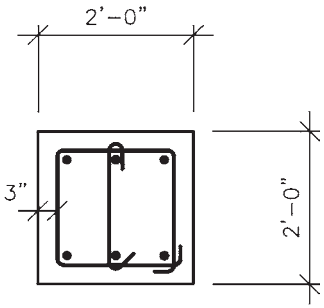

FIGURE 12.18 Column pier for Example 12.1.

$$\rho = \frac{1.80}{20.06 \times 24} = 0.00374$$

$$a_u = 4.35$$

$$\phi M_n = 1.80 \times 4.35 \times 20.06 = 157 \text{ kip-ft} > 102$$

Check shear:

$$V_u = 30 \times 1.7 = 51 \text{ (kip)}$$

$$\phi V_n = 0.107 \times 20.06 \times 24 = 5.15 \text{ kip} > V_u \qquad \text{OK}$$

provide #4 ties at code-mandated maximum spacing, $d/2$, or 10 in o.c.

Design column footing Case 1 governs the design. The pressures acting on each footing are shown in Fig. 12.19.

As determined above,

$$f_{p,\max} = 3.02 \text{ ksf}$$

$$f_{p,\min} = 0.06 \text{ ksf}$$

The pressure at the right face of the pier is 1.38 ksf (by interpolation).

$$M_{\max} = \frac{1.38 \times 5^2}{2} + \frac{(3.02 - 1.38) \times 5 \times 5 \times 2}{2 \times 3} = 30.95 \text{ kip-ft}$$

$$M_u = 30.95 \times 1.7 = 52.62 \text{ kip-ft/ft}$$

For a 24-in-thick footing with #7 bars and 3-in cover,

$$d = 24 - 3 - {}^7\!/_{16} = 20.56 \text{ in}$$

Try six #7 bars in a 48-in-wide footing:

$$\rho = \frac{3.61}{20.56 \times 48} = 0.0036$$

$$a_u = 4.35$$

$$\phi M_n = 3.61 \times 4.35 \times 20.56 = 322.9 \text{ kip-ft}$$

$$M_u = 52.62 \times 4 = 210.44 \text{ kip-ft} < \phi M_n \qquad \text{OK}$$

For bars in the short direction the minimum reinforcement is

$$A_{\min} = 0.0018 \times 24 \times 9 \times 12 = 4.66 \text{ (in}^2\text{)}$$

Use 11 #6 bars ($A_o = 4.84$ in^2).

Check shear (conservatively take at face of support):

$$V_u = \left(1.38 \times 5 + \frac{1.64 \times 5}{2}\right) 1.7 = 18.7 \text{ (kip per ft of width)}$$

$$\phi V_C = 0.107 \times 20.56 \times 12 = 26.4 > V_u \qquad \text{OK}$$

Finally, check the negative bending of the footing under wind uplift, when the footing has to resist its own weight and that of the soil on top, essentially being suspended in the air.

The downward ultimate load is

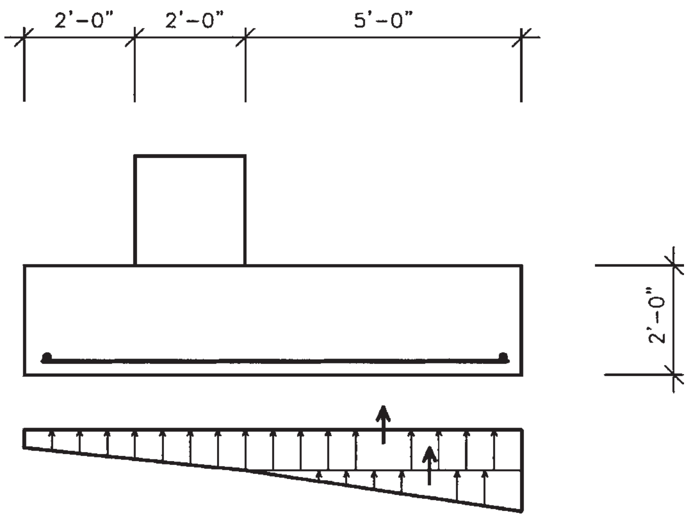

FIGURE 12.19 Soil pressures under column footing in Case 1 of Example 12.1.

$$W_n = 1.4\,(1.5 \times 0.12 + 2.0 \times 0.15) = 0.672 \text{ kip/ft}$$

$$l = 5 \text{ ft}$$

$$M_u = 0.672 \times 5^2/2 = 8.4 \text{ kip-ft} = 100{,}800 \text{ lb-in per ft of width}$$

Design the footing in negative pressure as plain concrete, following ACI 318,[9] Chap. 22.

Taking the effective thickness as 2 in smaller than actual,

$$S = \frac{12 \times 22^2}{6} = 968 \text{ (in}^3\text{)}$$

$$f_{t,u} = \frac{100{,}800}{968} = 104 \text{ psi} \qquad \phi = 0.55 \text{ for plain concrete in ACI 318-02}$$

$$F_{t,u} = 0.55 \times 5 \times \sqrt{4000} = 174 \text{ psi}$$

$$f_{t,u} < F_{t,u} \qquad \text{OK}$$

Check the beam-type shear:

$$V_u = 0.675\left(5 - \frac{22}{12}\right) = 2.14 \text{ kip/ft}$$

$$\phi V_n = 0.55 \times \frac{4}{3}\sqrt{4000} \times \frac{12 \times 22}{1000} = 12.24 \text{ kip/ft} > V_u \qquad \text{OK}$$

If the governing load combinations include a case of lateral column reactions acting toward the building, the foundation should be checked for that case as well.

The final design of the column foundation is shown in Fig. 12.20. The section through the exterior foundation wall is shown in Fig. 12.21.