Five-Story Light-Frame Wood Structure Over Podium

DESIGN EXAMPLE

WOODWORKS WOOD PRODUCTS COUNCIL ™

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Table of Contents

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1 Overview

1.1 Introduction

Buildings with five or more stories of light-frame wood construction over a single or multi-level concrete podium have gained popularity because they offer both density and value, helping address the need for cost-effective residential and mixed-use spaces. However, these projects have unique design and code considerations that are not always well understood.

This design example illustrates the seismic and wind design of a hotel that includes five stories of light-frame wood over a one-story concrete podium. The gravity framing system consists of light-frame wood bearing walls for the upper stories and concrete bearing walls for the lower story. The vertical lateral force-resisting system (LFRS) consists of light-frame wood shear walls for the upper stories and reinforced concrete shear walls for the lower story. The wood roof is framed with metal plate-connected wood trusses. The floor is framed with prefabricated wood I-joists. The floors have a 1-1⁄2-inch lightweight concrete topping. The roofing is modified bitumen membrane.

The term podium is not defined in the International Building Code (IBC) but is included in the commentary to Section 510.2. Also referred to as pedestal construction, these buildings include a slab, with or without dropped beams, supported by columns; the podium (or pedestal) portion of the building is designed to support the entire weight of the wood superstructure. Section 510.2 outlines the use of horizontal building separations, which allows the use of a 3-hour fire-resistance-rated assembly to create separate buildings for the purposes of allowable height and area.

This design example provides a detailed analysis of some of the important lateral requirements for the shear walls per the IBC and its referenced standards. It is not a complete building design. Many aspects are excluded, specifically the gravity framing system, and only certain steps related to the seismic and wind design of a selected shear wall are illustrated. To fully convey the process, these steps are presented in more detail than may be necessary for an actual building design.

1.2 Codes and Reference Documents

Unless otherwise noted, the following editions of code and standards are referenced in this document.

- 2024 International Building Code (IBC) (ICC, 2024)
- ASCE 7-22: Minimum Design Loads and Associated Criteria for Buildings and Other Structures (ASCE 7) (ASCE/SEI, 2022)
- 2024 National Design Specification for Wood Construction (NDS) (AWC, 2024)
- 2024 NDS Supplement: Design Values for Wood Construction (AWC, 2024)
- 2021 Special Design Provisions for Wind and Seismic (SDPWS) (AWC, 2021)
- AISC 360-22: Specification for Structural Steel Buildings (AISC 360) (AISC, 2022)
- Steel Construction Manual – Sixteenth Edition (Steel Manual) (AISC, 2023)
- ACI 318-19: Building Code Requirements for Structural Concrete (ACI 318) (ACI, 2019)

While this design example focuses on the 2024 IBC and its referenced standards, most code provisions are also applicable to the 2021 versions. Where significant differences exist, they are noted.

1.3 Factors That Influence Design

1.3.1 Lumber Species

The species of lumber used in this design example is Douglas fir-larch (DF-L), which is common on the West Coast. The authors do not intend to imply that this species should be used in all areas or for all markets. Species that are both appropriate for this type of construction and locally available vary by region and commonly include (among others) southern yellow pine (SYP), hem-fir (HF), and spruce-pine-fir (SPF).

1.3.2 Lumber Grade

The lower stories of the wood structure carry significantly higher gravity loads than the upper stories. One approach is to use a higher grade of lumber for the lower stories. This can produce designs that yield consistent wall construction over the height of the building, but requires careful attention during construction to ensure the correct grades are used at each location. Another approach is to choose one grade of lumber for all of the wood levels. This produces the need to change the size and/or spacing of the studs based on loading requirements. Plate crushing may control stud sizing at lower stories. For simplicity, this design example illustrates the use of one lumber grade for all wood levels.

1.3.3 Moisture Content and Wood Shrinkage

From a serviceability and performance perspective, wood shrinkage is an important issue in multi-story light-frame wood construction. It is impacted by the wood's moisture content (MC) and, more specifically, whether it is green or kiln dried.

There are three levels of wood seasoning (drying), which denote the MC of lumber at the time of surfacing:

- S-GRN = greater than 19% MC (unseasoned)
- S-DRY, KD or KD-HT = 19% maximum MC (seasoned)
- MC 15 or KD 15 = 15% maximum MC

These designations are found on the grade stamp.

Unseasoned lumber (S-GRN) is manufactured oversized so that when the lumber reaches 19% MC it will be approximately the same size as the dry (seasoned) size.

The word "DRY" indicates that the lumber was either kiln or air dried to a maximum MC of 19%.

Kiln-dried (KD) lumber has been placed in a closed chamber and heated until it reaches a pre-determined MC.

Kiln-dried heat-treated (KD-HT) lumber has been placed in a closed chamber and heated until it achieves a minimum core temperature of 132.8°F (56°C) for a minimum of 30 minutes to kill pathogens such as insects, fungi, or micro-organisms, and is primarily used in international trade.

MC restrictions apply at time of shipment, at time of dressing if dressed lumber is involved, and at time of delivery to the buyer unless shipped exposed to the weather.

The MC of wood products affects shrinkage calculations, which are discussed in Section 2.3.

1.4 Example Project: Given Information

Building elevation, floor plan, and cross section are shown in Figures 1.1, 1.2, and 1.3. The floor area of each level is 12,000 square feet, and Figure 1.3 includes floor-to-floor heights.

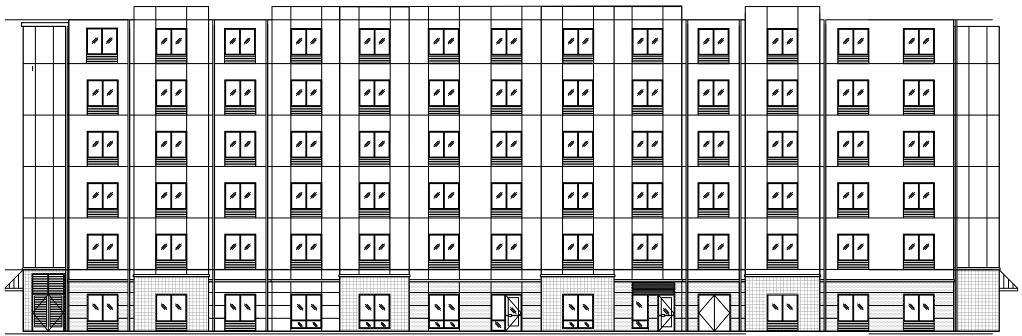


FIGURE 1.1: Building elevation

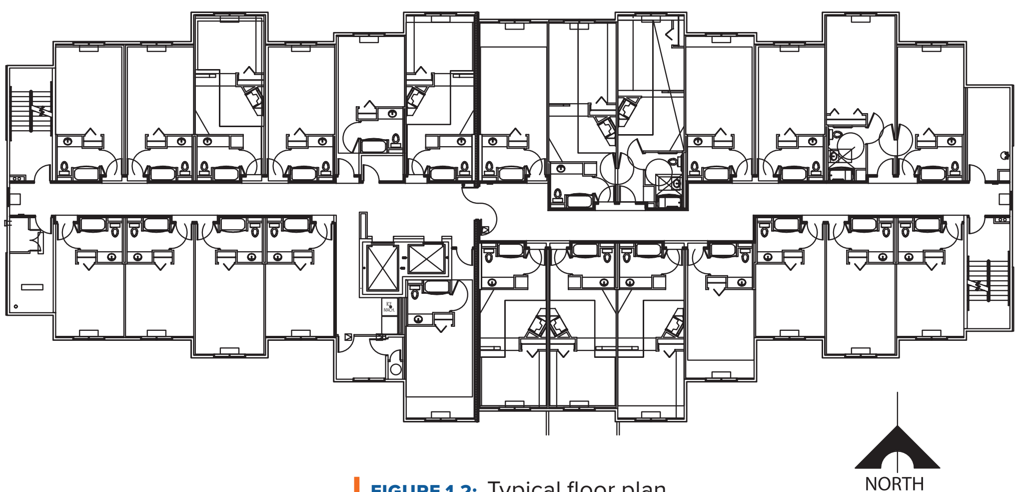


FIGURE 1.2: Typical floor plan

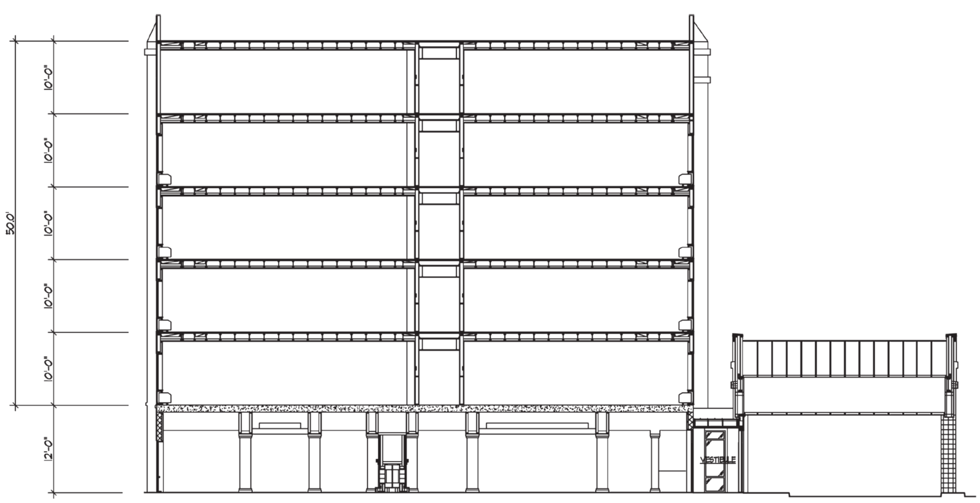
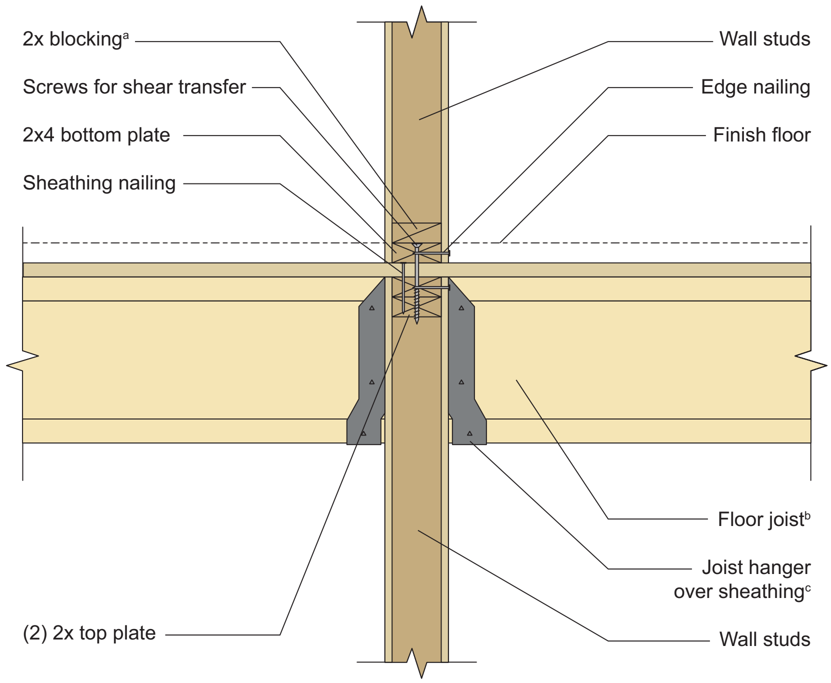


FIGURE 1.3: Typical cross section through the building

1.4.1 **Wood Structural Systems**

The floor system is comprised of engineered wood I-joists and 23/32-inch wood structural panel (WSP) sheathing, with joists spanning between wood stud bearing walls. The roof system is similarly comprised of engineered wood I-joists with 19/32-inch WSP sheathing, also spanning between wood stud walls. Given the short span of the floor and roof joists, sawn lumber joists could be used.

The walls are comprised of solid sawn studs with a single bottom plate (also known as a sole or sill plate) and a double top plate. This example uses modified balloon framing, where the floor and roof joists are hung from the double top plates (Figure 1.4A). This is in contrast to platform framing, where joists bear directly on the top plates (Figure 1.4B).



[a] Blocking above the bottom plate is to provide a nailing surface for the finishes as required. An alternative detail could use two bottom plates, but that would increase shrinkage amounts for the building.

[b] Web stiffeners at joist hangers may be required depending on joist size and manufacturer.

[c] Hangers for the floor joist are installed over the sheathing (gypsum, plywood, or OSB) and must be rated/approved for this installation (e.g., technical bulletin from joist hanger manufacturer listing reduced allowable hanger loads).

FIGURE 1.4A: Typical floor framing at wall

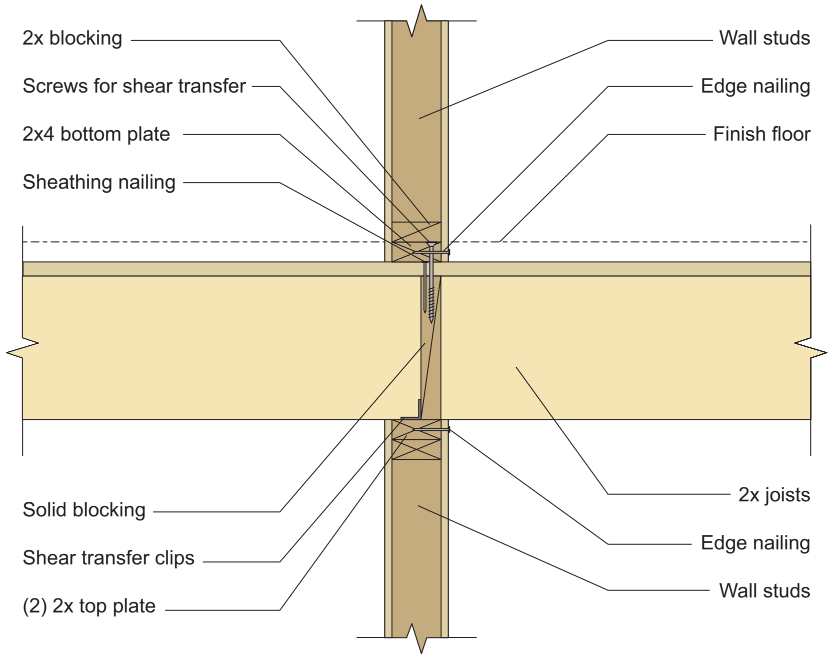


FIGURE 1.4B: Typical platform floor framing at wall using sawn joists
(Not used in this example)

In the high-seismic design example (Section 3), the vertical LFRS is comprised of light-frame wood shear walls sheathed with WSP. In the wind and low-seismic example (Section 4), the light-frame wood shear walls are sheathed with a combination of gypsum wall board (GWB) and WSP. In both, the horizontal LFRS is comprised of sheathed wood-frame diaphragms at each level.

Location of Shear Walls

This design example uses both interior and exterior walls for shear walls. Forces in the transverse direction (north-south) are resisted by unit demising walls (walls separating adjacent guest rooms). The design example illustrates the lateral design (seismic and wind) of a selected interior, transverse shear wall. Forces in the longitudinal direction (east-west) are resisted by the long interior corridor walls located at the center of the structure, on both sides of the corridor, in addition to shear walls on the exterior walls; the design of these walls is not shown in this example.

Another option is to use interior corridor walls only and not place shear walls on the exterior walls for lateral force resistance in the longitudinal direction. This approach relies on rigid diaphragm analysis to distribute lateral forces to the shear walls following the requirements of SDPWS Section 4.2.6 for open front structures. While this is permitted in the code, it requires careful consideration of the horizontal diaphragm deflections and overall building performance. Since open front diaphragm provisions were first introduced, additional clarity has been provided in SDPWS, including some explicit design considerations and reiteration that ASCE 7 story drift requirements for seismic design forces apply to all edges of the structure. See the WoodWorks publication, *A Design Example of a Cantilever Wood Diaphragm*, for more information.

TABLE 1.1: Gravity design loads

Roof Weights (psf)		Floor Weights (psf)	
Roofing	5.0	Flooring	1.0
Sheathing	3.0	Lightweight concrete	14.0
Trusses + blocking	2.0	Sheathing	2.5
Insulation + sprinklers	2.0	I-joist + blocking	4.0
Ceiling + misc.	15.0	Ceiling + misc.	7.0
Beams	1.0	Beams	1.5
Dead load	28.0	Dead load	30.0
Live load	20.0	Live load	40.0

Typical interior and exterior walls can weigh between 10 and 20 psf depending on the assemblies (number of GWB layers, etc.). Dead loads for typical wall assemblies can be found in the Commentary to Standard ASCE/SEI 7-22 (ASCE 7 Commentary) Table C3.1. Note that these weights are applied over the surface area of the wall and are therefore not shown in the floor loads above.

Seismic weights of floor diaphragms typically include one-half of the walls above and below. For this example, an additional 12.5 psf dead load is applied at the floors based on the expected wall assemblies, wall layouts, and 10-foot wall heights. Similarly, seismic weights of roof diaphragms typically include one-half the height of the walls from the sixth floor to the roof, plus the weight of parapets where they occur. In this example, an additional 7 psf dead load is applied at the roof level based on the expected wall assemblies, wall layouts, 10-foot wall heights, and 3-foot parapets at the perimeter. These weights are included in the respective seismic weights of each level, shown below.

TABLE 1.2: Story-level seismic weights

Flexible Upper Portion		Rigid Lower Portion	
W_{roof}	420 k		
W_{6th}	510 k		
W_{5th}	510 k		
W_{4th}	510 k	W_{upper}	2,460 k
W_{3rd}	510 k	W_{2nd}	2,632 k
W_{upper}	2,460 k	W_{total}	5,092 k

1.4.2 Structural Materials

Sheathing

Roof sheathing is 19/32-inch DOC PS 1 or PS 2-compliant sheathing, 40/20 span rating with Exposure 1 glue.

Floor sheathing is 23/32-inch DOC PS 1 or PS 2-compliant sheathing, APA-rated Sturd-I Floor 24 inches o.c. rating with Exposure 1 glue.

Shear wall sheathing, where it occurs, is 15/32-inch DOC PS 1 or PS 2-compliant sheathing, 32/16 span rating with Exposure 1 glue.

Framing Lumber

Throughout this document, the following notations are used to indicate reference design values in accordance with the NDS.

F_b = Reference bending design value, psi

F_c = Reference compression design value parallel to grain, psi

F_t = Reference tension design value parallel to grain, psi

F_v = Reference shear design value parallel to grain (horizontal shear), psi

$F_{c\perp}$ = Reference compression design value perpendicular to grain, psi

E = Reference modulus of elasticity, psi

E_{min} = Reference modulus of elasticity for beam stability and column stability calculations, psi

These notations may also be modified with a prime symbol (′)—e.g., F'_b, F'_c —which represents the adjusted design value (i.e., the reference design value multiplied by all applicable adjustment factors per NDS Section 2.3).

For Douglas fir-larch No. 1, these reference design values are as follows per NDS Supplement Table 4A:

F_b = 1,000 psi

F_c = 1,500 psi

F_t = 675 psi

F_v = 180 psi

$F_{c\perp}$ = 625 psi

E = 1,700,000 psi

E_{min} = 620,000 psi

Calculations throughout this design example show these values adjusted by applicable adjustment factors in NDS Section 2.3.

Fasteners

Common wire nails are used for shear walls, diaphragms, and straps. When specifying nails on a project, the specification should include nail diameter, nail length, and head diameter.

SDPWS lists values for shear walls and diaphragms in Tables 4.3A – 4.3D. In previous versions of SDPWS, these tables specified minimum fastener penetration length as well as a nominal fastener type and size, given in pennyweight. In the 2021 SDPWS, the minimum penetration requirement is unchanged, but fastener type and size have been clarified to require full-length common nails, with dimensions explicitly stated, or galvanized box nail substitutions as allowed by footnote 8. (Fastener dimensions match those found in Appendix A.) For values using fasteners and sheathing thickness not listed, see the WoodWorks articles, *Using Gun Nails in Wood-Framed Shear Walls and Diaphragms* and *Capacities for Shear Walls and Diaphragms with Thick Sheathing*.

2 Fire Protection, Life Safety, and Serviceability

2.1 Height and Area Analysis

Using the special design provision from IBC Section 510.2, this design utilizes a 3-hour fire separation at the first level above grade. This horizontal separation creates two buildings for the purposes of determining building area and number of stories; the overall building height (in feet) is still measured from the grade plane. The floor area of each level is 12,000 square feet; floor-to-floor heights and overall building heights are given in Figure 1.3. The structure is equipped throughout with an NFPA 13 sprinkler system.

2.1.1 Height and Area Assumptions

Lower Portion

Construction Type: I-A per IBC Section 510.2

Occupancies: Any occupancy allowed by code except Group H, per IBC Chapter 3

Height: 12 feet above grade plane (given)

Stories: One story above grade plane (given)

Area: 12,000 square feet (given)

Upper Portion

Construction Type: III-A proposed; to be verified in Sections 2.1.2 and 2.1.4

Occupancy: R-1 per IBC Chapter 3

Height: 62 feet above grade plane (given)

Stories: Five stories above podium (given)

Area: 12,000 square feet per floor (given)

Total area: 60,000 square feet above podium

Special Provision for One-Story Podiums with Parking Only

If the podium portion of a building is one story and only includes parking, the special provision in IBC Section 510.4 may be used instead of the provision in Section 510.2. This would allow the horizontal assembly to be Type IV heavy or mass timber construction with a fire-resistance rating (FRR) determined by the requirements for occupancy separations in IBC Section 508.4. Per IBC Table 508.4, the required separation between R and S-2 occupancies is 2 hours. However, because the ground floor of the hotel in this example includes other occupancies, the provision in Section 510.2 is used with a 3-hour-rated concrete slab as the horizontal separation.

2.1.2 Allowable Building Height

Lower Portion (Type I-A)

Allowable height limit: Unlimited *IBC Table 504.3*

Allowable number of stories: Unlimited *IBC Table 504.4*

The portion of the building below the horizontal assembly is not limited in height because it is Type I-A construction (and, as required for residential occupancies, has an NFPA 13 sprinkler system).

Upper Portion (Type III-A)

Allowable height limit: 85 feet *IBC Table 504.3*

Allowable number of stories: Five *IBC Table 504.4*

The portion of the building above the podium is five stories measured from the top of the podium and 62 feet measured from grade plane. When a building is equipped with an NFPA 13 sprinkler system throughout, the IBC allows five stories and 85 feet for R-2 occupancy in Type III-A construction, as noted. These heights represent an increase of one story and 20 feet over a hypothetical nonsprinklered condition. Because the upper structure is a residential occupancy, a sprinkler system is required. An NFPA 13R system might have been considered, but the use of this system would limit the overall height to four stories and 60 feet so it is not appropriate for this application.

Option for a Mezzanine

Mezzanines can be used to add another partial level to multi-story projects. IBC Section 505 indicates that a mezzanine can be up to one third of the floor area of the room or space below. It is not counted in the allowable building area, nor is it considered a story. However, it does need to be considered in the fire area outlined in IBC Chapter 9. This example does not include a mezzanine.

2.1.3 Seismic Height Limitation

In addition to building height limitations outlined in the IBC, ASCE 7 Table 12-2.1 lists the maximum allowable height of the structure for different seismic force-resisting systems (SFRS) based on the seismic design category (SDC). Section 11.2 defines *structural height* as the vertical distance from the base to the highest level of the SFRS, or the average height of the roof. It defines the *base* of the structure as “the level at which horizontal seismic ground motions are considered to be imparted on the structure.” Due to the rigidity of the concrete podium, the podium slab can be used as the base for light-frame wood walls sheathed with wood structural panels. While this was standard practice for two-stage lateral analysis under previous versions of ASCE 7, new language in ASCE 7-22 confirms that this approach is acceptable (Section 12.2.3.2, item f).

Upper Portion

Height limit in SDC D, E, and F: 65 feet — *ASCE 7 Table 12-2.1*

Structural height of the light-frame wood portion: 50 feet

65 > 50 *Okay*

Note that there is no height limit in SDC B and C, per ASCE 7 Table 12-2.1. The seismic design requirements of Chapter 12 do not apply for SDC A, per Section 11.7.

2.1.4 Allowable Building Area

Lower Portion

Allowable floor area: Unlimited — *IBC Table 506.2*

The portion of the building below the horizontal assembly is not limited in area because it is Type I-A construction and does not contain any group H (hazardous) occupancies.

Upper Portion

Allowable floor area: 72,000 square feet/floor — *IBC Table 506.2*

When a building is equipped throughout with an NFPA 13 sprinkler system, the IBC allows an area of 72,000 square feet per floor for R-2 occupancy, as noted. This represents a threefold increase over a hypothetical nonsprinklered condition. Because the upper structure is a residential occupancy, a sprinkler system is required.

IBC Sections 506.2 and 506.3 allow further increases to allowable area based on frontage, which increases the fire department’s access to the building in the event of a fire. However, the tabulated value of 72,000 square feet is more than sufficient to accommodate this design, which has only 12,000 square feet of R-2 occupancy per floor, without taking advantage of additional frontage increases.

IBC Section 506.2 also stipulates a total building area maximum that must be considered in addition to the floor area maximum. For this multi-story building, the allowable floor area may be multiplied by 3 to determine the total allowable floor area.

72,000 sqft/floor x 3 (for a building with three or more stories) = 216,000 sqft

216,000 sqft > 60,000 sqft *Okay*

In California, Sprinkler Increase is Height or Area (Not Both)

While the IBC allows both a height *and* area increase over the hypothetical nonsprinklered condition when an NFPA 13 sprinkler is installed, the California Building Code (CBC) limits the allowable increase to height *or* area. In this example, to take advantage of the five-story and 85-foot height limits of CBC Tables 504.4 and 504.3, respectively, the area of each floor is limited to 24,000 square feet per CBC Table 506.2. This is adequate for the individual floor areas in this example. However, for residential occupancies, the CBC also limits the allowable *total* building area to a factor of 2 instead of the factor of 3 allowed in the IBC. This results in:

[CBC] 24,000 sqft/floor x 2 (for a building with two or more stories) = 48,000 sqft

[CBC] 48,000 sqft < 60,000 sqft *No good*

In projects such as this, where design floor area exceeds the allowances, fire walls are commonly used to partition the building. In podium construction, fire walls used in the upper portion of the structure need to be vertically continuous and can terminate at the 3-hour horizontal assembly.

So, under the CBC, this building would need to take advantage of frontage increases (not calculated in this example) to increase the allowable building area to at least 60,000 square feet or include fire walls to divide the building into areas not exceeding 48,000 square feet.

2.2 Fire Protection

2.2.1 Fire-Resistance Ratings (FRRs)

FRR requirements for different construction types are outlined in IBC Table 601. For Type III-A construction, all primary structural framing elements, including floors, roofs, and interior bearing walls, require a 1-hour FRR, and exterior bearing walls require a 2-hour FRR. Exterior walls, whether bearing or nonbearing, must also comply with Table 705.5, which provides FRR requirements based on fire separation distance (FSD). Exterior walls in Type III-A residential occupancies must have a minimum 1-hour FRR if the FSD is less than 30 feet; otherwise, Table 705.5 does not require them to be rated (but the 2-hour FRR for exterior bearing walls from Table 601 still applies). For more on this topic, see the WoodWorks article, *How to Determine if Exterior Walls are Load or Non-Load Bearing and Why That's Important*. Additional FRR requirements for residential occupancies are outlined in IBC Section 420. For Type III-A construction, these provisions state that walls and floors separating dwelling or sleeping units must have a minimum 1-hour FRR.

There are several ways to achieve an FRR for a floor or wall assembly. IBC Section 703 outlines methods that include tested assemblies in accordance with ASTM E119 (ASTM, 2024), prescriptive assemblies in accordance with Section 721, and the component additive method in Section 722. Tested fire-rated assemblies can be found in a number of sources including the IBC, the Underwriters Laboratories (UL) Fire Resistance Ratings Guide, the Gypsum Association's Fire Resistance and Sound Control Design Manual (GA, 2024), AWC's 2024 Fire Design Specification for Wood Construction (FDS) (AWC, 2024), and others.

IBC Table 721.1(2) lists prescriptive assemblies and includes fire ratings for various wall construction types. For assemblies marked with footnote m, the design stress for studs with a slenderness ratio, l_e/d, greater than 33 shall be reduced to 78% of the allowable F'_c. For studs with an l_e/d not exceeding 33, the design stress shall be reduced to 78% of the adjusted stress F'_c calculated for studs having an l_e/d of 33. Note that this is an IBC requirement and not an NDS requirement. Table 721.1(2) includes some light-frame wood fire-rated assemblies that do not require footnote m because the walls were tested at 100% of full design load.

Determining Slenderness Ratio

Studs act as both compression members (under axial loads) and bending members (under wind loads or nominal live loads applied to the face of the wall). In both cases, slenderness is an important consideration in determining lateral stability. When a compression edge of a member is fully braced to prevent lateral displacement, the slenderness ratio, l_e/d, becomes infinitely small and the flexural stability and column stability factors can be assumed to be 1.0, per NDS Sections 3.3.3.2 and 3.7.1.1. Studs typically have sheathing on one or both sides, preventing lateral movement along the weak axis. (NDS Appendix A.11.3 states that sheathing on one side only, when adequately fastened, has been shown to provide adequate weak-axis bracing.) However, slenderness of the strong axis still needs to be considered (see NDS Figure 3F).

For a 2x4 stud wall with a 10-foot floor-to-floor height and a 2x4 bottom plate with a double 2x4 top plate:

$l_{e1} =$ the clear height of the studs $= 115.5 \text{ in.}$

$l_{e2} = 0 \text{ in.}$ (fully braced in the weak axis direction)

$l_{e1}/d_1 = 115.5 \text{ in.} / 3.5 \text{ in.} = 33$ (strong axis direction)

This is the controlling l_e/d per NDS Section 3.7.1.3

Meets l_e/d limits of NDS Section 3.7.1.4

Since these studs have an l_e/d of 33, their design stress needs to be reduced to 78% of F'_c if used in one of the prescriptive assemblies in IBC Table 721.1(2) where footnote m applies.

2.2.2 Fire Retardant-Treated Wood (FRTW)

As shown in the heights and areas analysis, the IBC requires buildings with five stories of light-frame wood construction to be classified as Type III. This construction type requires that exterior walls be built with noncombustible materials. However, IBC Section 602.3 provides an exception, stating that fire retardant-treated wood (FRTW) complying with IBC Section 2303.2 is permitted in exterior wall assemblies with ratings of 2 hours or less. This allows light-frame wood construction for many structures where noncombustible materials would otherwise be required. Note that the use of FRTW is independent of FRR considerations; using FRTW does not reduce the required FRR, nor does providing a fire-resistance-rated assembly negate the requirement to use FRTW.

In Type III construction, questions often arise regarding the FRR and FRTW requirements at exterior wall-to-floor intersections. For information on this topic, see the WoodWorks article, *Detailing Floor-to-Exterior Wall Conditions in Type III Projects*.

FRTW Strength Adjustments

IBC Section 2303.2.5 requires that lumber design values be adjusted for the fire-retardant treatment, including any effects of anticipated temperatures and humidity in service. Each manufacturer publishes adjustment factors for service and elevated temperatures in various conditions, such as those experienced by roof/attic framing members. Adjustment factors vary by manufacturer and should be obtained from the product evaluation report, and are in addition to the applicable adjustment factors from NDS Section 2.3.

In particular, an additional adjustment factor for incising may be necessary (C_i), which is not included in the reduction factors provided by the manufacturer. Incising is dependent on treatment formulation as well as species and size of the treated material. For this example, the 2x Douglas fir-larch members will not require incising so the incising factor is not used.

Exposure to Weather
IBC Section 2303.2.6 requires that, when FRTW is exposed to weather, damp, or wet conditions, the identifying label must indicate "EXTERIOR." For this example, all of the wood framing is within the building envelope; therefore, exterior-rated FRTW is not required.

Fasteners
IBC Section 2304.10.6.4 states that fasteners (including nuts and washers) in contact with FRTW used in interior locations shall be in accordance with the manufacturer's recommendations. In the absence of manufacturer recommendations, the requirements of Section 2304.10.6.3 shall apply. Hot-dip galvanized or stainless steel fasteners are the most common for this application. Note that rods in the tiedown system pass through an oversized hole in the wood. Therefore, they are not in direct contact with FRTW and do not need this additional protection.

Cutting and Notching
Treated lumber generally cannot be ripped or milled as this can expose an untreated surface, negating the flame spread resistance. However, ripping FRTW joists or rafters for drainage and placing FRTW plywood on top of the ripped edge is generally considered acceptable.

End cuts and holes are usually permitted; follow guidance from the FRTW manufacturer.

2.3 Vertical Displacement in Multi-Story Wood Framing

Vertical displacement can be a challenge in multi-level wood framing unless adequately accounted for during design and construction. It can be caused by shrinkage, settlement, creep, and/or bearing stresses.

2.3.1 Wood Shrinkage

Both the IBC and NDS require that consideration be given to the effects of cross-grain dimensional changes (shrinkage) when lumber is fabricated in a green condition. In addition, IBC Section 2304.3.3 requires that wood walls and bearing partitions supporting more than two floors and a roof be analyzed for shrinkage of the wood framing, and that possible adverse effects on the structure, installed systems such as mechanical/electrical/plumbing (MEP), and roof drainage be satisfactorily addressed and solutions provided to the building official.

The total potential shrinkage in light-frame wood buildings can be calculated by summing the estimated shrinkage of the *horizontal* lumber members in walls and floors (wall plates and floor joists). Most of the shrinkage is cross grain. The amount of shrinkage parallel to grain (length of studs) is approximately 1⁄40th of the shrinkage perpendicular to grain (cross grain) and can be neglected. For more information, see the WoodWorks paper, *Accommodating Shrinkage in Multi-Story Wood-Frame Structures*.

Following are two methods for determining the amount of wood shrinkage in a multi-story project.

Comprehensive Shrinkage Estimation
When the initial and final moisture content is between 6% and 14% (inclusive), the formula for dimensional change is:

$$S = D_i[C_T(M_F - M_i)]$$

Where:

S = Shrinkage (in.)

D_i = Initial dimension (in.)

C_T = Dimension change coefficient, tangential direction

$C_T = 0.00274$ for Douglas fir-larch

M_F = Final moisture content (%)

M_i = Initial moisture content (%)

When either the initial or final moisture content is less than 6% or greater than 14%, the formula for dimensional change is:

$$S = \frac{D_i(M_F - M_i)}{\frac{30(100)}{S_T} - 30 + M_i}$$

Where, in addition to variables already noted:

$S_T =$ Tangential shrinkage (%) from green to oven dry

$S_T =$ 7.775 for Douglas fir-larch

Both formulas as well as the species-specific variables are from the Wood Handbook: Wood as an Engineering Material (USDA FPL, 2021). Because the Handbook provides variables for individual species, the values for the species group shown are based on an average of relevant values.

The M_F for a building can be higher in coastal areas and lower in inland or desert areas, typically ranging from 6% to 11%. The Handbook provides recommended average moisture content for interior use of wood products in different areas of the U.S.

This design example uses an M_F of 11%.

Project specifications call for all top and bottom plates to be Douglas fir-larch kiln-dried (KD) or surface-dried (S-Dry). Both types have a maximum moisture content of 19% after processing at the lumber mill.

For the modified balloon-frame approach used in this example, the shrinkage zone consists of a double 2x4 top plate and a single 2x4 bottom plate (Figure 2.1A).

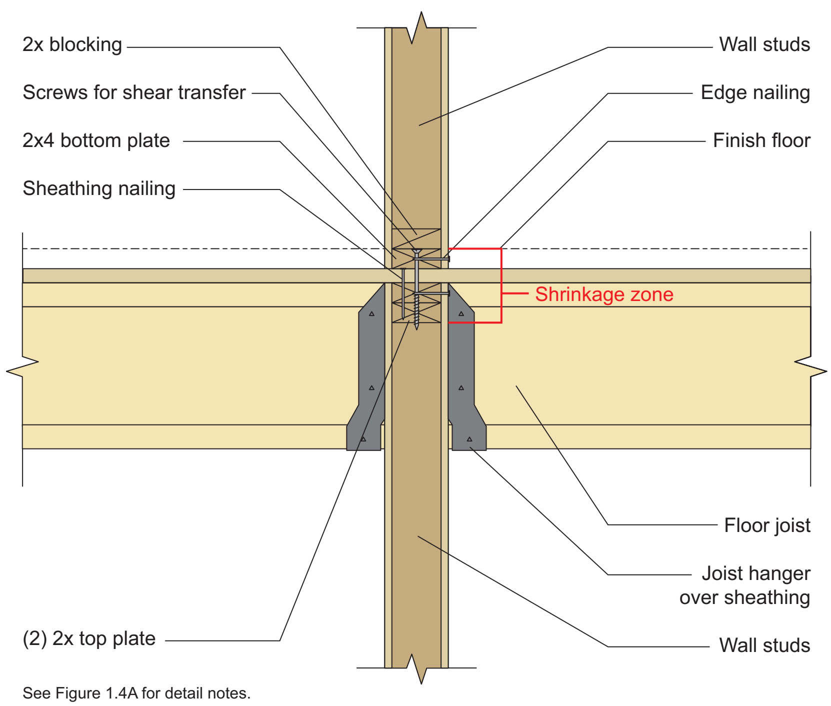


FIGURE 2.1A: Typical floor framing at wall

Assuming an M_i of 19% and an M_F of 11%, the equation is:

$$S = \frac{D_i(M_F - M_i)}{\frac{30(100)}{S_T} - 30 + M_i} = \frac{3(11 - 19)}{\frac{30(100)}{7.775} - 30 + 19} = \text{-0.064}$$

The final size of the double 2x4 top plate is:

$3.0 – 0.064 = 2.936$ in.

Simplified Shrinkage Estimation

A close approximation that is easier to use is:

$$S = C \times D_i(M_F - M_i)$$

Where, in addition to variables already noted:

C = Average shrinkage constant

$$C = 0.0025$$

Using the same M_i and M_F as before, the equation is:

$$S = C \times D_i(M_F - M_i) = 0.0025 \times 3.0(11 - 19) = -0.060 \text{ in.}$$

The final size of the double 2x4 top plate is:

$$3.0 - 0.060 = 2.940 \text{ in.}$$

This simplified estimate is within 0.1% of the calculated dimension of 2.936 inches using the more comprehensive formula. Therefore, we will use the simple estimation approach for the remainder of this example.

Shrinkage of the 2x4 bottom plate is calculated similarly:

$$S = C \times D_i\,(M_F - M_i) = 0.0025 \times 1.5(11 - 19) = -0.030 \text{ in.}$$

Total shrinkage per floor level with the double 2x4 top plate and 2x4 bottom plate:

$$S = 0.060 + 0.030 = 0.090 \text{ in.}$$

Shrinkage Under Platform framing Conditions

While this design example uses modified balloon framing as shown in Figure 2.1A, Figure 2.1B shows another common framing style.

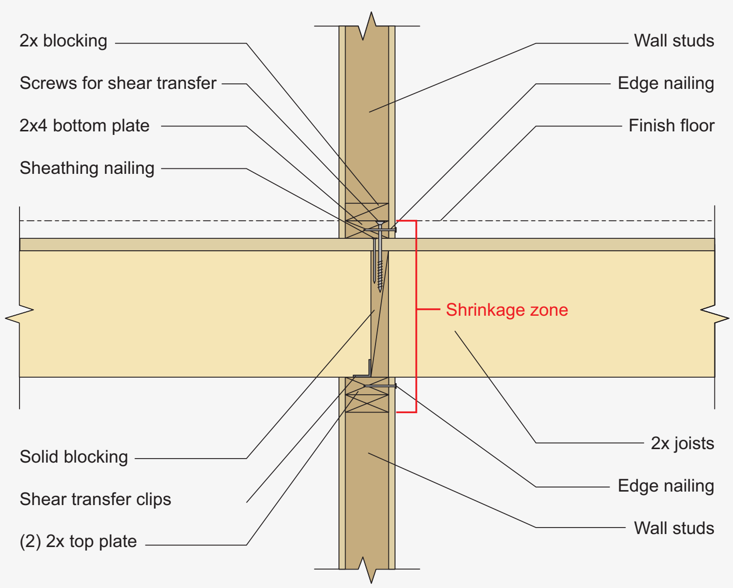


FIGURE 2.1B: Typical platform floor framing at wall using sawn joists

In this method of construction, floor joists sit on the top plate, which can contribute to shrinkage, as shown in the following calculation.

Assuming 2x12 solid sawn floor joists:

$$S = C \times D_i\,(M_F - M_i) = 0.0025 \times 11.25(11 - 19) = -0.225 \text{ in.}$$

Total shrinkage per floor level with the double 2x4 top plate, 2x12 sawn joists and 2x4 bottom plate:

$$S = 0.060 + 0.030 + 0.225 = 0.315 \text{ in.}$$

2.3.2 Settlement Under Construction Gaps

Small gaps can occur between plates and studs, caused by miscuts (short studs) and the lack of square-cut ends, among other things. These gaps can account for up to 1⁄8 inch per story, where "perfect" workmanship would be zero inches and "sloppier" workmanship would be 1⁄8 inch. This design example assumes gaps of 1⁄10 inch per floor.

Combining shrinkage (0.063 inch per floor) and settlement (0.10 inch per floor), vertical displacement at each level is 0.163 inches; cumulative displacements and approximate design displacements over the height of the building are shown in Table 2.1.

TABLE 2.1: Cumulative and approximate design displacements

Level	Vertical Displacement (in.)		Design Displacement (in.)
	Per Level	Cumulative	
Roof	0.190	0.950	1
6th	0.190	0.760	7/8
5th	0.190	0.570	5/8
4th	0.190	0.380	1/2
3rd	0.190	0.190	1/4

2.3.3 Deformation Under Sustained Loading (Creep)

Creep is caused by sustained loading and can occur when wood beams support bearing walls. The rate of creep is higher for beams that are loaded when the wood still has a relatively high moisture content. Where total deflection under long-term loading must be limited, NDS Section 3.5.2 recommends using a time-dependent deformation (creep) factor of between 1.5 and 2.0. In this example, walls are assumed to stack floor to floor; there are no walls supported by beams, so additional deformation is not included.

2.3.4 Reducing and Accounting for Vertical Displacement

Methods to Reduce Vertical Displacement

As discussed, shrinkage depends on three things: the initial moisture content, final moisture content, and thickness of wood in the shrinkage zone. While the designer has little control over the final moisture content, shrinkage can be controlled through careful consideration of the other two factors.

1. Consider using wood with a lower initial moisture content, such as kiln-dried plates (MC < 19%) or even MC15 (MC < 15%) lumber. Engineered lumber is also typically manufactured to a lower initial moisture content and could be used in certain applications.
2. Maintain proper storage of material on site to prevent it from getting wet, ensuring the material is at the desired initial moisture content at the time of installation.
3. To reduce the amount of wood in the shrinkage zone, consider balloon framing or modified balloon framing approaches, as shown in this example.
4. As noted in the discussion about settlement, good workmanship will decrease the amount of vertical movement the building experiences. Off-site construction practices can often provide a high level of quality control and can be a way to address workmanship concerns and build to tighter tolerances. For discussion and resources on off-site construction, see the webpage, *Off-Site/Panelized Construction* at woodworks.org.

Methods to Account for Vertical Displacement

Even with the methods above, some amount of vertical movement is expected. However, through proper design and detailing, the effects of shrinkage and other vertical displacement can be mitigated.

1. Use continuous tiedown systems with shrinkage compensating devices in shear walls. (See Section 3.3.2.)
2. Account for vertical displacement in architectural finish details near floor lines.
3. Ensure rough openings have a minimum 1⁄8-inch gap between window and door tops and the framing lumber.
4. Accommodate anticipated movement of pipes and their connections through wood framing members (such as wall studs). This mitigates the risk of cracked pipes and undesirable pipe sloping. Provide details such as enlarged holes to allow vertical movement and/or flexible plumbing connections. For sample details, see the WoodWorks paper, *Accommodating Shrinkage in Multi-Story Wood-Frame Structures*.
5. Accommodate movement of the structure with building envelope details. In particular, areas that may accumulate moisture need special attention to ensure that movement of the structure does not inhibit positive drainage away from the building. For an example, see the WoodWorks' paper, *Options for Brick Veneer on Mid-Rise Wood-Frame Buildings*.

3 Seismic Design

3.1 Seismic Design of Flexible Upper Portion and Rigid Lower Portion

To determine seismic demands, the flexible upper portion and rigid lower portion of the building are considered separately based on an initial design assumption that a two-stage analysis will be acceptable as permitted in ASCE 7, Section 12.2.3.2. Figure 3.1 illustrates a recommended workflow based on this approach, which will be confirmed in Section 3.2.

Seismic and Site Data

Site class and spectral response acceleration parameters:

For this design example, the authors chose a high-seismicity site with a site-specific soils investigation report, resulting in the following parameters.

Site Class:[1] D

$S_S = 1.808 \text{ g}$

$S_1 = 0.692 \text{ g}$

$S_{DS} = 1.206 \text{ g}$

$S_{D1} = 0.692 \text{ g}$

When not provided in a geotechnical report, the process for determining S_{DS} and S_{D1} is outlined in ASCE 7 Section 11.4.4. This process is outside the scope of this example.

Risk category, importance factor, and seismic design category:

Risk Category: II	*ASCE 7 Table 1.5-1*
Seismic importance factor, $I_e = 1.0$	*ASCE 7 Table 1.5-2*
SDC = D	*ASCE 7 Tables 11.6-1 and 11.6-2*

The process for determining SDC is outside the scope of this example but is outlined in ASCE 7 Section 11.6.

3.1.1 Seismic Design of Flexible Upper Portion

Design Coefficients and Factors for Seismic Force-Resisting Systems

For light-frame walls with wood structural panels that are both shear walls and bearing walls per ASCE 7 Table 12.2-1:

$R = 6.5$

$\Omega_0 = 3.0$*

Maximum height permitted in SDC D is 65 feet

As confirmed in Section 2.1.3, the example building is less than 65 feet.

**For buildings with flexible diaphragms, Ω_0 is permitted to be reduced to 2.5, per ASCE 7-22 Table 12.2-1 footnote b.*

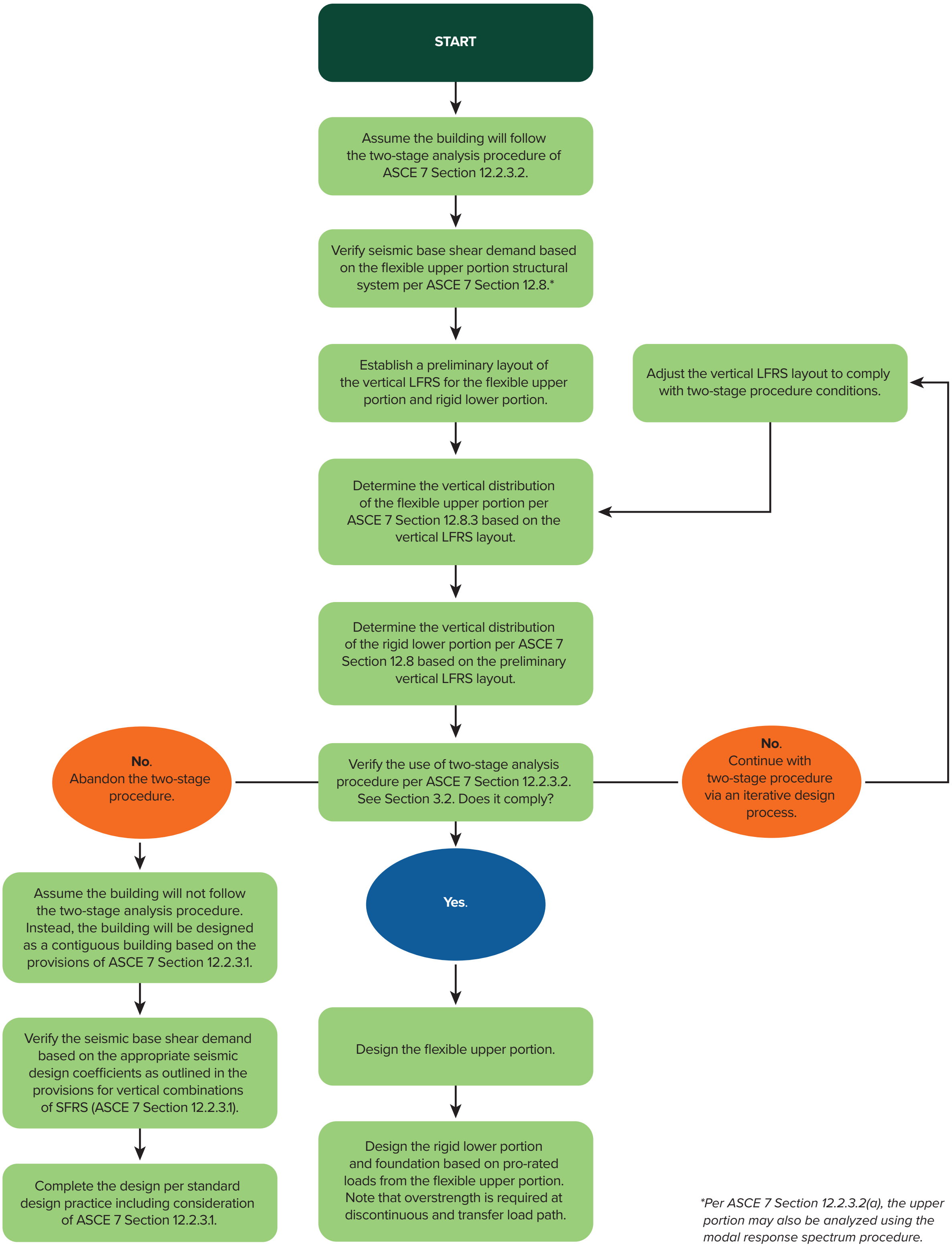


FIGURE 3.1: Seismic design workflow

Determining Period, T_a

The building period for the flexible upper portion is determined using the approximate fundamental period equation in ASCE 7 Section 12.8.2.1, where C_t and x are taken from Table 12.8-2.

$T_a = C_t(h_n)^x = 0.020(50.0)^{0.75} = 0.38 \text{ sec}$ *ASCE 7 Equation 12.8-8*

Seismic Response Coefficient, C_S, and Seismic Base Shear, V

For this design example, the seismic response coefficient, C_S, is determined using the two-period response spectrum per ASCE 7 Section 11.4.5.2 and Method 2 in Section 12.8.1.1. This approach is preferred for this building type over the multi-period response spectrum (MPRS) and Method 1. The MPRS approach can be more sensitive to swings in seismic demands associated with period shifts during design development when designing mid-rise buildings such as the one in this example.

$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)}$ *ASCE 7 Equation 12.8-3*

Upper bound of C_S:

C_S need not exceed

$C_s = \frac{S_{D1}}{T\left(\frac{R}{I_e}\right)}$ for $T \le T_L$ *ASCE 7 Equation 12.8-4*

or

$C_s = \frac{S_{D1}T_L}{T^2\left(\frac{R}{I_e}\right)}$ for $T > T_L$ *ASCE 7 Equation 12.8-5*

Lower bound of C_S:

C_S shall not be less than

$C_S = 0.044 S_{DS} I_e \ge 0.01$ *ASCE 7 Equation 12.8-6*

and when $S_1 \ge 0.6$, C_S shall not be less than

$C_s = \frac{0.5S_1}{\left(\frac{R}{I_e}\right)}$ *ASCE 7 Equation 12.8-7*

For this example, the upper and lower bounds for C_S (Equations 12.8-4, 12.8-5, 12.8-6, and 12.8-7) do not control; C_S is calculated based on Equation 12.8-3.

Therefore:

$C_S = 0.186$

and:

$V = C_S W = 0.186W$

For the flexible upper portion:

$W = 2{,}460 \text{ k}$

$V = C_S W = 0.186(2{,}460) = 458 \text{ k}$

Vertical Distribution of Forces

The main advantage of a two-stage design is that the base for the upper flexible portion is set on top of the podium slab. The heavy mass of the second-floor slab is not inverted into the upper flexible portion of the structure. That is, the base shear of the upper wood portion is based on its own weight (W) without the added weight of the podium.

The base shear is distributed to each level as follows:

$F_x = C_{vx} V$ *ASCE 7 Equation 12.8-12*

Where:

$$C_{vx} = \frac{w_x h_x}{\sum_{i=1}^{n} w_i h_i{}^k}$$ *ASCE 7 Equation 12.8-13*

and:

w_x, w_i is the portion of seismic weight of level x or i

h_x, h_i is the average height at level x or i of the sheathed diaphragm in feet above the base

k is a distribution exponent related to the building period

T_a is less than 0.5 seconds, therefore, k = 1 per ASCE 7 Section 12.8.3.

Story force calculations are summarized in Table 3.1.

TABLE 3.1: Vertical distribution of seismic forces (with base at second floor)

Level	w_x (k)	h_x (ft)	$w_x h_x{}^k$ (k-ft)	C_{vx}	F_x (k)	F_x/A (psf)
Roof	420	50	21,000	29%	133.5	11.12
6th	510	40	20,400	28%	129.6	10.80
5th	510	30	15,300	21%	97.2	8.10
4th	510	20	10,200	14%	64.8	5.40
3rd	510	10	5,100	7%	32.4	2.70
Sum	2,460		7,200	100%	458	

Where: A = Area of the floor plate, which is 12,000 ft²

3.1.2 Flexible vs. Rigid Diaphragm Analysis

Diaphragm flexibility influences how lateral forces are distributed among vertical and horizontal elements. Classification of diaphragm flexibility is addressed in the IBC, SDPWS, and ASCE 7. IBC Section 1604.4 refers to SDPWS for conditions when a wood diaphragm can be idealized as flexible or rigid. SDPWS Section 4.1.7.1 states that diaphragms can be idealized as flexible in accordance with ASCE 7. ASCE 7 Sections 12.3.1.1 and 12.3.1.3 allow diaphragms to be idealized as flexible based on prescriptive criteria or calculations showing the ratio of diaphragm deflection over average drift of the vertical LFRS is greater than two. SDPWS Section 4.1.7.2 provides the inverse of the flexible diaphragm calculation, allowing a diaphragm to be idealized as rigid. If none of the prescriptive or calculated methods of idealizing the diaphragm as flexible or rigid apply, IBC Section 1604.4 provides options for an envelope approach or semi-rigid diaphragm analysis.

In some cases, a diaphragm can meet the prescriptive requirements of ASCE 7 to be idealized as flexible while also qualifying to be idealized as rigid using the SDPWS calculation method. When this occurs, engineering judgment should be used to determine which diaphragm analysis method is most appropriate. For example, even if the project meets prescriptive conditions for a flexible diaphragm, a rigid diaphragm analysis may more accurately distribute forces to the vertical LFRS when the diaphragms have low aspect ratios. These diaphragms tend to be stiff relative to the shear walls and rigid diaphragm analysis may be justified via calculations.

For more background and guidance on diaphragm classification and deflections, see Chapter 6 of WoodWorks' *CLT Diaphragm Design Guide* and the article, *Classifying Wood-Sheathed Diaphragms as Flexible or Rigid*.

Example Project: Flexible Diaphragm Assumption

For structures with light-frame wood shear walls, ASCE 7 Section 12.3.1.1 allows wood diaphragms to be idealized as flexible when one of the following conditions exist:

1. Vertical (seismic) elements are either braced frames (steel or steel and concrete composite) or shear walls (concrete, masonry, steel, or steel and concrete composite).
2. The structure is a one- or two-family dwelling.
3. The structure is light-frame construction and all the following conditions are met:
   a. Concrete toppings are nonstructural and a maximum of 1-1⁄2 inches thick.
   b. Each line of the vertical LFRS complies with the allowable story drift.

In this design example, the third condition is met since our structure does not exceed 1-1⁄2 inches of concrete topping and each line of vertical lateral force-resisting elements complies with the allowable story drift (Section 3.4).

3.1.3 Flexible Upper Portion Redundancy Factor

The redundancy factor, ρ, for the flexible upper portion is 1.0. Both conditions of ASCE 7 Section 12.3.4.2 have been met, though designers are only required to meet one of the two provisions.[2]

3.1.4 Seismic Design of Rigid Lower Portion

Design of the concrete podium structural system is outside the scope of this design example. However, it is important to know that, when using the two-stage analysis procedure, the rigid lower portion must be analyzed and designed following the criteria outlined in Section 12.2.3.2.

Included in these criteria is the requirement that the equivalent lateral force (ELF) procedure be used for analysis. Additionally, whenever the R and ρ values differ between the upper wood structure and lower podium structure, as would be the case with a light-frame wood shear wall system ($R = 6.5$) over a special reinforced concrete shear wall system ($R = 5$), the engineer must scale the seismic reactions from the upper structure before applying them to the lower structure. In other words, seismic forces (i.e., shear and overturning) at the base of the wood structure are scaled up by the ratio of $(R/\rho)_{upper}$ to $(R/\rho)_{lower}$ and then applied to the top of the podium. Additionally, reactions of discontinuous or transfer load paths may require further amplification by the overstrength factor if certain irregularities exist at the transition from the upper to lower portion of the building. See Section 3.5 for further discussion of discontinuous systems.

Gravity loads from the upper portion are not scaled but are applied directly to the lower portion. The lower portion, including the scaled seismic forces from the upper portion, may then be analyzed and designed using the values of R, Ω_0, and C_d for the lower portion of the structure.

See Section 3.2 for a detailed discussion of two-stage design requirements.

3.2 Two-Stage Design for Lateral Analysis

The seismic response coefficient, R (referred to in this publication as the R factor), is 5.0 for the first-floor special concrete shear walls and 6.5 for the wood structural panel shear walls. For structures like the one in this example, with vertical combinations of SFRS, Section 12.2.3.1 of ASCE 7 provides two options:

1. If the lower system has a smaller R factor, the upper and lower systems can each be designed using their respective seismic design coefficients. Forces transferred from the upper system to the lower system must be scaled up by the ratio of the larger R factor to the smaller R factor.
2. If the upper system has a smaller R factor, the entire structure must be designed for this R factor and other corresponding seismic design coefficients.

In either case, Section 12.2.3.1 requires the vertical distribution of seismic forces to be based on the overall base shear of the building, meaning that some of the mass of the podium is inverted into the upper wood stories.

A more realistic approach (from both a seismic and economic perspective) is to design the structure using the two-stage procedure described in ASCE 7 Section 12.2.3.2 and demonstrated in Section 3.1 of this example. This procedure can be used where there is a flexible upper portion and rigid lower portion provided it meets the following criteria:

a. The stiffness of the lower portion must be at least 10 times that of the upper portion. The stiffness for each portion shall be computed as the ratio of the base shear for that portion to the elastic displacement, δ_e, computed at the top of that portion, considering the portion fixed at its base. For the lower portion, the applied forces shall include the reactions from the upper portion, modified as required in item (d). (See Section 3.2.1.)

b. The period of the entire structure must not be greater than 1.1 times the period of the upper portion alone. (See Section 3.2.2.)

c. The upper portion is designed as a separate structure using the appropriate values of R and ρ. (See Section 3.2.3.)

d. The lower portion is designed as a separate structure using the appropriate values of R and ρ while meeting the requirements of ASCE 7 Section 12.2.3.1. The reactions from the upper portion shall be those determined from the analysis of the upper portion, where the effects of the horizontal seismic load, E_h, are amplified as described in ASCE 7. (See Sections 3.2.4 and 3.1.4.)

e. The upper portion is analyzed using either the ELF or modal response spectrum procedure and the lower portion using ELF.

f. The structural height of the upper portion shall not exceed the limit in ASCE 7 Table 12.2-1 for the SFRS where the height is measured from the base of the upper portion. (See Section 2.1.3.)

g. Where a Type 4 horizontal irregularity or Type 3 vertical irregularity exists at the transition from the upper to lower portion, the reactions from the upper portion shall be amplified by Ω_0 in accordance with ASCE 7 Sections 12.3.3.4, 12.10.1.1, and 12.10.3.3. (See Section 3.5.)

For this design example, the upper flexible structure and lower rigid structure are each analyzed using the ELF procedure.[3]

3.2.1 Determining Stiffness

Stiffness of the lower portion must be at least 10 times the upper portion. The stiffness of each portion is computed as the ratio of the base shear to elastic displacement.

Stiffness of the Upper Portion

As discussed in Section 3.1.2, the diaphragm of the upper portion is idealized as flexible and lateral loads are distributed to each wall line based on tributary areas. For brevity, this design example only examines the strength and stiffness of a single transverse wall. This wall was chosen as representative of typical interior transverse walls, which have similar lengths and loads, and the deflection of the entire upper portion is assumed equal to the deflection of this representative shear wall. In a full design using flexible diaphragm analysis, each shear wall line in the upper portion will need to satisfy the strength, stiffness, and allowable drift requirements. In a design using rigid diaphragm analysis, a more holistic analysis incorporating the stiffnesses of all shear walls is needed to develop the stiffness of the upper portion in each direction. In both cases, the stiffness is based on the total base shear and the total elastic displacement of the top of the upper portion.

Upper portion rigidity (stiffness) derivation:

$F = k(\delta_e)$

Rearranging:

$k = F/\delta_e$

Where:

F = Applied force to the upper portion

k = Stiffness of the upper portion

δ_e = Elastic deflection of the upper portion

TABLE 3.2: Stiffness of typical interior wall (upper)

Level	F_x from Table 3.1 (k)	δ_e from Table 3.18 (in.)
Roof	133.5	0.19
6th	129.6	0.28
5th	97.2	0.26
4th	64.8	0.31
3rd	32.4	0.29
Sum	458	1.34

Note: δ_e is determined in Section 3.4 of this design example and presented here for the purpose of verifying the two-stage analysis assumptions. Refer to Figure 3.1 for the recommended workflow for two-stage analysis.

Using the deflection of this shear wall to represent the deflection of the entire upper structure, the equivalent stiffness of the upper portion can be estimated as the total applied force to the upper portion divided by the total elastic deflection of the shear wall, $k_{upper} = 458\text{ k} / 1.34\text{ in.} = 341\text{ k/in.}$

Stiffness of the Lower Portion

The elastic deflection of the rigid lower portion is taken from 3D finite element analysis and includes the effects due to accidental torsion and torsional amplification.

$\delta_e = 0.130\text{ in.}$

$V_{base} = 1{,}232\text{ kips}$

$k = 1{,}232\text{ k} / 0.130\text{ in.} = 9{,}479\text{ k/in.}$

Ratio of rigid lower portion stiffness to flexible upper portion stiffness:

$9{,}479/341 = 28 > 10$ *Okay*

3.2.2 **Determining Period**

Check that the period of the entire structure is not greater than 1.1 times the period of the upper portion.

Building periods are typically determined using the approximate fundamental period equations of ASCE 7. However, the following calculations will show that this often doesn't satisfy the two-stage requirement. The flexible portion was determined in Section 3.1.1 and is repeated below.

For the flexible upper portion:

$T_a = C_t(h_n)^x = 0.020(50.0)^{0.75} = 0.38\ \text{sec}$ *ASCE 7 Equation 12.8-8*

For the entire structure:

$T_a = C_t(h_n)^x = 0.020(62.0)^{0.75} = 0.44\ \text{sec}$ *ASCE 7 Equation 12.8-8*

Ratio of periods:

$0.44/0.38 = 1.18 > 1.1$ *No good*

Using ASCE 7 Equation 12.8-8 can produce period ratios greater than 1.1. This equation is problematic since the same coefficients are used for both wood and concrete shear walls. Instead, an alternative method can be used to determine building periods as shown below.

Alternate Method of Determining Period

$$T = 2\pi\sqrt{\frac{\sum_{i=1}^{n} w_i\delta_i^2}{g\sum_{i=1}^{n} F_i\delta_i}}$$ *ASCE 41 C7.4.1.2.3*

Where:

w_i = Seismic weight assigned to level i

δ_i = Elastic displacement at level i due to forces, F_i

g = Acceleration due to gravity = 32.2 ft/s^2

F_i = Seismic lateral force at level i

The above equation, based on Rayleigh's method, produces a more accurate building period than the approximate fundamental period per ASCE 7 Equation 12.8-8 because it accounts for the building's specific weight and deflections. This equation was in the Uniform Building Code (Equation 30-10 in the 1997 version) and can now be found in the commentary to ASCE 41, Seismic Evaluation and Retrofit of Existing Buildings (ASCE, 2023), as noted above, and in the commentary to FEMA 450, Recommended Provisions for Seismic Regulations for New Buildings and Other Structures (FEMA, 2004), Equation C5.2-1.

TABLE 3.3A: Period of the upper portion using Rayleigh's method

Level	w from Table 3.1 (k)	F from Table 3.1 (k)	Story Level δ_i from Table 3.18 (in.)	Cumulative δ (in.)	$w(\delta^2)$ (kip-in.^{2})	$F(\delta)$ (kip-in.)
Roof	420	133.5	0.19	1.34	752	179
6th	510	129.6	0.28	1.14	667	148
5th	510	97.2	0.26	0.86	376	84
4th	510	64.8	0.31	0.59	180	39
3rd	510	32.4	0.29	0.29	42	9
Sum	2,460	458			2,017	458

Note: The values for δ_i are determined in Section 3.4 of this design example and presented here for the purpose of verifying the two-stage analysis assumptions. Refer to Figure 3.1 for the recommended workflow for two-stage analysis.

$$T = 2\pi\sqrt{\frac{2017}{(32.2 \times 12\ )458}} = 0.67\ \text{sec}$$

TABLE 3.3B: Period of the entire structure using Rayleigh's method

Level	w from Table 3.1 (k)	F from Table 3.1 (k)	Story Level δ_i from Table 3.18* (in.)	Cumulative δ (in.)	$w(\delta^2)$ (kip-in.²)	$F(\delta)$ (kip-in.)
Roof	420	133.5	0.19	1.47	905	196
6th	510	129.6	0.28	1.27	827	165
5th	510	97.2	0.26	0.99	499	96
4th	510	64.8	0.31	0.72	268	47
3rd	510	32.4	0.29	0.42	89	14
2nd	2,632	1,232	0.13	0.13	44	160
Sum	5,092	1,690			2,631	678

**Story level deflection for Level 2 is not shown in Table 3.18, but instead taken from Section 3.2.1 above.*

$$T = 2\pi\sqrt{\frac{2631}{(32.2 \times 12)678}} = 0.63 \text{ sec}$$

Ratio of periods:

$0.63/0.67 = 0.94 < 1.1$ *Okay*

3.2.3 Design of Flexible Upper Portion

Per ASCE 7 Section 12.2.3.1, where the lower system has a lower R factor, the design coefficients (R, Ω_0, C_d) for the upper system can be used to calculate the forces and drifts of the upper system.

As defined in Section 3.1.1, design coefficients for the SFRS from ASCE 7 Table 12.2-1 are as follows:

Type A.16: Light-frame wood walls sheathed with wood structural panels rated for shear resistance

$R = 6.5$

$\Omega_0 = 3.0$*

$C_d = 4.0$

The flexible upper portion will be designed using $R = 6.5$ and the redundancy factor ρ for that portion.

**For buildings with flexible diaphragms, Ω_0 is permitted to be reduced to 2.5, per ASCE 7-22 Table 12.2-1 footnote b.*

3.2.4 Design of Rigid Lower Portion

For the design of the lower system, ASCE 7 Section 12.2.3.1 requires that, where the lower system has a lower R factor, the design coefficients (R, Ω_0, C_d) for the lower system shall be used. In addition, the seismic reactions from the upper portion shall be scaled up by the ratio of $(R/\rho)_{upper}$ to $(R/\rho)_{lower}$. See Section 3.1.4 for further discussion.

Design coefficients for the SFRS from ASCE 7 Table 12.2-1 are as follows:

Type A.1: Special reinforced concrete shear walls

$R = 5.0$

$\Omega_0 = 2.5$

$C_d = 5.0$

The rigid lower portion will be designed using $R = 5.0$ and the redundancy factor ρ for that portion.

3.3 Shear Wall Design Example

This design example uses the segmented shear wall approach described in SDPWS Section 4.3.2.1, where individual full-height wall segments with aspect ratio limitations from SDPWS Section 4.3.3 are used to resist the applied loads. The wall studied in this example is 29 feet long and stacked over the full five stories of the wood portion of the building, with a floor-to-floor height of 10 feet for each segment.

Check h/w ratio for shear wall segments:

Segment height, $h = 10.0 \text{ ft}$

Segment width, $w = 29.0 \text{ ft}$

$h/w = 10.0/29.0 = 0.34 < 2.0$ *Okay*

3.3.1 ASCE 7 Load Combinations

Wood buildings are traditionally designed using allowable stress design (ASD) with the following load combinations from ASCE 7 Section 2.4.5. These combinations have been simplified based on the loads being considered in this example (e.g., zero snow load).

$1.0D + 0.7E_v + 0.7E_h$ — *ASCE 7 Section 2.4.5 combination 8*

$1.0D + 0.525E_v + 0.525E_h + 0.75L$ — *ASCE 7 Section 2.4.5 combination 9*

$0.6D - 0.7E_v + 0.7E_h$ — *ASCE 7 Section 2.4.5 combination 10*

Where:

E_h = Horizontal seismic load effect, ρQ_E — *ASCE 7 Equation 12.4-3*

E_v = Vertical seismic load effect, $0.2S_{DS}D$ — *ASCE 7 Equation 12.4-4a*

From Section 3.1 of this example, $S_{DS} = 1.206$

Aspects of the design requiring the use of strength design (LRFD) or drift load combinations will be discussed throughout this example, most notably the deflection analysis in Section 3.4.

ASCE 7 also provides a horizontal seismic force effect including overstrength, represented as E_{hm}, which would replace E_h when the use of overstrength loads is required. This will be addressed in Section 3.5.

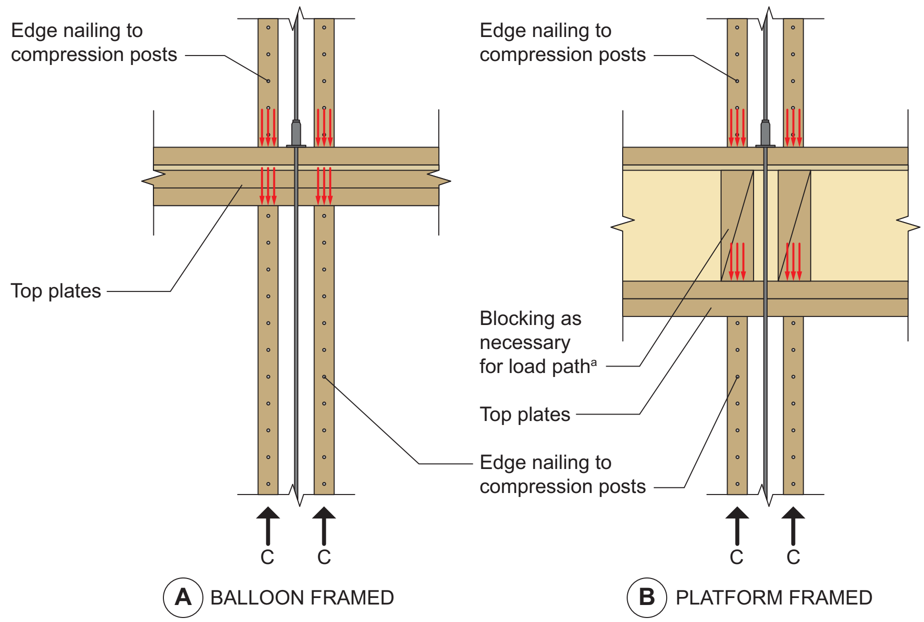
3.3.2 Shear Wall Load Path

Shear walls resist horizontally applied lateral forces in a similar manner to diaphragms: sheathing fastened to framing members acts as the web to resist shear across the length of wall, while chords (also called boundary elements) resist flexure at each end. The sheathing and nailing design is addressed in Section 3.3.4 while the design of chords is addressed in Sections 3.3.5 (compression) and 3.3.6 (tension).

In this example, a continuous tiedown system acts as the vertical chord, with boundary posts/studs taking the compression forces and tiedown rods taking the tension forces. The load path for the continuous tiedown system is as follows:

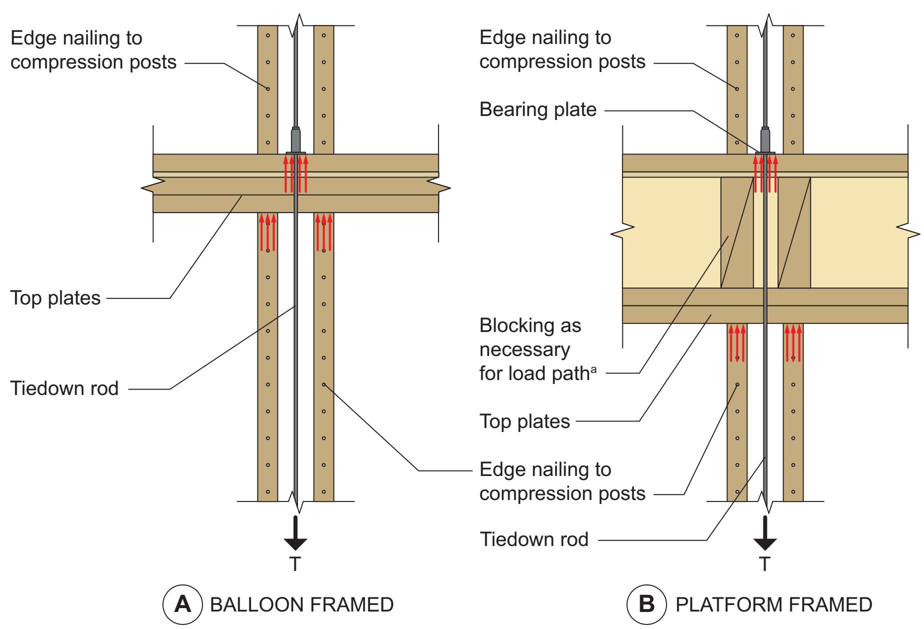
- Shear wall overturning forces are transferred to the chords via edge nailing of the sheathing to each chord member.
- When the shear wall end is in compression, the end chord members create a compression bearing path from the posts through blocking at the floor levels and then to the next set of posts below (Figure 3.2).
- When the shear wall end is in tension, the end chord members lift up and bear in compression on the floor (or roof) above. The bearing plate resists the individual story overturning force (i.e., the differential tension force) by restraining the posts below from uplifting (Figure 3.3). The bearing plates transfer the uplifting forces from the posts to the tension rod.

Therefore, both compression and tension loads are transferred via wood bearing through the thickness of the framing plates to the resisting elements (compression posts and tension rods, respectively). As a general rule, bearing stresses can be distributed over a bearing area located within a 45-degree plane with the loaded surface (Figure 3.4). If the forces require a bearing area larger than that plane, more investigation may be needed (e.g., bending and shear checks of the plates).



[a]Blocking at platform-framed detail (B) may have a single block with a drilled hole for the tiedown rod.

FIGURE 3.2: Compression load transfer



[a]Blocking at platform-framed detail (B) may have a single block with a drilled hole for the tiedown rod.

FIGURE 3.3: Tension load transfer

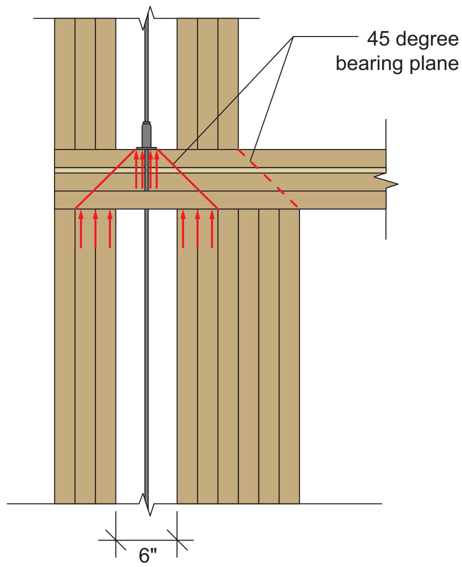


FIGURE 3.4: Bearing zones through framing

Avoid Skipping Levels

To reduce costs, some manufacturers "skip" levels, providing bearing plate restraints at every other floor. In this design example, a skipped design would omit bearing devices at the fourth and sixth floors and include restraints at the third, fifth, and roof levels. When floors are skipped, the magnitude of tiedown assembly displacement is cumulative between the bearing restraints, significantly increasing shear wall deflection(s). Skipping floors is not recommended and in some jurisdictions prohibited.

Continuous tiedowns have several advantages over conventional hold-downs—such as higher uplift capacities and reduced vertical deformation, leading to reduced lateral drift. Most conventional hold-downs do not offer the capacities needed for multi-level construction nor the shrinkage compensating devices that are available in continuous tiedown systems.

In shear walls with conventional hold-downs, overturning is resisted by chord posts that take both the compression and tension forces, and the hold-downs transfer tension forces between stories.

3.3.3 Lateral Load Demands on Shear Walls

The structure used in this design example has interior shear walls located at every other unit demising wall (i.e., walls between adjacent guest units). Using a tributary area approach to distribute story-level forces to each wall, the loads on a typical shear wall, based on a 845 square-foot tributary area, are shown in Table 3.4. This design example does not address the use of multiple individual wall segments within a given line of resistance; detailed information on analysis methods for that situation can be found in the WoodWorks article, *Calculating the Capacity of Multiple Shear Walls in a Line*.

TABLE 3.4: Seismic forces to shear walls

	Story-Level Forces		Cumulative Forces
Level	Designation in Figure 3.5	F (lb)	F_{total} (lb)
Roof	F_{roof}	9,397	9,397
6th	F_6	9,129	18,526
5th	F_5	6,847	25,373
4th	F_4	4,564	29,938
3rd	F_3	2,282	32,220

3.3.4 Shear Wall Design: Sheathing and Nailing

Shear Demand

As shown in Figure 3.5, the shear walls are designed based on *cumulative* shear forces, F_{total}, calculated in Table 3.4. For the ASD load combinations noted previously, the horizontal seismic load effect, E_h, will have a factor of 0.7. This is reflected in Table 3.5, where $V = 0.7(F_{total})/l$.

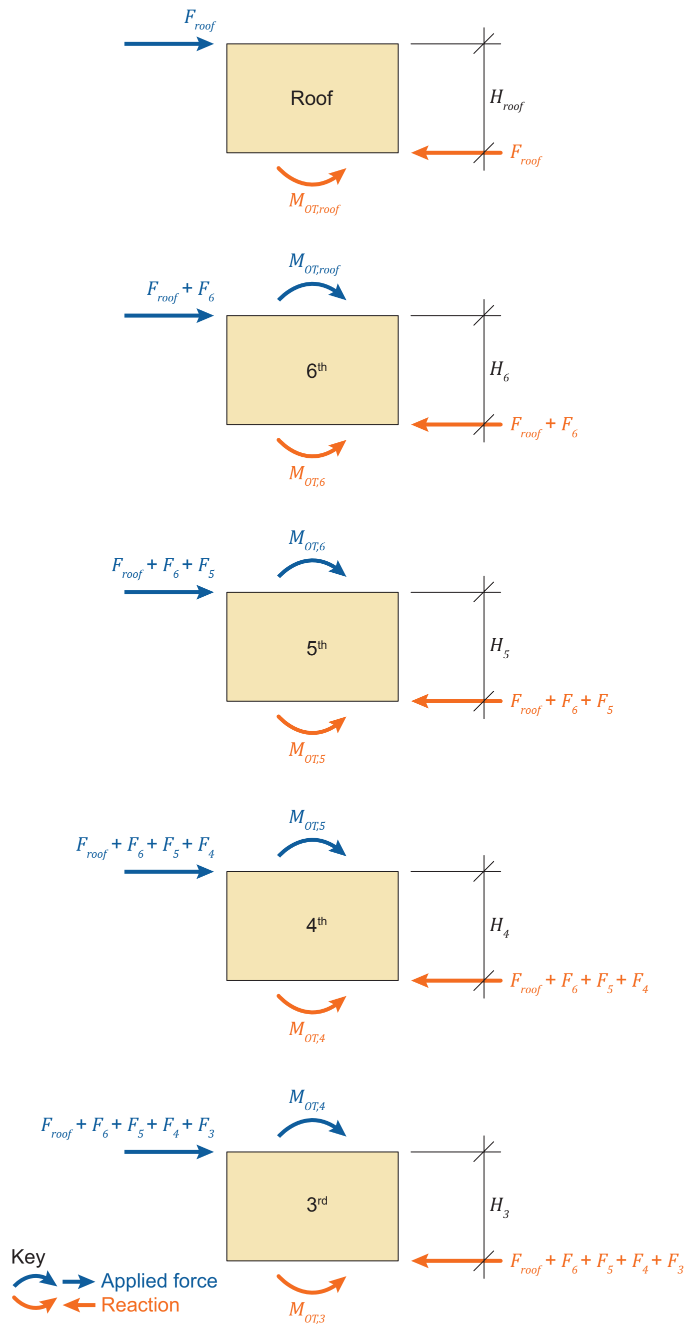


FIGURE 3.5: Cumulative shear and moment demands in multi-story shear walls

Shear Capacity

The shear walls will have 15⁄32-inch oriented strand board (OSB) sheathing nailed with 10d common nails (3 inches long x 0.148-inch shank diameter x 0.312-inch head diameter) with a minimum penetration of 1-1⁄2 inches into the framing members. Capacities are determined by taking the nominal unit shear capacities in SDPWS Table 4.3A and dividing by the seismic ASD reduction factor of 2.8.

While 2x framing is typical—e.g., top plates, bottom plate, and studs—SDPWS Section 4.3.7.1 item 5 lists conditions where 3x nominal framing and staggered nail spacing is required. Note that 3x nominal framing is required at abutting panel edges anytime 10d nails are spaced at 3 inches o.c. or less or when the required nominal shear capacity exceeds 980 plf in SDC D, E, or F. If two panels do not abut on a single framing member, 2x material is acceptable for these conditions.

Double-sided shear walls are often needed for buildings more than three or four stories tall in high-seismic regions. They are typically designed with the same sheathing and nailing pattern on each side of the wall, leading to double shear wall capacity in accordance with SDPWS Section 4.3.5.4.1. When single-sided shear wall fastener spacing is 2 inches o.c., some engineers use a double-sided wall with fasteners spaced at 4 inches o.c. for better performance and less drift.

TABLE 3.5: Shear wall forces and nailing (ASD)

Level	F_{total} from Table 3.4 (lb)	Wall Length, l (ft)	ASD Design Shear, V (plf)	Wall Sheathed 1 or 2 Sides	Fastener Edge Spacing (in. o.c.)	Allowable Shear (plf)
Roof	9,397	29	227	1	6	310
6th	18,526	29	447	1	4	460
5th	25,373	29	612	1	2	770
4th	29,938	29	723	1	2	770
3rd	32,220	29	778	2	4	920

3.3.5 Shear Wall Design: Chord Members in Compression

Compression Demand from Seismic Overturning

When designing multi-story structures, shear and the resulting overturning forces from wind and seismic loads are *cumulative*; shear forces applied at upper levels will generate much larger base overturning moments than the same shear forces applied at lower levels. Overturning moments are resolved into tension-compression (T-C) couples, resulting in axial forces that are transferred by vertical chord elements to the podium and ultimately the foundation, as previously described.

The overturning moments for the shear wall can be obtained by summing forces about the base of the wall for the level being designed (Figure 3.5).

Cumulative overturning moment for the roof level:

$$M_{OT,roof} = F_{roof}(H_{roof})$$

Cumulative overturning moment for the sixth-floor level:

$$M_{OT,6} = M_{OT,roof} + (F_{roof} + F_6)(H_6) = F_{roof}(H_{roof} + H_6) + F_6(H_6)$$

Cumulative overturning moment for the fifth-floor level:

$$M_{OT,5} = M_{OT,6} + (F_{roof} + F_6 + F_5)(H_5) = F_{roof}(H_{roof} + H_6 + H_5) + F_6(H_6 + H_5) + F_5(H_5)$$

Cumulative overturning moment for the fourth-floor level:

$$M_{OT,4} = M_{OT,3} + (F_{roof} + F_6 + F_5 + F_4)(H_4) = F_{roof}(H_{roof} + H_6 + H_5 + H_4) + F_6(H_6 + H_5 + H_4) + F_5(H_5 + H_4) + F_4(H_4)$$

Cumulative overturning moment for the third-floor level:

$$M_{OT,3} = M_{OT,4} + (F_{roof} + F_6 + F_5 + F_4 + F_3)(H_3) = F_{roof}\,(H_{roof} + H_6 + H_5 + H_4 + H_3) + F_6\,(H_6 + H_5 + H_4 + H_3) + F_5\,(H_5 + H_4 + H_3) + F_4\,(H_4 + H_3) + F_3\,(H_3)$$

The T-C couple used to design the chord elements (Table 3.6) is determined by dividing the overturning moment by the distance, d (referred to as b_{eff} in SDPWS), between the center of the tension rod and centroid of the compression posts (Figures 3.6 and 3.7). When considering only the horizontal component of the seismic forces, E_h, the tension and compression forces are equal and opposite.

TABLE 3.6: Overturning moments and tension-compression couple forces

Level	Story-Level Force, F, from Table 3.4 (lb)	Story Height (ft)	Cumulative M_{OT} (ft-k)	d (ft)	Seismic T-C Couple, + or – (k)
Roof	9,397	10	94.0	27.75	3.39
6th	9,129	10	279.2	27.65	10.10
5th	6,847	10	533.0	27.40	19.45
4th	4,564	10	832.3	27.18	30.63
3rd	2,282	10	1,154.5	26.97	42.81

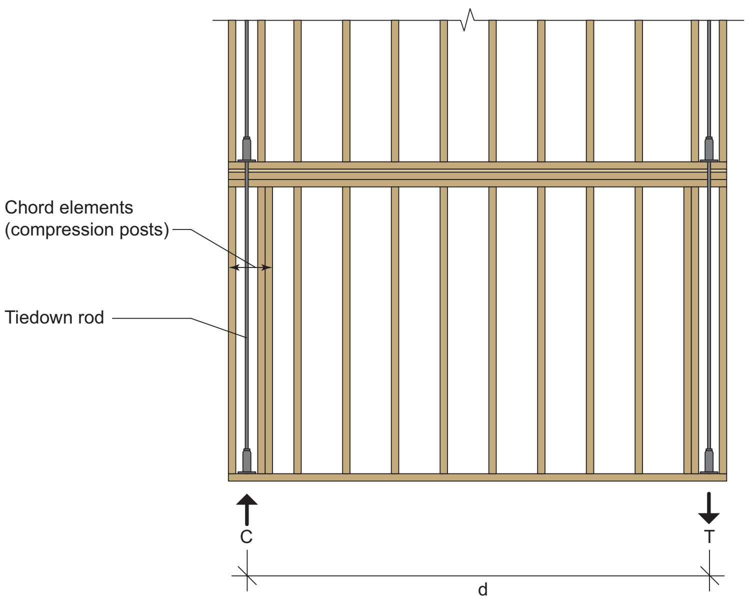


FIGURE 3.6: Shear wall elevation with distance, d

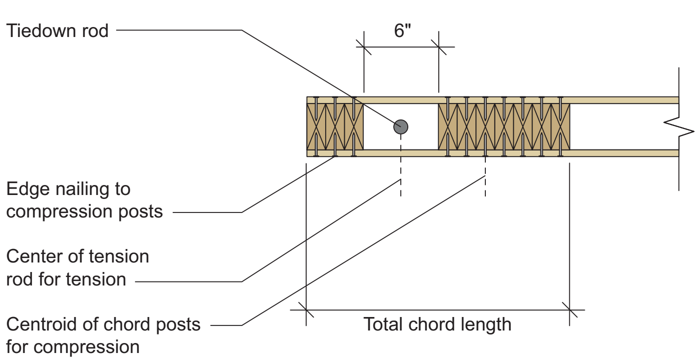


FIGURE 3.7: Example plan view at chords

In most designs, the size and number of chord elements change from story to story as shown in Figure 3.8, which can necessitate iterations to derive the actual distance, d. Some engineers will take a conservative estimate of d and use the same value for all levels to minimize iterations. However, this example uses actual distances based on the sizes of the as-designed compression posts.

In Figures 3.7 and 3.8, the compression posts are shown in an asymmetrical configuration about the tension rod. Most tiedown systems accommodate this type of arrangement, which allows the end-of-wall location to be consistent from floor to floor. This is a common approach and is used in this design example. Alternatively, the compression posts could be arranged symmetrically about the tension rod, which would result in longer shear walls at lower levels where more compression posts are used.

Most tiedown system manufacturers also permit small rod offsets from floor to floor, allowing for minor deviations in the size of the end-most compression posts at each level. In this example, (3) 2x compression posts were consistently used at the outermost ends of the shear walls, so rod offsets were not needed.

Lastly, in keeping with the assumption that bearing is distributed over a 45-degree angle through the framing plates (Figure 3.4), no more than three compression studs were added at each level, moving down the building as shown in Figure 3.8.

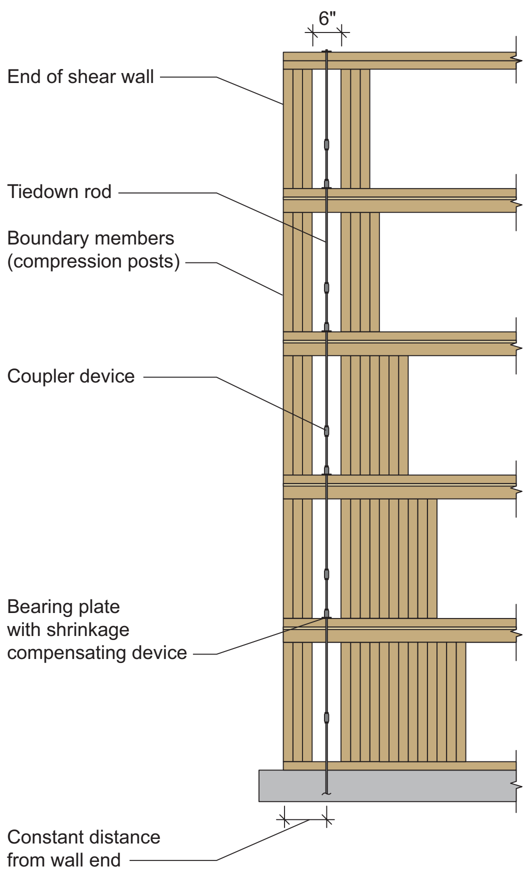


FIGURE 3.8: Example elevation of shear wall chords

Compression Demand from Gravity Loads

As mentioned, tension and compression forces in the chords are equal and opposite when considering horizontal seismic forces only. However, when considered in combination with dead, live, and other design loads, the compression and tension demands will be different.

Dead and live loads tributary to the wall are calculated as follows (line loads):

Dead:

$W_{Roof} = (28.0 \text{ psf})(2.0 \text{ ft}) = 56 \text{ plf}$

$W_{Floor} = (30.0 \text{ psf})(13.0 \text{ ft}) = 390 \text{ plf}$

$W_{Wall} = (10.0 \text{ psf})(10.0 \text{ ft}) = 100 \text{ plf}$

Live:

$W_{Floor} = (40.0 \text{ psf})(13.0 \text{ ft}) = 520 \text{ plf}$

Roof live:

$W_{Roof} = (20.0 \text{ psf})(2.0 \text{ ft}) = 40 \text{ plf}$

TABLE 3.7: Cumulative gravity loads

Level	D (plf)	L (plf)	L_r (plf)
Roof	156	0	40
6th	646	520	40
5th	1,136	1,040	40
4th	1,626	1,560	40
3rd	2,116	2,080	40

These gravity loads will be resisted by individual wall studs as well as the chords at the end of the shear wall. Therefore, each shear wall chord will be designed to resist a compression force equal to the line loads provided in Table 3.7 multiplied by the total length of the chord at that level (i.e., the length of the tiedown system—including all compression posts and the 6-inch gap for the tiedown rod, Figure 3.7). As with the calculation of d, it can be an iterative process to determine chord length. While some engineers use a conservative estimate, this design example uses the actual chord length based on the final design.

For this example, ASD load combination 8 will govern the compression forces on the chord members; the resulting forces are summarized in Table 3.8. (Floor and roof live loads are not used in this load combination but are shown for completeness.)

$1.0D + 0.7E_v + 0.7E_h$ *ASCE 7 Section 2.4.5 combination 8*

$= (1.0 + 0.7(0.2S_{DS}))\text{D} + 0.7E_h$

$= 1.17\text{D} + 0.7E_h$

TABLE 3.8: Compression chord forces (ASD)

Level	Total Chord Length (ft)	P_D (k)	P_L (k)	P_{Lr} (k)	Seismic Compression Force from Table 3.6, E_h (k)	Total Compression Demand (Load Combo 8) $1.17D + 0.7E_h$ (k)
Roof	1.250	0.20	0.00	0.05	3.39	2.60
6th	1.375	0.89	0.72	0.06	10.10	8.11
5th	1.750	1.99	1.82	0.07	19.45	15.94
4th	2.125	3.46	3.32	0.09	30.63	25.48
3rd	2.500	5.29	5.20	0.10	42.81	36.15

Compression Capacity of Chord

Assuming interior walls use 2x4 framing, chord posts will have a nominal 4-inch dimension to match the thickness of the wall. An example calculation for the capacity of a single 2x4 compression post is provided below.

2x4 Post – Douglas fir-Larch No. 1:

$d_1 = 3.5$ in.

$d_2 = 1.5$ in.

$l_1 = 10$ ft floor-to-floor height minus thickness of top and bottom plates

$= 10$ ft – 3.0 in. – 1.5 in. $= 9.625$ ft

Note that l_2 = 0 feet because the axis that is in-plane with the wall is fully braced by sheathing on one or both sides. (See Section 2.2.1 for more information.)

Therefore, the out-of-plane axis will control. (For a 2x4, the strong axis aligns with the out-of-plane direction. However, for a 4x6 or larger member, the weak axis will align with the out-of-plane direction and will control the design.)

$K_{e1} = 1.0$ *NDS Appendix G*

$l_{e1} = K_{e1}(l_1) = 9.625$ ft $= 115.5$ in.

$l_e/d = 115.5$ in. / 3.5 in. $= 33$

$E'_{min} = E_{min}\,(C_M)(C_t)(C_i)(C_T)$

For this example, C_i and C_T do not apply. Additionally:

$C_M = 1.0$

$C_t = 1.0$

From Section 1.4.2:

$E_{min} = 620{,}000$ psi

Therefore:

$E'_{min} = 620{,}000$ psi

Compression parallel to grain:

$F'_c = F_c(C_D)(C_M)(C_t)(C_F)(C_i)(C_P)$

In addition to the variables already noted:

$C_D = 1.6$ for seismic load combinations

$F_c = 1{,}500$ psi (Section 1.4.2)

Additionally,

$C_F = 1.15$ for a 2x4 member, from NDS Table 4A Adjustment Factors

C_P must be calculated based on the formula below (NDS equation 3.7-1).

$$C_P = \frac{1 + (F_{cE}/F_c^*)}{2c} - \sqrt{\left[\frac{1 + (F_{cE}/F_c^*)}{2c}\right]^2 - \frac{(F_{cE}/F_c^*)}{c}}$$

Where:

$c = 0.8$ for sawn lumber

$$F_{cE} = \frac{0.822E'_{min}}{(l_e/d)^2}$$

$= 468$ psi

$F_c^* = F_c(C_D)(C_M)(C_t)(C_F)(C_i)$ (i.e., the calculation for F'_c without the C_P factor applied)

$= 2{,}760 \text{ psi}$

$F_{cE}/F_c^* = 0.17$

$C_P = 0.16$

Therefore:

$F'_c = 450 \text{ psi}$

Compression perpendicular to grain:

$F'_{c\perp} = F_{c\perp}(C_M)(C_t)(C_i)(C_b)$

Where, in addition to the variables already noted:

$F_{c\perp} = 625 \text{ psi}$ (Section 1.4.2)

C_b, does not apply (NDS Section 3.10.4)

Therefore:

$F'_{c\perp} = 625 \text{ psi}$

The allowable compression capacity of the stud, P_{allow}, will be the lesser of F'_c and $F'_{c\perp}$ multiplied by the area of the stud. Similar calculations can be performed for any compression post size. This example is based on the use of multiple 2x4 members, which will be stitch-nailed together to act as the compression posts. In Table 3.9, individual post capacities are multiplied by the total number of posts provided at each end of the wall to determine total chord post capacity, P_{allow} (i.e., composite action is not assumed). In lieu of multiple 2x members, the designer may use fewer and larger compression posts to reduce the amount of stitch and edge nailing required.

TABLE 3.9: Compression chord design

Level	Chord Posts	Area (in.^{2})	F'_c (psi)	$F'_{c\perp}$ (psi)	Demand from Table 3.8 (k)	P_{allow} (k)	DCR*
Roof	(6) 2x4	31.5	450	625	2.60	14.19	18%
6th	(7) 2x4	36.8	450	625	8.11	16.55	49%
5th	(10) 2x4	52.5	450	625	15.94	23.65	67%
4th	(13) 2x4	68.3	450	625	25.48	30.74	83%
3rd	(16) 2x4	84.0	450	625	36.15	37.84	96%

**Demand capacity ratio*

Buckling of Rim Board

When a rim board is placed between the top plate(s) and floor sheathing and there is a large compressive load from boundary post(s), there is the potential for buckling of the rim board. This buckling effect can lead to loss of structural capacity and significantly increase deformations. Squash blocks should be used to prevent this from occurring (Figure 3.2, detail B). This design example utilizes modified balloon framing, which eliminates this effect (Figure 3.2, detail A).

3.3.6 Shear Wall Design: Chord Members in Tension

Tension Demand from Seismic Overturning and Dead Load Resistance

Similar to compression, the tension demand on the chord is determined by combining seismic overturning loads with gravity loads. For this example, ASD load combination 10 will govern the tension forces on the chord members.

$0.6D - 0.7E_v + 0.7E_h$ *ASCE 7 Section 2.4.5 combination 10*

$= (0.6 - 0.7(0.2S_{DS}))D + 0.7E_h$

$= 0.43D + 0.7E_h$

Using dead loads to resist overturning moment forces is common practice and allowed per SDPWS Section 4.3.6.4.2. To simplify calculations, some engineers choose to neglect dead loads when calculating uplift forces; however, they are included here for completeness. For more information, see the WoodWorks article, *Using Dead Loads to Resist Shear Wall Overturning*.

The resisting moment M_R at each level is calculated based on the dead load, D, as follows:

$M_R = wL^2/2 =$ [cumulative dead load][wall length]$^2/2$

The seismic overturning moment, M_{OT}, is the same as calculated in Table 3.6 but is now shown as a negative value (based on the sign convention used), indicating that it acts opposite the resisting moment.

Once the net resultant moment is calculated based on the appropriate load combinations, it is resolved into a tension force at the end of the wall by dividing by d. (Based on the sign convention used, tension is indicated as a negative value.) Calculations are summarized in Table 3.10.

TABLE 3.10: Tension chord forces (ASD)

Level	Cumulative Resisting Moment (Dead Load) M_R (ft-k)	Cumulative M_{OT} from Table 3.6 (ft-k)	Net Resultant Moment (Load Combination 10) $0.43M_R + 0.7M_{OT}$ (k-ft)	d (ft)	Net Resultant Tension Force $0.43D + 0.7E_h$ (k)	Differential Load Per Floor (k)
Roof	65.6	-94.0	-37.5	27.75	-1.351	-1.351
6th	271.6	-279.2	-78.3	27.65	-2.833	-1.482
5th	477.7	-533.0	-167.1	27.40	-6.099	-3.266
4th	683.7	-832.3	-287.8	27.18	-10.591	-4.492
3rd	889.8	-1,154.5	-424.5	26.97	-15.742	-5.151

Tension Capacity

Tiedown Rods

Smaller diameter tiedown rods can follow threaded rod or anchor rod specifications and are usually made from A36 or F1554 steel per Table 2-6 of the Steel Manual. A36 steel complies with the ASTM A36 standard (ASTM, 2019) and has a minimum yield strength of 36 ksi (i.e., grade 36). F1554 steel complies with the ASTM F1554 standard (ASTM, 2020) and is available in three grades: 36, 55, and 105 ksi. Standard strength rods are 36 ksi and should be considered the default unless the rods are marked otherwise. High-strength rods can be 55 ksi, 105 ksi, or other strengths, and should be marked as such. While a detailed discussion of steel grades is outside the scope of this example, it is not permissible to directly substitute high-strength rods for regular strength rods due to potential differences in weldability and the possibility of introducing brittle failures when ductile behavior is expected.

The tensile capacity of a rod, in ASD, is given via equation J3-1 in the Steel Manual:

$R_n/\Omega = F_n A_b/2.00$

Where:

$F_n = 0.75F_u$ *Steel Manual Table J3.2*

This example uses standard ASTM F1554, Grade 36 rods with F_y = 36 ksi and minimum F_u = 58 ksi. High-strength rods are not used in this example.

$A_b =$ Nominal area of the rod, which equals the gross rod area, A_g

$A_g = \frac{\pi d^2}{4}$, where d is the nominal rod diameter

Gross rod areas can also be found in Table 7-17 of the Steel Manual.

Rod capacities are calculated in Table 3.11. Rod diameters may need to be larger than the size required to meet tension demands in order to reduce rod elongations and resulting shear wall deflections. Spreadsheets allow the engineer to make rod diameter adjustments quickly without having to redo numerous calculations.

TABLE 3.11: Tension rod design

Level	Tension Demand from Table 3.10 (k)	Rod Diameter, d (in.)	A_g (in.^{2})	F_u (ksi)	F_{nt} (ksi)	Rod Capacity (k)	DCR
Roof	1.351	5/8	0.307	58	43.5	6.67	20%
6th	2.833	5/8	0.307	58	43.5	6.67	42%
5th	6.099	3/4	0.442	58	43.5	9.61	63%
4th	10.591	1	0.785	58	43.5	17.08	62%
3rd	15.742	1 1/8	0.994	58	43.5	21.62	73%

Rod Couplers

Couplers are used to connect the rods. They can be straight to connect two rods of the same size, or reducing to connect two different-sized rods, and come in different strengths. Couplers for high-strength rods need to be of high-strength steel and marked as such. Rods must be set far enough into the coupler that they are able to develop their full strength (usually the depth of a standard nut). “Pilot” or “witness” holes in the side of the coupler help confirm proper embedment.

In reducing couplers, the size of the threading changes in the middle. Rods should be embedded until they reach the center and can’t go any further.

Bearing Plates

Bearing plates transfer the tension load from the structure, via the bottom plate, top plates, or bridging, into the rod (Figure 3.3).

An example calculation for the design of bearing plates at Level 3 is shown below.

Bearing plate design is based on ASTM A36 steel with $F_y = 36$ ksi.

Initial bearing plate sizes (length and width) are estimated based on allowable bearing capacity. To fit within the 2x4 stud wall, plates are limited to 3 inches wide. The bearing plate at Level 3 will be checked as 3.0 x 4.0 x 5/8 inches.

Allowable bearing capacity (ASD):

$F'_{c\perp}\ (A_{Brg})$

Where:

$F'_{c\perp}$ is calculated as before except for C_b which is calculated as follows (from NDS Section 3.10.4):

Bearing area factor for $l_b < 6$ in.:

$$C_b = \frac{(l_b + 0.375)}{l_b}$$

Bearing area factor for $l_b \geq 6$ in.: $C_b = 1.0$

At level 3, for a bearing length of 4.0 inches, $C_b = 1.09$.

Therefore:

$F'_{c\perp} = 625(1.0)(1.0)(1.09){=}684$ psi

Bearing area, A_{Brg}, is calculated based on the hole in the wood bottom plate being 3/16-inch larger than the rod diameter given in Table 3.11. This is larger than a standard bolt hole, since the rod is meant to be able to move freely, vertically through the hole. Tiedown rods are not intended to transfer horizontal shear forces. (For steel plate bending calculations shown later, the hole in the steel plate is assumed to be 1/16-inch larger than the rod diameter, per Steel Manual Table J3.3.)

$$A_{Brg} = (3.0 \times 4.0) - \left(\pi \left(1\,{}^{5}\!/_{16}\right)^2 \Big/ 4 \right) = 10.65 \text{ in.}^2$$

Therefore, the allowable bearing capacity is:

$$F'_{c\perp}\,(A_{Brg}) = 684(10.65) = 7{,}278 \text{ lb}$$

Which is greater than the differential load of 5,151 lbs that occurs at Level 3, from Table 3.10, so the bearing area is adequately sized.

Bearing plate thickness shall be checked for bending, considering the plate as a cantilevered element.

Uniform load on the steel plate:

$$w = f_{c\perp} = \frac{5{,}151}{10.65} = 484 \text{ psi}$$

Steel plate bending moment (demand):

$$M = \frac{wL^2}{2} = \frac{(484 \times 3.0)\left(\frac{4.0}{2}\right)^2}{2} = 2{,}903 \text{ in.-lb}$$

Allowable bending capacity of the plate:

$$M_n/\Omega_b = {}^{F_y Z}\!/_{1.67} \leq {}^{1.6 F_y S}\!/_{1.67}$$ *Steel Manual Chapter F and Equation F11-1*

Where:

$$Z = \frac{bd^2}{4} = \frac{\left(3.0 - 1\,{}^{3}\!/_{16}\right)\left({}^{5}\!/_{8}\right)^2}{4} = 0.177 \text{ in.}^3$$

b is the width of the plate, minus the hole in the steel plate (1/16-inch larger than the rod)

d is the thickness of the plate

(By inspection, Z will control over 1.6^*S.)

$$M_n/\Omega_b = {}^{F_y Z}\!/_{1.67} = {}^{36(0.177)}\!/_{1.67} = 3{,}816 \text{ in.-lb} > 2{,}903 \text{ in.-lb}$$ *Okay*

Calculations for bearing plates at other levels are summarized in Table 3.12. (Plate bending calculations are not shown.)

TABLE 3.12: Bearing plate design

Level	Bearing Plate			Rod diameter from Table 3.11 (in.)	Hole diameter in bottom plate (in.)	A_{Brg} (in.^{2})	Bearing Factor, C_b	Bearing Demand (Tension from Table 3.10) (k)	Bearing Capacity (k)	DCR
	Width (in.)	Length (in.)	Thickness (in.)							
Roof	3.0	3.0	0.375	5/8	13/16	8.48	1.13	1.351	5.964	23%
6th	3.0	3.0	0.375	5/8	13/16	8.48	1.13	1.482	5.964	25%
5th	3.0	3.5	0.500	3/4	15/16	9.81	1.11	3.266	6.788	48%
4th	3.0	4.0	0.625	1	1 3/16	10.89	1.09	4.492	7.446	60%
3rd	3.0	4.0	0.625	1 1/8	1 5/16	10.65	1.09	5.151	7.278	71%

Take-Up Devices

Most continuous rod systems include proprietary expanding or contracting devices to compensate for shrinkage. Referred to as take-up devices, their purpose is to minimize gaps between the plate washer and nut on the tension rod, which can occur due to building settlement or shrinkage of the wood elements. By rotating the nut down or using a compression spring on the rod, they ensure the bearing plate remains tight to the wood surface so the tension rod engages in the event of uplift on the system.

These devices typically have International Code Council Evaluation Service (ICC-ES) reports indicating their allowable tension capacity, the amount of shrinkage they can accommodate, and deflections associated with the devices themselves. Designers should verify that the models being used meet the expected demands.

The use of take-up devices is highly desirable in multi-level light-frame wood construction. Since the total shrinkage of the building has to be accounted for in the wall anchorage deformation (Δ_a), it is impractical (and challenging) to meet code drift requirements for most shear walls—especially those with shorter lengths—without these devices.

Take-up devices deflect under load just like conventional hold-downs. Most manufacturers publish this information in their ICC-ES reports. The deformation or initial slack of these devices needs to be considered in the overall tiedown displacement.

Take-up devices have moving parts and may jam if not properly installed. Jamming typically occurs as a result of excessive out-of-plumbness of the tiedown rod. Take-up device specifications should include a requirement for the contractor to coordinate installation with the manufacturer instructions, as these devices often have specific installation instructions to ensure they perform as intended.

3.4 Shear Wall Deformations and Story Drift

3.4.1 Drift Load Combination and Resulting Forces

Previous editions of ASCE 7 (e.g., ASCE 7-16 Section 12.8.6) required story drift to be calculated based on strength-level seismic forces, calculated with the same base shear used for member design but without the reductions used in ASD load combinations. However, it was not clear which of the many strength load combinations should be used. ASCE 7-22 section 12.8.6.1 clarifies this by providing a new load combination for computing displacement and drift.

$1.0D + 0.5L + 1.0E_h$ *ASCE 7 Section 12.8.6.1*

Where:

L can be taken as $0.4L_o$ for live loads of 100 psf or less

L_o is the unreduced design live load

New compression, tension, and shear forces are calculated based on the drift load combination as shown in Tables 3.13A, 3.13B, and 3.13C (respectively). These values will be used for deflection, displacement, and drift calculations throughout this section.

Compression Forces

Vertical compression (i.e., crushing) due to dead and live loads is assumed to be the same under the two chord elements and therefore does not directly contribute to the rotation and resulting horizontal drift of the shear wall. Thus, the compression force used to determine vertical chord deformation and resulting horizontal drift is simply E_h. This approach is somewhat unconservative when calculating the vertical crushing deformation under each chord, since it neglects the precompression forces from dead and live loads that the system experiences prior to a seismic event. However, the authors deem this to be an appropriate estimation given more conservative assumptions being taken in other aspects of the deflection calculations. These loads were also considered when the compression posts were designed in Section 3.3.5 and the posts sized to provide necessary bearing area to limit compressive deformations.

TABLE 3.13A: Compression chord forces for drift calculations *(Analogous to Table 3.8)*

Level	Seismic Compression Force from Table 3.6, E_h (k)
Roof	3.39
6th	10.10
5th	19.45
4th	30.63
3rd	42.81

Tension Forces

At the end of the wall opposite the crushing effect, vertical deformation occurs due to net tension forces, which are the resultant of cumulative seismic overturning moments and the resisting cumulative dead and live loads at each level.

As noted previously, Section 12.8.6.1 states that, for the purposes of computing displacement and drift, the expected gravity loads shall be *no less than* $1.0D + 0.5L$. This is intended to capture P-delta effects, whereby increased gravity loads can *increase* the calculated drift. However, in the case of determining forces to resist overturning moments, including this much gravity load is unconservative, since it reduces the tension force on the system and *decreases* the calculated drift. Thus, for this design example, the authors propose using the strength-level (LRFD) load combination that results in the greatest amount of tension force in the chords. While this is a departure from the letter of the ASCE 7 standard, this approach results in a tension force that is not less than the tension force that would be developed under the ASCE 7 drift load combination, ultimately resulting in a more conservative estimate of drift.

$0.9D - E_v + E_h$ *ASCE 7 Section 2.3.6 combination 7*

$= (0.9 - 0.2S_{DS})D + E_h$

$= 0.66D + E_h$

TABLE 3.13B: Tension chord forces for drift calculations *(Analogous to Table 3.10)*

Level	Dead Load Resisting Moment from Table 3.10 $M_{R,D}$ (ft-k)	Seismic Overturning Moment from Table 3.6, $M_{OT,Eh}$ (ft-k)	Net Resultant Moment $0.66D + 1.0E_h$ (ft-k)	d (ft)	Tension load for Drift Calculations (k)	Differential Tension Load Per Floor (k)
Roof	65.6	-94.0	-50.8	27.75	-1.829	-1.829
6th	271.6	-279.2	-100.3	27.65	-3.627	-1.797
5th	477.7	-533.0	-218.3	27.40	-7.966	-4.339
4th	683.7	-832.3	-381.9	27.18	-14.052	-6.086
3rd	889.8	-1,154.5	-568.4	26.97	-21.075	-7.023

Based on the sign convention used, tension is indicated as a negative value.

Horizontal Shear Forces

Horizontal drift will also occur due to the lateral shear force applied to the wall. While previously calculated under ASD load combinations ($0.7E_h$), Table 3.13C shows the shear force under the new drift load combination ($1.0E_h$).

TABLE 3.13C: Shear wall forces for drift calculations *(Analogous to Table 3.5)*

Level	F_{total} from Table 3.4 (lb)	Wall Length, l (ft)	Drift Design Shear, V (plf)
Roof	9,397	29	324
6th	18,526	29	639
5th	25,373	29	875
4th	29,938	29	1,032
3rd	32,220	29	1,111

3.4.2 Shear Wall Deflection Equation

Considerable monotonic and cyclic testing has been done on wood structural panel shear walls in the last several decades. Test results and testing protocols used in the development of SDPWS provisions can be found in the references listed in the Commentary Section C4.3.1.

Historically, shear wall deflections were calculated based on a four-term equation that can still be found in the SDPWS Commentary (Equation C4.3.4-1). A similar equation can be found in IBC Section 2305.3 (Equation 23-2), although its use is only permitted for stapled shear walls. To determine the deflection of a nailed shear wall, SDPWS provides a three-term equation, which simplifies the previous four-term equation and is shown below.

$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b} \qquad \textit{SDPWS Equation 4.3-1}$$

Where:

v = Unit shear force induced by the design loads in the wall, plf

h = Shear wall height, measured from the bottom of the bottom plate to the top of the top plates, ft

E = Modulus of elasticity of the boundary posts, psi

From Part 1 of this design example, $E = 1{,}700{,}000$ psi for the species and grade being used.

A = Area of the chord posts/studs, in.^{2}

For this example, chord post sizes are given in Table 3.9.

b = Shear wall length, ft

G_a = Apparent shear wall stiffness from nail slip and panel deformation (k/in.), from SDPWS Tables 4.3A, 4.3B, 4.3C, or 4.3D. Per the footnotes to Tables 4.3A and 4.3B, when 4-ply or 5-ply plywood panels or composite panels are used, G_a values may be increased by 1.2.

This example uses G_a values for OSB sheathing from Table 4.3A. Note that, for the same nailing pattern and sheathing thickness, G_a values for OSB are higher than for plywood. In this example, OSB was chosen in lieu of plywood to help control drift.

Δ_a = Total vertical chord deformation due to vertical deformation of the overturning anchorage system (including fastener slip, device elongation, rod elongation, uncompensated shrinkage, etc.) plus the vertical compression deformation, as measured at the ends of the shear wall when the wall is subjected to the unit shear force induced by the design loads in the wall, in.

Four-Term vs. Three-Term Equation

The three-term equation is a simplified version of a more complex four-term equation. The four-term equation adds the effects of the four sources contributing to the deflection: cantilever bending of the chords, shear deformation of the wood structural panels, bending and slip of the fasteners, and deflection due to anchorage (tiedown) deformation. The original, more complex four-term shear wall deflection formula is:

$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{G_vG_t} + 0.75he_n + \frac{h}{b}\Delta_a$$

SDPWS Equation C4.3.4-1

Where:

G_vG_t = Shear stiffness (lb/in.) of panel depth (SDPWS Tables C4.2.2A and C4.2.2B)

e_n = Nail deformation (fastener slip)

Using the fastener slip equations from SDPWS Table C4.2.2D for 10d common nails, there are two basic equations.

When the nails are driven into *green* lumber $e_n = (V_n/977)1.894$

When the nails are driven into *dry* lumber $e_n = (V_n/769)3.276$

Where:

V_n is the fastener load in pounds per fastener

The simplified three-term equation (SDPWS Equation 4.3-1) combines the second and third terms of the four-term equation into one. The tabulated apparent shear wall shear stiffness value, G_a, is calculated so the three-term equation produces identical results as the four-term equation at the wall's LRFD seismic shear capacity.

Although Equation 4.3-1 is easier to use, the calculated deflection will be larger than deflections calculated using the four-term equation when forces in the wall are less than the wall's LRFD seismic capacity. For more accurate deflection estimates at lesser load levels, the four-term equation can be used where e_n values are calculated based on the applied unit shear demands of interest. Both equations need to be adjusted to site conditions (moisture content of the lumber, OSB vs. plywood panels, number of plies, etc.).

3.4.3 Vertical Chord Deformation, Δ_a

While the first two terms of the shear wall deflection equation are straightforward, the third warrants further discussion. Vertical chord deformation, Δ_a, is a collective accumulation of the deformation of the chord as each element crushes, elongates, shrinks, or otherwise deforms.

The SDPWS definition of Δ_a was provided in Section 3.4.2 and includes two significant changes compared to previous codes. First, it now explicitly includes the vertical compression deformation (i.e., crushing) effects at the compression end of the shear wall. Second, it clarifies that total chord deformation must be taken at the end of the wall, which is not typically where the calculated tension and compression forces occur. That is, the moment arm, d, used to calculate the T-C couple is different than the overall length of the shear wall, b, and deformation calculations must be adjusted accordingly.

Therefore, Δ_a can be thought of as the wall-length-adjusted sum of the tiedown system deflection, Δ_T, and the deformation from crushing, Δ_C, where Δ_T is vertical elongation of the anchorage system measured at the center of the tiedown rod and Δ_C is the vertical compression deformation of the chords measured at the centroid of the compression posts. The sum of Δ_T and Δ_C is multiplied by the ratio of d/b which scales this value to the sum of deformations that would be seen at the ends of the wall (Figure 3.9). The vertical movement results in a rotation of the shear wall as a rigid body, with the horizontal drift at the top of the wall (δ) equal to the aspect ratio times the vertical chord deformation (Δ_a), as shown in Figure 3.9.

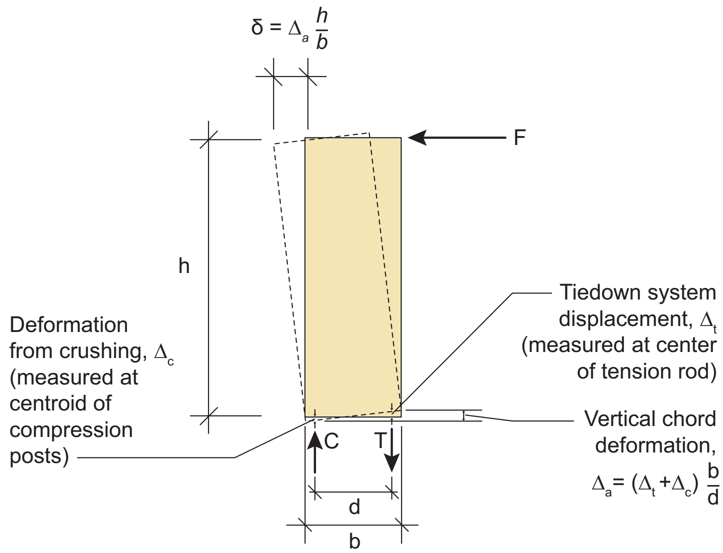


FIGURE 3.9: Effect of vertical chord deformation on drift

Plate Crushing

Plate crushing, Δ_C, is a downward effect at the compression chord that occurs at the opposite end of the wall from the tension forces. Walls that have no (net) uplift forces will still have a crushing effect at wall boundaries, which contributes to rotation of the wall. Per NDS Section 4.2.6, a bearing stress, $f_{c\perp}$, equal to $F_{c\perp}$ results in approximately 0.04 inch of crushing deformation. When $f_{c\perp} = 0.73\ F_{c\perp}$, crushing will be approximately 0.02 inch. The crushing effect on wood is not linear; a graph of load vs. deformation is shown in Figure 3.10. These are limit state values and are not adjustable for duration of load (C_D).

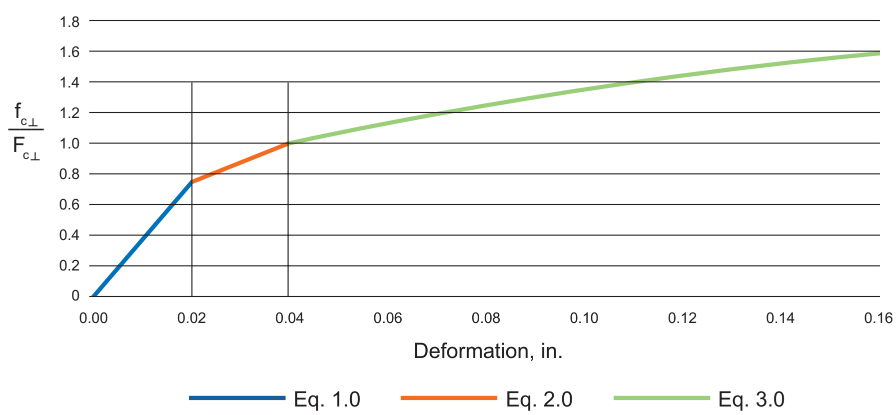


FIGURE 3.10: Bearing load deformation curve

For the three different segments of the load vs. deformation curve shown in Figure 3.10, equations for determining compression perpendicular-to-grain deformation (Δ) are calculated as follows:

For: $f_{c\perp} \leq F_{c\perp,0.02}$

$$\Delta = 0.02\left(\frac{f_{c\perp}}{F_{c\perp,0.02}}\right) \qquad \textit{Equation 1}$$

For: $F_{c\perp,0.02} < f_{c\perp} \leq F_{c\perp}$

$$\Delta = 0.04 - \frac{0.02\left(1-\frac{f_{c\perp}}{F_{c\perp}}\right)}{0.27} \qquad \textit{Equation 2}$$

For: $f_{c\perp} > F_{c\perp}$

$$\Delta = 0.04\left(\frac{f_{c\perp}}{F_{c\perp}}\right)^3 \qquad \textit{Equation 3*}$$

Where:

Δ = Deformation, in.

$f_{c\perp}$ = Compression stress (demand), psi

$F_{c\perp}$ = Reference design value at 0.04 in. deformation, psi

$F_{c\perp,0.02}$ = Reference design value at 0.02 in. deformation, psi = $0.73F_{c\perp}$

**Equation 3 is taken from Bendtsen et all, 1979*.

Note that the value of 0.04 inch is based on a metal plate bearing on wood perpendicular to grain under standard test conditions of ASTM D143 Standard Test Methods for Small Clear Specimens of Timber (ASTM, 2022). NDS Commentary section C4.2.6 states that when a joint is made of two wood members and both are loaded perpendicular to grain, the amount of deformation will be approximately 2.5 times that of a metal plate bearing joint (i.e., a 150% increase). It follows that when only one of the wood members is loaded perpendicular to grain, the amount of deformation will be approximately 1.75 times that of a metal plate bearing condition (i.e., a 75% increase).

While it has been recommended to use these factors for wood-to-wood bearing conditions when calculating shear wall deflections, more recent shear wall tests showed that this amplification of bearing deformation was not needed to calculate reasonable estimates of shear wall deflection. For this reason, the factor is excluded from Example C4.3.4-2 in the SDPWS Commentary. Whether to apply this amplification factor is at the judgement of the Engineer of Record, based on framing conditions (e.g., modified balloon framing vs. platform framing) and how closely design conditions match those of the shear walls tested. Information on the tests and their conditions can be found in ASTM D7989 Standard Practice for Demonstrating Equivalent In-Plane Lateral Seismic Performance to Wood-Frame Shear Walls Sheathed with Wood Structural Panels (ASTM, 2020). To be conservative, this design example will continue to use the deformation adjustment factor.

In the case of the shear wall in this example (Figure 3.2, detail A), the components of chord post crushing at each level are as follows:

- At the bottom of the chord posts, there is bearing on the bottom plates; this is a parallel-to-perpendicular wood bearing condition with a factor of 1.75.
- At the top of the chord posts, there is bearing on the underside of the top plates; this is another parallel-to-perpendicular wood bearing condition with a factor of 1.75.
- There is also the crushing effect of the floor sheathing. APA's Panel Design Specification (APA, 2020) provides similar deformation limits at given bearing stress values:
  - $f_{c\perp}$ of 210 psi results in approximately 0.02 inch of crushing deformation
  - $f_{c\perp}$ of 360 psi results in approximately 0.04 inch of crushing deformation

  It is assumed that deformations calculated based on these bearing stresses do not require an additional factor (1.75, 2.5, or otherwise). However, these deformations will be calculated separately from the sawn lumber deformations due to different limiting stress values. Also as a result of the different limiting stress values, Equation 2 above for solid sawn lumber is derived for sheathing, with the revised equation shown below.

$$\Delta = 0.04 - \frac{0.02\left(1-\frac{f_{c\perp}}{F_{c\perp}}\right)}{0.417} \qquad \textit{Equation 2 for sheathing}$$

Results for these three components and their combined impact at each floor level are shown in Table 3.14. Throughout this design example, tables note the level at which forces are imparted to the wall—i.e., the top of the wall being analyzed. For example, "roof level" is used to designate the wall between Level 6 and the roof. While this nomenclature is used in Table 3.14 for consistency, it is important to note that crushing occurs at the bottom of the wall. That is, the crushing noted at the "roof level wall" actually occurs at Level 6, and so on. Crushing of the bottom plate is based on the size of the chord posts within the wall being analyzed, while crushing of the top plate is based on the size of the chord posts in the wall below. Additionally, sheathing crushing is based on an increased bearing area, assuming that loads are distributed at a 45-degree angle through the thickness of the plates. Note that for the wall at Level 3 there is no crushing of the sheathing or top plates since that wall is supported by the concrete podium at the second floor, which does not contain sheathing or top plates.

TABLE 3.14: Crushing calculations

Crushing: Posts to Bottom Plate (perp-to-parallel condition = 1.75 factor)							
Level	**Chord Posts**	**Compression Demand from Table 3.13A (k)**	**Area (in.²)**	**$f_{c\perp}$ (psi)**	**Equation Number***	**Unfactored Crushing (in.)**	**Factored Crushing (in.)**
Roof	(6) 2x4	3.39	31.5	108	Eq. 1	0.005	0.008
6th	(7) 2x4	10.10	36.8	275	Eq. 1	0.012	0.021
5th	(10) 2x4	19.45	52.5	371	Eq. 1	0.016	0.028
4th	(13) 2x4	30.63	68.3	449	Eq. 1	0.020	0.034
3rd	(16) 2x4	42.81	84.0	510	Eq. 2	0.026	0.046

Crushing: Sheathing (1.0 factor)							
Level	**Chord Posts**	**Compression Demand from Table 3.13A (k)**	**Area (in.²)**	**$f_{c\perp}$ (psi)**	**Equation Number***	**Unfactored Crushing (in.)**	**Factored Crushing (in.)**
Roof	(6) 2x4	3.39	52.5	65	Eq. 1	0.006	0.006
6th	(7) 2x4	10.10	57.8	175	Eq. 1	0.017	0.017
5th	(10) 2x4	19.45	73.5	265	Eq. 2	0.027	0.027
4th	(13) 2x4	30.63	89.3	343	Eq. 2	0.038	0.038
3rd	(16) 2x4	42.81	N/A				

Crushing: Top Plate to Posts (perp-to-parallel condition = 1.75 factor)							
Level	**Chord Posts in Wall Below**	**Compression Demand from Table 3.13A (k)**	**Area (in.²)**	**$f_{c\perp}$ (psi)**	**Equation Number***	**Unfactored Crushing (in.)**	**Factored Crushing (in.)**
Roof	(7) 2x4	3.39	36.8	92	Eq. 1	0.004	0.007
6th	(10) 2x4	10.10	52.5	192	Eq. 1	0.008	0.015
5th	(13) 2x4	19.45	68.3	285	Eq. 1	0.012	0.022
4th	(16) 2x4	30.63	84.0	365	Eq. 1	0.016	0.028
3rd	N/A	42.81	N/A				

Total Crushing			
Level	**Chord Posts**	**Compression Demand from Table 3.13A (k)**	**Total Crushing (in.)**
Roof	(6) 2x4	3.39	0.021
6th	(7) 2x4	10.10	0.052
5th	(10) 2x4	19.45	0.078
4th	(13) 2x4	30.63	0.100
3rd	(16) 2x4	42.81	0.046

**Equations shown in Figure 3.10*

Rod Elongation

As with crushing deformation, the tiedown system deflection, Δ_T, has several components. The first is rod elongation, which is determined from the following expression:

$$\frac{PL}{AE}$$

Where:

P = Accumulated uplift tension force on the rod in kips (tension demand)

L = Rod length in inches from bearing restraint to bearing restraint, with the bearing restraint being where the load is transferred to the rod

For this design example, rod length is assumed to be equal to the floor-to-floor height of 10 feet. Note that the rod length could vary between floors, based on how far the anchor bolt or rod below projects above the bottom plate.

$E = 29{,}000$ ksi

A = Effective area of the rod

For threaded rods, the area is equal to the net tensile area (A_e) of the threaded rod, given in the Steel Manual Table 7-17.

Note that steel strength is not part of the expression above. While using higher strength rods will help achieve higher capacities, it will not help limit drift, since the modulus of elasticity, E, does not change with steel strength.

Some jurisdictions have limits on the amount of rod elongation that can occur between restraints; check local building department requirements.

As noted in the definition of P, rod elongation is based on the cumulative tension load on the rod at a given level.

TABLE 3.15: Rod elongation (drift demands)

Level	Tension Demand from Table 3.13B (k)	Rod Diameter, d from Table 3.11 (in.)	A_e (in.^{2})	Plate Height (ft)	Rod Elongation (in.)
Roof	1.829	5/8	0.226	10.0	0.033
6th	3.627	5/8	0.226	10.0	0.066
5th	7.966	3/4	0.334	10.0	0.099
4th	14.052	1	0.606	10.0	0.096
3rd	21.075	1 1/8	0.763	10.0	0.114

Crushing Effects of "Uplift" Boundary Members

Chord elements transfer differential uplift forces at each story to the metal bearing plate at the floor above (Figures 3.3 and 3.4 and Section 3.3.6).

Deformations due to wood crushing under the bearing plates are calculated in the same manner as crushing of the top and bottom plates in the previous section. However, because this is a metal plate bearing condition, no additional factors (1.75 or 2.5) are needed in this calculation.

TABLE 3.16: Bearing plate crushing (drift demands)

Level	Differential Tension Demand from Table 3.13B (k)	Area from Table 3.12 (in.^{2})	$f_{c\perp}$ (psi)	Equation Number*	Crushing (in.)
Roof	1.829	8.48	216	Eq. 1	0.009
6th	1.797	8.48	212	Eq. 1	0.009
5th	4.339	9.81	442	Eq. 1	0.019
4th	6.086	10.89	559	Eq. 2	0.032
3rd	7.023	10.65	660	Eq. 3	0.047

**Equations shown in Figure 3.10*

Other components of tiedown system deflection, Δ_T, include slack and elongation of the take-up device. Table 3.17A shows the slack is assumed to be 1/32 inch based on the distance the device would need to travel before catching in the next groove. It also shows a take-up device elongation of 0.03 inch; this value is provided by the take-up device manufacturer.

TABLE 3.17A: Total vertical chord deformation (with shrinkage compensating devices)

Level	Rod Elongation from Table 3.15 (in.)	Slack (in.)	Crushing from Table 3.14 (in.)	Bearing Plate Crushing from Table 3.16 (in.)	Take-up Device Elongation (in.)	Total Vertical Chord Deformation, Δ_a (in.)
Roof	0.033	0.031	0.021	0.009	0.030	0.131
6th	0.066	0.031	0.052	0.009	0.030	0.199
5th	0.099	0.031	0.078	0.019	0.030	0.272
4th	0.096	0.031	0.100	0.032	0.030	0.309
3rd	0.114	0.031	0.046	0.047	0.030	0.289

Impact of Not Including Take-Up Devices

If take-up devices are not installed, tiedown system deflection, Δ_T, needs to include shrinkage per the discussion in Section 2.3 and deformations due to gravity loads per the discussion in Section 3.4. In this case, slack and elongation of the take-up device would not apply. Additionally, if uplift bearing devices are not provided at each level, the vertical chord deformation would be cumulative moving up the building. These impacts are shown in Table 3.17B and clearly demonstrate the benefits of using take-up devices and bearing restraints at each floor in limiting vertical deformation and overall shear wall drift.

TABLE 3.17B: Total vertical chord deformation (without shrinkage compensating devices)

Level	Rod Elongation from Table 3.15 (in.)	Shrinkage from Table 2.1 (in.)	Crushing from Table 3.14 (in.)	Bearing Plate Crushing from Table 3.16 (in.)	Vertical Chord Deformation at Each Level (in.)	Cumulative Chord Deformation, Δ_a (in.)
Roof	0.033	0.190	0.021	0.009	0.266	1.88
6th	0.066	0.190	0.052	0.009	0.334	1.62
5th	0.099	0.190	0.078	0.019	0.408	1.28
4th	0.096	0.190	0.100	0.032	0.466	0.87
3rd	0.114	0.190	0.046	0.047	0.427	0.43

3.4.4 Shear Wall Deflections

Based on the three-term equation provided in Section 3.4.2 and vertical chord deformation values calculated in Table 3.17A, overall shear wall deflections are provided in Table 3.18.

TABLE 3.18: Shear wall deflection

Level	Drift Shear Force, v from Table 3.13C (plf)	h (ft)	A from Table 3.9 (in.²)	b (ft)	Fastener Spacing from Table 3.5	G_a (k/in.)	Vertical chord deformation Δ_a from Table 3.17A (in.)	Three-Term Shear Wall Deflection Equation			Shear wall deflection, δ_{SW} (in.)
								1st term (in.)	2nd term (in.)	3rd term (in.)	
Roof	324	10.0	31.5	29	6" o.c.	22	0.131	0.002	0.147	0.045	0.194
6th	639	10.0	36.8	29	4" o.c.	30	0.199	0.003	0.213	0.069	0.284
5th	875	10.0	52.5	29	2" o.c.	52	0.272	0.003	0.168	0.094	0.265
4th	1,032	10.0	68.3	29	2" o.c.	52	0.309	0.002	0.199	0.107	0.307
3rd	1,111	10.0	84.0	29	2x4" o.c.	60	0.289	0.002	0.185	0.100	0.287

Multi-Story Shear Wall Effects

The calculations above follow a standard practice in the U.S. of determining shear wall drift, whereby the wall at each story is treated as a separate wall based on the floor-to-floor height. However, some practitioners argue that more consideration should be given to the interaction of shear walls over multiple floors. Specifically, the *first term* of the shear wall deflection equation, which represents bending, only considers the shear force applied at the top of each wall; it does not include a bending moment at the top of the wall that could exist in response to the wall above also experiencing bending. Additionally, the *third term* of the shear wall deflection equation, which represents a rigid body rotation of the wall due to deformations at each end of the wall, could have compounding effects over a multi-story wall. As the wall segment at a lower level rotates, it could cause the wall segments above to rotate, regardless of any lateral loads applied at the upper levels. The *second term* of the shear wall deflection equation, which represents pure shear deformation without any rotational or bending component, would not contribute to multi-story effects.

How effectively the bending and rotational impacts are translated between floors is largely based on the out-of-plane stiffness of the floor assembly. For very rigid floor systems, it may be appropriate to assume these forces are transferred out through the floor; for very flexible floor systems, such as the modified balloon framing used in this design example, this assumption may not be appropriate. However, in keeping with standard U.S. practices and the precedent to analyze shear walls on a floor-to-floor basis, multi-story shear wall effects are not included in this design example.

Multi-story shear wall effects will be more pronounced in high aspect ratio shear walls (i.e., narrow and tall walls) because more of the shear wall deflection is contributed by the first and third terms. Low aspect ratio walls, such as the 29-foot-long by 10-foot-tall wall used in this example, will generally be by the second term, so multi-story effects are less pronounced.

3.4.5 Story Drift

ASCE 7 Section 12.8.6.5 defines the design story drift, Δ, as the difference between design earthquake displacements, δ_{DE}, excluding diaphragm deformation, at the center of mass of adjacent levels. Design earthquake displacement is defined in Section 12.8.6.3, Equation 12.8-16 as:

$$\delta_{DE} = \frac{C_d \delta_e}{I_e} + \delta_{di}$$

Where:

C_d = Deflection amplification factor from Table 12.2-1

For light-frame (wood) walls sheathed with wood structural panels rated for shear resistance, $C_d = 4$

I_e = Seismic importance factor

From Section 3.1, $I_e = 1.0$

δ_e = Elastic displacement computed under design earthquake forces, including the effects of accidental torsion and torsional amplification, as applicable

Note that in this example, torsion has not been considered.

δ_{di} = Displacement due to diaphragm deformation corresponding to the design earthquake

Recall that for the calculation of design story drift, δ_{di} may be neglected.

Previous editions of ASCE 7 did not explicitly include the diaphragm deflection term in the discussion of story drift, nor was there specific guidance on design earthquake displacement. However, the changes in ASCE 7-22 have no impact on the story drift calculations shown in this example. For information regarding *total deformation* of the building, the reader is encouraged to read Section 12.8.6 of the applicable version of ASCE 7.

Note that within ASCE 7, the δ symbol is used to refer to total deformation at a given point on the building (i.e., cumulative drift at any story) while Δ is refers to *story drift* (sometimes referred to as inter-story drift, or the difference in displacements between adjacent levels). At times, the nomenclature used in ASCE 7 conflicts with the nomenclature used in SDPWS, and the remaining sections of this example clearly state which is being referenced.

The shear wall deflection values, δ_{SW}, calculated per SDPWS and given in Table 3.18, can be used as a representative elastic story drift, which we can term Δ_e based on ASCE 7 nomenclature, at each level. Total elastic displacement at each level, δ_e, as defined by ASCE 7, would be calculated as the accumulation of story drifts as you move up the building. However, since story drift, Δ, is calculated as the *difference* between total elastic displacements at each level, we can simply scale the elastic story drifts we already have to design story drifts based on the following equation, which is analogous to ASCE 7 equation 12.8-16:

$$\Delta = \frac{C_d \Delta_e}{I_e}$$

The calculated story drift, Δ, at any level shall not exceed the allowable story drift, Δ_a, per ASCE 7 Section 12.12.1 and Table 12.12-1. For this design example, $\Delta_a = 0.020h_{sx}$; in other words, drift is limited to 2% of the story height, h_{sx}. These calculations are provided in Table 3.19.

TABLE 3.19: Design story drift vs. allowable story drift

Level	Elastic Story Drift from Table 3.18 (in.)	Design Story Drift (in.)	Story Height (ft)	Allowable Story Drift (in.)
Roof	0.194	0.78	10	2.40
6th	0.284	1.14	10	2.40
5th	0.265	1.06	10	2.40
4th	0.307	1.23	10	2.40
3rd	0.287	1.15	10	2.40

The 29-foot-long shear wall used in this design example meets the drift requirements.

3.5 Discontinuous Systems and the Overstrength (Ω_0) Factor

ASCE 7 Section 12.3.3.4 requires that structural elements supporting discontinuous walls be designed to resist seismic load effects including overstrength found in Section 12.4.3. Therefore, the podium that supports discontinuous wood shear walls must be designed for seismic forces (i.e., wood shear wall overturning forces in the chords—axial tension and compression—and associated horizontal shear forces) *including the corresponding overstrength factor*, Ω_0 of 3.0. Footnote b of ASCE 7 Table 12.2-1 states that, for structures with flexible diaphragms, this value may be 2.5. In some cases, the orthogonal directional combination procedure may be required per ASCE 7 Section 12.5.4. The overstrength factor resulting from discontinuous systems is applicable to both the independent directional and orthogonal directional combination procedures.

The overstrength factor does not need to be applied to the shear wall's connections. ASCE 7 Section 12.3.3.4 states that connections between the discontinuous wall and supporting element need only be adequate to resist the forces for which the discontinuous wall was designed. The commentary on Section 12.3.3.4 provides further explanation:

> *"For wood light-frame shear wall construction, the final sentence of Section 12.3.3.4 results in the shear and overturning connections at the base of a discontinued shear wall (i.e., shear fasteners and tiedowns being designed using the load combinations of Section 2.3 or 2.4 including seismic effect without over strength rather than the load combinations of Section 2.3 or 2.4 with overstrength of Section 12.4.3."*

However, an overstrength factor may still be required for anchorage to concrete, per ACI 318 Chapter 17.

As discussed in ASCE 7 Section 12.4.3.1, one way to reduce the calculated overstrength load is to demonstrate that yielding of associated elements (anchor, shear wall, diaphragm, collector, etc.) will occur below the overstrength-level forces. When this is the case, the seismic load effects including overstrength can be limited to this value. The commentary on Section 12.4.3 provides further explanation:

> *"The standard permits the horizontal seismic load effect including overstrength to be calculated directly using actual member sizes and expected material properties where it can be determined that yielding of other elements in the structure limits the force that can be delivered to the element in question. When calculated this way, the horizontal seismic load effect including overstrength is termed the capacity-limited seismic load effect, E_{cl}."*

Further discussion of concrete design, including anchorage, is outside the scope of this design example.

Section 3 demonstrated key aspects of the seismic analysis and design of a five-story light-frame wood structure over a one-story concrete podium located in a high-seismic region, using the two-stage lateral analysis procedure of ASCE 7. Wind loads are addressed in Section 4 along with seismic load effects that must still be considered even in areas of low to moderate seismicity.

4 Wind and Low-Seismic Design

4.1 Commentary on Wind-Controlled Regions

In regions with low to moderate seismicity (SDC A, B, and C), wind loads are typically the controlling force when it comes to designing the vertical LFRS, especially for lightweight structures. Section 3 highlighted the requirements and considerations when dealing with projects in areas of elevated seismic risk. Section 4 provides a design example for the same structure, but located in the Northeast where wind loads typically govern the design. Regardless of what's typical, it is important for designers to assess seismic demands on all projects. Nominal shear wall capacities are adjusted differently for wind and seismic loads (see SDPWS Section 4.1.4), and ASD load factors are 0.7 for seismic and 0.6 for wind. Because of these differences, seismic may control certain aspects of the design even when ultimate-level seismic forces are less than those from wind. The additional load factors applied to the seismic design of diaphragms and discontinuous systems may also result in seismic controlling the design of these elements, even when nominal wind forces are higher.

For low-seismicity projects, it is not uncommon for designers to leverage multiple SFRS—such as WSP and GWB shear walls—to minimize project costs. ASCE 7 Section 12.2.2 permits the use of different framing systems in different directions, which is common on projects where combined orthogonal effects are not required as part of the analysis, per ASCE 7 Section 12.5.1.2.

For example, light-frame wood multi-family projects are often rectangular in shape. The transverse direction tends to be more heavily loaded due to shorter wall segments supporting larger tributary areas, and the longitudinal direction typically has longer continuous lengths of shear wall. In these situations, it may be advantageous to use WSP shear walls ($R = 6.5$) in the transverse direction at demising walls, and GWB shear walls ($R = 2$) in the longitudinal direction.

The following example uses GWB shear walls except at lower levels where WSP shear walls are required due to the magnitude of lateral forces. The building is located in Providence, RI and is adjacent to open terrain consisting of grass fields and trees less than 30 feet tall, which qualifies as surface roughness category C per ASCE 7 Section 26.7. The project is located in SDC B. Design values for this example are taken from the ASCE Hazard Tool. However, designers should check with state and local codes to confirm the appropriate values for their project.

4.2 Using Gypsum Wall Board (GWB) for Lateral Resistance

IBC Section 2306.3 refers to SDPWS for the design and construction of light-frame wood shear walls. Alternatively, if shear walls are constructed using staples, the allowable shear values in IBC Table 2306.3(3) may be used. SDPWS includes nominal shear values in Table 4.3C, which must be converted to ASD or LRFD by applying the appropriate adjustment factors per SDPWS Section 4.1.4 for wind and seismic.

- Seismic design (SDPWS Section 4.1.4.1):
  - Divide tabulated nominal shear capacity by 2.8 for ASD
  - Multiply tabulated nominal shear capacity by 0.5 for LRFD
- Wind design (SDPWS Section 4.1.4.2):
  - Divide tabulated nominal shear capacity by 2.0 for ASD
  - Multiply tabulated nominal shear capacity by 0.8 for LRFD

While GWB may be used to resist lateral wind forces, allowable shear values for wind loads range from 60 to 250 pounds per lineal foot, which is significantly lower than those for WSP-sheathed walls.

Using GWB to resist seismic forces has limitations. The first is that shear walls are subject to the limitations of ASCE 7 Section 12.2.1, which requires that the SFRS be assigned the R factor and follow the height limitations and permitted uses in Table 12.2-1. Shear walls sheathed with “other materials,” such as GWB, plaster, and plaster over gypsum lath, are assigned an R of 2, which is significantly less than the R of 6.5 given for walls with WSP sheathing. This is because these “other materials” are less ductile than WSP shear walls. GWB has shown brittle failure in testing that does not allow elastic recovery of the shear wall. This can be a complete separation of the board from the framing studs caused by the sudden crushing of the material at the fasteners, fracture of the fasteners (most common with screws), or a combination of the two, making the wall unable to resist lateral loads. Compared to the R factor of 6.5 for walls with WSP sheathing, walls with GWB sheathing must be designed for a seismic force that is 225 percent higher.

The second limitation is that buildings in SDC D using shear walls with other materials are limited in height to 35 feet. The third is that using shear walls with other materials is not permitted in SDC E or F. The table below summarizes the differences between the systems.

TABLE 4.1: R factors for light-frame SFRS

Seismic Force-Resisting System	Response Modification Coefficient, *R*	System Limitations		
		Seismic Design Category		
		A, B, or C	D (ft)	E or F (ft)
A.16 – Light-frame (wood) walls sheathed with wood structural panels rated for shear resistance	6.5	No limit	65	65
A.18 – Light-frame walls with shear panels of all other materials	2	No limit	35	Not permitted

Partial ASCE 7 Table 12.2-1

4.2.1 Using Cooler Nails vs. Screws for GWB Fastening

Traditionally, GWB was fastened with cooler nails that were hand nailed using a hammer. The typical approach today is to use drywall screws installed with drywall screw guns. Panels are tacked in place with cooler nails, and drywall screws are added once the panels are all hung. Drywall screw guns operate at high speed and have sensors that make them stop driving at a specific depth. This allows the contractor to “set and forget,” making screw installation consistent and quick. Some screw guns also have auto feed devices.

While screw guns tend to be faster than nails overall, there is a caveat to their use. From a lateral force-resisting perspective, the allowable (and nominal) values are significantly lower for screw-fastened walls than nail-fastened walls. The reason is that, when a wall deflects under lateral loading, fasteners bend with the wall movement and screw threads tend to “cut” holes in the drywall material much larger than those caused by nails. It could be extremely problematic if the design engineer uses values for nail-fastened walls and the contractor uses screws. The design engineer may not even be aware of the fasteners used, since walls are often taped and mudded shortly after fastener installation. For this reason, it is recommended that the design engineer use values for screw-fastened GWB. It should also be noted that typical GWB is installed with “floating” corners and edges (i.e., builders don't attach panels to the plates, but rather float the corners together with tape and joint compound). Because these walls are not detailed and constructed as shear walls, they will not have GWP shear wall capacity.

4.3 Wind Loading Analysis – Main Wind Force-Resisting System

This design example uses the directional procedure to determine wind loads for the main wind force-resisting system (MWFRS) per ASCE 7 Section 27.2. This method can be used for structures that are not considered low-rise when the building is regular in shape and has no special wind effects. The steps for determining wind loads using this method are provided in ASCE 7 Table 27.2-1. For both the transverse and longitudinal directions, the following design parameters apply for this design example. In some cases, these parameters can vary between the two directions.

Risk Category: II	*ASCE 7 Table 1.5-1*
Wind velocity, $V = 120\text{ mph}$	*ASCE 7 Figure 26.5-1B*
Wind directionality factor, $K_d = 0.85$	*ASCE 7 Table 26.6-1*
Wind exposure = C	*ASCE 7 Section 26.7*
Topographic factor, $K_{zt} = 1.0$	*ASCE 7 Section 26.8.2*
Ground elevation factor, $K_e = 1.0$	*ASCE 7 Section 26.9*
Gust effect factor (rigid structures), $G = 0.85$	*ASCE 7 Section 26.11.1*
Enclosure classification: Enclosed	*ASCE 7 Section 26.2*

Design Parameter Notes

Elevation factor – It is conservative to assume a ground elevation factor of 1.0 for all projects. Rather, designers should use the site elevation to determine the appropriate ground elevation factor per ASCE 7 Table 26.9-1. For projects at elevations greater than 1,000 feet above sea level, significant reductions can be applied to the base velocity pressure.

Gust effect factor – It is common to assume a gust effect factor of 0.85 for low to mid-rise wood structures. ASCE 7 explicitly states that low-rise structures may be considered "rigid" if they meet the following criteria for low-rise buildings per Section 26.2:

1. Mean roof height, *h*, less than or equal to 60 feet, and
2. Mean roof height, *h*, does not exceed the least horizontal dimension

However, for most Type III light-frame wood projects, the structure is well above 60 feet in height—so confirming a rigid building assumption requires the designer to calculate frequency. While ASCE 7 provides approximate natural frequency equations for structural steel, concrete, and masonry buildings, it does not offer direct guidance for light-frame wood structures.

Once frequency is calculated, it can be used to determine the gust effect factor. Note that there is no two-stage analysis for wind design—the loads applied and frequency of the structure are based on overall structure height. It is also reasonable to look at the language in ASCE 7 Commentary on the definition of "Building or Other Structure, Flexible" which says in part:

"When buildings or other structures have a height exceeding 4 times the least horizontal dimension or when there is a reason to believe the natural frequency is less than 1 Hz (natural period greater than 1s), the natural frequency of the structure should be investigated."

Most Type III light-frame wood buildings do not exceed four times the least horizontal dimension in height and their approximate period can be determined through the FEMA 450 equation described in previous sections. This approach is one way to rationalize the rigid building assumption to utilize a gust effect factor of 0.85.

4.3.1 Wind Pressures for Transverse Direction

Wind pressures in the transverse direction are calculated for the parapets, leeward wall, windward wall, and flat roof as outlined below; results are tabulated in Table 4.2A.

TABLE 4.2A: Wind pressures on the building for the transverse direction

Level	Windward Wall Pressure					Leeward Wall Pressure					Total
	z (ft)	K_z	q_z (psf)	p Case (a) (psf)	p Case (b) (psf)	h_x (ft)	K_h	q_h (psf)	p Case (a) (psf)	p Case (b) (psf)	p (psf)
Parapet	65	1.15	42.3	54.0		65	1.15	42.3	-36.0		89.9
Roof	62	1.14	41.9	30.6	17.8	62	1.14	41.9	-8.7	-21.6	39.4
6th	52	1.10	40.4	29.8	17.0	62	1.14	41.9	-8.7	-21.6	38.5
5th	42	1.05	38.7	28.8	16.0	62	1.14	41.9	-8.7	-21.6	37.5
4th	32	0.99	36.6	27.6	14.8	62	1.14	41.9	-8.7	-21.6	36.3
3rd	22	0.92	33.9	26.0	13.2	62	1.14	41.9	-8.7	-21.6	34.8
2nd	12	0.85	31.4	24.5	11.7	62	1.14	41.9	-8.7	-21.6	33.3
Base	0	0.85	31.4	24.5	11.7	62	1.14	41.9	-8.7	-21.6	33.3

	Flat Roof Wind Pressure (psf)			
Horizontal Distance from Windward Edge	0 to $h/2$	$h/2$ to h	h to $2h$	> $2h$
Case (a)	-41.3	-29.8	-25.4	-23.1
Case (b)	1.0	1.0	1.0	1.0

Negative flat roof pressures indicate roof uplift
h = mean roof height

Parapet Wind Pressure

Parapet wall wind pressures are calculated as follows, based on a top of parapet elevation, h_p, of 65 ft:

Velocity pressure exposure coefficient, $K_z = 1.148$ — *ASCE 7 Table 26.10-1*

Velocity pressure, $q_p = 0.00256\ K_z\ K_{zt}\ K_e\ V^2$ — *ASCE 7 Equation 26.10-15*

Combined net pressure coefficients,

$GC_{pn} = +1.50$ for windward parapet — *ASCE 7 Section 27.3.4*

$GC_{pn} =$ -1.0 for leeward parapet — *ASCE 7 Section 27.3.4*

Combined net pressure on parapet $p_p = q_p K_d (GC_{pn})$ — *ASCE 7 Equation 27.3.3*

Leeward Wall Pressure

Leeward wall wind pressures are calculated as follows based on a mean roof height of $h = 62$ ft:

Velocity pressure exposure coefficient, $K_h = 1.137$ — *ASCE 7 Table 26.10-1*

Velocity pressure, $q = q_h = q_i = 0.00256\ K_h\ K_{zt}\ K_e\ V^2$ — *ASCE 7 Equation 26.10-1*

For $L/B = 189\text{ ft}/76\text{ ft} = 2.49$,

External pressure coefficient

$C_p = \text{-}0.276$ for leeward wall, from linear interpolation — *ASCE 7 Figure 27.3-1*

Internal pressure coefficient, $GC_{pi} = \pm 0.18$ — *ASCE 7 Table 26.13-1*

Design wind pressure, $p = q_h K_d G C_p - q_i K_d (GC_{pi})$ — ASCE 7 Equation 27.3-1

Two cases shall be considered to determine the critical load requirements for the appropriate condition: *ASCE 7 Table 26.13-1*

Case (a) – A positive value of (GC_{pi}) applied to all internal surfaces, or

Case (b) – A negative value of (GC_{pi}) applied to all internal surfaces

Windward Wall Pressure

Unlike leeward wall pressures, windward wall pressures change over the height of the building, with z varying from 0 to 62 feet. Windward wall wind pressures are calculated as follows:

Velocity pressure exposure coefficient, K_z, varies over the height of the building. See Table 4.2A. *ASCE 7 Table 26.10-1*

Velocity pressure, $q = q_z = 0.00256\ K_z\ K_{zt}\ K_e\ V^2$ *ASCE 7 Equation 26.10-1*

External pressure coefficient, $C_p = 0.80$ windward *ASCE 7 Figure 27.3-1*

Design wind pressure, $p = q_z K_d G C_p - q_i K_d (GC_{pi})$ *ASCE 7 Equation 27.3-1*

Similar to the leeward wall, cases (a) and (b) need to be evaluated and are shown in Table 4.2A.

Roof Wind Pressure

Roof wind pressures are calculated as follows based on a mean roof height of $h = 62$ ft:

Velocity pressure exposure coefficient, $K_h = 1.14$ *ASCE 7 Table 26.10-1*

Velocity pressure, $q = q_h = 0.00256\ K_h\ K_{zt}\ K_e\ V^2$ *ASCE 7 Equation 26.10-1*

For $h/L = 62\text{ ft}/76\text{ ft} = 0.82$ and q $< 10°$

External pressure coefficient varies along the length of the roof, per the table below. *ASCE 7 Figure 27.3-1*

Horizontal Distance from Windward Edge	C_p Transverse Direction		
	0 to $h/2$	**$h/2$ to h**	**h to $2h$**
Case (a)*	-1.15	-0.77	-0.63
Case (b)	-0.18	-0.18	-0.18

**Values determined by linear interpolation as permitted*

Design wind pressure, $p = q_h K_d G C_p - q_i K_d (GC_{pi})$ *ASCE 7 Equation 27.3-1*

Figure 4.1 provides a visual representation of how wind pressures are applied in the transverse direction for the structure being analyzed.

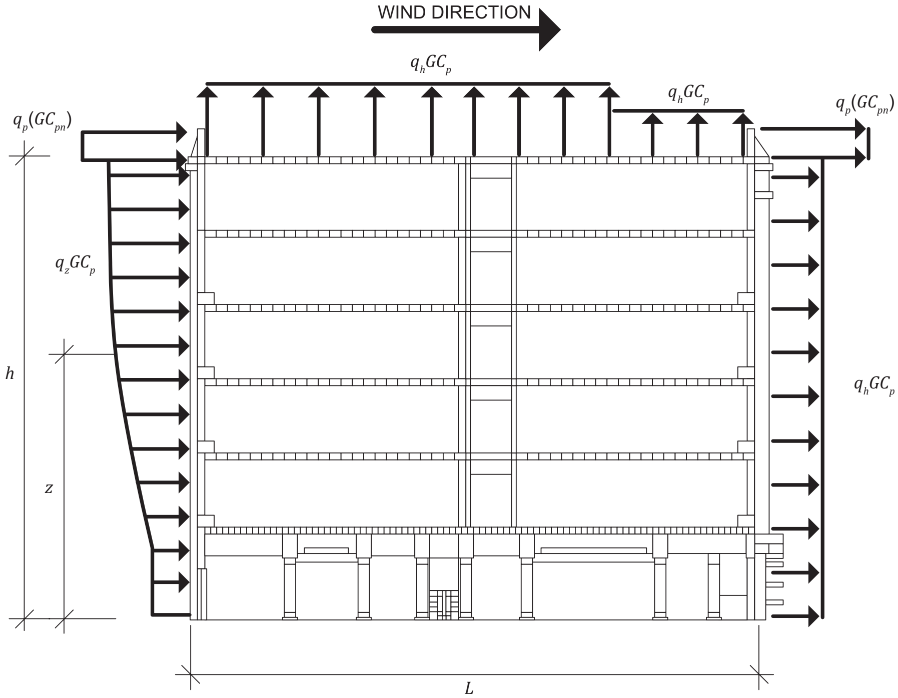


FIGURE 4.1: Transverse wind pressure diagram

4.3.2 Wind Pressures for Longitudinal Direction

Wind pressures in the longitudinal direction are calculated for the parapets, leeward wall, windward wall, and flat roof as outlined below, similar to the calculations for the transverse direction. Results are tabulated in Table 4.2B.

TABLE 4.2B: Wind pressures on the building for the longitudinal direction

Level	Windward Wall Pressure					Leeward Wall Pressure					Total
	z (ft)	K_z	q_z (psf)	p Case (a) (psf)	p Case (b) (psf)	h_x (ft)	K_h	q_h (psf)	p Case (a) (psf)	p Case (b) (psf)	p (psf)
Parapet	65	1.15	42.3	54.0		65	1.15	42.3	-36.0		89.9
Roof	62	1.14	41.9	30.6	17.8	62	1.14	41.9	-1.9	-14.8	32.6
6th	52	1.10	40.4	29.8	17.0	62	1.14	41.9	-1.9	-14.8	31.7
5th	42	1.05	38.7	28.8	16.0	62	1.14	41.9	-1.9	-14.8	30.7
4th	32	0.99	36.6	27.6	14.8	62	1.14	41.9	-1.9	-14.8	29.5
3rd	22	0.92	33.9	26.0	13.2	62	1.14	41.9	-1.9	-14.8	28.0
2nd	12	0.85	31.4	24.5	11.7	62	1.14	41.9	-1.9	-14.8	26.5
Base	0	0.85	31.4	24.5	11.7	62	1.14	41.9	-1.9	-14.8	26.5

	Flat Roof Wind Pressure (psf)			
Horizontal Distance from Windward Edge	0 to h/2	h/2 to h	h to 2h	> 2h
Case (a)	-33.7	-33.7	-21.6	-15.5
Case (b)	1.0	1.0	1.0	1.0

Negative flat roof pressures indicate roof uplift
h = mean roof height

Parapet Wind Pressure

Parapet wall wind pressures are the same in the longitudinal direction as previously calculated in the transverse direction. See Section 4.3.1 and Table 4.2B.

Leeward Wall Pressure

Leeward wall wind pressures for the longitudinal direction are calculated in the same manner as for the transverse direction (Section 4.3.1). However, the building length, L, and width, B, are now switched, resulting in a different L/B ratio and, therefore, different external pressure coefficients, as shown below.

For $L/B = 76 \text{ ft} / 189 \text{ ft} = 0.40$

External pressure coefficient, $C_p = \text{-}0.50$ leeward *ASCE 7 Figure 27.3-1*

As with leeward walls in the transverse direction, design wind pressures, p, need to be evaluated for cases (a) and (b), and are shown in Table 4.2B.

Windward Wall Pressure

Windward wall wind pressures are the same in the longitudinal direction as previously calculated in the transverse direction. See Section 4.3.1 and Table 4.2B.

Roof Wind Pressure

Roof wind pressures for the longitudinal direction are calculated in the same manner as for the transverse direction (Section 4.3.1). However, the h/L ratio is different, resulting in different external pressure coefficients, shown below.

For $h/L = 62 \text{ ft}/189 \text{ ft} = 0.33 \text{ and } \theta < 10°$

External pressure coefficient varies along the length of the roof as shown in the table below, per ASCE Figure 27.3-1.

	C_p Longitudinal Direction			
Horizontal Distance from Windward Edge	**0 to $h/2$**	**$h/2$ to h**	**h to 2h**	**> 2h**
Case (a)	-0.90	-0.90	-0.50	-0.30
Case (b)	-0.18	-0.18	-0.18	-0.18

The rest of this design example examines the transverse direction only, and does not utilize the longitudinal pressures noted above. Longitudinal pressures are shown to illustrate the impact building geometry has on the design wind pressures.

4.4 Seismic Design

4.4.1 Seismic Forces

To determine seismic loads, follow the procedure outlined in Section 3 of this example.

Seismic and Site Data

For this design example, the authors chose a high-wind, low-seismicity site (Providence, RI 02901) with a site-specific soils investigation report, resulting in the following parameters.

Site Class: D

$$S_S = 0.210$$

$$S_1 = 0.048$$

$$S_{DS} = 0.1673$$

$$S_{D1} = 0.0667$$

While a site-specific soils investigation report (geotechnical report) is recommended for each project, it is not always provided in advance of the structural design. When a report is not available, the information above can be determined in other ways.

Per ASCE 7 Section 20.1, where soil properties are not known in sufficient detail to determine the site class, the most critical site conditions of site classes C, CD, and D, as defined in Section 11.4.2, shall be used unless the Authority Having Jurisdiction (AHJ) or geotechnical data determine the presence of site class DE, E, or F soils.

Seismic ground motion and long-period transition maps can be found in Chapter 22 of ASCE 7. The ASCE Hazard Tool also provides these values either by zip code or longitude and latitude coordinates, and the latter (which can be obtained from the street address) is recommended. The tool provides S_S, S_1, S_{MS}, and S_{M1} values. S_{DS} and S_{D1} values can be calculated according to ASCE 7 Section 11.4.4 and are also provided in the tool's output.

Risk category, importance factor, and seismic design category:

Risk Category: II	*ASCE 7 Table 1.5-1*
Seismic importance factor, $I_e = 1.0$	*ASCE 7 Table 1.5-2*
SDC = B	*ASCE 7 Tables 11.6-1 and 11.6-2*

The process for determining SDC is outside the scope of this example but is outlined in ASCE 7 Section 11.6.

Seismic Response Coefficient, C_S, and Seismic Base Shear, V

Detailed seismic load derivation is not provided in this section, but readers can reference Section 3.1 for the steps needed to calculate the seismic forces applied at each level. It is assumed that the example building meets the requirements for two-stage analysis and the light-frame wood upper portion can be analyzed independently.

Based on the procedure outlined in Section 3.1.1:

$$C_S = 0.075$$

and:

$$V = C_S W = 0.084W$$

For the flexible upper portion:

$$W = 2{,}460 \text{ k}$$

$$V = C_S W = 0.075(2{,}460) = 185 \text{ k}$$

Vertical Distribution of Forces

Table 4.3 summarizes the seismic story forces at each level. Table 4.4 shows the tabulated seismic forces at each level, for reference and to illustrate that both wind and seismic loads need to be checked.

TABLE 4.3: Vertical distribution of seismic forces (with base at second floor)

Level	w_x (k)	h_x (ft)	$w_x h_x{}^k$ (k-ft)	C_{vx}	F_x (k)	F_x/A (psf)
Roof	420	50	21,000	29%	54.0	4.5
6th	510	40	20,400	28%	52.4	4.4
5th	510	30	15,300	21%	39.4	3.3
4th	510	20	10,200	14%	26.3	2.2
3rd	510	10	5,100	7%	13.1	1.1
Sum	2,460		72,000	100%	185	

Where C_{vx} = vertical distribution factor defined in Section 3.1.1 and A = area of the floor plate, which is 12,000 ft²

If a designer were to go through the previous steps for their project site and determine that SDC A is applicable, Section 11.7 of ASCE 7 notes that the structure is exempt from seismic design requirements and only require a minimum lateral force be applied to the structure as outlined in ASCE 7 Section 1.4.

4.5 Shear Wall Design Example

As described in Section 3.3, this design example uses a segmented shear wall approach, with individual full-height wall segments in the transverse direction. Table 4.4 shows the wind and seismic forces to a typical interior shear wall line based on the loads determined in Sections 4.3.1 and 4.4. The ASD load combinations applicable to seismic forces were provided in Section 3.3.1. The ASD load combinations appliable to wind forces are provided below (simplified based on the loads being considered in this example):

$1.0D + 0.6W$ *ASCE 7 Section 2.4.1 combination 5a*

$1.0D + 0.75L + 0.75(0.6W) + 0.75L_r$ *ASCE 7 Section 2.4.5 combination 6a*

$0.6D + 0.6W$ *ASCE 7 Section 2.4.5 combination 7a*

TABLE 4.4: Wind and seismic forces to shear wall

Level	Seismic					Wind				
	F_x/A from Table 4.3 (psi)	Tributary Area[a] (ft²)	Story-Level Force, F (lb)	Cumulative Force, F_{total} (lb)	ASD Force: $0.7F_{total}$ (lb)	p from Table 4.2A (psf)	Tributary Area[b] (ft²)	Story-Level Force, F_{total} (lb)	Cumulative Force, F_{total} (lb)	ASD Force: $0.6F_{total}$ (lb)
Parapet						89.9	39	3,508		
Roof	4.5	845	3,804	3,804	2,663	39.4	65	2,559	6,067	3,640
6th	4.4	845	3,687	7,491	5,243	38.5	130	5,007	11,074	6,644
5th	3.3	845	2,775	10,265	7,186	37.5	130	4,878	15,951	9,571
4th	2.2	845	1,850	12,115	8,481	36.3	130	4,720	20,672	12,403
3rd	1.1	845	925	13,040	9,128	34.8	130	4,518	25,189	15,114

Notes:

a. As described in Section 3.3.3, the tributary area for seismic loads in the transverse direction is 845 square feet.

b. The tributary area for wind is the story height times the shear wall spacing. Tributary area for the roof level is one-half the story height times the shear wall spacing plus the parapet height times the shear wall spacing.

For wind design, the design wind load cases prescribed in ASCE 7, Section 27.3.5 and defined in Figure 27.3-8 address the unidirectional loads as well as combinations of reduced magnitude unidirectional loads that are to be applied simultaneously to account for varying wind directions. In certain cases, torsional loads based on reduced lateral loads applied eccentrically about the center of the building are to be included in the load combination. Most light frame structures in multi-family applications, similar to this design example, meet the definition of a torsionally regular building under wind load per ASCE 7 Section 26.2. Designers should consider this on their projects and refer to ASCE 7, Appendix D to evaluate conditions where torsional wind load cases (Case 2 & Case 4) are exempt. The application of these torsional load cases is also dependent on the classification of the diaphragm: rigid, flexible, or semi-rigid. The diaphragm design is outside the scope of this worked example and the following transverse shear wall design is based on applying the wind unidirectionally (Case 1).

4.5.1 **Shear Wall Design: Sheathing and Nailing**

Unlike the seismic design example, which used a single 29-foot shear wall at each line of resistance, the wind design example will use two 29-foot shear walls at each line of resistance (i.e., the unit demising walls that are aligned across the corridor), for a total wall length of 58 feet. The shear walls will have 5/8-inch GWP or 7/16-inch WSP sheathing as noted in Table 4.5; WSP is used at the lower three levels because the shear values exceed the values for using drywall screws on the GWB. Fasteners for GWB are No. 6 1-1/4-inch-long drywall screws (Type W or S, where "W" stands for course **w**ood threads and "S" stands for fine **s**teel threads). Both screw types may be used, but the course wood threads are easier to install in wood studs. Fasteners for WSP are 8d common nails (dimensions) with 1-3⁄8-inch minimum penetration into the framing member. As described in section 4.2, capacities are determined by taking the nominal unit shear capacities in SDPWS Tables 4.3A and 4.3C (for WSP and GWP, respectively) and dividing by the ASD reduction factors of 2.0 for wind and 2.8 for seismic.

TABLE 4.5: Shear wall forces and nailing (ASD)

Level	Load Type	ASD Force from Table 4.4 (lb)	Wall Length (l) (ft)	ASD Design Shear, V (plf)	Wall Sheathed 1 or 2 Sides	Demand per Side (plf)	Panels Used	Fastener Spacing[a]	Blocking	Adjusted Allowable Shear (plf)
Roof	Seismic	2,663	58	46	2	23	5/8-in. GWB	8/12	Unblocked	50
	Wind	3,640		63	2	32		8/12	Unblocked	70
6th	Seismic	5,243	58	90	2	45	5/8-in. GWB	8/12	Unblocked	50
	Wind	6,644		115	2	58		8/12	Unblocked	70
5th	Seismic	7,186	58	124	1	124	7/16-in. WSP sheathing	6/12	Unblocked	156[b,c]
	Wind	9,571		166	1	166		6/12	Unblocked	219[b,c]
4th	Seismic	8,481	58	147	1	147	7/16-in. WSP sheathing	6/6	Unblocked	209[b,c]
	Wind	12,403		214	1	214		6/6	Unblocked	292[b,c]
3rd	Seismic	9,128	58	158	1	158	7/16-in. WSP sheathing	6/12	Blocked	261[b]
	Wind	15,114		261	1	261		6/12	Blocked	365[b]

a. *Fastener spacing is listed as two numbers ("X/X"). The first number is the fastener spacing at the panel edges and the second is the fastener spacing along intermediate (field) members.*

b. *Per SDPWS Table 4.3A, footnote 2, nominal unit shear capacities are permitted to be increased to values shown for 15/32-inch nominal sheathing if studs are spaced at 16 inches o.c. or the panels are applied with the long dimension across the stud.*

c. *Unblocked WSP shear walls have been adjusted based on SDPWS Table 4.3.5.3.*

Based on the tabulated loads in Table 4.4, wind forces are higher than seismic forces. But when selecting the shear wall panels and fastening pattern (Table 4.5) it is still important to check both since wind and seismic capacities are developed using different adjustment factors. For example, at the sixth floor the seismic DCR is 0.9 for the selected sheathing and fastening despite the ASD seismic force being 700 lbs less than the ASD wind force.

Another item worth considering is blocked vs. unblocked shear walls with WSP. SDPWS Table 4.3.5.3 permits the use of unblocked WSP shear walls with adjustment factors applied to the nominal unit shear capacity (Table 4.5, note c). Reducing the amount of blocking within the wall reduces material and labor and improves acoustic performance. However, blocking may still be required within the wall for non-structural reasons such as fire protection or to support interior components.

4.5.2 Shear Wall Design: Chord Members in Compression

Compression Demand from Overturning

As described in Section 3.3, shear and the resulting overturning forces from wind and seismic loads are cumulative in multi-story structures; shear forces applied at upper levels will generate much larger base overturning moments than the same forces applied at lower levels. Overturning moments are resolved into tension-compression (T-C) couples, resulting in axial forces that are transferred by vertical chord elements to the podium and ultimately the foundation.

Overturning moments for the shear wall can be obtained by summing forces about the base of the wall for the level being designed (Figure 3.5), based on the equations shown below. The final expression in the equation is unique to wind loading and accounts for the wind uplift forces at the roof level.

Cumulative overturning moment for the roof level:

$$M_{OT,roof} = F_{roof}(H_{roof}) + W_{uplift}(l_{roof})^2/2$$

Cumulative overturning moment for the sixth-floor level:

$$M_{OT,6} = M_{OT,roof} + (F_{roof} + F_6)(H_6) = F_{roof}(H_{roof} + H_6) + F_6(H_6) + W_{uplift}(l_6)^2/2$$

Cumulative overturning moment for the fifth-floor level:

$$M_{OT,5} = M_{OT,6} + (F_{roof} + F_6 + F_5)(H_5) = F_{roof}(H_{roof} + H_6 + H_5) + F_6(H_6 + H_5) + F_5(H_5) + W_{uplift}(l_5)^2/2$$

Cumulative overturning moment for the fourth-floor level:

$$M_{OT,4} = M_{OT,5} + (F_{roof} + F_6 + F_5 + F_4)(H_4) = F_{roof}(H_{roof} + H_6 + H_5 + H_4) + F_6(H_6 + H_5 + H_4) + F_5(H_5 + H_4) + F_4(H_4) + W_{uplift}(l_4)^2/2$$

Cumulative overturning moment for the third-floor level:

$$M_{OT,3} = M_{OT,4} + (F_{roof} + F_6 + F_5 + F_4 + F_3)(H_3) = F_{roof}(H_{roof} + H_6 + H_5 + H_4 + H_3) + F_6(H_6 + H_5 + H_4 + H_3) + F_5(H_5 + H_4 + H_3) + F_4(H_4 + H_3) + F_3(H_3) + W_{uplift}(l_3)^2/2$$

The T-C couple used to design the chord elements (Table 4.6) is determined by dividing the overturning moment by the distance, d (referred to as b_{eff} in SDPWS), between the center of the tension rod and centroid of the compression posts (Figures 3.6 and 3.7). Some engineers will take a conservative estimate of d and use the same value for all levels to minimize iterations. However, this example uses actual distances based on sizes of the as-designed compression posts.

TABLE 4.6: Overturning moments and tension-compression couple forces[a]

Level	Load Type	Shear Wall Story Force[b] (lb)	Story Height (ft)	Cumulative M_{OT} (ft-k)	d (ft)	Force Couple +/- (k)
Roof	Wind	3,033	10	65[c]	28.75	2.3
6th	Wind	2,504	10	120	28.75	4.2
5th	Wind	2,439	10	200	28.63	7.0
4th	Wind	2,360	10	304	28.50	10.7
3rd	Wind	2,259	10	430	28.38	15.1

a. *Only wind loads are shown as they govern the design of the shear wall boundary elements.*

b. *Shear wall story-level force is F_{total} from Table 4.4 divided by two for the two shear walls along the line of resistance.*

c. *Value includes contribution from flat roof uplift pressure over the length of the shear wall.*

Total Compression Demand

In addition to the overturning forces calculated in Table 4.6, the chords also resist gravity compression loads (Section 3.3.5). In this design example, ASD load combination 5a will govern the compression forces on the chord members; the resulting forces are summarized in Table 4.7.

TABLE 4.7: Compression chord forces (ASD)

Level	Total Chord Length[a] (ft)	P_D[b,c] (k)	P_L[b,c] (k)	P_{Lr}[b,c] (k)	Wind Compression Force, P_W from Table 4.6 (k)	Total Compression Demand (Load Combo 5a) $1.0D + 0.6W$ (k)
Roof	0.67	0.10	0.00	0.03	2.3	1.46
6th	0.67	0.43	0.35	0.03	4.2	2.94
5th	0.67	0.76	0.69	0.03	7.0	4.95
4th	0.67	1.08	1.04	0.03	10.7	7.47
3rd	0.67	1.41	1.39	0.03	15.1	10.49

a. Total chord length is taken as the larger of half of the typical stud spacing or the distance from the edge of the shear wall to the centroid of the compression post.

b. Dead, live, and roof live loads are taken as the tributary area of the interior transverse wall times the total chord length.

c. The cumulative line loads are taken from Section 3.3.5. See Table 3.7.

Compression Capacity of Chord

Compression chords are similar to those described in Section 3.3.5 for seismic. Chord posts are assumed to be comprised of multiple 2x4 members stitch-nailed together. The design is based on the following compressive capacities of a single 2x4, which were calculated in Section 3.3.5:

$F'_c = 450 \text{ psi}$

$F_{c\perp} = 625 \text{ psi}$

TABLE 4.8: Compression chord design

Level	Chord Posts	Area (in.²)	F'_c (psi)	$F'_{c\perp}$ (psi)	P_{allow} (k)	Demand From Table 4.7 (k)	DCR
Roof	(2) 2x4	10.5	450	625	4.73	1.46	31%
6th	(2) 2x4	10.5	450	625	4.73	2.94	62%
5th	(3) 2x4	15.8	450	625	7.09	4.95	70%
4th	(4) 2x4	21.0	450	625	9.46	7.47	79%
3rd	(5) 2x4	26.3	450	625	11.82	10.49	89%

4.5.3 Shear Wall Design: Chord Members in Tension

Similar to compression, the tension demand on the chord is determined by combining wind overturning loads with gravity loads. For this example, ASD load combination 7a will govern the tension forces on the chord members. As described in Section 3.3.6, dead load is used to resist the overturning moment. Once the net resultant moment is calculated based on the appropriate load combinations, it is resolved into a tension force at the end of the wall by dividing by d. Calculations are summarized in Table 4.9.

TABLE 4.9: Tension chord forces (ASD)

Level	Cumulative Resisting Moment (Dead Load) M_R (ft-k)	Cumulative M_{OT} (Wind) from Table 4.6 (ft-k)	d (ft)	Net Resultant Tension Force $0.6D + 0.6W$ (lb)[a]
Roof	65.6	-65.1	28.75	0
6th	271.6	-120.5	28.75	0
5th	477.7	-200.2	28.63	0
4th	683.7	-304.6	28.50	0
3rd	889.8	-430.5	28.38	0

a. Where $M_R + M_{OT} > 0$ there is no net overturning moment and the net resultant tension force is shown as zero.

As shown in Table 4.9, there is no net uplift at the shear wall ends. In situations where wind loads control, designers should closely examine the dead loads if they are being used to resist overturning. Overestimating the dead load will result in conservative seismic forces and gravity member design, but is unconservative for evaluating overturning for wind loads. A designer may opt to take a reduced dead load when evaluating overturning forces. In this example, if the dead loads were reduced by 2-3 psf at each level, there would be net tension at some levels. In these situations, designers may still opt to specify some form of floor-to-floor hold-down device such as straps or bucket style hold-downs vs. the more robust continuous rod system.

4.6 Shear Wall Deformations and Story Drift

Refer to Section 3.4 for a detailed overview of evaluating shear wall deformations and story drift. Although wind loads govern the design, seismic drift requirements in Chapter 12 of ASCE 7 still need to be met. However, there are no explicit criteria for wind loads within the IBC or ASCE 7. Limitations for wind loads are typically governed by serviceability—e.g., occupant comfort and finish requirements. Pathways that can be considered when evaluating drift from wind loads include:

- *IBC wind deflection criteria* – IBC Table 1604.3 (footnote f) permits using the 10-year mean return interval basic wind speed.
- *ASCE 7 Commentary to Appendix C* – ASCE 7 does not mandate a specific allowable story drift for wind but provides non-mandatory serviceability guidelines in its commentary. Drift limits commonly used in practice range from about $H/600$ to $H/400$ of the building or story height (0.17% – 0.25% drift per story).
- *Griffis Method* – Griffis Method for Evaluating Wind Drift (AISC, 1993), which is referenced in the ASCE 7 Commentary, uses a reduced wind load of $0.42W$ (equivalent to a 10-year event) or a representative return period for a given wind event. The Griffis paper also provides discussion on perceived motion by occupants and tabulated values for drift limits and associated serviceability concerns such as ceiling cracks and water leaks.

Photo: Gabe Borde

The Fowler | Boise, Idaho
ARCHITECT: Holst Architecture
DEVELOPER: Roundhouse
STRUCTURAL ENGINEER: Axiom
CONTRACTOR: Andersen Construction

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End Notes

[1] ASCE 7-22 has expanded its classification for site class to include intermediate site class values.

[2] Meaningful updates to the process for determining ρ were introduced in ASCE 7-22. Refer to Section 12.3.4 and the respective sub-sections.

[3] Marking a significant change from previous versions, ASCE 7-22 Section 12.6 permits use of the ELF procedure for all buildings, eliminating restrictions on analytical procedures found in Table 12.6-1 of ASCE 7-16 and earlier editions.

[4] In ASCE 7-22 the wind directionality factor, K_d, has been removed from the velocity pressure calculation. This factor is now within the individual wind pressure equations. ASCE 7-22 Commentary highlights the reasoning for this change.

Materials referenced throughout this design example can be found at woodworks.org. Use the search bar or visit the Resource Library to browse or search hundreds of resources by key word, project role, building type, and other filters.

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The Bishop | Atlanta, Georgia
ARCHITECT: Niles Bolton Associates
DEVELOPER: Pollack Shores Real Estate Group
STRUCTURAL ENGINEER: EM Structural
CONTRACTOR: Juneau Construction Company

University of Washington Student Housing | Seattle, WA
ARCHITECT: Mahlum
STRUCTURAL ENGINEER: Coughlin Porter Lundeen
CONTRACTOR: Walsh Construction; WG Clark Construction

WG Clark Construction

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