10

WOOD DESIGN

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This chapter examines the design of a variety of wood building elements. Section10.1 features a BSSC trial design prepared in the early 1980s as a starting point. Section 10.2 completes the roof diaphragm design for the building featured in Section 9.1. In both cases, only those portions of the designs necessary to illustrate specific points are included.

Typically, the weak links in wood systems are 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 ," which is as much an art as a science, 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. As referenced in the 2000 *NEHRP Recommended Provisions*, wood structural panel sheathing includes plywood and other products, such as oriented-strand board (OSB), that conform to the materials standards of Chapter 12. According to *Provisions* Chapter 12, panel materials other than wood structural panel sheathing do not have a recognized capacity for seismic-force resistance in engineered construction.

In addition to the 2000 *NEHRP Recommended Provisions* and *Commentary* (hereafter, the *Provisions* and *Commentary*), the documents edited below are either referenced directly, or are useful design aids for wood construction.

AF&PA Manual	American Forest & Paper Association 1996. *Manual for Engineered Wood Construction (LRFD)*, including supplements, guidelines, and ASCE 16-95, AF&PA.
[AF&PA Wind & Seismic	American Forest and Paper Association. 2001. *Special Design Provisions for Wind and Seismic,* ADS/LRFD Supplement. AF&PA.]

ANSI/AITC A190.1	American National Standards Institute/American Institute of Timber Construction. 1992. *American National Standard for Wood Products: Structural Glued-Laminated Timber*, A190.1. AITC.
APA PDS	American Plywood Association. 1998. *Plywood Design Specification*, APA.
APA 138	American Plywood Association. 1998. *Plywood Diaphragms*, APA Research Report 138. APA.
ASCE 7	American Society of Civil Engineers. 1998 [2002]. *Minimum Design Loads for Buildings and Other Structures*. ASCE.
ASCE 16	American Society of Civil Engineers. 1995. *Standard for Load and Resistance Factor Design (LRFD) for Engineered Wood Construction*. ASCE.
UBC Std 23-2	International Conference of Building Officials. 1997. *UBC Standard 23-2 Construction and Industrial Plywood, Uniform Building Code*. ICBO.
Roark	Roark, Raymond. 1985. *Formulas for Stress and Strain*, 4th Ed. McGraw-Hill.
USGS CD-ROM	United States Geological Survey. 1996. *Seismic Design Parameters*, CD-ROM. USGS.
WWPA Rules	Western Wood Products Association. 1991. *Western Lumber Grading Rules*. WWPA.

Although this volume of design examples is based on the 2000 *Provisions*, it has been annotated to reflect changes made to the 2003 *Provisions*. Annotations within brackets,[ ], indicated both organizational changes (as a result of a reformat of all the chapters of the 2003 *Provisions*) and substantive technical changes to the 2003 *Provisions* and it's primary reference documents. While the general concepts of the changes are described, the design examples and calculations have not been revised to reflect the changes to the 2003 *Provisions*.

The most significant change to the wood chapter in the 2003 *Provisions* is the incorporation by reference of the AF&PA, ADS/LRFD Supplement, *Special Design Provisions for Wind and Seismic* for design of the engineered wood construction. A significant portion of the 2003 *Provisions* Chapter 12, including the diaphragm and shear wall tables, has been replaced by a reference to this document. This updated chapter, however, does not result in significant technical changes , as the Supplement, (referred to herein as AF&PA Wind&Seismic) is in substantial agreement with the 2000 *Provisions.* There are, however, some changes to the provisions for perforated shear walls, which are covered in Section 10.1

Some general technical changes in the 2003 *Provisions* that relate to the calculations and/or design in this chapter include updated seismic hazard maps, changes to the Seismic Design Category classification for short period structures, revisions to the redundancy requirements, revisions to the wall anchorage design requirement for flexible diaphragms, and a new "Simplified Design Procedure" that could be applicable to the examples in this chapter.

Where the affect the design examples in this chapter, other significant changes to the 2003 *Provisions* and primary reference documents are noted. However, some minor changes to the 2003 *Provisions* and the reference documents may not be noted.

10.1 THREE-STORY WOOD APARTMENT BUILDING; SEATTLE, WASHINGTON

This example features a wood frame building with plywood diaphragms and shear walls. It is based on a BSSC trial design by Bruce C. Olsen, Structural Engineer, Seattle, Washington.

10.1.1 BUILDING DESCRIPTION

This three-story, wood frame apartment building has plywood floor diaphragms and shear walls. The building has a double-loaded central corridor. Figure 10.1-1 shows a typical floor plan, and Figure 10.1-2 shows a longitudinal section and elevation. The building is located in a neighborhood a few miles north of downtown Seattle. The site coordinates for determining the seismic design parameters are 47.69° N, 122.32° W.

The shear walls in the longitudinal direction are located on the exterior faces of the building and along the corridor. (In previous versions of this volume of design examples, the corridor walls were gypsum wallboard sheathed shear walls; however, gypsum wallboard sheathing, is no longer recognized for engineered design of shear walls per *Provisions* Sec. 12.3.5. Therefore, plywood shear walls are provided at the corridors.) The entire solid (non-glazed) area of exterior walls plywood sheathing, but only a portion of the corridor walls will require sheathing. For the purposes of this example, assume that each corridor wall will have a net of 55 ft of plywood (the reason for this is explained later). In the transverse direction, the end walls and one line of interior shear walls provide lateral resistance. (In previous versions of this example, only the end walls were shear walls. The interior walls now are required for control of diaphragm deflections given the increased seismic ground motion design parameters for the Seattle area.)

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 in. 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. 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 grade walls (Figure 10.1-3). The depth of the footings, and the height of the grade walls, are sufficient to provide crawl space clearance beneath the first floor.

The building is typical of apartment construction throughout the western United States, and has the weight necessary to balance potential overturning forces in the transverse direction. If the ground floor were a slab-on-grade, however, the resulting shallower grade wall might well require special attention, due to the possibility of overturning on some of the shear wall units.

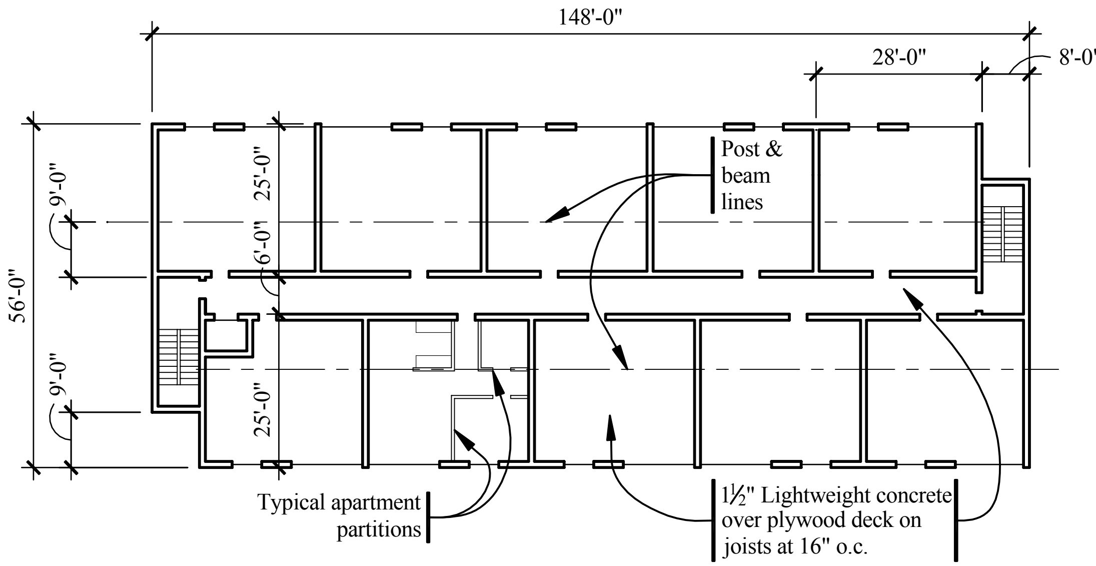

Figure 10.1-1 Typical floor plan (1.0 ft = 0.3048 m).

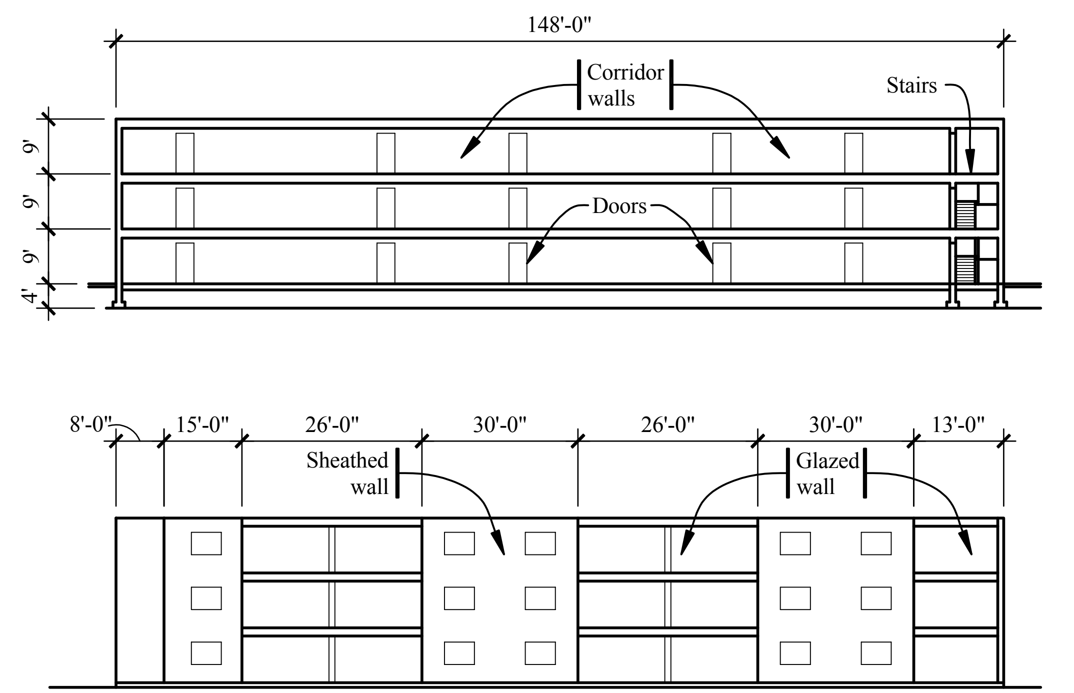

Figure 10.1-2 Longitudinal section and elevation (1.0 ft = 0.3048 m).

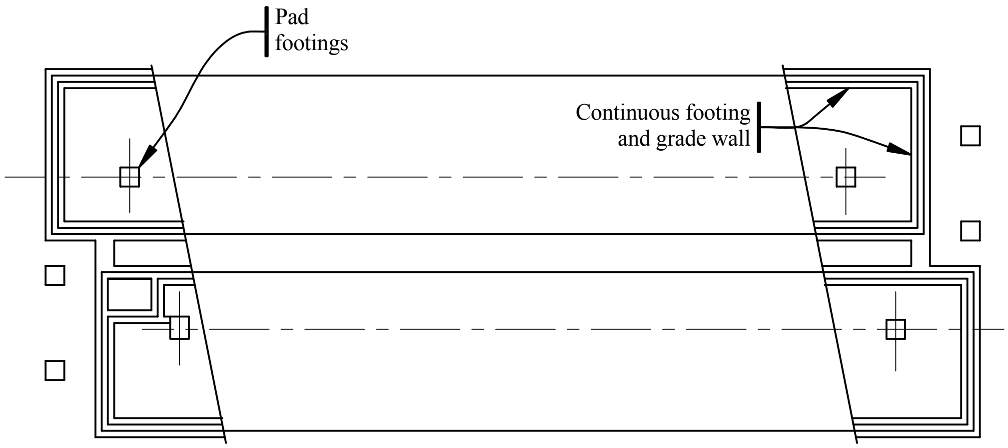

Figure 10.1-3 Foundation plan.

10.1.1.1 Scope

In this example, the structure is designed and detailed for forces acting in the transverse and longitudinal directions. However, greater attention is paid to the transverse direction, because of diaphragm flexibility. The example includes the following

1. Development of equivalent static loads, including torsional effects on plywood diaphragms,
2. Design and detailing of transverse plywood walls for shear and overturning moment,
3. Design and detailing of plywood floor and roof diaphragms,
4. Design and detailing of wall and diaphragm chord members,
5. Detailed deflection and P-delta calculations, and
6. Design and detailing of longitudinal plywood walls using the requirements for perforated shear walls.

[Note that the new "Simplified Design Procedure" contained in 2003 *Provisions* Simplified Alternate Chapter 4 as referenced by 2003 *Provisions* Sec. 4.1.1 is likely to be applicable to this example, subject to the limitations specified in 2003 *Provisions* Sec. Alt. 4.1.1.]

10.1.2 BASIC REQUIREMENTS

10.1.2.1 *Provisions* Parameters

Seismic Use Group (*Provisions* Sec. 1.3 [1.2])	= I
Occupancy Importance Factor, I (*Provisions* Sec. 1.4 [1.3])	= 1.0
Site Coordinates	= 47.69° N, 122.32° W
Short Period Response, S_S (USGS CD-ROM)	= 1.34
One Second Period Response, S_1 (USGS CD-ROM)	= 0.46
Site Class (*Provisions* Sec. 4.1.2.1 [3.5])	= D
Seismic Design Category (*Provisions* Sec. 4.2 [1.4])	= D
Seismic-Force-Resisting System (*Provisions* Table 5.2.2 [4.3-1])	= Wood panel shear wall
Response Modification Coefficient, R (*Provisions* Table 5.2.2 [4.3-1)	= 6.5
System Overstrength Factor, Ω_0 (*Provisions* Table 5.2.2 [4.3-1)	= 3
Deflection Amplification Factor, C_d (*Provisions* Table 5.2.2 [4.3-1)	= 4

10.1.2.2 Structural Design Criteria

10.1.2.2.1 Ground Motion (Provisions Sec. 4.1.2 [3.3])

Based on the site location, the spectral response acceleration values can be obtained from either the seismic hazard maps accompanying the *Provisions* or from the USGS CD-ROM. For site coordinates 47.69° N, 122.32° W, the USGS CD-ROM returns short period response, S_S = 1.34 and one second period response, S_1 = 0.46.

[The 2003 *Provisions* have adopted the 2002 USGS probabilistic seismic hazard maps, and the maps have been added to the body of the 2003 *Provisions* as figures in Chapter 3 (instead of the previously used separate map package). A CD-ROM containing the site response parameters based on the 2002 maps is also available.]

The spectral response factors are then adjusted for the site class (*Provisions* Sec. 4.1.2.4 [3.5]). For this example, it is assumed that a site class recommendation was not part of the soils investigation, which would not be uncommon for this type of construction. When soil properties are not known, *Provisions* Sec. 4.1.2.1 [3.5] defaults to Site Class D, provided that soft soils (Site Class E or F) are not expected to be present at the site (a reasonable assumption for soils sufficient to support a multistory building on shallow spread footings). The adjusted spectral response acceleration parameters are computed, according to *Provisions* Eq. 4.1.2.4-1 and 4.1.2.4-2 [3.3-1 and 3.3-4], for the short period and one second period, respectively, as:

$$S_{MS} = F_a S_S = 1.0(1.34) = 1.34$$
$$S_{M1} = F_v S_1 = 1.54(0.46) = 0.71$$

where F_a and F_v are site coefficients defined in *Provisions* Tables 4.1.2.4a and 4.1.2.4b [3.3-1 and 3.3-2], respectively. Note that Straight line interpolation was used for F_v.

Finally, the design spectral response acceleration parameters (*Provisions* Sec. 4.1.2.5 [3.3.3]) are determined in accordance with *Provisions* Eq. 4.1.2.5-1 and 4.1.2.5-2 [3.3-3 and 3.3-4], for the short period and one second period, respectively, as:

$$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3}(1.34) = 0.89$$
$$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3}(0.71) = 0.47$$

10.1.2.2.3 Seismic Design Category (Provisions Sec. 4.2 [1.4])

Based on the Seismic Use Group and the design spectral response acceleration parameters, the Seismic Design Category is assigned to the building based on *Provisions* Tables 4.2.1a and 4.2.1b [1.4-1 and 1.4-2]. For this example, the building is assigned Seismic Design Category D.

[Note that the method for assigning seismic design category for short period buildings has been revised in the 2003 *Provisions*. If the fundamental period, T_a, is less than $0.8T_s$, the period used to determine drift is less than T_s, and the base shear is computed using 2003 *Provisions* Eq 5.2-2, then seismic design category is assigned using just 2003 *Provisions* Table 1.4-1 (rather than the greater of 2003 *Provisions* Tables 1.4-1 and 1.4-2). The change does not affect this example.]

10.1.2.2.4 Load Path (Provisions Sec. 5.2.1 [14.2-1])

See Figure 10.1-4. 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.

10.1.2.2.5 Basic Seismic-Force-Resisting Systems (Provisions Sec. 5.2.2 [4.3])

Building Class (*Provisions* Table 5.2.2 [4.3-1]): Bearing Wall System

Seismic-Force-Resisting System (*Provisions* Table 5.2.2 [4.3-1]): Light frame walls with shear panels with R = 6.5, Ω_0 = 3, and C_d = 4 for both directions

10.1.2.2.6 Structure Configuration (Provisions Sec. 5.2.3 [5.3.2])

Diaphragm Flexibility (*Provisions* Sec. 5.2.3.1 [4.3.2.1]): Rigid (wood panel diaphragm in light frame structure with structural panels for shear resistance). *Provisions* Sec. 12.4.1.1 [4.3.2.1]defines a structural panel diaphragm as flexible, if the maximum diaphragm deformation is more than two times the average story drift. Due to the central shear wall, this is not expected to be the case in this building.

Plan Irregularity (*Provisions* Sec. 5.2.3.2 [4.3.2.2]): Since the shear walls are not balanced for loading in the transverse direction (see Figure 10.1-4), there will be some torsional response of the system. The potential for torsional response combined with the rigid diaphragm requires that the building be checked for a torsional irregularity (*Provisions* Table 5.2.3.2 [4.3-2]). This check will be performed following the initial determination of seismic forces, and the final seismic forces will be modified if required.

Vertical Irregularity (*Provisions* Sec. 5.2.3.3 [4.3.2.3]): Regular

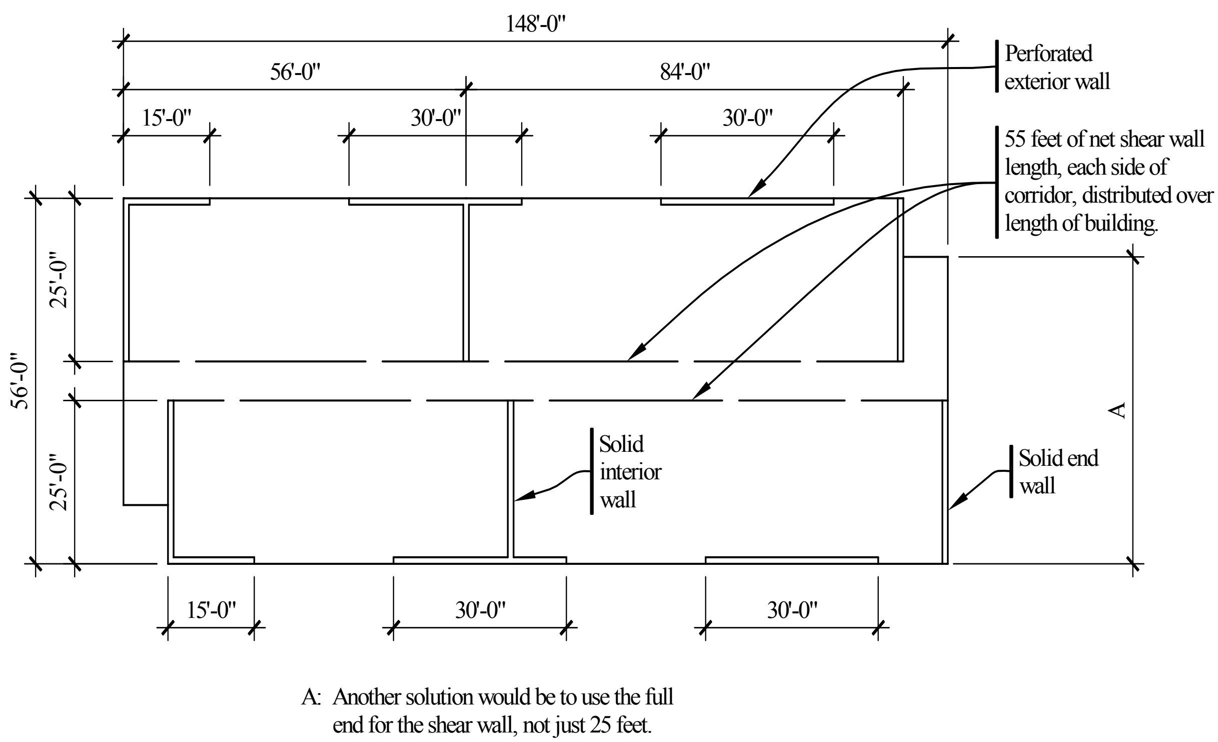

Figure 10.1-4 Load path and shear walls (1.0 ft = 0.3048 m).

10.12.2.7 Redundancy (Provisions Sec. 5.2.4 [4.3.3])

For Seismic Design Category D, the reliability factor, ρ, is computed per *Provisions* Eq. 5.2.4.2 [4.3.3.2]. Since the computation requires more detailed information than is known prior to the design, assume ρ = 1.0 for the initial analysis and verify later. If, in the engineer's judgement, the initial design appears to possess relatively few lateral elements, the designer may wish to use an initial ρ greater than 1.0 (but no greater than 1.5).

[The redundancy requirements have been substantially changed in the 2003 *Provisions*. For a shear wall building assigned to Seismic Design Category D, ρ = 1.0 as long as it can be shown that failure of a shear wall with height-to-length-ratio greater than 1.0 would not result in more than a 33 percent reduction in story strength or create an extreme torsional irregularity. The intent is that the aspect ratio is based on story height, not total height. Therefore, the redundancy factor would have to be investigated only in the longitudinal direction where the aspect ratios of the perforated shear walls would be interpreted as having aspect ratios greater than 1.0 at individual piers. In the longitudinal direct, where the aspect ratio is less than 1.0, ρ = 1.0 by default.]

10.1.2.2.8 Analysis Procedure (Provisions Sec. 5.2.5 [4.4.1])

Design in accordance with the equivalent lateral force (ELF) procedure (*Provisions* Sec. 5.4 [5.2]): No special requirements. In accordance with *Provisions* Sec. 5.2.5.2.3 [4.4.2], the structural analysis must consider the most critical load effect due to application of seismic forces in any direction for structures assigned to Seismic Design Category D. For the ELF procedure, this requirement is commonly satisfied by applying 100 percent of the seismic force in one direction, and 30 percent of the seismic force in the perpendicular direction; as specified in *Provisions* Sec. 5.2.5.2.2, Item a [4.4.2.2, Item 1]. For a light-framed shear wall building, with shear walls in two orthogonal directions, the only element affected by this directional combination would be the design of the shear wall end post and tie-down, located where the ends of two perpendicular walls intersect. In this example, the requirement only affects the shear wall intersections at the upper left and lower left corners of Figure 10.1-4. The directional requirement is satisfied using a two-dimensional analysis in the design of the remainder of the shear wall and diaphragm elements.

10.1.2.2.9 Design and Detailing Requirements (Provisions Sec. 5.2.6 [4.6])

See *Provisions* Chapters 7 and 12 for special foundation and wood requirements, respectively. As discussed in greater detail below, *Provisions* Sec. 12.2.1, now utilizes Load and Resistance Factor Design (LRFD) for the design of engineered wood structures. Therefore, the design capacities are consistent with the strength design demands of *Provisions* Chapter 5 [4 and 5].

10.1.2.2.10 Combination of Load Effects (Provisions Sec. 5.2.7 [4.2.2])

The basic design load combinations are as stipulated in ASCE 7 and modified by the *Provisions* Eq. 5.2.7-1 and 5.2.7-2 [4.2-1 and 4.2-1]. Seismic load effects according to the *Provisions* are:

$$E = \rho Q_E + 0.2 S_{DS} D$$

and

$$E = \rho Q_E - 0.2 S_{DS} D$$

when seismic and gravity are additive and counteractive, respectively.

For S_{DS} = 0.89 and assuming ρ = 1.0 (both discussed previously), the design load combinations are as stipulated in ASCE 7:

$$1.2D + 1.0E + 0.5L + 0.2S = 1.38D + 1.0Q_E + 0.5L + 0.2S$$

and

$$0.9D - 1.0E = 0.72D - 1.0Q_E$$

10.1.2.2.11 Deflection and Drift Limits (Provisions Sec. 5.2.8 [4.5.1])

Assuming that interior and exterior finishes have not been designed to accommodate story drifts, then the allowable story drift is (*Provisions* Table 2.5.8 [4.5-1]):

$$\Delta_a = 0.020\ h_{sx}$$

where interstory drift is computed from story drift as (*Provisions* Eq. 5.4.6.1 [5.2-15]):

$$\Delta_x = \delta_x - \delta_{x-1} = \frac{C_d\left[\delta_{xe} - \delta_{(x-1)e}\right]}{I}$$

where C_d is the deflection amplification factor, I is the occupancy importance factor, and δ_{xe} is the total elastic deflection at Level x.

10.1.2.3 Basic Gravity Loads

Roof:	
Live/Snow load (in Seattle, snow load governs over roof live load; in other areas this may not be the case)	= 25 psf
Dead load (including roofing, sheathing, joists, insulation, and gypsum ceiling)	= 15 psf
Floor:	
Live load	= 40 psf
Dead load (1½ in. lightweight concrete, sheathing, joists, and gypsum ceiling. At first floor, omit ceiling but add insulation.)	= 20 psf
Interior partitions, corridor walls (8 ft high at 11 psf) load	= 7 psf distributed floor
Exterior frame walls (wood siding, plywood sheathing, 2×6 studs, batt insulation, and 5/8-in. gypsum drywall)	= 15 psf of wall surface
Exterior double glazed window wall	= 9 psf of wall surface
Party walls (double-stud sound barrier)	= 15 psf of wall surface
Stairways	= 20 psf of horiz. projection

Perimeter footing (10 in. by 1 ft-4 in.) and grade beam (10 in. by 3 ft-2 in.) = 562 plf

Corridor footing (10 in by 1 ft-4 in.) and grade wall (8 in. by 1 ft-3 in.); 18 in. minimum crawl space under first floor = 292 plf

Applicable seismic weights at each level

W_{roof} = area (roof dead load + interior partitions + party walls) + end walls + longitudinal walls = 182.8 kips

$W_3 = W_2$ = area (floor dead load + interior partitions + party walls) + 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 crawl space with concrete foundation walls is quite stiff compared to the superstructure.

10.1.3 SEISMIC FORCE ANALYSIS

The analysis is performed manually following a step-by-step procedure for determining the base shear (*Provisions* Sec. 5.4.1 [5.2.1]), and the distribution of vertical (*Provisions* Sec. 5.4.3 [5.2.3]) and horizontal (*Provisions* Sec. 5.4.4 [5.2.4]) shear forces. Since there is no basic irregularity in the building mass, the horizontal distribution of forces to the individual shear walls is easily determined. These forces need only be increased to account for accidental torsion (see subsequent discussion).

No consideration is given to soil-structure interaction since there is no relevant soil information available; a common situation for a building of this size and type. The soil investigation ordinarily performed for this type of structure is important, but is not generally focused on this issue. Indeed, the cost of an investigation sufficiently detailed to permit soil-structure interaction effects to be considered, would probably exceed the benefits to be derived.

10.1.3.1 Period Determination and Calculation of Seismic Coefficient (*Provisions* Sec. 5.4.1 [5.2.1])

Using the values for S_{D1}, S_{DS}, R, and I from Sec. 10.1.2.1, the base shear is computed per *Provisions* Sec. 5.4.1 [5.2.1]. The building period is based on *Provisions* Eq. 5.4.2.1-1 [5.2-6]:

$$T_a = C_r h_n^x = 0.237$$

where C_r = 0.020, h_n = 27 ft, and x = 0.75.

According to *Provisions* Eq. 5.4.1.1-1 [5.2-2]:

$$C_s = \frac{S_{DS}}{R/I} = \frac{0.89}{6.5/1.0} = 0.137$$

but need not exceed *Provisions* Eq. 5.4.1.1-2 [5.2-3]:

$$C_s = \frac{S_{D1}}{T(R/I)} = \frac{0.47}{(0.237)(6.5/1.0)} = 0.305$$

Although it would probably never govern for this type of structure, also check minimum value according to *Provisions* Eq. 5.4.1.1-3:

$$C_s = 0.044IS_{DS} = (0.044)(1.0)(0.89) = 0.039$$

[This minimum C_s value has been removed in the 2003 *Provisions*. In its place is a minimum C_s value for long-period structures, which is not applicable to this example.]

The calculation of actual T as based on the true dynamic characteristics of the structure would not affect C_s; thus, there is no need to compute the actual period because the *Provisions* does not allow a calculated period that exceeds C_uT_a where C_u = 1.4 (see *Provisions* Sec. 5.4.2 [5.2.2]). Computing the base shear coefficient, per *Provisions* Eq. 5.4.1.1-1 [5.2-2], using this maximum period, would give C_s = 0.218, which still exceeds 0.137. Therefore, short period response governs the seismic design of the structure, which is common for low-rise buildings.

10.1.3.2 Base Shear Determination

According to *Provisions* Eq. 5.4.1 [5.2-1]:

$$V = C_sW = 0.137(751) = 103 \text{ kips (both directions)}$$

where effective total weight is W = 751 kips as computed previously.

10.1.3.3 Vertical Distribution of Forces

Forces are distributed as shown in Figure 10.1-5, where the story forces are calculated according to *Provisions* Eq. 5.4.3-1 and 5.4.3-2 [5.2-10 and 5.2-11]:

$$F_x = C_{vx}V = \left(\frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \right) V$$

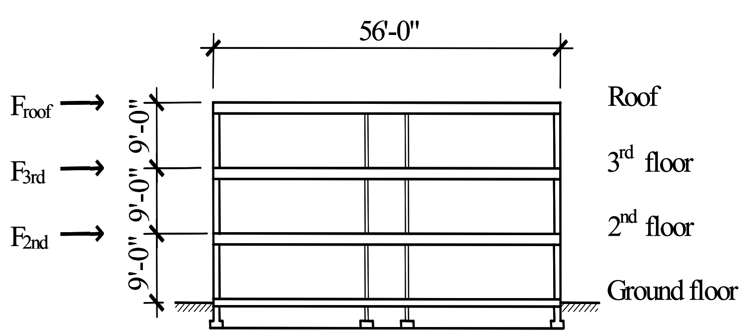

Figure 10.1-5 Vertical shear distribution (1.0 ft = 0.3048 m).

For $T < 0.5$, $k = 1.0$ and $\sum w_i h_i^k = 182.2(27) + 284.2(18) + 284.2(9) = 12{,}610$

F_{roof} = [182.8(27)/12,608](103.2) = 40.4 kips
F_{3rd} = [284.2(18)/12,608](103.2) = 41.9 kips
F_{2nd} = [284.2(9)/12,608](103.2) = 20.9 kips
Σ = 103 kips

It is convenient and common practice, to perform this calculation along with the overturning moment calculation. Such a tabulation is given in Table 10.1-1.

Table 10.1-1 Seismic Coefficients, Forces, and Moments

Level x	W_x (kips)	h_x (ft)	$w_x h_x^k$ (k = 1)	C_{vx}	F_x (kips)	V_x (kips)	M_x (ft-kips)
Roof	182.8	27	4,936	0.391	40.4		
						40.4	364
3	284.2	18	5,115	0.406	41.9		
						82.3	1,104
2	284.2	9	2,557	0.203	20.9		
						103.2	2,033
Σ	751.2		12,608				

1.0 ft = 0.3048 m, 1.0 kip = 4.45 kN, 1.0 ft-kip= 1.36 kN-m.

10.1.3.4 Horizontal Distribution of Shear Forces to Walls

Since the diaphragms are defined as “rigid” by the *Provisions* (Sec 5.2.3.1 [4.3.2.1] and 12.4.1.1), the horizontal distribution of forces must account for relative rigidity of the shear walls, and horizontal torsion must be included. As discussed below, for buildings with a Type 1a or 1b torsional irregularity (per *Provisions* Sec. 5.2.3.2 [4.3.2.2]) the torsional amplification factor (*Provisions* Sec. 5.4.4.3 [5.2.4.3]) must be calculated.

It has been common practice for engineers to consider wood diaphragms as flexible, regardless of the relative stiffness between the walls and the diaphragms. Under the flexible diaphragm assumption, loads are distributed to shear walls based on tributary area, without taking diaphragm continuity, or relative wall rigidity into account. Recognizing that diaphragm stiffness should not be ignored (even for wood structural panel sheathing), the *Provisions* provides limits on when the flexible diaphragm assumption may be used and when it may not be used.

The calculation of horizontal force distribution for rigid diaphragms, can be significantly more laborious than the relatively simple tributary area method. Therefore, this example illustrates some simplifications that can be used as relatively good approximations (as confirmed by using more detailed calculations). The design engineer is encouraged to verify all simplifying assumptions that are used to approximate rigid diaphragm force distribution.

For this example, forces are distributed as described below.

10.1.3.4.1 Longitudinal Direction

Based on the rigid diaphragm assumption, force is distributed based on the relative rigidity of the longitudinal walls, and the transverse walls are included for resisting torsion. By inspection, however, the center of mass coincides with the center of rigidity and , thus, the torsional demand is just the accidental torsional moment resulting from a 5 percent eccentricity of force from the center of mass (*Provisions* Sec. 5.4.4.2 [5.2.4.2]).

In this direction, there are four lines of resistance and the total torsional moment is relatively small. Although the walls are unequal in length, the horizontal distribution of the forces can be simplified by making two reasonable assumptions. First, for plywood shear walls, it is common to assume that stiffness is proportional to net in-plane length of sheathing (assuming sheathing thickness, nailing, and chord elements are roughly similar). Second, for this example, assume that all of the torsional moment is resisted by the end walls in the transverse directions. This is a reasonable assumption because the walls have a greater net length than the longitudinal exterior shear walls and are located much farther from the center of rigidity (and thus contribute more significantly to the rotational resistance).

Therefore, direct shear is distributed in proportion to wall length and torsional shear is neglected. Each exterior wall has 45 ft net length and each corridor wall has 55 ft net length for a total of 200 ft of net shear wall. The approximate load to each exterior wall is $(45/200)F_x = 0.225F_x$, and the load to each corridor wall is $(55/200)F_x = 0.275F_x$. (The force distribution was also computed using a complete rigidity model, including accidental torsion, with all transverse and longitudinal wall segments. The resulting distribution is $0.231F_x$ to the exterior walls and $0.276\ F_x$ to the corridor walls, for a difference of 2.7 percent and 0.4 percent, respectively).

10.1.3.4.2 Transverse Direction

Again, based on the rigid diaphragm assumption, force is to be distributed based on relative rigidity of the transverse walls, and the longitudinal walls are included for resisting torsion because the center of mass does not coincide with the center of rigidity. The torsional demand must be computed. Assuming that all six transverse wall segments have the same rigidity, the distance from the center of rigidity (*CR*) to the center of mass (*CM*) can be computed as:

$$CM - CR = 148/2 - \frac{4+60+144}{3} = 4.67\text{ft}$$

The accidental torsional moment resulting from a 5 percent eccentricity of force from the center of mass (*Provisions* Sec. 5.4.4.2 [5.2.4.2]) must also be considered.

As in the longitudinal direction, the force distribution in the transverse direction can be computed with reasonable accuracy by utilizing a simplified model. This simplification is made possible largely because the transverse wall segments are all of the same length and, thus, the same rigidity (assuming nailing and chord members are also the same for all wall segments). First, although the two wall segments at each line of resistance are offset in plan (Figure 10.1-4), assume that the wall segments do align, and are located at the centroid of the two segments. Second, assume that the longitudinal walls do not resist the torsional moment. Third, since interior transverse wall segments are located close to the center of rigidity, assume that they do not contribute to the resistance of the torsional moment.

To determine the forces on each wall, split the seismic force into two parts: that due to direct shear and that due to the torsional moment. Because their rigidities are the same, the direct shear is resisted by all six wall segments equally and is computed as $0.167F_x$. The torsional moment is resisted by the four end wall segments. Including accidental torsion, the total torsional moment is computed as:

Total Torsional Moment, $M_t + M_{ta} = F_x \times (4.67 \text{ ft} + 0.05 \times 148 \text{ ft}) = 12.07F_x$
Torsional Shear to End Walls = $12.07F_x/140 \text{ ft} = 0.086F_x$

Therefore, the simplified assumption yields a total design force of $0.167F_x + 0.086F_x/2 = 0.210F_x$, at each of the end wall segments on the right side of Figure 10.4-1, $0.167F_x - 0.086F_x/2 = 0.124F_x$ at each of the end wall segments on the left side of the figure, and $0.167F_x$ at each interior wall segment.

Next, determine the relationship between the maximum and average story drifts, to determine if a torsional irregularity exists, as defined in *Provisions* Table 5.2.3.2 [4.3-2]. Assuming that all of the shear walls will have the same plywood and nailing, the deflection of each wall will be proportional to the force in that wall. Therefore, the maximum drift for any level, will be proportional to $0.210F_x$, and the average drift proportional to $(0.210F_x + 0.124F_x)/2 = 0.167F_x$. The ratio of maximum to average deflection is $0.210F_x/0.167F_x = 1.26$. Since the ratio is greater than 1.2, the structure is considered to have a torsional irregularity Type 1a, in accordance with *Provisions* Table 5.2.3.2 [4.3-2]. Therefore, the horizontal force distribution must be computed again using the torsional amplification factor, A_x, from *Provisions* Sec. 5.4.4.3 [5.2.4.3]. (Diaphragm connections and collectors, not considered in this example, must satisfy *Provisions* Sec. 5.2.6.4.2. [4.6.3.2])

In accordance with *Provisions* Eq 5.4.4.3-1 [5.2-13]:

$$A_x = \left[\frac{\delta_{max}}{1.2\delta_{avg}}\right]^2 = \left[\frac{0.210F_x}{1.2(0.167F_x)}\right]^2 = 1.10$$

Therefore, recompute the torsional moment and forces as:

Total torsional moment, $A_x(M_t + M_{ta}) = 1.10[F_x \times (4.67 \text{ ft} + 0.05 \times 148 \text{ ft})] = 13.28F_x$
Torsional shear to End walls = $13.28F_x/140 \text{ ft} = 0.095F_x$

The revised simplified assumption yields a maximum total design force of $0.167F_x + 0.095F_x/2 = 0.214F_x$ at the end walls (right side of Figure 10.4-1), which will be designed in the following sections.

(The force distribution was also computed using a complete rigidity model including all transverse and longitudinal wall segments. Relative rigidities were based on length of wall and the torsional moment was resisted based on polar moment of inertia. The resulting design forces to the shear wall segments were $0.208F_x$, $0.130F_x$, and $0.168F_x$ for the right end, left end, and interior segments, respectively. The structure is still torsionally irregular ($\delta_{max} / \delta_{ave} = 1.23$), but the irregularity is less substantial and the resulting $A_x = 1.05$. This more rigorous analysis, has shown that the simplified approach provides reasonable results in this case. Since the simplified method is more likely to be used in design practice, the values from that approach will be used for the remainder of this example.)

10.1.3.5 Verification of Redundancy Factor

Once the horizontal force distribution is determined, the assumed redundancy factor must be verified. *Provisions* Eq. 5.2.4.2, is used to compute the redundancy factor, ρ, for each story as:

$$\rho_x = 2 - \frac{20}{r_{max_x}\sqrt{A_x}}$$

where A_x is the floor area in square feet and r_{max_x} is the ratio of the design story shear resisted by the single element carrying the most shear force in the story to the total story shear. As defined for shear walls in *Provisions* Sec. 5.2.4.2, r_{max_x} shall be taken as the shear in the most heavily loaded wall, multiplied by $10/l_w$, where l_w is the wall length in feet.

The most heavily loaded wall is the end wall, farthest away from the interior shear wall. Since the force distribution is the same for all three levels, the redundancy factor need only be determined for one level, in this case, at the first floor. The redundancy factor is computed as:

Floor area = (140)(56) = 7,840 ft^2
Max load to wall = 0.214V
Wall length = 25 ft
$r_{max_x} = 0.214V(10/25)/V = 0.0856$

Therefore:

$$\rho_x = 2 - \frac{20}{0.0856\sqrt{7,840}} = -0.64$$

Because the calculated redundancy factor is less than the minimum permitted value of 1.0, the initial assumption of ρ = 1.0 is correct, and the design can proceed using the previously computed lateral forces.

[See Sec. 10.1.2.2 for discussion of the changes to the redundancy requirements in the 2003 *Provisions*.]

10.1.3.6 Diaphragm Design Forces

As specified in *Provisions* Sec. 5.2.6.4.4 [4.6.3.4], floor and roof diaphragms must be designed to resist a force, F_{px}, in accordance with *Provisions* Eq. 5.2.6.4.4 [4.6-2]:

$$F_{px} = \frac{\sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} w_i} w_{px}$$

plus any force due to offset walls (not applicable for this example). The diaphragm force as computed above need not exceed $0.4S_{DS}Iw_{px}$, but shall not be less than $0.2S_{DS}Iw_{px}$. This latter value often governs at the lower levels of the building, as it does here. The maximum required diaphragm demand does not govern in this example.

The weight tributary to the diaphragm, w_{px}, 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.

Transverse direction

Roof Level
$\Sigma F_i = 40.4$
$\Sigma w_i = 182.8$
$F_{P,roof} = (40.4/182.8)(182.8)$ = 40.4 kips (controls for roof)
$0.2S_{DS}Iw_{px} = (0.2)(0.89)(1.0)(182.8)$ = 32.7 kips

Third Floor
$\Sigma F_i = 40.4 + 41.9 = 82.3$
$\Sigma w_i = 182.8 + 284.2 = 447.0$
$F_{P,3rd}$ =(82.3/447.0)(284.2) = 50.1 kips
$0.2S_{DS}Iw_{px}$ = (0.2)(0.89)(1.0)(284.2) = 50.8 kips (controls for 3rd floor)

Second Floor
$\Sigma F_i = 40.4 + 41.9 + 20.9 = 103{,}231$
$\Sigma w_i = 182.8 + 284.2 + 284.2 = 751.2$
$F_{P,2nd}$ = (103.2/751.2)(284.2) = 39.1 kips
$0.2S_{DS}Iw_{px}$ = (0.2)(0.89)(1.0)(284.2) = 50.8 kips (controls for 2nd floor)

Diaphragm forces in the longitudinal direction are computed in a similar manner. Since the weight of the exterior walls is more significant in the longitudinal direction, the designer may wish to subtract this weight from the story force in order to compute the diaphragm demands.

10.1.4 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. In doing so, some successive iteration of steps may be needed to satisfy the deflection limits.

Nailing patterns in diaphragms and shear walls have been established on the basis of tabulated requirements included in the *Provisions*. 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, *Provisions* Sec. 12.4.1.2 [12.4.1.1] places aspect ratio limits on plywood diaphragms (length/width shall not exceed 4/1 for blocked diaphragms), and *Provisions* Sec. 12.4.2.3 [12.4.2.3] places similar limits on shear walls (height/width shall not exceed 2/1 for full design capacities). However, it should be taken into consideration that compliance with these aspect ratios does not guarantee that the drift limits will be satisfied.

Therefore, diaphragms and shear walls have been analyzed for deflection as well as for shear capacity. A procedure for computing diaphragm and shear wall deflections is provided in *Commentary* Sec. 12.4.1. This procedure is illustrated below. The *Commentary* does not indicate how to compute the nail deformation (nail slip) factor, but there is a procedure contained in the commentary of ASCE16.

In the calculation of diaphragm deflections, the chord slip factor can result in large additions to the total deflection. This can be overcome by using “neat” holes for bolts, and proper shimming at butt joints. However, careful attention to detailing and field inspection are essential, to ensure that they are provided.

[AF&PA Wind & Seismic also contains procedures for computing diaphragm and shear wall deflections. The equations are slightly different from the more commonly used equations that appear in the *Commentary* and AF&PA LRFD *Manual*. In AF&PA Wind & Seismic, the shear and nail slip terms are combined using an “apparent shear stiffness” parameter. However, the apparent shear stiffness values are only provided for OSB. Therefore, the deflection equations in the *Commentary* or AF&PA LRFD *Manual* must be used in this example which has plywood diaphragms and shear walls. The apparent shear stiffness values for plywood will likely be available in future editions of AF&PA Wind & Seismic.]

10.1.4.1 Strength of Members and Connections

The 2000 *Provisions* has adopted Load and Resistance Factor Design (LRFD) for engineered wood structures. The *Provisions* includes the ASCE 16 standard by reference and uses it as the primary design procedure for engineered wood construction. Strength design of members and connections is based on the requirements of ASCE 16. The AF&PA Manual and supplements contain reference resistance values for use in design. For convenience, the *Provisions* contain design tables for diaphragms and shear walls that are identical to those contained in the AF&PA Structural-Use Panels Supplement. However, the modification of the tabulated design resistance for shear walls and diaphragms with framing other than Douglas Fir-Larch or Southern Pine is different between the two documents. This example illustrates the modification procedure contained in the *Provisions* tables, which is new to the 2000 edition.

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 ASCE 16 and the AF&PA Manual; with the exception of shear walls and diaphragms for which design resistance values are taken from the *Provisions*. The LRFD standard incorporates the notation *D*', *T*', *Z*', etc. to represent adjusted resistance values, which are then modified by a time effect factor, λ, and a capacity reduction factor, ϕ, to compute a design resistance, which is defined as "factored resistance" in the *Provisions* and ASCE 16. Additional discussion on the use of LFRD is included in *Commentary* Sec. 12.2 and 12.3, and in the ASCE 16 commentary. It is important to note that ACSE 16 and the AF&PA Manual use the term, "resistance," to refer to the design capacities of members and connections while "strength" refers to material property values.

It is worth noting that the AF&PA Manual contains a Pre-engineered Metal Connections Guideline for converting allowable stress design values for cataloged metal connection hardware (for example, tie-down anchors) into ultimate capacities for use with strength design. This procedure is utilized in this example.

[The primary reference for design of wood diaphragms and shear walls in the 2003 *Provisions* is AF&PA Wind & Seismic. Much of the remaining text in the 2003 *Provisions* results from differences between AF&PA Wind & Seismic and Chapter 12 of the 2000 *Provisions* as well as areas not addressed by AF&PA Wind & Seismic. Because the AF&PA Wind & Seismic tabulated design values for diaphragms and shear walls do not completely replace the tables in the 2000 *Provisions*, portions of the tables remain in the 2003 *Provisions*. Therefore, some diaphragm and shear wall design values are in the 2003 *Provisions* and some are in AF&PA Wind & Seismic. The design values in the tables are different between the two documents. The values in the 2003 *Provisions* represent factored shear resistance ($\lambda\varphi D'$), while the values in AF&PA Wind & Seismic represent nominal shear resistance that must then be multiplied by a resistance factor, φ, (0.65) and a time effect factor ,λ, (1.0 for seismic loads). Therefore, while the referenced tables may be different, the factored resistance values based on the 2003 *Provisions* should be the same as those in examples based on the 2000 *Provisions*. The calculations that follow are annotated to indicate from which table the design values are taken.]

10.1.4.2 Transverse Shear Wall Nailing

The design will focus on the more highly loaded end walls; interior walls are assumed to be similar.

10.1.4.2.1 Load to Any One of Four 25-ft End Walls

F_{roof} = 0.214(40.4) = 8.65 kips
F_{3rd} = 0.214(41.9) = 8.97 kips
F_{2nd} = 0.214(20.9) = 4.47 kips
Σ = 22.09 kips

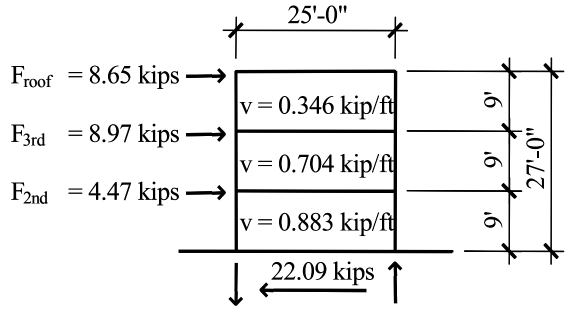

Figure 10.1-6 Transverse section: end wall (1.0 ft = 0.3048 m, 1.0 kip = 4.45 kN, 1.0 kip/ft = 14.6 kN/m).

10.1.4.2.2 Roof to Third Floor Design

V = 8.65 kips
v = 8.65/25 = 0.346 klf

Try a ½-in. (15/32) plywood rated sheathing (not Structural I) on blocked 2-in. Douglas fir-Larch members at 16 in. on center with 10d common nails at six-in. on center at panel edges and 12 in. on center at intermediate framing members. According to Note b of *Provisions* Table 12.4.3-2a [12.4-3a], the design shear resistance 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 Table 12A of the AF&PA Structural Connections Supplement, the SG = 0.43 for Hem-fir. Therefore, the adjustment factor is 1-(0.5-0.43) = 0.93.

From *Provisions* Table 12.4.3-2a [AF&PA Wind/Seismic Table 4.3A], $\lambda\phi D'$ = 0.40 klf
Adjusted shear resistance = 0.93(0.40) = 0.37 klf > 0.346 klf OK

8d nails could be used at this level but, because 10d nails are required below, 10d nails are used here for consistency.

Deflection of plywood shear panels depends, in part, on the slip at the nails which, in turn, depends on the load per nail. See Sec. 10.1.4.3 for a detailed discussion of the appropriate slip for use with the *Provisions* and see Table 10.1-2 for fastener slip equations used here.

Load per nail = 0.346(6/12)(1000) = 173 lb
Nail slip $e_n = 1.2(173/769)^{3.276}$ = 0.00904 in.

In the above equation, 1.2 = factor for plywood other than Structural I, and 769 and 3.276 are constants explained in Sec. 10.1.4.3.

[As indicated previously, AF&PA Wind & Seismic does not provide apparent shear stiffness values for plywood sheathing. Therefore, the deflection equations in the *Commentary* or AF&PA LRFD *Manual* must be used in this case.]

10.1.4.2.3 Third Floor to Second Floor

V = 8.65 + 8.97 = 17.62 kips

v = 17.62/25 = 0.704 klf

Try ½-in. (15/32) plywood rated sheathing (not Structural I) on blocked 2-in. Douglas fir-Larch members at 16 in. on center with 10d nails at 3-in. on center at panel edges, and at 12 in. on center at intermediate framing members.

From *Provisions* Table 12.4.3-2a [AF&PA Wind&Seismic Table 4.3A], $\lambda\phi D'$ = 0.78 klf
Adjusted shear resistance = 0.93(0.78) = 0.73 klf > 0.704 klf OK

Table Note e [AF&PA Wind&Seismic Sec. 4.3.7.1] requires 3-in. framing at adjoining panel edges.

Load per nail = 0.704(3/12)(1000) = 176 lb
Nail slip $e_n = 1.2(176/769)^{3.276}$ = 0.00960 in.

10.1.4.2 Second Floor to First Floor

V = 17.62 + 4.47 = 22.09 kips
v = 22.09/25 = 0.883 klf

Try a ½-in. (15/32) plywood rated sheathing (not Structural I) on blocked 2-in. Douglas fir-Larch members at 16 in. on center with 10d common nails at 2-in. on center at panel edges and 12 in. on center at intermediate framing members.

From *Provisions* Table 12.4.3-2a [AF&PA Wind&Seismic Table 4.3A], $\lambda\phi D'$ = 1.00 klf
Adjusted shear resistance = 0.93(1.00) = 0.93 klf > 0.883 klf OK

Table Notes d and e [AF&PA Wind&Seismic Sec. 4.3.7.1]require 3-in. framing at adjoining panel edges.

Load per nail = 0.883(2/12)(1000) = 147 lb
Nail slip $e_n = 1.2(147/769)^{3.276}$ = 0.00534 in.

10.1.4.3 General Note on the Calculation of Deflections for Plywood Shear Panels

The commonly used expressions for deflection of diaphragms and shear walls, which are contained in *Commentary* Sec. 12.4.1, and the ASCE 16 commentary standard (see also UBC Std 23-2 or APA 138), include a term for nail slip. The ASCE 16 commentary includes a procedure for estimating nail slip that is based on LRFD design values.

The values for e_n used in this example (and in Sec. 10.2) are calculated according to Table 10.1-2, which is taken from Table C9.5-1 of ASCE 16.

Table 10.1-2 Fastener Slip Equations

Fastener	Minimum Penetration (in.)	For Maximum Loads Up to (lb.)	Approximate Slip, e_n*	
			Green/Dry	Dry/Dry
6d common nail	1-1/4	180	$(V_n/434)^{2.314}$	$(V_n/456)^{3.144}$
8d common nail	1-7/16	220	$(V_n/857)^{1.869}$	$(V_n/616)^{3.018}$
10d common nail	1-5/8	260	$(V_n/977)^{1.894}$	$(V_n/769)^{3.276}$
14-ga staple	1 to 2	140	$(V_n/902)^{1.464}$	$(V_n/596)^{1.999}$
14-ga staple	2	170	$(V_n/674)^{1.873}$	$(V_n/361)^{2.887}$

*Fabricated green/tested dry (seasoned); fabricated dry/tested dry. V_n = fastener load in pounds. Values based on Structural I plywood fastened to Group II lumber. Increase slip by 20 percent when plywood is not Structural I.

1.0 in. = 25.4 mm, 1.0 lb = 4.45 N.

This example is based on the use of surfaced dry lumber so the "dry/dry" values are used. The appearance of the equations for both shear wall and diaphragm deflection, implies greater accuracy than is justified. The reader should keep in mind that the deflections calculated are only rough estimates.

10.1.4.4 Transverse Shear Wall Deflection

From *Commentary* Sec. 12.4, modified as described below, shear wall deflection is computed as:

$$\delta = \frac{8vh^3}{wEA} + \frac{vh}{Gt} + 0.75he_n + \frac{h}{w}d_a$$

The above equation produces displacements in inches and the individual variables must be entered in the force or length units as described below:

$v = V/w$ where V is the total shear on the wall and v is in units of pounds/foot

$8vh3/wEA$ = bending deflection, as derived from the formula $\delta_b = Vh^3/3EI,$ where V is the total shear in pounds, acting on the wall and $I = Aw^2/2$ (in.^{4})

vh/Gt = shear deflection, as derived from the formula $\delta_v = Vh/GA'$, where $A' = wt$ (in.^{2})

0.75 he_n = nail slip, in inches. Note that with h being given in feet, the coefficient 0.75 carries units of 1/ft.

$(h/w)d_a$ = deflection due to anchorage slip, in inches. Note that for use in the deflection equation contained in the *Commentary*, the term d_a represents the vertical deflection due to anchorage details. In the deflection equation contained in the commentary in the AF&PA Manual, there is no h/w factor, so it should be assumed that the term d_a represents the horizontal deflection at the top of the wall due to anchorage details.

For this example:

E = 1,600,000 psi
G = 75,000 psi
A (area) = 2(2.5)5.5 = 27.5 in.^{2} for assumed double 3×6 end posts

w (shear wall length) = 25 ft
h (story height) = 9 ft
t (effective thickness) = 0.298 in. for ½-in. unsanded plywood (not Structural I)
e_n = nail deformation factor from prior calculations, inches.

This equation is designed for a one-story panel and some modifications are in order for a multistory panel. The components due to shear distortion and nail slip are easily separable (see Table 10.1-3).

Table 10.1-3 Wall Deflection (per story) Due to Shear and Nail Slip

Story	v (plf)	vh/Gt (in.)	e_n (in.)	$0.75e_nh$ (in.)
Roof	346	0.139	0.00904	0.061
3	704	0.284	0.00960	0.065
2	883	0.356	0.00534	0.036

1.0 in. = 25.4 mm, 1.0 plf = 14.6 N/m.

Likewise, the component due to anchorage slip is easily separable; it is a rigid body rotation. If a 1/8-in. upward slip is assumed (on the tension side only), the deflection per story is (9/25)(1/8) = 0.045 in. (Table 10.1-4).

The component due to bending is more difficult to separate. For this example, a grossly simplified, distributed triangular load on a cantilever beam is used (Figure 10.1-7).

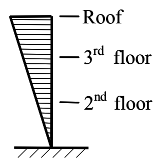

Figure 10.1-7 Force distribution for flexural deflections.

The total load V is taken as 21.3 kips, the sum of the story forces on the wall. The equation for the deflection, taken from Roark's *Formulas for Stress and Strain*, is:

$$\delta_x = \frac{2V}{5EAb^2h^2}(11h^5 - 15h^4x + 5hx^4 - x^5)$$

where x is the distance (ft) from the top of the building to the story in question, h is the total height (ft), b is the wall width (ft), A is the chord cross sectional area (in.^{2}), E is the modulus of elasticity (psi), and the resulting displacement, δ, is in inches.

This somewhat underestimates the deflections, but it is close enough for design. The results are shown in Table 10.1-4.

The total wall deflections, shown in Table 10.1-5, are combined with the diaphragm deflections (see Sec. 10.1.4.7, 10.1.4.8, and 10.1.4.9). Drift limits are checked after diaphragm deflections are computed.

Table 10.1-4 Wall Deflection (per story) Due to Bending and Anchorage Slip

Level	Effective $8vh^3/wEA$ (in.)	$(h/w)d_a$ (in.)
Roof	0.031	0.045
3	0.027	0.045
2	0.012	0.045

1.0 in. = 25.4 mm.

Table 10.1-5 Total Elastic Deflection and Drift of End Wall

Level	Shear (in.)	Bending (in.)	Nail Slip (in.)	Anchor Slip (in.)	Drift Δ_e (in.)	Total δ_e (in.)
Roof	0.139	0.031	0.061	0.045	0.277	1.145
3	0.284	0.027	0.065	0.045	0.420	0.869
2	0.356	0.012	0.036	0.045	0.448	0.448

1.0 in. = 25.4 mm.

10.1.4.5 Transverse Shear Wall Anchorage

Provisions Sec. 12.4.2.4 [12.4.2.4.1] 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 indicated in *Provisions* Eq. 5.2.7-2 [4.2-2].

Traditionally, tie-down devices were designed to resist this net overturning demand. However, *Provisions* Sec. 12.4.2.4 [12.4.2.4.1] requires that the nominal strength of tie-down devices exceeds the expected maximum vertical force that the shear panels can deliver to the tension-side end post. Specifically, the nominal strength ($\phi = 1.0$) of the tie-down device must be equal to, or greater than the net uplift forces resulting from $\Omega_0/1.3$ times the factored shear resistance of the shear panels, where Ω_0 is the system overstrength factor. These uplift forces are cumulative over the height of the building. Also, dead load forces are not used in these calculations to offset the uplift forces for the design of the tie-down anchorage.

[The requirements for uplift anchorage are slightly different in the 2003 *Provisions*. The tie down force is based on the "nominal strength of the shear wall" rather than the $\Omega_0/1.3$ times the factored resistance as specified in the 2000 *Provisions*. This change is primarily intended to provide consistent terminology and should not result in a significant impact on the design of the tie down.]

An additional requirement of *Provisions* Sec. 12.4.2.4 [12.4.2.4.1], is that end posts must be sized such that failure across the net section does not control the capacity of the system. That is, the tensile strength of the net section must be greater than the tie-down strength. Note that the tie-downs designed in the following section are not located at wall intersections where the directional combination requirements of *Provisions* Sec. 5.2.5.2.3 [4.4.2.3] would apply (see also the "Analysis Procedure" in Sec. 10.1.2.2).

10.1.4.5.1 Tie-down Anchors at Third Floor

For the typical 25-ft end wall segment, the overturning moment at the third floor is:

$$M_0 = 9(8.65) = 77.8 \text{ ft-kip} = Q_E$$

The counter-balancing moment, 0.72 Q_D = 0.72D(25 ft/2). The width of the floor contributing to this, is taken as half the span of the exterior window header equal to 6.5 feet (see Figures 10.1-1 and 10.1-2). For convenience, the same length is used for the longitudinal walls, the weight of which (interior and exterior glazed wall) is assumed to be 9 psf.

End wall self weight = 9 ft (25ft)15 psf/1000	= 3.4 kips
Tributary floor = 6.5 ft (25 ft) (15) psf/1000	= 2.4 kips
Tributary longitudinal walls = 9 ft (6.5 ft) 9 psf (2)/1000	= <u>1.1 kips</u>
Σ	= 6.9 kips

$0.72Q_D = 0.72(6.9)12.5 = 62.1$ ft-kip

M_0 (net) = 77.8 - 62.1 = 15.7 ft-kip

Therefore, uplift anchorage is required. Using the procedure described above, nominal tie-down strength must be equal to or exceed the tension force, T, computed as:

T = 0.37 klf (3/1.3)(8ft) = 6.83 kips

where

0.37 klf = factored shear wall resistance at the third floor
3/1.3 = $\Omega_0/1.3$
8 ft = net third floor wall height (9-ft story height minus approximately 1 ft of framing).

Use a double tie-down device to eliminate the eccentricity associated with a single tie-down. Also, use the same size end post over the full height of the wall to simplify the connections and alignments. As will be computed below, a 6×6 (Douglas fir-Larch) post is required at the first floor. At the third floor, try two sets of double tie-down anchors connected through the floor with a 5/8-in. threaded rod to the end posts with two 5/8-in. bolts similar to Figure 10.1-8.

According to *Provisions* Sec. 12.4.2.4 [12.4.2.4.1], the nominal tie-down strength is defined as the "maximum test load the device can resist under cyclic testing without connection failure by either metal or wood failure." [Note that this definition for nominal tie down strength has been removed in the 2003 *Provisions*. This change is primarily intended to provide consistent terminology and should not result in a significant impact on the design of the tie down.] For a single tie-down device as described above, the cataloged allowable uplift capacity is 2.76 kips and the cataloged average ultimate load is 12.15 kips. However, according to the documentation in this particular supplier's product catalog, the ultimate values are based on static testing and the type of failure is not indicated. Therefore, for this example, the cataloged ultimate value is not considered to satisfy the requirements of the *Provisions*. As an alternate approach, this example will utilize the methodology contained in the AF&PA Manual for converting cataloged allowable stress values to strength values.

Based on the procedure in the AF&PA Manual, Pre-Engineered Metal Connectors Guide, the strength conversion factor for a connector for which the catalog provides an allowable stress value with a 1.33 factor for seismic is 2.88/1.33.

Since the typical product catalogs provide design capacities only for single tie-downs, the design of double tie-downs requires two checks. First, consider twice the capacity of one tie-down, and second, the capacity of the bolts in double shear. ϕ = 1.0 for both these calculations.

For the double tie-down, the nominal strength is computed as:

$2\lambda\phi Z'$ = 2(1.0)(1.0)(2.76)(2.88/1.33) = 12.0 kips > 6.83 kips OK

For the two bolts through the end post in double shear, the AF&PA Manual gives:

$2\lambda\phi Z'$ = 2(1.0)(1.0)(4.90) = 9.80 kips > 6.83 kips OK

The factored capacity of the tie-downs must also be checked for the design loads, which in this case will not govern the design.

10.1.4.5.2 Tie-down Anchors at Second Floor

Since tie-downs are required at the third floor, it would be common practice to provide tie-downs at the second and first floors, whether or not calculations indicate that they are required. Nevertheless, the overturning calculations are performed for illustrative purposes. The overturning moment at the second floor is:

M_0 = 18(8.65) + 9(8.97) = 236 ft-kip.

The counter-balancing moment, 0.72 Q_D = 0.72(DL)12.5.

End wall self weight = 18 ft (25 ft) 15 psf/1000	= 6.75 kips
Tributary floor = 6.5 ft (25 ft) (15 + 27) psf/1000	= 6.83 kips
Tributary longitudinal walls = 18 ft (6.5 ft) 9 psf (2)/1000	= 2.10 kips
Σ	= 15.7 kips

$0.72Q_D$ = 0.72(15.7)12.5 = 141 ft-kips

M_0 (net) = 236-141 = 95 ft-kips

As expected, uplift anchorage is required. The design uplift force is computed using an adjusted factored shear resistance of 0.73 klf at the second floor, and a net length of wall height equal to 8 ft. Note that 8 ft. is appropriate for this calculation given the detailing for this structure. As shown in Figure 10.1-10, the plywood sheathing is not detailed as continuous across the floor framing, which results in a net sheathing height of about 8ft. If the sheathing were detailed across the floor framing, then 9 ft would be the appropriate wall height for use in computing tie-down demands. Combined with the uplift force at the third floor, the total design uplift force at the second floor is:

T = 6.83 kips + 0.73 klf (3/1.3)(8ft) = 20.3 kips

Use two sets of double tie-down anchors to connect the 6×6 end posts. Using the same procedure as for the third floor, tie-downs with a seven-eighths-in. threaded rod and three seven-eighths-in. bolts are computed to be adequate. See Figure 10.1-8.

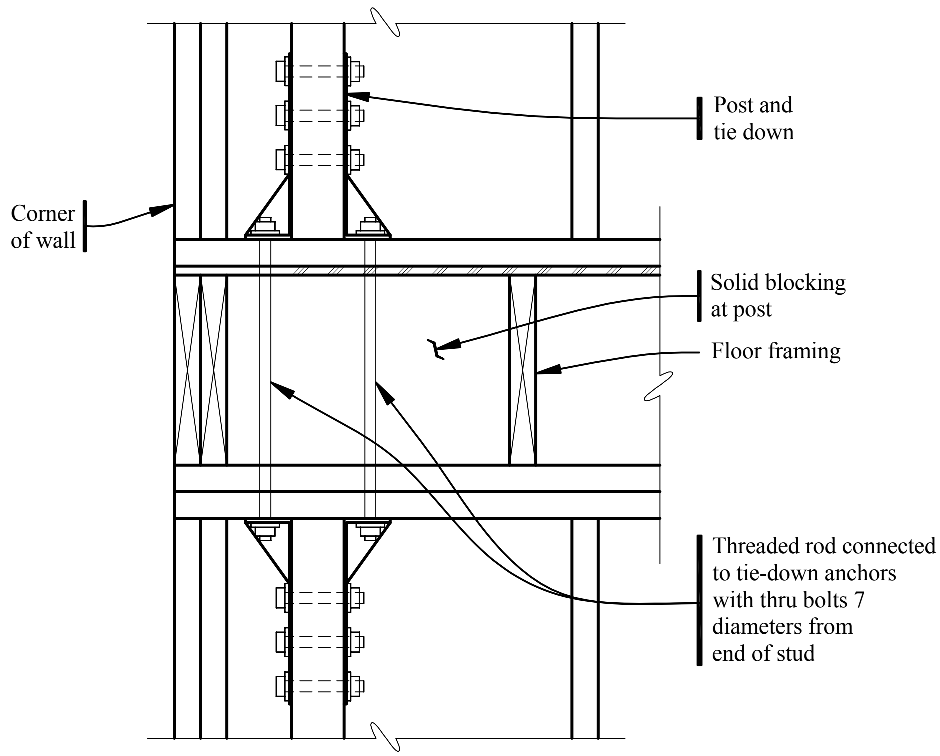

Figure 10.1-8 Shear wall tie down at suspended floor framing.

10.1.4.5.3 Tie-down Anchors at First Floor

The overturning moment at the first floor is:

M_0 = 27(8.65) + 18(8.97) + 9(4.47) = 435 ft-kip = $Q_{E.}$

The counter-balancing moment, 0.72 Q_D = 0.72D(25 ft/2).

End wall self weight = 27 ft (25ft)15 psf/1000	= 10.1 kips
Tributary floor = 6.5 ft (25 ft) (15 + 27 + 27) psf/1000	= 11.2 kips
Tributary longitudinal walls = 27 ft (6.5 ft) 9 psf (2)/1000	= 3.2 kips
Σ	= 24.5 kips

0.72Q_D = 0.72(24.5)12.5 = 221 ft-kip

M_0 (net) = 435 - 221 = 214 ft-kip

As expected, uplift anchorage is required. The design uplift force is computed using an adjusted factored shear resistance of 0.93 klf at the first floor, and a net wall height of 8ft. Combined with the uplift force at the floors above, the total design uplift force at the first floor is:

T = 20.3 kips + 0.93 klf (3/1.3)(8ft.) = 37.5 kips.

Use a double tie-down anchor that extends through the floor with an anchor bolt into the foundation. Tie-downs with a 7/8-in. threaded rod, and four 7/8-in. bolts are adequate.

The strength of the end post, based on failure across the net section, must also be checked (*Provisions* Sec. 12.4.2.4 [12.4.2.4.1]). For convenience, the same size post has been used over the height of the building, so the critical section is at the first floor. A reasonable approach to preclude net tension failure from being a limit state would be to provide an end post, whose factored resistance exceeds the nominal strength of the tie-down device. (Note that using the factored resistance rather than nominal strength of the end post provides an added margin of safety that is not explicitly required by the *Provisions*.) The nominal strength of the first floor double tie-down is 42.9 kips, as computed using the procedure described above. Therefore, the tension capacity at the net section must be greater than 42.9 kips.

Try a 6×6 Douglas Fir-Larch No. 1 end post. In some locations, the shear wall end post also provides bearing for the window header, so this size is reasonable. Accounting for 1-in. bolt holes, the net area of the post is 24.75 in.^{2} According to the AF&PA Manual, Structural Lumber Supplement:

$\lambda\phi T' = (1.0)(0.8)(2.23\text{ksi})(24.75\text{in}^2) = 44.2 \text{ kips} > 42.9 \text{ kips}$ OK

For the maximum compressive load at the end post, combine maximum gravity load, plus the seismic overturning load. In the governing condition, the end post supports the header over the glazed portion of the exterior wall (end wall at right side of Figure 10.1-1). Assume that the end post at the exterior side of the wall supports all the gravity load from the header, and resists one-half of the seismic overturning load.

Compute gravity loads based on a 6.5-foot tributary length of the header:

Tributary DL = ((27 ft)(9 psf) + (8 ft)(15 + 27 + 27)psf)(6.5ft)/1000 = 5.17 kips
Tributary LL = 8 ft (7 ft) (40 + 40) psf/1000 = 4.48 kips
Tributary SL = 8 ft (6.5 ft) (25 psf)/1000 = 1.30 kips

The overturning force is based on the seismic demand (not wall capacity as used for tension anchorage) assuming a moment arm of 23 ft:

Overturning, Q_E = 435/23 ft = 18.9 kips

Per load combination associated with *Provisions* Eq. 5.2.7.1-1 [4.2-1]:

Maximum compression = 1.38(5.17) + 1.0(18.9/2) + 0.5(4.48) + 0.2(1.30) = 19.1 kips.

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

$\lambda\phi P' = (1.0)(0.8)(1.30\text{ksi})(30.25 \text{ in.}^2) = 31.5 \text{ kips} > 19.1 \text{ kips}.$ OK

The 6×6 end post is slightly larger than the double 3×6 studs assumed above for the shear wall deflection calculations. Therefore, the computed shear wall deflection is slightly conservative, but the effect is minimal.

10.1.4.5.4 Check Overturning at the Soil Interface

A summary of the overturning forces is shown in Figure 10.1-9. To compute the overturning at the soil interface, the overturning moment must be increased for the 4-ft foundation height:

M_0 = 435 + 22.09(4.0) = 523 ft-kip

However, it then may be reduced in accordance with *Provisions* Sec. 5.3.6:

M_0 = 0.75(523) = 392 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 ft (6.5 ft) (27 - 4 + 1)psf /1000 = 3.9 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 carried by minimum nominal reinforcement in the foundation.

Foundation weight = (562 plf (13 ft + 25 ft) + 292 plf (13 ft))/1000	= 25.1 kips
First floor	= 3.9 kips
Structure above	= 24.5 kips
Σ = 53.5 kips	

Therefore, $0.72D - 1.0Q_E$ = 0.72(53.5)12.5 ft - 1.0(392) = 89.5 ft-kips, which is greater than zero, so the wall will not overturn.

10.1.4.5.5 Anchor Bolts for Shear

At the first floor:

v = 0.883 klf.

A common anchorage for non-engineered construction is a ½-in. bolt at 4ft-0 in. For a ½-in. bolt in a Douglas fir-Larch 3×6 plate, in single shear, parallel to the grain:

$\lambda\phi Z'$/4ft = (1.0)(0.65)(2.38 kips)/4 = 0.39 klf < 0.883 klf NG

Try a larger bolt and tighter spacing. For this example, use a 5/8-in. bolt at 32 in. on center:

$\lambda\phi Z'$/2.67ft = (1.0)(0.65)(3.72 kips)/2.67 = 0.91 klf > 0.883 klf OK

Provisions Sec. 12.4.2.4 [12.4.2.4.2]requires plate washers at all shear wall anchor bolts. A summary of the transverse shear wall elements is shown in Figure 10.1-9.

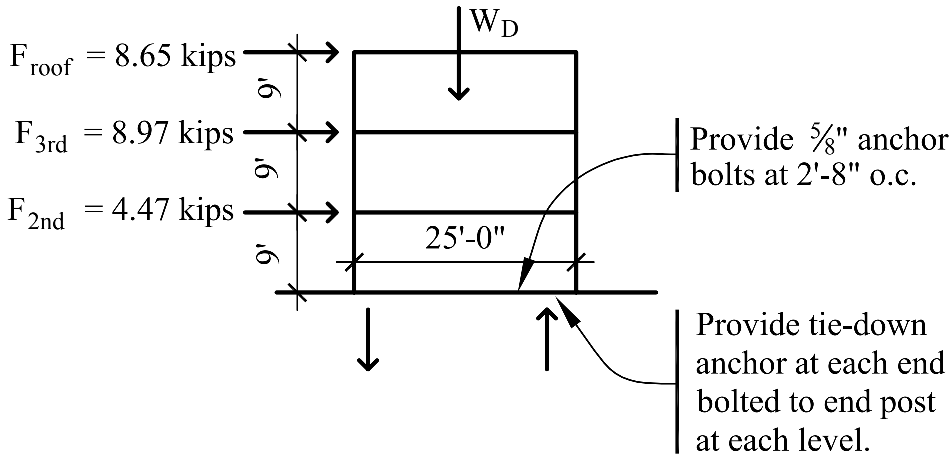

Figure 10.1-9 Transverse wall: overturning
(1.0 ft = 0.3048 m, 1.0 in. = 25.4 mm, 1.0 kip = 4.45 kN).

10.1.4.6 Remarks on Shear Wall Connection Details

In normal 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 Sec. 10.1.4.5 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 nine-ft with about one-ft occupied by the floor framing. Using standard 8-ft-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 10.1-10 and 10.1-11 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.

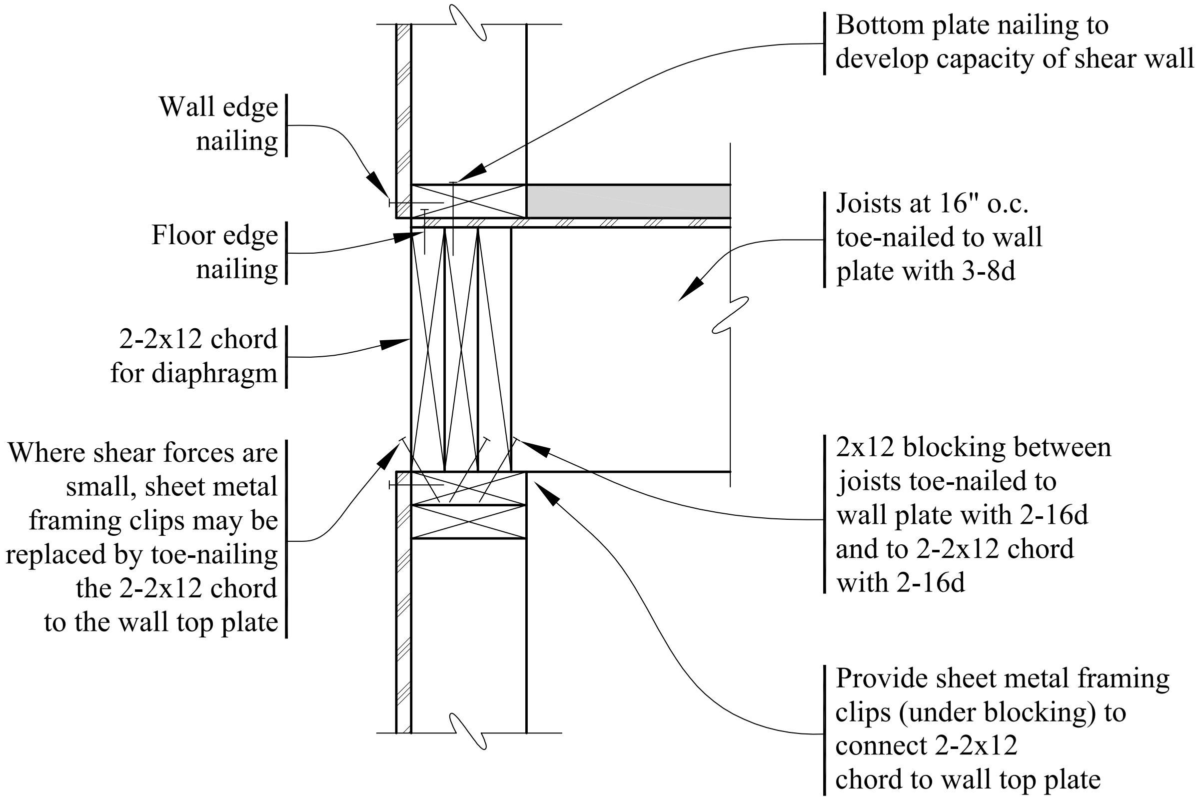

Figure 10.1-10 Bearing wall (1.0 in = 25.4 mm).

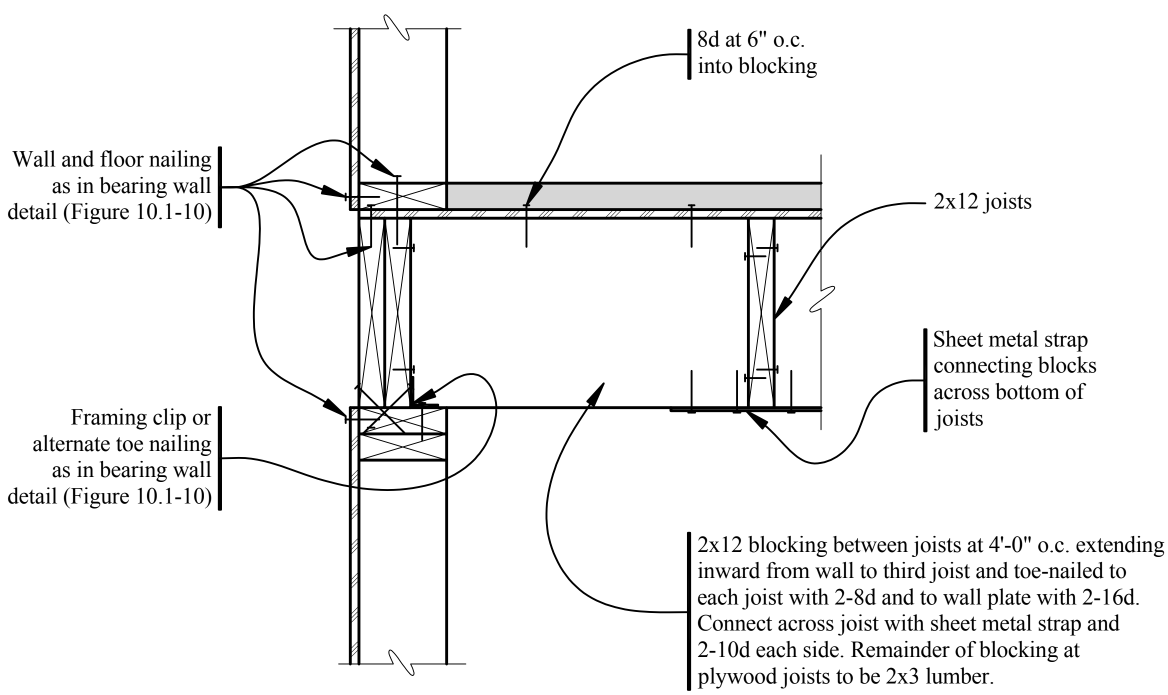

Figure 10.1-11 Nonbearing wall (1.0 in. = 25.4 mm).

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, using 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. Although some designers in the past may have used a short tier of plywood nailed to the plates of the stud walls to accomplish the transfer, *Provisions* Sec. 12.4.2.6 prohibits this type of detailing.

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, allow them to function as collectors to move the force from the full length of the diaphragm to the discrete shear walls. (According to *Provisions* Sec. 5.2.6.4.1 [4.6.2.2], the design of collector elements in wood shear wall buildings in Seismic Design Category D 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 10.1-11. (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 10.1-12).

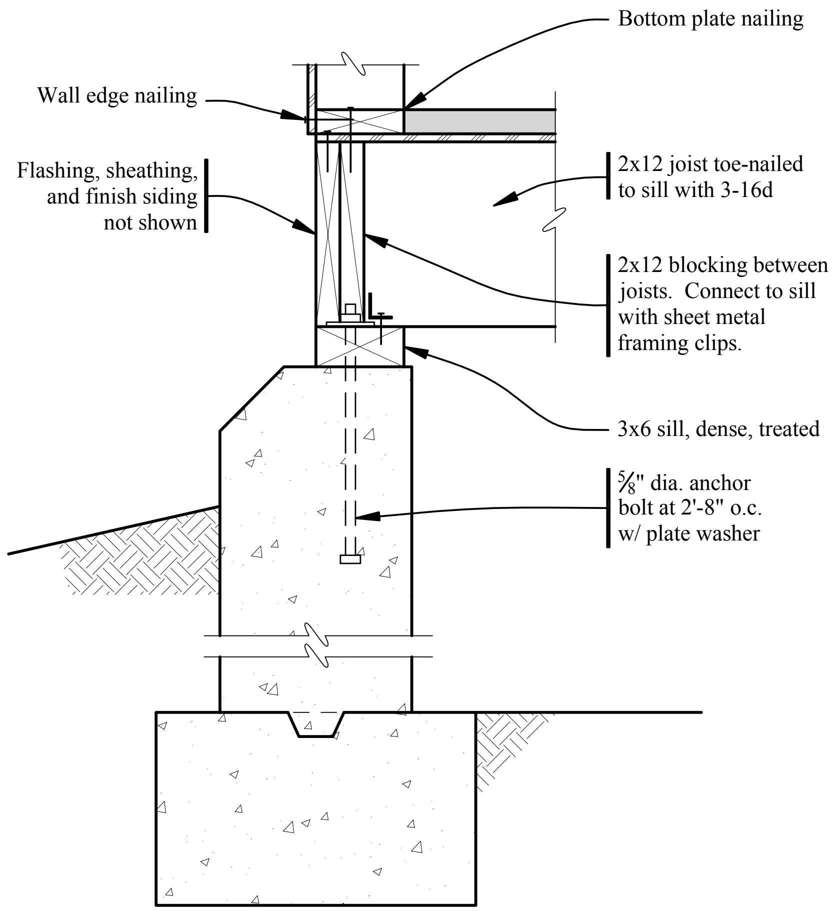

Figure 10.1-12 Foundation wall detail (1.0 in. = 25.4 mm).

The particular combinations of nails and bent steel framing clips shown in Figures 10.1-10, 10.1-11, and 10.1-12, to accomplish the necessary force transfers, are not the only possible solutions. A great amount of leeway exists for individual preference, as long as the load path has no gaps. Common carpentry practices often will provide most of the necessary transfers but, just as often, a critical few will be missed. As a result, careful attention to detailing and inspection is an absolute necessity.

10.1.4.7 Roof Diaphragm Design

While it has been common practice to design plywood diaphragms as simply supported beams spanning between shear walls, the diaphragm design for this example must consider the continuity associated with rigid diaphragms. The design will be based on the maximum shears and moments that occur over the entire diaphragm. From Sec. 10.1.3.5, the diaphragm design force at the roof is, $F_{P,roof}$ = 40.4 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.

10.1.4.7.1 Diaphragm Nailing

The maximum diaphragm shear occurs at the end walls. From Sec. 10.1.3.4, each 25-ft end wall segment resists 21 percent of the total story (diaphragm, in this case) force. Distributing the diaphragm force at the same rate, the diaphragm shear over the entire diaphragm width at the end walls is:

V = (0.214)(2)(40.4) = 17.3 kips
v = 17.3/56 ft. = 0.308 klf

Try ½-in. (15/32) plywood rated sheathing (not Structural I) on blocked 2-in. Douglas fir-Larch members at 16 in. on center, with 8d nails at 6 in. on center at panel edges and 12 in. on center at intermediate framing members.

From *Provisions* Table 12.4.3-1a [AF&PA Wind&Seismic Table 4.2A], $\lambda\phi D'$ = 0.35 klf > 0.308 klf OK

The determination of nail slip for diaphragms is included below.

10.1.4.7.2 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 = 40.4 kips /140 ft = 0.289 klf, the maximum positive moment is:

$0.289(84)^2 / 8$ = 255 ft-kip

and the maximum negative moment is:

$$\frac{0.289(84)^3 + 0.289(56)^3}{8(84+56)} = 198 \text{ ft-kip}$$

The positive moment controls, and the design chord force is 255/56 = 4.55 kips. Try a double 2×12 Douglas fir-Larch No. 2 chord. Due to staggered splices, compute the tension capacity based on a single 2×12, with a net area of A_n = 15.37 in.^{2} (accounting for 1-in. bolt holes). Per the AF&PA Manual, Structural Lumber Supplement:

$\lambda\phi T'$ = (1.0)(0.8)(1.55psi)(15.37in^2) = 19.1 kips > 4.55 kips OK

For chord splices, use 4 in. diameter split-ring connectors with 3/4-in. bolts. For split rings is single shear, the capacity of one connector is:

$\lambda\phi T'$ = (1.0)(0.65)(17.1) = 11.1 kips > 4.55 kips. OK

This type of chord splice connection, shown in Figure 10.1-13, is generally used only for heavily loaded chords and is shown here for illustrative purposes. A typical chord splice connection for less heavily loaded chords can be accomplished more easily by using 16d nails to splice the staggered chord members.

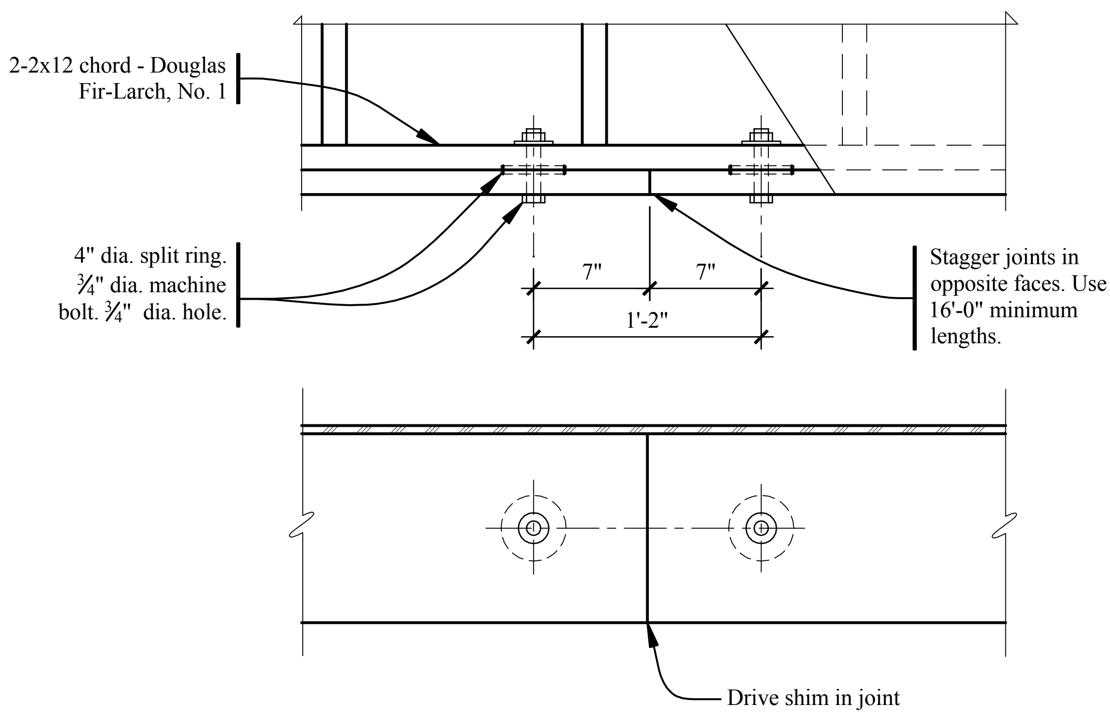

Figure 10.1-13 Diaphragm chord splice (1.0 ft = 0.3048 m, 1.0 in. = 25.4 mm).

10.1.4.7.3 Diaphragm Deflection

The procedure for computing diaphragm deflections, contained in *Commentary* Sec. 12.4 (similar to the commentary of ASCE 16), is intended for the single span, "flexible" diaphragm model that has been used in common practice. The actual deflections of multiple span "rigid" diaphragms may, in general, be similar to those of single-span diaphragms because shear deflection and nail slip (both based on shear demand) tend to dominate the behavior. As multiple span deflection computations tend to be cumbersome, it is suggested that the design engineer compute diaphragm deflections based on the single span model. This will result with a reasonable (and conservative), estimation of overall displacements. If these displacements satisfy the drift criteria, then the design is assumed to be adequate; if not, then a more rigorous computation of displacements could be performed.

It is the authors' opinion that for diaphragm displacement computations, the diaphragm loading should be according to *Provisions* Eq. 5.2.6.4.4 [5.2-11]; the minimum design demand ($0.2S_{DS}Iw_{px}$) need not be considered. Although the *Provisions* does not provide a specific requirement either way, this interpretation is consistent with *Provisions* Sec. 5.4.6.1 [5.2.6.1], which indicates that the minimum base shear equation (*Provisions* Eq. 5.4.1.1-3) need not be used for calculation of story drift.

[See Sec. 10.1.3.1 for a discussion of diaphragm deflection computation using the 2003 Provisions and AF&PA Wind & Seismic reference.]

From *Commentary* Sec. 12.4, the diaphragm deflection is computed as:

$$\delta = \frac{5vL^3}{8wEA} + \frac{vL}{4Gt} + 0.188Le_n + \frac{\Sigma(\Delta_c X)}{2w}.$$

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. For the single span approximation, consider the longer, 84 ft long span so that:

$$v = \frac{F_{px}(84/140)}{2w}$$

where F_{px} is the story diaphragm force at Level x, w is the diaphragm width, and v is in pounds/ft. For this calculation, the unit shear will be the same at both ends of the diaphragm. Therefore, it underestimates the actual unit shear at the end wall but overestimates the actual unit shear at the interior wall. These inaccuracies are assumed to be roughly offsetting. The individual terms of the above equation represent the following:

$5vL^3/8wEA$ = bending deflection, as derived from the formula $\delta_b = 5vL^4/384EI$, where v is the diaphragm unit force in pounds per ft and $I = Aw^2/2$ (in^2)

$vL/4Gt$ = shear deflection as derived from the formula $\delta_v = vL^2/8GA$, where v is the diaphragm unit force in pounds per ft and $A = wt$ (in^2)

$0.188Le_n$ = nail slip deflection in inches

$\Sigma(\Delta_c X)/2w$ = deflection due to chord slip in inches

For this example:

v = 40.4(84/140)/[(2(56))(1000)] = 216 plf (ignoring torsion)
L = 84 ft and w = 56 ft, diaphragm length and width
A = 2(1.5) 11.25 = 33.75 in.^{2} (double 2×12)
t = 0.298 in. for ½-in. unsanded plywood
E = 1,600,000 psi
G =75,000 psi

Nail slip is computed using the same procedure as for shear walls (Sec. 10.1.4.3), with the load per nail based on the diaphragm shear from this section. The diaphragm nailing is 8d at 6 in. on center.

Load per nail = 216(6/12) = 108 lb
Nail slip e_n = 1.2(108/616)$^{3.018}$ = 0.0063 in.

In the above equation, 1.2 is the factor for plywood other than Structural I and 616 and 3.018 are coefficients for seasoned lumber.

Although a tight chord splice has been specified, the chord slip is computed for illustrative purposes. Assume chord slip occurs only at the side tension since the compression splices are tightly shimmed as shown in Figure 10.1-13. For this example:

Δ_c = chord splice slip (1/16 in. used for this example)
X = distance from chord splice to nearest support

Assuming splices at 20 ft on center, along the 84-ft diaphragm length (ignore diaphragm continuity for this term), the sum of the chord splice slip is:

$\Sigma(\Delta_c X)$ = (1/16)(20 + 40 + 24 + 4) = 5.5

Thus:

$$\delta = \frac{5(216)(84^3)}{8(1,600,000)(33.75)(56)} + \frac{216(84)}{4(75,000)(0.298)} + 0.188(84)(0.0063) + \frac{5.5}{2(56)}$$

$$= 0.027 + 0.203 + 0.100 + 0.055 = 0.385 \text{ in.}$$

Wall and floor drifts are added, and checked in Sec. 10.1.4.9.

10.1.4.8 Second and Third Floor Diaphragm Design

The design of the second and third floor diaphragms follows the same procedure as the roof diaphragm. From Sec. 10.1.3.6, the diaphragm design force for both floors is $F_{p,3rd} = F_{p,2nd}$ = 50.8 kips.

10.1.4.8.1 Diaphragm Nailing

The maximum diaphragm shear occurs at the end walls. From Sec. 10.1.3.4, each end wall segment resists 21 percent of the total story (diaphragm, in this case) force. Distributing the diaphragm design force at the same rate, the diaphragm shear at the end walls is:

V = (0.214)(2)(50.8) = 21.7 kips
v = 21.7/56 ft = 0.388 klf

Try ½-in. (15/32) plywood rated sheathing (not Structural I) on blocked 2-in. Douglas fir-Larch members at 16 in. on center, with 8d nails at 4 in. on center at boundaries and continuous panel edges, at 6 in. on center at other panel edges, and 12 in. on center at intermediate framing members.

From *Provisions* Table 12.4.3-1a [AF&PA Wind&Seismic Table 4.2A], $\lambda\varphi D'$ = 0.47 klf > 0.388 klf OK

10.1.4.8.2 Chord and Splice Connection

Computed as described above for the roof diaphragm, the maximum positive moment is 320 kips and the design chord force is 5.71 kips.

By inspection, a double 2×12 chord spliced with 4-in. diameter split ring connectors, as at to the roof level, is adequate. A typical chord splice connection is shown in Figure 10.1-13.

10.1.4.8.3 Diaphragm Deflection

Using the same procedure as before:

L = 84 ft and w = 56 ft, diaphragm length and width
A = 2(1.5) 11.25 = 33.75 in.^{2} (double 2×12)
t = 0.298 in. for ½-in. unsanded plywood
E = 1,600,000 psi
G = 75,000 psi

As discussed previously, diaphragm deflection computations need not consider the minimum diaphragm design forces.

Therefore, at the third floor:

v = 50.1(84/140)/[(2(56))(1000)] = 268 plf
Load per nail = 268(4/12) = 89 lb
Nail slip e_n = 1.2(89/616)$^{3.018}$ = 0.0035 in.

$$\delta = \frac{5(268)(84^3)}{8(1,600,000)(33.75)(56)} + \frac{268(84)}{4(75,000)(0.298)} + 0.188(84)(0.0035) + \frac{5.5}{2(56)}$$

= 0.033 + 0.252 + 0.056 + 0.055 = 0.396 in.

At the second floor:

v = 39.1(84/140)/(2(56))(1000) = 209 plf
Load per nail = 209(4/12) = 70 lb
Nail slip e_n = 1.2(70/616)$^{3.018}$ = 0.0017 in.

$$\delta = \frac{5(209)(84^3)}{8(1,600,000)(33.75)(56)} + \frac{209(84)}{4(75,000)(0.298)} + 0.188(84)(0.0017) + \frac{5.5}{2(56)}$$

= 0.026 + 0.197 + 0.026 + 0.055 = 0.304 in.

10.1.4.9 Transverse Deflections, Drift, and P-delta Effects

Transverse deflections for walls and diaphragms were calculated above. The diaphragm deflections for the second and third floors are based on the seismic force analysis and not the minimum diaphragm design forces (*Provisions* Sec. 5.2.6.4.4 [5.2.3]).

To determine the maximum story deflections and drifts (see Section 10.1.2.2), the midspan diaphragm deflection is combined with the wall deflection. This is summarized in Table 10.1-6, which shows the drift below the level considered.

Table 10.1-6 Total Deflection and Drift

Level	Wall (in.)	Diaphragm (in.)	Total δ_e (in.)	$\delta = C_d\delta_e/I$ (in.)	Δ, drift (in.)

Roof	1.14	0.38	1.52	6.08	1.00
3	0.87	0.40	1.27	5.08	2.08
2	0.45	0.30	0.75	3.00	3.00

1.0 in. = 25.4 mm.

The maximum permissible drift is 0.020 h_s = 2.16 in. Therefore, the drift limitations are satisfied at the third floor and roof. The drift in the first story is about 38 percent too large so the design must be revised.

One beneficial aspect of the design that has been ignored in these calculations is the lightweight concrete floor fill. Although it is discontinuous at the stud walls, it will certainly stiffen the diaphragm. No studies that would support a quantitative estimate of the effect have been found. Because the shear deformation of the ½-in. plywood diaphragm represents a significant portion of the total drift at the first and second levels, any significant increase in shear stiffness that might be provided by the concrete would further reduce the expected drifts. In this case, about 60 percent of the drift is contributed by the shear walls. The diaphragm stiffness would need to be tripled to meet the drift criteria, so the shear walls will be revised.

If Structural I plywood is used for the shear walls in this direction, the drift criteria are satisfied at all levels. In Sec. 10.1.4.4, *G* becomes 90,000 ksi, and *t* (effective thickness) becomes 0.535 in. The deflections due to shear and nail slip (the 1.2 factor no longer applies) are reduced. The resulting total elastic deflection of the shear walls at Levels 2, 3, and Roof are, 0.242 in., 0.488 in., and 0.669 in., respectively. Table 10.1-6b shows the revised results, which satisfy the drift criteria.

Table 10.1-6b Total Deflection and Drift (Structural I Plywood Shear Walls)

Level	Wall (in.)	Diaphragm (in.)	Total δ_e (in.)	$\delta = C_d\delta_e/I$ (in.)	Δ, drift (in.)
Roof	0.67	0.38	1.05	4.20	0.64
3	0.49	0.40	0.89	3.56	1.40
2	0.24	0.30	0.54	2.16	2.16

1.0 in. = 25.4 mm.

Because the tie-down calculations (Sec. 10.1.4.5) depend on the tabulated capacities of the shear wall panels and Structural I panels have slightly higher capacities, the connection designs must be verified. Additional calculations (not shown here) confirm that the hardware that was previously selected still works. It is also worth noting that although many designers fail to perform deflection calculations for wood construction, the drift criteria control the selection of sheathing grade in this example.

The P-delta provision also must be examined. Following *Provisions* Sec. 5.4.6.2 [5.2.6.2] and assuming the total mass deflects two-thirds of the maximum diaphragm deflection, the P-delta coefficient is as shown in Table 10.1-7.

[Note that the equation to determine the stability coefficient has been changed in the 2003 *Provisions*. The importance factor, *I*, has been added to 2003 *Provisions* Eq. 5.2-16. However, this does not affect this example because *I* = 1.0.]

Table 10.1-7 P-delta Stability Coefficient

Level	P_D (kips)	P_L (kips)	ΣP (kips)	Δ (in.)	V (kips)	$\theta = P\Delta/VhC_d$

Roof	183	204	387	0.67	40.4	0.015
3	284	130	801	1.27	82.3	0.029
2	284	130	1,215	1.76	103.2	0.048

1.0 in. = 25.4 mm, 1.0 kip = 4.45 kN.

For example, to compute the effective P-delta drift at the roof:

$\delta_{roof} = C_d[\delta_{wall} + 2/3(\delta_{diaphragm})] = 4[0.67 + 2/3(0.38)] = 3.693$ in.
$\delta_3 = 4[0.49 + 2/3(0.40)] = 3.027$ in.
$\delta = 3.693 - 3.027 = 0.67$ in.

The story dead load is the same as shown in Table 10.1-1, and the story live load is based on 25 psf snow load for the roof and 16 psf reduced live load (0.4×40 psf) acting over the entire area of Levels 2 and 3. For $\theta < 0.10$, no deflection amplification due to P-delta effects is necessary.

10.1.4.10 Longitudinal Direction

Only one exterior shear wall section will be designed here. The design of the corridor shear walls would be similar to the transverse walls. (This example has assumed a greater length of corridor wall than would likely be required to resist the design forces. This increased length is intended to reduce the demand to the exterior walls, assuming rigid diaphragm distribution, to a level below the maximum permitted in-plane shear for the perforated shear wall design procedure as discussed below.) For loads in the longitudinal direction, diaphragm stresses and deflections are negligible.

The design of the exterior wall will utilize the guidelines for perforated shear walls (*Provisions* Sec. 12.4.3 [AF&PA Wind&Seismic Sec. 4.3]), which are new to the 2000 *Provisions*. [The provisions for perforated shear walls are contained in AF&PA Wind & Seismic and therefore have been removed from the 2003 *Provisions*. The design provisions are spread throughout AF&PA Wind & Seismic Sec. 4.3.3, but are not substantially different for the provisions contained in the 2000 *Provisions* except for the revisions to the tie down requirements as noted below.] The procedure for perforated shear walls applies to walls with openings that have not been specifically designed and detailed for forces 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 (*Provisions* Sec. 12.4.3.2 [AF&PA Wind&Seismic Sec. 4.3.5.2]), the most relevant to this example is that the factored design shear resistance shall not exceed 0.64 klf. This requirement essentially limits the demand on perforated shear walls such that the required factored design shear resistance is less than 0.64 klf. If the configuration required higher design values, then walls must be added in order to reduce the demand.

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 shall 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 with 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 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 $0.225F_x$. The exterior shear walls are composed of three separate perforated shear wall segments (two at 30 ft long and one at 15 ft long, all with the same relative length of full height sheathing), as shown in Figure 10.1-2. This section will focus on the design of a 30-ft section. Assuming that load is distributed to the wall sections based on relative length of shear panel, then the total story force to the 30-ft section is $(30/75)0.225F_x = 0.090F_x$ per floor. The load per floor is

$F_{roof} = 0.090(40.4) = 3.64$ kips
$F_{3rd} = 0.090(41.9) = 3.77$ kips
$F_{2nd} = 0.090(20.9) = \underline{1.88 \text{ kips}}$
$\Sigma = 9.29$ kips

10.1.4.10.1 Perforated Shear Wall Resistance

The design shear capacity (*Provisions* Sec. 12.4.3.3 [AF&PA Wind&Seismic Table 4.3.3.4]) 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. At each level, the design shear capacity, V_{wall}, is:

$$V_{wall} = (vC_0)\Sigma L_i$$

where

v = unadjusted factored shear resistance (*Provisions* Table 12.4.3-2a [Table 12.4.3a or AF&PA Wind&Seismic Table 4.3A).

C_0 = shear capacity adjustment factor (*Provisions* Table 12.4.3-1 [AF&PA Wind&Seismic Table 4.3.3.4]) = 0.83

The percent of full height sheathing is (4 + 10 + 4)/30 = 0.60, and the maximum opening height ratio is 4 ft /8 ft = 0.5. Per *Provisions* Table 12.4.3-1 [AF&PA Wind&Seismic Table 4.3.3.4], $C_0 = 0.83$.

ΣL_i = sum of widths of perforated shear wall segments = 4 + 10 + 4 = 18 ft

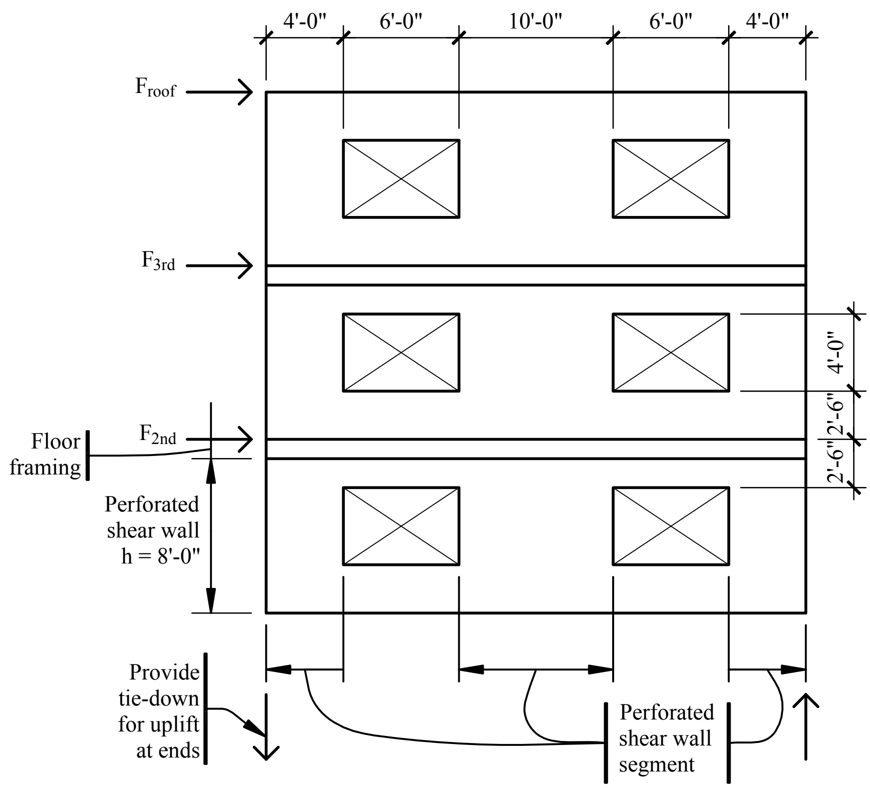

Figure 10.1-14 Perforated shear wall at exterior (1.0 ft = 0.3048 m, 1.0 in. = 25.4 mm)

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 10.1-14. Perforated shear wall plywood and nailing are determined below.

Roof to third floor:

V = 3.64 kips
Required v = 3.64/0.83/18 = 0.244 klf

Try ½-in. (15/32) Structural I plywood rated sheathing on blocked 2-in. Douglas fir-Larch members at 16 in. on center with 10d common nails at 6 in. on center at panel edges and 12 in. on center at intermediate framing members. (Structural I plywood would not be required to satisfy the strength requirements. However, to minimize the possibility for construction errors, the same grade of sheathing is used on walls in both directions.)

From *Provisions* Table 12.4.3-2a [AF&PA Wind&Seismic Table 4.3A], $\lambda\varphi D'$ = 0.44 klf > 0.244 klf OK

Third floor to second floor:

V = 3.64 + 3.77 = 7.41 kips
Required v = 7.41/0.83/18 = 0.496 klf

Try ½-in. (15/32) Structural I plywood rated sheathing on blocked 2 in. Douglas fir-larch members at 16 in. on center with 10d common nails at four in. on center at panel edges and 12 in. on center at intermediate framing members.

From *Provisions* Table 12.4.3-2a [AF&PA Wind&Seismic Table 4.3A]:

$\lambda \varphi D'$ = 0.66 klf (0.64 klf max) > 0.496 klf OK

As discussed above, *Provisions* Sec. 12.4.3.2, Item b [12.4.3], limits the factored shear resistance in *Provisions* Table 12.4.3-2a [AF&PA Wind&Seismic Table 4.3A] to 0.64 klf, which still exceeds the demand at this level, so the limitation is satisfied.

Second floor to first floor:

V = 7.41 + 1.88 = 9.29 kips
Required v = 9.29/0.83/18 = 0.622 klf

By inspection, the same plywood and nailing from above will work.

10.1.4.10.2 Perforated Shear Wall Uplift Anchorage

According to *Provisions* Sec. 12.4.3.4.1 [AF&PA Wind&Seismic Table 4.3A], the requirements for uplift anchorage must be evaluated at the ends of the wall only. Uplift at each wall segment is treated separately as described later. Uplift forces, based on the strength of the shear panels (per *Provisions* Sec. 12.4.2.4 [12.4.2.4.1]), are to be computed as discussed in Sec. 10.1.4.5. For this example, calculations involving seismic overturning and counter-balancing moments are assumed not to be applicable for perforated shear walls, as they are not expected to act as rigid bodies in resisting global overturning.

The tie-down design force is determined as $\Omega_0/1.3$ times the factored shear resistance of the shear panels, as discussed previously. For this example, the 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 10.1-8, would be used.

[The requirements for uplift anchorage are slightly different in the 2003 *Provisions*. The tie down force is based on the "nominal strength of the shear wall" rather than the $\Omega_0/1.3$ times the factored resistance as specified in the 2000 *Provisions*. This change is primarily intended to provide consistent terminology and should not result in a significant impact on the design of the tie down.]

The uplift forces are computed as:

Roof: T = 0.44 klf (3.0/1.3) (8 ft) = 8.12 kips
Third floor: T = 0.66 klf (3.0/1.3) (8 ft) = 12.18 kips
Second floor: T = 0.66 klf (3.0/1.3) (8 ft) = 12.18 kips
Σ = 32.48 kips

Since the chord member supports the window header as well, use a 6×6 Douglas fir-Larch No. 1 similar to the transverse walls. Try a double tie-down device with a 7/8-in. anchor bolt and three 7/8-in. stud bolts. Using the method described above for computing the strength of a double tie-down, the nominal design strength is 34.3 kips, which is greater than the demand of 32.48 kips.

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

10.1.4.10.3 Perforated Shear Wall Compression Chords

Provisions Sec. 12.4.3.4.4 [AF&PA Wind&Seismic Sec. 4.3.6.1], requires each end of a perforated shear wall to have a chord member designed for the following compression force from each story:

$$C = \frac{Vh}{C_0 \Sigma L_i}$$

where

V = design shear force in the shear wall (not wall capacity as used for uplift)
h = shear wall height (per floor)
C_0 = shear capacity adjustment factor
ΣL_i = sum of widths of perforated shear wall segments

For h = 8 ft, C_0 = 0.83 and ΣL_i = 18 ft, the compression force is computed as:

Third floor: C = 3.64(8)/0.83/18	= 1.95 kips
Second floor: C = (3.64 + 3.77)(8)/0.83/18	= 3.96 kips
First floor: C = (3.64 + 3.77 + 1.88)(8)/0.83/18	= 4.98 kips
Σ	= 10.89 kips

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 *Provisions* Sec. 12.4.3.4.4 [AF&PA Wind&Seismic Sec. 4.3.6.1], it is rational to combine the chord compression with gravity loading (using the load combination $1.38D + 1.0Q_E + 0.5L + 0.2S$ in accordance with *Provisions* Eq. 5.2.7.1-1 [4.2-1]), in order to design the chord member. The end post of the longitudinal shear wall supports the same tributary weight at the end post of the transverse shear walls. Using the weights computed previously in Sec. 10.1.4.5, the design compression force is:

1.38(5.17) + 1.0(10.89) + 0.5(4.48) + 0.2(1.30) = 20.5 kips.

The bearing capacity on the bottom plate was computed previously as 31.5 kips, which is greater than 20.5 kips. Where end posts are loaded in both directions, orthogonal effects must be considered in accordance with *Provisions* Sec. 5.2.5.2 [4.4.2.3].

10.1.4.10.4 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 *Provisions* Sec. 12.4.3.4.2 [AF&PA Wind&Seismic Sec. 4.3.6.4]. While this anchorage need only be provided at the full height sheathing, it is usually extended over the entire length of the perforated shear wall to simplify the detailing and reduce the possibility of construction errors.

$$v = \frac{V}{C_0 \sum L_i}$$

where

V = design shear force in the shear wall
C_0 = shear capacity adjustment factor
ΣL_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 = 0.244 klf
Second floor: v = 0.496 klf
First floor: v = 0.622 klf.

In addition to resisting the in-plane shear force, *Provisions* Sec. 12.4.3.4.3 [AF&PA Wind&Seismic Sec. 4.3.6.4] requires that the shear wall bottom plates be designed to resist a uniform uplift force, t, equal to the unit in-plane shear force. Per *Provisions* Sec. 12.4.3.4.5 [AF&PA Wind&Seismic Sec. 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:

Shear: v = 0.622 klf
Tension: t = 0.244 + 0.496 + 0.622 = 1.36 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. As for the transverse shear walls, try a 5/8-in. bolt at 32 in. on center with a 3-in. square plate washer (*Provisions* Sec. 12.4.2.4 [12.4.2.4.2] requires 1/4×3×3 in. plate washer for 5/8-in. anchor bolts). As computed previously, the shear capacity is 0.91 klf so the bolts are adequate.

For anchor bolts at 32 in. on center, the tension demand per bolt is 1.36 klf (32/12) = 3.63 kips. Bearing capacity of the plate washer (using a Douglas fir No. 2 bottom plate) is computed per the AF&PA Manual, Structural Connections Supplement, as:

$\lambda\phi P'$ = (1.0)(0.8)(1.30 ksi)(9 in.^{2}) = 9.36 kips > *3.63* kips OK

The anchor bolts themselves must be designed for combined shear, and tension in accordance with *Provisions* Sec. 9.2 [11.2].

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 10.1-15. 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.

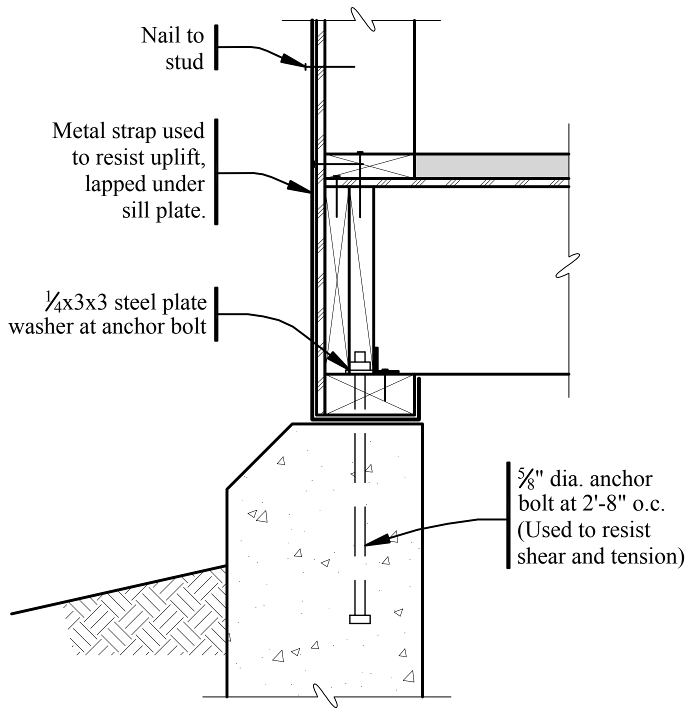

Figure 10.1-15 Perforated shear wall detail at foundation (1.0 ft = 0.3048 m, 1.0 in. = 25.4 mm).

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 10.1-16. For this type of connection (and the one shown in Figure 10.1-15) to be effective, shrinkage of the floor framing must be minimized using dry or manufactured lumber.

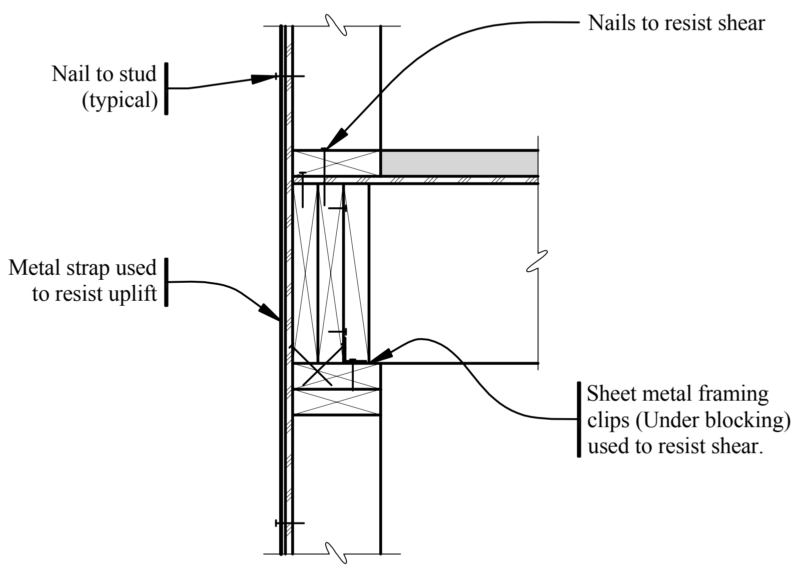

Figure 10.1-16 Perforated shear wall detail at floor framing.

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 1210d nails at each end.

10.2 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 Sec. 9.1 of this volume of design examples. Refer to that example for more detailed building information and the design of the masonry walls.

10.2.1 Building Description

This is a very simple rectangular warehouse, 100 ft by 200 ft in plan (Figure 10.2-1), with a roof height of 28 ft. The wood roof structure slopes slightly, but it is nominally flat. The long walls (side walls) are 8 in. thick and solid, and the shorter end walls are 12 in. thick and penetrated by several large openings.

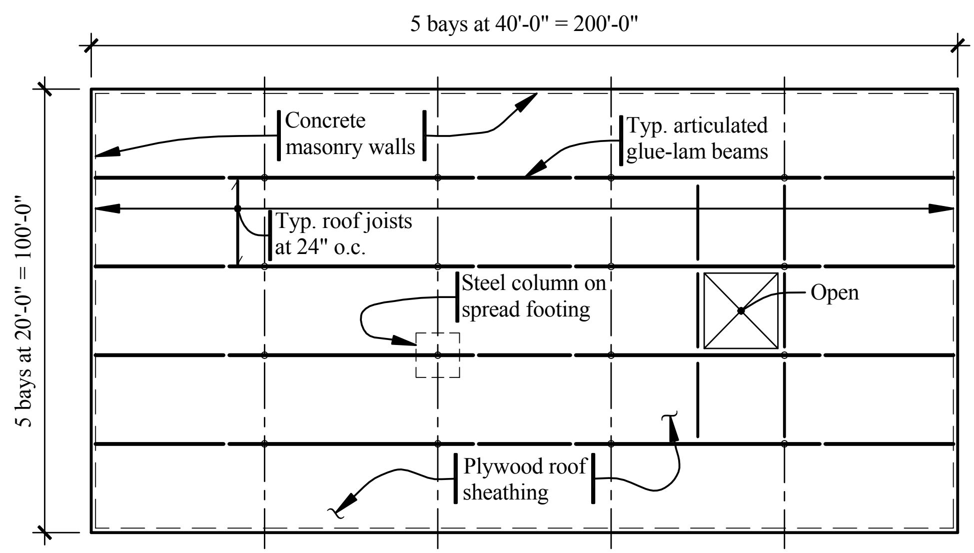

Figure 10.2-1 Building plan (1.0 ft = 0.3048 m, 1.0 in. = 25.4 mm).

Based on gravity loading requirements, the roof structure consists of wood joists, supported by 8¾-in. wide, by 24-in. deep, glued-laminated beams, on steel columns. The joists span 20 ft and the beams span 40 ft, as an articulated system. Typical roof framing is assumed to be Douglas fir-Larch No.1 as graded by the WWPA. The glued-laminated beams meet the requirements of combination 24F-V4 per ANSI/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. The details of these cross ties and the masonry wall-to-diaphragm anchorage are substantially different from those shown in previous versions of this example. This is primarily due to significant revisions to the *Provisions* requirements for anchorage of masonry (and concrete) walls to flexible wood diaphragms.

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

1. Development of diaphragm forces based on the equivalent lateral force procedure used for the masonry wall design (Sec. 9.1)
2. Design and detailing of a plywood roof diaphragm with a significant opening
3. Computation of drift and P-delta effects
4. Anchorage of diaphragm and roof joists to masonry walls and
5. Design of cross ties and subdiaphragms

[Note that as noted in Sec. 9.1, the new "Simplified Design Procedure" contained in 2003 *Provisions* Simplified Alternate Chapter 4 as referenced by 2003 *Provisions* Sec. 4.1.1 is likely to be applicable to this example, subject to the limitations specified in 2003 *Provisions* Sec. Alt. 4.1.1.]

10.2.2 Basic Requirements

10.2.2.1 *Provisions* Parameters

S_S (*Provisions* Maps [Figure 3.3.3])	= 1.50
S_1 (*Provisions* Maps [Figure 3.3.4])	= 0.60
Site Class (*Provisions* Sec. 4.1.2.1 [3.5])	= C
Seismic Use Group (*Provisions* Sec. 1.3 [1.2])	= I
Seismic Design Category (*Provisions* Sec. 4.2 [1.4])	= D
Seismic Force Resisting System (*Provisions* Table 5.2.2 [4.3-1])	= Special reinforced masonry shear wall
Response Modification Factor, R (*Provisions* Table 5.2.2 [4.3-1])	= 3.5
System Overstrength Factor, Ω_0 (*Provisions* Table 5.2.2 [4.3-1])	= 2.5
Deflection Amplification Factor, C_d (*Provisions* Table 5.2.2 4.3-1])	= 3.5

[The 2003 *Provisions* have adopted the 2002 USGS probabilistic seismic hazard maps, and the maps have been added to the body of the 2003 *Provisions* as figures in Chapter 3 (instead of the previously used separate map package).]

10.2.2.2 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 Sec. 9.1 of this volume of design examples.

10.2.2.2.1 Design and Detailing Requirements (Provisions Sec. 5.2.6 [4.6])

See *Provisions* Chapter 12, for wood design requirements. As discussed in greater detail in Sec. 10.1, *Provisions* Sec. 12.2.1 utilizes load and resistance factor design (LRFD) for the design of engineered wood structures. The design capacities are therefore, consistent with the strength design demands of *Provisions* Chapter 5.

The large opening in the diaphragm must be fitted with edge reinforcement (*Provisions* Sec. 5.2.6.2.2 [4.6.1.4]). However, the diaphragm does not require any collector elements that would have to be designed for the special load combinations (*Provisions* Sec. 5.2.6.4.1 [4.6.2.2]).

The requirements for anchoring of masonry walls to flexible diaphragms (*Provisions* Sec. 5.2.6.3.2 [4.6.2.1]) are of great significance in this example.

10.2.2.2.2 Combination of Load Effects (Provisions Sec. 5.2.7 [4.2.2])

The basic design load combinations for the lateral design, as stipulated in ASCE 7, and modified by the *Provisions* Eq. 5.2.7-1 and 5.2.7-2 [4.2-1 and 4.2-2], were computed in Sec. 9.1 of this volume of design examples as:

$1.4D + 1.0Q_E$

and

$0.7D - 1.0Q_E$.

The roof live load, L_r, is not combined with seismic loads, and the design snow load is zero for this Los Angeles location.

10.2.2.2.3 Deflection and Drift Limits (Provisions Sec. 5.2.8 [4.5.1])

In-plane deflection and drift limits for the masonry shear walls are considered in Sec. 9.1.

As illustrated below, the diaphragm deflection is much greater than the shear wall deflection. According to *Provisions* Sec. 5.2.6.2.6 [4.5.2], in-plane diaphragm deflection shall 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 *Provisions* Table 5.2.8 [4.5-1]. 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 (*Provisions* Sec. 5.4.6.2 [5.2.6.2]) are evaluated.

10.2.3 Seismic Force Analysis

Building weights and base shears are as computed in Sec. 9.1 of this volume of design examples. (The building weights used in this example are based on a preliminary version of Example 9.1 and, thus, minor numerical differences may exist between the two examples). *Provisions* Sec. 5.2.6.4.4 [4.6.3.4]specifies that floor and roof diaphragms be designed to resist a force, F_{px}, in accordance with *Provisions* Eq. 5.2.6.4.4 [4.6-2]as follows:

$$F_{px} = \frac{\sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} w_i} w_{px}$$

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:

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 *Provisions* Sec. 5.2.6.4.4 [4.6.3.4], because C_s exceeds the minimum factor of $0.2S_{DS}$.

10.2.4 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 *Provisions*. 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, *Provisions* Sec. 12.4.1.2 places aspect ratio limits on plywood diaphragms (length-to-width shall 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. A procedure for computing diaphragm deflections is illustrated in detail, in Sec. 10.1.4.7.

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.

10.2.4.1 Strength of Members and Connections

The 2000 *Provisions* have adopted Load and Resistance Factor Design (LRFD) for engineered wood structures. The *Provisions* includes the ASCE 16 standard by reference and uses it as the primary design procedure for engineered wood construction. Strength design of members and connections is based on the requirements of ASCE 16. The AF&PA Manual and supplements contain reference resistance values for use in design. For convenience, the *Provisions* contains design tables for diaphragms that are identical to those contained in the AF&PA Structural-Use Panels Supplement. Refer to Sec. 10.1.4.1 for a more complete discussion of the design criteria.

[The primary reference for design of wood diaphragms in the 2003 *Provisions* is AF&PA Wind & Seismic. Much of the remaining text in the 2003 *Provisions* results from differences between AF&PA Wind & Seismic and Chapter 12 of the 2000 *Provisions* as well as areas not addressed by AF&PA Wind & Seismic. Because the AF&PA Wind & Seismic tabulated design values for diaphragms do not completely replace the tables in the 2000 *Provisions*, portions of the tables remain in the 2003 *Provisions*. Therefore, some diaphragm design values are in the 2003 *Provisions* and some are in AF&PA Wind & Seismic. The design values in the tables are different between the two documents. The values in the 2003 *Provisions* represent factored shear resistance ($\lambda\varphi D'$), while the values in AF&PA Wind & Seismic represent nominal shear resistance that must then be multiplied by a resistance factor, φ, (0.65) and a time effect factor ,λ, (1.0 for seismic loads). Therefore, while the referenced tables may be different, the factored resistance values based on the 2003 *Provisions* should be the same as those in examples based on the 2000 *Provisions*. The calculations that follow are annotated to indicate from which table the design values are taken.]

10.2.4.2 Roof Diaphragm Design for Transverse Direction

10.2.4.2.1 Plywood and Nailing

The diaphragm design force, $F_{p,roof}$ = 233 kips. Accounting for accidental torsion (*Provisions* Sec. 5.4.4.2 [5.2.4.2]), the maximum end shear = 0.55$F_{p,roof}$ = 128 kips. This corresponds to a unit shear force v = (128/100) = 1.28 klf. Although there is not a specific requirement in the *Provisions*, it is the authors' opinion that accidental torsion should be considered, even for "flexible" diaphragms, to account for the possibility of a non-uniform mass distribution in the building.

Because the diaphragm shear demand is relatively high, the plywood sheathing and roof framing at the ends of the building, must be increased in size over the standard roof construction for this type of building. Assuming 3-in. nominal framing, try blocked 3/4-in. (23/32) Structural I plywood rated sheathing with two lines of 10d common nails at 2-1/2 in. on center at diaphragm boundaries, continuous panel edges, and two lines at 3 in. on center at other panel edges.

From *Provisions* Table 12.4.3-1a [12.4-1a], $\lambda\phi D'$ = 1.60 klf > 1.28 klf OK

Because the diaphragm shear decreases towards the midspan of the diaphragm, the diaphragm capacity may be reduced towards the center of the building. Framing size and plywood thickness are likely to have a more significant impact on cost than nail spacing, determine a reasonable location to transition to 2 in. nominal roof joists and ½ in. (15/32) plywood. A reasonable configuration for the interior of the building utilizes ½ -in. (15/32) Structural I plywood rated sheathing with a single line of 10d at 2 ½ in. on center nailing at diaphragm boundaries, continuous panels edges, and 4 in. on center nailing at other panel edges. Using 2×4 flat blocking at continuous panel edges, the requirements found in Notes f, and g of *Provisions* Table 12.4.3-1a [AF&PA Wind&Seismic Sec. 4.2.7.1] are met. Determine the distance, X, from the end wall where the transition can be made as:

$\lambda\phi D'$ = 0.83 klf (*Provisions* Table 12.4.3-1a [AF&PA Wind&Seismic Table 4.2A]),
Shear Capacity = 0.83(100) = 83.0 kips
Uniform Diaphragm Demand = 233/200 = 1.165 klf
X = (128 - 83)/1.165 = 38.6 ft, say 40 ft from the diaphragm edge

[Here is an example where both the *Provisions* tables and the AF&PA Wind & Seismic tables are required to complete the design. The design value for this plywood thickness and nailing pattern is contained in AF&PA Wind & Seismic, but the design value for the higher-capacity diaphragm at the ends is contained in the *Provisions*.]

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 10.2-1 contains a summary of the diaphragm framing and nailing requirements (All nails are 10d common). See Figure 10.2-2 for designation of framing and nailing zones, and Figure 10.2-3 for typical plywood layout.

Table 10.2-1 Roof Diaphragm Framing and Nailing Requirements

Zone*	Framing	Plywood	Nail Spacing (in.)	Capacity (kip/ft)

			Boundaries and Cont. Panel Edges	Other Panel Edges	Intermediate Framing Members	
a	3×12	¾ in.	2½ (2 lines)	3 (2 lines)	12 (1 line)	1.60
b	2×12	½ in.	2½ (1 line)	4 (1 line)	12 (1 line)	0.83

1.0 in. = 25.4 mm, 1.0 kip/ft = 14.6 kN/m.
* Refer to Figure 10.2-2 for zone designation.

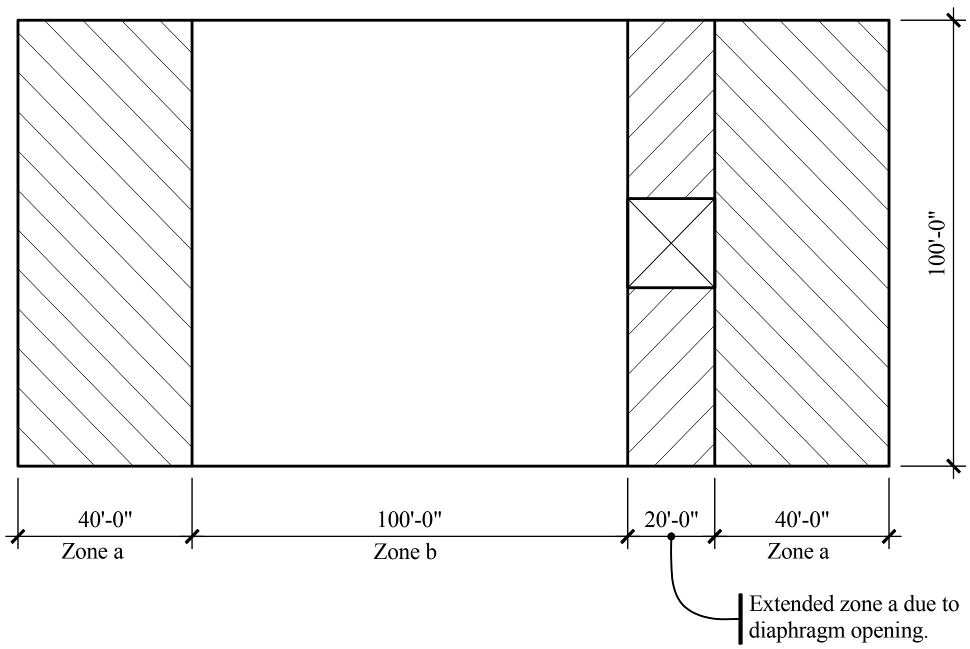

Figure 10.2-2 Diaphragm framing and nailing layout (1.0 ft = 0.3048 m).

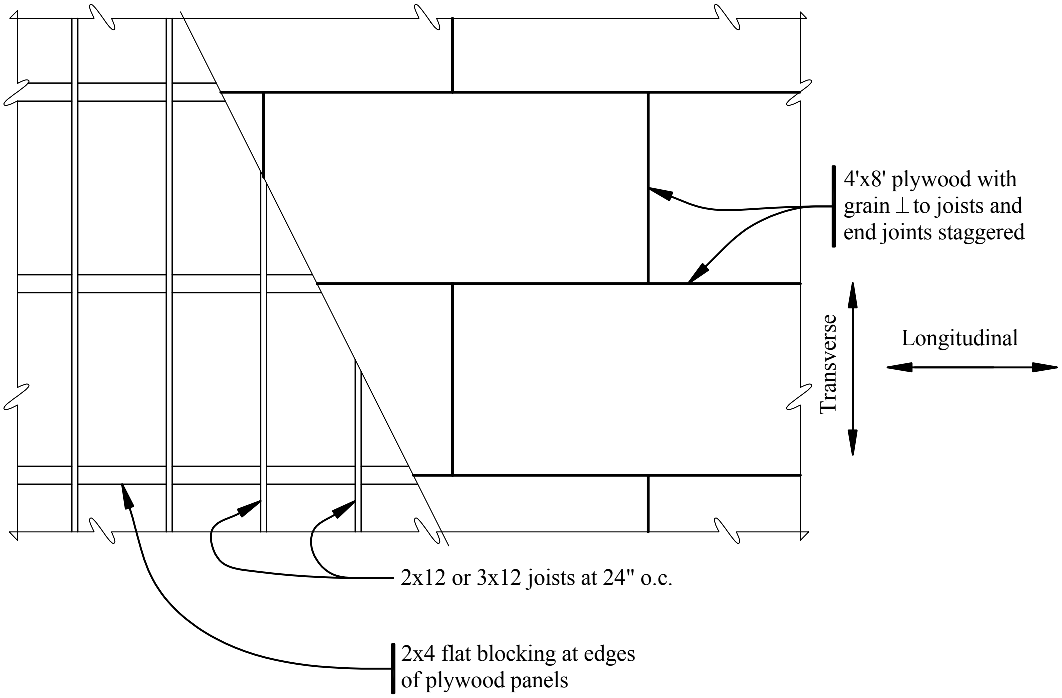

Figure 10.2-3 Plywood layout (1.0 ft = 0.3048 m, 1.0 in. = 25.4 mm).

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

Try a select structural Douglas fir-larch 4×12, for the chord. Assuming two 1/16 in. bolt holes (for 1-in. bolts) at splice locations, the net chord area is 31.9 in.^{2} Tension strength (parallel to wood grain), per the AF&PA Manual, Structural Lumber Supplement:

$\lambda\phi T'$ (1.0)(0.8)(2.70)(31.9) = 68.9 kips > 59.0 kips *OK*

Design the splice for the maximum chord force of 59.0 kips. Try bolts with steel side plates using 1 in. A307 bolts, with a 3½ in. length in the main member. The capacity, according to the AF&PA Manual, Structural Connections Supplement, is:

$\lambda\phi Z'$ = (1.0)(0.65)(16.29) = 10.6 kips per bolt.

Number of bolts required = 59.0/10.6 = 5.6

Use two rows of three bolts. The reduction (group action factor) for multiple bolts is negligible. Net area of the 4×12 chord with two rows of 1-1/16 in. holes is 31.9 in.^{2} as assumed above. Therefore, use six 1 in. A307 bolts on each side of the chord splice (Figure 10.2-4). 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.

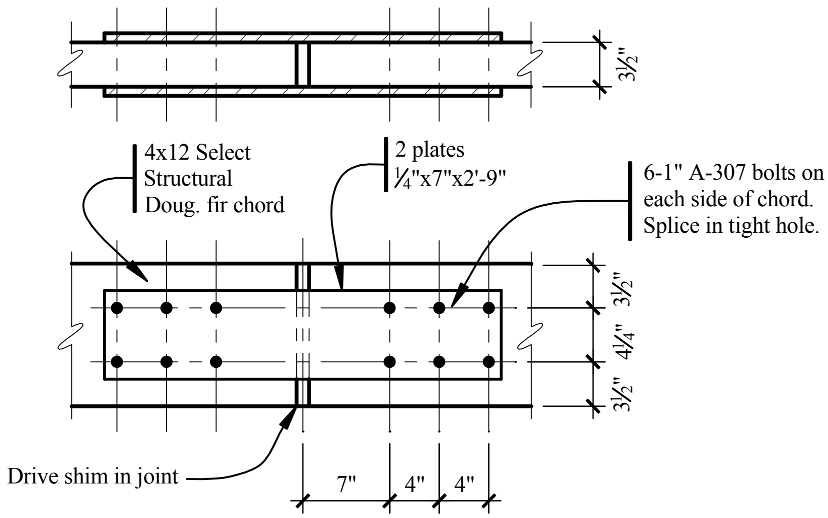

Figure 10.2-4 Chord splice detail (1.0 ft = 0.3048 m, 1.0 in. = 25.4 mm).

10.2.4.2.3 Diaphragm Deflection and P-delta Check

The procedure for computing diaphragm deflection is described in Sec. 10.1.4.7.

[AF&PA Wind & Seismic also contains procedures for computing diaphragm deflections. The equations are slightly different from the more commonly used equations that appear in the *Commentary* and AF&PA LRFD *Manual*. In AF&PA Wind & Seismic, the shear and nail slip terms are combined using an "apparent shear stiffness" parameter. However, the apparent shear stiffness values are only provided for OSB. Therefore, the deflection equations in the *Commentary* or AF&PA LRFD *Manual* must be used in this example which has plywood diaphragms. The apparent shear stiffness values for plywood will likely be available in future editions of AF&PA Wind & Seismic.]

As stated in *Commentary* Sec. 12.4, the diaphragm deflection is computed as:

$$\delta = \frac{5vL^3}{8wEA} + \frac{vL}{4Gt} + 0.188Le_n + \frac{\Sigma\left(\Delta_c X\right)}{2w}$$

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. An adjustment factor, F, for non-uniform nailing is applied to the third term of the above equation for this example.

Bending deflection = $5vL^3/8wEA$ = 0.580 in.
Shear deflection = $vL/4Gt$ = 1.210 in.
Effective nails slip deflection = 0.188 Le_nF = 0.265 in.
Deflection due to chord slip at splices = $\Sigma(\Delta_c X)/2w$ = 0.000 in.

(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:

v = (233/2)/100 = 1,165 plf (shear per foot at boundary, ignoring torsion)

L = 200 ft and w = 100 ft (diaphragm length and depth)
A = effective area of 4×12 chord and two-#6 rebars assumed to be in the bond beam
= 39.38 in.^{2} + 2(0.44)(29,000,000/1,900,000) = 52.81 in.^{2}
t = 0.535 in. (effective, for ½ in. Structural I plywood, unsanded; neglect ¾ in. plywood at edge)
E = 1,900,000 psi (for Douglas fir-larch select structural chord)
G = 90,000 psi (Structural I plywood)
e_n = nail slip for 10d nail at end of diaphragm (use one and one-half-in. nail spacing, two lines at 3 in., not the 1-1/4 in. perimeter spacing)

Design nail load = 1,165/(12/1.5) = 146 lb/nail

Using the procedure in Sec. 10.1.4.3 and the value for dry/dry lumber from Table 10.1-2, e_n = $(146/769)^{3.276}$ = 0.00433 in.

F = adjustment for non-uniform nail spacing

The 0.188 coefficient assumes a uniform nail spacing, which implies an average load per nail of one-half the maximum. One common practice for calculating deformation is simply to use the load per nail that would result from the larger spacing should that spacing be used throughout. However, this gives a large estimate for deformation due to nail slip. Figure 10.2-5 shows the graphic basis for computing a more accurate nail slip term. The basic amount is taken as that for the smaller nail spacing (1-1/2 in.), which gives 146 lbs per nail.

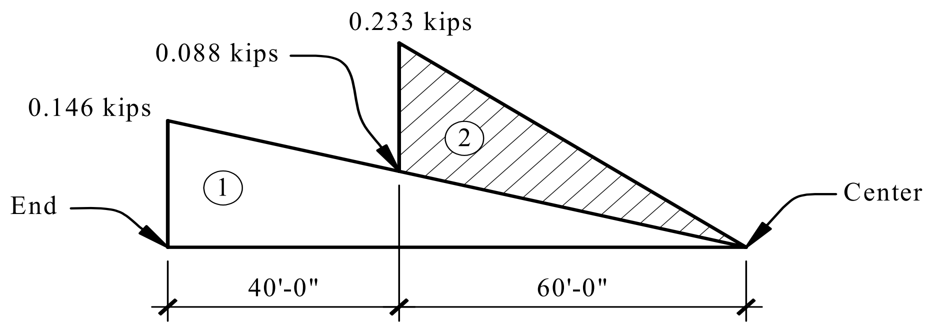

Figure 10.2-5 Adjustment for nonuniform nail spacing (1.0 ft = 0.3048 m, 1.0 kip = 4.45 kN)

Using a larger nail spacing at the interior increases the deformation. The necessary increase may be determined as the ratio of the areas of the triangles in Figure 10.2-5, which represents the load per nail along the length of the diaphragm. The ratio is for Triangle 2 representing zone b compared to Triangle 1 representing zone a, where zones a and b are as shown Figure 10.2-2.

Ratio = (233 - 88)60 / 146(100) = 0.63

thus, F = 1 + 0.63 = 1.63.

Total for diaphragm:

δ = 0.580 + 1.210 + 0.265 + 0.00 = 2.055 in.

End wall deflection = 0.037 in. (see Sec. 9.1 of this volume of design examples)

Therefore, the total elastic deflection δ_{xe} = 2.055 + 0.037 = 2.092 in.

Total deflection, $\delta_x = C_d\,\delta_{xe}/I$ = 3.5(2.092)/1.0 = 7.32 in. = Δ

P-delta effects are computed according to *Provisions* Sec. 5.4.6.2 [5.2.6.2]using the stability coefficient computed per *Provisions* Eq. 5.4.6.2-1 [5.2-16]:

$$\theta = \frac{P_x \Delta}{V_x h_{sx} C_d}$$

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
Δ= 7.32 in.
V = 233 kips (diaphragm force)

For the end walls, consider the weight of the end walls at the wall displacement:

P = 330 kips
Δ = (3.5)(0.037) = 0.13 in.
V = 264 kips (additional base shear for wall design)

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

$$\theta = \left(\frac{P\Delta}{V} + \frac{P\Delta}{V}\right) \Big/ hC_d = \left(\frac{816(7.32)}{233} + \frac{330(0.13)}{264}\right) \Big/ (28)(12)(3.5) = 0.022$$

For θ < 0.10, no deflection amplification due to P-delta effects is necessary.

[Note that the equation to determine the stability coefficient has been changed in the 2003 *Provisions*. The importance factor, I, has been added to 2003 *Provisions* Eq. 5.2-16. However, this does not affect this example because I = 1.0.]

Drift index =Δ/h_{sx} = 7.32/[28(12)] = 0.022.

This is slightly less than the limiting drift ratio of 0.025 applied for most low-rise buildings in Seismic Use Group I (*Provisions* Sec. 5.2.8 [4.5.1] and Table 5.2.8 [4.5-1]). However, for one-story buildings, Table 5.2.8, Note b [4.5-1, Note c], and *Provisions* Sec. 5.2.6.2.6 [4.5.2], permit 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.

Given the above conditions, and the fact that P-delta effects are not significant for this structure, the computed diaphragm deflections appear acceptable.

10.2.4.2.4 Detail at Opening

Consider diaphragm strength at the roof opening (Figure 10.2-6), as required by *Provisions* Sec. 5.2.6.2.2 [4.6.1.4].

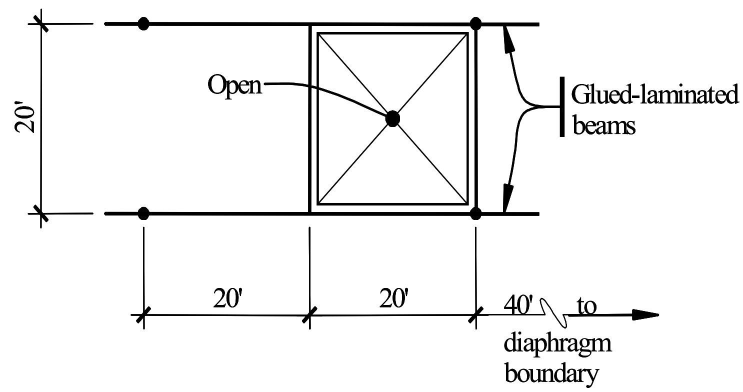

Figure 10.2-6 Diaphragm at roof opening (1.0 ft = 0.3048 m).

Check diaphragm nailing for
required shear area (shear in diaphragm at edge of opening):

Shear = 128 - [40(1.165)] = 81.4 kips
v = 81.4/(100 - 20) = 1.02 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 = 0.830 klf as computed previously (see Table 10.2-1 and Figure 10.2-2). 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 ft from the end wall). The diaphragm strength is now adequate for the reduced overall width at the opening.

10.2.4.2.5 Framing around Opening

The opening is located 40 ft from one end of the building and is centered in the other direction (Figure 10.2-6). 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.

10.2.4.2.6 Chord Forces at Opening

To determine the chord forces on the edge joists, model the diaphragm opening as a Vierendeel truss and assume the inflection points will be at the midpoint of the elements (Figure 10.2-7). Compute forces at the opening using a uniformly distributed diaphragm demand of 233/200 = 1.165 klf.

For Element 1 (shown in Figure 10.2-7):

w_1 = 1.165/2 = 0.582 kips/ft
V_{1B} = 0.5[128-(40)(1.165)] = 40.7 kips
V_{1A} = 40.7 - 20(0.582) = 29.1 kips
M_1 due to Vierendeel action = (½)[40.7(10) + 29.1(10)] = 349 ft-kips

Chord force due to M_1 = 349/40 = 8.72 kips. This is only 41 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.

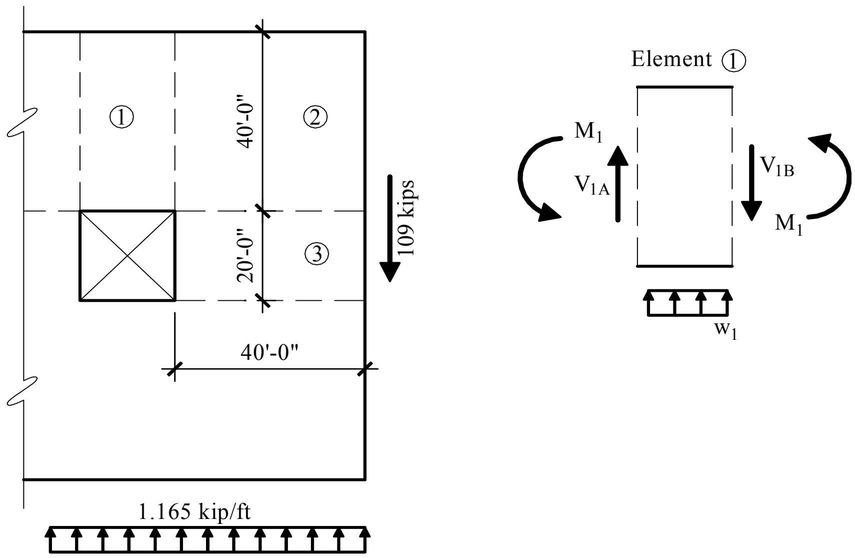

Figure 10.2-7 Chord forces and Element 1 free-body diagram (1.0 ft = 0.3048 m, 1.0 kip = 4.45 kN, 1.0 kip/ft = 14.6 kN/m).

For Element 3, analyze Element 2 (shown in Figure 10.2-8):

w_2 = 1.165(40/100) = 0.466 kips/ft
V_3 = 128(40/100) = 51.2 kips
V_{1B} = 40.7 kips
M_1 = 349 ft-kips.

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

$M = 128(40) - 1.165(40^2/2)$ = 4,188 ft-kips
T_{1B} = 4,188/100 = 41.9 kips
ΣM_0: $M_3 = M_1 + 40V_3 - 40T_{1B} - w_2 40^2/2$ = 348 ft-kips

Therefore, the chord force on the roof joist = 348/40 = 8.7 kips

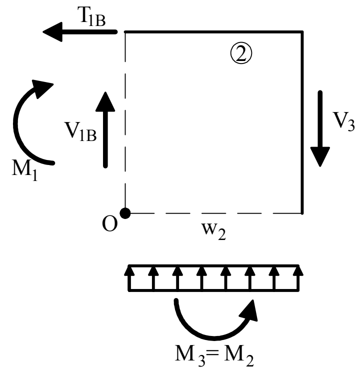

Figure 10.2-8 Free-body diagram for Element 2.

Alternatively, the chord design should consider the wall anchorage force interrupted by the opening. As described in Sec. 10.2.4.3, the edge members on each side of the opening are used as continuous cross-ties, with maximum cross-tie force of 25.0 kips. Therefore, the cross-tie will adequately serve as a chord at the opening.

10.2.4.3 Roof Diaphragm Design for Longitudinal Direction

Force = 209 kips
Maximum end shear = 0.55(209) = 115 kips
Diaphragm unit shear v = 115/200 = 0.58 klf

For this direction, the plywood layout is Case 3 in *Provisions* Table 12.4.3-1a [AF&PA Wind&Seismic Table 4.2A]. Using ½ in. Structural I plywood rated sheathing, blocked, with 10d common nails at 2 ½ in. on center at diaphragm boundaries and continuous panel edges parallel to the load (ignoring the capacity of the extra nails in the outer zones):

$\lambda\varphi D'$ = 0.83 plf > 0.58 plf, *Provisions* Table 12.4.3-1a [AF&PA Wind&Seismic Table 4.2A] 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.

10.2.4.4 Masonry Wall Anchorage to Roof Diaphragm

As stipulated in *Provisions* Sec. 5.2.6.3.2 [4.6.2.1], masonry walls must be anchored to flexible diaphragms to resist out-of-plane forces computed per *Provisions* Eq. 5.2.6.3.2 [4.6-1]as:

$F_P = 1.2S_{DS}IW_p = 1.2(1.0)(1.0)W_p = 1.2\ W_p$
Side walls, F_P = 1.2(65psf)(2+28/2)/1000 = 1.25 klf
End walls, F_P = 1.2(103psf)(2 + 28/2)/1000 = 1.98 klf

[In the 2003 *Provisions* the anchorage force for masonry walls connected to a flexible diaphragm has been reduced to $0.8S_{DS}IW_p$.]

10.2.4.3.1 Anchoring Joists Perpendicular to Walls (Side Walls)

Because the roof joists are spaced at 2 ft on center, provide a connection at each joist that will develop 2(1.25) = 2.50 kip/joist.

A common connection for this application is metal tension tie or hold-down device that is anchored to the masonry wall with an embedded bolt and is either nailed 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 in. headed anchor bolt, embedded in the bond beam and 18 10d nails into the side of the joist (Figure 10.2-10). Modifying the allowable values using the procedure in Sec. 10.1.4.5 results in a design strength of:

$\lambda\phi Z' = (1.0)(0.65)(4.73) = 3.07$ kips per anchor > 2.50 kips 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 in. and the joist length is 20 ft, so there are 20 nails per joist. From the AF&PA Manual, Structural Connections Supplement, the strength of a single 10d common nail is:

$\lambda\phi Z' = (1.0)(0.65)(0.298) = 0.194$ kips per nail
$20(0.194) = 3.88$ kips > 2.50 kips *OK*

The embedded 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 (2.50 kips/bolt), and the vertical gravity loading. Assuming 20 psf dead load (roof live load is not included), and a 10-foot tributary roof width, the vertical load per bolt = (20 psf)(10 ft)(2 ft)/1000 = 0.40 kip. Using the load combination described previously, the design horizontal tension and vertical shear on the bolt are:

$b_a = 1.0Q_E = 2.50$ kips
$b_v = 1.4D = 1.4(0.40) = 0.56$ kip

The anchor bolts in masonry, are designed according to *Provisions* Sec. 11.3.12. Using 3/4 in. headed anchor bolts, both axial strength, B_a, and shear strength, B_v, will be governed by masonry breakout. Per *Provisions* Eq. 11.3.12.3-1, and 11.3.12.1-1, respectively, the design strengths are:

$$B_a = 4\phi A_p \sqrt{f'_m}$$

$$B_v = 1750\phi\sqrt[4]{f'_m A_b}$$

where $\varphi = 0.5$, $f'_m = 2{,}000$ psi, A_b = tensile area of bolt = 0.44 in.^{2}, and A_p = projected area on the masonry surface of a right circular cone = 113 in.^{2} (assuming 6 in. effective embedment). Therefore, $B_a = 10.1$ kips and $B_v = 4.8$ kips. Shear and tension are combined per *Provisions* Eq. 11.3.12.4 as:

$$\frac{b_a}{B_a} + \frac{b_v}{B_v} = \frac{2.50}{10.1} + \frac{0.56}{4.8} = 0.36 < 1.0 \qquad \text{OK}$$

[The 2003 *Provisions* refers to ACI 530, Sec. 3.1.6 for strength design of anchorage to masonry, as modified by 2003 Provisions Sec. 11.2. In general, the methodology for anchorage design using ACI 530 should be comparable to the 2000 *Provisions*, though some of the equations and reduction factors may be different. In addition, the 2003 *Provisions* require that anchors are either controlled by a ductile failure mode or are designed for 2.5 times the anchorage force.]

Figure 10.2-9 summarizes the details of the connection. In-plane seismic shear transfer (combined with gravity) and orthogonal effects are considered in a subsequent section.

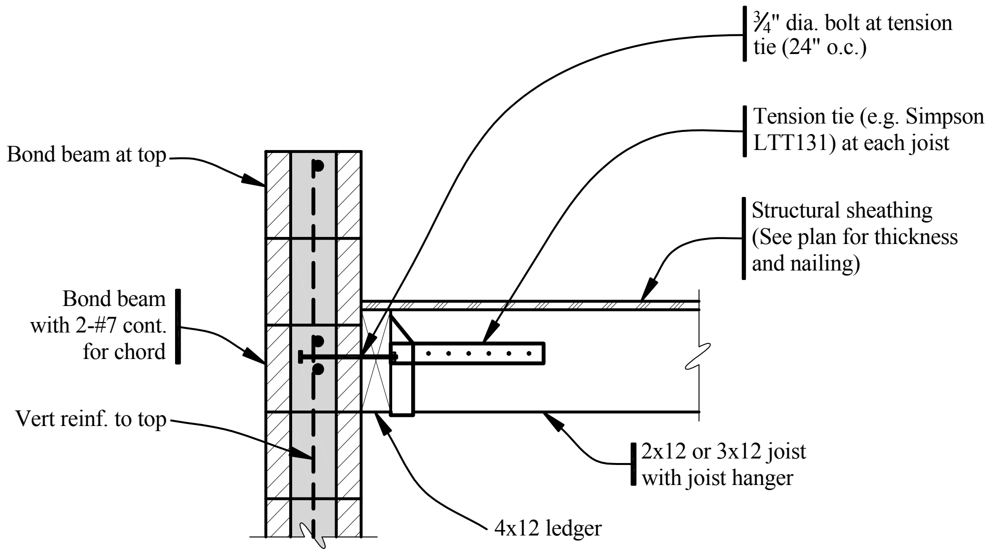

Figure 10.2-9 Anchorage of masonry wall perpendicular to joists (1.0 in. = 25.4 mm).

According to *Provisions* Sec. 5.2.6.3.2 [4.6.2.1, diaphragms must have continuous cross-ties to distribute the anchorage forces into the diaphragms. Although the *Provisions* do not specify a maximum spacing, 20 ft is common practice for this type of construction and Seismic Design Category.

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

(1.25 klf)(20 ft) = 25.0 kips

Try a 3×12 (Douglas fir-Larch No. 1) as a cross-tie. Assuming one row of 15/16 in. bolt holes, the net area of the section is 25.8 in.^{2} Tension strength (parallel to wood grain) per the AF&PA Structural Lumber Supplement, is:

$\lambda\phi T'$ = (1.0)(0.8)(1.82)(25.8) = 37.5 kips > 25.0 kips *OK*

At the splices, try a double tie-down device with four 7/8 in. bolts in double shear through the 3×12 (Figure 10.2-10). 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 Sec. 10.1.4.5 to modify the allowable values:

$2\lambda\phi Z'$ = 2(1.0)(0.65)(20.66) = 26.9 kips > 25.0 kips OK

The reduction (group action) factor for multiple bolts, C_g = 0.97.

For the four bolts, the AF&PA Manual, Structural Connections Supplement, gives:

$4\lambda\phi C_g Z'$ = 4(1.0)(0.65)(0.97)(10.17) = 25.6 kips > 25.0 kips

OK

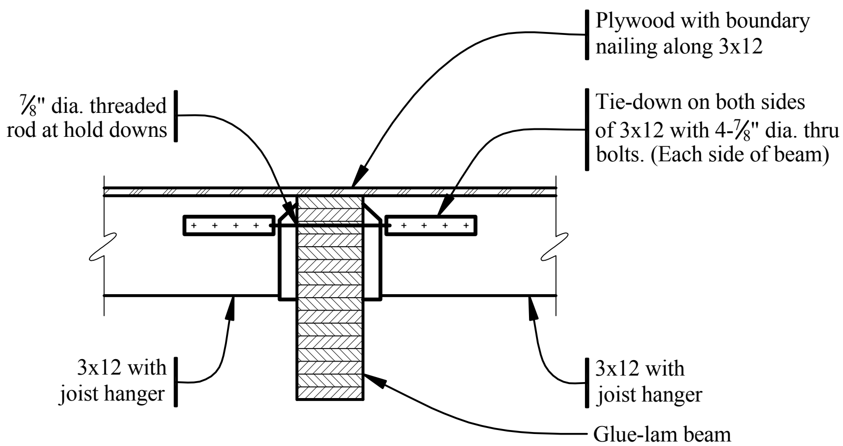

Figure 10.2-10 Chord tie at roof opening (1.0 in. = 25.4 mm).

In order to transfer the wall anchorage forces into the cross-ties, the subdiaphragms between these ties must be checked per *Provisions* Sec. 5.2.6.3.2 [4.6.2.1]. There are several ways to perform these subdiaphragm calculations. One method is illustrated in Figure 10.2-11. The subdiaphragm spans between cross-ties and utilizes the glued-laminated beam and ledger as it's chords. The 1-to-1 aspect ratio meets the requirement of 2½ to 1 for subdiaphragms per *Provisions* Sec. 5.2.6.3.2 [4.2.6.1].

For the typical subdiaphragm (Figure 10.2-11):

F_p = 1.25 klf
v = (1.25)(20/2)/20 = 0.625 klf.

The subdiaphragm demand is less than the minimum diaphragm capacity (0.83 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.

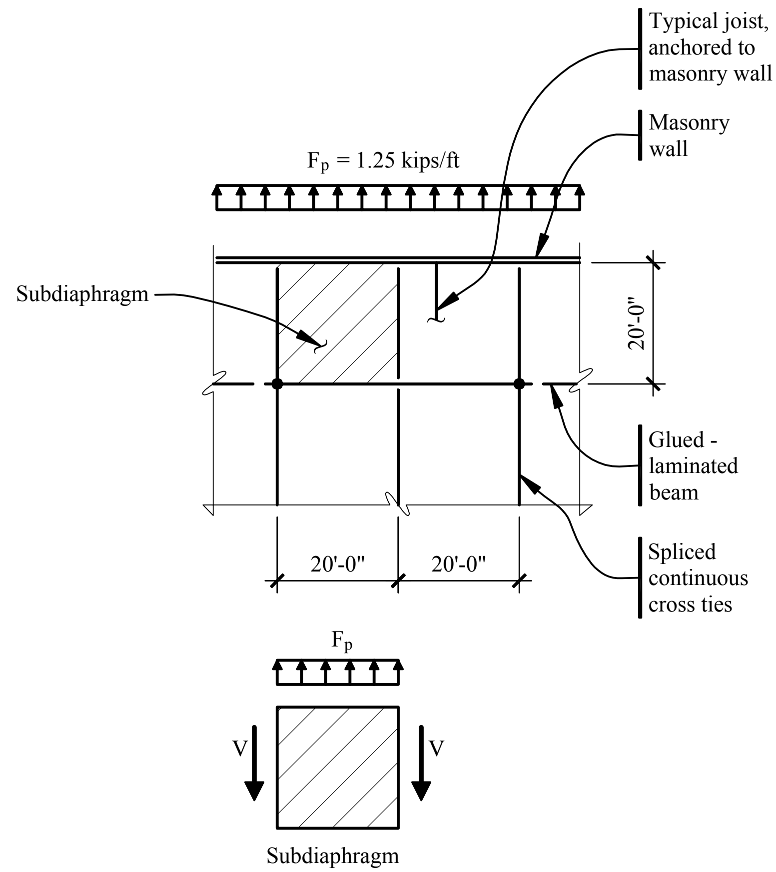

Figure 10.2-11 Cross tie plan layout and subdiaphragm free-body diagram for side walls (1.0 ft = 0.3048 m, 1.0 kip/ft = 14.6 kN/m).

10.2.4.3.2 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 10.2-12). 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 bearing plate at the innermost joist.

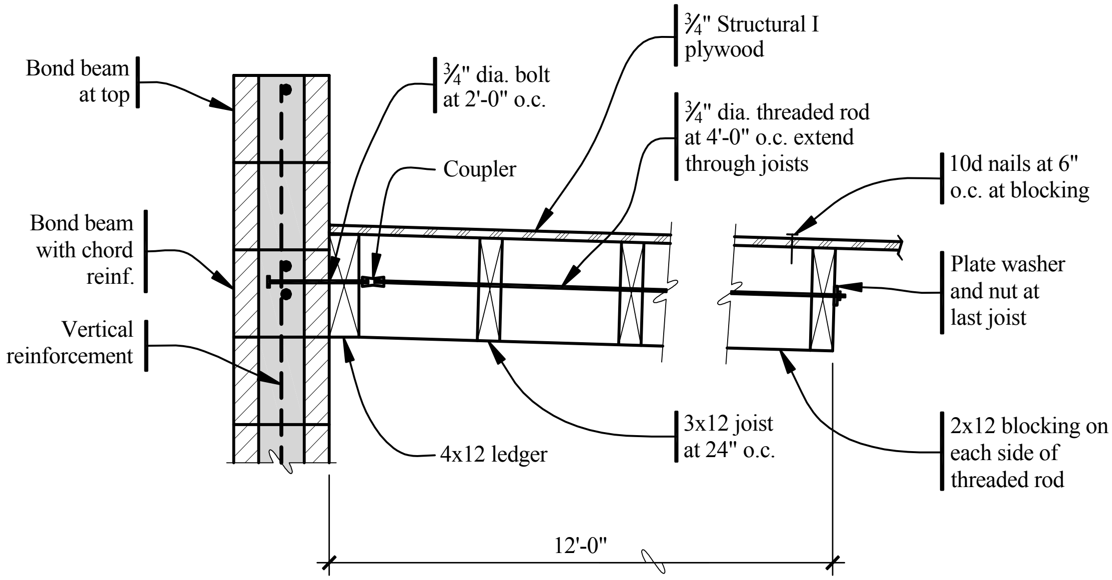

Figure 10.2-12 Anchorage of masonry wall parallel to joists (1.0 ft = 0.3048 m, 1.0 in. = 25.4 mm)

The anchorage force at the end walls is 1.98 klf. Space the connections at 4 ft on center so that the wall need not be designed for flexure (*Provisions* Sec. 5.2.6.1.2 [4.6.1.2]). Thus, the anchorage force is 7.92 kips per anchor.

Try a 3/4 in. 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 in. headed anchor bolts (with 6 in. embedment), the design axial strength B_a = 10.1 kips > 7.92 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 ft) into the roof framing. (The 12 ft are required for the subdiaphragm force transfer as discussed below.)

Nailing the blocking to the plywood sheathing is determined, using the AF&PA Structural Connections Supplement. As computed previously, the capacity of a single 10d common nail, $\lambda\varphi Z'$ = 0.194 kips. Thus, 7.92/0.194 = 41 nails are required. This corresponds to a nail spacing of about 7 in. for two 12-ft rows of blocking. Space nails at 6 in. for convenience.

Use the glued-laminated beams (at 20 ft on center) to provide continuous cross-ties, and check the subdiaphragms between the beams to provide adequate load transfer to the beams per Provisions Sec. 5.2.6.3.2 [4.6.2.1].

Design tension force on beam = (1.98 klf)(20 ft) = 39.6 kips

The stress on the beam is f_t = 39.600/[8.75(24)] = 189 psi, which is small. The beam is adequate for combined moment due to gravity loading and axial tension.

At the beam splices, try one-in. bolts with steel side plates. Per the AF&PA Manual, Structural Connections Supplement:

$\lambda \phi Z' = (1.0)(0.65)(18.35) = 11.93$ kips per bolt.

The number of bolts required = 39.6/11.93 = 3.3.

Use four bolts in a single row at midheight of the beam, with 1/4-in. by 4-in. steel side plates. The reduction (group action factor) for multiple bolts is negligible. Though 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* Sec. 5.2.6.3.2 [4.6.2.1]. The procedure is similar to that used for the side walls as described previously. The end wall condition is illustrated in Figure 10.2-13. 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. The maximum aspect ratio for subdiaphragms is 2½ to 1 per *Provisions* Sec. 5.2.6.3.2 [4.6.2.1]. Therefore, the minimum depth is 20/2.5 = 8 ft.

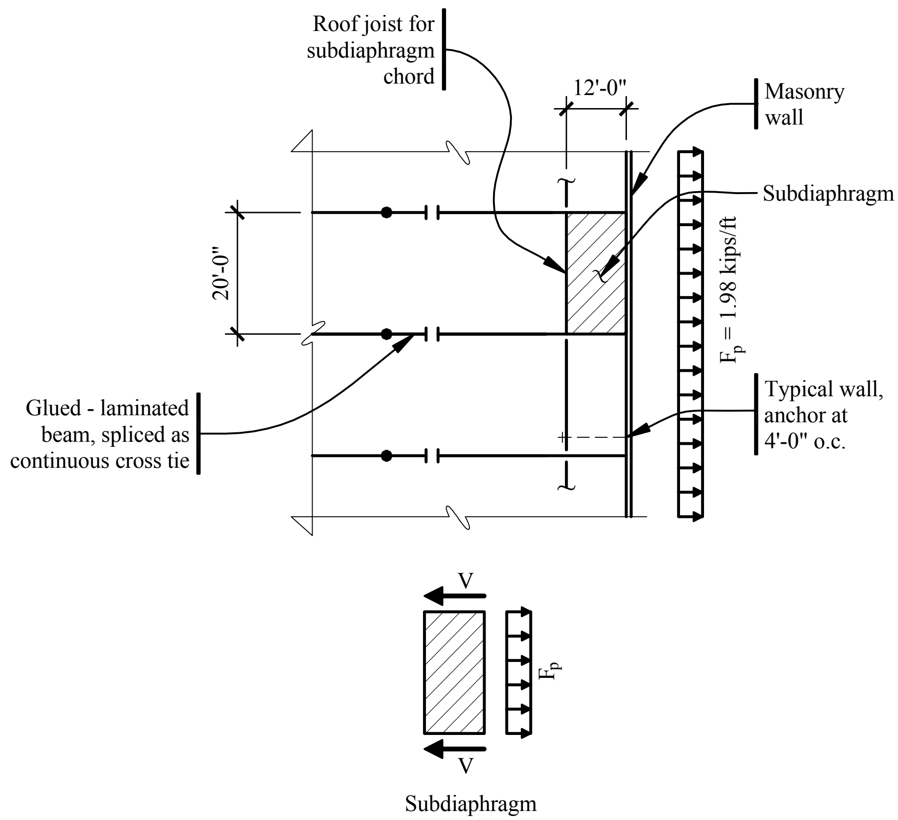

Figure 10.2-13 Cross tie plan layout and subdiaphragm free-body diagram for end walls (1.0 ft = 0.3048 m, 1.0 kip/ft = 14.6 kN/m).

For the typical subdiaphragm (Figure 10.2-13):

F_p = 1.98 klf
v = (1.98)(20/2)/8 = 2.48 klf

As computed previously (see Table 10.2-1 and Figure 10.2.2), the diaphragm strength in this area is 1.60 klf < 2.48 klf. Therefore, increase the subdiaphragm depth to 12 ft (six joist spaces):

v = (1.98)(20/2)/12 = 1.65 klf ≈1.60 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 ft into the building.

10.2.4.3.3 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:

Side walls = 0.575 klf
End walls = 1.282 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 *Provisions* Sec. 5.2.5.2.3 [4.4.2.3]. 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-in. bolts (Figure 10.2-9) 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 ft on center) is 0.575(2) = 1.15 kips.

Combining gravity shear and seismic shear, produces a resultant force of 1.38 kips at an angle of 26 degrees from the axis of the wood grain. Using the formulas for bolts at an angle to the grain per the AF&PA Structural Connections Supplement gives

lfZ' = (1.0)(0.65)(3.47) = 2.26 kips > 1.38. OK

This bolt spacing satisfies the load combination for gravity loading only.

For the check of the embedded anchor bolts, the factored demand on a single bolt is 1.15 kips in horizontal shear (in-plane shear transfer), 2.50 kips in tension (out-of-plane wall anchorage), 0.56 kip in vertical shear (gravity). Orthogonal effects are checked, using the following two equations:

$$\frac{(0.3)(2.5)}{19.1} + \frac{\sqrt{1.15^2 + 0.56^2}}{4.8} = 0.34$$

and

$$\frac{(2.5)}{10.1} + \frac{\sqrt{[0.3(1.15)]^2 + 0.56^2}}{4.8} = 0.38 \text{ (controls)} < 1.0$$ OK

At the end walls, the ledger with 3/4-in. bolts (Figure 10.2-12) need only be checked for in-plane seismic shear because gravity loading is negligible. For bolts spaced at 4 ft on center, the demand per bolt is 1.282(4) = 5.13 kips parallel to the grain of the wood. Per the AF&PA Structural Manual, Connections Supplement:

Z' = (1.0)(0.65)(5.37) = 3.49 kips < 5.13 kips NG

Therefore, add 3/4-in. headed bolts evenly spaced between the tension ties such that the bolt spacing is 2 ft on center and the demand per bolt is 1.282(2) = 2.56 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 2.56 kips in horizontal shear (in-plane shear transfer), 7.92 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:

$$\frac{(0.3)(7.92)}{10.1}+\frac{2.56}{4.8}=0.77$$

and

$$\frac{7.92}{10.1}+\frac{0.3(2.56)}{4.8}=0.94 \text{ (controls)} < 1.0 \qquad \text{OK}$$

Therefore, the wall connections satisfy the requirements for combined gravity, and seismic loading, including orthogonal effects.