Homework #10 FOUNDATIONS ENCE 4359

Solutions

(1)

A square spread footing supports an 18-in.-square column supporting a service dead load of 400 kips and a service live load of 270 kips. The column is built of 5000-psi concrete and has eight No. 9 longitudinal bars with $f_y = 60{,}000$ psi. Design a spread footing to be constructed by using 3000-psi concrete and Grade-60 bars. The top of the footing will be covered with 6 in. of fill with a density of 120 lb/ft^3 and a 6-in. basement floor (Fig. 16-13). The basement floor loading is 100 psf. The allowable bearing pressure on the soil is 6000 psf. Use load and resistanee factors from ACI Sections 9.2 and 9.3.

1. **Compute the factored loads and the resistance factors, ϕ.** From ACI Section 9.2, the applicable load combinations are as follows:

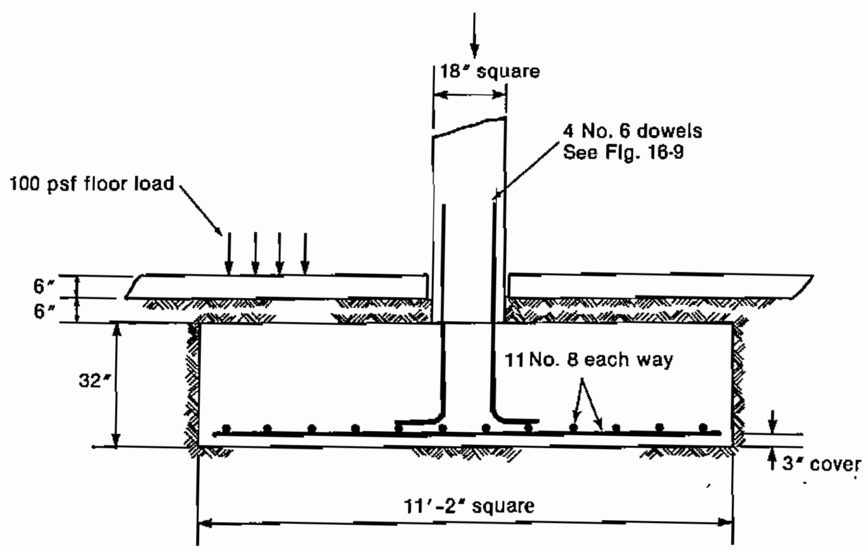
$$U = 1.4(D + F) \qquad \text{(ACI Eq. 9-1)}$$

$$U = 1.2(D + F + T) + 1.6(L + H) + 0.5(L_r \text{ or } S \text{ or } R) \qquad \text{(ACI Eq. 9-2)}$$

$$U = 1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (1.0L \text{ or } 0.8W) \qquad \text{(ACI Eq. 9-3)}$$

$$U = 1.2D + 1.6W + 1.0L + 0.5(L_r \text{ or } S \text{ or } R) \qquad \text{(ACI Eq. 9-4)}$$

$$U = 1.2D + 1.6W + 1.6H \qquad \text{(ACI Eq. 9-6)}$$



Because the statement of the problem mentioned only dead and live loads, we will assume these as the only applicable loads. In effect, we are assuming that the wind loads and roof loads are small compared to the dead loads. This reduces the set of load combinations to the following:

$$U = 1.4(D)$$
$$U = 1.2(D) + 1.6(L)$$

From these equations, the design loads are

$$U = 1.4 \times 400 = 560 \text{ kips}$$

and

$$U = 1.2 \times 400 + 1.6 \times 270 = 912 \text{ kips}$$

Strength-reduction factors ϕ are given in ACI Section 9.3. These were selected on the basis of whether flexure or shear is being considered. For shear, ACI Section 9.3.2.3 gives $\phi = 0.75$. For flexure, ϕ will be a function of the strain in the extreme-tension layer of bars, but ϕ will probably be 0.9 for footings.

2. Estimate the footing size and the factored net soil pressure. Allowable net soil pressure $q_n = 6$ ksf $-$ (weight/ft^2 of the footing and the soil and floor over the footing and the floor loading). Estimate the overall thickness of the footing at between one and two times the width of the column, say, 27 in.:

$$q_n = 6.0 - \left(\frac{27}{12} \times 0.15 + 0.5 \times 0.12 + 0.5 \times 0.15 + 0.100\right)$$
$$= 5.43 \text{ ksf}$$
$$\text{Area required} = \frac{400 \text{ kips} + 270 \text{ kips}}{5.43 \text{ ksf}} = 123 \text{ ft}^2$$
$$\approx 11.1 \text{ ft square.}$$

Try a footing 11 ft 2 in. square by 27 in. thick:

$$\text{Factored net soil pressure} = \frac{1.2 \times 400 + 1.6 \times 270}{11.17^2}$$
$$= 7.31 \text{ ksf}$$

3. Check the thickness for two-way shear. Generally, the thickness of a spread footing is governed by two-way shear. The shear will be checked on the critical perimeter at $d/2$ from the face of the column and, if necessary, the thickness will be increased or decreased. Since there is reinforcement in both directions, the average d will be used:

$$\text{Average } d = 27 \text{ in.} - (3 \text{ in. cover}) - (1 \text{ bar diameter})$$
$$= 23 \text{ in.}$$

The critical shear perimeter (ACI Section 11.12.1.2) is shown dashed in Fig. 16-14a. The tributary area for two-way shear is shown crosshatched. We have

$$V_u = 7.31 \text{ ksf}\left[11.17^2 - \left(\frac{41}{12}\right)^2\right] \text{ft}^2 = 827 \text{ kips}$$

Length of critical shear perimeter:

$$b_o = 4(18 + 23) \text{ in.} = 164 \text{ in.}$$

ϕV_c is the smallest of (a), (b), and (c) following:

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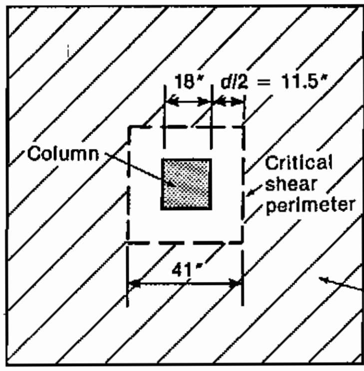
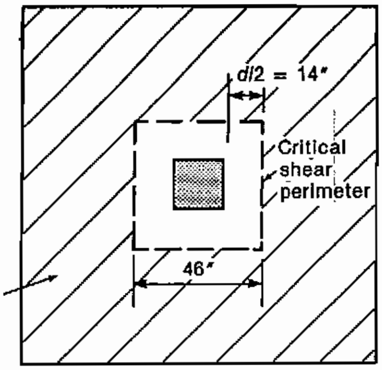
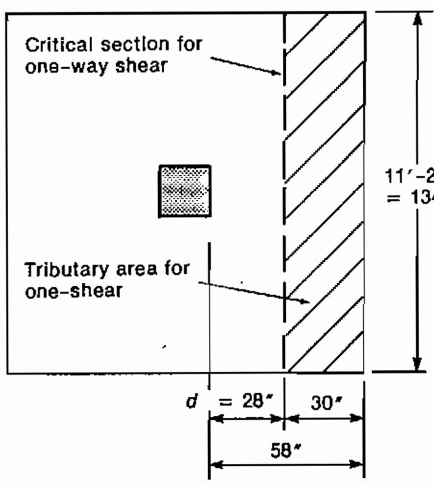


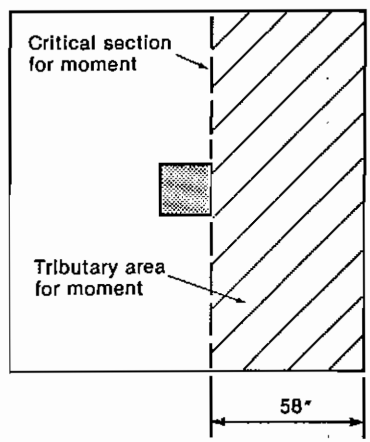
(a) Critical section for two-way shear—First trial.



(b) Critical section for two-way shear—Final trial



(c) Critical section for one-way shear



(d) Critical section for moment

(a) $$\phi V_c = \phi\left(2 + \frac{4}{\beta_c}\right)\sqrt{f'_c}b_o d$$ (13-15) (ACI Eq. 11-33)

$$\beta_c = \frac{\text{long side of column}}{\text{short side of column}} = 1.0$$

$$\phi V_c = \frac{0.75(2 + 4/1)\sqrt{3000} \times 164 \times 23}{1000} = 930 \text{ kips}$$

(b) $$\phi V_c = \phi\left(\frac{\alpha_s d}{b_o} + 2\right)\sqrt{f'_c}b_o d$$ (13-16) (ACI Eq. 11-34)

where α_s = 40 for interior columns, 30 for edge columns, and 20 for corner columns. Thus,

$$\phi V_c = 0.75\left(\frac{40 \times 23}{164} + 2\right)\frac{\sqrt{3000} \times 164 \times 23}{1000} = 1179 \text{ kips}$$

(c) $$\phi V_c = \phi(4)\sqrt{f'_c}b_o d = 620 \text{ kips}$$ (13-17) (ACI Eq. 11-35)

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Because $\phi V_c = 620$ kips is less than $V_u = 827$ kips, the footing is not thick enough. Try $h = 32$ in., $d = 28$ in., and $b_o = 184$ in. The footing is thicker, and it weighs more. Hence, a larger area may be required:

$$q_n = 6.0 - \left(\frac{32}{12} \times 0.15 + 0.5 \times 0.12 + 0.5 \times 0.15 + 0.100\right)$$
$$= 5.37 \text{ ksf}$$
$$\text{Area required} = \frac{400 + 270}{5.37}$$
$$= 124.8 \text{ ft}^2$$
$$= 11.17 \text{ ft square}$$

Try an 11-ft-2-in.-square footing, 32 in. thick:

$$\text{Factored net soil pressure, } q_{nu} = \frac{1.2 \times 400 + 1.6 \times 270}{11.17^2}$$
$$= 7.31 \text{ ksf}$$
$$\text{Average } d = 32 - 3 - 1$$
$$= 28 \text{ in.}$$

The new critical shear perimeter and tributary area for shear are shown in Fig. 16-14b. We have

$$V_u = 7.31\left[11.17^2 - \left(\frac{46}{12}\right)^2\right] = 804 \text{ kips}$$

ACI Eq. (11-35) governs again:

$$\phi V_c = 0.75 \times 4\sqrt{3000} \times (184) \times \frac{28}{1000} = 846 \text{ kips}$$

This is adequate. A check using $h = 30$ in. shows that a 30-in.-thick footing is not adequate. Use an **11-ft-2-in.-square footing, 32 in. thick.**

4. **Check the one-way shear.** Although one-way shear is seldom critical, we shall check it. The critical section for one-way shear is located at d away from the face of the column (ACI Section 11.12.1.1), as shown in Fig. 16-14c. Thus,

$$V_u = 7.31 \text{ ksf}\left(11.17 \text{ ft} \times \frac{30}{12}\text{ft}\right) = 204 \text{ kips}$$
$$\phi V_c = \phi 2\sqrt{f'_c}b_w d = 0.75 \times 2\sqrt{3000} \times 134 \times \frac{28}{1000}$$
$$= 308 \text{ kips}$$

Therefore, **OK in one-way shear.**

5. **Design the flexural reinforcement.** The critical section for moment and anchorage of the reinforcement is shown in Fig. 16-14d. The ultimate moment is

$$M_u = 7.31\left[11.17 \times \frac{(58/12)^2}{2}\right] = 954 \text{ ft-kips}$$

Assuming that $j = 0.9$ and $\phi = 0.90$, the area of steel required is

$$A_s = \frac{954 \times 12{,}000}{0.9 \times 60{,}000(0.9 \times 28)} = 8.41 \text{ in.}^2$$

The average value of d was used in this calculation for simplicity. The same reinforcement will be used in both directions:

$$\text{Minimum } A_s \text{ (ACI Sections 10.5.3 and 7.12.2)} = 0.0018bh$$

$$= 0.0018 \times 134 \times 32 = 7.72 \text{ in.}^2 \text{ (does not govern)}$$

Maximum spacing (ACI Section 7.6.5) = 18 in.

Try eleven No. 8 bars each way, $A_s = 8.69$ in.^{2}. Recompute ϕM_n as a check.

$$a = \frac{8.69 \times 60{,}000}{0.85 \times 3000 \times 134} = 1.53 \text{ in.}$$

because $a/d = 1.53$ in./28 in. = 0.055 is much less than a/d for the tension-controlled limit from Table A-4, the beam is tension-controlled, and $\phi = 0.90$.

$$\phi M_n = 1070 \text{ ft} - \text{kips}$$

This exceeds $M_u = 954$ ft-kip.

6. Check the development. The clear spacing of the bars being developed exceeds $2d_b$ and the clear cover exceeds d_b. Therefore, this is case 2 development in Tables 8-1 and A-11. From Table A-11, ℓ_d for a No. 8 bottom bar in 3000-psi concrete is 54.8 d_b. The development length is

$$\ell_d = 54.8\, d_b \beta \lambda$$

where $\beta = 1.0$ for uncoated reinforcement and $\lambda = 1.0$ for normal-weight concrete. Accordingly, we have

$$\ell_d = 54.8 \times 1.00 \times 1.0 \times 1.0$$
$$= 54.8 \text{ in.}$$

The bar extension past the point of maximum moment is (58 in. − 3 in.) = 55 in. This is OK. **Use eleven No. 8 bars each way; $A_s = 8.69$ in.^{2}.**

7. Design the column–footing joint. The column–footing joint is shown in Fig. 16-13. The factored load at the base of the column is

$$1.2 \times 400 + 1.6 \times 270 = 912 \text{ kips}$$

The maximum bearing load on the bottom of the column (ACI Section 10.17.1) is $\phi 0.85 f'_c A_1$, where A_1 is the area of the contact surface between the column and the footing and f'_c is for the column. When the the contact supporting surface on the footing is wider on all sides than the loaded area, the maximum bearing load on the top of the footing may be taken as

$$0.85 \phi f'_c A_1 \sqrt{\frac{A_2}{A_1}}, \text{ but not more than } 1.7 \phi f'_c A_1 \qquad (16\text{-}15)$$

where A_2 is the area of the lower base of a right pyramid or cone as defined in Fig. 16-10. ACI Section 9.3.2.4 defines ϕ equal to 0.65 for bearing. By inspection, $\sqrt{A_2/A_1}$ for the footing exceeds 2; hence, the maximum bearing load *on the footing* is $0.85 \times 0.65 \times 3 \times 18^2 \times 2 = 1074$ kips. The allowable bearing on the base of the column is

$$\phi(0.85 f'_c A_1) = 0.65 \times 0.85 \times 5 \times 18^2$$
$$= 895 \text{ kips}$$

Thus, the maximum load that can be transferred by bearing is 895 kips, and dowels are needed to transfer the excess load. Accordingly, we have

$$\text{Area of dowels required} = \frac{912 - 895}{\phi f_y} = 0.44 \text{ in.}^2$$

where $\phi = 0.65$ has been used. This is the ϕ value from ACI Sections 9.3.2.2(b) for compression-controlled tied columns and 9.3.2.4 for bearing. The area of dowels must also satisfy ACI Section 15.8.2.1:

$$\text{Area of dowels} \geq 0.005 A_g = 1.62 \text{ in.}^2$$

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$$= 0.0018 \times 134 \times 32 = 7.72 \text{ in.}^2 \text{ (does not govern)}$$

Maximum spaeing (ACI Section 7.6.5) = 18 in.

Try eleven No. 8 bars each way, A_s = 8.69 in.². Recompute ϕM_n as a eheek.

$$a = \frac{8.69 \times 60{,}000}{0.85 \times 3000 \times 134} = 1.53 \text{ in.}$$

because a/d = 1.53 in./ 28 in. = 0.055 is much less than a/d for the tension-controlled limit from Table A-4, the beam is tension-controlled, and $\phi = 0.90$.

$$\phi M_n = 1070 \text{ ft} - \text{kips}$$

This exceeds M_u = 954 ft-kip.

6. Check the development. The elear spacing of the bars being developed exceeds $2d_b$ and the elear cover exceeds d_b. Therefore, this is ease 2 development in Tables 8-1 and A-11. From Table A-11, ℓ_d for a No. 8 bottom bar in 3000-psi conerete is 54.8 d_b. The development length is

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where $\beta = 1.0$ for uncoated reinforeement and $\lambda = 1.0$ for normal-weight concrete. Accordingly, we have

$$\ell_d = 54.8 \times 1.00 \times 1.0 \times 1.0$$
$$= 54.8 \text{ in.}$$

The bar extension past the point of maximum moment is (58 in. − 3 in.) = 55 in. This is OK. Use **eleven No. 8 bars each way; A_s = 8.69 in.².**

7. Design the column–footing joint. The eolumn–footing joint is shown in Fig. 16-13. The factored load at the base of the column is

$$1.2 \times 400 + 1.6 \times 270 = 912 \text{ kips}$$

The maximum bearing load on the bottom of the eolumn (ACI Section 10.17.1) is $\phi 0.85 f'_c A_1$, where A_1 is the area of the contact surface between the column and the footing and f'_c is for the eolumn. When the the contact supporting surface on the footing is wider on all sides than the loaded area, the maximum bearing load on the top of the footing may be taken as

$$0.85\phi f'_c A_1 \sqrt{\frac{A_2}{A_1}}, \text{ but not more than } 1.7\phi f'_c A_1 \qquad (16\text{-}15)$$

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$$\phi(0.85 f'_c A_1) = 0.65 \times 0.85 \times 5 \times 18^2$$
$$= 895 \text{ kips}$$

Thus, the maximum load that can be transferred by bearing is 895 kips, and dowels are needed to transfer the excess load. Accordingly, we have

$$\text{Area of dowels required} = \frac{912 - 895}{\phi f_y} = 0.44 \text{ in.}^2$$

where $\phi = 0.65$ has been used. This is the ϕ value from ACI Sections 9.3.2.2(b) for compression-eon-trolled tied columns and 9.3.2.4 for bearing. The area of dowels must also satisfy ACI Section 15.8.2.1:

$$\text{Area of dowels} \geq 0.005 A_g = 1.62 \text{ in.}^2$$

Try four No. 6 dowels (A_s = 1.76 in²); dowel eaeh eorner bar. The dowels must extend into the footing by the compression-development length for a No. 6 bar in 3000-psi concrete, or 16 in. The bars will be extended down to the level of the main footing steel and hooked 90°. The hooks will be tied to the main steel to hold the dowels in plaee. The dowels must extend into the column a distance equal to the greater of a compression splice for the dowels (23 in.) or the compression-development length of the column bars (25 in.). **Use four No. 6 dowels; dowel each corner bar. Extend dowels 25 in. into column.** (See Fig. 16-13.) ∎

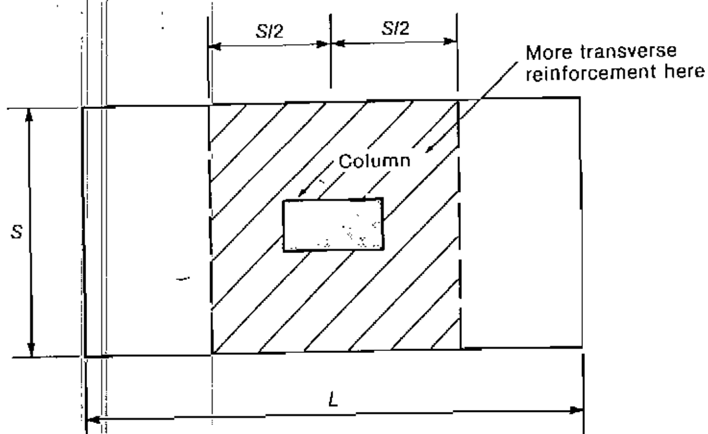
(2)

1. **Check the one-way shear.** One-way shear may be critical in a rectangular footing and must be checked. The critical section and tributary area for one-way shear are shown in Fig. 16-16a. We have

$$V_u = 7.31\left(\frac{45}{12} \times 9\right) = 247 \text{ kips}$$

$$\phi V_c = 0.75 \times 2\sqrt{3000} \times 108 \times \frac{28}{1000} = 248 \text{ kips}$$

This is just OK in one-way shear.



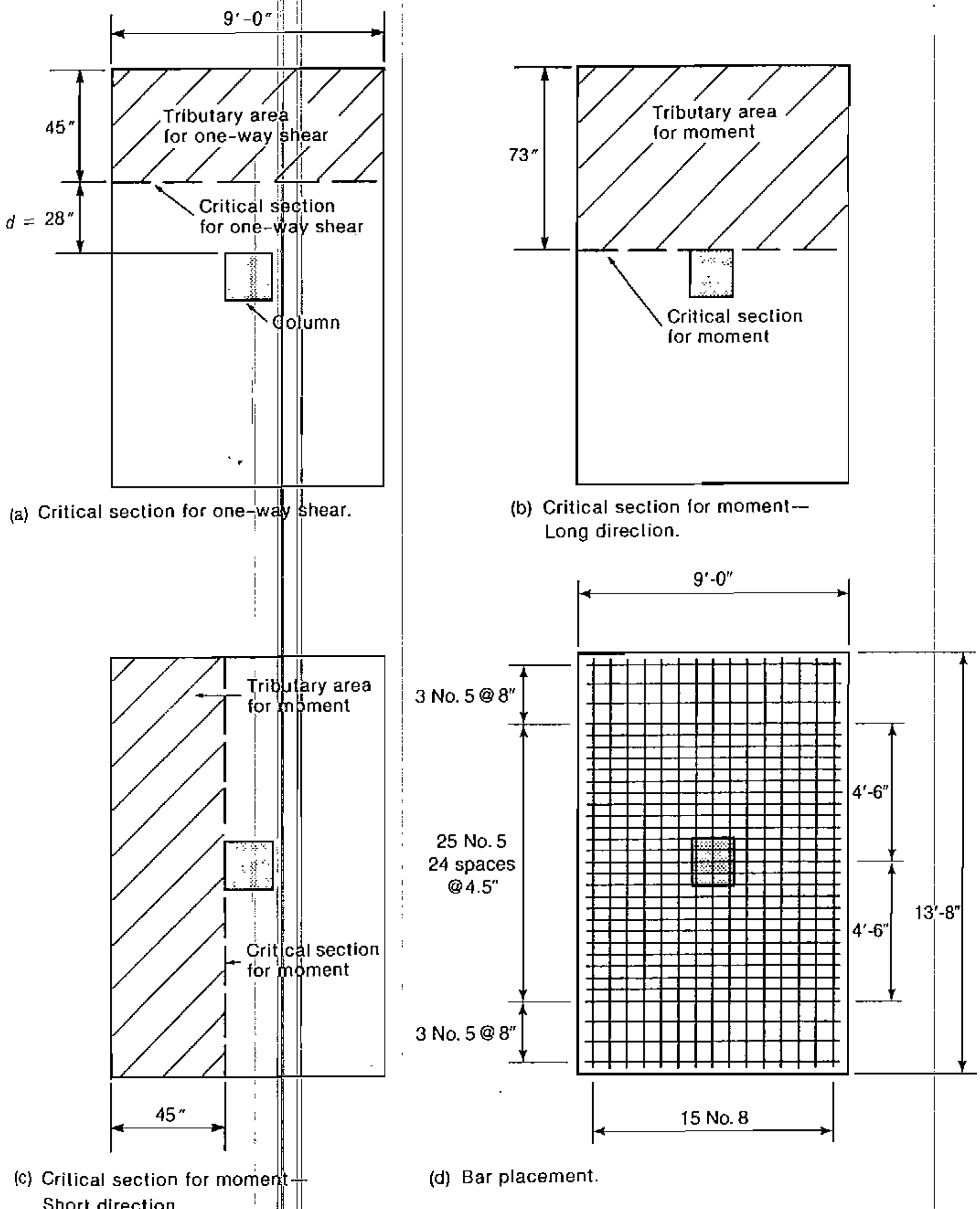


Fig. 16-16
Rectangular footing—Example 16-3.

2. Design the reinforcement in the long direction. The critical section for moment and reinforcement anchorage is shown in Fig. 16-16b. The ultimate moment is

$$M_u = 7.31\left(9 \times \frac{(73/12)^2}{2}\right) = 1217 \text{ ft-kips}$$

Assuming that $\phi = 0.9$ and $j = 0.9$, the area of steel required is

$$A_s = \frac{1217 \times 12{,}000}{0.9 \times 60{,}000(0.9 \times 28)} = 10.73 \text{ in.}^2$$

$$A_{s(\min)} = 0.0018 \times 108 \times 32 = 6.22 \text{ in.}^2 \text{ (does not govern)}$$

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We could use

14 No. 8 bars, $A_s = 11.1$ in.^{2}
11 No. 9 bars, $A_s = 11.0$ in.^{2}
9 No. 10 bars, $A_s = 11.4$ in.^{2}

Try 14 No. 8 bars—$\phi M_n = 1330$ ft-kips. Check development:

$$\ell_d = 54.8 \times 1.0 \times 1.0 \times 1.0 = 54.8 \text{ in.}$$

The length available is 70 in.—therefore, OK. **Use 14 No. 8 bars in the long direction.**

3. Design the reinforcement in the short direction. The critical section for moment and reinforcement anchorage is shown in Fig. 16-16c. We have

$$M_u = 7.31\left(13.67 \times \frac{(45/12)^2}{2}\right) = 702 \text{ ft-kips}$$

Assuming that $j = 0.9$, $A_s = 6.19$ in.^{2}.

$$A_{s(\min)} = 0.0018 \times 164 \times 32 = 9.45 \text{ in.}^2 \text{ (This governs.)}$$

Try 12 No. 8 bars: $A_s = 9.48$ in.^{2}.

Check development. $\ell_d = 54.8$ in. and the length available is 28.5 in.—therefore, not OK. We must consider smaller bars. Try 31 No. 5 bars, $A_s = 9.61$ in.^{2}, $\ell_d = 27.4$ in.—therefore OK. Use 31 No. 5 bars in the short direction of the footing. For the arrangement of the bars in the transverse direction (ACI Section 15.4.4.2),

$$\beta = \frac{\text{long side}}{\text{short side}} = 1.519$$

In the middle strip of width 9 ft, provide

$$\frac{2}{1.519 + 1} \times 31 \text{ bars} = 24.6 \text{ bars}$$

Provide 25 No. 5 bars in the middle strip, and provide three No. 5 bars in each end strip. The final design is shown in Fig. 16-16d. ■

A 12-in.-thick concrete wall carries a service (unfactored) dead load of 10 kips per foot and a service live load of 12.5 kips per foot. The allowable soil pressure, q_a, is 5000 psf at the level of the base of the footing, which is 5 ft below the final ground surface. Design a wall footing, using $f'_c = 3000$ psi and $f_y = 60{,}000$ psi. The density of the soil is 120 lb/ft^3. Frequently, the strength of the concrete in the footing is lower than that in the column. Dowels are used to accommodate this change in strength. Most strip footings on soil have one mat of reinforcement.

1. **Estimate the size of the footing and the factored net pressure.** Consider a 1-ft strip of footing and wall. Allowable soil pressure is 5 ksf; allowable net soil pressure is 5 ksf − weight/ft^2 of the footing and of the soil over the footing. Since the thickness of the footing is not known at this stage, it is necessary to guess a thickness for a first trial. Generally, the thickness will be 1 to 1.5 times the wall thickness. We shall try a 12-in.-thick footing. Therefore, $q_n = 5 - (1 \times 0.15 + 4 \times 0.12) = 4.37$ ksf, and we have:

$$\text{Area required} = \frac{10 \text{ kips} + 12.5 \text{ kips}}{4.37 \text{ ksf}}$$
$$= 5.15 \text{ ft}^2 \text{ per foot of length of wall}$$

Try a footing 5 ft 2 in. = 62 in. wide.

Using the load factors in ACI Section 9.2.1:

$$\text{Factored net pressure, } q_{nu} = \frac{1.2 \times 10 + 1.6 \times 12.5}{5.167} = 6.19 \text{ ksf}$$

In the design of the concrete and reinforcement, we shall use $q_{nu} = 6.19$ ksf.

2. **Check the shear.** Shear usually governs the thickness of footings. Only one-way shear is significant in a wall footing. Check it at d away from the face of the wall (section B–B in Fig. 16-11)

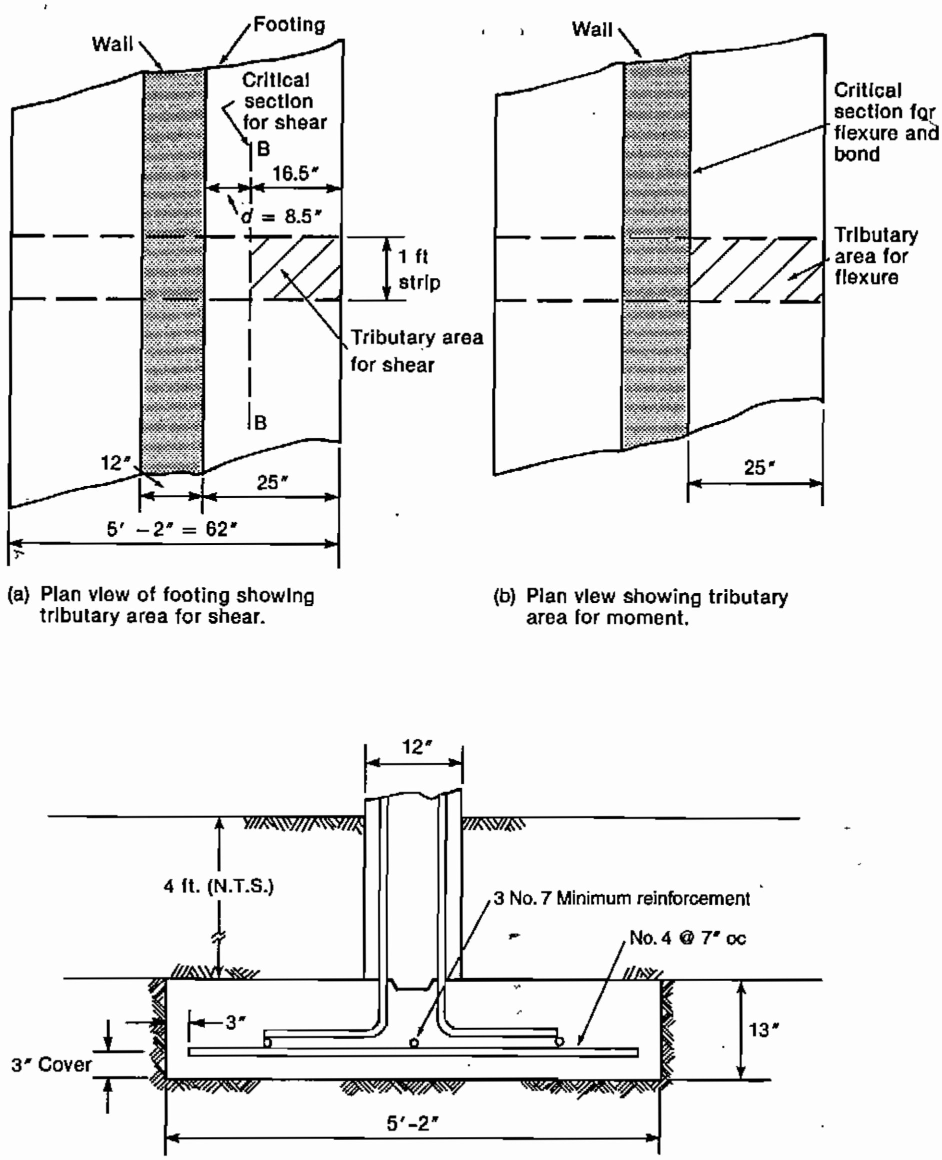
$$d = 12 \text{ in.} - 3 \text{ in cover} - \tfrac{1}{2} \text{ bar diameter} \simeq 8.5 \text{ in.}$$

The tributary area for shear is shown crosshatched in Fig. 16-12a.

$$V_u = 6.19\left(\frac{16.5}{12} \times 1\right) \text{ft}^2 = 8.51 \text{ kips/ft}$$

$$\phi V_c = \phi \times 2\sqrt{f'_c}\, b_w d = 0.75 \times 2 \times \sqrt{3000} \times 12 \times 8.5/1000$$
$$= 8.38 \text{ kips/ft}$$

where, from ACI Section 9.3.2.3, $\phi = 0.75$ for shear design when the load factors from ACI Section 9.2.1 are used.



(a) Plan view of footing showing tributary area for shear.

(b) Plan view showing tributary area for moment.

(c) Reinforcement details.

Since $V_u > \phi V_c$, the footing depth is too small. If V_u is larger or considerably smaller than ϕV_c, choose a new thickness and repeat steps 1 and 2. **Try a 13-in.-thick footing 5 ft 2 in. wide.** A 13-in.-thick footing has $d = 9.50$ in. and $\phi V_c = 9.37$ kips/ft. Because ϕV_c exceeds $V_u = 8.51$ kips/ft, **a 13-in.-thick footing 5 ft 2 in. wide is adequate for shear.** Recompute d:

$$d = 13 \text{ in.} - 3 \text{ in. cover} - \frac{1}{2} \text{bar diameter} \simeq 9.5 \text{ in.}$$

3. Design the reinforcement. The critical section for moment is at the face of the wall (section A–A in Fig. 16-11). The tributary area for moment is shown crosshatched in Fig. 16-12b.

The required moment is

$$M_u = 6.19 \times \frac{(25/12)^2}{2} \times 1 \text{ ft-kips/ft} = 13.4 \text{ ft-kips/ft of length}$$

$$M_u = \phi M_n = \phi A_s f_y jd$$

Footings are generally very lightly reinforced. Therefore, assume that $j = 0.925$. Therefore,

$$A_s = \frac{13.4 \times 12{,}000}{0.9 \times 60{,}000(0.925 \times 9.5)} = 0.339 \text{ in.}^2/\text{ft}$$

From ACI Secs. 10.5.4 and 7.12.2

$$\begin{aligned} \text{Minimum } A_s &= 0.0018bh \\ &= 0.0018 \times 12 \times 13 = 0.281 \text{ in}^2/\text{ft} \end{aligned}$$

Maximum spacing of bars = $2h$, or 18 in.
We could use:

No. 5 bars at 11 in. o.c., $A_s = 0.34 \text{ in.}^2/\text{ft}$
No. 4 bars at 7 in. o.c., $A_s = 0.34 \text{ in.}^2/\text{ft}$

Try No. 4 bars at 7 in. o.c., $A_s = 0.34 \text{ in.}^2/\text{ft}$

Because the calculation of A_s was based on assumptions, recompute the moment capacity:

$$a = \frac{0.34 \times 60{,}000}{0.85 \times 3000 \times 12} = 0.667 \text{ in.}$$

Since $a/d = 0.667/9.5 = 0.070$ is much less than the $a/d = 0.319$ for the tension-controlled limit from Table A-4, the section is tension-controlled, and $\phi = 0.9$.

$$\phi M_n = \frac{0.9 \times 0.34 \times 60{,}000(9.5 - 0.667/2)}{12{,}000} = 14.0 \text{ ft-kips/ft}$$

Because this exceeds $M_n = 13.4$ ft-kips/ft, the moment capacity is OK.
For completeness, we could compute ε_t directly and use it to check ϕ. From similar triangles

$$\frac{c}{0.003} = \frac{d_t - c}{\varepsilon_t}$$

and

$$\varepsilon_t = \frac{(d_t - c)}{c} \times 0.003$$

where $d_t = 9.5$ in. and $c = a/\beta_1 = 0.667/0.85 = 0.785$ in.
Calculations show $e_t = 0.333$.
This far exceeds ε_t limit = 0.005. Therefore, $f_s = f_y$ and $\phi = 0.90$.

4. **Check the development.** The clear spacing of the bars being developed exceeds $2d_b$ and the clear cover exceeds d_b. Therefore, this is case 2 development in Tables 8–1 and A–11. From Table A–11, ℓ_d for a No. 4 bottom bar in 3000-psi concrete is 21.9 in.

The distance from the point of maximum bar stress (at the face of the wall) to the end of the bar is 25 in. − 3 in. cover on the ends of the bars = 22 in. This is more than $\ell_d = 21.9$. **Use No. 4 bars at 7 in. on centers.** This is still case 2 development, and 22 in. just satisfies ℓ_d.

5. **Select the minimum (temperature) reinforcement.** By ACI Section 7.12.2 we require that

$$\begin{aligned} A_s &= 0.0018bh = 0.0018 \times 62 \times 13 \text{ in.} \\ &= 1.45 \text{ in.}^2 \end{aligned}$$

The maximum spacing is $5 \times 12 = 60$ in. or 18 in. **Provide three No. 7 bars for shrinkage reinforcement.**

6. **Design the connection between the wall and the footing.** ACI Section 15.8.2.2 requires that reinforcement equivalent to the minimum vertical wall reinforcement extend from the wall into the footing. A cross section through the wall footing designed in this example is shown in Fig. 16-12e.

The section through the strip footing shows a shear key in the top surface of the footing. This is formed by a two-by-four pushed down into the surface. The shear key is intended to resist some shear to prevent the wall from being dislocated laterally. Sometimes the shear key is omitted, and shear friction is used to resist dislocation of the wall during construction. ■