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Lec.2

تصميم خرسانة مسلحة

Footings

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FOOTINGS

1. Introduction

Footings are structural members used to support columns and walls and transmit their loads to the underlying soils. Reinforced concrete is a material admirably suited for footings and is used as such for both reinforced concrete and structural steel buildings, bridges, towers, and other structures.

The permissible pressure on a soil beneath a footing is normally a few tons per square meter. The compressive stresses in the walls and columns of an ordinary structure may run as high as a few hundred tons per square meter. It is, therefore, necessary to spread these loads over sufficient soil areas to permit the soil to support the loads safely.

Not only is it desired to transfer the superstructure loads to the soil beneath in a manner that will prevent excessive or uneven settlements and rotations, but it is also necessary to provide sufficient resistance to sliding and overturning.

To accomplish these objectives, it is necessary to transmit the supported loads to a soil of sufficient strength and then to spread them out over an area such that the unit pressure is within a reasonable range. If it is not possible to dig a short distance and find a satisfactory soil, it will be necessary to use piles to do the job.

The closer a foundation is to the ground surface, the more economical it will be to construct. However, it is necessary to excavate a sufficient distance so that a satisfactory bearing material is reached.

2. Types of Footings

Among the several types of reinforced concrete footings in common use are the wall, isolated, combined, raft, and pile-cap types.

A *wall footing* is simply an enlargement of the bottom of a wall that will sufficiently distribute the load to the foundation soil. Wall footings are normally used around the perimeter of a building and perhaps for some of the interior walls.

An *isolated or single-column footing* is used to support the load of a single column. These are the most commonly used footings, particularly where the loads are relatively light and the columns are not closely spaced.

Mat (or raft) foundation

Combined footings are used to support two or more column loads. A combined footing might be economical where two or more heavily loaded columns are so spaced that normally designed single-column footings would run into each other. Single-column footings are usually square or rectangular and, when used for columns located right at property lines, extend across those lines. A footing for such a column combined with one for an interior column can be designed to fit within the property lines.

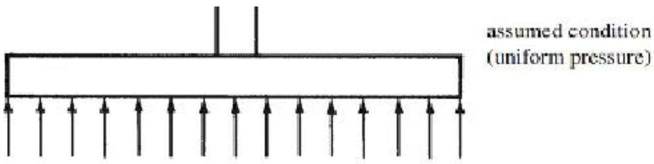
A *mat or raft or floating foundation* is a continuous reinforced concrete slab over a large area used to support many columns and walls. This kind of foundation is used where soil strength is low or where column loads are large but where piles are not used. For such cases, isolated footings would be so large that it is more economical to use a continuous raft or mat under the entire area. The cost of the formwork for a mat footing is far less than is the cost of the forms for a large number of isolated footings. If individual footings are designed for each column and if their combined area is greater than half of the area contained within the perimeter of the building, it is usually more economical to use one large footing or mat. The raft or mat foundation is particularly useful in reducing differential settlements between columns.

Pile caps are slabs of reinforced concrete used to distribute column loads to groups of pile.

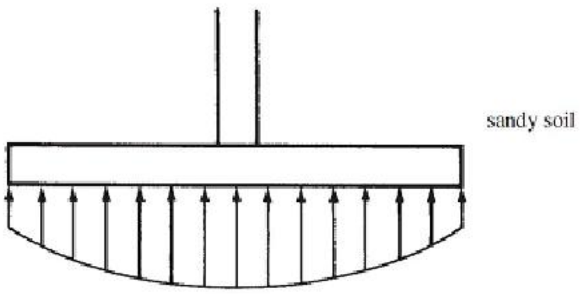
3. Actual Soil Pressures

The soil pressure at the surface of contact between a footing and the soil is assumed to be uniformly distributed as long as the load above is applied at the center of gravity of the footing. This assumption is made even though many tests have shown that soil pressures are unevenly distributed due to variations in soil properties.

When footings are supported by sandy soils, the pressures are larger under the center of the footing and smaller near the edge. The sand at the edges of the footing tends to move from underneath the footing edges, with the result that more of the load is carried near the center of the footing.



(a)



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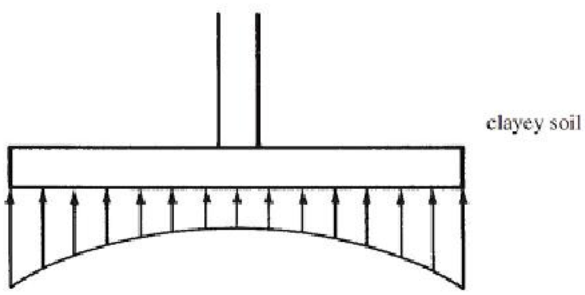


Just the opposite situation is true for footings supported by clayey soils. The clay under the edges of the footing sticks to or has cohesion with the surrounding clay soil. As a result, more of the load is carried at the edge of the footing than near the middle.

The designer should clearly understand that the assumption of uniform soil pressure underneath footings is made for reasons of simplifying calculations.

4. Allowable Soil Pressures

The allowable soil pressures to be used for designing the footings for a particular structure are preferably obtained by using the services of a geotechnical engineer. He or she will determine safe values from the principles of soil mechanics on the basis of experimental investigations.

Because such investigations often may not be feasible, most building codes provide certain approximate allowable bearing pressures that can be used for the types of soils and soil conditions occurring in that locality. The table below shows a set of allowable values that are typical of such building codes. It is thought that these values usually provide factors of safety of approximately three against severe settlements.

Section 15.2.2 of the ACI Code states that the required area of a footing is to be determined by dividing the anticipated total load, including the footing weight, by a permissible soil pressure or permissible pile capacity determined using the principles of soil mechanics. It will be noted that this total load is the unfactored load, and yet the design of footings is based on strength design, where the loads are multiplied by the appropriate load factors. **It is obvious that an ultimate load cannot be divided by an allowable soil pressure to determine the bearing area required**.

The designer can determine the bearing area required by summing up the actual or unfactored dead and live loads and dividing them by the allowable soil pressure. Once this area is determined and the dimensions are selected, an ultimate soil pressure can be computed by dividing the factored or ultimate load by the area provided.

	Maximum Allowable Soil Pressure	
Class of Material	**U.S. Customary Units ($kips/ft^2$)**	**SI Units (kN/m^2)**
Rock	20% of ultimate crushing strength	20% of ultimate crushing strength
Compact coarse sand, compact fine sand, hard clay, or sand clay	8	385
Medium stiff clay or sandy clay	6	290
Compact inorganic sand and silt mixtures	4	190
Loose sand	3	145
Soft sand clay or clay	2	95
Loose inorganic sand–silt mixtures	1	50
Loose organic sand–silt mixtures, muck, or bay mud	0	0

5. Design of Wall Footings

The theory used for designing beams is applicable to the design of footings with only a few modifications. The upward soil pressure under the wall footing tends to bend the footing into the deformed shape shown. The footings will be designed as shallow beams for the moments and shears involved. However, footing depth seems to be controlled by shear rather than bending moment due to larger loads and smaller spans in footings.

It appears that the maximum moment in this footing occurs under the middle of the wall. However, it is considered satisfactory to compute the moments at the faces of the walls (ACI Code 15.4.2).

To compute the bending moments and shears in a footing, it is necessary to compute only the net upward pressure, q_u, caused by the factored wall loads above. In other words, the weight of the footing and soil on top of the footing can be neglected.

Since the failure will not occur on a vertical plane at the wall face but rather at an angle of approximately 45° with the wall, as shown in Figure. Shear may be calculated at a distance d from the face of the wall (ACI Code 11.1.3.1).

The use of stirrups in footings is usually considered impractical and uneconomical. For this reason, the effective depth of wall footings is selected so that V_u is limited to the design shear strength, $ØV_c$, that the concrete can carry without web reinforcing, that is:

$$V_c = 0.17\lambda\sqrt{f'_c}\, b_w d \qquad (11\text{-}3)$$

Although the equation for V_c contains the term λ, it would be unusual to use lightweight concrete to construct a footing.

It is convenient to design wall footings for 1-m-wide sections of the walls. The depth of such footings above the bottom reinforcing may not be less than 150 mm for footings on soils or 300 mm for those on piles. As a result, minimum footing depths are at least 250 mm for regular spread footings and 400 mm for pile caps.

The determination of a footing depth is a trial-and-error problem. The designer assumes an effective depth, *d,* computes the *d* required for shear, tries another *d,* computes the *d* required for shear, and so on, until the assumed value and the calculated value are within each other.

Example:

Design a wall footing to support a 300mm-wide reinforced concrete wall with a dead load D = 300kN/m and a live load L = 200kN/m. The bottom of the footing is to be 1.2m below the final grade, the soil weighs 16kN/m^3, the allowable soil pressure, q_a, is 200kN/m^2, f_y = 420MPa and f'_c = 28 MPa, normal-weight concrete.

SOLUTION

Assume a 300mm thick footing (d = 225 mm).The cover is determined by referring to the code (7.7.1), which says that for concrete cast against and permanently exposed to the earth, a minimum of 75mm clear distance outside any reinforcing is required.

The footing weight is:

$(0.3)(24) = 7.2 kN/m^2$,

The soil fill on top of the footing is:

$(0.9)(16) = 14.4 kN/m^2$.

So $21.6 kN/m^2$ of the allowable soil pressure q_a is used to support the footing itself and the soil fill on top. The remaining soil pressure is available to support the wall loads. It is called q_e, the effective soil pressure.

$$q_e = 200 - 7.2 - 14.4 = 178.4 kN/m^2$$

Width of foundation required is:

$$q_u = \frac{300+200}{178.4} = 2.8m \Rightarrow \text{ Use 2.8 m.}$$

Bearing pressure:

$$q_u = \frac{(1.2)(300)+(1.6)(200)}{2.8} = 242.86 kN/m^2$$

Shear at distance d from face of wall:

$$V_u = (\frac{242.86}{1000})(\frac{2800}{2} - \frac{300}{2} - 225) = 248.9 kN/1m - width$$

Section Capacity for shear is:

$$\phi V_c = \phi 0.17 \sqrt{f_c'} b_w d$$

$$= (0.75)(0.17\sqrt{28})(1000)(225)/1000 = 151.8 < 248.9 kN/1m - width$$

<u>Try another d</u>, let d=375mm (h=450mm):

$$q_e = 200 - (0.45)(24) - (0.75)(16) = 177.2 kN/m^2$$

Width of foundation required is:

$$q_u = \frac{300+200}{177.2} = 2.83m \Rightarrow \text{ Use 3.0 m.}$$

Bearing pressure:

$$q_u = \frac{(1.2)(300)+(1.6)(200)}{3} = 226.67 kN/m^2$$

Shear at distance d from face of wall:

$$V_u = (\frac{226.67}{1000})(\frac{3000}{2} - \frac{300}{2} - 375) = 221 kN/1m - width$$

Section Capacity for shear is:

$$\phi V_c = \phi 0.17\sqrt{f_c'} b_w d$$

$$= (0.75)(0.17\sqrt{28})(1000)(375)/1000 = 253 > 221 kN/1m - width \quad \text{O.K.}$$

The moment at face of wall is:

$$M_u = (226.67)\frac{(1.5-0.15)^2}{2} = 206.6 kN.m$$

$$\frac{M_u}{\phi b d^2} = \frac{(206.6)(10)^6}{(0.9)(1000)(375)^2} = 1.63 MPa \Rightarrow \rho \cong 0.004$$

$$(A_s)_{rqd.} = (0.004)(1000)(375) = 1500 mm^2/m$$

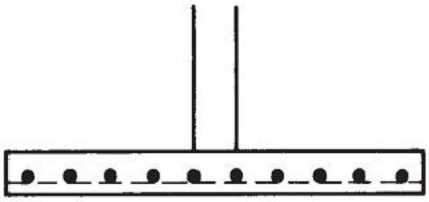
Using Ø16/125 $\Rightarrow (A_s)_{Prov.} = 1608 mm^2/m$

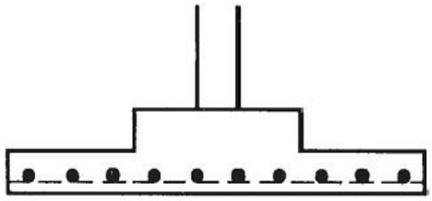
The development length should be checked and $l_{rqd.} < l_{prov.}$

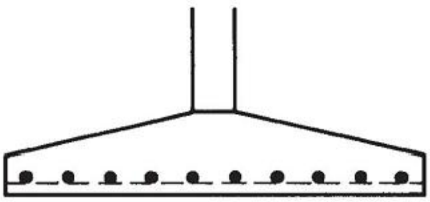
6. Design of Square Isolated Footings

Single-column footings usually provide the most economical column foundations. Such footings are generally square in plan, but they can just as well be rectangular or even circular or octagonal.

Most footings consist of slabs of constant thickness, but if calculated thicknesses are greater than 1m, it may be economical to use stepped footings, as illustrated in Figure below. The shears and moments in a footing are obviously larger near the column, with the result that greater depths are required in that area as compared to the outer parts of the footing. Occasionally, sloped footings are used instead of stepped ones, but labor costs can be a problem. Whether stepped or sloped, it is considered necessary to place the concrete for the entire footing in a single pour to ensure the construction of a monolithic structure, thus avoiding horizontal shearing weakness.

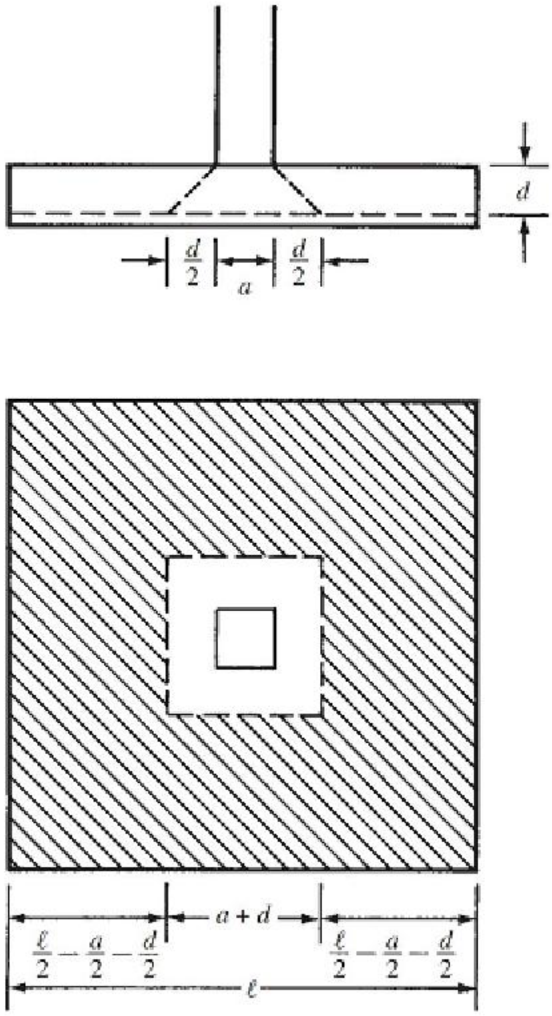

(a) Single-slab footing


(b) Stepped footing


(c) Sloped footing

Shear:

Two shear conditions must be considered in column footings, regardless of their shapes. The first of these is one-way or beam shear, which is the same as that considered in wall footings in the preceding section. For this discussion, reference is made to the footing of Figure below. The total shear to be taken along section 1-1 equals the net soil pressure, q_u, times the hatched area outside the section. In the expression to follow, b_w is the whole width of the footing.



11.11.2.1 — For nonprestressed slabs and footings, V_c shall be the smallest of (a), (b), and (c):

(a) $$V_c = 0.17\left(1 + \frac{2}{\beta}\right)\lambda\sqrt{f'_c}\,b_o d \qquad (11\text{-}31)$$

where β is the ratio of long side to short side of the column, concentrated load or reaction area;

(b) $$V_c = 0.083\left(\frac{\alpha_s d}{b_o} + 2\right)\lambda\sqrt{f'_c}\,b_o d \qquad (11\text{-}32)$$

where α_s is 40 for interior columns, 30 for edge columns, 20 for corner columns; and

(c) $$V_c = 0.33\lambda\sqrt{f'_c}\,b_o d \qquad (11\text{-}33)$$

Moments:

The bending moment in a square reinforced concrete footing with a square column is the same about both axes because of symmetry**. If the column is not square, the moment will be larger in the direction of the shorter column dimension**. It should be noted, however, that the effective depth of the footing cannot be the same in the two directions because the bars in one direction rest on top of the bars in the other direction. The effective depth used for calculations might be the average for the two directions or, more conservatively, the value for the bars on top.

The critical section for bending is taken at the face of a reinforced concrete column or halfway between the middle and edge of a masonry wall or at a distance halfway from the edge of the base plate and the face of the column if structural steel columns are used (Code 15.4.2).

The determination of footing depths by the procedure described here will often require several cycles of a trial-and-error procedure. There are, however, many tables and handbooks available with which footing depths can be accurately estimated. In addition, there are many rules of thumb used by designers for making initial thickness estimates, such as 20% of the footing width or the column diameter plus 75mm.

The reinforcing steel ratio (ρ) calculated for footings will often be appreciably less than the minimum values ($1.4/f_y$) or ($0.25\sqrt{f'_c}/f_y$) specified for flexural members in ACI Section 10.5.1 & Section 10.5.4, however, the code states that in slabs of uniform thickness, the minimum area and maximum spacing of reinforcing bars in the direction of bending need only be equal to those required for shrinkage and temperature reinforcement. The maximum spacing of this reinforcement may not exceed the lesser of three times the footing thickness, or 450mm. Many designers feel that the combination of high shears and low ρ values that often occurs in footings is not a good situation. Because of this, they specify steel areas at least as large as the flexural minimums of ACI Section 10.5.1.

Example

Design a square column footing for a 400mm square tied interior column that supports a dead load P_D = 800 kN and a live load P_L = 600 kN. The base of the footing is 1.5 m below grade, the soil weight is 16 kN/m^3, f_y = 420MPa, f'_c = 21MPa, and q_a = 250 kN/m^2.

SOLUTION:

Assume 600mm thickness (d=500mm),

$q_e = 250 - (0.6)(24) - (0.9)(16) = 221.2kN/m^2$

$A_{req.} = (800 + 600)/221.2 = 6.33m^2$ Use 2.6x2.6m , $A_{prov.}$=6.76m^2

$q_u = \frac{(1.2)(800)+(1.6)(600)}{6.76} = 284kN/m^2$

Check punching shear,

$b_o = (4)(400 + 500) = 3600mm$

V_u=(284)(2.6^2-0.9^2)=1690kN

$\varphi V_c = (0.75)(0.33)(\sqrt{21})(3600)(500)/1000 = 2041.5kN > 1690kN$ OK

or $\varphi V_c = (0.083)\left(\frac{(40)(900)}{3600} + 2\right)(\sqrt{21})(3600)(500)/1000 = 3880kN$

Check one way shear:

V_u=(284)(2.6)(1.3-0.2-0.5)=443kN

$\varphi V_c = (0.75)(0.17)(\sqrt{21})(2600)(500)/1000 = 759kN > 443kN$ OK

$M_u = (284)(1.1)(0.55)(2.6) = 446.8\text{kN.m}$

$$\frac{M_u}{\varphi bd^2} = \frac{(446.8)(10)^6}{(0.9)(2600)(500)^2} = 0.76 \rightarrow \rho = 0.0019$$

Check min. ratio which is larger of ($\rho = \frac{1.4}{420} = 0.0033$, $\rho = 0.25\frac{\sqrt{21}}{420} = 0.00273$)

A_s=(0.0033)(2600)(500)=4290mm^2 or 1650mm^2/m

Use Ø22/200, $(A_s)_{Prov.}$=1900mm^2

Check development length.

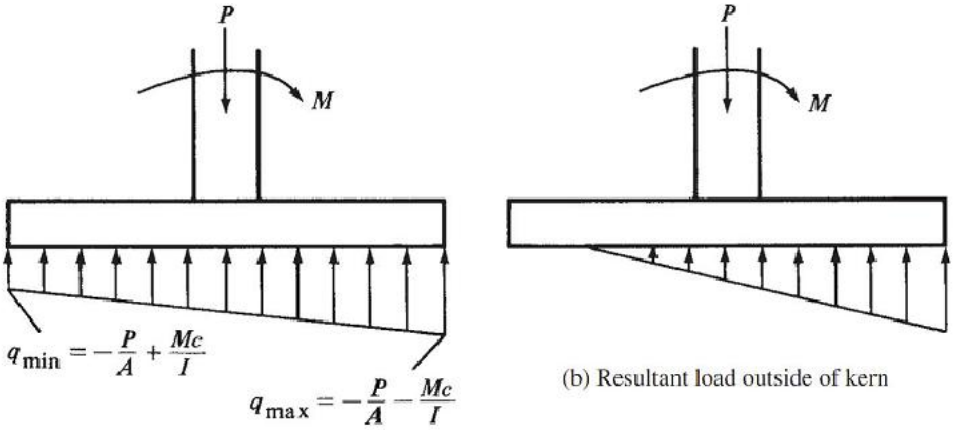
9. Footings Subjected to Axial Loads and Moments

Walls or columns often transfer moments as well as vertical loads to their footings. These moments may be the result of gravity loads or lateral loads. Such a situation is represented by the vertical load *P* and the bending moment *M* shown in Figure below. Moment transfer from columns to footings depends on how the column-footing connection is constructed.

If a column-footing joint is to behave as a pin or hinge, it would have to be constructed accordingly. The reinforcing in the column might be terminated at the column base instead of continuing into the footing. Dowels would be provided, but these would not be adequate to provide a moment connection.

To provide continuity at the column-footing interface, the reinforcing steel would have to be continued into the footing. This is normally accomplished by embedding hooked bars into the footing and having them extend into the air where the columns will be located. The length they extend into the air must be at least the lap splice length; sometimes this can be a significant length. These bars are then lap spliced or mechanically spliced with the column bars, providing continuity of tension force in the reinforcing steel.

If there is a moment transfer from the column to the footing, the resultant force will not coincide with the centroid of the footing. Of course, if the moment is constant in magnitude and direction, it will be possible to place the center of the footing under the resultant load and avoid the eccentricity, but lateral forces such as wind and earthquake can come from any direction, and symmetrical footings will be needed.



(a) Resultant load in kern

(b) Resultant load outside of kern

7. Rectangular Isolated Footings

Isolated footings may be rectangular in plan if the column has a very pronounced rectangular shape or if the space available for the footing forces the designer into using a rectangular shape. **Should a square footing be feasible, it is normally more desirable than a rectangular one because it will require less material and will be simpler to construct.**

The design procedure is almost identical with the one used for square footings. After the required area is calculated and the lateral dimensions are selected, the depths required for one-way and two-way shear are determined by the usual methods. **One-way shear will very often control the depths for rectangular footings, whereas two-way shear normally controls the depths of square footings.**

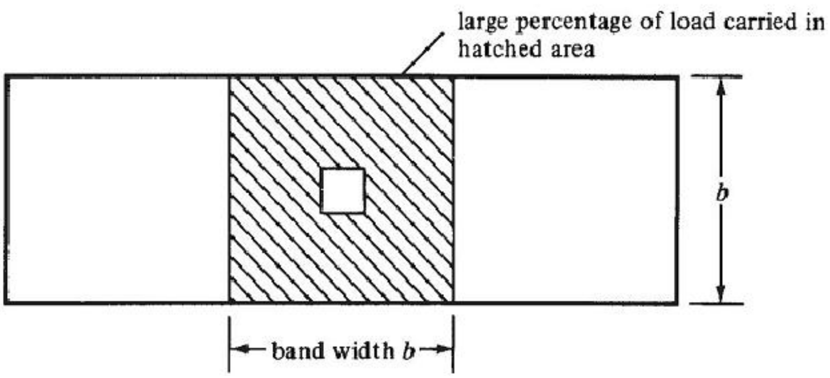
The next step is to select the reinforcing in the long direction. These longitudinal bars are spaced uniformly across the footing, but such is not the case for the short-span reinforcing. In Figure below, it can be seen that the support provided by the footing to the column will be concentrated near the middle of the footing, and thus the moment in the short direction will be concentrated somewhat in the same area near the column.

As a result of this concentration effect, it seems only logical to concentrate a large proportion of the short-span reinforcing in this area. The code (15.4.4.2) states that a certain minimum percentage of the total short-span reinforcing should be placed in a band width equal to the length of the shorter direction of the footing. The amount of reinforcing in this band is to be determined with the following expression, in which β is the ratio of the length of the long side to the width of the short side of the footing:

$$\frac{\text{Reinforcing in band width}}{\text{Total reinforcing in short direction}} = \frac{2}{\beta + 1} = \gamma_s$$

(ACI Equation 15-1)

The remaining reinforcing in the short direction should be uniformly spaced over the ends of the footing,



First, the required area of the footing is determined for the service loads, and the footing dimensions are selected so that the centroids coincide. The various loads are then multiplied by the appropriate load factors, and the shear and moment diagrams are drawn along the long side of the footing for these loads. After the shear and moment diagrams are prepared, the depth for one- and two-way shear is determined, and the reinforcing in the long direction is selected.

In the short direction, it is assumed that each column load is spread over a width in the long direction equal to the column width plus $d/2$ on each side if that much footing is available. Then the steel is designed, and a minimum amount of steel for temperature and shrinkage is provided in the remaining part of the footing.

The ACI Code does not specify an exact width for these transverse strips, and designers may make their own assumptions as to reasonable values. The width selected will probably have very little influence on the transverse bending capacity of the footing, but it can affect appreciably its punching or two-way shear resistance.

Example

Design a rectangular footing for 450mm square interior column with a dead load of 750 kN and a live load of 600 kN. Make the length of the long side equal to twice the width of the short side, f_y = 420 MPa, f'_c = 28MPa, normal weight, and q_a=200kN/m^2. Assume the base of the footing is 1.5m below grade.

Solution:

Assume 600mm thickness (d=500mm),

$$q_e = 200 - (0.6)(24) - (0.9)(16) = 171.2kN/m^2$$

$$A_{req.} = \frac{750+600}{171.2} = 7.886m^2 = 2a^2 \rightarrow a = 1.99m \qquad \text{Use 4.0x2.0m , } A_{prov.}\text{=8m}^2$$

$$q_u = \frac{(1.2)(750)+(1.6)(600)}{8} = 232.5kN/m^2$$

Check punching shear,

$$b_o = (4)(450 + 500) = 3800mm$$

V_u=(232.5)(8-0.95^2)=1650kN

$$\varphi V_c = (0.75)(0.33)(\sqrt{28})(3800)(500)/1000 = 2488kN > 1650kN \quad \text{OK}$$

Check one way shear:

$$V_u = (232.5)(2)\left(\frac{4}{2} - \frac{0.45}{2} - 0.5\right) = 593kN$$

$$\varphi V_c = (0.75)(0.17)(\sqrt{28})(2000)(500)/1000 = 674.7kN > 593kN \quad \text{OK}$$

Moments

a) Long Direction

$$lever\ arm = \left(\frac{4}{2} - \frac{0.45}{2}\right) = 1.775m$$

$$\mathrm{M_u} = (232.5)\frac{(1.775)^2}{2}(2.0) = 732.5\mathrm{kN.m}$$

$$\frac{\mathrm{M_u}}{\varphi \mathrm{bd}^2} = \frac{(732.5)(10)^6}{(0.9)(2000)(500)^2} = 1.63 \rightarrow \rho = 0.004$$

Check min. ratio which is larger of ($\rho = \frac{1.4}{420} = 0.0033$, $\rho = 0.25\frac{\sqrt{28}}{420} = 0.0032$)

A_s=(0.004)(2000)(500)=4000mm^2 or 2000mm^2/m

Use Ø25/200, $(A_s)_{Prov.}$=2450mm^2/m

b) Short Direction

$$lever\ arm = \left(\frac{2}{2} - \frac{0.45}{2}\right) = 0.775m$$

$$\mathrm{M_u} = (232.5)\frac{(0.775)^2}{2}(4.0) = 279\mathrm{kN.m}$$

$$\frac{\mathrm{M_u}}{\varphi \mathrm{bd}^2} = \frac{(279)(10)^6}{(0.9)(2000)(500)^2} = 0.31 \rightarrow \rho = 0.00075$$

Check min. ratio which is larger of ($\rho = \frac{1.4}{420} = 0.0033$, $\rho = 0.25\frac{\sqrt{28}}{420} = 0.0032$)

A_s=(0.0033)(4000)(500)=6600mm^2

Use 21-Ø20, $(A_s)_{Prov.}$≈6600mm^2

$$\frac{\text{Reinf. in band width}}{\text{Total Reinf. in short direction}} = \frac{2}{\frac{4}{2}+1} = 0.67$$

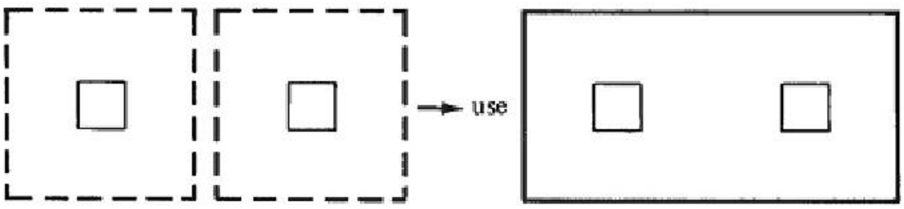
$$(0.67)(20) = 14 bars \rightarrow use\ 14\ bars\ in\ band\ width = 2.0m$$

Check development length.

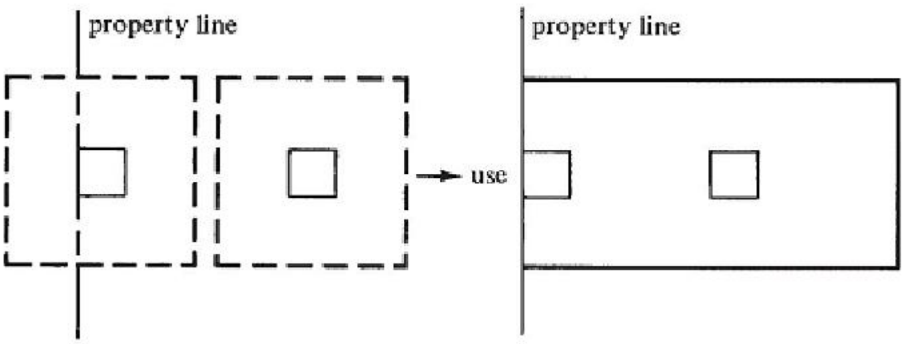
8. Combined Footings

Combined footings support more than one column. One situation in which they may be used is when the columns are so close together that isolated individual footings would run into each other [see Figure below]. Another frequent use of combined footings occurs where one column is very close to a property line, causing the usual isolated footing to extend across the line. For this situation, the footing for the exterior column may be combined with the one for an interior column.

Because it is desirable to make bearing pressures uniform throughout the footing, the centroid of the footing should be made to coincide with the centroid of the column loads to attempt to prevent uneven settlements. This can be accomplished with combined footings that are rectangular in plan. Should the interior column load be greater than that of the exterior column, the footing may be so proportioned that its centroid will be in the correct position by extending the inward projection of the footing.



(a)



(b)

The largest shear force =1340kN, at distance "d" from the face of support:

$V_u = 1340 - (800)(0.6) = 860kN$

$\varphi V_c = (0.75)(0.17)(\sqrt{21})(2750)(600)/1000 = 964kN > 860kN$ OK

Check punching shear,

At right column,

$V_u=2320-(1.1)^2(290.9)=1968kN$

$\varphi V_c = (0.75)(0.33)(\sqrt{21})(1100)(4)(600)/1000 = 2994kN$ OK

At left column,

$V_u=1520-(0.75)(1.05)(290.9)=1290.9kN$

$\varphi V_c = (0.75)(0.33)(\sqrt{21})(750 + 750 + 1050)(600)/1000 = 1735.3kN$ OK

Moments

a) Longitudinal Direction

- Negative Reinf.

Max. $M_u = 1102kN.m$

$$\frac{M_u}{\varphi bd^2} = \frac{(1102)(10)^6}{(0.9)(2750)(600)^2} = 1.236 \rightarrow \rho = 0.00305$$

Check min. ratio which is larger of ($\rho = \frac{1.4}{420} = 0.0033$, $\rho = 0.25\frac{\sqrt{28}}{420} = 0.0032$)

$A_s=(0.0033)(2750)(600)=5455mm^2$

Use 15-Ø22, $(A_s)_{Prov.}=5700mm^2$

- Positive Reinf.

At face of column, max. $M_u = 210.3kN.m$

$\rho = 0.00056$

Check min. ratio which is larger of ($\rho = \frac{1.4}{420} = 0.0033$, $\rho = 0.25\frac{\sqrt{28}}{420} = 0.0032$)

$A_s=(0.0033)(2750)(600)=5500mm^2$

Use 15-Ø22, $(A_s)_{Prov.}=5700mm^2$

b) Transverse Direction

Assuming steel spread over width of 500+d/2+d/2=1100mm

$q_u = \frac{2320}{2.75} = 844kN/m$

$$M_u = (844)\left(\frac{1.125^2}{2}\right) = 534kN.m \rightarrow \rho = 0.0037$$

A_s=(0.0037)(1100)(600)=2442mm^2

Use 7-Ø22, $(A_s)_{Prov.}$=2660mm^2

- A similar procedure is used for moments at exterior column with b=450+d/2 = 750mm.

Check development length.

A_s=(0.0035)(3000)(600)=6300mm^2

Use 13-Ø25, $(A_s)_{Prov.}$=6383mm^2

b) Short Direction

$$lever\ arm = \left(\frac{3}{2} - \frac{0.4}{2}\right) = 1.3m$$

$$\mathrm{M_u} = (308)\frac{(1.3)^2}{2}(4.0) = 1040\mathrm{kN.m}$$

$$\frac{\mathrm{M_u}}{\varphi b d^2} = \frac{(1040)(10)^6}{(0.9)(4000)(500)^2} = 0.8 \rightarrow \rho = 0.002$$

Check min. ratio which is larger of ($\rho = \frac{1.4}{420} = 0.0033$, $\rho = 0.25\frac{\sqrt{28}}{420} = 0.0032$)

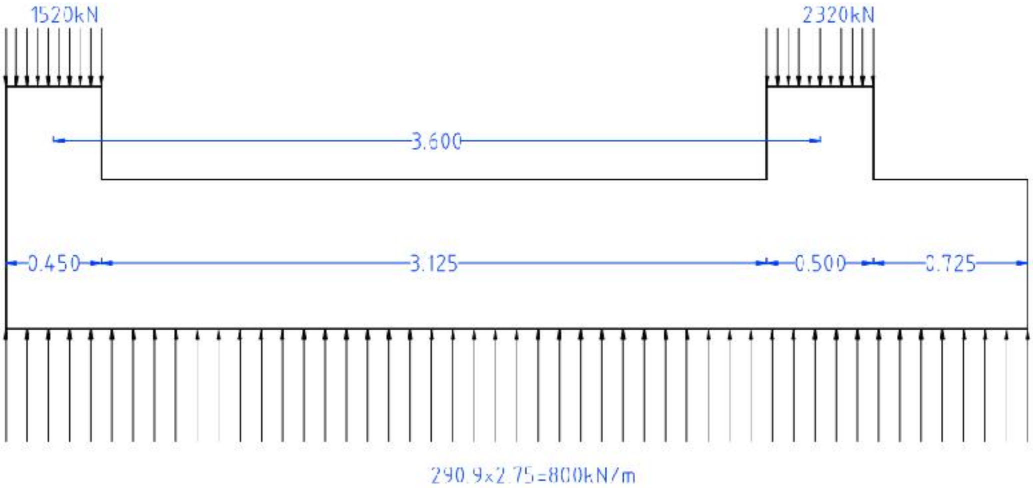
A_s=(0.0033)(4000)(600)=7920mm^2

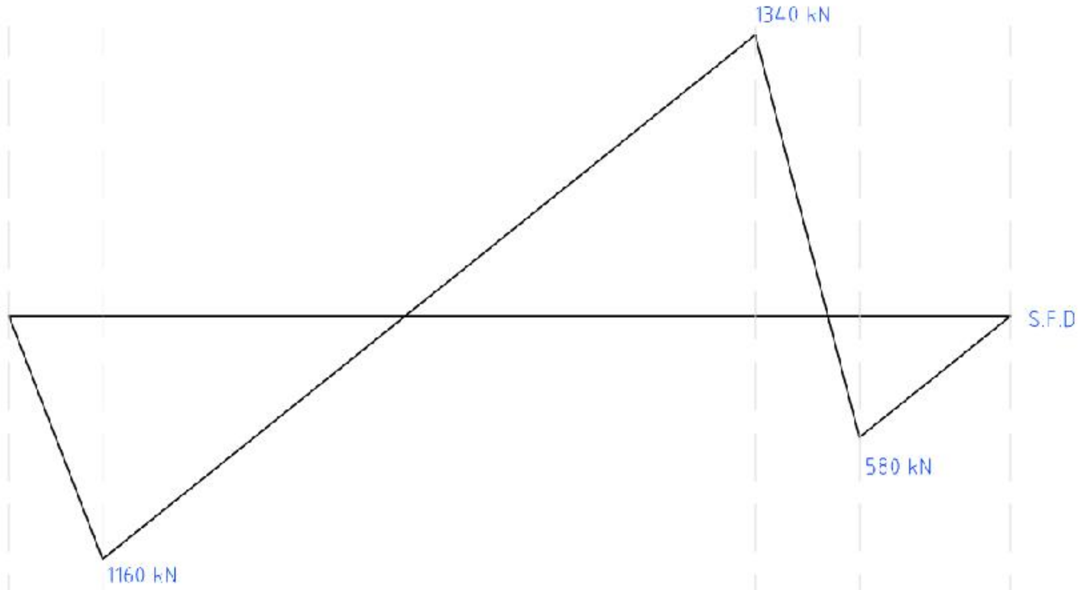
Use 21-Ø22, $(A_s)_{Prov.}$≈7980mm^2

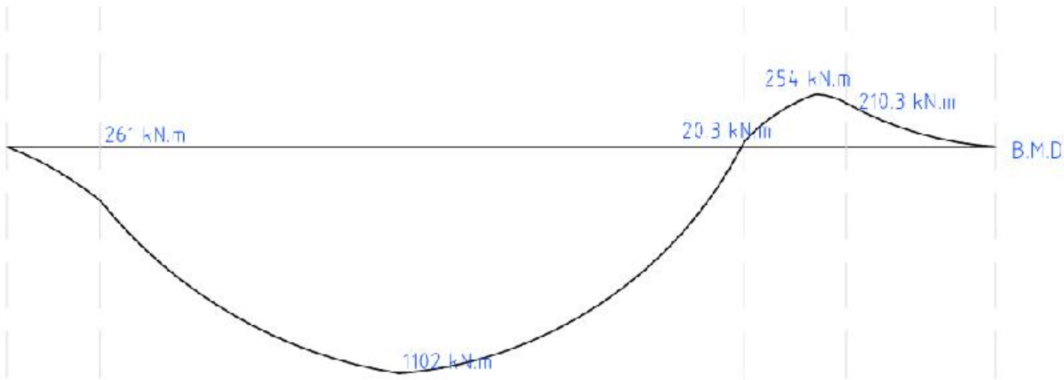
$$\frac{\text{Reinf. in band width}}{\text{Total Reinf. in short direction}} = \frac{2}{\frac{4}{3}+1} = 0.86$$

$$(0.86)(21) = 18bars \rightarrow use\ 18\ bars\ in\ band\ width = 3.0m$$

<u>Check development length.</u>


1520kN
2320kN
3.600
0.450
3.125
0.500
0.725
290.9x2.75=800kN/m

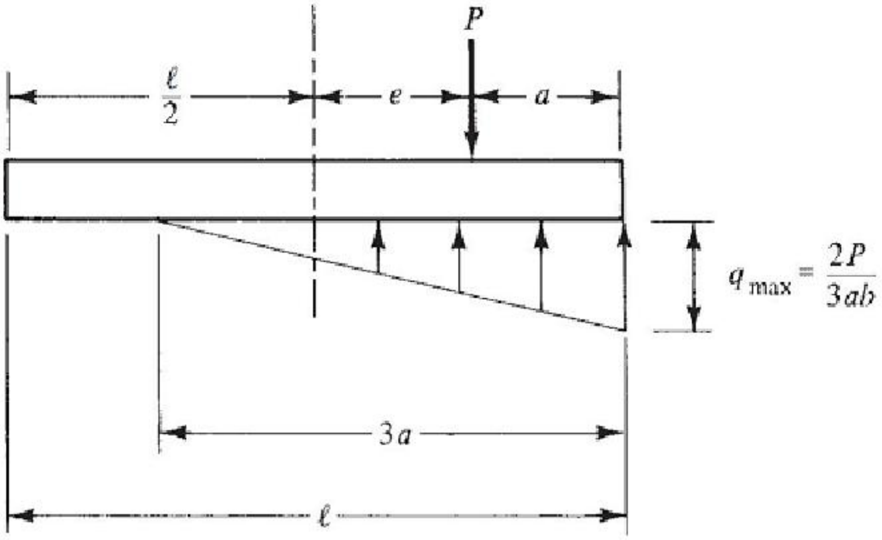

1340 kN
S.F.D
580 kN
1160 kN


254 kN.m
210.3 kN.m
261 kN.m
20.3 kN.m
B.M.D
1102 kN.m

The effect of the moment is to produce a linearly varying soil pressure, which can be determined at any point with the expression

$$q = -\frac{P}{A} \pm \frac{Mc}{I}$$

If the resultant force strikes the footing base within the kern, the value of *-P/A* is larger than *+Mc/I* at every point, and the entire footing base is in compression, as shown in Figure below (a). If the resultant force strikes the footing base outside the kern, the value of + *Mc/I* will at some points be larger than *-P/A,* and there will be uplift or tension. The soil-footing interface cannot resist tension, and the pressure variation will be as shown in Figure below (b). The location of the kern can be determined by replacing *Mc/I* with *Pec/I,* equating it to *P/A,* and solving for *e.*



Should the eccentricity be larger than this value, the method described for calculating soil pressures [(*-P/A)* ± *(Mc/I*)] is not correct. To compute the pressure for such a situation, it is necessary to realize that the centroid of the upward pressure must for equilibrium coincide with the centroid of the vertical component of the downward load. In Figure, it is assumed that the distance to this point from the right edge of the footing is *a.* Since the centroid of a triangle is located at one-third of its base, the soil pressure will be spread over the distance 3a as shown. For a rectangular footing with dimensions $L \times b$, the total upward soil pressure is equated to the downward load, and the resulting expression is solved for q_{max} as follows:

$$\left(\frac{1}{2}\right)(3ab)(q_{max}) = P$$

$$q_{max} = \frac{2P}{3ab}$$

Once the area has been established, the remaining design will be handled as it was for other footings.

Example

Determine the width needed for a wall footing to support loads: *D.L*= 260 kN/m and L.L = 175 kN/m. In addition, a moment of 50kN.m must be transferred from the column to the footing. Assume the footing is 450 thick, its base is 1.2m below the final grade, and $q_a = 200$ kN/m^2.

$q_e = 200 - (0.45)(24) - (0.75)(16) = 177.2kN/m^2$

$width_{req.} = \frac{260+175}{177.2} = 2.45m$

Use 2.5m width

$A = (2.5)(1) = 2.5m^2$

$I = (\frac{1}{12})(1)(2.5)^3 = 1.3m^3$

$$q_{max.} = -\frac{P}{A} - \frac{Mc}{I} = -\frac{435}{2.5} - \frac{(50)(1.25)}{1.3} = -222\frac{kN}{m^2} > 177.2\frac{kN}{m^2}\ Not\ OK!$$

$$q_{min.} = -\frac{P}{A} + \frac{Mc}{I} = -\frac{435}{2.5} + \frac{(50)(1.25)}{1.3} = -125.9kN/m^2$$

Use 3.5m width

$A = (3.5)(1) = 3.5m^2$

$I = (\frac{1}{12})(1)(3.5)^3 = 3.57m^3$

$$q_{max.} = -\frac{435}{3.5} - \frac{(50)(1.75)}{3.57} = -148.8kN/m^2$$

$$q_{min.} = -\frac{435}{3.5} + \frac{(50)(1.75)}{3.57} = -99.8kN/m^2$$

Example

Determine the width needed for a wall footing to support loads: *D.L*= 260 kN/m and L.L = 175 kN/m. In addition, a service moment of 50kN.m must be transferred from the column to the footing. Assume the footing is 450 thick, its base is 1.2m below the final grade, and q_a = 200 kN/m^2.

$$q_e = 200 - (0.45)(24) - (0.75)(16) = 177.2kN/m^2$$

$$width_{req.} = \frac{260+175}{177.2} = 2.45m$$

Use 2.5m width

$$A = (2.5)(1) = 2.5m^2$$

$$I = (\frac{1}{12})(1)(2.5)^3 = 1.3m^4$$

$$e = \frac{50}{260+175} = 0.115m = 115mm < \frac{2500}{6} = 416.7mm$$

$$q_{max.} = -\frac{P}{A} - \frac{Mc}{I} = -\frac{435}{2.5} - \frac{(50)(1.25)}{1.3} = -222\frac{kN}{m^2} > 177.2\frac{kN}{m^2}\ Not\ OK!$$

$$q_{min.} = -\frac{P}{A} + \frac{Mc}{I} = -\frac{435}{2.5} + \frac{(50)(1.25)}{1.3} = -125.9kN/m^2$$

Use 3.5m width

$$A = (3.5)(1) = 3.5m^2$$

$$I = (\frac{1}{12})(1)(3.5)^3 = 3.57m^4$$

$$q_{max.} = -\frac{435}{3.5} - \frac{(50)(1.75)}{3.57} = -148.8kN/m^2$$

$$q_{min.} = -\frac{435}{3.5} + \frac{(50)(1.75)}{3.57} = -99.8kN/m^2$$

Example

Design a rectangular footing for 400*500mm interior column with a service loads of (dead load =1300 kN, live load= 1100 kN , dead load moment=100 kN.m & live load moment = 80kN.m). Assume the length of the short side equal to 0.75 the length of the long side, f_y = 420 MPa, f'_c = 28MPa, normal weight, and q_a=260kN/m^2. The base of the footing is 1.5m below grade.

Solution:

Assume 700mm thickness (d=600mm),

$q_e = 200 - (0.7)(24) - (0.8)(16) \cong 230kN/m^2$

$A_{req.} = \frac{1300+1100}{230} = 10.4m^2 = 0.75L^2 \rightarrow L = 3.72m$ Use 4.0x3.0m , $A_{prov.}$=12m^2

$e = \frac{180}{2400} = 0.075m = 75mm < \frac{4000}{6} = 666.7mm$

$q_{max.} = -\frac{2400}{12} - \frac{(6)(180)}{(3)(4)^2} = -222.5\frac{kN}{m^2} < 230\frac{kN}{m^2}\ OK$

$q_{min.} = 177.5kN/m^2$

$q_{u_{max.}} = -\frac{(1.2)(1300) + (1.6)(1100)}{12} - \frac{(6)[(1.2)(100) + (1.6)(80)]}{(3)(4)^2} = 308\frac{kN}{m^2}$

$q_{min.} = -246kN/m^2$

Check punching shear,

$b_o = 4200mm$

$V_u = (\frac{308+246}{2})((12 - (1)(1.1))$=3019kN

$\varphi V_c = (0.75)(0.33)(\sqrt{28})(4200)(600)/1000 = 3303kN > 3019kN$ OK

Check one way shear:

q_u at the center of column = = $\frac{308+246}{2}$=277kN/m^2

q_u at distance d from column face =290.175kN/m^2

$V_u = (\frac{308+290.2}{2})(1.15)(3)$=1031$kN$

$\varphi V_c = (0.75)(0.17)(\sqrt{28})(3000)(600)/1000 = 1204kN$ OK

Moments

a) Long Direction

q_u at column face =281kN/m^2

$$M_u = \left[(281)\frac{(1.75)^2}{2} + \frac{\frac{(27)(1.75)}{2}(2)(1.75)}{3}\right](3) = 1374\text{kN.m}$$

$$\frac{M_u}{\varphi bd^2} = \frac{(1374)(10)^6}{(0.9)(3000)(600)^2} = 1.3 \rightarrow \rho = 0.0035$$

Check min. ratio which is larger of ($\rho = \frac{1.4}{420} = 0.0033, \rho = 0.25\frac{\sqrt{28}}{420} = 0.0032$)

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