The pier sizes (2 ft × 2 ft) were chosen to provide a generous allowance for column sizes, to facilitate development of anchor bolts, and to provide a 2-in space for the slab seat on the pier. Despite the ample size, a note was added to the drawings that limited the column base plate size to 12 in × 12 in, also quite adequate for the building span and load. To maximize the design capacities of the anchor bolts, the same note also required a minimum edge distance of 7 in for sidewall columns, 6 in for endwall columns, and 4 in for door jambs (see Fig. 14.16). Details of foundation piers at corner and middle columns are shown in Fig. 14.18.

Since the exterior foundation walls were partly exposed to match existing, they acted as basement walls retaining soil and had to be tied to the slab on grade with #4 dowels spaced at 12 ft o.c. Soil compaction near the walls with the dowels sticking out is very difficult, and the dowels were to be bent in the field (see Fig. 14.19).

The final step in the foundation design involved an uplift check at the new frame piers. The design uplift force was estimated first. From BOCA 1996, the gross uplift loading on the roof was determined by the following formula:

$$P = P_v I \,[K_h G_h C_p - K_h (GC_{pi})\,]$$

where $P_v = 18.5$ psf for 85-mph wind
$I = 1.08$ (importance factor)

From the appropriate tables in the code:

$$K_h = 0.87$$

$$G_h = 1.29$$

$$GC_{pi} = 0.25$$

$$C_p = -0.7$$

$$P = 18.5 \times 1.08\,[0.87 \times 1.29 \times (-0.7) - 0.87 \times 0.25] = -20.0 \text{ psf (uplift)}$$

The net unit uplift can be found by subtracting two-thirds of the unit building dead load:

$$P_{\text{net}} = 20.0 - {}^2\!/_3 \times 5 = 16.67 \text{ (psf)}$$

The total uplift force on foundation is then

$$\frac{16.67}{1000}\,\frac{60}{2}\,29.25 = 14.63 \text{ kip}$$

This uplift force was resisted by two-thirds of the weight of the foundation pier, footing, soil on top of the footing, and new and existing foundation walls doweled to the pier. The height of the walls was taken as 4.75 ft to the top of the wall footing. As can be easily checked, the available “ballast” was more than adequate.

14.6.6 Developing Horizontal Frame Reactions

The available total horizontal reinforcement in the existing walls was four #5 bars, with a combined area of 1.23 in^2. Assuming the bars were properly spliced and using the allowable steel tension stress of 24 ksi, the bars could safely develop a tension force of

$$24 \times 1.23 = 29.5 \text{ kip} > 17 \text{ kip} \qquad \text{(OK)}$$

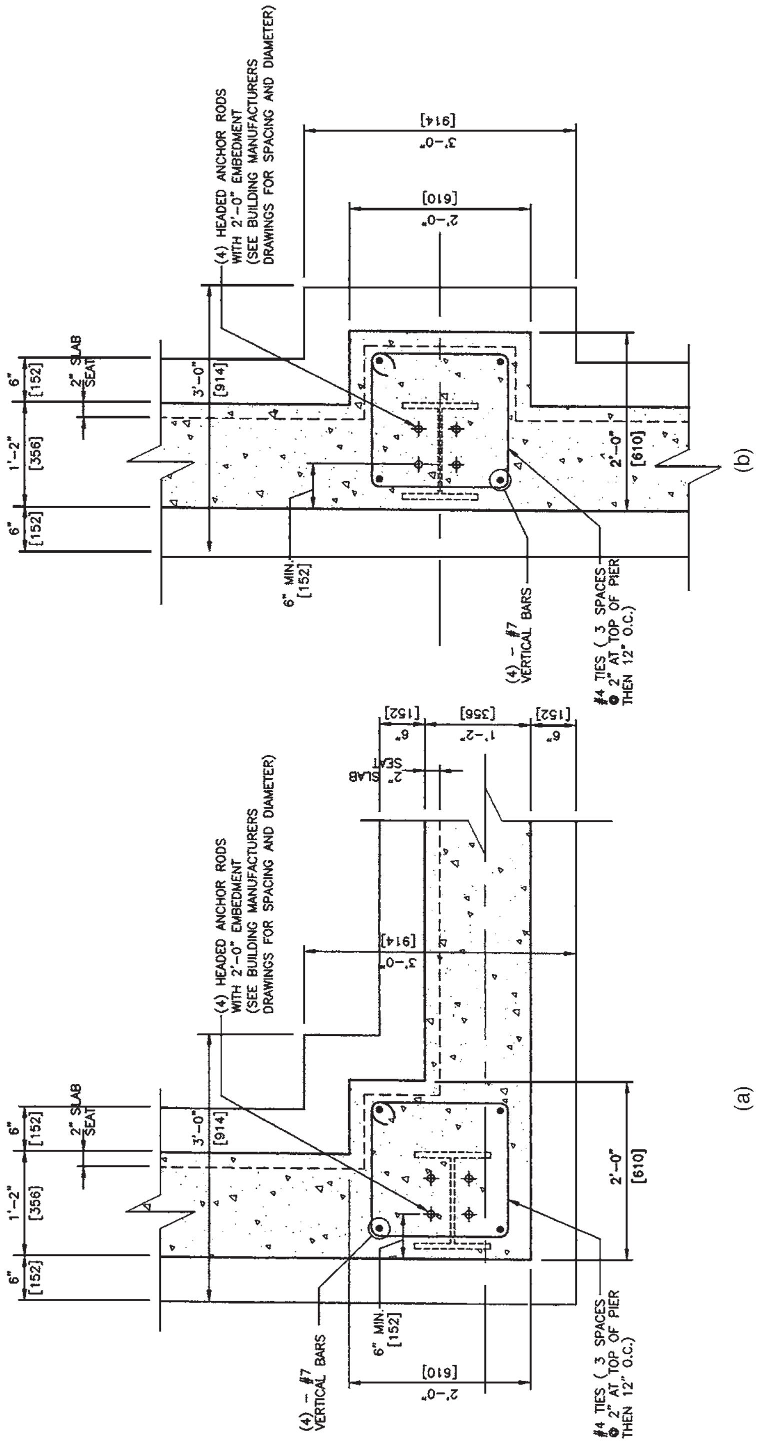

FIGURE 14.18 Details of foundation piers at new endwall in the Case Study: (*a*) corner; (*b*) middle.