increase the flexural capacity of the wall, it is usually more economical to make it thicker than to brace it with horizontal girts or wind columns. (Note that the design thickness of the ribbed blocks does not include the ribs, and the ribbed blocks must be thicker than the regular blocks for the same span and loading.)

Bracing rigid masonry by sufficiently strong but overly flexible steel members is not effective because of stiffness incompatibility, a consideration frequently missing in some sophomoric calculations that treat masonry no differently than metal siding. Masonry tends to crack at relatively small lateral deflections, while metal siding and girts can deflect a lot without breaking. As a result, instead of bracing the masonry when it deforms under horizontal loading, flexible girts simply move along without becoming fully stressed. By the time the girts are stressed enough to apply the intended restraining forces to the masonry, the brittle wall may have already cracked.

To be effective as lateral wall bracing, the girts must laterally deflect under load less than the CMU walls they are intended to strengthen. This typically requires deeper and heavier girt sections than needed for strength alone. The common 8-in-deep cold-formed Z girts usually do not provide adequate lateral support for 8-in CMU walls, and deep structural steel sections are normally required. As already stated, the maximum allowable horizontal deflection of steel members used for bracing masonry is typically taken as $L/600$ and perhaps even less. The design process parallels that of Example 7.2, below.

Whenever CMU walls are present, substantial lateral stiffness is required not only of the girts, but also of the primary building frames. Otherwise, sturdy girts will be framed into a structure that moves ("drifts") excessively under lateral load and renders them ineffective. The issue of lateral drift criteria for metal buildings with masonry walls is discussed in Chap. 11.

Example 7.1 Design an exterior single-wythe CMU wall for a pre-engineered building with single-span rigid frames, 24-ft eave height, and 80-ft frame width. The design wind load is 25 psf, and the roof live load is 20 psf. The wall spans vertically, with the top girt placed behind the masonry and below the knee, as shown in Fig. 7.26*a*. The wall carries no vertical load, and its own weight may be neglected. Use ACI 530[7] Seismic Performance Category D to determine the maximum spacing of vertical and horizontal wall reinforcement and the minimum reinforcement percentages. For this example, consider only the wind loading normal to the wall; neglect seismic loading and any shear-wall behavior. Assume the specified compressive strength of masonry f'_m of 2000 psi and "partially grouted" masonry.

solution First, determine the approximate location of the top girt to establish the design wall span (the distance L in Fig. 7.26*a*). Consulting the frame tables in Chap. 4, find the distance from the column base to the bottom of the knee to be 21 ft. Since this number is approximate, conservatively locate the girt 20 ft above the base. The wall can then be analyzed as a cantilevered beam with $L = 20$ ft and $a = 4$ ft subjected to uniform load $w = 25$ lb/ft (Fig. 7.26*b*).

The horizontal reactions R_1 and R_2 and the maximum design bending moments M_1 and M_2 can be found by standard beam formulas:

$$R_1 = \frac{25}{(2)\,20}(20^2 - 4^2) = 240 \text{ lb/ft}$$

$$R_2 = \frac{25}{(2)\,20}(20^2 + 4^2) = 360 \text{ lb/ft}$$

$$M_1 = \frac{25}{(8)\,20^2}(20^2 + 4^2)(20^2 - 4^2) = 1152 \text{ ft-lb/ft} = 13{,}824 \text{ in-lb/ft}$$

$$M_2 = \frac{(25)4^2}{2} = 200 \text{ ft-lb/ft} = 2400 \text{ in-lb/ft}$$

The required moment-resisting capacity of the wall can be found by any accepted masonry design methods. In lieu of hand calculations or computer programs, one can use the *Concrete Masonry*

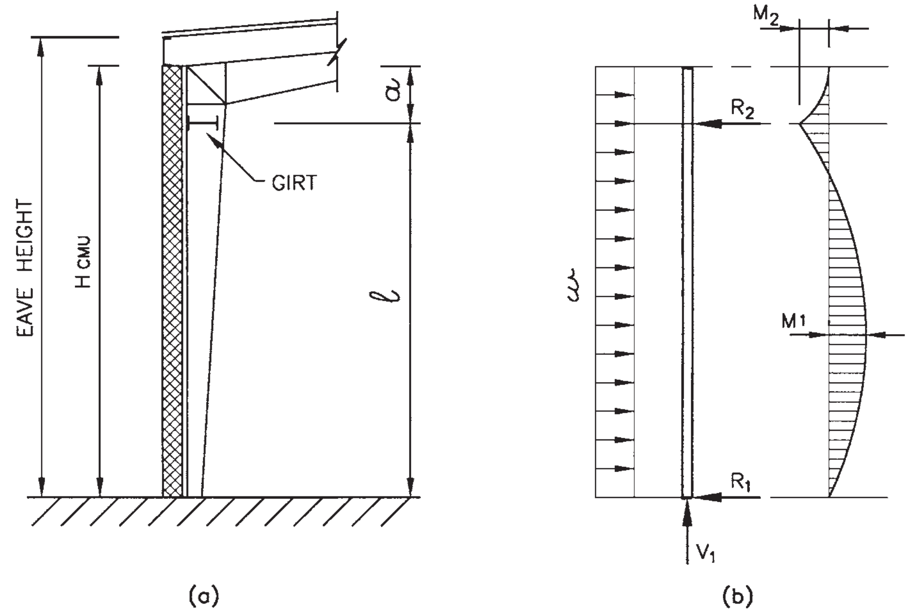

FIGURE 7.26 CMU wall for Example 7.1.

Design Tables of the NCMA[8] for 8-in CMU with f_m' of 2000 psi with bars placed in the mid-depth of the wall. Because the building is in Seismic Performance Category D, the maximum spacing of vertical and horizontal bars is 48 in.

Since the wall is non-loadbearing, the load combination includes only wind and dead load (the latter is neglected for this example). Most latest codes do not allow a one-third increase in allowable stresses (or a 25 percent reduction in total loading) for this case, as the previous practice permitted. (If the governing building code still allows such increase in stresses or reduction of loading, multiply the allowable moments by 1.33.) From NCMA Table 3.2.13, find the most economical vertical reinforcement:

#7 bars spaced 40 in o.c. (resisting moment of 14,833 in-lb/ft)

Check the minimum reinforcement percentages. The vertical #7 bars spaced 40 in o.c. provide a reinforcement ratio (using the actual block size) of:

$$\frac{0.60}{(40)(7.625)} = 0.00196 > 0.0007 \qquad \text{OK}$$

Also

$$0.00196 > (2/3)(0.002) = 0.0013$$

Since the vertical bars provide more than two-thirds of the total required, the area of the horizontal reinforcement needs to be only:

$$A_h = (0.0007)(48)(7.625) = 0.256 \text{ in}^2$$

This can be satisfied by two #4 bars (A_h = 0.39 in^2) or one #5 bar (A_h = 0.31 in^2). It is customary to provide two bars in bond-beam units, so select two #4's.