Segment 2

h = 8 ft
d = 2 ft
x = 2 ft – 0.5 ft = 1.5 ft
$F'_{s,2,wind}$ = 250 plf
$F'_{s,2,seismic}$ = 178 plf

T = C = (2 ft / 1.5 ft)(250 plf)(8 ft) = 2,667 lb (wind)
T = C = (2 ft / 1.5 ft)(178 plf)(8 ft) = 1,899 lb (seismic)

Segment 3

h = 8 ft
d = 8 ft
x = 8 ft – 0.5 ft = 7.5 ft
$F'_{s,2,wind}$ = 352 plf
$F'_{s,2,seismic}$ = 250 plf

T = C = (8 ft / 7.5 ft)(352 plf)(8 ft) = 3,004 lb (wind)
T = C = (8 ft / 7.5 ft)(250 plf)(8 ft) = 2,133 lb (seismic)

Notes:

1. In each of the above cases, the seismic tension and compression forces on the shear wall chords are less than that determined for the wind loading condition. This occurrence is the result of using a larger safety factor to determine the shear wall design capacity and the practice of not including the interior sheathing (GWB) design shear capacity for seismic design. Thus, the chord forces based on the seismic shear wall design capacity may be under-designed unless a sufficient safety factor is used in the manufacturer’s rated hold-down capacity to compensate. In other words, the ultimate capacity of the hold-down connector should be greater than the overturning force that could be created based on the ultimate shear capacity of the wall, including the contribution of the interior GWB finish. This condition should be verified by the designer since the current code practice may not provide explicit guidance on the issue of balanced design on the basis of system capacity (i.e., connector capacity relative to shear wall capacity). This issue is primarily a concern with seismic design because of the higher safety factor used to determine design shear wall capacity and the code practice not to include the contributing shear capacity of the interior finish.
2. The compression chord force should be recognized as not being a point load at the top of the stud(s) comprising the compression chord. Rather, the compression chord force is accumulated through the sheathing and begins at the top of the wall with a value of zero and increases to C (as determined above) at the base of the compression chord. Therefore, this condition will affect how the compression chord is modeled from the standpoint of determining its capacity as a column using the column equations in the NDS.
3. The design of base shear connections and overturning forces assume that the wind uplift forces at the base of the wall are offset by 0.6 times the dead load (ASD) at that point in the load path or that an additional load path for uplift is provided by metal strapping or other means.
4. As mentioned in Step 2 for the design of base shear connections, the wind load on the designated shear wall segments may be distributed according to the design capacity of each segment in proportion to that of the total shear wall line. This method is particularly useful when the design shear capacity of the wall line is substantially higher than the shear demand required by the wind load as is applicable to this hypothetical example. Alternatively, a shear wall segment may be eliminated from the analysis by not specifying restraining devices for the segment (i.e., hold-down brackets). If the former approach is taken, the wind load is distributed as follows:

Fraction of design wind load to Segment 1:

$F_{ssw,1,wind} / F_{ssw,total,wind}$ = (921 lb)/(4,237 lb) = 0.22

Fraction of wind load to Segment 2:

$F_{ssw,2,wind} / F_{ssw,total,wind}$ = (500 lb)/(4,237 lb) = 0.12

Fraction of wind load to Segment 3:

$F_{ssw,3,wind} / F_{ssw,total,wind}$ = (2,816 lb)/(4,237 lb) = 0.66

Thus, the unit shear load on each shear wall segment due to the design wind shear of 3,000 lb on the total wall line is determined as follows:

Segment 1: 0.22(3,000 lb)/(3 ft) = 220 plf
Segment 2: 0.12(3,000 lb)/(2 ft) = 180 plf
Segment 3: 0.66(3,000 lb)/(8 ft) = 248 plf

Now, the overturning forces (chord forces) determined above and the base shear connection requirements determined in Step 2 may be recalculated by substituting the above values, which are based on the design wind loading. This approach only applies to the wind loading condition when the design wind loading on the wall line is less than the design capacity of the wall line. As mentioned, it may be more efficient to eliminate a designed shear wall segment to bring the total design shear capacity more in line with the design wind shear load on the wall. Alternatively, a lower capacity shear wall construction may be specified to better match the loading condition (i.e., use a thinner wood structural sheathing panel, etc.). This decision will depend on the conditions experienced in other walls of the building such that a single wall construction type may be used throughout for all exterior walls (i.e., simplified construction).

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

Only the load-drift behavior for wind design is shown below. For seismic design, a simple substitution of the design shear capacities of the wall segments and the safety factor for seismic design (as determined previously) may be used to determine a load-drift relationship for use in seismic design.

The basic equation for load-drift estimation of a shear wall segment is as follows:

$$\Delta = 2.2\left(\frac{0.5}{G}\right)\sqrt[4]{a}\left(\frac{V_d}{F_{SSW,ULT}}\right)^{2.8}\left(\frac{h}{8}\right) \qquad \text{(Equation 6.5-9)}$$

h = 8 ft
G = 0.42 (Spruce-Pine-Fir)

Aspect ratios for the wall segments

$a_1 = 2.67$
$a_2 = 4.0$
$a_3 = 1.0$

$F_{ssw,ult,1,wind} = F_{ssw,1,wind}(SF)$ = (921 lb)(2.0) = 1,842 lb
$F_{ssw,ult,2,wind} = F_{ssw,2,wind}(SF)$ = (500 lb)(2.0) = 1,000 lb
$F_{ssw,ult,3,wind} = F_{ssw,3,wind}(SF)$ = (2,816 lb)(2.0) = 5,632 lb