Therefore, select #7 vertical bars spaced 40 in o.c. and two #4 bars placed in horizontal bond beams spaced 48 in o.c.

Example 7.2 Design the horizontal steel beam used as lateral top wall support in Example 7.1. Use steel with yield strength of 50 ksi. Use the wind-load deflection limit of $L/600$.

solution Since the reaction R_2 per foot of the beam was found in Example 7.1. as 360 lb/ft, the maximum design bending moment for a horizontally spanning beam is

$$M_{\max} = \frac{(360)25^2}{8} = 28{,}125 \text{ lb-ft} = 28.125 \text{ kip-ft}$$

If the beam is fully braced,

$$F_{bx} = 0.66(50) = 33 \text{ ksi}$$

The required section modulus is

$$S_{x,\,rq} = (28.125)12/33 = 10.23 \text{ in}^3$$

Using a deflection limit of $L/600$, the required moment of inertia is approximately

$$I_{x,\,rq} = \frac{(10.23)(25)}{1.17} = 219 \text{ in}^4$$

Using the AISC manual,[9] try W14 × 26 ($S_x = 35.3$ in^3, $I_x = 245$ in^4).

Determine the lateral bracing requirements for the beam. Since the beam size is governed by stiffness rather than strength, the beam is lightly stressed, and its lateral bracing can be spaced farther apart than the distance L_c. Using the allowable moment tables in the manual for W14 × 26 with $M_{all} = 28.125$ kip-ft, find the unbraced length to be about 12 ft, about half the span. Use the equations of AISC Specification[10] Chapter F to check if bracing only at midspan is acceptable. For $l = 12.5$ ft, $r_T = 1.28$, $d/A_f = 6.59$, $C_b = 1.0$, and $F_y = 50$ ksi, compute:

$$\sqrt{\frac{510{,}000 C_b}{F_y}} = 101$$

$$\frac{l}{r_T} = \frac{(12.5)12}{1.28} = 117.2 > 101$$

so use equations F1-7 and F1-8.

$$F_b = \frac{170{,}000(1.0)}{(117.2)^2} = 12.38 \text{ ksi} < 0.6\,F_y \qquad \text{(F1-7)} \qquad \leftarrow \text{Use}$$

$$F_b = \frac{12{,}000(1.0)}{(12.5)(12)(6.59)} = 12.14 \text{ ksi} < 0.6\,F_y \qquad \text{(F1-8)}$$

Use $F_b = 12.38$ ksi.

$$f_b = \frac{28.125(12)}{35.3} = 9.56 < 12.38 \text{ ksi} \qquad \text{OK}$$

Check the beam's lateral deflection at the design wind load:

$$\Delta_{hor} = \frac{5(0.360)24^4(1728)}{(384)(29{,}000)(245)} = 0.445 \text{ in} \qquad \text{or} \qquad \frac{0.445}{(25)(12)} = L/674 < L/600 \qquad \text{OK}$$

Check the beam's vertical deflection under its own weight for $l = 12.5$ ft and $I_y = 8.91$ in^4:

$$\Delta_{vert} = \frac{0.0054(0.026)12.5^4(1728)}{29{,}000(8.91)} = 0.023 \text{ in} \qquad \text{(negligible)}$$

Therefore, select the W14 × 26 girt with flange braces spaced at midspan.

7.4.4 Horizontally Spanning CMU Walls

As discussed in Sec. 7.3.5, hard walls can be designed to span horizontally. A couple of CMU-related design nuances are worth examining. The first is the attachment of the CMU wall to frame columns at the bond beams locations. A note on the typical connection detail (see inset on Fig. 7.27) states that the masonry ties or anchors are excluded from the manufacturer's scope of work. This means that the responsibility for their design partly falls on the architect/engineer. We say "partly," because these attachments should be coordinated with the column flange bracing design (Fig. 7.24), which often originates with the manufacturer.

The second nuance concerns a need for base flashing in single-wythe CMU walls, as shown in Fig. 7.27. According to one school of thought, hollow CMU acts akin to a cavity wall, and any moisture penetrating the outer shell must be removed—hence the flashing. However, the CMU walls reinforced in two directions, as required by contemporary building codes, may not have a series of full-height vertical cavities where water can flow. Unless the walls contain a lot of heavy horizontal joint reinforcement, they may have fully grouted bond beams placed at close intervals (perhaps 4 ft on-centers, measured vertically). When bond beams are present, the cavity-wall analogy applies only to each wall segment between them. Logically, flashing should then either be provided above every bond beam—an unusual design—or nowhere at all.

There could also be another, unrelated reason for having the base flashing: to break the bond between the wall and foundation, in order to allow wall rotation under horizontal loads, the issue discussed in Chap. 11.

Example 7.3 Using the design loading, masonry strength, and Seismic Performance Category from Example 7.1, design an 8-in CMU wall to span horizontally between the columns spaced 25 ft o.c. No steel girts or wind columns are provided.

solution The maximum bending moment for the wall spanning horizontally a distance of 25 ft and subjected to a wind load of 25 psf is

$$M_{max} = \frac{(25)25^2}{8} = 1953.125 \text{ lb-ft} = 23{,}438 \text{ lb-in} \qquad \text{(per ft of wall height)}$$

This moment is much larger than the moment computed in Example 7.1 for a vertically spanning wall, but the situation is helped by the fact that a double curtain of the bond beam bars provides a larger effective beam depth. Using NCMA Table 3.2.15 and neglecting the one-third increase in allowable stresses, find the most economical horizontal bond-beam reinforcement:

Two #8 bars spaced 40 in o.c. (resisting moment of 23,544 in-lb/ft)

Note that the bars larger than #8 may be difficult to fit inside an 8-in bond beam and still provide the required grout envelopment around them. (Some might feel that even the #8 bars may be too large for the task.) This provides a horizontal reinforcement percentage of