



D E S I G N

E X A M P L E

Four-story Wood-frame Structure over Podium Slab



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Overview

This design example illustrates the seismic design of a four story wood framed hotel over one story of concrete podium slab which is assigned to Seismic Design Category D. The gravity load framing system consists of wood-frame bearing walls for the upper stories and concrete bearing walls for the lower story. The lateral load resisting system consists of wood-framed shear walls for the upper stories and concrete shear walls for the lower story. Typical building elevation and floor plan of the structure are shown in Figures 1 and 2 respectively. A typical section showing the heights of the structure is shown in Figure 4. The wood roof is framed with pre-manufactured 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 composition shingles.

This design example uses the term “podium slab” which, while not included in the *2006 International Building Code (IBC)* or *2007 California Building Code (CBC)*, is commonly used in the building industry. This type of construction is also referred to as an uppermost “structural slab” or “transfer slab” (both can have a slab and beam system) that is designed to support the entire weight of the wood superstructure.

When designing this type of “mid-rise” wood-frame structure, there are several unique design elements to consider. The following steps provide a detailed analysis of some of the important seismic requirements of the shear walls per the 2006 IBC and 2007 CBC.

This example is not a complete building design. Many aspects have not been included, specifically the gravity load framing system, and only certain steps of the seismic design related to portions of a selected shear wall have been illustrated. In addition, the lateral requirements for wind design related to the selected shear wall have not been illustrated (only seismic). The steps that have been illustrated may be more detailed than what is necessary for an actual building design but are presented in this manner to help the design engineer understand the process.

Codes and Reference Documents Used

2006 International Building Code (IBC)

2005 National Design Specification® (NDS®) for Wood Construction – ASD/LRFD

American Institute of Steel Construction Steel Construction Manual – Thirteenth Edition

2007 California Building Code (CBC)

This design example focuses on the IBC and NDS requirements. Where there is a difference between the IBC and CBC, a comment and reference is made.

Interior and exterior wall weights have not been included in the above loads; they have been included in the diaphragm weights shown below. Typical interior and exterior partition weights can vary between 10 psf and 20 psf depending on room sizes, number of layers of gypsum board on walls, etc.

Weights of respective diaphragm levels, including tributary exterior and interior walls:

FLEXIBLE UPPER PORTION

$$\begin{array}{rcl}
 W_{roof} & = & 587 \text{ k} \\
 W_{5th \text{ floor}} & = & 639 \text{ k} \\
 W_{4th \text{ floor}} & = & 647 \text{ k} \\
 \underline{W_{3rd \text{ floor}}} & = & 647 \text{ k} \\
 W & = & 2,520 \text{ k}
 \end{array}$$

RIGID LOWER PORTION

$$\begin{array}{rcl}
 W_{upper} & = & 2,520 \text{ k} \\
 \underline{W_{2nd \text{ floor}}} & = & 2,632 \text{ k} \\
 W & = & 5,152 \text{ k}
 \end{array}$$

Weights of roof diaphragms are typically determined by taking one half the height of the walls from the fifth floor to the roof. Weights of floor diaphragms are typically determined by taking one-half of the walls above and below for the fifth, fourth and third floor diaphragms. The weights of all walls, including interior non-bearing partitions, are included in the respective weights of the various levels. The weight of parapets (where they occur) has been included in the roof weight.

The roof is 1/2-inch-thick DOC PS 1 or DOC PS 2-rated sheathing, 32/16 span rating with Exposure I glue.

The floor is 23/32-inch-thick DOC PS 1 or DOC PS 2-rated Sturd I Floor 24 inches o.c. rating, 48/24 span rating with Exposure I glue.

DOC PS 1 and DOC PS 2 are the U.S. Department of Commerce (DOC) Prescriptive and Performance-based standards for plywood and oriented strand board (OSB), respectively.

Wall framing is a modified balloon framing where the joists hang from the walls in joist hangers (see Figure 6).

Framing lumber for studs and posts

NDS Table 4A

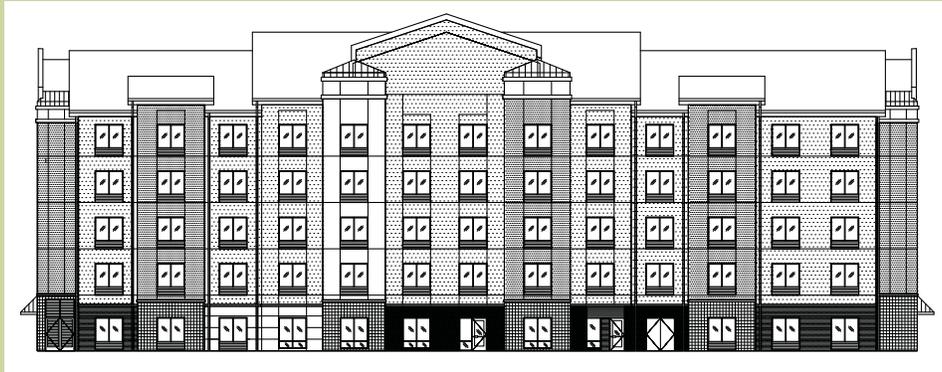
DOUGLAS FIR LARCH-NO. 1 GRADE:

$$\begin{array}{l}
 F_b = 1,450 \text{ psi} \\
 F_c = 1,500 \text{ psi} \\
 F_t = 1,500 \text{ psi} \\
 E = 1,700,000 \text{ psi} \\
 E_{min} = 620,000 \text{ psi} \\
 C_m = 1.0 \\
 C_t = 1.0
 \end{array}$$

Common wire nails are used for shear walls, diaphragms and straps. When specifying nails on a project, specification of the penny weight, type, diameter and length (example 10d common = 0.148" x 3") are recommended.

The IBC, NDS and associated *Special Design Provisions for Wind and Seismic (SDPWS)* list values for shear walls and diaphragms. For values using nail and sheathing thickness not listed in the IBC and NDS/SDPWS, the engineer can also consider using the values listed in *International Code Council-Evaluation Service (ICC-ES) Report ESR-1539* from the International Staple, Nail and Tool Association (ISANTA). This report can be downloaded from ISANTA's website at <http://www.isanta.org> or from the International ICC-ES website at <http://www.icc-es.org>.

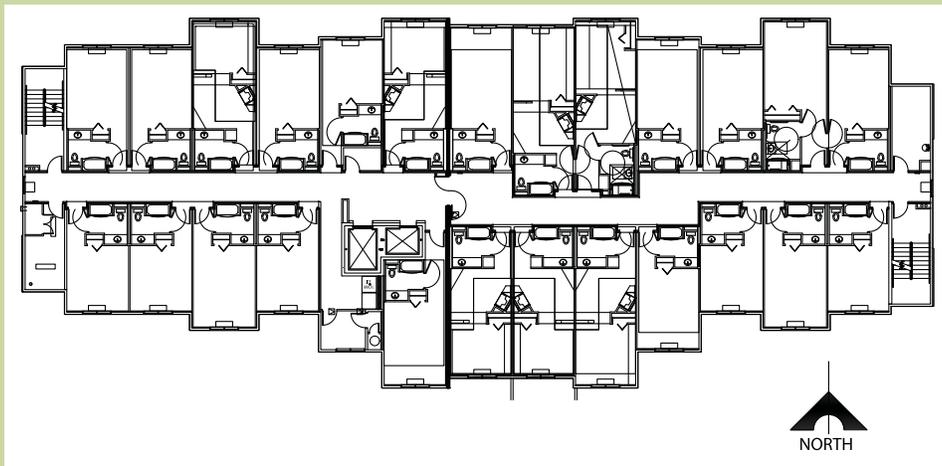
Figure 1. Building Elevation



NOTE FOR FIGURE 1:

See Figure 2 for building plan dimensions and Figure 4 for building height dimensions.

Figure 2. Typical Floor Plan



NOTE FOR FIGURE 2:

In Figure 2, the prefabricated wood I-joists run east-west spanning to the wood-bearing walls separating the hotel guest units running north-south at 13 feet o.c. The floor area is 12,000 square feet.



Factors That Influence Design

Prior to starting the seismic design of a structure, the following must be considered:

Species of Lumber

The species of lumber used in this design example is Douglas Fir-Larch (DF-L), which is common on the west coast. The author does not intend to imply that this species can or 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 also commonly include (among others) Southern Yellow Pine (SLP) and Spruce Pine Fir (SPF).

Grade of Lumber

The lower two stories of the wood-frame structure carry significantly higher gravity loads than the upper two stories. One approach is to use a higher grade of lumber for the lower two stories than the upper two stories. This approach can produce designs that yield a constant wall construction over the height of the building. Another approach would be to choose one grade of lumber for all four wood-frame stories. This approach produces the need to change the size and/or spacing of the studs based on the loading requirements. Sill plate crushing may control stud sizing at lower levels. For simplicity, this design example illustrates the use of one lumber grade for all floor levels.

Figure 3. Typical Grade Stamp



NOTES FOR FIGURE 3:

- a. Certification Mark:** Certifies association quality supervision
- b. Mill Identification:** Firm name, brand or assigned mill number
- c. Grade Designation:** Grade name, number or abbreviation
- d. Species Identification:** Indicates species by individual species or species combination
- e. Condition of Seasoning:** Indicates condition of seasoning at the time of surfacing

Moisture Content and Wood Shrinkage

From a serviceability and performance perspective, the most significant issue related to multistory wood-frame construction is wood shrinkage—which is impacted by the moisture content (MC) and, more specifically, whether the wood used is “green” or “kiln dried.”

The availability of both types is largely dependent on the region and associated market conditions. Typically, wood used in construction in the U.S. southwest is “green” (S-GRN) and kiln dried (KD) wood is relatively rare, while the opposite is true in other parts of the country. The engineer should consider the availability of kiln dried lumber in the area of the proposed construction. The WoodWorks website provides access to technical support offered either one-on-one or via wood associations nationwide, to help designers looking for this type of information. To learn more, visit: www.Woodworks.org/aboutWoodworks/technical-support.aspx

Condition of Seasoning

There are three levels of wood seasoning (drying), which denotes the moisture content of the lumber at the time of surfacing. The identification “stamps” are as follows:

S-GRN = over 19% moisture content (unseasoned)

S-DRY, KD or KD-HT = 19% maximum moisture content (seasoned)

MC 15 or KD 15 = 15% maximum moisture content

These designations may be found in the grade stamp.

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

Heat treated (HT) lumber is lumber that has been placed in a closed chamber and heated until it attains a minimum core temperature of 56°C for a minimum of 30 minutes.

The word “DRY” indicates that the lumber was either kiln or air dried to a maximum moisture content of 19 percent.

Kiln dried (KD) lumber is lumber that has been seasoned in a chamber to a pre-determined moisture content by applying heat.

Kiln dried heat treated (KD-HT) lumber has been placed in a closed chamber and heated until it achieves a minimum core temperature of 56°C for a minimum of 30 minutes.

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

Engineered I-joists were used for this design example; however, given the short span on the floor joists, sawn lumber could have been used. In this case, the joist shrinkage perpendicular to grain would need to be included in the overall shrinkage calculation. Also, sawn lumber joists can be supported in joist hangers (see Figure 6) so as not to contribute to the overall building shrinkage. For this design example, “sawn” lumber is used for the stud-framed walls and pre-manufactured roof trusses.

For further explanation of moisture content and wood shrinkage, see section 5.

Location of Shear Walls

The lateral force-resisting system in this design example uses both interior and exterior walls for shear walls (see Figure 2). The seismic force-resisting system for the transverse direction (north-south) utilizes the interior walls between the hotel guest rooms. A seismic design of a selected interior shear wall in the transverse direction is illustrated in this design example. The seismic force-resisting system for the longitudinal direction (east-west) utilizes the long interior corridor walls located at the center of the structure, with shear walls on both sides of the corridor in addition to shear walls on the exterior walls and shear walls at the bathroom walls.

Related to the lateral force-resisting system in the longitudinal direction for structures similar to this design example, it is recognized that some structural engineers will only utilize the interior corridor walls and not place shear walls on the exterior walls. This type of design uses a *rigid diaphragm* approach to the distribution of lateral forces to the shear walls. Code requirements for *semi-rigid* or *flexible* diaphragms on structures similar to this design example would not allow the elimination of the exterior shear walls in the longitudinal direction. While the code does not explicitly prohibit the elimination of exterior shear walls for wood-framed structures, from a performance perspective, the elimination is not recommended.



Support of Floor Joists

This design example uses balloon framing. The floor joists are supported in joist hangers hung from the top plates (see Figure 6). The wall studs and posts have a simple span between the top of the sole plate and the bottom of the lower top plate.

For wood-frame structures built with regular platform construction, the floor joists are supported by direct bearing onto the top plate(s) (see Figure 6A).

Calculations and Discussion

Code Reference

1. Four Stories Type V Wood Framing over Type I Concrete

ASCE 7-05 §12.2.3.1

1a. Structural/Seismic Height Limitation

The heights of the floors and roof are shown in Figure 4.

MAXIMUM HEIGHT OF STRUCTURE:

ASCE 7-05 Table 12.2.1

Table 12.2.1 of ASCE 7-05 lists the maximum height of a structure, measured from its base, related to the seismic force-resisting system (SFRS) and the Seismic Design Category (SDC). Section 11.2 defines the *base* of the structure as “the level at which horizontal seismic ground motions are considered to be imparted on the structure.”

Industry standard for the height of the wood-framed building is measured from the top of the podium slab to the *average* roof sheathing elevation. Using the podium slab as the base for the light-framed walls sheathed with wood structural panels:

The height limit in *SDC 'D'* is 65 feet

The average (mean) height of the structure is 49.1 feet

$65 > 49.1$ okay

1b. Fire and Life Safety Height and Area Limitations

TYPE V OVER TYPE IA (OR TYPE II) CONSTRUCTION:

IBC Table 503

Lower Portion

Type 1A construction

Occupancy is S2, B, E and A2

Per IBC Table 503:

Allowable height is unlimited

Allowable number of stories is unlimited

Per IBC Table 503:

Allowable area is unlimited

Upper Portion-try as Type VA Construction

Type VA construction

Occupancy is R-2

Per IBC Table 503:

Allowable height is 50 feet

Allowable number of stories is three

Allowable area is 12,000 square feet per story \geq 12,000 square feet okay

IBC §504.2 states that for Group R buildings equipped with an automatic sprinkler system, the value specified in Table 503 for the maximum height can be increased by 20 feet and the maximum number of stories increased by one, but shall not exceed 60 feet in height or four stories.

Modified allowable height is 60 feet (max) $>$ 49.1 feet okay

Modified number of stories is four \leq four stories okay

IBC Table 503 defines the height limit and story limit as being *above grade plane*; hence it cannot count the upper structure only, but must consider the building as a whole. However, IBC §509.2 defines the number of stories allowed as pertaining to the wood portion above the podium slab horizontal assembly.

SUMMARY:

Structural/seismic height limit controls the building height at 65 feet.

This structure will be a Type VA structure over a Type I structure with automatic fire sprinklers.

1c. Wood Studs in Fire-resistance-rated Walls

IBC Table 720.1(2)

When wood-frame structures exceed the limits for Type V construction, the code requires either Type III or Type IV construction.

IBC §602.3 defines Type III construction as buildings with exterior walls made from non-combustible materials. Therefore, the use of fire-retardant-treated wood (FRTW) is required for the exterior load-bearing wall assemblies.

Fire-rated assemblies can be found in a number of sources including the IBC, the Underwriters Laboratories (UL) *Fire-resistance-rated Systems and Products*, the UL *Fire Resistance Directory*, and the Gypsum Association's *Fire Resistance Design Manual*.

Table 720.1(2) of the IBC lists fire ratings for various wall construction types. Many of the wall construction types using wood construction reference footnote 'm.' Footnote 'm' of the table requires the reduction of F'_c to be 78 percent of the allowable when the slenderness ratio $l_e/d > 33$.

The American Forest & Paper Association (AF&PA) has tested a number of wood-frame fire-rated assemblies. There is a disparity between the IBC and publications such as AF&PA's *Fire-Rated Wood-Frame Wall and Floor/Ceiling Assemblies*, which does not require the reduction in allowable stress. The building's architect and/or engineer should check with the local jurisdiction to determine the accepted approach. The AF&PA procedure is detailed at: <http://www.awc.org/pdf/CalculatingtheSuperimposedLoadonWoodFrameWalls.pdf>.



DETERMINATION OF C_L :

NDS-05 3.3.3.2

When studs have gypsum sheathing or structural panel sheathing on *both* sides of the studs and posts, where the compressive edges are held in line, C_L may be assumed to be 1.0.

$l_e = l_U$ = the clear height of the studs

This design example has sheathing on both sides, therefore $C_L = 1.0$.

However, when a sound wall is used and the studs are staggered where one edge of the stud does not have its compressive edge held in line, C_L needs to be calculated. For this loading condition, the effective unbraced length l_e for the studs and posts is listed in NDS-05 Table 3.3.3 as follows:

For a 10'-0" floor-to-floor height with a 2x4 sole plate with a 4x4 top plate:

$$\frac{l_U}{d} = \frac{114in}{3.5in} = 33 > 7$$

Therefore:

$$l_e = 1.631l_U + 3d$$

Solving for $l_e/d = 33$ yields the following stud and post lengths for the footnote 'm' reduction in F'_c :

For 4x studs and posts:

$$l_U > 5'-4"$$

For 6x studs and posts:

$$l_U > 8'-5"$$

Since most wall heights for new buildings are 9 to 10 feet, this reduction in F'_c is basically applied to all bearing walls in a *fire-rated* wall.

It should be noted that this is an IBC requirement and not an NDS requirement.

NOTE:

American Wood Council publication DCS3 (which can be downloaded at www.awc.org) provides wood stud walls tested to 100 percent design load. These walls can be used without the 0.78 reduction factor. Local building department requirements should be checked.

2. Two-stage Design for Lateral Analysis

ASCE 7-05 §12.2.3.1

Due to IBC Table 508.3.3 requirements for building occupancies, a one-hour area separation between the first floor (A-3 Occupancy) and the second floor (R-1 Occupancy) is necessary. Also, if the sub-structure (first floor) is for parking, a three-hour separation is required per IBC §509.2.

The seismic response coefficient R for the first floor special concrete shear walls and special reinforced masonry shear walls is 5.0. The seismic response coefficient R for the wood structural panel shear walls is 6.5. Section 12.2.3.1 of ASCE 7-05 requires the least value of R to be used for the building for the seismic design in that direction.

One approach that can be used for the seismic design would be to design the entire structure for the R value of 5.0. However, this would require the upper wood-framed portion of the structure to be designed for 30 percent higher forces in addition to inverting more of the building's mass (second floor) into the upper stories.

A more realistic approach (from both a seismic and economic perspective) would be to design the structure using the two-stage equivalent lateral force procedure prescribed in ASCE 7-05. This procedure can be used where there is a flexible upper portion and a rigid lower portion. This structure type (two-stage design) would be in effect the structural opposite to the “soft story” structures that are not desirable.

The allowance of two-stage-equivalent lateral force procedure for a flexible upper portion above a rigid lower portion has been in the building code since the 1988 Uniform Building Code with essentially the same variables. This procedure is permitted when the structure complies with the following criteria:

- A. The stiffness of the lower portion must be at least 10 times the upper portion.
- B. The period of the entire structure shall not be greater than 1.1 times the period of the upper portion.
- C. The flexible upper portion shall be designed as a separate structure using the appropriate values of R and ρ .
- D. The rigid lower portion shall be designed as a separate structure using the appropriate values of R and ρ .

For the purpose of this design example, the building is regular and qualifies for the *Equivalent Lateral Force Procedure* to be used.

2a. Stiffness Determinations

Stiffness of the lower portion must be at least 10 times the upper portion.

Wall rigidity (stiffness):

$$F = k\delta$$

Or

$$k = \frac{F}{\delta}$$

Where: F = the applied force to the wall
 k = the stiffness of the wall
 δ = deflection of the wall

STIFFNESS OF FLEXIBLE UPPER PORTION:

Determine stiffness of typical interior cross wall:

Table 1. Determine stiffness of typical interior wall

Level	F (k)	Deflection δ_{xe} (in)	$k = \frac{F}{\delta}$ (k/in)
Roof	13.935	0.27	51.61
5th Floor	23.415	0.30	78.05
4th Floor	29.820	0.31	96.19
3rd Floor	33.045	0.31	106.60

Where: F = the applied force to the wall as determined from Table 6
 δ = the computed shear wall deflection from Table 16



STIFFNESS OF RIGID LOWER PORTION:

Determine stiffness of typical interior cross wall:

From 3-D finite element analysis of the rigid lower portion, the average deflection of the first floor transverse shear wall at design seismic loading:

$$\delta_{walls} = 0.02 \text{ in}$$

$$F_{wall} = 190 \text{ kips}$$

$$k = \frac{190k}{0.02 \text{ in}} = 9,500 \frac{k}{in}$$

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

$$ratio = \frac{9,500}{106.60} = 89 > 10 \Rightarrow \text{okay}$$

2b. Period Determinations

Check for conformance to the requirement that the period of the entire structure must not be greater than 1.1 times the period of the upper portion.

First determine building periods (see Figure 4 for section through structure) using the approximate fundamental period equations of ASCE 7-05 as opposed to computer model calculations.

For the flexible upper portion:

$$T_a = C_t(h_n)^x = 0.020(62.84)^{3/4} = 0.45 \text{ sec} \quad \text{ASCE 7-05 Eq. 12.8-7}$$

For the entire structure:

$$T_a = C_t(h_n)^x = 0.020(74.84)^{3/4} = 0.50 \text{ sec} \quad \text{ASCE 7-05 Eq. 12.8-7}$$

Ratio of periods:

$$\frac{0.50}{0.45} = 1.14 \cong 1.1 \Rightarrow \text{close-enough}$$

Using the ASCE 7-05 equation can produce period ratios > 1.1. This equation is problematic since the same equation is used for both wood and concrete shear walls to determine the building period.

ALTERNATE METHOD OF PERIOD DETERMINATION:

$$T = 2\pi \sqrt{\left(\sum_{i=1}^n w_i \delta_i^2\right) \div \left(g \sum_{i=1}^n f_i \delta_i\right)} \quad \text{FEMA 450 Eq. C5.2-1}$$

The above equation, which produces a more accurate building period, is based on Rayleigh's method and was the equation that appeared in the Uniform Building Codes (Eq. 30-10 in the 1997 UBC).

Table 2. Determine period of flexible upper portion

Level	w (k)	f (k)	δ(in)	w(δ) ²	fδ
Roof	587	197.9	0.27	42.79	53.43
5th Floor	639	134.6	0.30	57.51	40.38
4th Floor	647	91.0	0.31	62.18	28.21
3rd Floor	647	45.5	0.31	62.18	14.10
Σ	2,520	469.0		224.66	136.12

$$T = 2\pi \sqrt{\frac{224.66}{(32.2 \times 12) 136.12}} = 0.41 \text{ sec}$$

Table 2A. Determine period of entire structure

Level	w (k)	f (k)	δ(in)	w(δ) ²	fδ
Roof	587	197.9	0.27	42.79	53.43
5th Floor	639	134.6	0.30	57.51	40.38
4th Floor	647	91.0	0.31	62.18	28.21
3rd Floor	647	45.5	0.31	62.18	14.10
2nd Floor	2,632	489.5	0.02	1.05	9.79
Σ	5,152	958.50		225.71	145.91

$$T = 2\pi \sqrt{\frac{225.71}{(32.2 \times 12) 145.91}} = 0.40 \text{ sec}$$

Ratio of periods:

$$\frac{0.40}{0.41} = 0.98 \leq 1.1 \Rightarrow \text{okay}$$

2c. Design of Flexible Upper Portion

Design coefficients for the Seismic Force-Resisting System (SFRS) from ASCE 7-05 Table 12.2-1 are as follows:

Type A-13: Light-framed walls with wood sheathing

$$R = 6.5$$

$$\Omega_0 = 3.0$$

$$C_d = 4.0$$

Maximum building height:

No height limit for seismic design categories B & C

65 feet for seismic design categories D, E & F



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

2d. Design of Rigid Lower Portion

Design coefficients for the SFRS:

Usually A1/A7:

For special reinforced concrete shear walls

$$R = 5.0$$

$$\Omega_0 = 2.5$$

$$C_d = 5.0$$

For special reinforced masonry shear walls

$$R = 5.0$$

$$\Omega_0 = 2.5$$

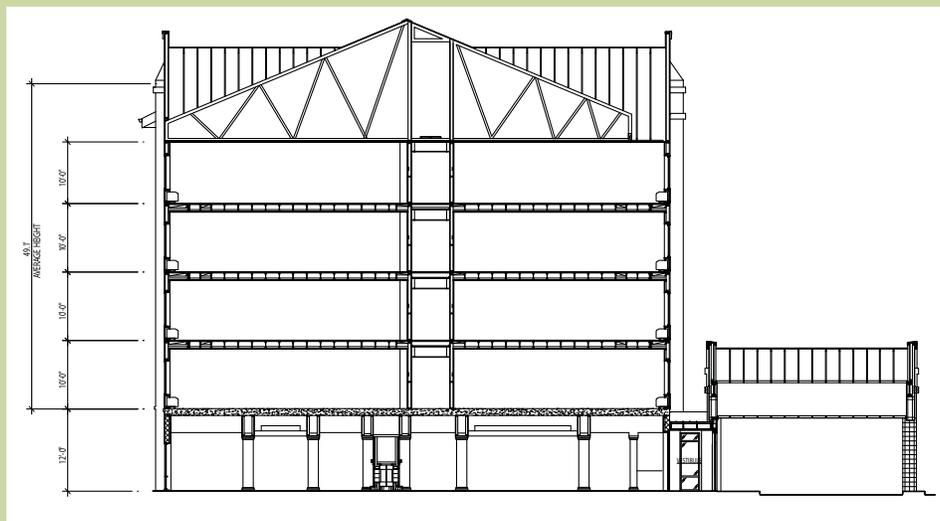
$$C_d = 3.5$$

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

3. Seismic Design of Flexible Upper Portion and Lower Rigid Portion

3a. Seismic Design of Flexible Upper Portion

Figure 4. Typical Cross-section through Building



NOTE FOR FIGURE 4:

If parallel chord trusses are used instead of pitched chord trusses, the overall building height can be reduced.

SEISMIC AND SITE DATA:

Seismic Design Category D

For building frame systems with light-frame walls sheathed with wood structural panels

$R = 6.5$ **ASCE 7-05 Table 12-2.1**

Redundancy factor $\rho = 1.0$ **ASCE 7-05 §12.3.4.2**
(See section 3d)

Design base shear is:

$V = C_s W$ **ASCE 7-05 Eq.12.8-1**

Note: design base shear is a strength design basis.

$C_s = \frac{S_{DS}}{\left(\frac{R}{I}\right)}$ **ASCE 7-05 Eq.12.8-2**

Where:

Site Class D (stiff soil)

Site Class D has been determined by a geotechnical investigation. Without a geotechnical investigation, Site Class D shall be used as the default value.

$I = 1.0$

$R = 6.5$

Values for S_s and S_1 can be determined from ASCE 7-05 maps or from the U.S. Geological Survey (USGS) website, which provides the values by either zip code or longitude and latitude coordinates. It is recommended that the longitude and latitude coordinates (which can be obtained from the street address) be used.

USGS website link:

<http://earthquake.usgs.gov/research/hazmaps/design>

Download the JAVA Ground Motion Parameter Calculator and enter latitude and longitude.

$S_s = 1.809$ **ASCE 7-05 Figure 22-1**

MAXIMUM VALUE IN DETERMINATION OF C_s

ASCE 7-05 §12.8.1.3

For regular structures five stories or less in height and having a period of 0.5 seconds or less, ASCE 7-05 permits the value of S_s to be limited to 1.5. Since the structure in this design example has a "Type II" weight (mass) irregularity between the second and third floors, a vertical irregularity exists. It is not clear whether a building that is designed using the two-stage analysis (ASCE 7-05 §12.2.3.1) should be exempted from this provision. Since each structure can be treated separately, it seems reasonable to conclude that the weight mass irregularity does not apply in the two-stage design approach. However, this design example does not exempt the irregularity and hence the cutoff value for S_s is not used. For actual projects, building officials in the local jurisdiction should be contacted for their interpretation of the code.



$$S_1 = 0.692$$

ASCE 7-05 Figure 22-2

$$F_a = 1.0$$

ASCE 7-05 Table 11.4-1

$$F_v = 1.5$$

ASCE 7-05 Table 11.4-2

$$S_{MS} = F_a S_s = 1.0(1.809) = 1.809$$

ASCE 7-05 Eq. 11.4-1

$$S_{M1} = F_v S_1 = 1.5(0.692) = 1.038$$

ASCE 7-05 Eq. 11.4-2

$$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3} (1.809) = 1.206$$

ASCE 7-05 Eq. 11.4-3

$$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3} (1.038) = 0.692$$

ASCE 7-05 Eq. 11.4-4

Values for T_L (long-period transition period) are obtained from ASCE 7-05 maps and are used in formula 12.8-3 for determining the cut-off value of C_s below.

$$T_L = 4 \text{ sec}$$

ASCE 7-05 Figure 22-15

$$C_s = \frac{1.206}{\left(\frac{6.5}{1.0}\right)} = 0.186$$

The seismic response coefficient need not exceed:

$$C_s = \frac{S_{D1}}{T\left(\frac{R}{I}\right)} = \frac{0.692}{0.45\left(\frac{6.5}{1.0}\right)} = 0.236$$

For $T \leq T_L$

ASCE 7-05 Eq. 12.8-3

The seismic response coefficient shall not be less than:

$$C_s = 0.01$$

ASCE 7-05 Eq. 12.8-5

In addition, for structures located where S_1 is equal to or greater than 0.6g:

$$C_s = \frac{0.5S_1}{\left(\frac{R}{I}\right)} = \frac{0.5 \times 0.692}{\left(\frac{6.5}{1.0}\right)} = 0.053$$

ASCE 7-05 Eq. 12.8-6

$$\therefore V = 0.186W$$

For the flexible upper portion:

$$W = 2,520 \text{ k}$$

$$V = C_s W = 0.186 \times 2,520 = 469 \text{ k}$$

For the building as a whole using the same $R = 6.5$:

$$W = 5,152 \text{ k}$$

$$V = C_S W = 0.186 \times 5,152 = 958 \text{ k}$$

VERTICAL DISTRIBUTION OF FORCES

ASCE 7-05 §12.8.3

The biggest advantage of using 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 podium slab (second floor) is not inverted into the upper flexible portion of the structure. Hence, the base shear is based on the weight (W) of the structure that is above the podium slab.

The base shear must be distributed to each level. This is done as follows:

$$F_x = C_{VX} V$$

ASCE 7-05 Eq.12.8-11

$$C_{VX} = \frac{w_x h_x}{\sum_{i=1}^n w_i h_i^k}$$

ASCE 7-05 Eq.12.8-12

Where h_x is the average height at level i of the sheathed diaphragm in feet above the base, k is a distribution exponent related to the building period.

Since $T = 0.45$ second < 0.5 seconds, $k = 1$

Determination of F_x is shown in Table 3.

ASCE 7-05 §12.8.3

Note that the vertical distribution of seismic forces using the base of the structure at the first floor (Table 3A) produces overly conservative results due to the tall first floor of 22 feet. For illustrative purposes, the vertical distribution of seismic forces including the second floor (without the two-stage analysis) and using the R coefficient of 6.5 for the wood sheathed walls is included in Table 3B. However, this design example uses the vertical distribution of seismic forces using the base of the structure at the second floor (Table 3) using the two-stage analysis.

Table 3. Vertical distribution of seismic forces (with base at second floor)

Level	w_x (k)	h_x (ft)	$w_x h_x$ (k-ft)	$\frac{w_x h_x}{\sum w_i h_i}$ (%)	F_x (k)	$\frac{F_x}{w_x}$	F_{tot} (k)	$\frac{F_x}{A}$ (psf)
Roof	587	48	28,176	42.2	197.9	0.337	197.9	16.49
5th Floor	639	30	19,170	28.7	134.6	0.211	332.5	11.22
4th Floor	647	20	12,940	19.4	91.0	0.141	423.5	7.58
3rd Floor	647	10	6,470	9.7	45.5	0.070	469.0	3.79
Σ	2,520		66,756	100.0	469.0		469.0	

Where: A = area of the floor plate which is 12,000 square feet



Not used in this design example – for illustrative purposes only

Table 3A. Vertical distribution of seismic forces (with base at first floor) not including second floor in distribution

Level	w_x (k)	h_x (ft)	$w_x h_x$ (k-ft)	$\frac{w_x h_x}{\sum w_i h_i}$ (%)	F_x (k)	$\frac{F_x}{w_x}$	F_{tot} (k)	$\frac{F_x}{A}$ (psf)
Roof	587	60	35,220	36.3	170.2	0.290	170.2	14.18
5th Floor	639	42	26,868	27.7	130.0	0.203	300.2	10.83
4th Floor	647	32	20,704	21.3	99.9	0.154	400.1	8.33
3rd Floor	647	22	14,234	14.7	68.9	0.106	469.0	5.74
Σ	2,520		96,996	100.0	469.0		469.0	

Not used in this design example – for illustrative purposes only

Table 3B. Vertical distribution of seismic forces (with base at first floor) including second floor in distribution

Level	w_x (k)	h_x (ft)	$w_x h_x$ (k-ft)	$\frac{w_x h_x}{\sum w_i h_i}$ (%)	F_x (k)	$\frac{F_x}{w_x}$	F_{tot} (k)	$\frac{F_x}{A}$ (psf)
Roof	587	60	35,220	27.4	262.5	0.447	262.5	21.9
5th Floor	639	42	26,868	20.9	200.3	0.313	462.8	16.7
4th Floor	647	32	20,704	16.1	154.2	0.238	617.0	12.9
3rd Floor	647	22	14,234	11.1	106.3	0.164	723.3	8.86
2nd Floor	2,632	12	31,584	24.5	234.7	0.089	958.0	19.6
Σ	5,152		128,610	100.0	958.0		958.0	

3b. Assumption of Flexible Diaphragms

IBC §1613.6.1

ASCE 7-05 §12.3.1.1 allows wood diaphragms in one and two-family dwellings to be idealized as flexible diaphragms. Section 1613.6.1 of the IBC amends §12.3.1.1 of ASCE 7-05 by extending the use of flexible diaphragm design assumptions to most wood-framed structures, provided *all* of the following conditions are met:

1. Toppings of concrete are nonstructural and are a maximum of 1-1/2 inches thick.
2. Each line of vertical elements of the lateral force-resisting system complies with the allowable story drift.
3. Vertical elements of the lateral force-resisting system are light-frame structural walls sheathed with wood structural panels rated for shear resistance or steel sheets.
4. Portions of wood structural panel diaphragms that cantilever beyond the vertical elements of the lateral force-resisting system are designed in accordance with IBC §2305.2.5.

In this design example, the first condition is met since our structure does not exceed 1-1/2 inches of light-weight concrete.

Condition 2 is met since section 8c of this design example for drift check of typical shear wall complies with the allowable story drift.

Condition 3 is met since the design example includes wood-sheathed walls.

Condition 4 is met since the structure does not have any cantilever portions of the diaphragms.

3c. Flexible vs. Rigid Diaphragm Analysis

With the IBC's extension of assumption of flexible diaphragms for most wood structures, the engineer is left to use judgment regarding whether to use flexible or rigid diaphragm analysis to determine shear distributions to the shear walls. With the uniformity of shear wall lengths and spacing in the building's transverse direction (north-south), flexible diaphragm assumptions are certainly justifiable from a code compliance perspective.

Current industry standard is to consider rigidities of the shear walls in determining the horizontal distribution of lateral forces, either from an envelope method (highest from flexible diaphragm assumptions and rigid diaphragm assumptions) or by a distribution solely based on relative rigidities.

Some engineers designing structures similar to this design example will place shear walls at interior corridor walls (see Figure 2) and not place any lateral-resisting elements at the exterior walls (for longitudinal forces). This approach must utilize a rigid diaphragm design. Although some jurisdictions allow this type of design, it is not recommended from a performance perspective.

Engineers now have sophisticated design software available for designing structures of this type. With all that is available, many engineers still analyze "individual units." Some engineers perform a rigid diaphragm analysis and a few perform envelope solutions. These varying designs all get permitted by local building officials and there is not a lot of continuity in the design process even within cities.

For this case study, an "envelope" design was utilized.

3d. Flexible Upper Portion Redundancy Factor

The redundancy factor (p) for the flexible upper portion is 1.0. Both conditions of ASCE 7-05 §12.3.4.2 have been met, though designers are only required to meet one of the two provisions.

3e. Seismic Design of Rigid Lower Portion

Since the center of mass of the flexible upper structure coincides with the center of mass of the rigid lower portion, the entire structure mass can be lumped together and applied at the center of the podium's rigid diaphragm with the code-required eccentricities.

4. Code Requirements for use of Fire-Retardant-Treated Wood (FRTW)

IBC §602.3

Depending on the "code check" analysis performed by the architect, the proposed building may require a Type III construction. Type III construction requires the exterior walls to be constructed with noncombustible materials. As an exception to using noncombustible construction, section 602.3 of the IBC states that fire-retardant-treated wood (FRTW) framing complying with IBC §2303.2 is permitted for exterior wall assemblies with ratings of two-hours or less, basically allowing wood-frame construction for many structures where noncombustible materials are required.



The FRTW must comply with conditions in IBC sections 2303.2 and 2304.9.5 as follows:

1) LABELING

IBC §2303.2.1

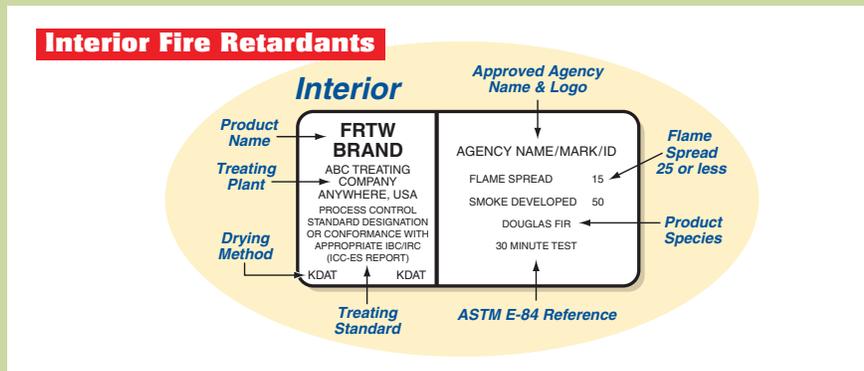
Fire-retardant-treated lumber and wood structural panels must be labeled and contain the following items:

- A. Identification mark of the approved agency
- B. Identification of the treating manufacturer
- C. Name of the fire-retardant treatment
- D. Species of the wood treated
- E. Flame spread and smoke-developed index
- F. Method of drying after treatment
- G. Conformance with appropriate standards

If exposed to weather, damp or wet conditions, it must also include the words “No increase in the listed classification when subjected to the Standard Rain Test.”

Sample labels for solid sawn framing lumber and plywood are shown in Figure 5. It should be noted that FRTW sheathing is only available in plywood; the amount of resins and waxes in oriented strand board (OSB) is too high for the treatment process.

Figure 5. Sample Labels for FRTW



2) STRENGTH ADJUSTMENTS

IBC §2303.2.2

The IBC requires that lumber design values be adjusted for the treatment and take into account the anticipated temperatures and humidity. Each manufacturer must publish the adjustment factors for service temperatures (not less than 80°F) and for roof-framing members (elevated temperatures). The adjustment factors vary from manufacturer to manufacturer, and should be obtained from the *ICC-ES Evaluation Report*. A sample of two manufacturers' strength adjustments are shown in Table 4.

Table 4. Sample strength reduction factors for FRTW

Design Property	FRTW Brand A			FRTW Brand B		
	Douglas Fir-Larch	Southern Pine	Spruce-Pine-Fir	Douglas Fir-Larch	Southern Pine	Spruce-Pine-Fir
F_b	0.97	0.91	0.88	0.90	0.89	0.89
F_t	0.95	0.88	0.83	0.87	0.92	0.87
F_c	1.00	0.94	0.94	0.91	0.94	0.91
F_v	0.96	0.95	0.93	0.94	0.95	0.94
E	0.96	0.95	0.94	0.98	0.98	0.98
F_c	0.95	0.95	0.95			
Fasteners	0.90	0.90	0.90	0.92	0.92	0.92

3) EXPOSURE TO WEATHER

IBC §2303.2.3

When FRTW is exposed to weather, damp or wet conditions, the identifying label needs to indicate "EXTERIOR."

4) FASTENERS

IBC §2304.9.5

Fasteners in preservative-treated and fire-retardant-treated wood shall be of hot-dipped zinc-coated galvanized steel, stainless steel, silicon bronze or copper. Fasteners in contact with treated wood need to meet this requirement. Rods in the tie-down system pass through an oversized hole in the wood and do not need to meet this requirement.

5) CUTTING AND NOTCHING

Treated lumber must not be ripped or milled as this will invalidate the flame spread. However, where FRTW joists or rafters are ripped for drainage conditions and FRTW plywood is placed on top of the ripped edge, this is considered acceptable.

End cuts and holes are usually not permitted; check the product evaluation report for requirements.

Some treated wood suppliers require the untreated wood to be shipped to their plant (from the framing contractor) for treatment, then shipped to the site.

Some suppliers stock most "sawn lumber" (2x, 3x and 4x) for immediate shipping.

Treatment adds about 50 percent to the cost of the material for interior and 80 percent for exterior applications.

5. Vertical Displacement (Shrinkage) in Multi-level Wood Framing

IBC §2303.7

Vertical displacement can be a significant problem in multi-level wood framing unless special considerations are accounted for during design and construction. Vertical displacement may be caused by one or a combination of the following:



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 §2304.3.3 requires that bearing walls supporting more than two floors and a roof be analyzed for shrinkage of the wood framing, and that possible adverse effects on the structure be satisfactorily demonstrated to the building official.

A free “shrinkage calculator” can be downloaded from the Western Wood Products Association website at: www2.wwpa.org.

The total shrinkage in wood-framed buildings can be calculated by summing the estimated shrinkage of the *horizontal* lumber members in walls and floors (wall plates, sills and floor joists). Most of the shrinkage is cross grain. The amount of shrinkage parallel to grain (length of studs) is approximately 1/40 of the shrinkage perpendicular to grain (cross grain) and can be neglected.

This case study illustrates two methods for determining the amount of wood shrinkage:

5a. Comprehensive Shrinkage Estimation

For a dimensional change with the moisture content limits of 6 to 14 percent, the formula is:

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

Where:

- S = shrinkage (in inches)
- D_i = initial dimension (in inches)
- C_T = dimension change coefficient, tangential direction
- $C_T = 0.00319$ for Douglas Fir-Larch
- $C_T = 0.00323$ for Hem-Fir
- $C_T = 0.00263$ for Spruce-Pine-Fir
- M_F = final moisture content (%)
- M_i = initial moisture content (%)

The formulas are from the *Wood Handbook: Wood as an Engineering Material and Dimensional Stability of Western Lumber Products*.

For a dimension change with moisture content limits greater than 6 to 14 percent where one of the values is outside of those limits, the formula is:

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

Where:

- S = shrinkage (in inches)
- D_i = initial dimension (in inches)
- S_T = tangential shrinkage (%) from green to oven dry
- $S_T = 7.775$ for Douglas Fir-Larch
- M_F = final moisture content (%)
- M_i = initial moisture content (%)

The final moisture content (M_F) for a building is referred to as the equilibrium moisture content (*EMC*). The final equilibrium moisture content can be higher in coastal areas and lower in inland or desert areas. These ranges are normally from 6 to 15 percent (low to high). The Western Wood Products Association

has downloadable documents listing EMC for all major U.S. cities for each month of the year. At the web address after login, click "Shrinkage" followed by "EMC Charts" (free user login with password is required): www2.wvpa.org/Shrinkage/EMCUSLocations1997/tabid/888/Default.aspx

The EMC can be calculated with this formula:

$$EMC = \frac{1800}{W} \left[\frac{KH}{1-KH} + \frac{(K_1KH + 2K_1K_2K^2H^2)}{(1 + K_1KH + K_1K_2K^2H^2)} \right]$$

Where:

$$W = 330 + (0.452)T + (0.00415)T^2$$

$$K = 0.791 + (0.000463)T - (0.000000844)T^2$$

H = relative humidity (%)

$$K_1 = 6.34 + (0.000775)T - (0.0000935)T^2$$

$$K_2 = 1.09 + (0.0284)T - (0.0000904)T^2$$

T = temperature (°F)

For this design example, a final moisture content M_F (EMC) of 12.0 percent is used.

Project specifications call for all top plates and sill (sole) plates to be Douglas Fir-Larch "kiln dried" (KD) or "surfaced dried" (S-Dry). Kiln dried lumber or surfaced dried has a maximum moisture content of 19 percent and an average of 15 percent.

It might be more realistic to use a lower number than 19 percent in the calculation so as to not overestimate the shrinkage.

Typical floor framing has a 4x4 top plate and a 2x4 sole plate (see Figure 6).

Find the individual shrinkage of the two members:

DETERMINE SHRINKAGE OF 4X4 TOP PLATE:

Since our initial MC (M_i) is 19 percent and the final MC (M_F) is 12 percent, the equation is:

$$S = \frac{D_i (M_F - M_i)}{\frac{30 (100)}{S_T} - 30 + M_i} = \frac{3.5 (12 - 19)}{\frac{30 (100)}{7.775} - 30 + 19} = -0.065 \text{ inch}$$

The final size of our 4x4 is:

$$3.5 - 0.065 = 3.435 \text{ inches}$$

5b. Quick Shrinkage Estimation

A close approximation that is much more easily used to determine amount of shrinkage is:

$$S = CD_i (M_F - M_i)$$

Where:

S = shrinkage (inches)

C = average shrinkage constant

$C = 0.002$

M_F = final moisture content (%)

M_i = initial moisture content (%)



DETERMINE SHRINKAGE OF 4X4 TOP PLATE:

Since our initial MC (M_i) is 19 percent and the final MC (M_f) is 12 percent, the equation is:

$$S = CD_i (M_f - M_i) = 0.002 \times 3.5 (12-19) = -0.049 \text{ inch}$$

The final size of our 4x4 is:

$$3.5 - 0.049 = 3.451 \text{ inches}$$

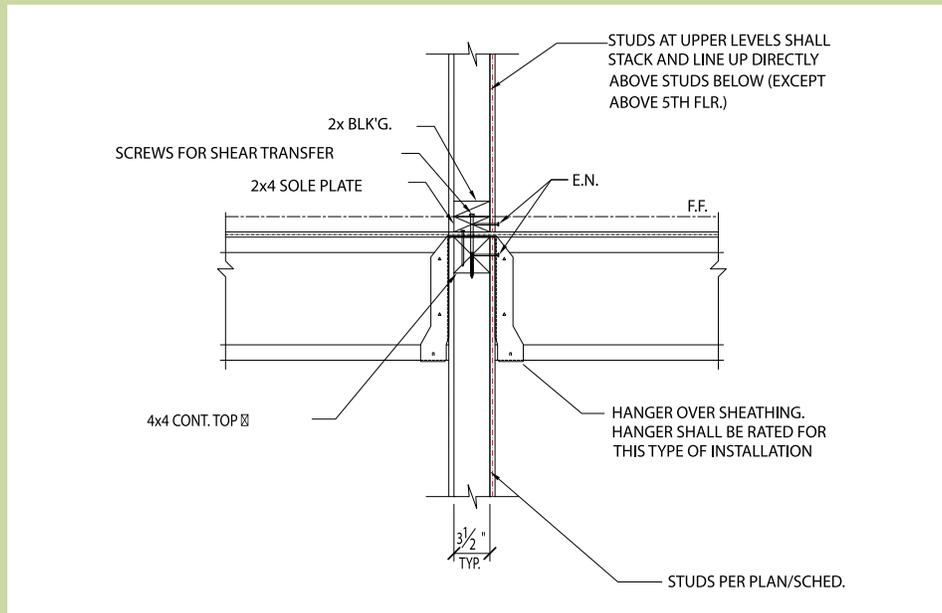
Note that this quick estimation is within 0.5 percent of the actual calculated dimension of 3.435 inches using the comprehensive formulas.

$$S = CD_i (M_f - M_i) = 0.002 \times 1.5 (12-19) = -0.021 \text{ inch}$$

DETERMINE SHRINKAGE OF 2X4 SOLE PLATE:

$$S = CD_i (M_f - M_i) = 0.002 \times 1.5 (12 - 19) = -0.021 \text{ inch}$$

Figure 6. Typical Floor Framing at Wall



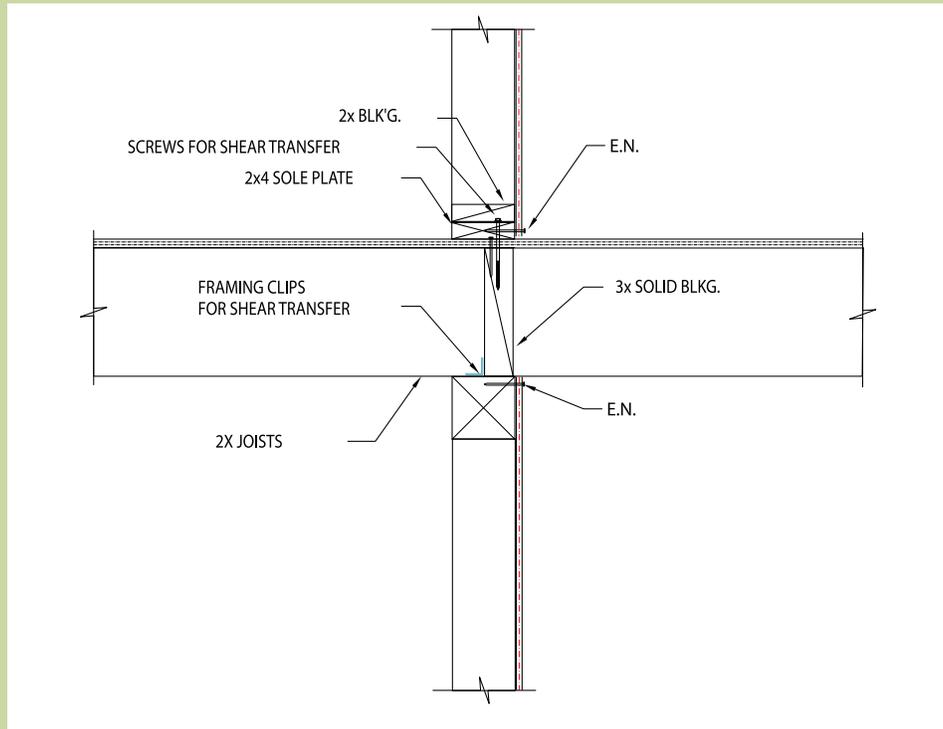
NOTES FOR FIGURE 6:

1. Blocking above the sole plate is to provide a nailing surface for the finishes. An alternative detail could use two sole plates, but this will increase shrinkage amounts for the building.
2. Web stiffeners at joist hangers may be required depending on joist size and manufacturer.
3. 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).
4. This detail uses a 4x4 top plate. Use of double 2x plates (not depicted) is also common.

Total shrinkage per floor level with the 4x4 top plate and 2x4 sole plate:

$$S = 0.049 + 0.021 = 0.07 \text{ inch}$$

Figure 6A. Typical Platform Floor Framing at Wall Using Sawn Joists



EXAMPLE CALCULATION

DETERMINE SHRINKAGE OF SAWN JOISTS WITH PLATFORM FRAMING (Figure 6A):

$$S = CD_i (M_F - M_i) = 0.002 \times 11.25 (12-19) = -0.158 \text{ inch}$$

Total shrinkage per floor level with the 4x4 top plate, 2x12 sawn joists and 2x4 sole plate:

$$S = 0.049 + 0.021 + 0.158 = 0.228 \text{ inch}$$

SETTLEMENT UNDER CONSTRUCTION GAPS (Consolidation):

Small gaps can occur between plates and studs, caused by (among other things) mis-cuts (short studs) and the lack of square-cut ends. These gaps can account for up to 1/8 inch per story, where “perfect” workmanship would be 0 inches and a more “sloppy” workmanship would be 1/8 inch. This case study factors in gaps of 1/10 inch.

DEFORMATION UNDER SUSTAINED LOADING:

Wood beams that support walls can creep from the sustained loading. The “rate” of creep can be higher for beams that are loaded while “drying” under load, because the modulus of elasticity is lower for higher moisture contents. Appendix F of the NDS provides commentary related to creep in wood and recommends a (creep) deflection amplification factor of between 1.5 and 2.0 for computing deflections under sustained loads.



Table 5. Vertical Displacements

Level	Vertical Displacement		Design Displacement (in.)
	Per Floor	Cumulative	
5th Floor	0.170	0.68	3/4
4th Floor	0.170	0.51	5/8
3rd Floor	0.170	0.34	3/8
2nd Floor	0.170	0.17	1/4

Where: Shrinkage of 0.07 inch + settlement of 0.10 inch = 0.170 inch

METHODS TO REDUCE VERTICAL DISPLACEMENT:

1. Use kiln-dried plates (MC < 19%) or even MC15 (MC < 15%) lumber or engineered lumber for plates.
2. Consider a single top plate instead of double top plate.
3. Consider balloon framing or a modified balloon framing.
4. Place floor joists in metal hangers bearing on beams or top plates instead of bearing on the top plates.
5. The site storage of the material stock can negate all design and planning when the material is not properly stored on the site. Lumber should be kept away from moisture sources and rain.

METHODS TO ACCOUNT FOR VERTICAL DISPLACEMENT:

1. Use continuous tie-down systems with shrinkage compensating devices in shear walls.
2. Architectural finish details near the floor lines need to account for vertical displacement.
3. Provide a 1/8-inch gap between window and door tops to the framing lumber.

6. Shear Wall Design Example

This design example features a four-story “segmented shear wall” with an out-to-out length of 29.0 feet and floor-to-floor heights of 10.0 feet. NDS-05 SDPWS §4.3.5.1 categorizes this wall type as having full-height wall segments with aspect ratio limitations of NDS-05 SDPWS §4.3.4 applying to each full height segment.

CHECK H/W RATIO FOR SHEAR WALL SEGMENTS:

Segment height = 10.0 feet

Segment width = 29.0 feet

$$h/w = \frac{10.0}{29.0} = 0.34 < 2.0 \Rightarrow \text{okay}$$

6a. Determination of Lateral Loads to Shear Wall

IBC §1613.6.1

The structure used in this design example has interior shear walls located at every other wall between hotel guest units. The walls are spaced at 13 feet o.c., with the depth of the building equal to 65 feet.

Based on an “envelope” design using flexible diaphragm assumptions and a rigid diaphragm analysis, the critical forces to the interior shear wall (Figure 7) are shown in Table 6.

Figure 7. Typical Interior Shear Wall Elevation

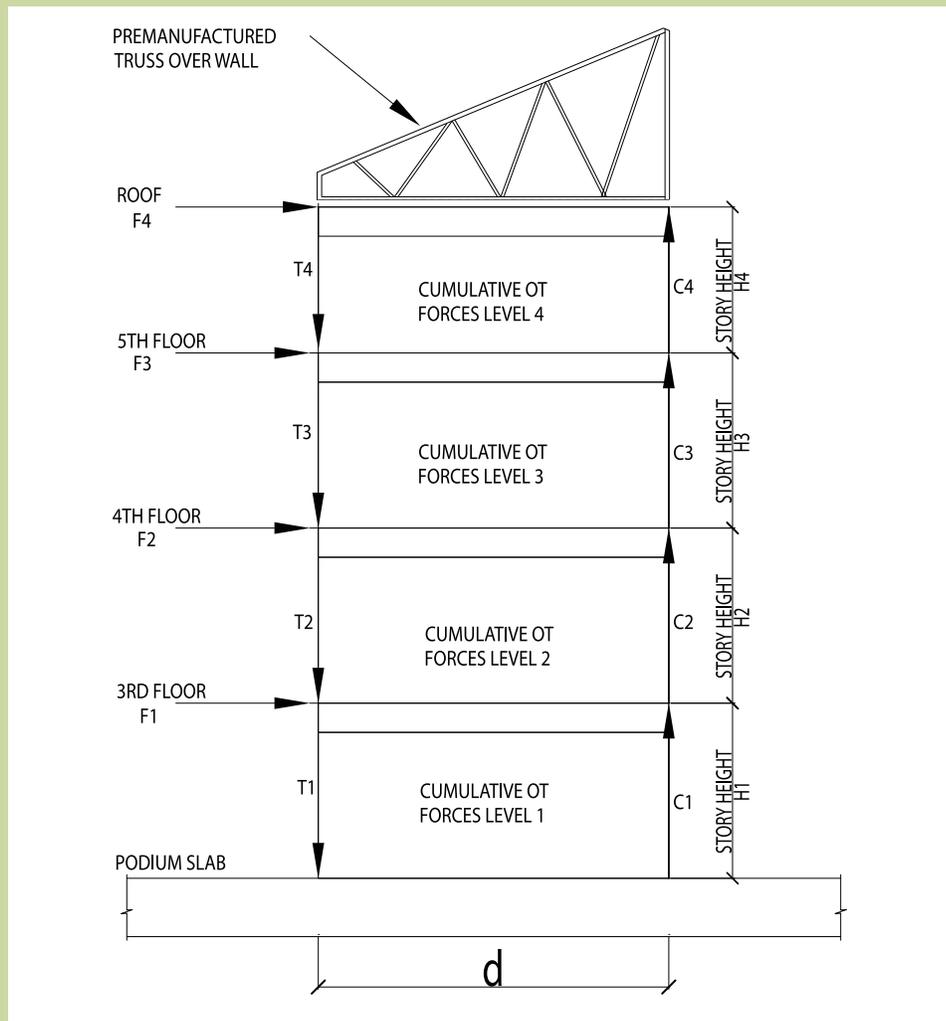


Table 6. Distribution of seismic forces for both shear walls

Level	Designation	F_{Total} (lb)
Roof	F_4	13,935
5th Floor	F_3	23,415
4th Floor	F_2	29,820
3rd Floor	F_1	33,045



6b. Determination of Shear Wall Sheathing and Nailing

The shear wall to be designed will use 15/32-inch Structural I rated sheathing using 10d common nails with a minimum penetration of 1-1/2 inches into the framing members.

A 2x4 sole plate (sill plate) will be used at the base of the shear wall. There is a discrepancy between the IBC and the SDPWS on 3x nominal framing requirements:

Footnote e in IBC Table 2306.4.1 reads:

Framing at adjoining panel edges shall be 3 inches nominal or wider, and nails shall be staggered where nails are 2 inches o.c.

SDPWS section 4.3.7.1, item 3c, states that:

3x nominal framing is required when the required nominal shear capacity exceeds 700 plf in Seismic Design Category (SDC) D, E or F.

Table 7. Determination of shear wall nailing

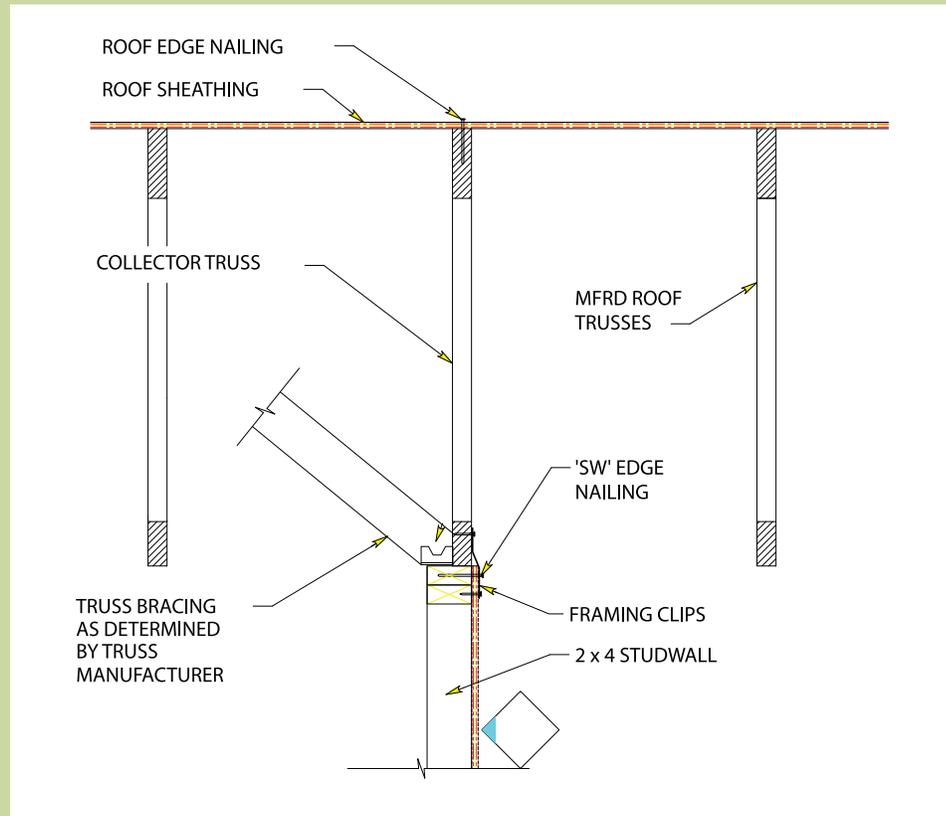
Designation	F_{Total} (lb)	Wall Length l (ft)	ASD Design			
			$V = \frac{F_{Total}(0.7)}{l}$ (plf)	Wall Sheathed 1 or 2 sides	Allowable Shear ^a (plf)	Fastener Edge Spacing ^b
F_4	13,935	29.0	340	1	340	6
F_3	23,415	29.0	565	1	665	3
F_2	29,820	29.0	720	1	870	2 ^c
F_1	33,045	29.0	800	1	870	2 ^c

- a. Allowable shear values are obtained by taking the nominal unit shear capacities in NDS-05 SDPWS Table 4.3A and dividing by the ASD reduction factor of 2.0.
- b. A 2x4 sole plate (sill plate) will be used at the base of walls (see Figure 6) with the exception of the bottom wall (on the podium slab) which requires a 2x sill plate. For 10d common nails spaced at 2 inches o.c., the nails are staggered. From a constructability standpoint (framer bent over to install nails) and for improved structural performance (larger edge distance), the use of a 3x sole plate is recommended.
- c. Where fastener spacing is 2 inches o.c., some engineers may use sheathing on both sides of the wall with fasteners spaced at 4 inches o.c. for better performance and less drift.

6c. Shear Transfer at Top of Wall

The shear transfer at the top of the fifth floor wall is achieved with framing clips located at the bottom of the roof truss chord (see Figure 8). The collector truss is specified based on the vertical and lateral loading combinations including lateral collector loads.

Figure 8. Shear Transfer at Top of Wall



NOTES FOR FIGURE 8:

1. Diaphragm shear at collector truss:

$$v = \frac{27,870\text{lb} \times 0.7}{65.0\text{ ft}} = 300\text{ plf}$$

where 27,870 lb is from Table 6.

2. Diaphragm nailing needs to accommodate one row of fasteners (boundary nailing). Using 8d common nails with 15/32-inch rated sheathing spaced at 4 inches o.c. with 2x nominal framing members, the allowable shear is 360 plf (IBC Table 2306.3.1).
3. The collector truss should be looked at for possible uplift forces and strapping from truss to shear wall ends may be necessary.
4. Area separation walls may need a special truss with "flat members" or a doubled truss.



6d. Shearwall Cumulative Overturning Forces

When designing overturning forces in multi-level structures, shear and the respective overturning forces due to seismic (or wind) must be carried down to the foundation, or in this design example the podium slab, by the boundary studs and continuous tie-down system. These forces are cumulative over the height of the building, and shear forces applied at the upper levels will generate much larger base overturning moments than if the same shear forces were applied at the lower story.

The overturning forces for the shear wall (Figure 7) can be obtained by summing forces about the base of the wall for the level being designed.

Cumulative overturning force for the fifth floor level:

$$M_{ot} = F_4 (H_4)$$

Cumulative overturning force for the fourth floor level:

$$M_{ot} = F_4 (H_4 + H_3) + F_3(H_3)$$

Cumulative overturning force for the third floor level:

$$M_{ot} = F_4 (H_4 + H_3 + H_2) + F_3(H_3 + H_2) + F_2(H_2)$$

Cumulative overturning force for the second floor level:

$$M_{ot} = F_4 (H_4 + H_3 + H_2 + H_1) + F_3(H_3 + H_2 + H_1) + F_2(H_2H_1) + F_1(H_1)$$

In shear walls with continuous tie-down systems, the overturning resistance in the shear wall is resisted by the posts and/or end studs resisting the compression forces and the tension rods resisting the tension forces.

In shear walls with conventional holdown systems, the overturning resistance in the shear wall is resisted by the posts and/or end studs resisting the compression forces and the tension forces.

6e. Load Combinations using 2006 IBC

IBC Section 1605.3.2 has alternative basic load combinations to ASCE 7-05. For allowable stress design, the earthquake load combinations are:

$$D + L + S + \frac{E}{1.4} \quad \text{IBC Eq.16-20}$$

Since S is not present, the simplified load combination is:

$$D + L + \frac{E}{1.4}$$

Where E = the horizontal seismic force (F):

$$0.9D + \frac{E}{1.4} \quad \text{IBC Eq.16-21}$$

6f. Load Combinations using ASCE 7-05

§12.4.2.3

Per Section 12.4.2.3, the following load combinations shall be used for basic combinations for allowable stress design:

$$(1.0 + 0.14S_{DS})D + H + F + 0.7pQ_E \quad \text{ASCE 7-05 Eq. 5}$$

$$(1.0 + 0.105S_{DS})D + H + F + 0.525pQ_E + 0.75L + 0.75(L_r \text{ or } S \text{ or } R) \quad \text{ASCE 7-05 Eq. 6}$$

$$(0.6 - 0.14S_{DS})D + 0.7pQ_E + H \quad \text{ASCE 7-05 Eq. 8}$$

Where the dead load D is increased (or decreased) for vertical accelerations by the S_{DS} coefficient.

Since H , F , S and R are not present, the simplified load combinations are:

$$(1.0 + 0.14S_{DS})D + 0.7pQ_E \quad \text{ASCE 7-05 Eq. 5}$$

$$(1.0 + 0.105S_{DS})D + 0.525pQ_E + 0.75L + 0.75L_r \quad \text{ASCE 7-05 Eq. 6}$$

$$(0.6 + 0.14S_{DS})D + 0.7pQ_E \quad \text{ASCE 7-05 Eq. 8}$$

Where Q_E = the horizontal seismic force F .

ASCE 7-05 §12.4.2.1

$$0.105S_{DS} = 0.105 (1.206) = 0.13$$

$$0.14S_{DS} = 0.14 (1.206) = 0.17$$

6g. Shearwall Chord (Boundary) Members

The vertical members at the end of the shear walls are the walls' chords (boundary members). As in a diaphragm, the chords resist flexure and the sheathing (web) resist the shear. The overturning moment is resolved into a T-C couple creating axial tension and compression forces. When considering only the horizontal component of the seismic forces, the tension and compression forces are equal and opposite. The overturning compressive force is determined by dividing the overturning moment by the distance "d" between the center of the tension rod and the center of the compression posts (Figure 9). However, in most designs, the size and number of chords (boundary members) change from story to story as shown in Figures 10 and 11, which can necessitate iterations to derive the actual distance "d." Many engineers will take a "conservative average" distance "d" and use the same value for all cases to minimize iterations.

Figure 9 illustrates multiple boundary members that are common to multi-level wood-framed shear walls.

The axial loads to the bearing wall and boundary members are determined from the following loads:

Dead loads:

$$W_{Roof} = (28.0 \text{ psf})(2.0 \text{ ft}) = 56.0 \text{ plf}$$

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

$$W_{Wall} = (10.0 \text{ psf})(10.0 \text{ ft}) = 100.0 \text{ plf}$$

Live loads:

$$W_{Roof} = (20.0 \text{ psf})(2.0 \text{ ft}) = 40.0 \text{ plf}$$

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

Dead + live loads:

$$W_{Roof} = (28.0 \text{ psf} + 20.0 \text{ psf})(2.0 \text{ ft}) = 96.0 \text{ plf}$$

$$W_{Floor} = (30.0 \text{ psf} + 40.0 \text{ psf})(13.0 \text{ ft}) = 910 \text{ plf}$$

$$W_{Wall} = 10.0 \text{ psf} (10.0 \text{ ft}) = 100.0 \text{ plf}$$

(1.2 + 0.2 S_{DS}) dead + live loads:

Per section 12.4.2.3 of ASCE 7-05, the load factor on L is permitted to be 0.5 since the live load is equal to or less than 100 psf and not of public assembly. The 0.5 factor will be used in the live load determinations below:

$$W_{Roof} = ((1.4 \times 28.0 \text{ psf}) + (0.5 \times 20.0 \text{ psf}))(2.0 \text{ ft}) = 98.5 \text{ plf}$$

$$W_{Floor} = ((1.4 \times 30.0 \text{ psf}) + (0.5 \times 40.0 \text{ psf}))(13.0 \text{ ft}) = 806 \text{ plf}$$

$$W_{Wall} = 1.4 \times 10.0 \text{ psf}(10.0 \text{ ft}) = 140.0 \text{ plf}$$



Figure 9. Shear Wall Elevation with Distance D

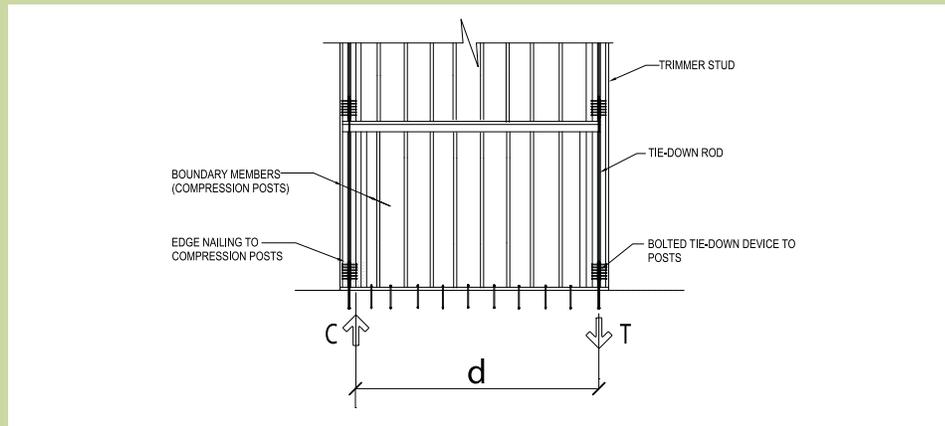
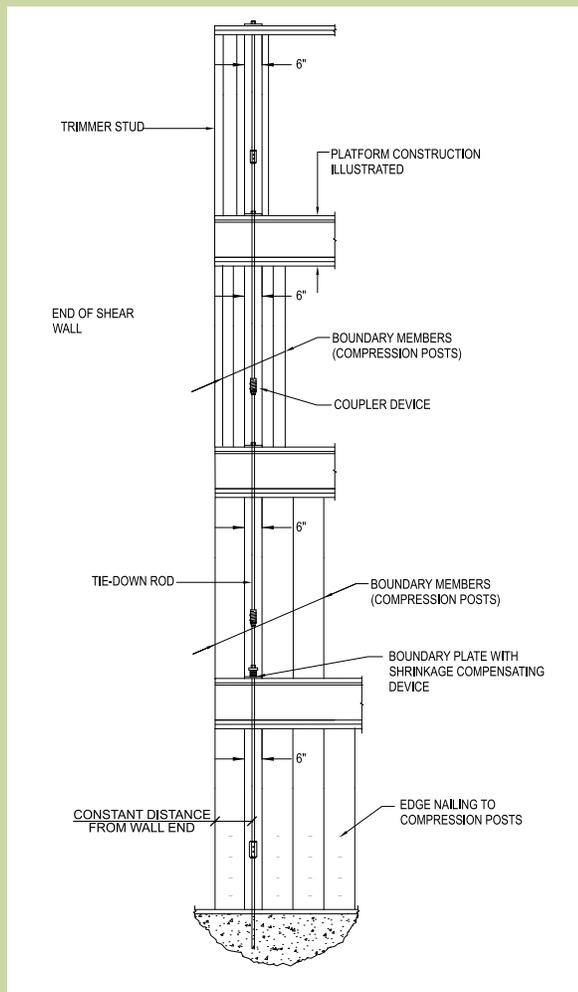


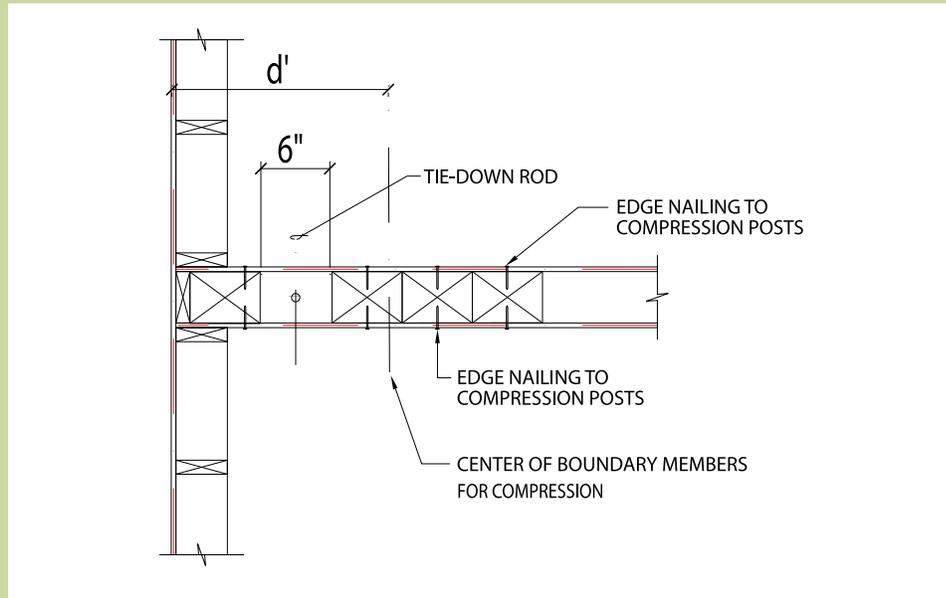
Figure 10. Example Elevation of Shear Wall Boundary Members



NOTES FOR FIGURES 10 AND 11:

1. Some continuous rod systems favor centering the rod between symmetrical amounts of posts (concentric with the tension rod), while other continuous rod systems favor an asymmetrical orientation of posts (shown in Figures 10 and 11).
2. See Figures 13, 14 and 15 for comments on blocking at the floor framing.

Figure 11. Example Plan Section at Boundary Members



For ASD compression on the chord members, the alternate basic load combination is used.

$$D + L + \frac{E}{1.4} \quad \text{IBC Eq. 16-20}$$

For strength compression on the chord members, the ASCE 7-05 seismic load combination will be used. The strength compression loads are used later in this example to determine the shear wall deflection at strength loads (sill plate crushing). Per ASCE 7-05 sections 12.8.6 and 12.12.1, strength level forces are required for the determination of shear wall deflections.

$$(1.2 + 0.2S_{DS})D + \rho Q_E + L + 0.2S$$

Where:

$$\rho Q_E = E$$

Since S is not present, the simplified load combination is:

$$(1.2 + 0.2S_{DS})D + L + E$$

Where:

$$(1.2 + 0.2S_{DS}) = (1.2 + 0.2 \times 1.206) = 1.4$$

ASCE 7-05 §12.4.2.3

$$E = \frac{M_{OT}}{d}$$



Table 8. Determination of shear wall chord member forces

Level	M_{OT} (ft-k)	ASD P_{D+L} (k)	d' (ft)	d (ft)	ASD Demand Compression	Strength Demand Compression
					$C = \frac{M_{OT}}{1.4d} + P_{D+L}$ (k)	$\frac{M_{OT}}{d} + (1.2 + 0.2S_{DS})D + L$ (k)
Roof	139.35	0.38	0.98	27.04	4.06	5.62
5th Floor	373.50	2.36	0.98	27.04	12.23	16.13
4th Floor	671.70	7.00	1.58	26.44	25.15	32.13
3rd Floor	1,002.1	10.19	1.58	26.44	37.26	47.62

Where: $P_{D+L} = w(d')^2$

FOR ASD DEMAND (see section 6g):

$$P_{D+L \text{ Roof}} = (96 \text{ plf} + 100 \text{ plf})(0.98 \times 2) = 0.384 \text{ k}$$

$$P_{D+L \text{ 5thFloor}} = (910 \text{ plf} + 100 \text{ plf})(0.98 \times 2) + P_{\text{Roof}} = 2.36 \text{ k}$$

$$P_{D+L \text{ 4thFloor}} = ((910 \text{ plf} + 100 \text{ plf})^2 + (96 + 100))(1.58 \times 2) = 7.00 \text{ k}$$

$$P_{D+L \text{ 3rdFloor}} = ((910 \text{ plf} + 100 \text{ plf})^3 + (96 + 100))(1.58 \times 2) = 10.19 \text{ k}$$

FOR STRENGTH DEMAND (see section 6g):

$$(1.2 + 0.02S_{DS})D + L = 1.4D + L$$

$$P_{D+L \text{ Roof}} = (98.5 \text{ plf} + 140 \text{ plf})(0.98 \times 2) = 0.467 \text{ k}$$

$$P_{D+L \text{ 5thFloor}} = (806 \text{ plf} + 140 \text{ plf})(0.98 \times 2) + P_{\text{Roof}} = 2.32 \text{ k}$$

$$P_{D+L \text{ 4thFloor}} = ((806 \text{ plf} + 140 \text{ plf})^2 + (98.5 + 140))(1.58 \times 2) = 6.73 \text{ k}$$

$$P_{D+L \text{ 3rdFloor}} = ((806 \text{ plf} + 140 \text{ plf})^3 + (98.5 + 140))(1.58 \times 2) = 9.72 \text{ k}$$

Table 9. Determination of shear wall chord members

Level	Chord Posts	Total Area	l_e (ft)	C_f	C_p	Bearing Cap. (kips)	ASD Demand (kips)	Stability Capacity (kips)	D/C Ratio
Roof	4-3x4	35.0	9.625	1.15	0.163	21.88	4.06	15.79	0.26
5th Floor	4-3x4	35.0	9.625	1.15	0.163	21.88	12.23	15.79	0.77
4th Floor	4-4x8	101.5	9.625	1.05	0.187	63.44	25.15	46.48	0.54
3rd Floor	4-4x8	101.5	9.625	1.05	0.187	63.44	37.26	46.48	0.80

Notes:

1. $C_d = 1.6$

2. Bearing capacity (on sole plate) = $F'_{c\perp} AC_b$

3. Column bearing factor $C_b = 1.0$

4. Column stability factor $C_p = \frac{1 + (F_{cE}/F_c^*)}{2c} - \sqrt{\left[\frac{1 + (F_{cE}/F_c^*)}{2c} \right]^2 - \frac{F_{cE}/F_c^*}{c}}$

5. Column stability capacity = $F_c C_D C_F C_p A$

Example for four 4x8 posts: $4 \times 11.62 = 46.48$ kips

6. The typical interior stud wall is framed with 4-inch nominal framing studs.

7. Interior bearing walls for this design example are non-rated and, as such, would not require the reduction in allowable loads.

6h. Example Compression Member Capacity Determination

4X8 POST – DOUGLAS FIR-LARCH NO. 1:

Where:

$$A = 25.375 \text{ in}^2$$

$$C_D = 1.6$$

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

$$d_1 = 3.5 \text{ in.}$$

The following coefficients for C_m and C_t are not referenced in the NDS formulas (for simplicity).

$$C_m = 1.0$$

$$C_t = 1.0$$

$$K_e = 1.0$$

The members' span between the top of the 2x4 sill plate and the underside of the 4x4 top plate (see Figure 5).

$$l = 9.52 \text{ feet}$$

$$l_{ei} = 9.52 \times 12 = 114 \text{ inches}$$

$$l_{ei} / d_1 = 114 / 3.5 = 32.64$$

Slenderness is controlled by the minor axis and is thus used in the F_{cE} calculation.

Compression parallel to grain:

$$F'_c = F_c C_D C_F C_p$$

$$F_c^* = F_c C_D C_F = 1,500 \times 1.6 \times 1.05 = 2,520 \text{ psi}$$

$$C_p = \frac{1 + (F_{cE}/F_c^*)}{2c} - \sqrt{\left[\frac{1 + (F_{cE}/F_c^*)}{2c} \right]^2 - \frac{F_{cE}/F_c^*}{c}} = 0.1817 \quad \text{NDS Eq. 3.7-1}$$



Where:

$$c = 0.8$$

$$F_{cE} = \frac{0.822E_{min}}{(l_e/d)^2} = \frac{0.822 \times 620,000}{32.64^2} = 478.4 \text{ psi}$$

NDS Eq. 3.7-1

$$F_{cE}/F_c = \frac{478.4}{2,520} = 0.1898$$

$$F'_c = F_c C_D C_F C_P = 1,500 \times 1.6 \times 1.05 \times 0.1817 = 458 \text{ psi}$$

FOR A 4X8 POST:

$$P_{allow} = A \times F'_c = 25.375 \times 458 = 11,620 \text{ lbs}$$

Compression perpendicular to grain:

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

FOR A 4X8 POST:

$$P_{allow} = A \times F'_{c\perp} = 25.375 \times 625 = 15,860 \text{ lbs}$$

6i. Determine Resisting Moments and Uplift Forces

The resisting moment M_R is determined from the following dead loads:

$$W_{Roof} = 28.0 \text{ psf (2.0 ft)} = 56.0 \text{ plf}$$

$$W_{Floor} = 30.0 \text{ psf (13.0 ft)} = 390.0 \text{ plf}$$

$$W_{Wall} = 10.0 \text{ psf (10.0 ft)} = 100.0 \text{ plf}$$

Tables 10 and 10A illustrate the differences in ASD uplift values that can be calculated from using the ASCE 7-05 formula and the alternate IBC formula. For this case study, the ASCE 7-05 equation in Table 11 is used.

Table 10. Determine shear wall uplift forces using ASCE 7-05 load combinations

Level	M_R (ft-lb)	d (ft)	Strength		ASD Uplift	Differential Load Per Floor (lbs)
			M_{OT} (ft-lb)	$M_R(0.6-0.14S_{DS})^{\dagger}$ (ft-lb)	$\frac{(M_{OT} \times 0.7) - (0.6 - 0.14S_{DS})M_R}{d}$ (lb)	
Roof	65,600	27.04	139,350	28,210	2,565	0
5th Floor	271,645	27.04	373,500	116,805	5,350	2,785
4th Floor	477,690	26.44	671,700	205,405	10,015	4,665
3rd Floor	683,735	26.44	1,002,100	294,005	15,410	5,395

[†]Where $(0.6 - 0.14S_{DS}) = (0.6 - 0.14 \times 1.206) = 0.43$

Table 10A. Determine shear wall uplift forces using IBC alternate load combinations

Level	M_R (ft-lb)	d (ft)	Strength	ASD Uplift	Differential Load Per Floor (lbs)
			M_{OT} (ft-lb)	$\left(\frac{M_{OT}}{1.4} \right) - 0.9M_R$ d (lb)	
Roof	65,600	27.04	139,350	1,500	0
5th Floor	271,645	27.04	373,500	825	-675
4th Floor	477,690	26.44	671,700	1,885	2,560
3rd Floor	683,735	26.44	1,002,100	3,800	1,915

Notes for Table 10A: A “negative” differential load is a result of a higher resisting moment and occurs at a lower level than above.

6j. Shearwall Tie-down System Components

TIE-DOWN RODS

Tie-down rods are usually made from A36/A307 steel. This is called standard rod strength. Unless marked, rods should be considered standard rod strength. High-strength rods are A449 or A193-B7 and are usually marked on the end with an embossed stamp, though some rod manufacturers stamp the rod grade on the side. If the rod is stamped at the end and is cut, it needs to be re-marked. High-strength rods should have special inspection to confirm the rod type since the ends of these rods may be embedded into a coupler where the marks cannot be seen after installation. It should be noted that high-strength rods are not weldable. Proprietary systems have special rod colors and markings on the sides. The rods and tie-down systems are not proprietary, but the manufactured components are.

TIE-DOWN ELONGATION

Tie-down rod elongation is computed between bearing plates (restraints). This design example has bearing plates located at each floor. Table 11 computes the rod capacities and elongations (per floor) between the bearing plates.

Table 11. Determine rod sizes, capacities and elongations

Level	Plate Height (ft)	Tension Demand (kips)	Rod Dia. d (in)	Eff. Dia. d_e (in)	A_g (in ²)	A_e (in ²)	F_u (ksi)	F_y (ksi)	Allow Rod Capacity	Rod Elong. (in)
									$75 \cdot F_u \cdot A_g / 2$ (kips)	
Roof	10.0	2.56	0.625	0.560	0.307	0.226	60	43	6.91	0.047
5th Floor	10.0	5.35	0.625	0.560	0.307	0.226	60	43	6.91	0.098
4th Floor	10.0	10.0	0.625	0.560	0.307	0.226	120	92	13.82	0.183
3rd Floor	10.0	15.4	0.875	0.796	0.601	0.462	120	92	27.05	0.138



Notes:

1. Tension demand (ASD uplift) values are computed in Table 10.

2. Rod area:
$$A_g = \frac{3.14d^2}{4}$$

3. Net tensile area A_e is from AISC Table 7-18.

4. Standard rod is ASTM A36 rod with minimum $F_u = 58 \text{ ksi}$, $F_y = 36 \text{ ksi}$.

High-strength rod is ASTM A193 rod with minimum $F_u = 125 \text{ ksi}$, $F_y = 105 \text{ ksi}$ for rods up to 2-1/2 inches in diameter and A449 rod with minimum $F_u = 120 \text{ ksi}$, $F_y = 105 \text{ ksi}$ for rods up to 1 inch in diameter then drops to $F_y = 105 \text{ ksi}$ for larger rods.

5. Allowable rod capacity for the *AISC Steel Construction Manual Thirteenth Edition* is:

$$\frac{0.75F_uA_g}{2}$$

6. Rod elongation:
$$\Delta = \frac{PL}{A_eE}$$

Where:

Δ = the elongation of the rod in inches

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

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

$E = 29,000 \text{ ksi}$

A_e = the effective area of the rod in square inches

When smooth rods are used, the area is equal to the gross area (A_g). When threaded (all-thread) rods are used, the area is equal to the tension area (A_e) of the threaded rod. Since many of the proprietary systems that have smooth rods have long portions threaded at the ends, it is recommended that A_e be used when calculating rod elongation.

7. Rod elongation is based on using the effective area (A_e) and the following lengths:

a. For the first level, the anchor bolt is projecting 4 inches above the foundation (height of coupler nut to anchor bolt at podium slab).

b. For the framed floors, the rod from below is projecting 6 inches above the sole plate.

ROD COUPLERS

Couplers are used to connect the rods. Couplers can either be straight or reducing and can be supplied in different strengths or grades. Couplers for high-strength rods need to be of high-strength steel and are marked with notches or marks on the coupler. For a rod to develop its full strength, the rod must be a set amount (usually the depth of a standard nut). It is recommended that, when couplers are used, they have "pilot" or "witness" holes in the side so the threads of the rods can be witnessed in the holes to ensure proper embedment.

Reducing couplers are used when the rod size is changed. In reducing couplers, the size of the threading changes at the middle of the coupler device. It is intended that the rods be embedded until they bottom out at the center of the coupler. If the rods are installed in this fashion, "witness" holes will not be necessary; however, it is recommended that couplers with witness holes be used so that proper installation can be confirmed by an inspector. Reducing couplers should have the same notches and identifying marks as straight couplers when used with high-strength rods.

BEARING PLATES

Bearing plates transfer the tension load from the structure, the sole plate or the top plates into the rod (see Figure 14). Premanufactured bearing plates are usually identified by paint color or by a number marked on the plate. However, paint colors or unpainted plates vary among different rod system manufacturers.

Table 12. Determine bearing plate sizes and capacities

Level	Bearing Plate					Bearing Factor	Bearing Load	Allowable Capacity
	Width	Length	Thickness	Hole dia.	A_{Brg}			
	(in)	(in)	(in)	(in)	(in ²)	C_b	(kips)	(kips)
Roof	3.0	5.5	0.6	0.6875	15.788	1.07	2.565	10.558
5th Floor	3.0	3.5	0.4	0.6875	9.788	1.11	2.785	6.6791
4th Floor	3.0	3.5	0.4	0.6875	9.788	1.11	4.665	6.6791
3rd Floor	3.0	5.5	0.6	0.9375	15.396	1.07	5.395	10.296

Notes for Table 12:

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

2. Bearing area factor for $l_b < 6$ inches:
$$C_b = \frac{(l_b + 0.375)}{l_b}$$

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

3. Bearing plate thicknesses shall be checked for bending using lengths governed by the area satisfaction check and the associated hole in the plate.

Example bending check of bearing plate at third floor:

Bearing plate size = 3.0 inches x 5.5 inches x 0.6 inches thick

Bearing load = 5,395 lbs (Table 12)

Bearing area for wood: subtracting for 3/16-inch oversized hold in wood plate

$$(16.5 - 1.104) = 15.396 \text{ sq in}$$

$$f_{c\perp} = \frac{5,395}{15.396} = 350 \text{ psi}$$

$$F'_c = F_c C_b = 625 \times 1.07 = 669 \text{ psi} > 350 \text{ psi okay}$$

Steel plate bending check:

$$(350 \times 3.0) \times \frac{\left(\frac{5.5}{2}\right)^2}{2} = 3,970 \text{ in/lb}$$

$$Z_{plate} = \frac{bd^2}{4} = \frac{(3.0 - 0.9375) \times 0.6^2}{4} = 0.1856 \text{ in}^3$$

$$\frac{M}{Z} = \frac{3,970}{0.1856} = 21.4 \text{ ksi okay}$$



4. Allowable capacity: $F'_{c\perp} A_{Br} C_b$

Where: $F'_{c\perp} = 0.625 \text{ ksi}$

5. The bearing area is based upon the sill plate hole diameter being 1/4-inch larger than the rod diameter.

6. Bearing load = differential load from Table 10.

BOLTED TIE-DOWN DEVICE ELEMENTS

Another type of tie-down device, illustrated in Figure 15, utilizes bolts instead of bearing plates to transfer the overturning forces to the continuous rods. In this system, posts need to transfer tension forces. Although this type of system is still available, most framing contractors prefer the bearing plate devices due to quicker/easier installation in the field.

TAKE-UP DEVICES

Most continuous rod systems have methods of compensating for shrinkage with proprietary expanding or contracting devices.

The purpose of these devices is to minimize the clearance created between the holdown, tension tie connector, or plate washer and the anchor bolt/nut due to building settlement or wood shrinkage. They keep rotating the nut down (or use a compression spring) on the rod so the holdown, tension tie or bearing plate remains tight to the wood surface.

ICC Evaluation Service has acceptance criteria (AC 316) for shrinkage compensating (take-up) devices. The design engineer should check to see that the proprietary devices conform to these criteria.

The use of take-up devices is highly desirable in multi-level wood-framed construction. Since the total shrinkage of the building has to be accounted for in the tie-down displacement (d_a), it is very difficult to meet the code drift requirements for most shear walls without take-up devices, especially for short-length shear walls.

Take-up devices deflect under load just like the conventional holdown. Most manufacturers publish this information either in their brochures or Evaluation Service approvals. The deformation or initial slack of these devices needs to be considered in the overall tie-down displacement (d_a).

Take-up devices have moving parts and may jam if not properly installed. Jamming typically occurs as a result of excessive continuous tie rod angle (out-of-plumb). See the manufacturer's instructions for proper installation.

7. Considerations with Continuous and Discontinuous Anchor Tie-downs

Continuous tie-downs have several advantages over conventional tie-downs—such as ease of installation and the achievement of higher uplift capacities. Most conventional tie-downs (hold downs) do not offer the capacities needed for multi-level construction, or the shrinkage compensating devices that are available in continuous tie-down systems.

SKIPPING OF FLOORS FOR BEARING RESTRAINTS

To reduce costs, some manufacturers “skip” floors with the bearing restraint devices. In this design example, bearing devices may be omitted at the third and fifth floors with restraints at the fourth floor and roof locations. When floors are skipped, the magnitude of tie-down assembly displacement is accumulative between the bearing restraints and hence significantly increases the shear wall deflection(s). Skipping floors is not recommended.

BEARING ZONE THROUGH FRAMING

Compression loads to the boundary members (posts) are achieved by nailing the shear wall sheathing to each boundary member, thus transferring the overturning (compression) forces, and are accumulative to the stories below. As the shear wall transfers the overturning (tension) forces to the boundary members, these forces collect at each level (between restraint devices) and transfer the differential loads (see Table 10) to the bearing plates at the level above (see Figure 14). The engineer should consider how the differential uplift forces are transferred from the boundary members to the bearing plate. As a general rule, when the differential uplift forces can be transferred within a bearing area located within a 45 degree plane from the bearing plate, no further investigation is necessary (see Figure 14A). When the transfer of forces requires an area larger than the 45 degree plane, some sort of further investigation is necessary (e.g., bending and shear checks of top plates etc.).

Example bearing check (See Figure 14A):

Differential load at third floor = 5,395 lb (from Table 10)

Bearing plate width = 5.5 inches (from Table 12)

Bearing width at bottom of 4x4 top plate = (5.5 + 5.7 + 5.7) = 16.9 inches

Net bearing area = (16.9 – 6.0) x 3.5 = 38.1 square inches

Bearing stress = 5,395/38.1 = 142 psi < 625 psi *okay*

Posts at plate = 5,395/(2 x 3.5 x 3.5) = 220 psi < 625 psi *okay*

8. Shear Wall Deflection, Tie-Down and Take-up Devices

8a. Continuous Tie-down Assembly Displacement

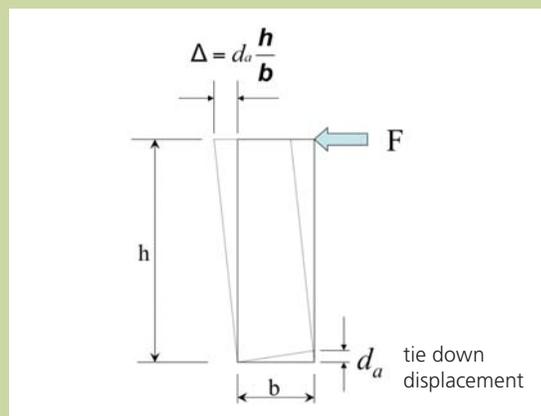
The continuous tie-down assembly displacement (d_a) is a collective accumulation of the deformation of tie-down elements. Each of these elements deforms, elongates and/or shrinks.

2006 IBC now has a revised definition of d_a stated as follows:

“Vertical elongation of overturning anchorage (including fastener slip, device deflection, anchor rod elongation, etc.) at the design shear load (v).”

The net effect of the tie-down assembly displacement is a rotation of the shear wall, as a rigid body, with the displacement at the top of the wall (Δ) equal to the aspect ratio times the tie-down assembly displacement (d_a).

Figure 12. Effect of d_a on drift



NOTES FOR FIGURE 12:

Where: h = floor-to-floor height
 b = the out-to-out dimension of the shear wall



ROD ELONGATION

Some jurisdictions have limits on the amount of rod elongation that can occur between restraints, and some require that the “allowable stress area” (A_e vs. A_g) be used in rod elongation calculations. As such, local building department requirements should always be checked. This design example uses A_e for rod elongation and A_g or A_n for rod capacity. Many manufactures will vary the yield strength of the tension rods. It should be noted that the use of a higher strength rod can actually increase the drift of the shear wall, due to increased elongation from higher loads that can be placed on the same size rod diameters and the modulus of elasticity of the steel, which does not change.

For further discussion on rod elongation see section 6j.

SILL PLATE CRUSHING

Per NDS-05 § 4.2.6, when compression perpendicular to grain $f_{c\perp}$ is less than $0.73 F'_{c\perp}$, crushing will be approximately 0.02 inch. When $f_{c\perp} = F'_{c\perp}$, crushing is approximately 0.04 inch. The effect of sill plate crushing is the downward effect at the opposite end of the wall (resulting from the boundary chords) and has the same rotational effect as the tie-down displacement (d_a). Short walls that have no (net) uplift forces will still have a crushing effect at wall boundaries and contribute to rotation of the wall.

Table 13. Determine sill plate crushing

Level	Chord Posts	ASD Demand (kips)	Strength Demand (kips)	Total Area (in ²)	$f_{c\perp}$ (ksi)	$0.73F'_{c\perp}$ (ksi)	Crush (in)
Roof	4-3x4	4.06	5.62	35.0	0.160	0.456	0.007
5th Floor	4-3x4	11.53	16.13	35.0	0.461	0.456	0.023
4th Floor	4-4x8	22.88	32.13	101.5	0.317	0.456	0.015
3rd Floor	4-4x8	33.85	47.62	101.5	0.469	0.456	0.025

Where:

1. ASD demand and strength demand values are obtained from Table 8.
2. Crushing value ranges from 0.00 to 0.02 inch when $f_{c\perp}$ ranges from 0.0 psi to $0.73F'_{c\perp}$ and ranges from 0.02 to 0.04 inch when $f_{c\perp}$ ranges from $0.73 F'_{c\perp}$ to $F'_{c\perp}$. Values are interpolated to obtain the crushing values listed (crush).

Table 14. Determine bearing plate crushing

Level	ASD Bearing Load (kips)	Strength Bearing Load (kips)	Bearing Plate A_{Brg} (in ²)	$f_{c\perp}$ (ksi)	$0.73F'_{c\perp}$ (ksi)	Crush (in)
Roof	2.565	3.664	15.788	0.232	0.456	0.010
5th Floor	2.785	3.979	9.788	0.406	0.456	0.018
4th Floor	4.665	6.664	9.788	0.681	0.456	0.047
3rd Floor	5.395	7.707	15.396	0.501	0.456	0.025

Where:

1. ASD bearing load values are obtained from the differential loads of Table 11.
2. Strength bearing loads are obtained by dividing ASD bearing loads by the conversion factor of 0.7.
3. Note that the "allowable" $F_{c\perp}$ has been exceeded at the fourth floor; however, this design example uses "strength" (LRFD) loads where the bearing resistance is:

$$F'_{c\perp} = \lambda \phi_c K_F F_{c\perp} C_b = 1.0 \times 0.9 \left(\frac{1.875}{0.9} \right) 625 \times 1.11 = 1,300 \text{ psi} > 681 \text{ psi okay}$$

Also see ASD bearing plate capacities and bearing factors from Table 12.

Table 15. Determine tie-down assembly displacement (with shrinkage compensators)

Level	Rod Elong. (in)	Shrinkage (Vertical Displacement) (in)	Chord Crushing (in)	Bearing Plate Crushing (in)	Take-up Deflection Elongation (in)	Total Displacement d_a (in)
Roof	0.047	0.03	0.007	0.010	0.03	0.124
5th Floor	0.098	0.03	0.023	0.018	0.03	0.199
4th Floor	0.183	0.03	0.015	0.047	0.03	0.305
3rd Floor	0.138	0.03	0.025	0.025	0.03	0.248

Table 15A. Determine tie-down assembly displacement (without shrinkage compensators)

Level	Rod Elong. (in)	Shrinkage (Vertical Displacement) (in)	Chord Crushing (in)	Bearing Plate Crushing (in)	Total Displacement d_a (in)	Accumulative Displacement d_a (in)
Roof	0.047	0.170	0.007	0.010	0.234	1.311
5th Floor	0.098	0.170	0.023	0.018	0.309	1.077
4th Floor	0.183	0.170	0.015	0.042	0.410	0.768
3rd Floor	0.138	0.170	0.025	0.025	0.358	0.358

Notes for Tables 15 and 15A:

Where:

1. Rod Elongation values are obtained from Table 11.
2. Shrinkage values (vertical displacement) are obtained from Table 5; where shrinkage compensating devices are used, a value of 1/32 inch is used, recognizing that most devices have to travel a distance before they get to the next "groove" in the device to re-adjust.
3. Chord crushing (crush) values are obtained from Table 13.
4. Bearing plate (crush) values are obtained from Table 14.
5. Take-up deflection elongation in Table 15A is 0.00 inches because the device has been omitted.
6. Without shrinkage compensators (Table 15A), the tie-down assembly displacements are accumulative from floor-to-floor level.



8b. Shear Wall Deflection

DEFLECTION EQUATION:

The IBC lists the following well-known four-term equation to determine the shear wall deflection:

$$\delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75 he_n + d_a \frac{h}{b} \quad \text{IBC Eq. 23-2}$$

Where:

E = Modulus of Elasticity of diaphragm chords

$E = 1,700,000$ psi

A = area of chord cross-section (3x4 posts or 4x8 posts in this design example)

Gt = sheathing shear stiffness from nail slip and panel shear deformation (from IBC Table 2305.2.2(2)).

$Gt = 44,500$ lb/in for 15/32" Structural I, 5-ply plywood and 83,500 lb/in for OSB

b = shear wall length

h = height of the wall in feet

v = incurred unit shear in diaphragm

d_a = continuous tie-down assembly displacement (see section 8a)

e_n = nail deformation

The NDS-05 SDPWS lists the following three-term equation to determine the shear wall deflection:

$$\delta = \frac{8vh^3}{EAb} + \frac{vh}{1,000 G_a} + d_a \frac{h}{b} \quad \text{SDPWS Eq. C4.3.2-2}$$

Where:

E = Modulus of elasticity of diaphragm chords

$E = 1,700,000$ psi

A = area of chord cross-section (3x4 posts or 4x8 posts in this design example)

G_a = apparent diaphragm shear stiffness from nail slip and panel shear deformation (from Column A – Table 4.2A); for 6-inch nailing in a blocked diaphragm:

$G_a = 17.0$ kips/in

Note: G_a values are to be multiplied by 0.5 if the moisture content is 19% at time of installation of nails/fasteners.

b = shear wall length

v = incurred unit shear in diaphragm

h = height of the wall in feet

d_a = continuous tie-down assembly displacement (see section 8b for discussion)

The new simplified three-term equation combines the second and third terms (of the four-term equation) into one term. The computed deflections by using either the four-term equation or the three-term equation produce nearly identical results.

This design example uses the SDPWS three-term equation.

Table 16. Determine shear wall deflection (using shrinkage compensating devices)

Level	ASD Shear (<i>plf</i>)	Strength Shear (<i>plf</i>)	<i>h</i> (<i>ft</i>)	<i>A</i> (<i>in</i> ²)	<i>b</i> (<i>ft</i>)	<i>G_a</i> (<i>k/in</i>)	Nail Spacing (<i>in</i>)	Total Displacement <i>d_a</i> (<i>in</i>)	Deflection δ_{xe} (<i>in</i>)
Roof	340	485	10.0	35.0	29.0	22.0	6	0.124	0.27
5th Floor	565	807	10.0	35.0	29.0	36.0	3	0.199	0.30
4th Floor	720	1,028	10.0	101.5	29.0	51.0	2	0.305	0.31
3rd Floor	800	1,142	10.0	101.5	29.0	51.0	2	0.248	0.31

Table 16A. Determine shear wall deflection (without shrinkage compensating devices)

Level	ASD Shear (<i>plf</i>)	Strength Shear (<i>plf</i>)	<i>h</i> (<i>ft</i>)	<i>A</i> (<i>in</i> ²)	<i>b</i> (<i>ft</i>)	<i>G_a</i> (<i>k/in</i>)	Nail Spacing (<i>in</i>)	Total Displacement <i>d_a</i> (<i>in</i>)	Deflection δ_{xe} (<i>in</i>)
Roof	340	485	10.0	35.0	29.0	22.0	6	1.311	0.68
5th Floor	565	807	10.0	35.0	29.0	36.0	3	1.077	0.60
4th Floor	720	1,028	10.0	101.5	29.0	51.0	2	0.768	0.47
3rd Floor	800	1,142	10.0	101.5	29.0	51.0	2	0.358	0.35

Where:

$$\delta = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + d_a \frac{h}{b}$$

Comparing shear wall deflections, the shear walls without shrinkage compensating devices were found to deflect 2½ times more at the roof level than those with these devices. Further, the magnitude of the increased deflection increases significantly as the length of the shear wall decreases and the ratio of *h/b* becomes larger.

Note that some jurisdictions require the calculated drifts to be increased by 1.25 to account for dynamic cyclic effects on the wall that could reduce its stiffness.

8c. Story Drift Determination

ASCE 7-05 §12.8.6

The code states that when allowable stress design is used, the computed story drift δ_{xe} shall be computed using strength-level seismic forces specified in ASCE 7-05 §12.8 without the reduction for allowable stress design.

For light-frame walls sheathed with wood structural panels rated for shear resistance, the design story drift is computed as follows:

$$\delta_x = \frac{C_d \delta_{xe}}{I}$$



Where:

δ = design story drift

C_d = deflection amplification factor from ASCE 7-05 Table 12.2-1

$C_d = 4.0$

I = occupancy factor

$I = 1.0$

δ_{xe} = calculated deflection at the top of the wall

$$\delta_x = \frac{4.0 \delta_{xe}}{1.0} = 4.0 \delta_{xe}$$

The calculated story drift using δ_x shall not exceed the maximum allowable which is 0.025 times the story height h for structures four stories or less in height. The calculated story drift shall not exceed 0.020 times the story height h for structures five stories or more in height. Since the overall building is five stories, the drift limit is 0.020 h .

DETERMINATION OF MAXIMUM DRIFTS

ASCE 7-05 Table 12.12-1

Table 17. Determine shear wall drift vs. allowable drifts (with shrinkage compensators)

Level	Deflection δ_{xe} (in)	h (ft)	Story Design Drift $4.0\delta_{xe}$ (in)	Code Max. Allowable (in)
Roof	0.27	10.0	1.08	2.40
5th Floor	0.30	10.0	1.20	2.40
4th Floor	0.31	10.0	1.24	2.40
3rd Floor	0.31	10.0	1.24	2.40

Table 17A. Determine shear wall drift vs. allowable drifts (without shrinkage compensators)

Level	Deflection δ_{xe} (in)	h (ft)	Story Design Drift $4.0\delta_{xe}$ (in)	Code Max. Allowable (in)
Roof	0.68	10.0	2.72	2.40
5th Floor	0.60	10.0	2.40	2.40
4th Floor	0.47	10.0	1.88	2.40
3rd Floor	0.35	10.0	1.40	2.40

Notes for Tables 17 and 17A:

Shear wall drifts do not include the diaphragm deflections between the shear walls but are considered negligible for this design example.

For the 29-foot-long wall used in this design study, the shear wall with the shrinkage compensating devices meets the drift requirements but the shear wall without the shrinkage compensating devices exceeds the drift requirements at roof level.

8d. Load Path for Rod Systems

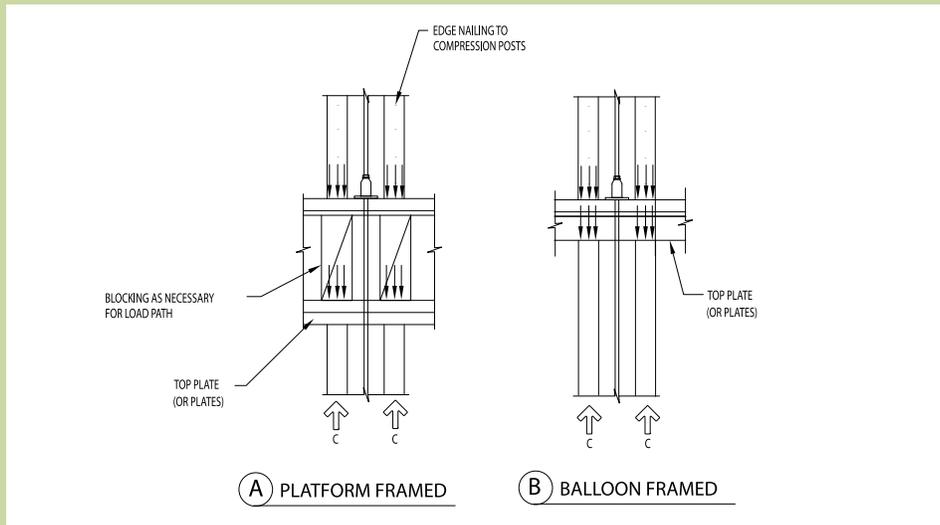
COMPRESSION MEMBERS

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 13).

TENSION RODS

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 (load transfer device) resists the individual story overturning by restraining the posts below from uplifting (Figure 14). The bearing plates transfer the uplifting forces from the posts to the tension rod.

Figure 13. Load Transfer from Compression Posts to Compression Posts

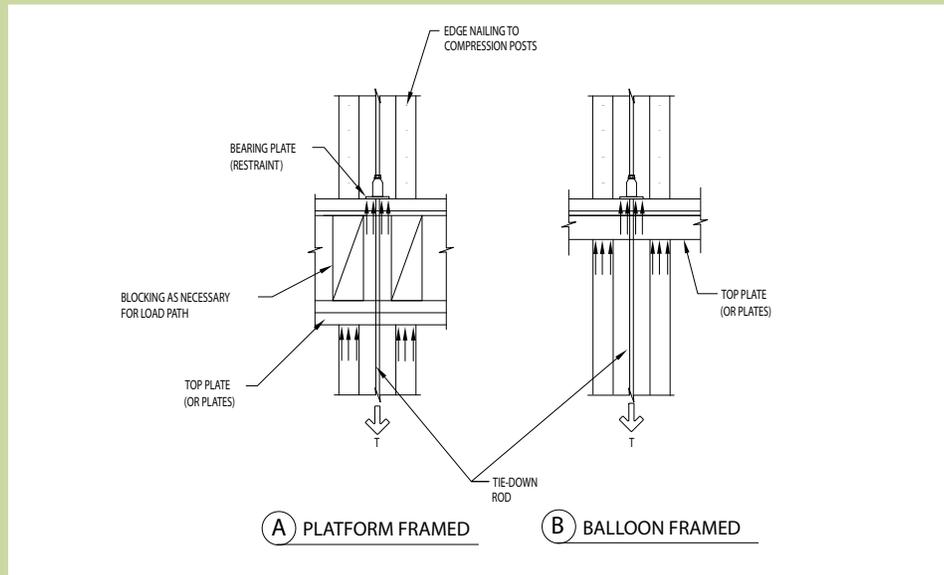


NOTES FOR FIGURE 13:

Detail A (at platform framed) may have a single block with a drilled hole for the tie-down rod (see Figure 15).



Figure 14. Load Transfer from Uplifting Posts to Bearing Device



NOTES FOR FIGURE 14:

Detail A (at platform framed) may have a single block with a drilled hole for the tie-down rod (see Figure 15).

Figure 14A. Bearing Zone Through Framing from Uplifting Posts to Bearing Device

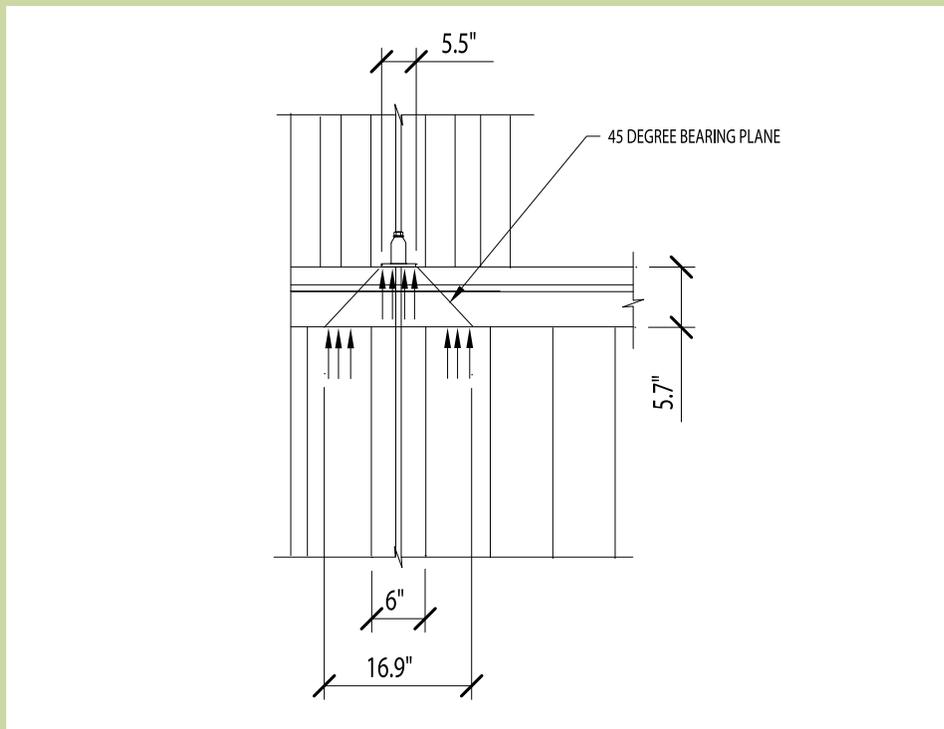
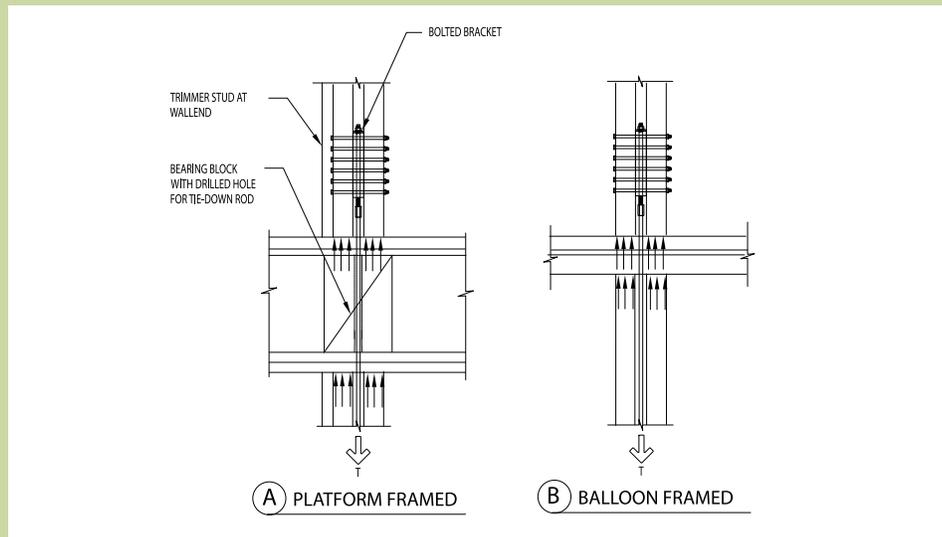


Figure 15. Load Transfer from Uplifting Posts to Bolted Device



8e. Proprietary Software for Continuous Tie-down Systems

Several continuous tie-down system manufacturers offer design software to aid the design engineer in the proper selection of their products as well as the proper selection of the compression chord members. The use of these software programs can be a big time saver for the engineer.

9. Discontinuous System Considerations and the Over Strength (Ω) Factor

9a. Anchor Forces to Podium Slab

For over 20 years, the building codes have had requirements for amplified forces to elements supporting discontinuous systems. Earlier editions of the codes used the term $3R_w/8$, while current codes use the term Ω . Previous editions of the IBC and the 97 UBC exempted concrete slabs supporting light-framed construction from these requirements. However, ASCE 7-05 has added “slabs” to the list of elements needing the design strength to resist the maximum axial force that can be delivered per the load combinations with the overstrength factor (Ω) in §12.4.3.2.

This means that the shear wall boundary overturning forces (axial uplift and axial compression) need to have the Ω factor of 3.0 applied to the supporting slab design. Footnote g of ASCE 7-05 Table 12.2-1 states that, for structures with flexible diaphragms, this value may be 2.5. However, ASCE 7-05 has added commentary to the requirements in §12.4.3.2:

Section C12.4.3 of the ASCE 7-05 commentary states that:

This standard permits the special seismic loads to be taken as less than the amount computed by the Ω_0 coefficient ... when it can be shown that yielding of other elements in the structure will limit the amount of load that can be delivered to the element.



In Addition, § C12.3.3.3 of the ASCE 7-05 commentary states that:

Connection between shear wall and supporting member need only be designed to transmit the loads associated with the shear wall and not the special seismic loads.

What the ASCE 7-05 commentary is stating is that the Ω factor of 3.0 need not be applied when it can be shown that yielding of other elements (diaphragm, collector, collector tie, etc.) will occur below the Ω level forces. In addition, the commentary is also stating that the “tie-down” to the slab need not be designed for the special seismic loads. The provision of §12.3.3.3 only requires that the connection be adequate to “transmit” the forces for which the discontinuous elements were required to be designed. The ability to “transmit” such forces addresses the need for the “strength” of the connection to be adequate (rather than ensuring elastic type of response in the connection— e.g., Omega factor increase).

Different jurisdictions interpret the application of the Ω factor to podium slabs differently and it is recommended that the engineer discuss the requirements with local building officials prior to starting the podium slab design.

It is common to have an anchor bolt (to the podium slab) not meet the requirements of ACI 318 Appendix D because of edge distances or embedment lengths available. Other means of bolt anchorage commonly used include “through bolting” or “sleeves” for a post installed through bolt, embed plates with welded studs, bearing plate washers at the bolt nut, or special steel reinforcing bars used in conjunction with the anchor bolts/bearing plates.

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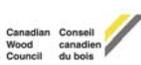
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Suggestions for Improvement

Comments and suggestions for improvement are welcome and should be e-mailed to WoodWorks at info@woodworks.org.

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