Find

1. Using the "total shear method" of horizontal shear load distribution, determine the total length of shear wall required and the required shear wall construction in the N-S direction.
2. Using the "tributary area method" of horizontal shear load distribution, determine the shear resistance and wall construction required in each N-S shear wall line.
3. Using the "relative stiffness method" of horizontal shear load distribution, determine the shear loads on the N-S shear wall lines.

Solution

1. Using the total shear approach, determine the unit shear capacity required based on the given amount of available shear wall segments in each N-S wall line and the total N-S shear load.

In this part of the example, it is assumed that the wall lines will be designed as segmented shear wall lines. From the given information, the total length of N-S shear wall available is 45 ft. It is typical practice in this method to not include segments with aspect ratios greater than 2 since stiffness effects on the narrow segments are not explicitly considered. This would eliminate the 2 ft segments and the total available length of shear wall would be 45 ft – 8 ft = 37 ft in the N-S direction.

The required design unit shear capacity of the shear wall construction and ultimate capacity is determined as follows for the N-S lateral design loads:

Wind N-S

$F'_{s,wind}$ = (21,339 lb)/37 ft = 576 plf
$F_{s,\ wind} = (F'_{s,wind})(SF)$ = (576 plf)(2.0) = 1,152 plf

Thus, the unfactored (ultimate) and unadjusted unit shear capacity for the shear walls must meet or exceed 1,152 plf. Assuming that standard 1/2-thick GWB finish is used on the interior wall surfaces (80 plf minimum from Table 6.3), the required ultimate capacity of the exterior sheathing is determined as follows:

$F_{s,wind} = F_{s,ext} + F_{s,int}$
$F_{s,ext}$ = 1,152 plf – 80 plf = 1,072 plf

From Table 6.1, any of the wall constructions that use a 4 inch nail spacing at the panel perimeter exceed this requirement. By specifying and 3/8-thick Structural I wood structural panel with 8d common nails spaced at 4 inches on center on the panel edges (12 inches on center in the panel field), the design of the wall construction is complete and hold-down connections and base shear connections must be designed. If a different nail is used or a framing lumber species with G < 0.5, then the values in Table 6.1 must be multiplied by the C_{ns} and C_{sp} factors. For example, assume the following framing lumber and nails are used in the shear wall construction:

lumber species:	Spruce-Pine-Fir (G=0.42)	C_{sp} = 0.92
nail type:	8d pneumatic, 0.113-inch-diameter	C_{ns} = 0.75

Thus, values in Table 6.1 would need to be multiplied by (0.92)(0.75) = 0.69. This adjustment requires a 15/32-inch-thick sheathing with the 8d nails (i.e., 1,539 plf x 0.69 = 1,062 plf which is close enough to the required 1,072 plf for practical design purposes). Alternatively, a 7/16-inch thick wood structural panel sheathing could be used in accordance with footnote 5 of Table 6.1; however, the horizontal joint between panels would need to be blocked. In extreme lateral load conditions, it may be necessary (and more efficient) to consider a "double sheathed" wall construction (i.e., structural wood panels on both sides of the wall framing) or to consider the addition of an interior shear wall line (i.e., design the interior walls along wall line C as shear walls).

Seismic N-S

$F'_{s,seismic}$ = (8,983 lb)/37 ft = 243 plf
$F_{s, seismic}$ = ($F'_{s,seismic}$)(SF) = (243 plf)(2.5) = 608 plf

Thus, the unfactored (ultimate) and unadjusted unit shear capacity for the wall line must meet or exceed 608 plf. Since seismic codes do not permit the consideration of a 1/2-thick GWB interior finish, the required ultimate capacity of the exterior sheathing is determined as follows:

$F_{s,seismic} = F_{s,ext}$ = 608 plf

From Table 6.1, any of the wood structural panel wall constructions that use a 6 inch nail spacing at the panel perimeter exceed this requirement. By specifying 3/8-inch-thick Structural I wood structural panels with 8d common nails spaced at 6 inches on center on the panel edges (12 inches on center in the panel field), the design of the wall construction is complete and hold-down connections and base shear connections must be designed. If a different nail is used or a framing lumber species with G < 0.5, then the values in Table 6.1 must be multiplied by the C_{ns} and C_{sp} factors as demonstrated above for the N-S wind load case.

The base shear connections may be designed in this method by considering the total length of continuous bottom plate in the N-S shear wall lines. As estimated from the plan, this length is approximately 56 feet. Thus, the base connection design shear load (parallel to the grain of the bottom plate) is determined as follows:

Base wind design shear load = (21,339 lb)/(56 ft) = 381 plf
Base seismic design shear load = (8,983 lb)/(56 ft) = 160 plf

The base shear connections may be designed and specified following the methods discussed in Chapter 7 – Connections. A typical 5/8-inch-diameter anchor bolt spaced at 6 feet on center or standard bottom plate nailing may be able to resist as much as 800 plf (ultimate shear capacity) which would provided a "balanced" design capacity of 400 plf or 320 plf for wind and seismic design with safety factors of 2.0 and 2.5, respectively. Thus, a conventional wall bottom plate connection may be adequate for the above condition; refer to Chapter 7 for connection design information and the discussion in Section 7.3.6 for more details on tested bottom plate connections.

If the roof uplift load is not completely offset by 0.6 times the dead load at the base of the first story wall, then strapping to transfer the net uplift from the base of the wall to the foundation or construction below must be provided.

The hold-down connections for the each shear wall segment in the designated shear wall lines are designed in the manner shown in Example 6.1. Any overturning forces originating from shear walls on the second story must also be included as described in Section 6.4.2.4.