Wood Design

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This chapter examines the design of a variety of wood building elements. Section 11.1 features a three-
story, wood-frame apartment building. Section 11.2 illustrates the design of the roof diaphragm and wall-
to-roof anchorage for the masonry building featured in Section 10.1. In both cases, only those portions of
the designs necessary to illustrate specific points are included.

Typically, the weak link in wood systems is the wood strength at the connections, but the desired ductility
must be developed by means of these connections. Wood members have some ductility in compression
(particularly perpendicular to grain) but little in tension. Nailed plywood shear panels develop
considerable ductility through yielding of nails and crushing of wood adjacent to nails. Because wood
structures are composed of many elements that must act as a whole, the connections must be considered
carefully to ensure that the load path is complete. Tying the structure together is essential to good
earthquake-resistant construction.

Wood elements often are used in low-rise masonry and concrete buildings. The same basic principles
apply to the design of wood elements, but certain aspects of the design (for example, wall-to-diaphragm
anchorage) are more critical in mixed systems than in all-wood construction.

Wood structural panel sheathing is referred to as “plywood” in this chapter. However, sheathing can
include plywood and other products, such as oriented-strand board (OSB), that conform to the appropriate
materials standards.

The calculations herein are intended to provide a reference for the direct application of the design
requirements presented in the 2009 NEHRP Recommended Provisions (hereafter, the Provisions) and its
primary reference document, ASCE 7-05 Minimum Design Loads for Buildings and Other Structures
(hereafter, the Standard) and to assist the reader in developing a better understanding of the principles
behind the Provisions and the Standard.

In addition to the Provisions, the documents below are referenced in this chapter. Although the Standard
references the 2005 edition of the AF&PA SDPWS, this chapter utilizes the 2008 edition, which is the
more recent, updated version. Note that the 2005 editions of the AF&PA NDS and AF&PA NDS
Supplement are the latest versions.

ACI 318  
American Concrete Institute. 2008. Building Code Requirements and Commentary for Structural Concrete.

ACI 530  

ANSI/AITC A190.1  

ASCE 7  

AF&PA Guideline  

AF&PA NDS  
11.1 THREE-­­STORY WOOD APARTMENT BUILDING, SEATTLE, WASHINGTON

This example features a wood-frame building with plywood diaphragms and shear walls.

11.1.1 Building Description

This three-story wood-frame apartment building has a double-loaded central corridor. The building is typical stick-frame construction consisting of wood joists and stud bearing walls supported by a concrete foundation wall and strip footing system. The seismic force-resisting system consists of plywood floor and roof diaphragms and plywood shear walls. Figure 11.1-1 shows a typical floor plan and Figure 11.1-2 shows a longitudinal section and elevation. The building is located in a residential neighborhood a few miles north of downtown Seattle.

The shear walls in the longitudinal direction are located on the exterior faces of the building and along the corridor. The entire solid (non-glazed) area of the exterior walls has plywood sheathing, but only a portion of the corridor walls will require sheathing. In the transverse direction, the end walls and one line of interior shear walls provide lateral resistance. It should be noted that while plywood sheathing generally is used at the exterior walls for reasons beyond just lateral load resistance, the interior longitudinal (corridor) and transverse shear walls could be designed using gypsum wallboard as permitted by AF&PA SDPWS Section 4.3.7.5. However, the corridor shear walls are not included in this example and the interior transverse walls are designed using plywood sheathing, largely due to the required shear capacity.

The floor and roof systems consist of wood joists supported on bearing walls at the perimeter of the building, the corridor lines, plus one post-and-beam line running through each bank of apartments. Exterior walls are framed with 2×6 studs for the full height of the building to accommodate insulation. Interior bearing walls require 2×6 or 3×4 studs on the corridor line up to the second floor and 2×4 studs above the second floor. Apartment party walls are not load-bearing; however, they are double walls and are constructed of staggered 2×4 studs at 16 inches on center. Surfaced, dry (seasoned) lumber is used for all framing to minimize shrinkage. Floor framing members are assumed to be composed of Douglas Fir-Larch material and wall framing is Hem-Fir No. 2, as graded by the WWPA Rules. The material and grading of other framing members associated with the lateral design is as indicated in the example. The lightweight concrete floor fill is for sound isolation and is interrupted by the party walls, corridor walls and bearing walls.

The building is founded on interior footing pads, continuous strip footings and concrete foundation walls (Figure 11.1-3). The depth of the footings and the height of the walls are sufficient to provide crawlspace clearance beneath the first floor.
Figure 11.1-1 Typical floor plan
(1.0 ft = 0.3048 m)

Figure 11.1-2 Longitudinal section and elevation
(1.0 ft = 0.3048 m)
11.1.1.1 Scope. In this example, the structure is designed and detailed for forces acting in the transverse and longitudinal directions, including the following:

- Design and detailing of transverse plywood walls for shear and overturning moment.
- Design and detailing of plywood floor and roof diaphragms.
- Design and detailing of wall and diaphragm chord members.
- Design and detailing of longitudinal plywood walls using the requirements for perforated shear walls.

The simplified procedure, new to the 2005 edition of the Standard, is permitted for relatively short, simple and regular structures utilizing shear walls or braced frames. The seismic analysis and design procedure is much less involved than a building utilizing a seismic force resisting system listed in Standard Section 12.2 and analyzed using one of the procedures listed in Standard Section 12.6. See Section 11.1.2.2 for a more detailed discussion of what is and is not required for the seismic design. In accordance with Standard Section 12.14.1.1, the subject building qualifies for the simplified procedure because of the following attributes:

- Residential occupancy
- Three stories in height
- Bearing wall lateral system
- At least two lines of lateral force-resisting elements in both directions, at least one on each side of the center of mass
- No cantilevered diaphragms or structural irregularities
11.1.2 Basic Requirements

11.1.2.1 Seismic Parameters

<table>
<thead>
<tr>
<th>Table 11.1-1 Seismic Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design Parameter</td>
</tr>
<tr>
<td>Occancy Category (Standard Sec. 1.5.1)</td>
</tr>
<tr>
<td>Short-Period Response, $S_S$</td>
</tr>
<tr>
<td>Site Class (Standard Sec. 11.4.2)</td>
</tr>
<tr>
<td>Seismic Design Category (Standard Sec. 11.6)</td>
</tr>
<tr>
<td>Seismic Force-Resisting System (Standard Table 12.14-1)</td>
</tr>
<tr>
<td>Shear Walls</td>
</tr>
<tr>
<td>Response Modification Coefficient, $R$</td>
</tr>
</tbody>
</table>

11.1.2.2 Structural Design Criteria

11.1.2.2.1 Ground Motion Parameter. Unlike the typical design procedures in Standard Chapter 12, the simplified procedure requires consideration of just one spectral response parameter, $S_D$. This is because the behavior of short, stiff buildings for which the simplified procedure is permitted will always be governed by short-period response. In accordance with Standard Section 12.14.8.1:

$$S_D = \frac{2}{3}F_wS_S$$

The site coefficient, $F_w$, can be determined using Standard Section 12.14.8.1 with simple default values based on soil type or using Standard Table 11.4-1 if the site class is known. Since Standard Table 11.4-1 generally will result in more favorable value, that method is used for this example. Using $S_S = 1.34$ and Site Class D, Standard Table 11.4-1 lists a short-period site coefficient, $F_w$, of 1.0. Therefore, in accordance with Standard Equation:

$$S_D = \frac{2}{3}(1.0)(1.34) = 0.89$$

11.1.2.2.2 Seismic Design Category (Standard Sec. 11.6). Where the simplified procedure is used, Standard Section 11.6 permits the Seismic Design Category to be determined based on Standard Table 11.6-1 only. Based on the Occancy Category and the design spectral response acceleration parameter, the subject building is assigned to Seismic Design Category D.

11.1.2.2.3 Seismic Force-Resisting Systems (Standard Sec. 12.14.4). See Figure 11.1-4. For both directions, the load path for seismic loading consists of plywood floor and roof diaphragms and plywood shear walls. Because the lightweight concrete floor topping is discontinuous at each partition and wall, it is not considered to be a structural diaphragm. In accordance with Standard Table 12.14-1, building has a bearing wall system comprised of light-framed walls sheathed with wood structural panels. The response modification factor, $R$, is 6.5 for both directions.
11.1.2.2.4 Diaphragm Flexibility (Standard Sec. 12.14.5). Standard Section 12.14.5 defines a diaphragm comprised of wood structural panels as flexible. Because the lightweight concrete floor topping is discontinuous at each partition and wall, it is not considered to be a structural diaphragm.

11.1.2.2.5 Application of Loading (Standard Sec. 12.14.6). For the simplified procedure, seismic loads are permitted to be applied independently in two orthogonal directions.

11.1.2.2.6 Design and Detailing Requirements (Standard Sec. 12.14.7). The plywood diaphragms are designed for the forces prescribed in Standard Section 12.14.7.4. The design of foundations is per Standard Section 12.13 and wood design requirements are based on Standard Section 14.4 as discussed in greater detail below. This example does not require any collector elements (Standard Sec. 12.14.7.3).

11.1.2.2.7 Analysis Procedure (Standard Sec. 12.14.8). For the simplified procedure, only one analysis procedure is specified and it is described in greater detail in Section 11.1.3.1 below.

11.1.2.2.8 Drift Limits (Standard Sec. 12.14.8.5). Where the simplified procedure is used, there are not any specific drift limitations because the types of structures for which the simplified procedure is applicable are generally not drift-sensitive. As specified in Standard Section 12.14.8.5, if a determination of expected drift is required (for the design of cladding for example), then drift is permitted to be computed as 1 percent of the building height unless a more detailed analysis is performed.

11.1.2.2.9 Combination of Load Effects (Standard Sec. 12.14.3). The basic design load combinations are as stipulated in Standard Chapter 2 as modified by the Standard Sec. 12.14.3.1.3. Seismic load effects according to the Standard Equations 12.14-5 and 12.14-6 are as follows:

\[ E = Q_E + 0.2S_{DS}D \]

\[ E = Q_E - 0.2S_{DS}D \]
where seismic and gravity are additive and counteractive, respectively.

For $S_{DS} = 0.89$, the design load combinations are as follows:

$$(1.2 + 0.2S_{DS})D + 1.0Q_E + 0.5L + 0.2S = 1.38D + 1.0Q_E + 0.5L + 0.2S$$

$$(0.9 - 0.2S_{DS})D - 1.0Q_E = 0.72D - 1.0Q_E$$

Note that there is no redundancy factor for the simplified procedure.

### 11.1.2.3 Basic Gravity Loads

- **Roof:**

<table>
<thead>
<tr>
<th>Load Type</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Live/Snow Load</td>
<td>25 psf</td>
</tr>
<tr>
<td>(in Seattle, snow load governs over roof live load; in other areas this may not be the case)</td>
<td></td>
</tr>
<tr>
<td>Dead Load</td>
<td>15 psf</td>
</tr>
<tr>
<td>(including roofing, sheathing, joists, insulation and gypsum ceiling)</td>
<td></td>
</tr>
</tbody>
</table>

- **Floor:**

<table>
<thead>
<tr>
<th>Load Type</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Live Load</td>
<td>40 psf</td>
</tr>
<tr>
<td>Dead Load</td>
<td>20 psf</td>
</tr>
<tr>
<td>(1-1/2-in. lightweight concrete, sheathing, joists and gypsum ceiling. At first floor, omit ceiling but add insulation.)</td>
<td></td>
</tr>
<tr>
<td>Interior Partitions and Corridor Walls</td>
<td>7 psf distributed floor load</td>
</tr>
<tr>
<td>(8 ft high at 11 psf)</td>
<td></td>
</tr>
<tr>
<td>Exterior Frame Walls</td>
<td>15 psf of wall surface</td>
</tr>
<tr>
<td>(wood siding, plywood sheathing, 2×6 studs, batt insulation and 5/8-in. gypsum wallboard)</td>
<td></td>
</tr>
<tr>
<td>Exterior Double Glazed Window Wall</td>
<td>9 psf of wall surface</td>
</tr>
<tr>
<td>Party Walls</td>
<td>15 psf of wall surface</td>
</tr>
<tr>
<td>(double-stud sound barrier)</td>
<td></td>
</tr>
<tr>
<td>Stairways</td>
<td>20 psf</td>
</tr>
</tbody>
</table>
Typical Footing (10 in. by 1 ft-6 in.) and 690 plf
Stem Wall (10 in. by 4 ft-0 in.)

Applicable Seismic Weights at Each Level

\[ W_{\text{roof}} = \text{Area (roof dead load + interior partitions + party walls) +} \]
\[ \text{End Walls + Longitudinal Walls} \]
182.8 kips

\[ W_2 = W_3 = \text{Area (floor dead load + interior partitions + party walls) +} \]
\[ \text{End Walls + Longitudinal Walls} \]
284.2 kips

Effective Total Building Weight, \( W \)
751 kips

For modeling the structure, the first floor is assumed to be the seismic base, because the short crawlspace with concrete foundation walls is stiff compared to the superstructure.

### 11.1.3 Seismic Force Analysis

The analysis is performed manually following a step-by-step procedure for determining the base shear (Standard Sec. 12.14.8.1), vertical distribution of forces (Standard Sec. 12.14.8.2) and horizontal distribution of forces (Standard Sec. 12.14.8.3). For a building with flexible diaphragms, Standard Section 12.14.8.3.1 allows the horizontal distribution of forces to be based on tributary areas and accidental torsion need not be considered for the simplified procedure.

#### 11.1.3.1 Base Shear Determination

According to Standard Equation 12.14-11:

\[ V = \frac{F S_{DS}}{R} W \]

Where \( F = 1.2 \) for a three-story building, \( R = 6.5 \) and \( W = 751 \) kips as determined previously. Therefore, the base shear is computed as follows:

\[ V = \frac{(1.2)(0.89)}{6.5} (751) = 123.4 \text{ kips (both directions)} \]

#### 11.1.3.2 Vertical Distribution of Forces

Forces are distributed as shown in Figure 11.1-5, where the story forces are calculated according to Standard Equation 12.14-12 as follows:

\[ F_s = \frac{w_s}{W} V \]

This results in a uniform vertical distribution of forces, where the story force is based on the relative seismic weight of the story with all stories at the same seismic acceleration (as opposed to the triangular or parabolic vertical distribution used in the Equivalent Lateral Force procedure of Standard Sec. 12.8).
The story force at each floor is computed as:

\[
\begin{align*}
F_{\text{roof}} &= \left[\frac{182.8}{751}\right](123.4) = 30.0 \text{ kips} \\
F_{3\text{rd}} &= \left[\frac{284.2}{751}\right](123.4) = 46.7 \text{ kips} \\
F_{2\text{nd}} &= \left[\frac{284.2}{751}\right](123.4) = 46.7 \text{ kips} \\
\Sigma &= 123.4 \text{ kips}
\end{align*}
\]

11.1.3.3 Horizontal Distribution of Shear Forces to Walls. Since the diaphragms are defined as flexible by Standard Section 12.14.5, the horizontal distribution of forces is based on tributary area to the individual shear walls in accordance with Standard Section 12.14.8.3.1. For this example, forces are distributed as described below.

11.1.3.3.1 Longitudinal Direction. In this direction, there are four lines of resistance, but only the exterior walls are considered in this example. The total story force tributary to the exterior wall is determined as follows:

\[
(25/2)/56F_x = 0.223F_x
\]

The distribution to each individual shear wall segment along this exterior line is discussed in Section 11.1.4.7 below.

11.1.3.3.2 Transverse Direction. Again, based on the flexible diaphragm assumption, force is to be distributed based on tributary area. As shown in Figure 11.1-4, there are three sets of two shear walls, each offset in plan by 8 feet. For the purposes of this example, each set of walls is assumed to be in alignment, resisting the same tributary width. The result is that the building is modeled with a diaphragm consisting of two simple spans, which provides a more reasonable horizontal distribution of force than a pure tributary area distribution.

For a two-span, flexible diaphragm, the central walls will resist one-half of the total load, or 0.50\(F_x\). The other walls resist story forces in proportion to the width of diaphragm between them and the central walls. The left set of walls in Figure 11.1-4 resists \(\frac{60}{2}/148F_x = 0.203F_x\) and the right set resists \(\frac{88}{2}/148F_x = 0.297F_x\), where 60 feet and 88 feet represent the dimension from the ends of the building to the centroid of the two central walls. Note that this does not exactly match the existing diaphragm spans, but is a reasonable simplification to account for the three sets of offset shear walls at the ends and middle of the building.
11.1.3.4 Diaphragm Design Forces. As specified in Standard Section 12.14.7.4, the design forces for floor and roof diaphragms are the same forces as computed for the vertical distribution in Section 11.1.3.2 above plus any force due to offset walls (not applicable for this example).

The weight tributary to the diaphragm, \( w_{p,x} \), need not include the weight of walls parallel to the force. For this example, however, since the shear walls in both directions are relatively light compared to the total tributary diaphragm weight, the diaphragm force is computed based on the total story weight, for convenience. Therefore, the diaphragm forces are exactly the same as the story forces shown above.

11.1.4 Basic Proportioning

Designing a plywood diaphragm and plywood shear wall building principally involves the determination of sheathing thicknesses and nailing patterns to accommodate the applied loads. This is especially the case where the simplified procedure is utilized, since there are not any deflection checks and possible subsequent design iterations.

In addition to the wall and diaphragm design, this design example features framing member and connection design for elements including shear wall end posts and hold-downs, foundation anchorage and diaphragm chords.

Nailing patterns in diaphragms and shear walls have been established on the basis of tabulated requirements included in the AF&PA SDPWS. It is important to consider the framing requirements for a given nailing pattern and capacity as indicated in the notes following the tables. In addition to strength requirements, AF&PA SDPWS Section 4.2.4 places aspect ratio limits on plywood diaphragms (length/width must not exceed 4/1 for blocked diaphragms) and AF&PA SDPWS Section 4.3.4 places similar limits on shear walls (height/width must not exceed 2/1 for full design capacities).

11.1.4.1 Strength of Members and Connections. The Standard references the AF&PA NDS and AF&PA SDPWS for engineered wood structures. These reference standards support both Allowable Stress Design (ASD) and Load and Resistance Factor Design (LRFD) as permitted by the Standard. For this example, LRFD is utilized. The AF&PA NDS and AF&PA Supplement contains the material design values for framing members and connections, while the AF&PA SDPWS contains the diaphragm and shear wall tables as well as detailing requirements for shear wall and diaphragm systems.

Throughout this example, the resistance of members and connections subjected to seismic forces, acting alone or in combination with other prescribed loads, is determined in accordance with the AF&PA NDS and AF&PA SDPWS. The methodology is somewhat different between the AF&PA NDS for framing members and connections and the AF&PA SDPWS for shear walls and diaphragms.

For framing members and connections, the AF&PA NDS incorporates the notation \( F_r, F_t, Z, \) etc., for reference design values, which are then modified using standard wood adjustment factors, \( C_M, C_r, C_F, \) etc. (used for both ASD and LRFD) and then for LRFD are modified by a format conversion factor, \( K_F, \) a resistance factor, \( \phi \) and a time effect factor, \( \lambda, \) to compute an adjusted design resistance, \( F_r', F_t', Z'. \) These factors are defined in AF&PA NDS Appendix N.

For shear walls and diaphragms, the AF&PA SDPWS contains tabulated unit shear values, \( v_s, \) which are multiplied by a resistance factor, \( \phi_D, \) equal to 0.8. This is the only modification to the tabulated design values since this building utilizes Douglas Fir Larch framing. Additional modification would be required for other species in accordance with the footnotes to the tabular values in the AF&PA SDPWS.
For pre-engineered connection elements, the AF&PA NDS does not contain a procedure for converting the manufacturer’s cataloged values (typically as ASD values) to LRFD. However, such a procedure is contained in a guideline published with the 1996 edition of the LRFD wood standard (AF&PA Guideline). The AF&PA Guideline contains a method for converting allowable stress design values for cataloged metal connection hardware (for example, tie-down anchors) into ultimate capacities for use with strength design. The procedure, which is used for this example, can generally be described as taking the catalog ASD value, multiplying by 2.88 and dividing the by the load duration factor on which the cataloged value is based (typically 1.33 or 1.60 for pre-engineered connection hardware often used for wind or seismic design).

11.1.4.2 Transverse Shear Walls. The design will focus on the more highly loaded interior walls; the end walls would be designed in a similar manner.

11.1.4.2.1 Load to Interior Transverse Walls. As computed in Section 11.1.3.3.2, the total story force resisted by the central walls is \(0.50F_x\). Since the both walls are the same length and material, each individual wall will resist one-half of the total or \(0.25F_x\). Therefore:

\[
\begin{align*}
F_{roof} &= 0.25(30.0) = 7.50 \text{ kips} \\
F_{3rd} &= 0.25(46.7) = 11.68 \text{ kips} \\
F_{2nd} &= 0.25(26.7) = 11.68 \text{ kips} \\
\Sigma &\approx 30.86 \text{ kips}
\end{align*}
\]

The story forces and story shears resisted by the individual wall segment is illustrated in Figure 11.1-6.

11.1.4.2.2 Roof to Third Floor

\[V = 7.50 \text{ kips}\]

\[v = 7.50/25 = 0.300 \text{ klf}\]

Try a 1/2-inch (15/32) plywood rated sheathing (not Structural I) on blocked 2× Hem-Fir members at 16 inches on center with 8d common nails at 6 inches on center at panel edges and 12 inches on center at intermediate framing members. From AF&PA SDPWS Table 4.3A, this shear wall assembly has a nominal unit shear capacity, \(v_u\), of 0.520 klf. However, according to Note 3 of AF&PA SDPWS Table 4.3A, the design shear resistance values are for Douglas Fir-Larch or Southern Pine and must be adjusted for Hem-Fir wall framing. The specific gravity adjustment factor equals 1-(0.5-SG) where SG is...
the specific gravity of the framing lumber. From AF&PA NDS Table 11.3.2A, the SG for Hem-Fir is 0.43. Therefore, the adjustment factor is $1-(0.5-0.43) = 0.93$. The adjusted shear capacity is computed as follows:

$$0.93 \phi D_v s = 0.93(0.8)(0.520) = 0.387 \text{ klf} > 0.300 \text{ klf}$$

OK

While 3/8- or 7/16-inch plywood could be used at this level, 1/2-inch is used for consistency with the lower floors.

11.1.4.2.3 Third Floor to Second Floor

$$V = 7.50 + 11.68 = 19.18 \text{ kips}$$

$$v = 19.18/25 = 0.767 \text{ klf}$$

Try 1/2-inch (15/32) plywood rated sheathing (not Structural I) on blocked 2× Hem-Fir members at 16 inches on center with 10d nails at 3 inches on center at panel edges and at 12 inches on center at intermediate framing members. From AF&PA SDPWS Table 4.3A, this shear wall assembly has a nominal unit shear capacity, $v_n$, of 1.200 klf. The adjusted shear capacity is computed as follows:

$$0.93 \phi D_v s = 0.93(0.8)(1.200) = 0.893 \text{ klf} > 0.767 \text{ klf}$$

OK

For this shear wall assembly, the width of framing at panel edges needs to be checked relative to AF&PA SDPWS Section 4.3.7.1. In accordance with Item 4 of that section, 3× framing is required at adjoining panel edges since the wall has 10d nails spaced at 3 inches or less and because the unit shear capacity exceeds 0.700 klf for a building assigned to Seismic Design Category D.

However, an exception to this section permits double 2× framing to be substituted for the 3× member, provided that the 2× framing is adequately stitched together. Since the double 2× framing is often preferred over the 3× member, this procedure will be utilized for this example. The exception requires the double 2× members to be connected to “transfer the induced shear between members.” For the purposes of this example, the induced shear along the vertical plane between adjacent panels will assumed to be equal to the adjusted design shear of 0.893 klf.

Using 16d common wire nails and 2× Hem-Fir framing, AF&PA NDS Table 11N specifies a lateral design value, $Z$, of 0.122 kips per nail. The adjusted design capacity is:

$$Z' = ZK_f \phi \lambda = (0.122)(2.16/0.65)(0.65) = 0.264 \text{ kips per nail}$$

and the number of nails per foot is $0.893/0.264 = 3.4$, so provide 4 nails per foot. Therefore, use double 2× framing at panel edges fastened with 16d at 3 inches on center and staggered (as required by the exception where the nail spacing is less than 4 inches).

11.1.4.2.4 Second Floor to First Floor

$$19.18 + 11.68 = 30.86 \text{ kips}$$

$$v = 30.86/25 = 1.236 \text{ klf}$$

Try 5/8-inch (19/32) plywood rated sheathing (not Structural I) on blocked 2-inch Hem-Fir members at 16 inches on center with 10d common nails at 2 inches on center at panel edges and 12 inches on center at
intermediate framing members. From AF&PA SDPWS Table 4.3A, this shear wall assembly has a nominal unit shear capacity, \( v_{u} \), of 1.740 klf. The adjusted shear capacity is computed as follows:

\[
0.93 \phi D v_s = 0.93(0.8)(1.740) = 1.294 \text{ klf} > 1.236 \text{ klf} \quad \text{OK}
\]

This shear wall assembly also requires 3× or stitched double 2× framing at panel edges. In this case, 3× framing is recommended, since the tight nail spacing required to stitch the double 2× members could lead to splitting and bolts or lag screws would not be economical.

Rather than increasing the plywood thickness at this level, adequate capacity could be achieved by using Douglas Fir-Larch framing members or using 1/2-inch plywood on both sides of the shear wall framing.

### 11.1.4.3 Transverse Shear Wall Anchorage.

AF&PA SDPWS Section 4.3.6.4.2 requires tie-down (hold-down) anchorage at the ends of shear walls where net uplift is induced. Net uplift is computed as the combination of the seismic overturning moment and the dead load counter-balancing moment using the load combination \( 0.72D - 1.0Q_E \).

The design requirements for the shear wall end posts and tie-downs have evolved over the past several code cycles. The 2008 AF&PA SDPWS requires the tie-down devices (Sec. 4.3.6.4.2) and end posts (Sec. 4.3.6.1.1) to be designed for a tension or compression force equal to the induced unit shear multiplied by the shear wall height. It can be inferred from AF&PA SDPWS Figure 4E, that the shear wall height, \( h \), refers to the sheathing height and not the story height, since the end post load is a function of the length of shear wall sheathing that engages the end post.

#### 11.1.4.3.1 Tie-down Anchors at Third Floor.

For the typical 25-foot interior wall segment, the overturning moment at the third floor is:

\[
M_0 = 9(7.50) = 67.5 \text{ ft-kip} = Q_E
\]

For the counter-balancing moment, it is assumed that the interior transverse walls will engage a certain length of exterior and corridor bearing wall for uplift resistance. The width of floor is taken as the length of solid wall panel at the exterior, or 10 feet. See Figures 11.1-1 and 11.1-13. For convenience, the same length is used for the longitudinal walls. The designer should take care to assume a reasonable amount of tributary dead loads that can be engaged considering the connections and stiffness of the cross wall elements. In this situation, considering that the exterior and corridor walls are plywood-sheathed shear walls, the assumption noted above is considered reasonable.

The weight of interior wall, 11 psf, is used for both conditions.

\[
\begin{align*}
\text{Shear wall self weight} &= (9 \text{ ft})(25 \text{ ft})(11 \text{ psf})/1,000 = 2.47 \text{ kips} \\
\text{Tributary floor} &= (10 \text{ ft})(25 \text{ ft})(15 \text{ psf})/1,000 = 3.75 \text{ kips} \\
\text{Tributary longitudinal walls} &= (9 \text{ ft})(10 \text{ ft})(11 \text{ psf})(2)/1,000 = 1.98 \text{ kips} \\
\Sigma &= 8.20 \text{ kips} \\
0.72Q_D &= 0.72(8.20)(12.5) = 73.8 \text{ ft-kip}
\end{align*}
\]

Since the dead load stabilizing moment exceeds the overturning moment, uplift anchorage is not required at the third floor. An end post for shear wall boundary compression is required, but since the design is similar to the second floor end post, it is not illustrated here.

#### 11.1.4.3.2 Tie-down Anchors at Second Floor
The overturning moment at the second floor is:

\[ M_0 = 18(7.50) + 9(11.68) = 240 \text{ ft}-\text{kip} \]

The counter-balancing moment is computed using the same assumptions as for the third floor.

\[
\begin{align*}
\text{Shear wall self weight} &= (18 \text{ ft})(25 \text{ ft})(11 \text{ psf})/1,000 = 4.95 \text{ kips} \\
\text{Tributary floor} &= (10 \text{ ft})(25 \text{ ft})(15 \text{ psf})(2)/1,000 = 7.50 \text{ kips} \\
\text{Tributary longitudinal walls} &= (18 \text{ ft})(10 \text{ ft})(11 \text{ psf})(2)/1,000 = 3.96 \text{ kips} \\
\Sigma &= 16.41 \text{ kips}
\end{align*}
\]

\[ 0.72Q_b = 0.72(16.41)(12.5) = 148 \text{ ft-kips} \]

\[ M_0 \text{ (net)} = 240 - 148 = 92 \text{ ft-kips} \]

As would be expected, uplift anchorage is required.

As described above, the design uplift force is computed using a unit shear demand of 0.768 klf at the second floor and a net length of wall height equal to 8 feet. Note that 8 feet is appropriate for this calculation given the detailing for this structure. As shown in Figure 11.1-10, the plywood sheathing is not detailed as continuous across the floor framing, which results in a net sheathing height of approximately 8 feet. If the sheathing were detailed across the floor framing, then 9 feet would be the appropriate wall height for use in computing tie-down demands. Since there is no net uplift force at the third floor, the third floor load need not be considered. Therefore, the design uplift force at the second floor is:

\[ T = 0.768 \text{ klf (8 ft)} = 6.14 \text{ kips} \]

Note that this uplift force exceeds the forces determined using the net overturning moment, which would be equal to 92 ft-kips / 25 ft = 3.68 kips, thus providing the intended added level of conservatism for the end posts and tie-downs.

Use a double tie-down anchor to connect the end posts. For ease of construction, select a tie-down device that screws to the end post. See Figure 11.1-7. A tie-down with a 5/8-inch threaded rod and fourteen 1/4-inch screws has a cataloged ASD capacity of 5.645 kips for Douglas Fir-Larch framing based on a load duration factor of 1.6. Using the AF&PA Guideline procedure for pre-engineered connections described in Section 11.1.4.1 \((K_F = 2.88/1.60)\), the LRFD capacity is determined as follows:

\[ ZK_F\phi\lambda = (5.645)(2.88/1.60)(0.65)(1.0) = 6.60 \text{ kips} > 6.14 \text{ kips} \]
11.4.3.3 Tie-down Anchors at First Floor. The overturning moment at the first floor is:

\[ M_0 = 27(7.50) + 18(11.68) + 9(11.68) = 517 \text{ ft-kip} = Q_E \]

The counter-balancing moment is computed using the same assumptions as for the second floor.

Shear wall self weight = (27 ft)(25 ft)(11 psf)/1,000 = 7.42 kips

Tributary floor = (10 ft)(25 ft)(15 psf)(3)/1,000 = 11.25 kips

Tributary longitudinal walls = (27 ft)(10 ft)(11 psf)(2)/1,000 = 5.94 kips

\[ 0.72Q_d = 0.72(24.61)12.5 = 222 \text{ ft-kip} \]

\[ M_0 \text{ (net)} = 517-222 = 295 \text{ ft-kip} \]

As expected, uplift anchorage is required. The design uplift force is computed using a unit shear force of 1.236 klf at the first floor and a net wall height of 8 feet. Combined with the uplift force at the second floor, the total design uplift force at the first floor is:

\[ T = 6.14 \text{ kips} + 1.236 \text{ klf (8 ft)} = 16.0 \text{ kips} \]

Use a double tie-down anchor that extends down into the foundation with an anchor bolt. Tie-downs with a 7/8-inch threaded rod anchor and three 1-inch bolts through a 6\(\times\)6 Douglas Fir-Larch end post have a
cataloged capacity of 12.1 kips based on a load duration factor of 1.6. The LRFD capacity of two tie downs is computed as follows:

\[ 2ZK_f\phi\lambda = 2(12.1)(2.88/1.60)(0.65)(1.0) = 28.3 \text{ kips} > 16.0 \text{ kips} \quad \text{OK} \]

Next, check the LRFD capacity of the bolts in double shear. For the three bolts, the AF&PA NDS gives the following equation:

\[ 3ZK_f\phi\lambda = 3(5.50)(2.16/0.65)(0.65)(1.0) = 35.6 \text{ kips} > 16.0 \text{ kips} \quad \text{OK} \]

The strength of the end post, based on failure across the net section, must also be checked. A reasonable approach to preclude net tension failure from being a limit state would be to provide an end post whose nominal resistance exceeds the nominal strength of the tie-down device. The nominal strength of the first-floor double tie-down is 28.3/0.65 = 43.5 kips. Therefore, the nominal tension capacity at the net section should be greater than 43.5 kips.

Try a 6×6 Douglas Fir-Larch No. 1 end post. Accounting for 1-1/16-inch bolt holes, the net area of the post is 24.4 in². Using \( \phi = 1.0 \) for nominal strength, according to the AF&PA NDS Supplement:

\[ F' = F_cK_f\phi\lambda = (0.825)(2.16/0.8)(1.0)(1.0) = 2.228 \text{ ksi} \]

\[ T' = F'A = 2.228(24.4) = 44.5 \text{ kips} > 54.4 \text{ kips} \quad \text{OK} \]

Not shown here but for a group of bolts, the row and/or group tear-out capacity must be checked for all bolted connections with multiple fasteners. Refer to AF&PA NDS Section 10.1.2 and Appendix E.

For the maximum compressive load at the end post, combine the maximum gravity load plus the seismic overturning load. However, since the exterior and interior longitudinal walls are load-bearing stud walls, the gravity load demand on the shear wall end post is minimal.

Therefore, without any significant gravity load, the compression force on the end post is the same as the tension force per AF&PA SDPWS Section 4.3.6.1.1 and equal to 16.0 kips at the first floor.

Due to the relatively short clear height of the post, the governing condition is bearing perpendicular to the grain on the bottom plate. Check the bearing of the 6×6 end post on a 3×6 Douglas Fir-Larch No. 2 plate, per the AF&PA NDS Supplement:

\[ F'_{\perp} = F_c\phi\lambda = (0.625)(1.875/0.9)(0.9)(1.0) = 1.17 \text{ ksi} \]

\[ C' = F'_{\perp}A = 1.17(5.5)(5.5) = 35.4 \text{ kips} > 16.0 \text{ kips} \quad \text{OK} \]

**11.1.4.3.4 Check Overturning at the Soil Interface.** A summary of the overturning forces is shown in Figure 11.1-8. To compute the overturning at the soil interface, the overturning moment must be increased for the 4-foot foundation height:

\[ M_0 = 517 + 30.9(4.0) = 640 \text{ ft-kip} \]
However, it then may be reduced in accordance with Standard Section 12.14.8.4:

\[ M_0 = 0.75(640) = 480 \text{ ft-kip} \]

To determine the total resistance, combine the weight above with the dead load of the first floor and foundation.

Load from first floor = \((25 \text{ ft})(10 \text{ ft})(20-4+1) \text{ psf} / 1,000 = 4.25 \text{ kips}\)

where 4 psf is the weight reduction due to the absence of a ceiling and 1 psf is the weight of insulation.

The length of the longitudinal foundation wall included is a conservative approximation of the amount that can be engaged assuming minimum nominal reinforcement in the foundation.

Foundation weight = \((690 \text{ plf}[10 \text{ ft} + 10 \text{ ft} + 25 \text{ ft}]) / 1,000 = 31.05 \text{ kips}\)
First floor = 4.25 kips
Structure above = 24.61 kips
\( \Sigma \) = 59.91 kips

Therefore, \(0.72D - 1.0Q_e = 0.72(59.91)(12.5 \text{ ft}) - 1.0(480) = 59.2 \text{ ft-kips}\), which is greater than zero, so the overturning check is acceptable.

11.1.4.3.5 Anchor Bolts for Shear. At the first floor, the unit shear demand, \(v\), is 1.236 klf.

Try 5/8-inch bolts in a 3×6 Douglas Fir-Larch sill plate, in single shear, parallel to the grain. In accordance with the AF&PA NDS:

\[ ZK_F \phi \lambda = (1.11)(2.16/0.65)(0.65)(1.0) = 2.40 \text{ kips per bolt}\]

The required bolt spacing is \(2.4/(1.236/12) = 23.3 \text{ in}\). Therefore, provide 5/8-inch bolts at 16 inches on center to match the joist layout.

AF&PA SDPWS Section 4.3.6.4.3 requires plate washers at all shear wall anchor bolts and where the nominal unit shear capacity exceeds 400 plf, the plate washer needs to extend within 1/2 inch of the edge of the plate on the side with the sheathing, so provide 4.5-inch-square plate washers.
Note that in addition to the capacity of the bolt in the wood sill, the bolt capacity in the concrete foundation wall should be checked based on ACI 318 Appendix D.

**11.1.4.6 Remarks on Shear Wall Connection Details.** In typical platform frame construction, details must be developed that will transfer the lateral loads through the floor system and, at the same time, accommodate normal material sizes and the cross-grain shrinkage in the floor system. The connections for wall overturning in Section 11.1.4.5 are an example of one of the necessary force transfers. The transfer of diaphragm shear to supporting shear walls is another important transfer, as is the transfer from a shear wall on one level to the level below.

The floor-to-floor height is 9 feet with approximately 1 foot occupied by the floor framing. Using standard 8-foot-long plywood sheets for the shear walls, a gap occurs over the depth of the floor framing. It is common to use the floor framing to transfer the lateral shear force. Figures 11.1-9 and 11.1-10 depict this accomplished by nailing the plywood to the bottom plate of the shear wall, which is nailed through the floor plywood to the double 2×12 chord in the floor system.

![Diagram of shear wall connection](image)

*Figure 11.1-9 Bearing wall (1.0 in = 25.4 mm)*
The top plate of the lower shear wall also is connected to the double 2×12 by means of sheet metal framing clips to the double 2×12 to transfer the force back out to the lower plywood. (Where the forces are small, toe nails between the double 2×12 and the top plate may be used for this connection.) This technique leaves the floor framing free for cross-grain shrinkage.

The floor plywood is nailed directly to the framing at the edge of the floor, before the plate for the upper wall is placed. Also, the floor diaphragm is connected directly to framing that spans over the openings between shear walls. The axial strength and the connections of the double 2×12 chords, allows them to function as collectors to move the force from the full length of the diaphragm to the discrete shear walls. (According to Standard Sec. 12.14.7.3, the design of collector elements in wood shear wall buildings need not consider increased seismic demands due to overstrength.)

The floor joist is toe nailed to the wall below for forces normal to the wall. Likewise, full-depth blocking is provided adjacent to walls that are parallel to the floor joists, as shown in Figure 11.1-10. (Elsewhere, the blocking for the floor diaphragm only need be small pieces, flat 2×4s for example.) The connections at the foundation are similar (see Figure 11.1-11).

Figure 11.1-10 Nonbearing wall
(1.0 in. = 25.4 mm)
11.1.4.5 Roof Diaphragm. While it has been common practice to design plywood diaphragms as simply supported beams spanning between shear walls, the diaphragm design for this example will consider some continuity at the central shear walls. The design will be based on the shears associated with the tributary area distribution of force to the shear walls and will account for moments at the diaphragm midspan as well as the central shear walls. (Note that this diaphragm assumption would result in a slightly different distribution of lateral loads to the shear walls, which is not accounted for in this example.)

From Section 11.1.3.4, the diaphragm design force at the roof is the same as the roof story force, so $F_{p,\text{roof}} = 30.0$ kips.
As discussed previously, the design force computed in this example includes the internal force due to the weight of the walls parallel to the motion. Particularly for one-story buildings, it is common practice to remove that portion of the design force. It is conservative to include it, as is done here.

11.1.4.5.1 Diaphragm Nailing. Idealizing the building as a two-span diaphragm with three sets of walls as described previously, the maximum diaphragm shear occurs at the ends of the 88-foot diaphragm span. Assuming a uniform distribution of the diaphragm force across the building, the maximum shear over the entire diaphragm width is computed as follows:

\[ V = \frac{(30.0)(88/148)}{2} = 8.93 \text{ kips} \]

\[ v = \frac{8.93}{56 \text{ ft}} = 0.160 \text{ klf} \]

Try 1/2-inch (15/32) plywood rated sheathing (not Structural I) on blocked 2-inch Douglas Fir-Larch members at 16 inch on center, with 8d nails at 6 inches on center at all boundaries and panel edges and 12 inches on center at intermediate framing members. From AF&PA SDPWS Table 4.2A, this diaphragm assembly has a nominal unit shear capacity, \( v_s \), of 0.540 klf. The adjusted shear capacity is computed as follows:

\[ \phi_Dv_s = 0.8(0.540) = 0.432 \text{ klf} > 0.160 \text{ klf} \quad \text{OK} \]

11.1.4.5.2 Chord and Splice Connection. Diaphragm continuity is an important factor in the design of the chords. The design must consider the tension/compression forces, due to positive moment at the middle of the span as well as negative moment at the interior shear wall. It is reasonable (and conservative) to design the chord for the positive moment assuming a simply supported beam and for the negative moment accounting for continuity. The positive moment is \( wl^2/8 \), where \( w \) is the unit diaphragm force and \( l \) is the length of the governing diaphragm span. For a continuous beam of two unequal spans, under a uniform load, the maximum negative moment is:

\[ M^{-} = \frac{wl_1^3 + wl_2^3}{8(l_1 + l_2)} \]

where \( w \) is the unit diaphragm force and \( l_1 \) and \( l_2 \) are the lengths of the two diaphragm spans. For \( w = 30.0 \text{ kips} / 148 \text{ ft} = 0.203 \text{ klf} \), the maximum positive moment is:

\[ 0.203(88)^2 / 8 = 197 \text{ ft-kip} \]

and the maximum negative moment is:

\[ \frac{0.203(88)^3 + 0.203(60)^3}{8(88 + 60)} = 154 \text{ ft-kips} \]

The positive moment controls and the design chord force is 197/56 = 3.51 kips. Try a double 2\times12 Douglas Fir-Larch No. 2 chord. Due to staggered splices, compute the tension capacity based on a single 2\times12, with an area of \( A_n = 16.88 \text{ in}^2 \). According to the AF&PA NDS Supplement:

\[ F_i' = F_iK_x\phi\lambda = (0.575)(2.16/0.8)(0.8)(1.0) = 1.537 \text{ ksi} \]

\[ T' = F_i'A = 1.537(16.88) = 25.9 \text{ kips} > 3.51 \text{ kips} \quad \text{OK} \]
For chord splices, use 16d nails in the staggered chord members. According to the AF&PA NDS, the capacity of one 16d common wire nail in single shear with two 2× Douglas Fir-Larch members is 0.141 kips. The adjusted strength per nail is:

\[ Z' = Z_K \phi \lambda = (0.141)(2.16/0.65)(0.65)(1.0) = 0.305 \text{ kips} \]

The number of required nails at the splice is \( 3.51/0.305 = 11.5 \), so use twelve 16d nails. Assuming a 4-foot splice length, provide two rows of six 16d nails at 8 inches on center. A typical chord splice connection is shown in Figure 11.1-12.

**Figure 11.1-12** Diaphragm chord splice

(1.0 ft = 0.3048 m, 1.0 in. = 25.4 mm)

### 11.1.4.6 Second- and Third-Floor Diaphragm

The design of the second- and third-floor diaphragms follows the same procedure as for the roof diaphragm. From Section 10.1.3.4, the diaphragm design force for both floors is

\[ F_{p,3rd} = F_{p,2nd} = 46.7 \text{ kips} \]

#### 11.1.4.6.1 Diaphragm Nailing

The maximum diaphragm shear is computed as follows:

\[ V = (46.7)(88/148)/2 = 13.88 \text{ kips} \]

\[ v = 13.88 / 56 \text{ ft} = 0.248 \text{ klf} \]

With an adjusted capacity of 0.432 klf, the same diaphragm as at the roof also works for the floors.
11.1.4.6.2 Chord and Splice Connection. Computed as described above for the roof diaphragm, the maximum positive moment is 306 ft-kips and the design chord force is 5.48 kips.

By inspection, a double 2×12 chord spliced with 16d nails similar to the roof level is adequate. The number of nails at the floors is 5.48/0.305 = 18 nails, so for the 4-foot splice length, provide two rows of nine 16d nails at 5 inches on center on each side of the splice joint.

11.1.4.7 Longitudinal Direction. Only one exterior shear wall section will be designed here. The design of the corridor shear walls would be similar to that of the transverse walls. For loads in the longitudinal direction, diaphragm stresses are negligible and the nailing provided for the transverse direction is more than adequate.

The design of the exterior wall utilizes the provisions for perforated shear walls as defined in AF&PA SDPWS Sections 4.3.4.1 and 4.3.5.3. The procedure for perforated shear walls applies to walls with openings that have not been specifically designed and detailed for force transfer around the openings. Essentially, a perforated wall is treated in its entirety rather than as a series of discrete wall piers. The use of this design procedure is limited by several conditions as specified in AF&PA SDPWS Section 4.3.5.3.

The main aspects of the perforated shear wall design procedure are as follows. The design shear capacity of the shear wall is the sum of the capacities of each segment (all segments must have the same sheathing and nailing) reduced by an adjustment factor that accounts for the geometry of the openings. Uplift anchorage (tie-down) is required only at the ends of the wall (not at the ends of all wall segments), but all wall segments must resist a specified tension force (using anchor bolts at the foundation and strapping or other means at upper floors). Requirements for shear anchorage and collectors (drag struts) across the openings are also specified. It should be taken into account that the design capacity of a perforated shear wall is less than that of a standard segmented wall with all segments restrained against overturning. However, the procedure is useful in eliminating interior hold downs for specific conditions and thus is illustrated in this example.

The portion of the story force resisted by each exterior wall was computed previously as 0.223Fx. The exterior shear walls are composed of three separate perforated shear wall segments (two at 30 feet long and one at 15 feet long, all with the same relative length of full-height sheathing), as shown in Figure 11.1-2. This section will focus on the design of a 30-foot section. Assuming that load is distributed to the wall sections based on relative length of the shear panel, then the total story force to the 30-foot section is (30/75)0.223Fx = 0.089Fx per floor. The load per floor is:

\[
F_{\text{roof}} = 0.089(30.0) = 2.67 \text{ kips}
\]
\[
F_{3\text{rd}} = 0.089(46.7) = 4.16 \text{ kips}
\]
\[
F_{2\text{nd}} = 0.089(46.7) = 4.16 \text{ kips}
\]
\[
\Sigma = 10.99 \text{ kips}
\]

11.1.4.7.1 Perforated Shear Wall Resistance. The design shear capacity for perforated shear walls is computed as the factored shear resistance for the sum of the wall segments, multiplied by an adjustment factor that accounts for the percentage of full-height (solid) sheathing and the ratio of the maximum opening to the story height as described in AF&PA SDPWS Section 4.3.3.5. At each level, the design shear capacity, \( V_{\text{wall}} \), is:

\[
V_{\text{wall}} = (vC_{0})\Sigma L_i
\]

where:
\( v \) = factored shear resistance (AF&PA SDPWS Table 4.3A)
\( C_0 \) = shear capacity adjustment factor (AF&PA SDPWS Table 4.3.3.5)
\( \Sigma L_i \) = sum of shear wall segment lengths

For the subject wall, the widths of perforated shear wall segments are \( 4 + 10 + 4 = 18 \) feet, the percent of full-height sheathing is \( 18/30 = 0.60 \) and the maximum opening height is 4 feet. Therefore, per AF&PA SDPWS Table 4.3.3.5, \( C_0 = 0.83 \).

**Figure 11.1-13** Perforated shear wall at exterior
(1.0 ft = 0.3048 m, 1.0 in. = 25.4 mm)

The wall geometry (and thus the adjustment factor and total length of wall segments) is the same at all three levels, as shown in Figure 11.1-13. Perforated shear wall plywood and nailing are determined below.

- Roof to third floor:
  \[ V' = 2.67 \text{ kips} \]
  Required \( v = 2.67/0.83/18 = 0.179 \text{ klf} \)
  Try 1/2-inch (15/32) plywood rated sheathing (not Structural I) on blocked 2× Hem-Fir members at 16 inches on center with 8d nails at 6 inches on center at panel edges and at 12 inches on center at
intermediate framing members. From AF&PA SDPWS Table 4.3A, this shear wall assembly has a nominal unit shear capacity, \( v_s \), of 0.520 klf. Adjusting for framing material, the shear capacity is computed as follows:

\[
0.93 \phi_D v_s = 0.93(0.8)(0.520) = 0.387 \text{ klf} > 0.179 \text{ klf}
\]

- Third floor to second floor:

\[
V = 2.67 + 4.16 = 6.83 \text{ kips}
\]

Required \( v = 6.83 / 0.83 / 18 = 0.457 \text{ klf} \)

Try 1/2-inch (15/32) plywood rated sheathing (not Structural I) on blocked 2\( \times \) Hem-Fir members at 16 inches on center with 8d nails at 4 inches on center at panel edges and at 12 inches on center at intermediate framing members. From AF&PA SDPWS Table 4.3A, this shear wall assembly has a nominal unit shear capacity, \( v_s \), of 0.760 klf. Adjusting for framing material, the shear capacity is computed as follows:

\[
0.93 \phi_D v_s = 0.93(0.8)(0.760) = 0.565 \text{ klf} > 0.457 \text{ klf}
\]

- Second floor to first floor:

\[
V = 6.83 + 4.16 = 10.99 \text{ kips}
\]

Required \( v = 10.99 / 0.83 / 18 = 0.735 \text{ klf} \)

Try 1/2-inch (15/32) plywood rated sheathing (not Structural I) on blocked 2\( \times \) Hem-Fir members at 16 inches on center with 10d nails at 3 inch on center at panel edges and at 12 inch on center at intermediate framing members. From AF&PA SDPWS Table 4.3A, this shear wall assembly has a nominal unit shear capacity, \( v_s \), of 1.200 klf. Adjusting for framing material, the shear capacity is computed as follows:

\[
0.93 \phi_D v_s = 0.93(0.8)(1.200) = 0.893 \text{ klf} > 0.735 \text{ klf}
\]

Note that the nominal unit shear capacity of 1.200 klf is less than the maximum permitted nominal shear capacity of 1.740 klf in accordance with AF&PA SDPWS Section 4.3.5.3, Item 3.

11.1.4.7.2 Perforated Shear Wall Tension Chord. According to AF&PA SDPWS Section 4.3.6.1.2, tension and compression chords and associated anchorage must be evaluated at the ends of the wall only. Uplift anchorage at each wall segment is treated separately as described later. The tension and compression forces at the wall ends are determined per AF&PA SDPWS Equation 4.3-8 as follows:

\[
T = C = \frac{V h}{C_0 \sum L_i}
\]

where:

\( V \) = design shear force in the shear wall
\( h \) = shear wall height (per floor)
\( C_0 \) = shear capacity adjustment factor
\( \Sigma L_i \) = sum of widths of perforated shear wall segments
For this example, the tension chord and tie-down will be designed at the first floor only; the other floors would be computed similarly and tie-down devices, as shown in Figure 11.1-7, would be used. For $h = 8$ ft, $C_0 = 0.83$ and $\Sigma L_i = 18$ ft, the tension force is computed as follows:

Third floor: $T = \frac{2.67(8)}{(0.83 \times 18)} = 1.42$ kips
Second floor: $T = \frac{(2.67 + 4.16)(8)}{(0.83 \times 18)} = 3.66$ kips
First floor: $T = \frac{(2.67 + 4.16 + 4.16)(8)}{(0.83 \times 18)} = 6.42$ kips
$\Sigma = 11.50$ kips

For the dead load to resist the tension chord uplift, assume a tributary floor width equal to the half the span of the window header at the end wall segment. The tributary width is 6.5 feet and the tributary joist span is 8 feet. The tributary weight is computed as follows:

Exterior wall weight = $(27$ ft$)(6.5$ ft$)(9$ psf$)/1,000 = 1.58$ kips
Tributary roof = $(8$ ft$)(6.5$ ft$)(15$ psf$)/1,000 = 0.78$ kips
Tributary floor = $(8$ ft$)(6.5$ ft$)(20$ psf$)(2)/1,000 = 2.08$ kips
$\Sigma = 4.44$ kips

The net uplift is computed as follows:

$0.72D + 1.0E = 0.72(4.44) - 11.5 = 8.30$ kips

Therefore, uplift anchorage is required per AF&PA SDPWS Section 4.3.6.4.2. Since the chord member resists the perforated shear wall compression load and supports the window header as well, use a 6$\times$6 Douglas Fir-Larch No. 1, similar to the transverse walls. The post has ample tension capacity. For the anchorage, try a tie-down device with a 7/8-inch anchor bolt and twenty 1/4-inch screws into the post. Using the method described above for computing the strength of a pre-engineered tie-down, the capacity is computed as follows:

$ZK_F \phi \lambda = (7.87)(2.88/1.60)(0.65)(1.0) = 9.2$ kips $> 8.3$ kips OK

The design of the tie-downs at the second and third floors is similar.

**11.1.4.7.3 Perforated Shear Wall Compression Chord.** The force in the compression chord is the same as the tension chord equal to 11.5 kips at the first floor. Again, just the chord at the first floor will be designed here; the design at the upper floors would be similar. Although not explicitly required by AF&PA SDPWS Section 4.3.6.1.2, it is rational to combine the chord compression with gravity loading, using the load combination $1.4D + 1.0Q_E + 0.5L + 0.2S$, in order to design the chord member. The dead load is as computed above and the live load and snow load are 4.16 kips and 1.30 kips, respectively. Therefore, the design compression force is as follows:

$1.38(4.44) + 1.0(11.5) + 0.5(4.16) + 0.2(1.30) = 20.0$ kips

The bearing capacity on the bottom plate was computed previously as 35.4 kips, which is greater than 20.0 kips. Note that where end posts are loaded in both directions, orthogonal effects must be considered in accordance with *Standard* Section 12.5.

**11.1.4.7.4 Anchorage at Shear Wall Segments.** The anchorage at the base of a shear wall segment (bottom plate to floor framing or foundation wall) is designed per AF&PA SDPWS Section 4.3.6.4. This section requires two types of anchorage: in-plane shear anchorage (AF&PA SDPWS Sec. 4.3.6.4.1.1)
and distributed uplift anchorage (AF&PA SDPWS Sec. 4.3.6.4.2). While both types of anchorage need only be provided at the full-height sheathing, the shear anchorage is usually extended at least over the entire length of the perforated shear wall to simplify the detailing and reduce the possibility of construction errors.

The in-plane shear anchorage is required to resist the following:

\[ v = \frac{V}{C_0 \sum L_i} \]

where:

- \( V \) = design shear force in the shear wall
- \( C_0 \) = shear capacity adjustment factor
- \( \sum L_i \) = sum of widths of perforated shear wall segments

This equation is the same as was previously used to compute unit shear demand on the wall segments. Therefore, the in-plane anchorage will be designed to meet the following unit, in-plane shear forces:

- **Third floor**: \( v = 0.179 \text{ klf} \)
- **Second floor**: \( v = 0.457 \text{ klf} \)
- **First floor**: \( v = 0.735 \text{ klf} \)

The required distributed uplift force, \( t \), is equal to the in-plane shear force, \( v \). Per AF&PA SDPWS Section 4.3.6.4, this uplift force must be provided with a complete load path to the foundation. That is, the uplift force at each level must be combined with the uplift forces at the levels above (similar to the way overturning moments are accumulated down the building).

At the foundation level, the unit in-plane shear force, \( v \) and the unit uplift force, \( t \), are combined for the design of the bottom plate anchorage to the foundation wall. The design unit forces are as follows:

- **Shear**: \( v = 0.735 \text{ klf} \)
- **Tension**: \( t = 0.179 + 0.457 + 0.735 = 1.371 \text{ klf} \)

Assuming that stresses on the wood bottom plate govern the design of the anchor bolts, the anchorage is designed for shear (single shear, wood-to-concrete connection) and tension (plate washer bearing on bottom plate). The interaction between shear and tension need not be considered in the wood design for this configuration of loading.

Try a 5/8-inch bolt at 32 inches on center with a 4.5-inch square plate washer (AF&PA SDPWS Section 4.3.6.4.3 requires plate washer to extend within 1/2 inch of the 5.5-inch-wide bottom plate). As computed previously, the shear capacity of a 5/8-inch bolt in a 3×6 Douglas Fir-Larch sill plate is 2.40 kips. The demand per bolt is 0.735 klf \((32/12) = 1.96 \text{ kips} \), so the 32-inch spacing is adequate for shear.
For anchor bolts at 32 inches on center, the tension demand per bolt is 1.371 klf (32/12) = 3.66 kips. Bearing capacity of the plate washer (using a Douglas Fir No. 2 bottom plate) is computed per AF&PA NDS Supplement as follows:

\[
F'_{c,v} = F_{c,v}K_{f}F_{v} = (0.625)(1.875/0.9)(0.9)(1.0) = 1.17 \text{ ksi}
\]

\[
C' = F'_{c,v}A = 1.17(4.5)(4.5) = 23.7 \text{ kips} > 3.66 \text{ kips} \quad \text{OK}
\]

The anchor bolts themselves must be designed for combined shear and tension in accordance with ACI 318-08.

In addition to designing the anchor bolts for uplift, a positive load path must be provided to transfer the uplift forces into the bottom plate. One method for providing this load path continuity is to use metal straps nailed to the studs and lapped around the bottom plate, as shown in Figure 11.1-14. Attaching the studs directly to the foundation wall (using embedded metal straps) for uplift and using the anchor bolts for shear only is an alternative approach.

Figure 11.1-14 Perforated shear wall detail at foundation  
(1.0 ft = 0.3048 m, 1.0 in. = 25.4 mm)

At the upper floors, the load transfer for in-plane shear is accomplished by using nailing or framing clips between the bottom plates, rim joists and top plates in a manner similar to that for standard shear walls. The uniform uplift force can be resisted either by using the nails in withdrawal (for small uplift demand) or by providing vertical metal strapping between studs above and below the level considered. This type of connection is shown in Figure 11.1-15. For this type of connection (and the one shown in
Figure 11.1-14) to be effective, shrinkage of the floor framing must be minimized using dry or manufactured lumber.

![Diagram of floor framing](image)

**Figure 11.1-15 Perforated shear wall detail at floor framing**

For example, consider the second floor. The required uniform uplift force, $t = 0.244 + 0.496 = 0.740$ klf. Place straps at every other stud, so the required strap force is $0.740(32/12) = 1.97$ kips. Provide an 18-gauge strap with twelve 10d nails at each end.

### 11.2 WAREHOUSE WITH MASONRY WALLS AND WOOD ROOF, LOS ANGELES, CALIFORNIA

This example features the design of the wood roof diaphragm and wall-to-diaphragm anchorage for the one-story masonry building described in Section 10.1 of this volume of design examples. Refer to that example for more detailed building information and the design of the masonry walls.

#### 11.2.1 Building Description

This is a very simple rectangular warehouse, 100 feet by 200 feet in plan (see Figure 11.2-1), with a roof height of 28 feet. The wood roof structure slopes slightly, but it is nominally flat. The long walls (side walls) are 8 inches thick and solid and the shorter end walls are 12 inches thick and penetrated by several large openings.
Based on gravity loading requirements, the roof structure consists of wood joists, supported by 8-3/4-inch-wide by 24-inch-deep glued-laminated timber beams on steel columns. The joists span 20 feet and the beams span 40 feet, as an articulated system. Typical roof framing is assumed to be Douglas Fir-Larch No 1 as graded by the WWPA Rules. The glued-laminated timber beams meet the requirements of Combination 24F-V4 per AITC A190.1.

The plywood roof deck acts as a diaphragm to carry lateral loads to the exterior walls. There are no interior walls for seismic resistance. The roof contains a large opening that interrupts the diaphragm continuity. The diaphragm contains continuous cross ties in both principal directions that serve as part of the wall anchorage system.

The following aspects of the structural design are considered in this example:

- Development of diaphragm forces based on the Equivalent Lateral-Force Procedure used for the masonry wall design (Sec. 10.1)
- Design and detailing of a plywood roof diaphragm with a significant opening
- Computation of drift and P-delta effects
- Anchorage of diaphragm and roof joists to masonry walls
- Design of cross ties and subdiaphragms

11.2.2 Basic Requirements
11.2.2.1 Seismic Parameters
Table 11.2-1  Seismic Parameters

<table>
<thead>
<tr>
<th>Design Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$S_S$</td>
<td>1.50</td>
</tr>
<tr>
<td>$S_I$</td>
<td>0.60</td>
</tr>
<tr>
<td>Site Class (Standard Sec. 11.4.2)</td>
<td>C</td>
</tr>
<tr>
<td>Occupancy Category (Standard Sec. 1.5.1)</td>
<td>II</td>
</tr>
<tr>
<td>Seismic Design Category (Standard Sec. 11.6)</td>
<td>D</td>
</tr>
<tr>
<td>Seismic Force-Resisting System (Standard Table 12.2-1)</td>
<td>Special Reinforced Masonry Shear Walls</td>
</tr>
<tr>
<td>Response Modification Factor, $R$</td>
<td>5</td>
</tr>
<tr>
<td>System Overstrength Factor, $\Omega_0$</td>
<td>2.5</td>
</tr>
<tr>
<td>Deflection Amplification Factor, $C_d$</td>
<td>3.5</td>
</tr>
</tbody>
</table>

11.2.2.2 Structural Design Criteria. A complete discussion on the criteria for ground motion, seismic design category, load path, structural configuration, redundancy, analysis procedure and shear wall design is included in Section 10.1 of this volume of design examples.

11.2.2.2.1 Design and Detailing Requirements. Since this building has a wood structural panel diaphragm with masonry shear walls, the diaphragm can be considered flexible in accordance with Standard Section 12.3.1.1. There are not any irregularities (Standard Sec. 12.3.2) that would impact the diaphragm design and the diaphragm and wall anchorage system is permitted to be designed with the redundancy factor equal to 1.0 per Standard Section 12.3.4.1.

The design of the diaphragm is based on Standard Section 12.10. The large opening in the diaphragm must be fitted with edge reinforcement (Standard Sec. 12.10.1). However, the diaphragm does not require any collector elements that would have to be designed for the special load combinations (Standard Sec. 12.10.2.1).

The requirements for anchorage of masonry walls to flexible diaphragms (Standard Sec. 12.11.2) are of great significance in this example.

11.2.2.2.2 Seismic Load Effects and Combinations. The basic design load combinations for the seismic design, as stipulated in Standard Section 12.4.2.3, were computed in Section 10.1 of this volume of design examples, as follows:

$$1.4D + 1.0Q_E$$

where gravity and earthquake are additive and

$$0.7D - 1.0Q_E$$

where gravity and earthquake counteract.

The roof live load, $L_r$, is not combined with seismic loads (see Standard Chapter 2) and the design snow load is zero for this Los Angeles location.
11.2.2.2.3 Deflection and Drift Limits. In-plane deflection and drift limits for the masonry shear walls are considered in Section 10.1.

As illustrated below, the diaphragm deflection is much greater than the shear wall deflection. According to Standard Section 12.12.2, in-plane diaphragm deflection must not exceed the permissible deflection of the attached elements. Because the walls are essentially pinned at the base and simply supported at the roof, they are capable of accommodating large deflections at the roof diaphragm.

For illustrative purposes, story drift is determined and compared to the requirements of Standard Table 12.12-1. However, according to this table, there is essentially no drift limit for a single-story structure as long as the architectural elements can accommodate the drift (assumed to be likely in a warehouse structure with no interior partitions). As a further check on the deflection, P-delta effects (Standard Sec. 12.8.7) are evaluated.

11.2.3 Seismic Force Analysis

Building weights and base shears are as computed in Section 10.1. (The building weights used in this example are based on a preliminary version of Example 10.1 and thus minor numerical differences may exist between the two examples). Standard Section 12.10.1.1 specifies that floor and roof diaphragms be designed to resist a force, \( F_{px} \), computed in accordance with Standard Equation 12.10-1 as follows:

\[
F_{px} = \frac{\sum_{i=1}^{n} F_i}{\sum_{i=1}^{n} W_{i}} w_{px}
\]

plus any force due to offset walls (not applicable for this example). For one-story buildings, the first term of this equation will be equal to the seismic response coefficient, \( C_s \), which is 0.286. The effective diaphragm weight, \( w_{px} \), is equal to the weight of the roof plus the tributary weight of the walls perpendicular to the direction of the motion. The tributary weights are as follows:

- Roof = 20(100)(200) = 400 kips
- Side walls = 2(65)(28/2+2)(200) = 416 kips
- End walls = 2(103)(28/2+2)(100) = 330 kips

The diaphragm design force is computed as:

- Transverse: \( F_{p,roof} = 0.286(400+416) = 233 \) kips
- Longitudinal: \( F_{p,roof} = 0.286(400+330) = 209 \) kips

These forces exceed the minimum diaphragm design forces given in Standard Section 12.10.1.1, because \( C_s \) exceeds the minimum factor of 0.2S_{DS}. 
11.2.4 Basic Proportioning of Diaphragm Elements

The design of plywood diaphragms primarily involves the determination of sheathing sizes and nailing patterns to accommodate the applied loads. Large openings in the diaphragm and wall anchorage requirements, however, can place special requirements on the diaphragm capacity. Diaphragm deflection is also a consideration.

Nailing patterns for diaphragms are established on the basis of tabulated requirements included in the AF&PA SDPWS. It is important to consider the framing requirements for a given nailing pattern and capacity as indicated in the notes following the tables. In addition to strength requirements, AF&PA SDPWS Section 4.2.4 places aspect ratio limits on plywood diaphragms (length-to-width must not exceed 4/1 for blocked diaphragms). However, it should be taken into consideration that compliance with this aspect ratio does not guarantee that drift limits will be satisfied.

While there is no specific limitation on deflection for this example, the diaphragm has been analyzed for deflection as well as for shear capacity.

In the calculation of diaphragm deflections, the chord splice slip factor can result in large additions to the total deflection. This chord splice slip, however, is often negligible where the diaphragm is continuously anchored to a bond beam in a masonry wall. Therefore, chord splice slip is assumed to be zero in this example.

11.2.4.1 Strength of Members and Connections. As described in more detail in Section 11.1.4.1, the Standard references the AF&PA NDS and AF&PA SDPWS for engineered wood structures. Diaphragm design is based on AF&PA SDPWS Section 4.2, which provides design criteria for both ASD and LRFD methods. This example utilizes LRFD as the design basis, so the diaphragm design is based on the tabulated unit shear values, $v_s$, which are multiplied by a resistance factor, $\phi_D$, equal to 0.80.

Refer to Section 11.1.4.1 for a summary of the design methodology in the AF&PA NDS for framing members and connections.

11.2.4.2 Roof Diaphragm Design for Transverse Direction

11.2.4.2.1 Plywood and Nailing. The diaphragm design force is $F_{p,roof} = 233$ kips and the maximum end shear is $0.5F_{p,roof} = 116.5$ kips. This corresponds to a unit shear force of $v = (116.5/100) = 1.165$ klf. (Note that per Standard Sec. 12.8.4.2, accidental torsion need not be considered for flexible diaphragms.)

Due to the relatively high diaphragm shears, closely spaced nailing will be required, so in accordance with AF&PA SDPWS Section 4.2.7.1.1, Item 3, 3-inch nominal framing will be provided. Assuming 3-inch nominal framing, try blocked 1/2-inch (15/32) Structural I plywood rated sheathing with 10d common nails at 2 inches on center at diaphragm boundaries and continuous panel edges and at 3 inches on center at other panel edges. The use of 2×4 flat blocking at continuous panel edges satisfies the requirements for blocked diaphragms. From AF&PA SDPWS Table 4.2A:

$$\phi_D v_s = 0.80(1.640) = 1.31 \text{ klf} > 1.165 \text{ klf}$$

Because the diaphragm shear decreases towards the midspan of the diaphragm, the diaphragm capacity may be reduced towards the center of the building. A reasonable configuration for the interior of the building utilizes 2-inch nominal framing and 1/2-inch (15/32) Structural I plywood rated sheathing plywood with 10d at 4 inches on center at diaphragm boundaries and continuous panels edges and
6 inches on center nailing at other panel edges. Determine the distance, $X$, from the end wall where the transition can be made, as follows:

- $\phi_d V_s = 0.80(0.850) = 0.68$ klf (AF&PA SDPWS Table 4.2A)
- Shear capacity = $0.68(100) = 68.0$ kips
- Uniform diaphragm demand = $233/200 = 1.165$ klf
- $X = (117-68)/1.165 = 42.1$ ft (assumed as 50 ft from the diaphragm edge)

In a building of this size, it may be beneficial to further reduce the diaphragm nailing towards the middle of the roof. However, due to the requirements for subdiaphragms (see below) and diaphragm capacity in the longitudinal direction and for simplicity of design, no additional nailing pattern is used.

Table 11.2-1 contains a summary of the diaphragm framing and nailing requirements (all nails are 10d common). See Figure 11.2-2 for designation of framing and nailing zones and Figure 11.2-3 for typical plywood layout.

**Table 11.2-2** Roof Diaphragm Framing and Nailing Requirements

<table>
<thead>
<tr>
<th>Zone*</th>
<th>Framing</th>
<th>Structural 1 Plywood</th>
<th>Nail Spacing (in.)</th>
<th>Capacity (kip/ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Boundaries and Cont. Panel Edges</td>
<td>Other Panel Edges</td>
<td>Intermediate Framing Members</td>
</tr>
<tr>
<td>A</td>
<td>$3 \times 12$</td>
<td>15/32 in.</td>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td>B</td>
<td>$2 \times 12$</td>
<td>15/32 in.</td>
<td>4</td>
<td>6</td>
</tr>
</tbody>
</table>

1.0 in. = 25.4 mm, 1.0 kip/ft = 14.6 kN/m.
* Refer to Figure 11.2-2 for zone designation.
Figure 11.2-2 Diaphragm framing and nailing layout
(1.0 ft = 0.3048 m)

Figure 11.2-3 Typical diaphragm plywood layout
(1.0 ft = 0.3048 m, 1.0 in = 25.4mm)
11.2.4.2.2 Chord Design. Although the bond beam at the masonry wall could be used as a diaphragm chord, this example illustrates the design of the wood ledger member as a chord. Chord forces are computed using a simply supported beam analogy, where the design force is the maximum moment divided by the diaphragm depth.

- Diaphragm moment, \( M = \frac{wL^2}{8} = \frac{F_{p,\text{roof}}L}{8} = 233(200/8) = 5,825 \text{ ft-kips} \)
- Chord force, \( T = C = \frac{5,825}{(100 - 16/12)} = 59.0 \text{ kips} \)

Try a select structural Douglas Fir-Larch 4×12 for the chord. Assuming two 1-1/16-inch bolt holes (for 1-inch bolts) at splice locations, the net chord area is 31.9 in². Tension strength (parallel to wood grain), per the AF&PA NDS, is as follows:

\[
F_t' = F_t C_T K_T \phi_T = (1,000 \text{ psi})(1.0)(2.16/0.8)(0.8)(1.0) = 2,160 \text{ psi}
\]

\[
T' = F_t' A = 2,160(31.9)/1000 = 68.9 \text{ kips} > 59.0 \text{ kips}
\]

Design the splice for the maximum chord force of 59.0 kips. Try bolts with steel side plates using 1-inch A307 bolts, with a 3-1/2-inch length in the main member. The capacity, according to the AF&PA NDS, is as follows:

\[
Z' = Z K_T \phi_T = (4.90)(2.16/0.65)(0.65)(1.0) = 10.6 \text{ kips per bolt}
\]

The number of bolts required (at each side of the splice joint) is 59.0/10.6 = 5.6.

Use two rows of three bolts. The edge distance, end distance and spacing meet the AF&PA NDS requirements to avoid capacity reductions and the reduction for multiple bolts (group action factor) is negligible. The net area of the 4×12 chord with two rows of 1-1/16-inch holes is 31.9 in² as assumed above. Therefore, use six 1-inch A307 bolts on each side of the chord splice (see Figure 11.2-4).

In addition to the bolt checks, the steel splice plates would need to be check for tension. Although it is shown for illustration, this type of chord splice may not be the preferred splice against a masonry wall since the bolts and side plate, would have to be recessed into the wall.
11.2.4.2.3 Diaphragm Deflection and P-delta Check. Based on the procedure for contained in AF&PA SDPWS Section 4.2.2, diaphragm deflection is computed as follows:

$$\delta_{dia} = \frac{5vL^2}{8EAW} + \frac{0.25vL}{1000G_a} + \sum \left( \frac{x\Delta_c}{2W} \right)$$

The equation produces the midspan diaphragm displacement in inches and the individual variables must be entered in the force or length units as described below. A small increase in diaphragm deflection due to the large opening is neglected and the effects of the variable nail spacing are neglected for simplicity. The chord slip deflection is assumed to be zero because the chord is connected to the continuous bond beam at the top of the masonry wall.

The variables above and associated units used for computations are as follows:

- $v = (233/2)/100 = 1,165$ plf (shear per foot at boundary)
- $L = 200$ ft (diaphragm length)
- $W = 100$ ft (diaphragm width)
- $A =$ effective area of $4\times12$ chord and two-#6 bars assumed to be in the bond beam
  
  \[= 39.38 \text{ in}^2 + 2(0.44)(29,000,000/1,900,000) = 52.81 \text{ in}^2\]
- $E = 1,900,000$ psi (for Douglas Fir-Larch select structural chord)
- $G_a = 1.2(18) = 21.6$ kips/in. (apparent diaphragm shear stiffness from AF&PA SDPWS Table 4.2A accounting for nail slip and panel shear deformation, based on sheathing and nailing at the outer zone and increased by 1.2 per Footnote 3, assuming four-ply minimum sheathing)
Bending deflection = \(5vL^3/8EA W = 0.58\) in.

Shear/nail slip deflection = \(0.25vL/1000G_a = 2.71\) in.

Deflection due to chord slip at splices = \(\Sigma(x\Delta_c)/2W \approx 0.00\) in. (as noted above)

Total for diaphragm:

\[\delta_{dia} = 0.58+2.71+0.00 = 3.29\] in.

End wall deflection = 0.037 in. (see Sec. 10.1 of this volume of design examples)

Therefore, the total elastic deflection \(\delta_{xe} = 3.29+0.037 = 3.29\) in.

Total deflection, \(\delta = C_d\delta_{xe}/I = 3.5(3.29)/1.0 = 11.53\) in.

The drift ratio at the center of the diaphragm = \(\delta/h_s = 11.53/[28(12)] = 0.034\).

This exceeds the maximum drift ratio of 0.025 permitted for most low-rise buildings in Occupancy Category II (Standard Table 12.12-1). However, for one-story buildings, Standard Table 12.12-1, Footnote c permits unlimited drift, provided that the structural elements and finishes can accommodate the drift. The limit for masonry cantilever shear wall structures (0.007) should only be applied to the in-plane movement of the end walls (0.13\(h = 0.0004 < 0.007\)). The construction of the out-of-plane walls allows them to accommodate very large drifts. It is further expected that the building does not contain interior elements that are sensitive to drift.

P-delta effects are computed according to Provisions Section 12.8.7, which modifies Standard Section 12.8.7 for determining the stability coefficient, \(\theta\), per Provisions Equation 12.8-16:

\[
\theta = \frac{P \Delta I}{V_s h_s C_d}
\]

(Note that the Provisions adds the importance factor, \(I\), that was missing in the Standard equation.) Because the midspan diaphragm deflection is substantially greater than the deflection at the top of the masonry end walls, it would be overly conservative to consider the entire design load at the maximum deflection. Therefore, the stability coefficient is computed by splitting the P-delta product into two terms: one for the diaphragm and one for the end walls.

For the diaphragm, consider the weight of the roof and side walls at the maximum displacement. (This overestimates the P-delta effect. The computation could consider the average displacement of the total weight, which would lead to a reduced effective delta. Also, the roof live load need not be included.)

\[P = 400+416 = 816\] kips

\[\Delta = 11.53\] in.

\[V = 233\] kips (diaphragm force)

For the end walls, consider the weight of the end walls at the wall displacement:
$P = 330 \text{ kips}$

$\Delta = (3.5)(0.037) = 0.13 \text{ in.}$

$V = 264 \text{ kips (additional base shear for wall design)}$

For story height, $h = 28$ feet, the stability coefficient is:

$$\theta = \frac{(P\Delta \div V) + (P\Delta \div V)}{hC_d} = \frac{816(11.53) + 330(0.13)}{233 + 264} \div (28)(12)(3.5) = 0.034$$

For $\theta < 0.10$, P-delta effects need not be considered based on Provisions Section 12.8.7.

Since the P-delta effects are not significant for this structure and the Standard does not impose drift limitations for this type of structure, the computed diaphragm deflections appear acceptable.

**11.2.4.2.4 Detail at Opening.** Consider diaphragm strength at the roof opening as required by Standard Section 12.10.1. The diaphragm nailing must be checked for the reduced total width of diaphragm sheathing and the chords must be checked for bending forces at the opening.

Check diaphragm nailing for the shear in the diaphragm at edge of opening. The maximum shear at the exterior-side edge of the opening is computed as follows:

$$\text{Shear} = 116.5 - [40(1.165)] = 69.9 \text{ kips}$$

$$v = \frac{69.9}{(100-20)} = 0.874 \text{ klf}$$

Because the opening is centered in the width of the diaphragm, half the force to the diaphragm must be distributed on each side of the opening.

Diaphragm capacity in this area is 0.680 klf as computed previously (see Table 11.2-1 and Figure 11.2-2). Because the diaphragm demand at the reduced section exceeds the capacity, the extent of the Zone A nailing and framing should be increased. For simplicity, extend the Zone A nailing to the interior edge of the opening (60 feet from the end wall). The diaphragm strength is now adequate for the reduced overall width at the opening.

**11.2.4.2.5 Framing around Opening.** The opening is located 40 feet from one end of the building and is centered in the other direction (Figure 11.2-5). This does not create any panels with very high aspect ratios.

In order to develop the chord forces, continuity will be required across the glued-laminated beams in one direction and across the roof joists in the other direction.
11.2.4.2.6 Chord Forces at Opening. To determine the chord forces on the edge joists, split the diaphragm into smaller free-body sections, assume the inflection points will be at the midpoint of the elements (Figure 11.2-6) and compute the forces at the opening using a uniformly distributed diaphragm demand of \( \frac{233}{200} = 1.165 \) klf.

For Element 1 (shown in Figure 11.2-7):

\[
\begin{align*}
  w_f &= \frac{1.165}{2} = 0.582 \text{ kips/ft} \text{ (assuming half the diaphragm load on each side of the opening)} \\
  V_{IB} &= 0.5[116.5-(40)(1.165)] = 35.0 \text{ kips} \text{ (based on diaphragm unit shear on right side of opening)} \\
  V_{IA} &= 35.0-20(0.582) = 23.3 \text{ kips} \text{ (based on diaphragm unit shear on left side of opening)} \\
  M_f &= \frac{1}{2}[35.0(10) + 23.3(10)] = 291 \text{ ft-kips} \text{ (assuming equal moments at each edge of the section)}
\end{align*}
\]

The chord force due to \( M_f = 291/40 = 7.28 \) kips. This is only 35 psi on the glued-laminated beam on the edge of the opening. This member is adequate by inspection. On the other side of this diaphragm element, the chord force is much less than the maximum global chord force (59.0 kips), so the ledger and ledger splice are adequate.
For Element 3, analyze Element 2 (shown in Figure 11.2-7) in the same manner as Element 1:

\[ w_2 = 1.165 \times \frac{40}{100} = 0.466 \text{ kips/ft} \]
\[ V_3 = 116.5 \times \frac{40}{100} = 46.6 \text{ kips} \]
\[ V_{1B} = 35.0 \text{ kips} \]
\[ M_I = 291 \text{ ft-kips.} \]

\( T_{1B} \) is the chord force due to moment on the total diaphragm:

\[ M = 116.5 \times 40 - 1.165 \times \frac{40^2}{2} = 3,728 \text{ ft-kips} \]
\[ T_{1B} = 3,728/100 = 37.3 \text{ kips} \]
\[ \Sigma M_0: M_3 = M_I + 40V_3 - 40T_{1B} - w_2\frac{40^2}{2} = 291 \text{ ft-kips} \]

Therefore, the chord force on the roof joist = 291/40 = 7.26 kips
Alternatively, the chord design should consider the wall anchorage force interrupted by the opening. As described in Section 11.2.4.4.1, the edge members on each side of the opening are used as continuous cross-ties, with maximum cross-tie force of 16.6 kips. Therefore, the cross-tie will adequately serve as a chord at the opening.

11.2.4.3 Roof Diaphragm Design for Longitudinal Direction

Force = 209 kips

Maximum end shear = 0.50(209) = 104.5 kips

Diaphragm unit shear, \( v = \frac{104.5}{200} = 0.523 \text{ klf} \)

For this direction, the plywood layout is Case 3 in AF&PA SDPWS Table 4.2A. Using 1/2-inch Structural I plywood rated sheathing, blocked, with 10d common nails at 4 inches on center at diaphragm boundaries and continuous panel edges parallel to the load (ignoring the capacity of the extra nails in the outer zones), per AF&PA SDPWS Table 4.2A:

\[
\phi D v_s = 0.80(0.850) = 0.68 \text{ klf} > 0.523 \text{ klf} \quad \text{OK}
\]

Therefore, use the same nailing designed for the transverse direction. Compared with the transverse direction, the diaphragm deflection and P-delta effects will be satisfactory.

11.2.4.4 Masonry Wall Anchorage to Roof Diaphragm. As stipulated in Standard Section 12.11.2.1, masonry walls must be anchored to flexible diaphragms to resist out-of-plane forces computed per Standard Equation 12.11-1 as follows:

\[
F_p = 0.8 S_{DS} W_p = 0.8(1.0)(1.0) W_p = 0.8 \ W_p
\]

Side walls, \( F_p = 0.8(65\text{psf})(2+28/2)/1,000 = 0.83 \text{ klf} \)

End walls, \( F_p = 0.8(103\text{psf})(2 + 28/2)/1,000 = 1.32 \text{ klf} \)

11.2.4.4.1 Anchoring Joists Perpendicular to Walls (Side Walls). The roof joists are spaced at 2 feet on center, so as a preliminary design, consider a connection at every other joist that will develop 4(0.83) =
3.32 kip/ joist.  Note that 4 feet is the maximum anchor spacing allowed without having to check the walls for resistance to bending between anchors (Standard Sec. 12.11.2).

A common connection for this application is a metal tension tie-down or hold-down device that is anchored to the masonry wall with an embedded bolt and is either nailed, screwed, or bolted to the roof joist. Other types of anchors include metal straps that are embedded in the wall and nailed to the top of the joist. The ledger is not used for this force transfer because the eccentricity between the anchor bolt and the plywood creates tension perpendicular to the grain in the ledger (cross-grain bending), which is prohibited. Also, using the edge nails to resist tension perpendicular to the edge of the plywood is not permitted.

Try a tension tie with a 3/4-inch headed anchor bolt, embedded in the bond beam and with 18 10d nails into the side of the joist (Figure 11.2-8). The cataloged ASD tension capacity of this connector is 3.61 kips based on a load duration factor of 1.60. Modifying the allowable values using the procedure in Section 11.1.4.5 results in a design LRFD capacity of:

\[ Z'K_f\phi \lambda = (3.61)(2.88/1.60)(0.65)(1.0) = 4.22 \text{ kips per anchor} > 3.32 \text{ kips} \quad \text{OK} \]

The joists anchored to the masonry wall must also be adequately connected to the diaphragm sheathing. Determine the adequacy of the typical nailing for intermediate framing members. The nail spacing is 12 inches and the joist length is 20 feet, so there are 20 nails per joist. From the AF&PA NDS, the LRFD capacity of a single 10d common nail in 1/2-inch plywood is:

\[ Z'K_f\phi \lambda = (0.090)(2.16/0.65)(0.65) = 0.194 \text{ kips per nail} \]

\[ 20(0.194) = 3.88 \text{ kips} > 3.32 \text{ kips} \quad \text{OK} \]

The embedded anchor bolt also serves as the ledger connection, for both gravity loading and in-plane shear transfer at the diaphragm. Therefore, the strength of the anchorage to masonry and the strength of the bolt in the wood ledger must be checked.

For the anchorage to masonry, check the combined tension and shear resulting from the out-of-plane seismic loading (3.32 kips per bolt) and the vertical gravity loading. Assuming 20 psf dead load (roof live load need not be combined with seismic loads), a 10-foot tributary roof width and ledger bolts at 2 feet on center (at tension ties and in between) the vertical load per bolt = (20 psf)(10 ft)(2 ft)/1,000 = 0.40 kip. Using the load combinations described previously, the design horizontal tension and vertical shear on the bolt are as follows:

\[ b_{tf} = 1.0Q_e = 3.32 \text{ kips} \]

\[ b_{vf} = 1.4D = 1.4(0.40) = 0.56 \text{ kip} \]

The anchor bolts in masonry are designed according to ACI 530 as adopted by the Standard (Sec. 14.4) and as modified by Standard Sections 14.4.7.6 and 14.4.7.7. Standard Section 14.4.7.6 requires the strength of the anchorage connecting diaphragms to other parts of the seismic force-resisting system to be governed by steel tensile or shear yielding unless the anchorage is designed for 2.5 times the required forces. For this example, the anchorage is proportioned such that the steel governs the capacity. Standard Section 14.4.7.7 modifies the shear strength requirements for anchorage, requiring that the shear capacity is not more than 2 times the strength due to masonry pry-out.
Using 3/4-inch headed anchor bolts with an effective embedment depth of 6 inches, both tensile strength, $B_{an}$ and shear strength, $B_{av}$, will be computed assuming the masonry strength, $f'_{m}$, is 2,000 psi and the steel strength, $f_y$, is 36,000 psi. Tensile strength per ACI 530 Section 3.1.6.2 is taken as the lesser of the following:

$$B_{an} = A_b f_y = 0.44(36) = 15.8 \text{ kips}$$

$$B_{an} = 4 A_{pt} \sqrt{f_m} = 4(113)\sqrt{2,000} = 20.2 \text{ kips}$$

where $A_{pt}$ is the projected area of the right cone and is equal to $\pi (l_b)^2$, where $l_b$ is the effective embedment depth. Therefore, $A_{pt} = \pi (6)^2 = 113 \text{ in}^2$.

Since the steel strength governs, Standard Section 14.4.7.6 is met and $\phi = 0.9$. Therefore the design strength in tension is $0.9(15.8) = 14.2 \text{ kips}$.

Shear strength per ACI 530 Section 3.1.6.3 is taken as the lesser of the following:

$$B_{av} = 0.6 A_b f_y = 0.6(0.44)(36) = 9.50 \text{ kips}$$

$$B_{av} = 4 A_{pv} \sqrt{f_m} = 4(56.5)\sqrt{2,000} = 10.1 \text{ kips}$$

where $A_{pv}$ is one half of the projected area of the right cone and is equal to $113/2 = 56.5 \text{ in}^2$. Since the steel strength governs, Standard Section 14.4.7.6 is met for shear and $\phi = 0.9$. Therefore, the design strength in shear is $0.9(9.50) = 8.55 \text{ kips}$.

Shear and tension are combined per ACI 530 Section 3.1.6.4 as:

$$\frac{h_{an}}{q B_{an}} + \frac{h_{av}}{q B_{av}} = \frac{3.32}{14.2} + \frac{0.56}{8.55} = 0.30 < 1.0$$

OK

Figure 11.2-8 summarizes the details of the connection. In-plane seismic shear transfer (combined with gravity) and orthogonal effects are considered in a subsequent section.
According to Standard Section 12.11.2.2.1, diaphragms must have continuous cross-ties to distribute the anchorage forces into the diaphragms. Although the Standard does not specify a maximum spacing, 20 feet is common practice for this type of construction and seismic design category.

For cross-ties at 20 feet on center, the wall anchorage force per cross-tie is:

\[ (0.83 \text{ klf})(20 \text{ ft}) = 16.6 \text{ kips} \]

Try a 3\times12 Douglas Fir-Larch No. 1 as a cross-tie. Assuming one row of 1-1/8-inch bolt holes, the net area of the section is 25.3 in\(^2\). Tension strength (parallel to wood grain) per the AF&PA NDS Supplement is:

\[ F'_t = F_t K_F \phi \lambda = (0.675)(2.16/0.8)(0.8) = 1.46 \text{ ksi} \]

\[ T' = F' t A = 1.46(25.3) = 36.9 \text{ kips} > 16.6 \text{ kips} \quad \text{OK} \]

However, the cross-tie must be checked for combined gravity and lateral loads. The governing case for combined loads is midspan where the maximum gravity moment is combined with seismic tension. The 3\times12 cross-tie has the following properties:

\[ A = 28.1 \text{ in}^2 \]

\[ S = 52.7 \text{ in}^3 \]

\[ F'_t = 1.46 \text{ ksi} \]

\[ F'_{b} = F_b C_f K_F \phi \lambda = (1.000)(1.15)(2.16/0.85)(0.85) = 2.48 \text{ kips} \]
The factored dead load moment is computed using the load combinations described above as:

\[ M_u = 1.4(20 \text{ psf})(2 \text{ ft})(20 \text{ ft})^2/8 = 2.80 \text{ ft-kips} \]

The factored stresses are computed as:

\[ f_t = 16.6/28.1 = 0.591 \text{ ksi} \]

\[ f_b = (2.80)(12)/52.7 = 0.638 \text{ ksi} \]

Combined stresses are checked in accordance with AF&PA NDS Section 3.9.1 as follows:

\[ \frac{f_t}{F_t} + \frac{f_b}{F_b} = \frac{0.591}{1.46} + \frac{0.638}{2.48} = 0.66 < 1.0 \]

OK

At the splices, try a double tie-down device with three 1-inch bolts in double shear through the 3×12 member (Figure 11.2-9). Product catalogs provide design capacities for single tie-downs only; the design of double hold-downs requires two checks. First, consider twice the capacity of one tie-down and, second, consider the capacity of the bolts in double shear.

For the double tie-down, use the procedure in Section 11.1.4.5 to modify the allowable values:

\[ 2ZK_f\phi\lambda = 2(8.81)(2.88/1.60)(0.65)(1.0) = 20.6 \text{ kips} > 16.6 \text{ kips} \]

OK

For the four bolts, the AF&PA NDS gives:

\[ 4ZK_f\phi\lambda = 4(3.50)(2.16/0.65)(0.65)(1.0) = 30.2 \text{ kips} > 16.6 \text{ kips} \]

OK

Figure 11.2-9 Chord tie at roof opening
(1.0 in. = 25.4 mm)

In order to transfer the wall anchorage forces into the cross-ties, the subdiaphragms between these ties must be checked per Standard Section 12.11.2.2.1. There are several ways to perform these
subdiaphragm calculations. One method is illustrated in Figure 11.2-10. The subdiaphragm spans between cross-ties and utilizes the glued-laminated beam and ledger as its chords. The 1-to-1 aspect ratio meets the requirement of 2.5 to 1 for subdiaphragms per Standard Section 12.11.2.2.1.

For the typical subdiaphragm (Figure 11.2-10):

\[ F_p = 0.83 \text{ klf} \]

\[ v = (0.83)(20/2)/20 = 0.415 \text{ klf}. \]

The subdiaphragm demand is less than the minimum diaphragm capacity (0.68 klf along the center of the side walls). In order to develop the subdiaphragm strength and boundary nailing must be provided along the cross-tie beams.

![Cross tie plan layout and subdiaphragm free-body diagram for side walls](image)

**Figure 11.2-10** Cross tie plan layout and subdiaphragm free-body diagram for side walls

(1.0 ft = 0.3048 m, 1.0 kip/ft = 14.6 kN/m)

11.2.4.4.2 Anchorage at Joists Parallel to Walls (End Walls). Where the joists are parallel to the walls, tied elements must transfer the forces into the main body of the diaphragm, which can be accomplished by using either metal strapping and blocking or metal rods and blocking. This example uses threaded rods that are inserted through the joists and coupled to the anchor bolt (Figure 11.2-11). Blocking is
added on both sides of the rod to transfer the force into the plywood sheathing. The tension force in the rod causes a compression force on the blocking through the nut and on the bearing plate at the innermost joist.

Figure 11.2-11 Anchorage of masonry wall parallel to joists  
(1.0 ft = 0.3048 m, 1.0 in. = 25.4 mm)

The anchorage force at the end walls is 1.32 klf. Space the connections at 4 feet on center so that the wall need not be designed for flexure. Thus, the anchorage force is 5.28 kips per anchor.

Try a 3/4-inch headed anchor bolt, embedded into the masonry. In this case, gravity loading on the ledger is negligible and can be ignored and the anchor can be designed for tension only. (In-plane shear transfer and orthogonal effects are considered later.)

As computed for 3/4-inch headed anchor bolts (with 6 inch embedment), the design axial strength is $\phi B_{an} = 14.2$ kips > 5.28 kips. Therefore, the bolt is acceptable.

Using couplers rated for 125 percent of the strength of the rod material, the threaded rods are then coupled to the anchor bolts and extend six joist spaces (12 feet) into the roof framing. (This length of 12 feet is required for the subdiaphragm force transfer discussed below.)

Nailing the blocking to the plywood sheathing is determined using nail capacities from the AF&PA NDS. As computed previously, the LRFD capacity of a single 10d common nail, $Z'K_f\phi \lambda = 0.194$ kips per nail. Thus, 28 nails are required ($5.28/0.194$). This corresponds to a nail spacing of approximately 10 inches for two 12-foot rows of blocking. Space nails at 8 inches for convenience.

Use the glued-laminated timber beams (at 20 feet on center) to provide continuous cross-ties and check the subdiaphragms between the beams to provide adequate load transfer to the beams per Standard Section 12.11.2.2.1:
Design tension force on beam = (1.32 klf)(20 ft) = 26.4 kips

The stress on the beam is $f_e = 26,400/[8.75(24)] = 126$ psi, which is small. The beam is adequate for combined moment due to gravity loading and axial tension.

At the beam splices, try 3/4-inch bolts with steel side plates. Per the AF&PA NDS:

$$ZK_f \phi \lambda = (3.34)(2.16/0.65)(0.65)(1.0) = 7.21 \text{ kips per bolt}$$

The number of bolts required (on each side of the splice joint) is $26.4/7.21 = 3.7$.

Use four bolts in a single row at mid-height of the beam, with 1/4-inch by 4-inch steel side plates. The reduction (group action factor) for multiple bolts is negligible. Although not included in this example, the steel side plates should be checked for tension capacity on the gross and net sections. There are pre-engineered hinged connectors for glued-laminated beams that could provide sufficient tension capacity for the splices.

In order to transfer the wall anchorage forces into the cross-ties, the subdiaphragms between these ties must be checked per Provisions Section 12.11.2.2.1. The procedure is similar to that used for the side walls as described previously. The end wall condition is illustrated in Figure 11.2-12. The subdiaphragm spans between beams and utilizes a roof joist as its chord. In order to adequately engage the subdiaphragm, the wall anchorage ties must extend back to this chord. Since the maximum aspect ratio for subdiaphragms is 2.5 to 1, the minimum depth is $20/2.5 = 8$ feet.
For the typical subdiaphragm (Figure 11.2-12):

\[ F_p = 1.32 \text{ klf} \]

\[ v = \frac{(1.32)(20/2)}{8} = 1.65 \text{ klf} \]

As computed previously (see Table 11.2-1 and Figure 11.2.2), the diaphragm strength in this area is 1.17 klf < 1.65 klf. Therefore, increase the subdiaphragm depth to 12 feet (six joist spaces):

\[ v = \frac{(1.32)(20/2)}{12} = 1.10 \text{ klf} > 1.17 \text{ klf} \]

OK

In order to develop the subdiaphragm strength, boundary nailing must be provided along the cross-tie beams. There are methods of refining this analysis using multiple subdiaphragms so that all of the tension anchors need not extend 12 feet into the building.

11.2.4.4.3 Transfer of Shear Wall Forces. The in-plane diaphragm shear must be transferred to the masonry wall by the ledger, parallel to the wood grain. The connection must have sufficient capacity for the diaphragm demands as follows:

- Side walls: 0.523 klf
End walls: 1.165 klf

For each case, the capacity of the bolted wood ledger and the capacity of the anchor bolts embedded into masonry must be checked. Because the wall connections provide a load path for both in-plane shear transfer and out-of-plane wall forces, the bolts must be checked for orthogonal load effects in accordance with Standard Section 12.5. That is, the combined demand must be checked for 100 percent of the lateral load effect in one direction (e.g., shear) and 30 percent of the lateral load effect in the other direction (e.g., tension).

At the side walls, the wood ledger with 3/4-inch bolts (Figure 11.2-8) must be designed for gravity loading (0.56 kip per bolt as computed above) as well as seismic shear transfer. The seismic load per bolt (at 2 feet on center) is 0.523(2) = 1.05 kips.

Combining gravity shear and seismic shear produces a resultant force of 1.19 kips at an angle of 28 degrees from the axis of the wood grain. The bolt capacity in the wood ledger can be determined using the formulas for bolts at an angle to the grain per the AF&PA NDS (either adjusting for dowel bearing strength per Section 11.3.3 or adjusting the tabulated bolt values per Appendix J). The resulting design value, \( Z = 1.41 \) kips and the LRFD capacity is determined as follows:

\[
Z_{K_F\phi\lambda} = (1.41)(2.16/0.65)(0.65)(1.0) = 3.05 \text{ kips} > 1.19 \text{ kips} \quad \text{OK}
\]

This bolt spacing also satisfies the load combination for gravity loading (dead and roof live) only.

For the check of the embedded anchor bolts, the factored demand on a single bolt is 1.05 kips in horizontal shear (in-plane shear transfer), 3.32 kips in tension (out-of-plane wall anchorage) and 0.56 kip in vertical shear (gravity). Orthogonal effects are checked, using the following two equations:

\[
\frac{0.3(3.32)}{8.75} + \frac{\sqrt{1.05^2 + 0.56^2}}{2.66} = 0.56
\]

and

\[
\frac{3.32}{8.75} + \frac{\sqrt{(0.3 \times 1.05)^2 + 0.56^2}}{2.66} = 0.62 \quad \text{(controls) < 1.0} \quad \text{OK}
\]

At the end walls, the ledger with 3/4-inch bolts (Figure 11.2-11) need only be checked for in-plane seismic shear because gravity loading is negligible. For bolts spaced at 4 feet on center, the demand per bolt is 1.165(4) = 4.66 kips parallel to the grain of the wood. Per the AF&PA NDS:

\[
Z_{K_F\phi\lambda} = (1.61)(2.16/0.65)(0.65)(1.0) = 3.48 \text{ kips} < 4.66 \text{ kips} \quad \text{NG}
\]

Therefore, add 3/4-inch headed bolts evenly spaced between the tension ties such that the bolt spacing is 2 feet on center and the demand per bolt is 1.165(2) = 2.33 kips. These added bolts are used for in-plane shear only and do not have coupled tension tie rods.

For the check of the embedded bolts, the factored demand on a single bolt is 2.33 kips in horizontal shear (in-plane shear transfer), 5.28 kips in tension (out-of-plane wall anchorage), 0 kip in vertical shear (gravity is negligible). Orthogonal effects are checked using the following two equations:
\[
\frac{0.3(5.28)}{8.75} + \frac{2.33}{2.66} = 1.06 \text{ (controls)} > 1.0
\]

\[
\frac{5.28}{8.75} + \frac{0.3(2.33)}{2.66} = 0.86
\]

Since one of the equations is slightly more than unity, the bolt capacity can be increased by using a larger bolt or more embedment depth, or more bolts can be added. With this minor revision, the wall connections satisfy the requirements for combined gravity and seismic loading, including orthogonal effects.