \text{kN/m}^2$$

Pressure due to self weight of footing of 700 mm depth = 25 × 0.7 = 17.5 kN/m^2

Pressure due to soil backfill of height = (= 1000 − 700 = 300 mm) = 18 × 0.3 = 5.4 kN/m^2

$\therefore$ Maximum and minimum pressures are,

$$206.97 \pm 48.41 + 17.5 + 5.4$$

$$= \left[\begin{array}{l} 278.25\ \text{kN/m}^2\ (p_{max}) < 1.5(190) = 285\ \text{kN/m}^2\ \text{(ok)} \\ 181.46\ \text{kN/m}^2\ (p_{min}) < 285\ \text{kN/m}^2\ \text{(ok)} \end{array}\right.$$

Reinforcement requirement in long direction

The critical section for bending moment is at the face of column itself.

$$\text{Soil pressure at the column face} = 276.08 - \left[\left(\frac{276.08-179.26}{2700}\right)1100\right] = 236.64\ \text{kN/m}^2$$

$$\text{BM at column face } (M) = \left[236.64 \times 1.1 \times 2.55 \times \frac{1.1}{2} + (276.08 - 236.64)\frac{1.1}{2}(2.55)\frac{2}{3}(1.1)\right]\text{kNm}$$

$$= (365.08 + 40.56)\ \text{kNm} = 405.64\ \text{kNm}$$

$\therefore$

$$R = \frac{M}{bd^2} = \frac{405.64 \times 10^6}{(2550)620^2} = 0.413825\ \text{N/mm}^2$$

$\therefore$

$$\frac{p_t}{100} = \frac{A_{st}}{bd} = \frac{25}{2(415)}\left[1 - \sqrt{\frac{1 - 4.598(0.413825)}{25}}\right]$$

$$= 0.00116892476$$

$\Rightarrow$ $p_t = 0.1169\%$

$\Rightarrow$

$$A_{st} = \frac{0.1169}{100} \times 2550 \times 620 = 1848.189\ \text{mm}^2$$

$\therefore$

$$\text{No. of 20 } \Phi \text{ bars required} = \frac{1848.189}{\frac{\pi}{4}(20)^2} = 5.88 \simeq 6\ \text{bars}$$

$\therefore$

$$\text{Spacing} = \frac{2550 - 2\times 50 - 6\times 20}{5} = 466\,\text{mm} > 300\,\text{mm}$$

$$\text{If 8-20 }\Phi\text{ bars are provided, then spacing} = \frac{2550 - 2\times 50 - 8\times 20}{7} = 327.14 > 300\text{ mm}$$

$$\text{If 10-20 }\Phi\text{ bars are provided, then spacing} = \frac{2550 - 2\times 50 - 10\times 20}{9} = 250\,\text{mm} < 300\,\text{mm}$$

$\therefore$

$$A_{st\,provided} = 10\times\frac{\pi}{4}\times 20^2 = 3141.6\,\text{mm}^2 > 1848.189\,\text{mm}^2$$

Thus provide 10-20 Φ bars @ 250 c/c.

Reinforcement requirement in short direction

Average soil pressure between edge and column centre line

$$= \frac{227.67 + 276.08}{2} = 251.875\,\text{kN/m}^2$$

$$\text{BM} = 251.875\times 2.7\times 1.1\times\frac{1.1}{2} = 411.44\text{ kNm}$$

$$R = \frac{411.44\times 10^6}{2700\times 620^2} = 0.3964\,\text{N/mm}^2$$

$\therefore$

$$\frac{p_t}{100} = \frac{A_{st}}{bd} = \frac{25}{2(415)}\left[1 - \sqrt{\frac{1 - 4.598(0.3964)}{25}}\right]$$

$$= 0.111875\times 10^{-2}$$

$\therefore$

$$p_t = 0.11188\%$$

$\Rightarrow$

$$A_{st} = \frac{0.11188}{100}\times 2700\times 620 = 1872.87\,\text{mm}^2$$

$$\text{No. of 20 }\Phi\text{ bars required} = \frac{1872.87}{\frac{\pi}{4}(20)^2} = 5.96 \simeq 6\,\text{bars}$$

$$\text{Spacing of 6-20 }\Phi\text{ bars} = \frac{2700 - 2\times 50 - 6\times 20}{5} = 496\,\text{mm} > 300\,\text{mm}$$

$$\text{Using 10-20 }\Phi\text{ bars, spacing} = \frac{2700 - 2\times 50 - 10\times 20}{9} = 266.67\,\text{mm} < 300\,\text{mm}$$

Provide 10-20 Φ @ 250 c/c.

Development Length

Development length required for M25 concrete (L_d)

$$= \frac{0.87(415)\Phi}{4\times 1.6\times 1.4} = 40.296\,\Phi = 40.296\,(20) = 805.92\text{ m}$$

$$\text{Development length available} = 1100 - 50 - \frac{20}{2} = 1040\text{ mm} > 805.92\text{ mm} \quad \text{(OK)}$$

Load transfer at column base

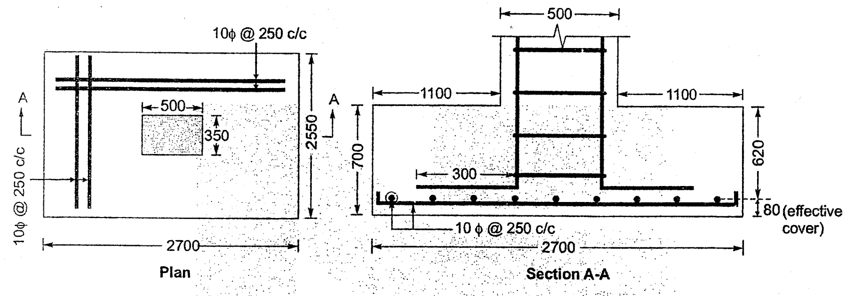
Since the column is subjected to axial load along with moments thus some column bars are in tension due to moment. Transfer of tensile force is not possible at the interface of column and footing and thus column bars are extended into the footing.

Development length for 20 Φ bars in M25 concrete = 805.92 mm

Development length available = 620 mm < 805.92 mm

Thus column bars must be given a 90° bend (anchorage value = 8 ϕ) and then extended upto 300 mm so that total development length provided = 620 + 8 (20) + 300 = 1080 mm > 805.92 mm (OK)

(Anchorage value of 90° bend)



Example 13.5 The size of a RC column is 300 x 600 mm. The column has to support a load of 1400 kN. Design the foundation for this column if safe bearing capacity of the soil is 150 kN/m². The width of the footing can not exceed 2.5 m. Use M 25 for concrete and Fe 500 steel. Use LSM.

Solution:

Size of foundation

Column load, $P = 1400\text{ kN}$

Let weight of the foundation, $P_f = 10\%$ of column load $(P) = 0.1 \times 1400\text{ kN} = 140\text{ kN}$

$\therefore$ Total load, $P_t = 1400 + 140\text{ kN} = 1540\text{ kN}$

$$\text{Area of footing required,} \quad A = \frac{1540}{150} = 10.27\text{ m}^2$$

Note: Do not use factored load of 1.5 x 1540 kN, here since safe bearing capacity of soil itself takes into account the factor of safety.

Assume $B = 2.5\text{ m}$

$\therefore$

$$D = \frac{A}{B} = \frac{10.27}{2.5} = 4.11\text{ m} = 4.2\text{ m (say)}$$

$\therefore$ Area provided, $A = 4.2 \times 2.5\text{ m}^2 = 10.5\text{ m}^2 > 10.27\text{ m}^2$ (OK)

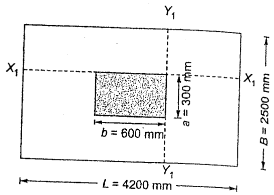
Net soil pressure

$$w_0 = \frac{P}{A} = \frac{1400}{10.5} = 133.33\text{ kN/m}^2$$

$$< 150 \text{ kN/m}^2 \text{ (= safe bearing capacity of the soil)} \quad \text{(OK)}$$

Factored soil pressure, $w_{u0} = 1.5 \times 133.33 = 200 \text{ kN/m}^2$

Depth of foundation from bending moment criterion



Moment at section x_1-x_1, $$M_{ux} = w_{u0}\left(\frac{B-b}{2}\right)\left(\frac{B-b}{4}\right)$$

$$= 200 \times \frac{(2.5-0.300)^2}{8} = 121 \text{ kNm}$$

Moment at section y_1-y_1, $$M_{uy} = w_{u0} \times \left(\frac{L-a}{2}\right)\left(\frac{L-a}{4}\right)$$

$$= 200 \times \frac{(4.2-0.6)^2}{8} = 324 \text{ kNm}$$

$$Q = 0.36 f_{ck} \times 0.46 \times (1 - 0.42 \times 0.46)$$
$$= 0.36 \times 25 \times 0.46 (1 - 0.42 \times 0.46)$$
$$= 3.34$$

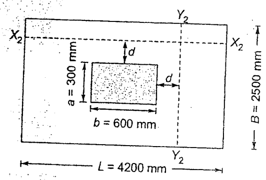
Depth of foundation, $$d = \sqrt{\frac{M_{uy}}{QB}} = \sqrt{\frac{324 \times 10^6}{3.34 \times 1000}} = 311.46 \text{ mm}$$

Take $d = 320$ mm

Check for one way shear

Critical section for one way shear will be at a distance of 'd' from the face of the column

Maximum shear force at section y_2-y_2



$$V_{uy} = w_{u0} \times 1 \text{ m} \times \left[\frac{L-a}{2} - d\right]$$

$$= 200 \times 1\left[\frac{4.2-0.6}{2} - 0.32\right]$$

$$= 296 \text{ kN}$$

Nominal shear stress

$$\tau_{vu} = \frac{V_{uy}}{Bd} = \frac{296 \times 10^3}{1000 \times 320} = 0.925 \text{ N/mm}^2$$

$$k = 1.0 \text{ for d} > 300 \text{ mm}$$

$$k\tau_c = 1 \times 0.29 \text{ N/mm}^2 < \tau_{vu} (= 0.925 \text{ N/mm}^2) \text{ Failed}$$

Revised depth of the footing

$$k\tau_c = 0.29 = \frac{V_{uy}}{Bd}$$

$$\therefore \quad d = \frac{V_{uy}}{B \times 0.29} = \frac{296 \times 10^3}{1000 \times 0.29} = 1020.69 \text{ mm}$$

Average of 320 and 1020 mm $= \dfrac{320+1020}{2} = 670 \text{ mm} \approx 700 \text{ mm}$

Check for $d = 700$ mm

$$V_{uy} = 200 \times \left[\frac{4.2-0.6}{2} - 0.7\right] = 220 \text{ kN}$$

$$\tau_v = \frac{V_{uy}}{Bd} = \frac{220 \times 10^3}{1000 \times 700} = 0.31 \text{ N/mm}^2$$

Try $d = 750$ mm

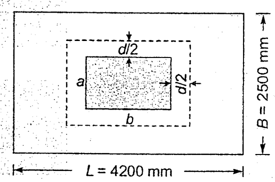
$$V_{uy} = 200 \times \left[\frac{4.2-0.6}{2} - 0.75\right] = 210 \text{ kN}$$

$$\tau_v = \frac{V_{uy}}{Bd} = \frac{210 \times 10^3}{1000 \times 750} = 0.28 \text{ N/mm}^2 < 0.29 \text{ N/mm}^2 \quad \text{(OK)}$$

$\therefore$ Adopt effective depth, $d = 750$ mm

Check for punching/two way shear

Net punching force $= P_u - w_{u0}(a+d)(b+d)$
$= 1.5 \times 1400 - 200 \times (0.6 + 0.75)(0.3 + 0.75)$
$= 1816.5$ kN



Punching shear stress $$= \tau_{vp\,(dev)} = \frac{\text{Net punching force}}{\text{Perimeter} \times \text{depth}}$$

$$= \frac{1816.5 \times 10^3}{2[(a+d)(b+d)] \times d}$$

$$= \frac{1816.5 \times 10^3}{2[(600+750)(300+750)] \times 750} = 0.505 \text{ N/mm}^2$$

$$\tau_{vp} = k_s 0.25\sqrt{f_{ck}}$$

where $$k_s = \left(0.5 + \frac{300}{600}\right) \not> 1 = 1$$

$$\tau_{vp\,(per)} = 1 \times 0.25 \times \sqrt{25} = 1.25 \text{ N/mm}^2 > 0.505 \text{ N/mm}^2 \quad \text{(OK)}$$

Steel Reinforcement:

Reinforcement required along long direction

$$M_{uy} = 324 \text{ kNm}$$

$$R_y = \frac{M_{uy}}{bd^2} = \frac{324 \times 10^6}{1000 \times 750^2} = 0.576$$

$$\therefore \quad \frac{p_t}{100} = \frac{A_{sty}}{bd} = \frac{f_{ck}}{2f_y}\left[1 - \sqrt{1 - \frac{4.598 R_y}{f_{ck}}}\right]$$

$$= \frac{25}{2 \times 500}\left[1 - \sqrt{1 - 4.598 \times \frac{0.576}{25}}\right]$$

$$p_t = 0.136\%$$

$$\therefore \quad A_{sty} = \frac{0.136}{100} \times 1000 \times 750 = 1020 \text{ mm}^2$$

$$\therefore \quad \text{Spacing of } 20\phi \text{ bars} = \frac{1000 \times \frac{\pi}{4} \times 20^2}{1020} = 308 \text{ mm c/c}$$

Provide 20φ bars @ 250 m c/c along long direction

$$\therefore \quad A_{sty\,provided} = \frac{1000 \times \frac{\pi}{4} \times 20^2}{250} = 1256.6 \text{ mm}^2 > 1020 \text{ mm}^2 \quad \text{(OK)}$$

Reinforcement required along short direction

$$M_{ux} = 121 \text{ kNm}$$

$$R_x = \frac{M_{ux}}{bd^2} = \frac{121 \times 10^6}{1000 \times (750-20)^2} = 0.22706$$

$$\therefore \quad p_t = \frac{25}{2 \times 500}\left[1 - \sqrt{1 - 4.598 \times \frac{0.22706}{25}}\right]100$$

$$= 0.05\% < 0.12\% \text{ (minimum reinforcement)}$$

Provided minimum reinforcement of 0.12%

$$\therefore \quad A_{stx} = \frac{0.12}{100} \times 1000 \times (750-20) = 876 \text{ mm}^2$$

$$\therefore \quad \text{Spacing of } 20\phi \text{ bars} = \frac{1000 \times \frac{\pi}{4} \times 20^2}{876} = 358.6 \text{ mm c/c but} \ngtr 300 \text{ mm c/c}$$

∴ Provide 20φ bars @ 250 mmc/c

$$\therefore \quad A_{stx\,provided} = \frac{1000 \times \frac{\pi}{4} \times 20^2}{250}$$

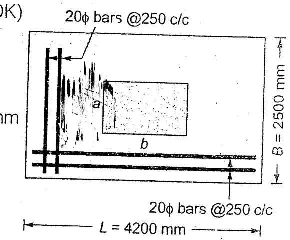
$$= 1256.6 \text{ mm}^2 > 876 \text{ mm}^2 \quad \text{(OK)}$$

$$\text{Development length required } (L_d) = \frac{0.87 f_y \phi}{4\tau_{bd}} = \frac{0.87(500)\phi}{4(1.6 \times 1.4)}$$

$$= 48.55\phi = 48.55 \times 20 = 917 \text{ mm}$$

$$\text{Available length} = \left(\frac{2500-300}{2}\right) - 75$$

$$= 1025 \text{ mm} > 971 \text{ mm} \quad \text{(OK)}$$



Example 13.6 Design the foundation of a brick masonry wall which is one brick thick and transfers a load of 110 kN/m length of the wall. The allowable bearing capacity of the soil below the footing is 90 kN/m². Take unit weight of earth as 16 kN/m³ and use M 25 concrete and Fe 415 steel.

Solution:

Let the footing is founded at a depth of 1 m below the natural ground level (NGL).

$$\text{Axial load} = 110 \text{ kN/m length of the wall}$$

Assuming weight of the footing and soil backfill as 10% of axial load, then width of the footing required per meter length of the footing along the wall $= 1.1 \times \frac{110}{90} = 1.3444$ m

$$\text{Adopt width of the wall footing } (B) = 2 \text{ m}$$

$$\text{Factored axial load} = 1.5 \times 110 \text{ kN/m} = 165 \text{ kN/m}$$

$$\text{Net upward soil pressure} = \frac{165}{(2 \times 1)} = 82.5 \text{ kN/m}^2$$

Now since the wall is one brick thick, so thickness of the wall = 230 mm (Neglecting the thickness of wall plaster etc.)

Critical bending moment in masonry wall:

In masonry wall, the critical section for bending moment is located half way between the wall center line and face/edge of the wall.

$$\text{Factored moment } (M_u) = 82.5 \times \frac{\left(\frac{2}{2} - \frac{0.15}{2}\right)^2}{2} = \frac{82.5 \times 0.925^2}{2} = 35.29 \text{ kNm}$$

Equating the factored moment to moment of resistance of the section for Fe 415 steel,

$$M_u = \text{Moment of resistance (MOR)}$$

$$35.29 \times 10^6 = 0.138 f_{ck} bd^2$$

$$35.29 \times 10^6 = 0.138 \times 25 \times 1000 \times d^2$$

$$d = 101.138 \text{ mm}$$

Adopt an overall depth of the footing (D) as 200 mm

Taking the effective cover as 50 mm, effective depth of footing (d) = 200 – 50 mm = 150 mm

$$R = \frac{M_u}{bd^2} = \frac{35.29 \times 10^6}{1000 \times 150^2} = 1.5684 \text{ N/mm}^2$$

$$\frac{p_t}{100} = \frac{A_{st}}{bd} = \frac{f_{ck}}{2f_y}\left[1 - \sqrt{1 - 4.598\frac{R}{f_{ck}}}\right]$$

$$\frac{p_t}{100} = \frac{A_{st}}{bd} = \frac{25}{2 \times 415}\left[1 - \sqrt{1 - 4.598\frac{1.5684}{25}}\right] = 0.004713$$

$$p_t = 0.4713\%$$

$$A_{st} = 706.95 \text{ mm}^2$$

Provide 12 mm diameter bars @ 150 mm c/c.

Thus, $A_{st,\,provided} = 753.98 \text{ mm}^2 > 706.95 \text{ mm}^2$

Development length:

The required development length is given by:

$$L_d = \frac{0.87 f_y \phi}{4\tau_{bd}} = \frac{0.87 \times 415 \times 12}{4 \times 1.4 \times 1.6} = 483.55 \text{ mm}$$

$$\text{Length available} = 1000 - 230/2 \text{ mm} = 885 \text{ mm} > 483.55 \text{ mm} \quad \text{(OK)}$$

Shrinkage reinforcement:

Provide longitudinal shrinkage reinforcement @ 0.12% = 0.0012 x 2000 x 200 mm² = 480 mm²

Provide 8-10Φ bars, $A_{st\,dist.} = 8\times\frac{\pi}{4}\times10^2 = 628.32\text{ mm}^2 > 480\text{ mm}^2$ (OK)

Shear check:

Now since the wall footing bends only in one direction normal to the wall, thus only one way shear is critical for wall footings.

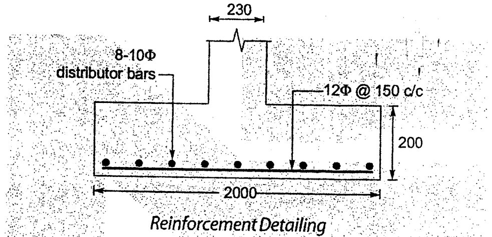
The critical section for one-way shear is located at a distance 'd' from the face of the wall.

Shear force (V_u) = $82.5\times(0.885-0.15)$ kN/m = 60.6375 kN/m=60.64 kN/m

Nominal shear stress (τ_v) = $60.64\times1000/(1000\times150) = 0.4043$ N/mm²

$p_{t,\,provided}$ = $753.98\times100/1000\times150 = 0.503\%$

Design shear strength of concrete for p_t = 0.503% and M25 concrete $(\tau_c) = 0.49096$ N/mm² (Table 19 of IS 456: 2000) $> \tau_v$(0.4043 N/mm²) (O.K.)



Reinforcement Detailing

Example 13.7 Find the thickness of uniform footing slab for a square column of size 400 × 400 mm carrying an axial load of 1100 kN. The safe bearing capacity of the soil is 150 kN/sq.m. Use M20 and Fe415.

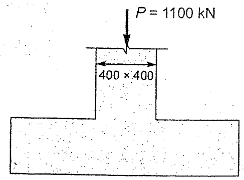
Solution:

Safe bearing capacity of soil (spc) = 150 kN/m² (M20 concrete and Fe415 steel)

Axial load (P) = 1100 kN

Weight of footing = 10% of 1100 kN = 110 kN

∴ Total load = 1210 kN



∴ Footing area required $= \frac{1210}{150} = 8.07\text{ m}^2$

Using square footing, size of the footing

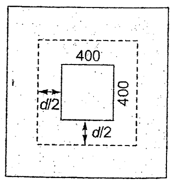
$= \sqrt{8.07} = 2.84\text{m} \simeq 2.9\text{ m (say)}$

∴ Provide a footing of size = 2.9 m × 2.9 m

∴ Footing area provided = 2.9 × 2.9 = 8.41 m²

∴ Net soil pressure $= \frac{1100}{8.41} = 130.8\text{ kN/m}^2$

∴ Factored soil pressure = 1.5 × 130.8 = 196.2 kN/m²



Projection of footing beyond column face along $x = O_y$

$= \frac{2900-400}{2} = 1250$ mm

∴ $M_{yy} = 196.2\times1.25\times\frac{1.25}{2} = 153.28$ kNm

Projection of footing beyond column face along $y = O_y$

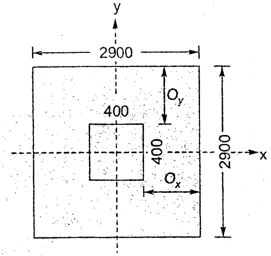
$= \frac{2900-400}{2} = 1250$ mm

∴ $M_{xx} = 196.2\times\frac{1.25^2}{2} = 153.28$ kNm

$M_{u\,lim} = 0.138\,f_{ck}\,bd^2$

⇒ $153.28\times10^6 = 138\,(20)\,(1000)\,d^2$

⇒ $d = 235.7$ mm



Let clear cover = 75 mm and using 20 mm ϕ bars

$D = 235.7+75+\frac{20}{2}$

= 320.7 mm = 350 mm (say)

∴ Effective depth (d) $= 350-75-\frac{20}{2} = 265$ mm

One way shear check:

Shear force along x-x = shear force along y-y

$= 196.2\times2.9\times\left(\frac{2.9-0.4}{2}-0.265\right) = 560.45\text{ kN} = v_u$ (say)

∴ $\tau_{xx} = \tau_{yy} = \frac{560.45\times1000}{2900\times265} = 0.729\text{ N/mm}^2$

Design shear stress of concrete for minimum tensile reinforcement (0.15%)

For M20 concrete = $\tau_c = 0.28$ N/mm²

∴ Depth required $= \frac{v_u}{B\tau_c} = \frac{560.45\times1000}{2900\times0.28} = 690.2\text{ mm} > 265\text{ mm}$ (Not OK)

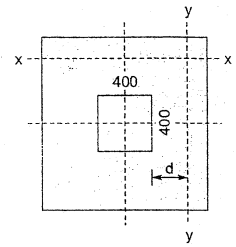
Let d = 700 mm

$D = 700+75+\frac{20}{2} = 785$ mm

∴ $\tau_v = \frac{560.45\times1000}{2900\times700} = 0.276\text{ N/mm}^2 < 0.28\text{ N/mm}^2$ (OK)

∴ d = 700 m is OK.

Two way shear check



Punching shear force = $1.5\times1100 - 196.2\,(0.4+d)^2$

$= 1650 - 196.2\,(0.4+d)^2$

Punching shear stress $= \frac{[1650-196.2(0.4+d)^2]1000}{4(0.4+0.7)0.7}$

= 0.4656 N/mm²

Permissible shear stress $= k_s\tau_c$ where, $k_s = 0.5+\dfrac{b}{a} \ngtr 1 = 0.5+\dfrac{2.9}{2.9} \ngtr 1 = 1.5 \ngtr 1$

$$\therefore \quad k_s = 1$$

$$= 1\times\tau_c$$

$$= 0.25\sqrt{f_{ck}}$$

$$= 0.25\sqrt{20} = 1.118 \text{ N/mm}^2$$

$$> 0.4656 \text{ N/mm}^2 \qquad \text{(OK)}$$

Thus, d = 700 mm is sufficient.

Reinforcement

$$M_u = 153.28 \text{ kNm}$$

$$R_u = \frac{M_u}{Bd^2} = \frac{153.28\times10^6}{2900\times700^2} = 0.1079 \text{ N/mm}^2$$

$$\therefore \quad \frac{p_t}{100} = \frac{A_{st}}{Bd} = \frac{20}{2(415)}\left[1-\sqrt{1-4.598\times\frac{0.1079}{20}}\right]$$

$$\therefore \quad p_t = 0.03\% \text{ which is very less}$$

$$\frac{A_{st\,min}}{Bd} \geq \frac{0.85}{f_y}$$

$$\therefore \quad p_{t\,min} \geq \frac{85}{f_y} = \frac{8.5}{415} = 0.2048\%$$

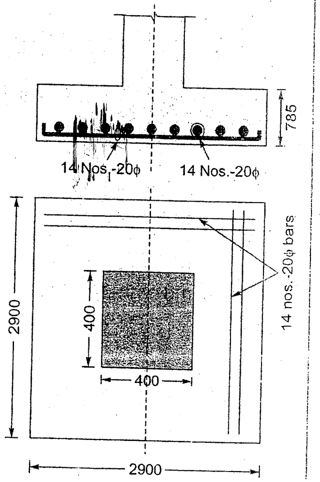
$$\Rightarrow \quad A_{st\,min} = \frac{0.2048}{100}\times2900\times700 = 4157.44 \text{ mm}^2$$

$$\therefore \quad \text{No. of 20 mm } \phi \text{ bars} = \frac{41157.44}{\frac{\pi}{4}\times20^2} = 13.2-14(\text{say})$$

$$\therefore \quad \text{Spacing of bars} = \frac{2900-75-75-14\times20}{13}$$

$$= 190 \text{ mm c/c}$$

$$< 3d (= 3\times700 = 2100 \text{ mm}) \text{ (OK)}$$



Development length

$$L_d = \frac{0.87\times415\phi}{4\tau_{bd}(1.6)} = \frac{0.87\times415\phi}{4\times1.6\times1.2}$$

$$= 47\,\phi = 47\times20 = 940 \text{ mm}$$

$$\text{Length available} = \left(\frac{2900-400}{2}\right)-75$$

$$= 1175 \text{ mm} > 940 \text{ mm (OK)}$$

13.12 Design of Combined Footings

A footing which supports more than a single column or wall is called as combined footing. This combined footing is called as continuous strip footing when columns are aligned in a straight line and it is called as raft or mat foundation when columns are aligned in two directions. Combined footing is required when columns are closely spaced and when exterior column lies on the property line of the structure.

13.13 Soil Pressure Distribution in Combined Footing

Assessment of exact determination of soil pressure below the footing is very difficult. In fact it depends on the rigidity of the footing and the soil properties. However for convenience, a straight line pressure distribution is assumed below the footing. This assumption is quite satisfactory for the design of rigid footings. But for flexible footings, this assumption is not valid and the problem is very complex involving soil-structure interactions aspects.

13.14 Geometric Design of Combined Footings

The geometry of the combined footing should be so selected that the centroid of the footing area should coincide with the resultant of column loads. This results in uniform soil pressure distribution which prevents the tilting of the footing. Depending on the relative magnitude of column loads, the footing may be rectangular or trapezoidal in plan.

13.15 Design Aspects of Two Column Combined Footing

1. **Fixing the plan dimensions:** The plan dimension of the footing should be such that the resulting pressure due to service loads is less than the safe bearing capacity of the soil and the centroid of the footing plan should coincide with the line of action of resultant column loads.
2. **Load transfer from column to footing:** Factored net soil pressure is calculated from the factored loads by dividing the factored load by the factored net soil pressure. This pressure is assumed to be uniformly distributed.

The base slab of the combined footing is subjected to the two way bending moments, two and one way shear. For simplicity, the footing is assumed to be a uniformly loaded longitudinal beam (of width B, length L and factored load $q_u \cdot B$ per unit length) supported on two columns strips.

The thickness of the footing is governed by shear considerations (generally two way or punching shear). Just like in uniformly loaded beams, in footings also, the critical section for one-way shear is located at a distance 'd' (effective depth) from the column face and for two way (punching) shear, the critical section is at a distance of d/2 from the column face along the periphery of the column. Flexural reinforcement is designed for the positive moments at the face of the column and negative moments occurring in between the columns. Reinforcement is placed at the bottom for the former case of bending moment and at the top for the later case of bending moment. Nominal transverse reinforcement is also provided to tie the main reinforcement.

13.16 Combined Footing of Beam-Slab

For large combined footings, providing a uniform depth for entire area of the footing results in un-economical design. For such cases, beam-slab combined footing is resorted to. In this, the footing consists of a base slab stiffened by a longitudinal beam of adequate depth interconnecting the two columns footings.

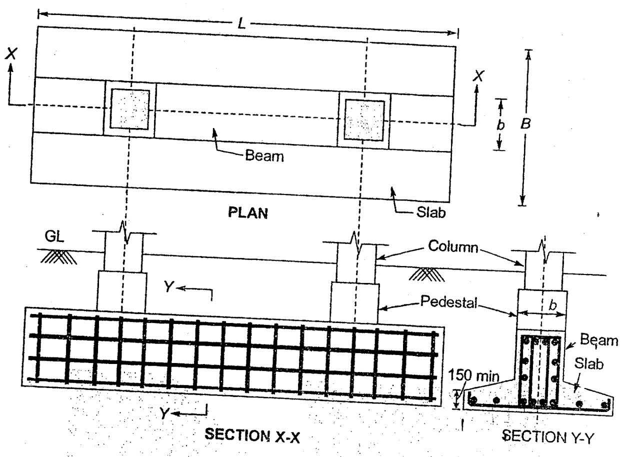


Fig.13.18 *Beam slab combined footing*

Base slab acts like a one way slab supported by the beam and bends in the transverse direction under the uniform soil pressure which acts from the below. Loads transferred by the slab are resisted by the longitudinal beam. The size of the beam generally depends on one way shear at a section located at a distance 'd' from the column face. The width of the footing beam is generally kept equal to the column width. High shear in the longitudinal beam is resisted by the heavy shear reinforcement provided in the form of multi-legged stirrups.

Thickness of the base slab is checked for one way shear at a distance 'd' (of the slab) from the face of the beam. Flexural reinforcement in the footing slab is designed for the cantilever moment coming in the footing slab at the face of the beam.

Example 13.8 **Design a reinforced concrete combined footing for two columns C_1 and C_2 of size 450 mm × 450 mm and 550 mm × 550 mm respectively. The column C_1 is reinforced with 4-25 mm diameter bars and column C_2 with 8-25 mm diameter bars (of Fe 415 in both the columns) and carries an axial load of 850 kN and 1550 kN respectively. The centre to centre distance between the two columns is 5 m. Permissible soil pressure below the base of the footing at 1.5 m depth below the ground is 270 kN/m². Use M 25 grade of concrete and Fe 415 steel.**

Solution:

Preliminary size of the combined footing:

Let weight of the footing and soil backfill is about 10% of the total column load.

Thus area of footing required $\quad A_{reqd} = (850+1550)\times\dfrac{1.1}{270} = 9.78\text{ m}^2$

For a uniform soil pressure distribution, the line of action of column loads must pass through the centroid of the footing. Let centroid is at a distance of x_c from the centre line of column C_1.

Determining the value of x_c:

Factored column load of column C_1 $= P_{u1} = 1.5 \times 850\text{ kN} = 1275\text{ kN}$

Factored column load of column C_2 $= P_{u2} = 1.5 \times 1550\text{ kN} = 2325\text{ kN}$

Total factored column load $= P_{u1} + P_{u2} = 1275 + 2325\text{ kN} = 3600\text{ kN}$

Centre to centre spacing between the columns (s) = 5 m

$$x_c = \frac{P_{u2}s}{(P_{u1}+P_{u2})} = \frac{2325\times 5000}{3600}$$

$$= 3229.167\text{ mm} > \frac{s}{2}\left(=\frac{5000}{2}\text{mm} = 2500\text{ mm}\right)$$

Thus adopt a rectangular footing of length $(L) = 2\left(x_c + \dfrac{450}{2}\right) = 2(3229.167+225) = 6908.334\text{ mm}$

$= 6.908\text{ m}$

Provide length of the footing (L) $=$ 7 m.

Thus width of the footing required (B) $= \dfrac{A_{reqd.}}{L} = \dfrac{9.78}{7} = 1.397\text{ m}$

Provide width of the footing (B) $=$ 1.5 m

Determination of Stresses in the Longitudinal Direction of the Footing

If the footing is assumed to be a wide beam of width 1500 mm spanning along the longitudinal direction then uniformly distributed load from beneath the base of the footing is given by:

$$\sigma_u B = \frac{(P_{u1}+P_{u2})}{L} = \frac{3600}{7}$$

$= 514.28757\text{ kN/m}$

$= 514.3\text{ kNm}$

The **critical section for shear** is located at a distance 'd' from the inner face of the column C_2 and has the value:

$$V_{u1} = 2325 - 514.3\times(1.775 + 0.275 + d)\text{ kN}$$

$= (1270.685 - 0.5143d)\text{ kN}$

Maximum positive bending moment:

$$M_u^+ = 514.3 \times 1.5^2/2 = 578.6\text{ kNm}$$

Maximum negative bending moment:

Maximum positive bending moment occurs where shear forces is zero i.e. at a distance x_1 from the outer edge of column C_1 where x_1 is given by:

$514.3x_1 - 1275 = 0$

or $\quad x_1 = 2.4791\text{ m} = 2479.1\text{ mm}$

$M_u^- = 514.3 \times 2.4791^2/2 - 1275\times(2.4791 - 0.225)$

$= -1293.5\text{ kNm} = -1294\text{ kNm}$

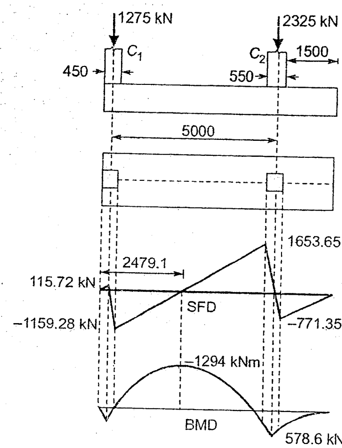


Fig.13.19 *Bending Moment Diagram and Shear Force Diagram*

Depth of the footing based on one way shear consideration:

Let percentage of tension reinforcement (p_t) = 0.5%

For 0.5% p_t and M 25 concrete, $\tau_c = 0.49\text{ N/mm}^2$ (Table 19 of IS: 456-2000) and

for M 25, τ_{cmax} = 3.1 N/mm^2

Design shear strength of concrete (V_{uc}) = $0.49 \times 1500 \times d = 735\, d$ N

Now $V_{u1} \leq V_{uc}$

or $(1270.685 - 0.5143d) \times 1000\text{ N} \leq 735d$

or $d \geq 1017.12$ mm

Depth of the footing based on two way shear consideration:

Critical section for two way shear is located at a distance of '$d/2$' from the face of columns.

Factored soil pressure (q_u) = 514.3/1.5 kN/m^2 = 342.87 kN/m^2

Taking d = 1018 mm (>1017.12 mm),

V_{u2} (for col. C_1) = $1275 - 342.87 \times (0.45 + 1.018) \times (0.45 + 1.018/2)$

= 792.3 kN

V_{u2} (for col. C_2) = $2325 - 342.87 \times (0.55 + 1.018)^2 = 1482.01$ kN

Maximum permissible two way shear stress (τ_{c2}) = $k_s(0.25\sqrt{f_{ck}})$

where k_s = 1 for square columns

= $1 \times (0.25\sqrt{25}) = 1.25$ N/mm^2

Two way shear strength of concrete (v_{c2}):

V_{c2} (for col. C_1) = $1.25 \times (450 + 1018/2 + 450 + 1018/2 + 450 + 1018 + 450 + 1018) \times 1018$ N

= $1.25 \times 4854 \times 1018$ N = 6176.715 kN > 792.3 kN

V_{c2} (for col. C_2) = $1.25 \times (550 + 1018) \times 4 \times 1018$ N

= 7981.112 kN > 1482.01 kN

Thus the depth of footing is sufficiently safe for two way shear also.

Taking clear cover to reinforcement in footing as 75 mm and using 25 mm diameter bars, overall depth of the footing (D) is:

$$D = 1017.12 + 75 + 25/2 \text{ mm} = 1104.62 \text{ mm}$$

Adopt an overall depth of footing as 1200 mm

Thus effective depth of the footing (d) = 1200 − 75 − 25/2 mm = 1112.5 mm > 1017.12 mm (OK)

Base pressure check:

Taking unit weight of concrete as 25 kN/m^3 and that of backfill soil as 18 kN/m^3, gross pressure under service loads is given by:

$$q = \frac{850+1550}{7\times1.5} + 25 \times 1.2 + 18 \times (1.5 - 1.2) \text{ kN/m}^2$$

$$= 228.57 + 30 + 5.4 \text{ kN/m}^2$$

$$= 263.97 \text{ kN/m}^2 < 270 \text{ kN/m}^2 \quad \text{(OK)}$$

Design of flexural reinforcement:

Maximum negative bending moment (M_u^-) = 1294 kNm

$$R = \frac{M_u^-}{Bd^2} = \frac{1294\times10^6}{(1500\times1112.5^2)} = 0.697 \text{ N/mm}^2$$

$$\frac{p_t}{100} = \frac{A_{st}}{Bd} = \frac{f_{ck}}{2f_y}\left[1-\sqrt{1-4.598\frac{R}{f_{ck}}}\right]$$

$$\frac{p_t}{100} = \frac{A_{st}}{1500x1112.5} = \frac{25}{2x415}\left[1-\sqrt{1-4.598\left(\frac{0.697}{25}\right)}\right]$$

$$p_t = 0.1997\% < 0.5\%$$

0.5% steel was assumed and is required for one way shear.

Thus provide p_t = 0.5% so that A_{streqd} = 8343.75 mm^2

$A_{st,\,min}$ = 0.12% of Bd = 0.0012 × 1500 × 1112.5 mm^2 = 2002.5 mm2 < 8343.75 mm^2 (OK)

Provide 18 nos. 25 mm diameter bars.

Spacing of bars = (1500 − 2 x 75 − 18 x 25)/17 = 52.94 mm

Thus provide 18 nos. 25 mm diameter bars at a spacing of 50 mm c/c.

Development length:

$$L_d = \frac{0.87f_y\phi}{4\tau_{bd}} = \frac{0.87x415\phi}{4x1.6x1.4} = 40.296\phi = 40.296\times25 = 1007.4 \text{ mm}$$

Sufficient length is available on both the sides of maximum bending moment section.

Maximum positive bending moment (M_u^+) = 578.6 kNm

$R = M_u/Bd^2$ = $578.6 \times 10^6/(1500 \times 1112.5^2) = 0.3117$ N/mm^2

$$\frac{p_t}{100} = \frac{A_{st}}{Bd} = \frac{f_{ck}}{2f_y}\left[1-\sqrt{1-4.598\frac{R}{f_{ck}}}\right]$$

$$\frac{p_t}{100} = \frac{A_{st}}{1500x1112.5} = \frac{25}{2x415}\left[1-\sqrt{1-4.598\left(\frac{0.3117}{25}\right)}\right]$$

p_t = 0.0879% < 0.12%

Provide minimum reinforcement of 0.12% = 0.0012 × 1500 × 1112.5 mm^2 = 2002.5 mm^2

Provide 8-25 mm diameter bars with spacing (s)

= (1500 − 2 x 75 − 8 x 25)/7 = 164.29 mm c/c

Development length (L_d) = 40.296ϕ = 40.296 × 25 mm = 1007.4 mm

Design of column strips as transverse beams:

Transverse beam under column C_1

Factored load per unit beam length = $\dfrac{1275}{1.5} = 850$ kN/m

Projection of beam beyond column face = $\dfrac{150-450}{2} = 525$ mm

Maximum moment at the face of column = $M_u = 850\times\dfrac{0.525^2}{2} = 117.14$ kNm

Using 20 mm dia. bars placed over 25 mm dia. bars,

Effective beam depth (d) = $1200 - 75 - 25 - \dfrac{20}{2} = 1090$ mm

Width of the beam (b) = Column width + 0.75 d

= 450 + 0.75 × 1090 = 1267.5 mm

$$\therefore \quad R = \frac{M_u}{bd^2} = \frac{117.14\times10^6}{1267.5\times1090^2} = 0.078 \text{ N/mm}^2$$

$$\therefore \quad \frac{p_t}{100} = \frac{A_{st}}{bd} = \frac{25}{2(415)\left[1-\sqrt{1-4.598\times\frac{0.078}{25}}\right]} = 2.1693\times10^{-4}$$

$$\therefore \quad p_t = 0.021683\% < 0.12\% \quad \text{(min. reinforcement)}$$

$\therefore$ Provide $p_t = 0.12\%$

$$\therefore \quad A_{st} = \frac{0.12}{100}\times1267.5\times1090 = 1657.89\ \text{mm}^2$$

$$\therefore \quad \text{No. of 20 mm dia. bars required} = \frac{1657.89}{\frac{\pi}{4}\times20^2} = 5.3 = 6(\text{say})$$

$$\therefore \quad \text{Spacing of bars} = \frac{1267.5-75\times2-6\times20}{5} = 199.5\ \text{mm c/c}$$

This spacing is comparatively large

Using 16 mm dia. bars, no. of bars required

$$= \frac{1657.89}{\frac{\pi}{4}\times16^2} = 8.2 = 10(\text{say})$$

$$\therefore \quad \text{Spacing} = \frac{1267.5-75\times2-16\times10}{9} = 106.4\ \text{mm c/c}$$

$\therefore$ Provide 10-16ϕ bars

$$\text{Development length required} = \frac{0.87\times415\phi}{4\times1.6\times1.4} = 40.29\phi\times16$$

$$= 644.64 > \text{length available } (=525-75=450\ \text{mm})$$

$$\text{Using } 12\phi \text{ bars, no. of bars required} = \frac{1657.89}{\frac{\pi}{4}\times12^2} = 14.7 = 15(\text{say})$$

$$\therefore \quad \text{Spacing} = \frac{1267.5-75\times2-15\times12}{14} = 66.96\ \text{mm c/c}$$

$$\text{Development length required} = 40.29\phi = 40.29\times12 = 483.48\ \text{mm} < 450\ \text{mm} \quad \text{(OK)}$$

The critical section for one way shear located at a distance d(= 1090 mm) lines outside the footing and thus no need to check for one way shear.

Transverse beam under column C_2

$$\text{Factored load per unit length} = \frac{2325}{1.5} = 1550\ \text{kN/m}$$

$$\text{Projection beyond column face} = \frac{1500-550}{2} = 475\ \text{mm}$$

$$\text{Maximum moment at column face} = M_u = 1550\times\frac{0.475^2}{2} = 174.86\ \text{kNm}$$

$$\text{Beam width} = 550 + 1.5\times1090 = 2185\ \text{mm}$$

$$\therefore \quad R = \frac{174.86\times10^6}{2185\times1090^2} = 0.06735\ \text{N/mm}^2$$

$$\therefore \quad \frac{p_t}{100} = \frac{A_{st}}{bd} = \frac{25}{2(415)}\left[1-\sqrt{1-4.598\times\frac{0.06735}{25}}\right]$$

$$= 1.8713\times10^{-4}$$

$$\therefore \quad p_t = 0.018713\% < 0.12\%\ (= p_{t\,min})$$

$$\therefore \quad A_{st\,min.} = \frac{0.12}{100}\times2185\times1090 = 2857.98\ \text{mm}^2$$

$$\therefore \quad \text{No. of } 12\phi \text{ dia. bars required} = \frac{2857.98}{\frac{\pi}{4}\times12^2} = 25.3\ \text{nos. } 26(\text{say})$$

$$\text{Development length required} = \frac{0.87 f_y\phi}{4\times1.6\tau_{bd}} = \frac{0.87\times415\phi}{4\times1.6\times1.4} = 40.29\phi$$

$$= 40.29\times12 = 483.48\ \text{mm} \quad \text{which is not available}$$

Increase width of the footing from 1500 mm to 1800 mm

$$\therefore \quad \text{Projection beyond column face} = \frac{1800-550}{2} = 625\ \text{mm}$$

$$= 625\ \text{mm} > 483.48\ \text{mm}\ (= L_d\ \text{required})$$

Transfer of force at column base

Column C_1

Permissible bearing stress at

(i) Column face $= 0.45 f_{ck} = 0.45\times25 = 11.25\ \text{N/mm}^2$

(ii) Footing face $= 0.45 f_{ck}\sqrt{\frac{A_1}{A_2}}$ where $A_1 = 450^2\ \text{mm}^2$, $A_2 = 450^2\ \text{mm}^2$

$$\therefore \quad \sqrt{\frac{A_1}{A_2}} = \frac{450}{450} = 1 \qquad \text{But } \sqrt{\frac{A_1}{A_2}} \not> 2$$

$\therefore$ Permissible bearing stress at footing face

$$= 0.45 f_{ck}\times1 = 11.25\ \text{N/mm}^2$$

$\therefore$ Limiting bearing resistance at column-footing interface

$$= 11.25\times450^2 = 2278.125\ \text{kN} > 1275\ \text{kN} \quad \text{(OK)}$$

Thus no reinforcement at interface is required but usually column bars are taken upto the bottom of footing.

Column C_2

Permissible bearing stress at

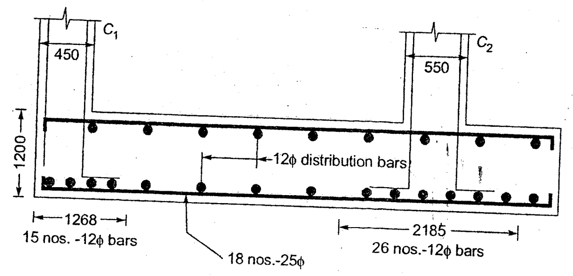
(i) Column face $= 0.45 f_{ck} = 0.45\times25 = 11.25\ \text{N/mm}^2$

(ii) Footing interface $= 0.45 f_{ck}\sqrt{\frac{A_1}{A_2}} = 0.45\times25\times2 = 22.5\ \text{N/mm}^2$

$$\text{where } \sqrt{\frac{A_1}{A_2}} = \frac{1800}{550} = 3.27 \text{ but } \not> 2$$

$\therefore$ Limiting bearing resistance at column footing interface

$$= 11.25\times550^2 = 3403\ \text{kN} > 2325\ \text{kN} \quad \text{(OK)}$$



Reinforcement detailing

Objective Brain Teasers

Q.1 The thickness of the footing at the edge shall not be less than:
(a) 100 mm (b) 150 mm
(c) 200 mm (d) 300 mm

Q.2 The design soil pressure acting upwards is:
(a) More than the ultimate bearing capacity of the soil
(b) Less than the ultimate bearing capacity of the soil
(c) Dependent on the type of soil
(d) Cannot be said

Q.3 In the limit state design of footings, the permissible bearing stress at the interface of column and footing is ________ $0.45f_{ck}$.
(a) Less than
(b) More than
(c) Equal to
(d) Cannot be determined

Q.4 The check for development length in footing is critical when:
(a) Footing depth is large
(b) Footing depth is less
(c) Soil bearing capacity is high
(d) Soil bearing capacity is low

Q.5 The depth of an isolated footing is generally governed by:
(a) Bending moment consideration
(b) Shear consideration
(c) Development length consideration
(d) All of the above

Q.6 The *safe bearing capacity* of soil as reported in a typical *soil report*:
(a) Includes the factor of safety
(b) Excludes the factor of safety
(c) Is same as *net ultimate bearing capacity of soil*
(d) Is determined wholly by empirical expressions without undergoing field tests

Q.7 Foundation design of a power plant generator involves the **additional** consideration of:
(a) Static loading
(b) Dynamic loading
(c) Deformation characteristics of the generator
(d) Stiffness characteristics of the generator

Q.8 In a two column combined footing, the resultant of column axial loads passes through the geometric centroid of the footing. The resultant soil pressure in this case will be:
(a) Uniform
(b) Trapezoidal
(c) Parabolic
(d) Of any shape depending on the soil.

Q.9 The minimum clear cover requirement as per IS recommendations in case of footings is:
(a) 20 mm (b) 25 mm
(c) 75 mm (d) 50 mm

Q.10 Raft foundation is usually provided where:
(a) Total footing area of individual footings exceeds 60% of total plan area.
(b) Total footing area of individual footings exceeds 30% of total plan area.
(c) Soil strata is rocky.
(d) Differential settlements is not a big issue.

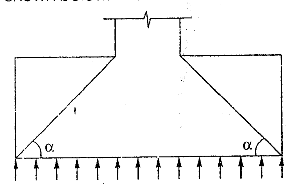
Q.11 A RC square footing of side length 2000 mm and 200 mm effective depth supports a 300 mm × 300 mm column. The line of action of load passes through the centroidal axis of footing and column. The magnitude of the load is 320 kN. The nominal transverse one way shear stress in the footing is
(a) 0.32 MPa (b) 3.2 MPa
(c) 0.26 MPa (d) 5.5 MPa

Q.12 The minimum edge thickness of footing should be more than
(a) 300 mm (b) 200 mm
(c) 250 mm (d) 150 mm

Q.13 If foundations of all the columns are designed on the basis of total dead and live loads then
(a) settlement of exterior columns will be more than interior columns
(b) there will be a uniform settlement
(c) there will not be any settlement at all
(d) settlement of interior columns will be more then exterior columns

Q.14 Punching shear is an important consideration in case of
(a) footings (b) beams
(c) columns (d) retaining wall

Q.15 A concrete pedestal is made of M25 concrete as shown below. The value of tanα for this case is



(a) > 4.5 (b) ≥ 3
(c) < 4.5 (d) ≤ 2

Q.16 A thin layer of PCC is provided below the footings because
(a) it lowers down the amount of total settlement
(b) it avoids point reactions from the soil
(c) it acts as a cushion for the footing
(d) All of the above

Q.17 Design of foundations for power plant turbo generators, is guided by the following parameter
(a) stiffness (b) strength
(c) flexibility (d) frequency

Answers

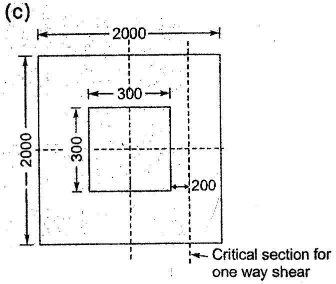
1. (b)	2. (a)	3. (b)	4. (b)	5. (b)
6. (a)	7. (b)	8. (a)	9. (d)	10. (a)
11. (c)	12. (d)	13. (a)	14. (a)	15. (a)
16. (b)	17. (d)			

Hints:

3. (b)
At interface, permissible bearing stress

$= 0.45f_{ck}\sqrt{\left(\frac{A_1}{A_2}\right)}$ which is greater than $0.45f_{ck}$.

11. (c)



$$p = \frac{P}{A} = \frac{320000}{2000 \times 2000} = 0.08 \text{ N/mm}^2$$
$$V_u = (0.08 \times 2000)(1000 - 200 - 150)$$
$$= 104000 \text{ N}$$
$$\tau_v = \frac{104 \times 1000}{2000 \times 200} = 0.26 \text{ N/mm}^2$$

15. (a)

$$\tan\alpha > 0.9\left[\frac{100q_0}{f_{ck}} + 1\right]^{1/2}$$
$$= 0.9\left[\frac{100 \times 6}{25} \times 1\right]^{1/2} = 4.5$$

Conventional Practice Questions

Q.1 Design an isolated footing for a column 350 mm × 450 mm, having 20 bars of 16 mm diameter carrying an axial load of 1600 kN and a moment of 150 kNm. Use M20 and Fe415. The safe bearing capacity of the soil is 170 kN/m^2 at 1 m below the ground level.

Q.2 Design a combined footing for two columns of size 400 × 400 mm and 350 × 450 mm. The columns carry axial loads of 800 kN and 1200 kN respectively and c/c spacing between the columns is 3.7 m. The 400 × 400 mm is flushed with the property line. The safe bearing capacity of the soil is 150 kN/m^2 at 1.5 m below the ground level. Use M25 and Fe500.

Q.3 Design an isolated footing for a rectangular column of size 300× 550 mm which is reinforced with 16 nos. 20 mm diameter bars of Fe415 and carries an axial load of 2000 kN. Take safe bearing capacity of the soil as 250 kN/m^2 at 1 m below the ground level. Use M30 and Fe415.

Q.4 Design a sloped footing for a square column of size 350 × 350 mm carrying an axial load of 1100 kN. The column is reinforced with 16 nos. 20 mm diameter bars of Fe415. The safe bearing capacity of the soil is 150 kN/m^2.

Q.5 Design a plain concrete footing for a column of size 350 × 350 mm carrying an axial load of 300 kN. The safe bearing capacity of the soil is 250 kN/m^2 at 1 m depth below the ground level. Use M25 and Fe415. Use M30

Q.6 Design a combined trapezoidal footing for two columns of size 300 × 400 mm and 450 × 550 mm carrying axial loads of 800 kN and 1100 kN respectively. The safe bearing capacity of the soil is 110 kN/m^2. Use M25 and Fe500.

Q.7 Design an isolated circular footing for a circular column of 400 mm diameter carrying an axial load of 1050 kN. Use M30 and Fe415. Take safe bearing capacity of the soil as 80 kN/m^2.

Q.8 Design an isolated stepped footing for a rectangular column of size 350 × 550 mm carrying an axial load of 1250 kN and moments of 120 kNm and 100 kNm in the major and minor axes respectively. Take safe bearing capacity of the soil as 150 kN/m^2. Use M30 and Fe415

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