As an alternative, the portion of the wall to the left of the door opening can be treated as a separate perforated shear wall for the left-to-right loading condition. In doing so, the design shear capacity of the left portion of the wall may be determined to be 1,224 lb and the base shear connection required is (1,224 lb)/8ft = 153 plf, much less than the 340 lb required in the wind load condition. The right side of the wall is solid sheathed and, for the right-to-left loading condition, the base shear is equivalent to the design shear capacity of the wall or 340 plf. These calculations can also be performed using the seismic design values for the perforated shear wall. This approach is based on the behavior of a perforated shear wall where the leading edge and the immediately adjacent shear wall segments are fully restrained as in the segmented shear wall approach for one direction of loading. Thus, these segments will realize their full unit shear capacity for one direction of loading. Any interior segments will contribute, but at a reduced amount do to the reduced restraint condition. This behavior is represented in the adjustment provided by the C_{op} factor which is the basis of the perforated shear wall method. Unfortunately, the exact distribution of the uplift forces and shear forces within the wall are not known. It is for this reason that they are assigned conservative values for design purposes. Also, to accommodate potential uplift forces on the bottom plate in the regions of interior perforated shear wall segments, the base shear connections are required to resist an uplift load equivalent to the design unit shear capacity of the wall construction. In the case of this example, the base shear connection would need to resist a shear load of 340 plf (for the wind design condition) and an uplift force of 340 plf (even if under a zero wind uplift load).

Testing has shown that for walls constructed similar to the one illustrated in this example, a bottom plate connection of 2 16d pneumatic nails (0.131 inch diameter by 3 inches long) at 16 inches on center or 5/8-inch-diameter anchor bolts at 6 feet on center provides suitable shear and uplift resistance – at least equivalent to the capacity of the shear wall construction under conditions of no dead load or wind uplift (NAHBRC, 1999). For other conditions, this connection must be designed following the procedures given in Chapter 7 using the conservative assumptions as stated above.

As an alternative base connection that eliminates the need for hold-down brackets at the ends of the perforated shear wall, straps can be fastened to the individual studs to resist the required uplift force of 340 plf as applicable to this example. If the studs are spaced 16 inches on center, the design capacity of the strap must be (340 plf)(1.33 ft/stud) = 452 lb per stud. If an uplift load due to wind uplift on the roof must also be transferred through these straps, the strap design capacity must be increased accordingly. In this example, the net wind uplift at the top of the wall was assumed to be 265 plf. At the base of the wall, the uplift is 265 plf – 0.6(8 ft)(8 psf) = 227 plf. Thus, the total design uplift restraint must provide 340 plf + 227 plf = 567 plf. On a per stud basis (16 inch on center framing), the design load is 1.33 ft/stud x 567 plf = 754 lb/stud. This value must be increased for studs adjacent to wall openings where the wind uplift force in increased. This can be achieved by using multiple straps or by specifying a larger strap in these locations. Of course, the above combination of uplift loads assumes that the design wind uplift load on the roof occurs simultaneously with the design shear load on the wall. However, this condition is not usually representative of actual conditions depending on wind orientation, building configuration, and the shear wall location relative to the uplift load paths.

3. Determine the chord tension and compression forces

The chord tension and compression forces are determined following the same method as used in Example 6.1 for the segmented shear wall design method, but only for the first wall segment in the perforated shear wall line (i.e. the restrained segment). Therefore, the tension forces at the end of the wall are identical to those calculated in Example 6.1 as shown below:

Left end of the wall (Segment 1 in Example 6.1):

T = 2,947 lb (wind design)
T = 2,093 lb (seismic design)

Right end of the wall (Segment 3 in Example 6.1):

T = 3,004 lb (wind design)
T = 2,133 lb (seismic design)

Note: One tension bracket (hold-down) is required at each the end of the perforated shear wall line and not on the interior segments. Also, refer to the notes in Example 6.1 regarding "balanced design" of overturning connections and base shear connections, particularly when designing for seismic loads.

4. Determine the load-drift behavior of the perforated shear wall line.

The basic equation for load-drift estimation of a perforated shear wall line is as follows (Section 6.5.2.6):

$$\Delta = 1.8\left(\frac{0.5}{G}\right)\left(\frac{1}{\sqrt{r}}\right)\left(\frac{V_d}{F_{PSW,ULT}}\right)^{2.8}\left(\frac{h}{8}\right) \qquad \text{(Eq. 6.5-8)}$$

h = 8 ft
G = 0.42(specific gravity for Spruce-Pine-Fir)
r = 0.73 (sheathing area ratio determined in Step 1)

$F_{psw,ult,wind}$ = $(F_{psw,wind})(SF)$ = (3,036 lb)(2.0) = 6,072 lb
$F_{psw,ult,seismic}$ = $(F_{psw,seismic})(SF)$ = (2,389 lb)(2.5) = 5,973 lb

Substituting in the above equation,

$\Delta_{wind} = 6.4\text{x}10^{-11}(V_{d,wind})^{2.8}$
$\Delta_{seismic} = 6.7\text{x}10^{-11}(V_{d,seismic})^{2.8}$

For the design wind load of 3,000 lb and the design seismic load of 1,000 lb (assumed for the purpose of this example), the drift is estimated as follows:

$\Delta_{wind} = 6.4\text{x}10^{-11}(3{,}000)^{2.8} = 0.35$ inches
$\Delta_{seismic} = 6.7\text{x}10^{-11}(1{,}000)^{2.8} = 0.02$ inches

Note: The reader is reminded of the uncertainties in determining drift as discussed in Example 6.1 and also in Chapter 6. For seismic design, some codes may require the design seismic drift to be amplified (multiplied by) a factor of 4 to account for the potential actual forces that may be experienced relative to the design forces that are determined using an R factor; refer to Chapter 3 for additional discussion. Thus, the amplified drift may be determined as 4 x 0.02 inches = 0.08 inches. However, if the seismic shear load is magnified (i.e., 4 x 1,000 lb = 4,000 lb) to account for a possible actual seismic load (not modified for the seismic response of the shear wall system), the seismic drift calculated in the above equation becomes 0.8 inches which is an order of magnitude greater. The load adjustment is equivalent to the use of an R of 1.5 instead of 6 in Equation 3.8-1 of Chapter 3. However, this latter approach of magnifying the load