CIVL471 DESIGN OF RC STRUCTURES

LECTURE NOTE #14

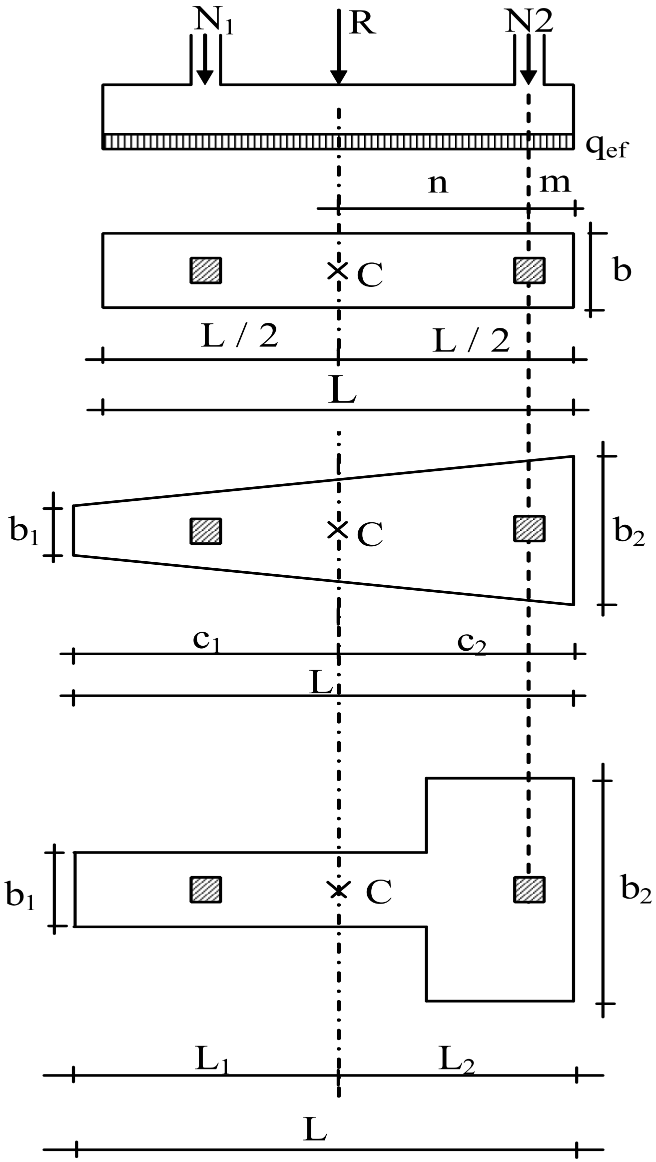
CHAPTER V

FOOTINGS and FOUNDATIONS (CONT.)

5.6.2 DESIGN PRINCIPLES OF COMBINED FOOTINGS (TS500 REQUIREMENTS)

In combined footings there may be beam parts and plate parts. If there is a beam part, total height of the beam should not be less than 1/10 of the clear span. Plate part of a continuous footing may not be thinner than 20 cm. Design shear force should be calculated at the face of the column. If there is not a beam part in a continuous footing, plate thickness should not be less than 30 cm. Requirements of TS500 for the reinforcement of the beams and the plates can also apply to the beam and plate parts of the footings. There must be reinforcement in the compression part of the `continuous footing equal to at least one third of the tension reinforcement.

5.6.3 TWO-COLUMN COMBINED FOOTINGS



$$L = 2(n + m)$$

$$b = R / (q_{ef} L)$$

$$\frac{b_2}{b_1} = \frac{3(n+m) - L}{2L - 3(n+m)} \qquad b_1 + b_2 = \frac{2R}{q_{ef} L}$$

$$c_1 = \frac{L(b_1 + 2b_2)}{3(b_1 + b_2)} \qquad c_2 = \frac{L(2b_1 + b_2)}{3(b_1 + b_2)}$$

$$b_1 = \frac{R}{q_{ef}} \; \frac{2(n+m) - L_2}{L_1(L_1 + L_2)}$$

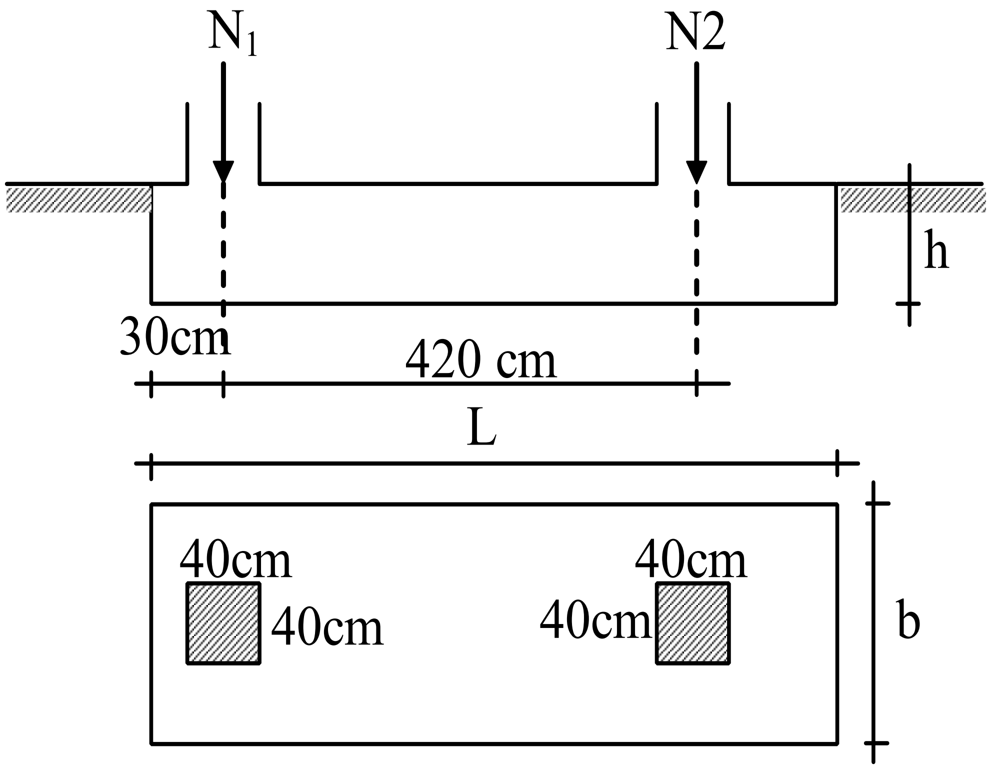
$$b_2 = \frac{R}{L_2 q_{ef}} - \frac{L_1}{L_2} b_1$$

$$L_1 b_1 + L_2 b_2 = \frac{R}{q_{ef}}$$

Figure 5.25

These footings generally combine two very close columns or two columns one of which is very close to the next building (or property line). They are usually designed such that the resultant of the column loads passes through the centroid of the bearing area. Thus it is assumed that the soil stresses under the footing are uniformly distributed. The shape of the bearing area may be rectangular, trapezoidal or T shaped depending on the relative values of the column loads. In Fig. 5.25 some equations are given to simplify the determination of the bearing area dimensions for these shapes. These equations are based on the assumption that the resultant (R) of the column loads passes through the centroid of the bearing area.

Example 5.4



Dead loads: $N_{d1} = 235$ kN
$N_{d2} = 365$ kN

Live loads: $N_{l1} = 180$ kN
$N_{l2} = 280$ kN

$q_a = 245\ kN/m^2$

Materials: C18 S420

Figure 5.26

One end of the footing is limited by the property line which is 30 cm away from the column center as shown in Fig.5.26. Design this combined footing.

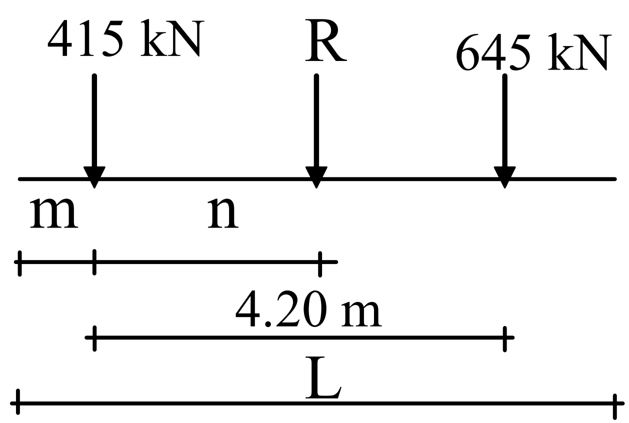
Solution:

Let h =100 cm >380/10=38cm q_{ef} = 245−1*25 = 245 − 25 = 220 kN/m^2

Resultant of the column loads and the location of the resultant force:

N_1 = 235 + 180 =415 kN N_2 = 365 + 280 =645 kN

R = 415 + 645 = 1060 kN



Moment equilibrium:

$$R.n = 1060n = 645*4.2 = 2709 \text{ kN-m}$$

$$n = 2709 / 1060 = 2.55 \text{ m.} \qquad m + n = 0.3 + 2.55 = 2.85 \text{ m}$$

$$L = 2(n + m) = 2*2.85 = 5.70 \text{ m.}$$

Required minimum bearing area and minimum b:

$$\min(bL) = \frac{R}{q_{ef}} = \frac{1060}{220} = 4.82 \text{ m}^2 \qquad \min b = \frac{4.82}{L} = \frac{4.82}{5.7} = 0.85 \text{ m}$$

Selected b = 95 cm.

Two column combined footings may be analyzed like beams. The soil pressure corresponds to distributed load and column loads correspond to support reactions. First shear force diagram is drawn and then moments are calculated from the shear areas. In the following these calculations are given.

Design in long direction:

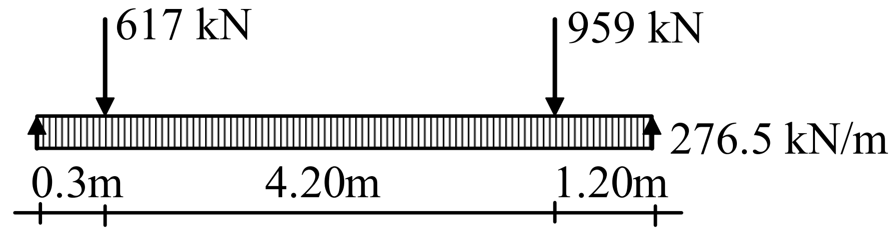
$N_{1u} = 1.4*235 + 1.6*180 = 617$ kN $N_{2u} = 1.4*365 + 1.6*280 = 959$ kN

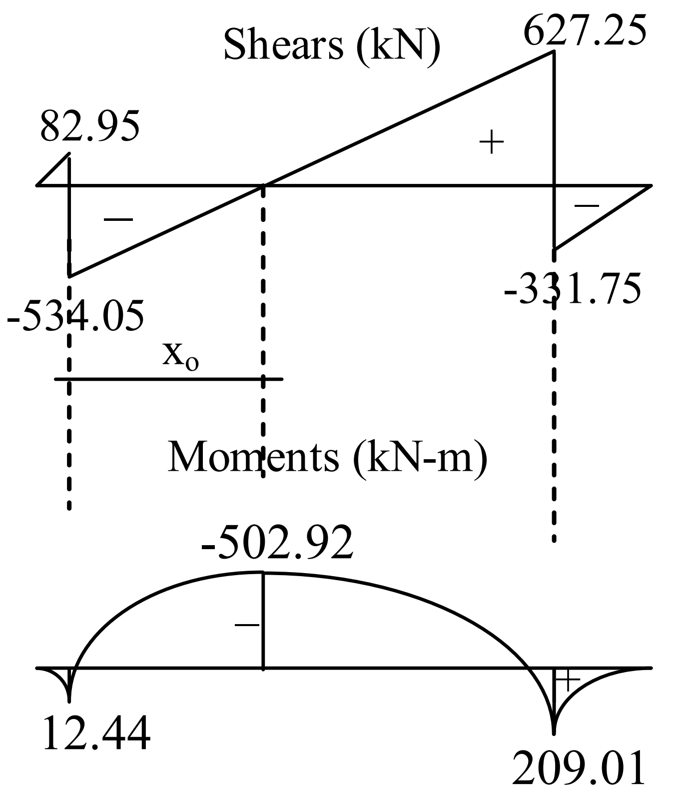
$R_u = 617 + 959 = 1576$ kN

Design soil pressure per linear meter:

$$q_u = \frac{R_u}{L} = \frac{1576}{5.7} = 276.5 \text{ kN / m}$$

Shear forces and bending moments:





Shear forces:

$276.5*.0.30 = 82.95$ kN

$82.95 - 617 = - 534.05$ kN

$- 534.05 + 276.5*4.2 = 627.25$ kN

$627.25 - 959 = - 331.75$ kN

$- 331.75 + 276.5*1.2 = 0$

$$x_o = \frac{534.05}{276.5} = 1.93 \text{ m}$$

Moments:

$82.95*0.3/2 = 12.44$ kN-m

$12.44 - 1.93*534.05/2 = - 502.92$ kN-m

$- 502.92 + 627.25* (4.2 - 1.93) /2 = 209.01$ kN-m

Punching shear:

Punching perimeter can not develop in this narrow footing. Therefore punching shear failure is not possible.

One-way shear check:

Design shear force will be calculated at the face of the second column since maximum shear force is 627.35 kN in this column. If d = 100 − 7 = 93 cm

$$V_d = 627.25 - 0.20*276.5 = 572.2 \text{ kN}$$

$$V_{cr} = (0.65*1*950*930) / 1000 = 574.28 \text{ kN} > V_d$$

Shear reinforcement is not necessary but stirrups will be provided for assembling purposes.

Bending design:

- $M = 12.44$ kN-m is too small to be considered.

- $M = 502.92$ kN-m = 5029200 kg-cm $\qquad R = \dfrac{5029200}{95*93^2} = 6.12 \text{ kg/cm}^2$

$\rho < \rho_{min} = 0.0022 \qquad A_s = 0.0022*95*93 = 19.44 \text{ cm}^2$

Selected: 6Ø22 (22.81 cm^2) (At the top)

- $M = 209.10$ kN-m < 502.92 kN-m Select : 6Ø22 (At the bottom)

Four of these six bottom bars (more than one third of the tension reinforcement) will continue across the compression zone and be cut at the other end of the footing. The height of the beam is more than 60 cm; therefore longitudinal web reinforcement should be provided. According to TS500 minimum area of these bars is $0.001 b_w d$.

$0.001*95*93 = 8.84 \text{ cm}^2$ Selected: 4Ø18 (10.18 cm^2)

For assembling the reinforcement stirrups with 30 cm spacing will be provided. Two stirrups per set will be used since the footing is rather wide. Details are shown in Fig. 5.27. Development lengths should be checked especially for the bars provided for the positive moments.

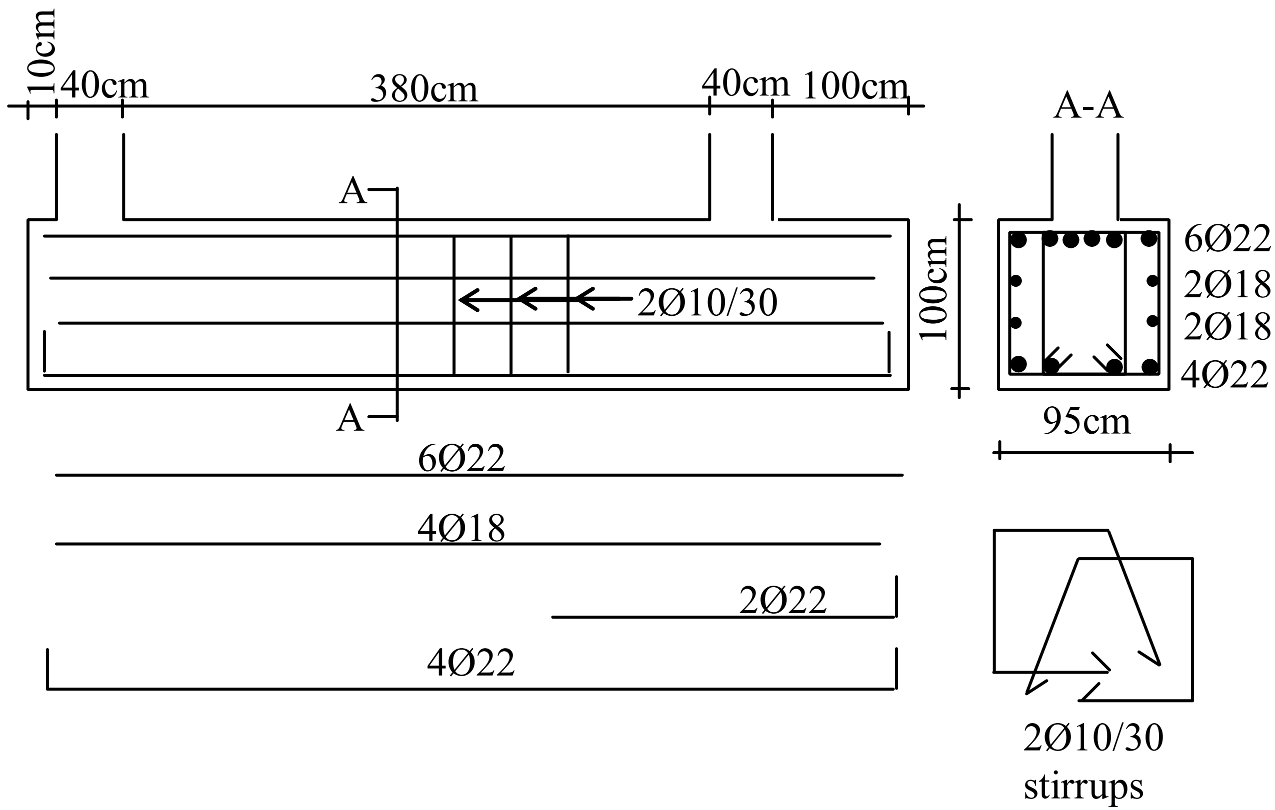


Figure 5.27

If column dimensions are large and support moments are high adjustments may be made on support moments. Shear diagrams between the faces of columns may be drawn more accurately taking the column loads as uniformly distributed loads as shown in Fig.5.28. Then maximum moment is calculated at the section where shear force is zero. In the figure shear forces and the moments at the bottom of the right column are shown just as an example.

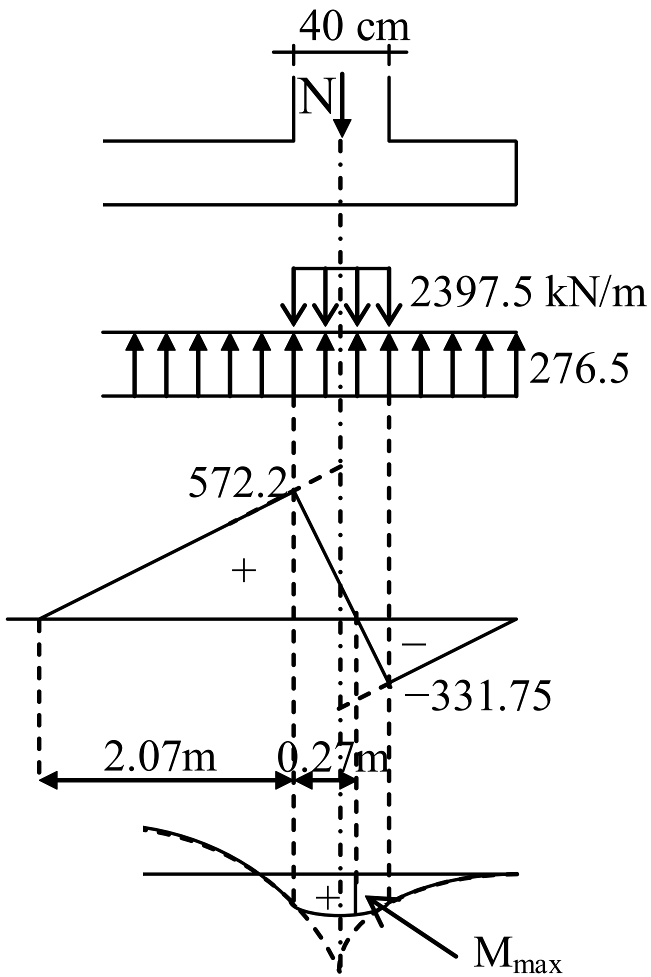


Figure 5.28

Distribured column load:

$$\frac{N}{0.40} = \frac{959}{0.40} = 2397.5 \text{ kN/m}$$

Uniformly distributed load across the column width:
$2397.5 - 276.5 = 2121$ kN/m

Distance of the section where shear force is zero:

$$572.2 / 2121 = 0.27 \text{ m}$$

$$M_{max} = -502.92 + \frac{(2.07 + 0.27) * 572.2}{2}$$
$$= 166.55 \text{ kN-m}$$

Footing designed in this example was a narrow one. Therefore in the short direction no calculation was necessary. There may be cases where the width of the footing is much larger than the column width. In such cases in column areas reinforcement parallel to the short sides should also be provided. In Fig. 5.29 these areas are shown. They are transverse strips and designed as cantilevers. The widths of these cantilevers are equal to the widths of columns plus d/2 at two sides. The pressure acting below the cantilever is found dividing the column load by b. Shear force and bending moment are computed at section I-I. Calculated reinforcement is placed over the main reinforcement.

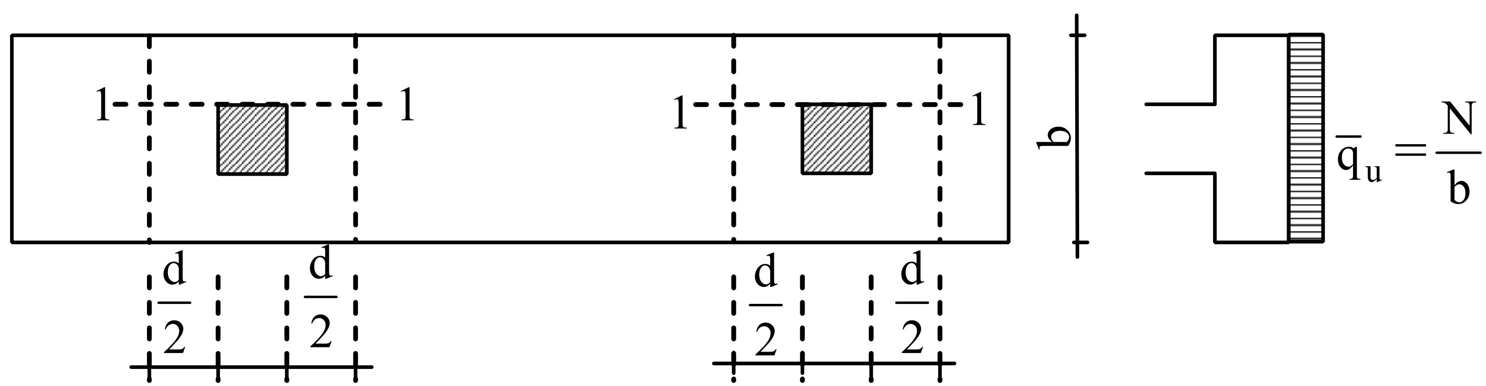


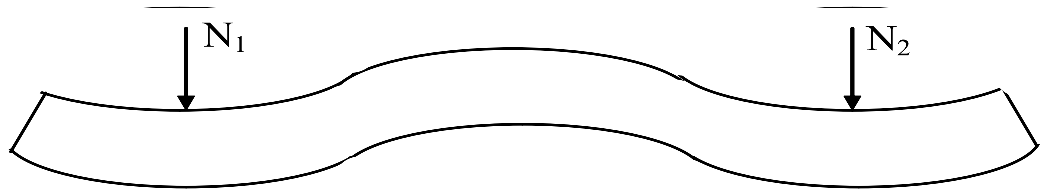
Figure 5.29

5.7 CONTINUOUS STRIP FOOTINGS

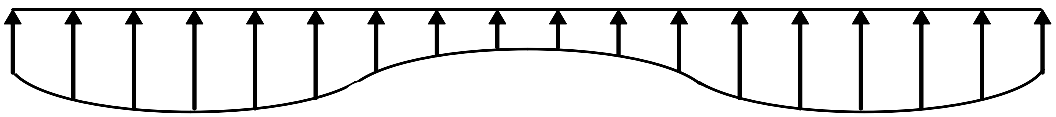
Continuous strip footings are the combined footings that support some or all columns of a frame. As mentioned earlier if soil is not suitable for making single column footings, continuous footings may be the solution for the foundation problem. Besides, instead of making single column footings and tying them with tie-beams, making strip footings in one direction and tying them in the other direction may be more suitable in earthquake zones. Soil pressure distribution under a strip footing depends on the rigidity of the footing and on the compressibility of the soil. For compressible soils it is possible to assume that the soil pressure and the settlement of the footing at any point are proportional to each other. By this assumption and by using the theory of the beams on elastic foundations, a very reasonable solution can be obtained. However this method is not very practical and the results will not be realistic if true soil properties are not defined very well. For this reason in practice generally some approximate and easier methods are used.

If number of columns are small and spans are not too large footings behave rigidly. Deformations of the rigid footings are linear. Therefore soil stress distribution may also be assumed as linearly varying. As a special case uniformly distributed pressures may be assumed if the resultant of the column loads is passing across the centroid of the footing. Rigid continuous strip footings may be designed exactly in the same way as the two-column combined footings.

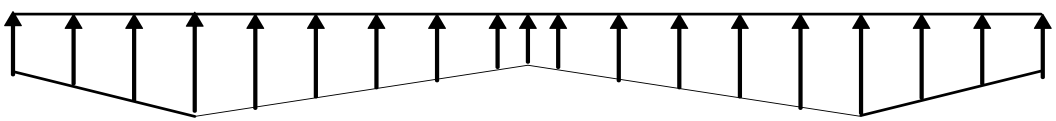
On the other hand if spans are large, footing may not behave as a rigid one. It behaves as a flexible footing. Under the flexible footings soil pressures vary as shown in Fig.5.30b. This distribution may be approximated by triangular or uniform distributions as shown in Figs.5.30c and 5.30d.



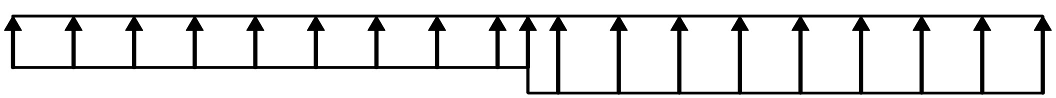
a) Deformation of flexible footing



b) Real distribution



c) Approximation as linear distribution



d) Approximation as uniform distribution

Figure 5.30

The following approximate design method assumes uniform soil distributions under the columns as shown in Fig. 5.30d.

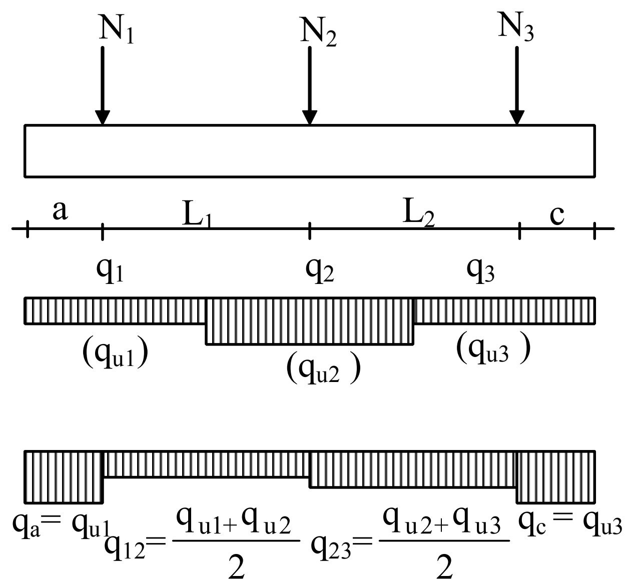


Figure 5.31

At first, it is assumed that each column has an effective length on the footing. This length is equal to the distance between the middle points of the spans. If there are cantilevering parts at the ends, they are included in the effective lengths of the external columns as shown in the Fig. 5.31. For each column uniformly distributed linear soil load is determined dividing the column load by the effective length. Thus q_1, q_2, q_3 etc. are calculated. The width of the footing is computed dividing maximum of them by q_{ef}. For design q_{u1}, q_{u2}, q_{u3} etc. are calculated like q_1, q_2, q_3 etc. are calculated but this time by using design column loads (factored loads). At last, average soil loads are found as shown in Fig.5.31 and shear forces and bending moments are calculated by using these average loads.

5.8 GRID FOUNDATIONS

Grid foundations are essentially strip footings arranged in two directions. They are more effective than one-way strip footings for the prevention of differential settlement. The theory of beams on elastic foundations may also be applied to grid foundations. However in practice generally approximate, simple but sufficiently accurate methods are used. As an example, one of these widely used methods is given below. In Fig.5.34 a grid foundation example supporting 9 columns is shown. Design method will be described on this example.

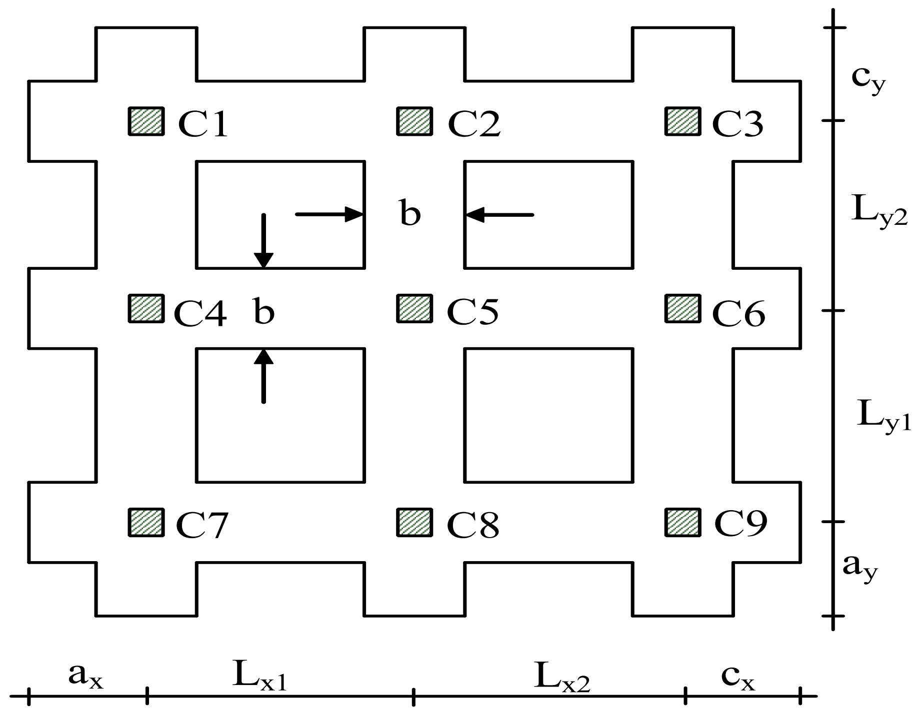


Figure 5.34

It is assumed in this method that while one part of the column load is spread in one direction the remaining part is spread in the other direction. After determining these portions of the column loads, each strip footing is analyzed and designed independently. To simplify the computations it is advised to select the footing widths equal in both directions. For each column effective lengths are defined in both directions as the lengths between the midspans. For example effective lengths for the column C1 shown in Fig.5.34 are:

$$L_{ex} = a_x + \frac{L_{x1}}{2} \qquad \text{and} \qquad L_{ey} = c_y + \frac{L_{y2}}{2}$$

Effective lengths for the column C2:

$$L_{ex} = \frac{L_{x1} + L_{x2}}{2} \qquad \text{and} \qquad L_{ey} = c_y + \frac{L_{y2}}{2}$$

Effective lengths for the column C5:

$$L_{ex} = \frac{L_{x1} + L_{x2}}{2} \qquad \text{and} \qquad L_{ey} = \frac{L_{y1} + L_{y2}}{2}$$

Effective lengths for all other columns are found similarly. The bearing area under the effective lengths of a column is defined as "effective area" of the column. If "b" is the widths of the strips the effective area "A" is:

$$A = L_{ex}b + L_{ey}b - b^2 \tag{5.9}$$

If a uniform pressure distribution is assumed under the effective area,

$$A = \frac{N}{q_{ef}} \tag{5.10}$$

By equating the right sides of two equations:

$$\frac{N}{q_{ef}} = (L_{ex} + L_{ey})b - b^2 \tag{5 .11}$$

By solving Eq.(5.11) the value of "b" for this particular column can be calculated. It is most likely that for each column a different "b" will be calculated. Maximum of them is selected for the widths of all the strips. The portion of the column loads considered in x and y direction strips are found as follows:

$$N_x = \frac{A_x}{A_x + A_y} N = \frac{L_{ex} b}{L_{ex} b + L_{ey} b} N = \frac{L_{ex}}{L_{ex} + L_{ey}} N \tag{5.12}$$

$$N_y = \frac{A_y}{A_x + A_y} N = \frac{L_{ey} b}{L_{ex} b + L_{ey} b} N = \frac{L_{ey}}{L_{ex} + L_{ey}} N \tag{5.13}$$

Using factored loads in Eqs.(5.12) and (5.13) column loads can be computed for each footing. Then they can be designed by the methods given in the previous section. That is either rigid beam or flexible beam approach is employed according to the rigidity of the footing.

5.9 MAT FOUNDATIONS

Mat foundations are thick solid plates as mentioned earlier. The design of them also depends on the rigidities of the plates. Under the rigid foundations soil pressure distributions may be assumed as uniform or linearly varying. They may be designed like beamless slabs. Sometimes mat foundations may have beam parts as shown in Fig. 5.35. These foundations may be designed like slabs supported by the beams. If soil pressure is not uniform because of eccentricity, average soil pressure may be used in design.

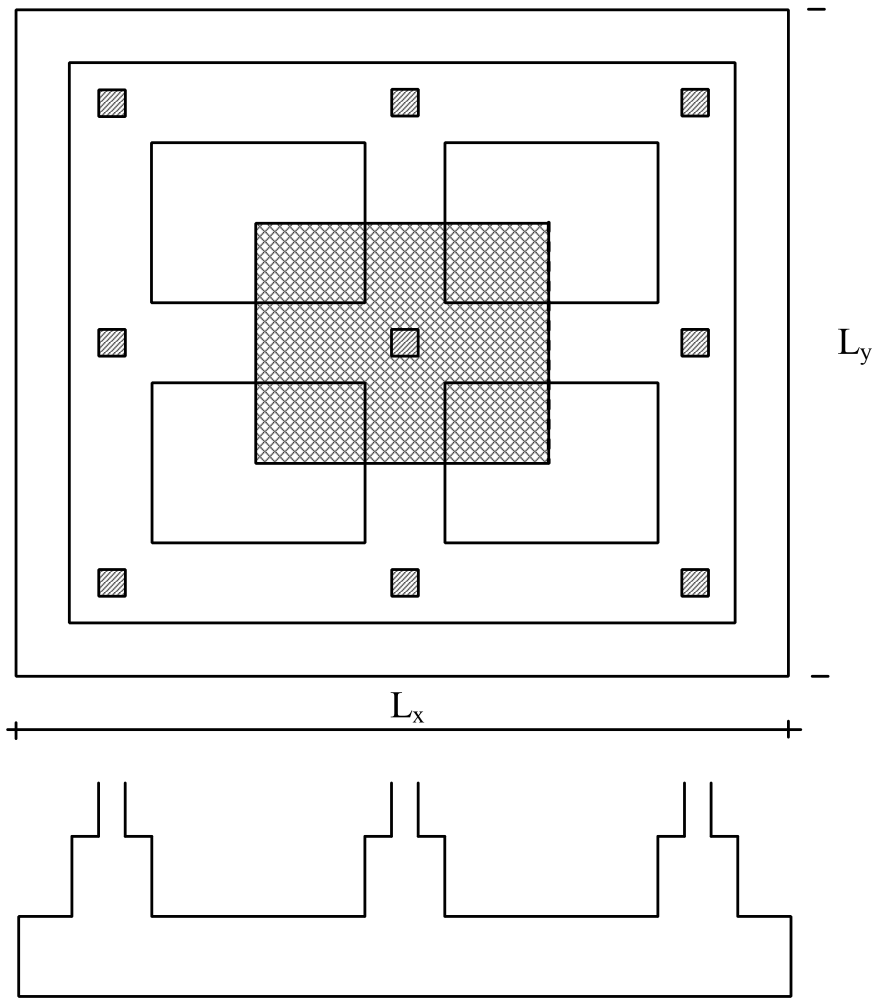


Figure 5.35

These foundations may be designed like slabs supported by the beams. If soil pressure is not uniform because of eccentricity, average soil pressure may be used in design.

If foundation is assumed flexible, analysis may be done as follows: Effective areas are defined for columns which are the areas within the middle lines of the slab parts as shown in Fig.5.35 (shaded area). The soil pressure for each effective area is calculated. Thus four different pressure values are computed for every corner quarter of a slab, but slabs are designed by using average of these four pressure values. The design of the beam parts is similar to those of the grid foundations.

Bearing area of mat foundation may be computed by using average soil pressure:

$$q_{av} = \frac{\Sigma N}{L_x L_y} \leq q_{ef} \tag{5.14}$$

where ΣN is the sum of the column loads. From this equation,

$$L_x L_y \geq (\Sigma N / q_{ef}) \tag{5.15}$$

During the calculation of q_{ef} usual reductions should be made in allowable soil pressure. However if there is basement floor over the foundation, live load of the basement floor should also be substracted from the allowable soil pressure.