$$K_a \text{ (active pressure coefficient)} = 0.33$$
$$K_p \text{ (passive pressure coefficient)} = 3.00$$
$$\mu \text{ (sliding friction coefficient)} = 0.55$$

Then, for a 4-ft-wide strip of soil,

$$F_R = 55.6 \times 0.55 + {}^1\!/_2\,(3.00 - 0.33) \times 0.12 \times 3.5^2 \times 4(\text{ft}) = 34.9 \text{ kip} > 30 \text{ kip}$$

However, the safety factor of 34.9/30 = 1.16 is insufficient. At this point, we can introduce a shear key cut into the soil (see Fig. 12.11) or rely on passive pressure of foundation walls spanning horizontally between the column foundations.

Design of shear keys is explained in many references, such as Ref. 7, so the second approach is illustrated.

Find the required length of wall that, acting as a horizontal cantilever, engages enough soil to provide a factor of safety against sliding equal to 1.5. The total required resistance is

$$F_{\text{req'd}} = 30 \times 1.5 = 45 \text{ kip}$$

The amount of sliding resistance added by one linear-foot of wall is

$$f_w = {}^1\!/_2\,(3.00 - 0.33) \times 0.12 \times 3.5^2 = 1.96 \text{ kip/ft}$$

The required length of wall on each side of a pier is

$$L_{w,\text{rq}} = \frac{(45 - 34.9)}{2 \times 1.96} = 2.58 \text{ (ft)}$$

Check the horizontal bending of a 12-in wall, 3.5 ft deep, with at least three #4 horizontal bars placed in interior layers behind vertical #4 bars:

$$w_u = 1.96 \times 1.7 = 3.33 \text{ kip/ft}$$

$$M_u = \frac{3.33 \times 2.58^2}{2} = 11.05 \text{ kip-ft}$$

$$d = 12 - 2 - {}^1\!/_2 - {}^1\!/_4 = 9.25 \text{ (in)}$$

$$A = 0.58 \text{ in}^2$$

$$\rho = \frac{0.58}{9.25 \times 3.5 \times 12} = 0.0015 \rightarrow a_u = 4.44$$

$$\phi M_n = 0.58 \times 9.25 \times 4.44 = 23.82 \text{ kip-ft} > M_u \qquad \text{OK}$$

Case 2: Dead + wind uplift. Check the foundation tentatively selected for Case 1 for wind uplift and dead load. The forces acting on the foundation (see Fig. 12.17) are: $U = 14$ kip, $H = 11$ kip, $\Sigma W = 18.6$ kip, as computed for Case 1 (55.6 − 37 = 18.6 kip). Also from Case 1, subtracting the restoring moment from the column reaction:

$$\Sigma M_R = 290.38 - 203.5 = 86.88 \text{ (kip-ft)}$$

$$\bar{x}_{\text{c.g.}} \text{ from right edge} = \frac{86.88}{18.6} = 4.67 \text{ (ft), or}$$

$$0.17 \text{ ft to left of footing centerline } (l = 4.33 \text{ ft})$$

Taking overturning and restoring moments about point *B:*

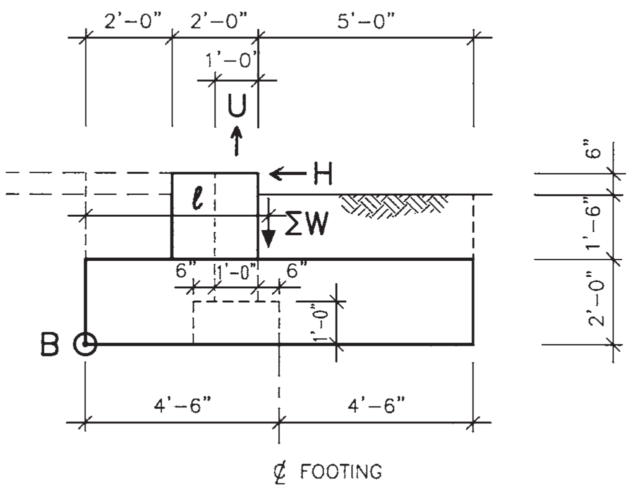

FIGURE 12.17 Forces acting on foundation for Case 2 of Example 12.1.

$$M_{OT} = 11 \times 4 + 14 \times 3.5 = 93 \text{ (kip-ft)}$$

$$M_R = 18.6 \times 4.33 = 80.5 \text{ (kip-ft)} < M_{OT} \qquad \text{N.G.}$$

Again, we must rely on help from the foundation walls shown dotted in Fig. 12.17. The walls are 12 in deep with 2-ft-wide footings, also 12-in deep. The weight of walls, wall footings, and soil above the footing ledges is

$$W_{wall} = [(3' \times 1 + 2' \times 1)\,0.15 + (0.5 \times 2.5 + 0.5 \times 3)\,0.12]\,(25 - 4) = 22.68 \text{ (kip)}$$

Then

$$\Sigma W = 18.6 + 22.68 = 41.28 \text{ (kip)}$$

$$M_R = 18.6 \times 4.33 + 22.68 \times 3.5 = 159.9 \text{ (kip-ft)} > 93 \text{ (kip-ft)} \qquad \text{OK}$$

$$\text{F.S.} = \frac{159.9}{93} = 1.72 > 1.5 \qquad \text{OK}$$

Sliding is OK by observation.

Design concrete pier The maximum service-load moment on each pier is

$$M_{max} = 30 \text{ kip} \times 2 \text{ ft} = 60 \text{ kip-ft}$$

Since we do not know what percentage of this is dead load, conservatively use an overall load factor of 1.7:

$$M_u = 60 \times 1.7 = 102 \text{ kip-ft}$$

Try a 2-ft × 2-ft pier (Fig. 12.18) with three #7 bars each face, #4 ties and 3 in clear cover:

$$d = 24 - 3 - {}^1\!/_2 - {}^7\!/_{16} = 20.06 \text{ in}$$

$$A_S = 1.80 \text{ in}^2$$