In general, the distances m and n are given, the former being the distance from the center of the exterior column to the property line and the latter the distance from that column to the resultant of both column loads.

Another expedient that is used if a single footing cannot be centered under an exterior column is to place the exterior column footing eccentrically and to connect it with the nearest interior column footing by a beam or strap. This strap, being counterweighted by the interior column load, resists the tilting tendency of the eccentric exterior footings and equalizes the pressure under it. Such foundations are known as *strap, cantilever,* or *connected footings.*

The two examples that follow demonstrate some of the peculiarities of the design of two-column footings.

Example 16.3. Design of a combined footing supporting one exterior and one interior column. An exterior 24 × 18 in. column with $D = 170$ kips, $L = 130$ kips, and an interior 24 × 24 in. column with $D = 250$ kips, $L = 200$ kips are to be supported on a combined rectangular footing whose outer end cannot protrude beyond the outer face of the exterior column (see Fig. 16.1). The distance center to center of columns is 18 ft 0 in., and the allowable bearing pressure of the soil is 6000 psf. The bottom of the footing is 6 ft below grade, and a surcharge of 100 psf is specified on the surface. Design the footing for $f'_c = 3000$ psi, $f_y = 60{,}000$ psi.

Solution. The space between the bottom of the footing and the surface will be occupied partly by concrete (footing, concrete floor) and partly by backfill. An average unit weight of 125 pcf can be assumed. Hence, the effective portion of the allowable bearing pressure that is available for carrying the column loads is $q_e = q_a -$ (weight of fill and concrete + surcharge) $= 6000 - (6 \times 125 + 100) = 5150$ psf. Then the required area $A_{req} =$ sum of column loads/$q_e = 750/5.15 = 145.5$ ft^2. The resultant of the column loads is located from the center of the exterior column a distance $450 \times 18/750 = 10.8$ ft. Hence, the length of the footing must be $2(10.8 + 0.75) = 23.1$ ft. A length of 23 ft 3 in. is selected. The required width is then $145.5/23.25 = 6.3$ ft. A width of 6 ft 6 in. is selected (see Fig. 16.15).

Longitudinally, the footing represents an upward-beam spanning between columns and cantilevering beyond the interior column. Since this beam is considerably wider than the columns, the column loads are distributed crosswise by transverse beams, one under each column. In the present relatively narrow and long footing it will be found that the required minimum depth for the transverse beams is smaller than is required for the footing in the longitudinal direction. These "beams," therefore, are not really distinct members but merely represent transverse strips in the main body of the footing, so reinforced that they are capable of resisting the transverse bending moments and the corresponding shears. It then becomes necessary to decide how large the effective width of this transverse beam can be assumed to be. Obviously, the strip directly under the column does not deflect independently and is strengthened by the adjacent parts of the footing. The effective width of the transverse beams is therefore evidently larger than that of the column. In the absence of definite rules for this case, or of research results on which to base such rules, the authors recommend conservatively that the load be assumed to spread outward from the column into the footing at a slope of 2 vertical to 1 horizontal. This means that the effective width of the transverse beam is assumed to be equal to the width of the column plus $d/2$ on either side of the column, d being the effective depth of the footing.

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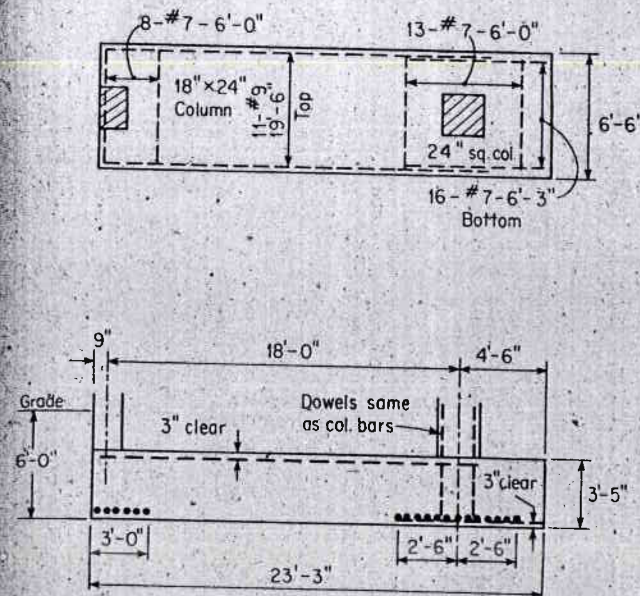

FIGURE 16.15
Combined footing of Example 16.3.

Strength design in longitudinal direction

The net upward pressure caused by the factored column loads is

$$q_u = \frac{1.4(170 + 250) + 1.7(130 + 200)}{23.25 \times 6.5} = 7.60 \text{ kips/ft}^2$$

Then the net upward pressure per linear foot in the longitudinal direction is $7.60 \times 6.5 = 49.4$ kips/ft. The maximum negative moment between the columns occurs at the section of zero shear. Let x be the distance from the outer edge of the exterior column to this section. Then (see Fig. 16.16)

$$V_u = 49{,}400x - 459{,}000 = 0$$

results in $x = 9.3$ ft. The moment at this section is

$$M_u = \left[49{,}400\frac{9.3^2}{2} - 459{,}000(9.30 - 0.75)\right]12 = -21{,}400{,}000 \text{ in-lb}$$

The moment at the right edge of the interior column is

$$M_u = 49{,}400\frac{3.5^2}{2}12 = 3{,}630{,}000 \text{ in-lb}$$

and the details of the moment diagram are as shown in Fig. 16.16. Try $d = 37.5$ in.

From the shear diagram of Fig. 16.16 it is seen that the critical section for flexural shear is at a distance d to the left of the left face of the interior column. At that point the

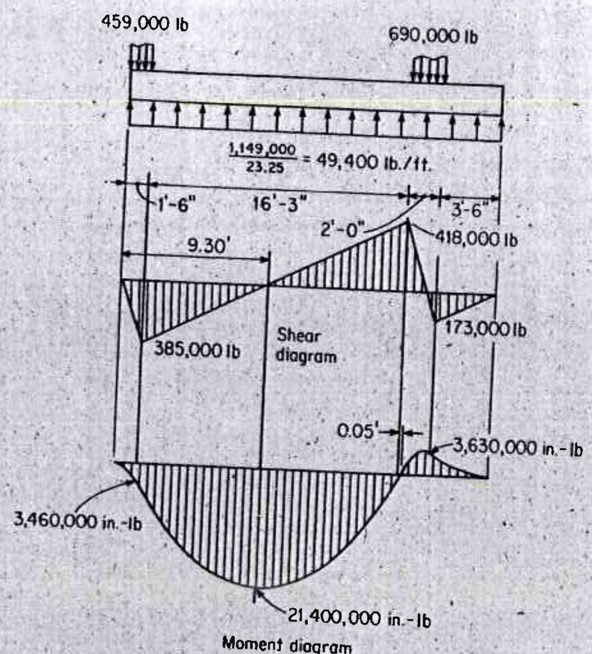

FIGURE 16.16
Moment and shear diagrams for footing of Example 16.3.

required shear strength is

$$V_u = 418,000 - \frac{37.5}{12}49,400 = 264,000 \text{ lb}$$

and the design shear strength

$$\phi V_c = 0.85 \times 2\sqrt{3000} \times 78 \times 37.5 = 272,000 \text{ lb} > V_u$$

indicating that $d = 37.5$ in. is adequate.

Additionally, as in single footings, punching shear should be checked on a perimeter section a distance $d/2$ around the column, on which the nominal shear stress $v_c = 4\sqrt{3000} = 220$ psi. Of the two columns, the exterior one with a three-sided perimeter a distance $d/2$ from the column is more critical in regard to this punching shear. The perimeter is

$$b_o = 2\left(1.5 + \frac{37.5/12}{2}\right) + \left(2.0 + \frac{37.5}{12}\right) = 11.24 \text{ ft}$$

and the shear force, being the column load minus the soil pressure within the perimeter, is

$$V_u = 459,000 - 3.06 \times 5.12(7600) = 340,000 \text{ lb}$$

On the other hand, the design shear strength on the perimeter section is

$$\phi V_c = 0.85 \times 220 \times 11.24 \times 12 \times 37.5 = 946,000 \text{ lb}$$

considerably larger than the required strength V_u.

With $d = 37.5$ in., and with 3.5 in. insulation from the center of the bars to the top surface of the footing, the total thickness is 41 in.

To determine the required steel area, $M_u/\phi bd^2 = 21,400,000/(0.9 \times 78 \times 37.5^2) = 217$ is used to enter Graph A.1*b* of App. A. For this value, the curve 60/3 gives the steel ratio $\rho = 0.0037$. The required steel area is $A_s = 0.0037 \times 37.5 \times 78 = 10.8$ in^2. Eleven No. 9 bars furnish 11.00 in^2. The required development length is found to be 6.7 ft. From Fig. 16.16, the distance from the point of maximum moment to the nearer left end of the bars is seen to be $9.30 - \frac{3}{12} = 9.05$ ft, much larger than the required minimum development length. The selected reinforcement is therefore adequate for both bending and bond.

For the portion of the longitudinal beam that cantilevers beyond the interior column, the minimum required steel area controls. Here

$$A_{s,min} = \frac{3\sqrt{3000}}{60,000} \times 78 \times 37.5 = 8.01 \text{ in}^2$$

but not less than

$$A_{s,min} = \frac{200}{60,000} \times 78 \times 37.5 = 9.75 \text{ in}^2$$

Sixteen No. 7 bars, with $A_s = 9.62$ in^2 are selected; their development length is computed and for bottom bars is found satisfactory.

Design of transverse beam under interior column

The width of the transverse beam under the interior column can now be established as previously suggested and is $24 + 2(d/2) = 24 + 2 \times 18.75 = 61.5$ in. The net upward load per linear foot of the transverse beam is $690,000/6.5 = 106,000$ lb/ft. The moment at the edge of the interior column is

$$M_u = 106,000\frac{2.25^2}{2}12 = 3,220,000 \text{ in-lb}$$

Since the transverse bars are placed on top of the longitudinal bars (see Fig. 16.15), the actual value of d furnished is $37.5 - 1.0 = 36.5$ in. The minimum required steel area controls; i.e.,

$$A_s = \frac{200}{60,000}61.5 \times 36.5 = 7.48 \text{ in}^2$$

Thirteen No. 7 bars are selected and placed within the 61.5 in. effective width of the transverse beam.

Punching shear at the perimeter a distance $d/2$ from the column has been checked before. The critical section for regular, flexural shear, at a distance d from the face of the column, lies beyond the edge of the footing, and therefore no further check on shear is needed.

The design of the transverse beam under the exterior column is the same as the design of that under the interior column, except that the effective width is 36.75 in. The details of the calculations are not shown. It will be easily checked that eight No. 7 bars, placed within the 36.75 in. effective width, satisfy all requirements. Design details are shown in Fig. 16.15.