Shear Wall Design with Examples DES 413-1

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American Wood Council

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Learning Objectives

• At the end of this program, participants will be better able to:

  • Identify and understand the basic shear wall system to resist lateral loads
  • Understand the difference between segmented and perforated shear wall design
  • Understand hold down design
  • Identify and analyze shear walls per the IBC, WFCM, and SDPWS and understand the differences between them

Polling Question

1. What is your profession?
   a) Architect
   b) Engineer
   c) Code Official
   d) Building Designer
   e) Other
Outline

- Shear Walls Design Examples:
  - 2012 IBC/IRC Recognition
  - 2012 WFCM Prescriptive
  - 2012 WFCM Engineered
  - 2008 SDPWS

- Anatomy of a Shear Wall
  - Elements of a shear wall
  - Strength and stiffness
  - Detailing and limitations

AWC Design Standards
WFCM and IRC/IBC

2012 WFCM is referenced in 2012 IRC/IBC

WFCM and IRC

IRC R301.1.1 Alternative Provisions

**R301.1.1 Alternative provisions.** As an alternative to the requirements in Section R301.1 the following standards are permitted subject to the limitations of this code and the limitations therein. Where engineered design is used in conjunction with these standards, the design shall comply with the *International Building Code*.

WFCM and IRC

IRC R301.2.1.1 Wind limitations

R301.2.1.1 Wind limitations and wind design required.

In regions where wind design is required in accordance with Figure R301.2(4)B or where the basic wind speed shown on Figure R301.2(4)A equals or exceeds 110 miles per hour (49 m/s), the design of buildings for wind loads shall be in accordance with one or more of the following methods:

1. AF&PA Wood Frame Construction Manual (WFCM); or

WFCM and IBC

IBC Chapter 16

1609.1.1 Determination of wind loads. Wind loads on every building or structure shall be determined in accordance with Chapters 26 to 30 of ASCE 7 or provisions of the alternate all-heights method in Section 1609.6. The type of opening protection required, the ultimate design wind speed, \( V_{aw} \), and the exposure category for a site is permitted to be determined in accordance with Section 1609 or ASCE 7. Wind shall be assumed to come from any horizontal direction and wind pressures shall be assumed to act normal to the surface considered.

Exceptions:

2. Subject to the limitations of Section 1609.1.1.1, residential structures using the provisions of AF&PA WFCM.
WFCM and IBC

IBC Section 2301.2

2301.2 General design requirements. The design of structural elements or systems, constructed partially or wholly of wood or wood-based products, shall be in accordance with one of the following methods:

1. Allowable stress design in accordance with Sections 2304, 2305 and 2306.
2. Load and resistance factor design in accordance with Sections 2304, 2305 and 2307.
3. Conventional light-frame construction in accordance with Sections 2304 and 2308.

Exception: Buildings designed in accordance with the provisions of the AF&PA WFCM shall be deemed to meet the requirements of the provisions of Section 2308.

WFCM and IBC

IBC Section 2308

SECTION 2308
CONVENTIONAL LIGHT-FRAME CONSTRUCTION

2308.1 General. The requirements of this section are intended for conventional light-frame construction. Other methods are permitted to be used, provided a satisfactory design is submitted showing compliance with other provisions of this code. Interior nonload-bearing partitions, ceilings and curtain walls of conventional light-frame construction are not subject to the limitations of this section. Alternatively, compliance with AF&PA WFCM shall be permitted subject to the limitations therein and the limitations of this code. Detached one- and two-family dwellings and multiple single-family dwellings (townhouses) not more than three stories above grade plane in height with a separate means of egress and their accessory structures shall comply with the International Residential Code.
2012 WFCM

- Wood Frame Construction Manual
- 2012 WFCM uses ASCE 7-10 wind design provisions

ASCE 7-05 vs ASCE 7-10

<table>
<thead>
<tr>
<th>Table C1.2 Wind Speed Conversion Table</th>
</tr>
</thead>
<tbody>
<tr>
<td>ASCE 7-05 Basic Wind Speeds (mph)</td>
</tr>
<tr>
<td>85</td>
</tr>
<tr>
<td>---------------------------------------</td>
</tr>
<tr>
<td>Equivalent ASCE 7-10 Basic Wind Speeds (mph)</td>
</tr>
<tr>
<td>110</td>
</tr>
</tbody>
</table>
2012 SDPWS is referenced in 2012 IBC

2306.3 Wood structural panel shear walls. Wood-frame shear walls. Wood structural panel—Wood-frame shear walls shall be designed and constructed in accordance with AF&PA SDPWS. Wood structural panel shear walls are permitted to resist horizontal forces using the allowable capacities. Where panels are fastened to framing members with staples, requirements and limitations of AF&PA SDPWS shall be met and the allowable shear values set forth in Table 2306.3—2306.3(1), 2306.3(2) or 2306.3(3) shall be permitted. Allowable capacities in Table 2306.3 The allowable shear values in Tables 2306.3(1) and 2306.3(2) are permitted to be increased 40 percent for wind design. Panels complying with ANSI/APA PRP-210 shall be permitted to use design values for Plywood Siding in the AF&PA SDPWS.

Significant Changes to 2012 IBC

NEW ANSI/APA PRP-210 Plywood Siding
- Durability
- Thickness by thickness
- Siding shear walls
Significant Changes to 2012 IBC

- **Shear wall deflection with staples**
- **Wood structural panels - Wood-frame**
- **Allowable shear tables - nails and staples only**
- **SDPWS**

2385.3 Shear wall deflection. The deflection of wood-frame shear walls shall be determined in accordance with AF&PA SDPWS. The deflection (\(\Delta\)) of a blocked wood structural panel shear wall uniformly fastened throughout with staples is permitted to be calculated in accordance with Equation 23-2.

\[
\Delta = \frac{V S h_1^2}{E a b} + \frac{F T}{G a} + 0.75 d_a ^{b} + d_b ^{b}
\]  
(Equation 23-2)

**SECTION 2307**

**LOAD AND RESISTANCE FACTOR DESIGN**

2307.1 Load and resistance factor design. The design and construction of wood elements and structures using load and resistance factor design shall be in accordance with AF&PA NDS and SDPWS.
Significant Changes to 2012 IBC

4.3.7 Shear Wall Systems

4.3.7.1 Wood Structural Panel Shear Walls: Shear walls sheathed with wood structural panel sheathing shall be permitted to be used to resist seismic and wind forces. The size and spacing of fasteners at shear wall boundaries and panel edges shall be as provided in Table 4.3A. The shear wall shall be constructed as follows:

4. The width of the nailed face of framing members and blocking shall be 2” nominal or greater at adjoining panel edges except that a 3” nominal or greater width at adjoining panel edges and staggered nailing at all panel edges are required where:
   a. Nail spacing of 2” on center or less at adjoining panel edges is specified, or
   b. 10d common nails having penetration into framing members and blocking of more than 1-1/2” are specified at 3” on center, or less at adjoining panel edges, or
   c. Required nominal unit shear capacity on either side of the shear wall exceeds 700 psf in Seismic Design Category D, E, or F.

Exception: Where the width of the nailed face of framing members is required to be 3” nominal, two framing members that are 2” in nominal thickness shall be permitted to be used provided they are fastened together with fasteners designed in accordance with the NDS to transfer the induced shear between members. Where fasteners connecting the two framing members are spaced less than 4” on center, they shall be staggered.

Polling Question

2. Does the 2012 IBC have ASD capacity tables for wood structural panel shear walls with nail fasteners?
Outline

- 2012 IBC/IRC Recognition
- **2012 WFCM Prescriptive**
- 2012 WFCM Engineered
- 2008 SDPWS

Segmented Shear Wall Method
Perforated Shear Wall Method

WFCM Prescriptive

NOTE:
- Wind Speeds 110-195 mph Exp. B & C
- Segmented & Perforated Shear Walls
- Other Application Limits
Design Example

Assumptions

- 130 mph (700-yr, 3-second gust) Exposure B
- L=36’
- W=30’
- 5/12 roof pitch
- Top plate to ridge = 6.25’
- 2-story
- 8’ wall height
- 6’8” door height
- 4’ window height
- Wood Structural Panel Exterior Sheathing
- Vary interior walls – with and without gypsum

Design Example

Design first floor shear wall
**WFCM Prescriptive**

2012 WFCM Prescriptive – Segmented

**Table 3.17A Segmented Shear Wall Requirements for Wind**

<table>
<thead>
<tr>
<th>700-yr. Wind Speed</th>
<th>110</th>
<th>115</th>
<th>120</th>
<th>130</th>
<th>140</th>
<th>150</th>
<th>160</th>
<th>170</th>
<th>180</th>
<th>195</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear Wall Line Beneath</td>
<td>20</td>
<td>32</td>
<td>38</td>
<td>48</td>
<td>58</td>
<td>68</td>
<td>78</td>
<td>88</td>
<td>98</td>
<td>108</td>
</tr>
<tr>
<td>Roof, Ceiling, &amp; 1 Floor</td>
<td>87</td>
<td>96</td>
<td>106</td>
<td>115</td>
<td>125</td>
<td>135</td>
<td>145</td>
<td>155</td>
<td>165</td>
<td>175</td>
</tr>
</tbody>
</table>

Interpolate = 12.3'

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**Footnotes to Table 3.17A**

<table>
<thead>
<tr>
<th>Roof Only</th>
<th>Roof + 1 Floor</th>
<th>Roof + 2 Floors</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall Height</td>
<td>Top Plate to Ridge Height (ft)</td>
<td>Adjustment Factor</td>
</tr>
<tr>
<td>Roof Pitch</td>
<td></td>
<td></td>
</tr>
<tr>
<td>≤:12 0' (Flat)</td>
<td>0.35</td>
<td>0.43</td>
</tr>
<tr>
<td>≤:12 0' 10'</td>
<td>0.50</td>
<td>0.59</td>
</tr>
<tr>
<td>≤:12 ≤:12</td>
<td>0.65</td>
<td>0.74</td>
</tr>
</tbody>
</table>

Interpolate = 0.68

Adjusted = 12.3' (0.68) = 8.4'
WFCM Prescriptive

2012 WFCM Prescriptive – Segmented – required = 8.4'

4' + 4' + 2.5' + 2.5' = 13' > 8.4' OK
Assumes gypsum on interior
What if we don’t count the gypsum on interior?

\[ 8.4' \times 1.3 = 10.9' \]

Segmented shear wall – assuming no interior gypsum

\[ 4' + 2.5' + 2.5' + 4' = 13' > 10.9' \text{ OK} \]
Segmented shear wall – requires hold downs on each segment

% Full-height sheathing
8.4' / 36' = 23%
Interpolated = 1.86
8.4'(1.86) = 15.6'
w/ gypsum

10.9' / 36' = 30%
Interpolated = 1.72
10.9'(1.72) = 18.7'
w/o gypsum

21' Full-height sheathing > 18.7'
PSW requires fully sheathed wall

PSW requires hold-downs only at the ends
Hold-downs = 3,488 lbs w/ gypsum

\[
\frac{3,488}{1.3} = 2,683 \text{ lbs w/o gypsum}
\]

- Need to combine with top floor hold-down requirements
- Based on capacity of first shear wall panel
- Does not include dead load

Outline

- 2012 IBC/IRC Recognition
- 2012 WFCM Prescriptive
  - 2012 WFCM Engineered
- 2008 SDPWS
2012 WFCM Engineered

\[ w_{\text{roof}} = 94 \text{ plf} \]
\[ w_{\text{floor}} = 128(0.82)^* = 105 \text{ plf} \]
\[ w_{\text{total}} = 199 \text{ plf} \]
\[ 199(30')/2 = 2,985 \text{ lbs} \]

*Footnote 2: \((H+1)/11\) adjustment = \((8+1)/11\)
WFCM Engineered

2012 WFCM Engineered – Segmented

Required Capacity = 2,985 lbs
7/16” WSP Capacity = 336 plf
1/2” Gypsum Capacity = 100 plf
Total = 436 plf

\[
\frac{2,985}{436} = 6.8' \text{ (w/ gypsum)} \\
\frac{2,985}{336} = 8.9' \text{ (w/o gypsum)}
\]

4' + 4' = 8' > 6.8'
assuming interior gypsum  OK
WFCM Engineered

2012 WFCM Engineered - Segmented

\[ 4' + 2.5' + 2.5' + 4' = 13' > 8.9' \]
assuming NO interior gypsum OK

2,985 lbs

WFCM Engineered

2012 WFCM Engineered - Perforated

Reference SDPWS Capacities and Adjustments

\[ V = 2,985 \text{ lbs} \]
\[ v = 436 \text{ plf (w/ gypsum)} \]
\[ v = 336 \text{ plf (w/o gypsum)} \]
\[ \%FHS = 21' / 36' = 58\% \]
Interpolated Factor = 0.62

436 (0.62) = 270 plf

2,985/270 = 11.1' (w/ gypsum)

336 (0.62) = 208 plf

2,985/208 = 14.4' (w/o gypsum)
### WFCM Engineered

#### 2012 WFCM Engineered - Perforated

![Diagram of sheathing](image)

21' Full-height sheathing > 14.4' OK

---

### WFCM Engineered

#### 2012 WFCM Engineered – Hold-downs

\[ T = v \times h \]

- \( v = 436 \text{ plf (w/ gypsum)} \)
- \( v = 336 \text{ plf (w/o gypsum)} \)
- \( h = 8' \)

\[ T = 436(8') = 3,488 \text{ lbs} \]

\[ T = 336(8') = 2,688 \text{ lbs} \]

- Need to combine with top floor hold-down requirements
- Based on capacity of first shear wall panel
- Can account for dead load (WFCM 2.2.4)
Polling Question

3. The 2012 WFCM Prescriptive provisions include which of the following shear wall design methods?
   a) Perforated  
b) Segmented  
c) Force transfer around openings  
d) All of the above  
e) a and b

Outline

• 2012 IBC/IRC Recognition
• 2012 WFCM Prescriptive
• 2012 WFCM Engineered
• 2008 SDPWS
SDPWS

2008 SDPWS

- Engineered
- Res and Non-Res
- ASD & LRFD
- Efficiencies in designs
- Shear wall provisions
  - Segmented
  - Perforated
  - Force Transfer Around Openings

Minimum Design Loads

ASCE 7-10 Minimum Design Loads for Buildings and Other Structures

14.5.1 Reference Documents

The quality, testing, design, and construction of members and their fastenings in wood systems that resist seismic forces shall conform to the requirements of the applicable following reference documents:

1. AF&PA NDS
2. AF&PA SDPWS
### 2008 SDPWS - WSP Capacity

**ASD Capacity = 670/2 = 335 plf**

### 2008 SDPWS - Gypsum Capacity

**ASD Capacity = 200/2 = 100 plf**
2008 SDPWS - Gypsum Capacity

Required Capacity = 2,985 lbs
WSP = 335 plf
Gypsum = 100 plf
Total = 435 plf

\[
\frac{2,985}{435} = 6.9' \text{ (w/ gypsum)}
\]
\[
\frac{2,985}{335} = 8.9' \text{ (w/o gypsum)}
\]

2008 SDPWS - Segmented Shear Wall

\[
4' + 4' = 8' > 6.9'
\]
assuming interior gypsum  OK
2008 SDPWS – Segmented Shear Wall

4' + 2.5' + 2.5' + 4' = 13' > 8.9'
assuming NO interior gypsum OK

2,985 lbs

2008 SDPWS – Perforated Shear Wall

Shear Capacity Adjustment Factor

\[ C_o = \left( \frac{r}{3 - 2r} \right) \frac{L_{tot}}{\sum L_i} \]

\[ r = \frac{1}{1 + \frac{A_o}{n \sum L_i}} \]

\[ A_o = 4(4')(2.5') + (5')(6.67') = 73.4 \text{ ft}^2 \]

r = 0.70

C_o = 0.75

Comparison: WFCM Engineered (tabulated) C_o = 0.62
**SDPWS**

**2008 SDPWS – Perforated Shear Wall**

\[ C_0 = 0.75 \]

**w/ gypsum**

\[ 435 \times 0.75 = 326 \]

\[ \frac{2,985}{326} = 9.2' \]

**w/o gypsum**

\[ 335 \times 0.75 = 251 \]

\[ \frac{2,985}{251} = 11.9' \]

21' Full-height sheathing > 11.9' OK
**SDPWS**

**2008 SDPWS – Hold-downs (Segmented)**

\[ T = v \times h \]

- \( v = \frac{2,985}{8'} = 347 \text{ plf (w/ gyp)} \)
- \( v = \frac{2,985}{13'} = 230 \text{ plf (w/o gyp)} \)
- \( h = 8' \)
- \( T = 347(8') = 2,985 \text{ lbs (w/ gyp)} \)
- \( T = 230(8') = 1,840 \text{ lbs (w/o gyp)} \)

- Need to combine with top floor hold-down requirements
- Based on loads
- Can account for dead load (4.3.6.4.2)

**2008 SDPWS – Hold-downs (Perforated)**

\[ T = \frac{V \times h}{C_a \times \sum L_i} \]

- \( V = 2,985 \text{ lbs} \)
- \( h = 8' \)
- \( C_a = 0.75 \)
- \( L_i = 21' \)
- \( T = 1,516 \text{ lbs} \)

- Need to combine with top floor hold-down requirements
- Based on loads
- Can account for dead load (4.3.6.4.2)
### Design Example - Summary

**2012 WFCM**

1st of 2 Story; 30' span; 5/12 pitch; 130 mph Exp. B

<table>
<thead>
<tr>
<th>AWC Standard</th>
<th>Segmented</th>
<th>Perforated</th>
<th>Hold-downs</th>
</tr>
</thead>
<tbody>
<tr>
<td>2012 WFCM</td>
<td>8.4' (10.9')</td>
<td>15.6' (18.7')</td>
<td>3,488 (2,683) lbs</td>
</tr>
<tr>
<td>2012 WFCM</td>
<td>6.8' (8.9')</td>
<td>11.1' (14.4')</td>
<td>3,488 (2,688) lbs</td>
</tr>
<tr>
<td>2008 SDPWS</td>
<td>6.9' (8.9')</td>
<td>9.2' (11.9')</td>
<td>2,985 (1,840) lbs</td>
</tr>
</tbody>
</table>

Parenthetical values assume NO interior gypsum

### Design Example - Comparison

**2001 WFCM**

1st of 2 Story; 30' span; 5/12 pitch; 100 mph Exp. B

<table>
<thead>
<tr>
<th>AWC Standard</th>
<th>Segmented</th>
<th>Perforated</th>
<th>Hold-downs</th>
</tr>
</thead>
<tbody>
<tr>
<td>2001 WFCM</td>
<td>6.9' (9')</td>
<td>13.4' (16.4')</td>
<td>3,488 (2,683) lbs</td>
</tr>
<tr>
<td>2001 WFCM</td>
<td>6.4' (8.3')</td>
<td>10.3' (13.3')</td>
<td>3,488 (2,688) lbs</td>
</tr>
<tr>
<td>2008 SDPWS</td>
<td>6.4' (8.3')</td>
<td>8.5' (11.0')</td>
<td>2,775 lbs</td>
</tr>
</tbody>
</table>

Parenthetical values assume NO interior gypsum
Polling Question

4. The 2008 SDPWS provides shear wall design provisions for which of the following?
   a) Perforated
   b) Segmented
   c) Force transfer around openings
   d) All of the above
   e) a and b
Special Design Provisions for Wind and Seismic (SDPWS)

• 2008 SDPWS and Commentary:
  
• Article on changes in 2008 SDPWS:

Wood Frame Shear Walls

• Shear walls - Seismic
  • Elements of a shear wall
  • Strength and stiffness
  • Detailing and limitations
• NEESWood House – 2-story University of Buffalo
• Constructed in 3 days (time-lapse) [http://youtu.be/l-cXpJJIFU]
• [http://youtu.be/KUJ1dfdZbhI](http://youtu.be/KUJ1dfdZbhI)
Wood Frame Shear Walls

Elements of a wood shear wall

- Specific stud species
- Sheathing panels of specific grade and thickness
- Specific nail size and spacing requirements
- Hold-down anchors
- Base shear anchors

Shear Wall Test

8 ft x 8 ft wood structural panel shear wall cyclic test

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Shear Wall Test

Typical failure of sheathing nailing

a) Nail yielding at adjoining panel edge
b) Nail yielding and head pull through at panel to bottom plate location

Nominal Unit Shear Capacity

- Example determination of nominal unit shear capacity, $v_s$
  - 7/16 in. Structural I WSP, 8d common Nail, 6 in. Panel Edge Fastener Spacing, Framing $G = 0.5$, Studs at 24 in. o.c.
  - Nominal unit shear capacity, $v_s$: 510 plf
Apparent Shear Stiffness

Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls

Example determination of apparent shear stiffness, $G_a$

- 7/16 in. Structural I OSB, 8d common Nail, 6 in. Panel Edge Fastener Spacing, Framing $G = 0.5$, Studs at 16 in. o.c.

Apparent shear stiffness, $G_a$: 16 kips/in.

Table Footnotes are Important

1. Nominal unit shear values shall be adjusted in accordance with 4.3.3 to determine ASD allowable unit shear capacity and LRFD factored unit resistance. For general construction requirements see 4.3.6. For specific requirements, see 4.3.7.1 for wood structural panel shear walls, 4.3.7.2 for particleboard shear walls, and 4.3.7.3 for fiberboard shear walls. See Appendix A for common and box nail dimensions.

2. Shears are permitted to be increased to values shown for 15/32 inch sheathing with same nailing provided (a) studs are spaced a maximum of 16 inches on center, or (b) panels are applied with long dimension across studs.

3. For species and grades of framing other than Douglas-Fir-Larch or Southern Pine, reduced nominal unit shear capacities shall be determined by multiplying the tabulated nominal unit shear capacity by the Specific Gravity Adjustment Factor = $[1-(0.5-G)]$, where $G$ = Specific Gravity of the framing lumber from the NDS (Table 11.3.2A). The Specific Gravity Adjustment Factor shall not be greater than 1.
4. Apparent shear stiffness values $G_a$, are based on nail slip in framing with moisture content less than or equal to 19% at time of fabrication and panel stiffness values for shear walls constructed with either OSB or 3-ply plywood panels. When 4-ply or 5-ply plywood panels or composite panels are used, $G_a$ values shall be permitted to be increased by 1.2.

5. Where moisture content of the framing is greater than 19% at time of fabrication, $G_a$ values shall be multiplied by 0.5.

6. Where panels are applied on both faces of a shear wall and nail spacing is less than 6 in. on center on either side, panel joints shall be offset to fall on different framing members. Alternatively, the width of the nailed face of framing members shall be 3 in. nominal or greater at adjoining panel edges and nails at all panel edges shall be staggered.

7. Galvanized nails shall be hot-dipped or tumbled.

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### Table Footnotes are Important

<table>
<thead>
<tr>
<th>Adjustment for Design Level</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nominal unit shear values adjusted in accordance with 4.3.3 to determine ASD allowable unit shear capacity and LRFD factored unit resistance.</td>
</tr>
<tr>
<td><strong>ASD unit shear capacity, $v_u$:</strong></td>
</tr>
<tr>
<td>$v_u = \frac{510 \text{ plf}}{2.0} = 255 \text{ plf}$</td>
</tr>
<tr>
<td><strong>LRFD unit shear capacity, $v_r$:</strong></td>
</tr>
<tr>
<td>$v_r = 510 \text{ plf} \times 0.80 = 408 \text{ plf}$</td>
</tr>
</tbody>
</table>

#### 4.3.3 Unit Shear Capacities

The ASD allowable unit shear capacity shall be determined by dividing the tabulated nominal unit shear capacity by applicable footnotes, by the ASD reduction factor of 2.0. The LRFD factored unit resistance shall be determined by multiplying the tabulated nominal unit shear capacity, modified by applicable footnotes, by a resistance factor $\phi$ of 0.40. No further increases shall be permitted.
Adjustment for Framing G

- Reduced nominal unit shear capacities determined by multiplying the tabulated nominal unit shear capacity by the Specific Gravity Adjustment Factor
  - SG Adjustment Factor = \([1.0 - (0.50 - G)]\) < 1.0

- Example SG Adjustment Factors

<table>
<thead>
<tr>
<th>Species Combination</th>
<th>Specific Gravity, G</th>
<th>FACTOR = 1.0 - (0.50 - G)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Southern Pine</td>
<td>0.55</td>
<td>1.00</td>
</tr>
<tr>
<td>Douglas Fir-Larch</td>
<td>0.50</td>
<td>1.00</td>
</tr>
<tr>
<td>Hem Fir</td>
<td>0.43</td>
<td>0.93</td>
</tr>
<tr>
<td>Spruce Pine-Fir</td>
<td>0.42</td>
<td>0.92</td>
</tr>
<tr>
<td>Western Woods</td>
<td>0.36</td>
<td>0.86</td>
</tr>
</tbody>
</table>

Adjustment for Aspect Ratio

Aspect Ratio: \(h:b_s\)

For wood structural panel resisting seismic where \(2:1 < h:b_s \leq 3.5:1\), multiply \(v_s\) by \(2b_s/h\)

Table 4.3.4 Maximum Shear Wall Aspect Ratios

| Shear Wall Sheathing Type            | Maximum          | \(|h|/b_s Ratio| |
|--------------------------------------|------------------|--------------|
| Wood structural panels, unblocked    | 2:1              |              |
| Wood structural panels, blocked      | 3.5:1            |              |
| Particleboard, blocked               | 2:1              |              |
| Gypsum wallboard                     | 2.1              |              |
| Portland cement plaster              | 2.1              |              |
| Structural Fiberboard                | 3.5:1            |              |

1. For design to resist seismic forces, the shear wall aspect ratio shall not exceed 2:1 unless the nominal unit shear capacity is multiplied by \(2b_s/h\).
2. Walls having aspect ratios exceeding 1.5:1 shall be blocked shear walls.
3. For design to resist seismic forces, the shear wall aspect ratio shall not exceed 1:1 unless the nominal unit shear capacity is multiplied by the Aspect Ratio Factor (Seismic) = 0.1+.96/h. The value of the Aspect Ratio Factor (Seismic) shall not be greater than 1.0. For design to resist wind forces, the shear wall aspect ratio shall not exceed 1:1 unless the nominal unit shear capacity is multiplied by the Aspect Ratio Factor (Wind) = 1.09/.09b. The value of the Aspect Ratio Factor (Wind) shall not be greater than 1.0.
Example aspect ratio factors for wood structural panel

<table>
<thead>
<tr>
<th>Shear wall height, h, and width, b_s</th>
<th>h, (ft)</th>
<th>b_s, (ft)</th>
<th>h/b_s</th>
<th>ASPECT RATIO FACTOR = 2b_s/h</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>8</td>
<td>4</td>
<td>2</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>3.2</td>
<td>2.5</td>
<td>0.80</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>2.7</td>
<td>3</td>
<td>0.67</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>2.3</td>
<td>3.5</td>
<td>0.57</td>
</tr>
</tbody>
</table>

Example Strength Calculation

Given

• 2'-4" w x 8' h Wall, 7/16 in. Structural I OSB, 8d common nail, 6 in. panel edge fastener spacing, studs at 24 in. o.c.
• Tabulated unit shear capacity, v = 510 plf
• Spruce Pine Fir Framing G = 0.42
• Aspect ratio, h:b_s = 3.5

What is the adjusted LRFD unit shear capacity, v_s?

• A) 232 plf  B) 408 plf  C) 214 plf
Adjusted Design Unit Shear

- Solution: Design unit shear adjusted for framing specific gravity (G=0.42) and aspect ratio (3.5:1)
  
  \[ \text{SG Adjustment Factor} = [1.0-(0.50-0.42)] = .92 \]

- LRFD unit shear capacity
  
  \[ v_s = 510 \text{ plf} \times 0.80 \times 0.92 \times 0.57 = 214 \text{ plf} \]

- ASD unit shear capacity
  
  \[ v_s = \frac{510 \text{ plf}}{2.0} \times 0.92 \times 0.57 = 134 \text{ plf} \]

Shear Wall Deflection

- Shear Wall (IBC 2305.3) – staples only

\[ \Delta = \Delta_b + \Delta_v + \Delta_n + \Delta_a \]

\[ \Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_a + \frac{h}{b} \]

\[ \frac{v \cdot h}{1000Ga} \]
Shear Wall Deflection

Shear wall deflection, Equation 4.3-1

\[ \Delta = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b} \]

- \( b \) = shear wall length, ft
- \( h \) = shear wall height, ft
- \( E \) = modulus of elasticity of end posts, psi
- \( A \) = area of end post cross-section, in\(^2\)
- \( G_a \) = apparent shear wall shear stiffness, kips/in.
- \( \Delta \) = total vertical elongation of wall anchorage system, in.
- \( v \) = induced unit shear, lbs/ft

Non-linear 4-term equation in Commentary
Polling Question

5. The three-term diaphragm and shear wall deflection equations are an algebraic simplification of the four-term equation.

Example Deflection Calculation

Given

- 8 ft x 8 ft Wall
- Apparent shear stiffness, $G_o$: 16 kips/in.
- (2) 2x4 framing as end post area, $A$: $(2) \times 1.5 \times 3.5 = 10.5$ in.$^2$
- Stud modulus of elasticity, $E$: 1,600,000 psi
- Hold-down anchor slip, $\Delta_a$: 0.125 in.
- Induced unit shear, $v$, 408 plf (LRFD)
- Wall on rigid foundation

Question: What is the horizontal deflection at the top of the wall?
Example Deflection Calculation

- Solution
- Shear wall deflection from Eq. 4.3-1:
  - Bending: \( \frac{8vh^3}{EAb} = 0.012 \text{ in.} \)
  - Shear: \( \frac{vh}{1000G_a} = 0.204 \text{ in.} \)
  - Anchor slip: \( \frac{h}{b} \Delta_a = 0.125 \text{ in.} \)
- Total deflection = 0.34 in.

Other Shear Wall Material
Strength and stiffness information provided for other materials
- Structural Fiberboard
- Wood structural panel over gypsum sheathing board
- Gypsum Wallboard and Gypsum Sheathing (blocked and unblocked)
- Gypsum lath
- Expanded metal or woven wire lath and Portland cement plaster
- Particleboard Sheathing
- Plywood Siding
- Lumber sheathing (horizontal, diagonal and double diagonal)
- Vertical lumber siding
- Wood structural panel - unblocked

Data shown from Report WMEL-2002-03, Washington State University, Dolan
Shear Wall Types

- Individual Full height segments (SDPWS 4.3.5.1)
  - Reference wall construction, other methods adjust values from the full height reference case
- Force transfer around openings (SDPWS 4.3.5.2)
  - Design for force transfer and resulting strength and stiffness calculations by rational analysis
- Perforated (SDPWS 4.3.5.3)
  - Empirically based strength and stiffness reductions

Individual Full-height Shear Walls

- Only full height segments are considered
- Reference wall type – basis of tabulated unit shear values
Force Transfer Around Opening

- Hold-downs typically only at ends
- Extra calculations and added construction details (connections & blocking)
- Uses reference design values

Perforated Shear Walls

- Hold-downs only at ends
- Empirically based strength and stiffness reduction factor applied to full height panels to account for effect of opening
- Bottom plate attachment for uplift
- Uses reference design values
Additional Resources

- Force transfer around openings
  - Design of Wood Structural Panel Shear Walls with Openings - a Comparison of Methods; Wood Design Focus 2005
  - Design of Wood Structures; McGraw Hill 2007
- Perforated shear wall method
  - Perforated Shear Wall Design; Wood Design Focus Spring 2002

3x at Adjoining Panel Edge

Table 4.3A footnote 6. 3x framing required to reduce potential for splitting at adjoining panel edge where WSP is nailed on each face and nail spacing is less than 6 in. o.c.

Figure C4.3.3  Detail for Adjoining Panel Edges where Structural Panels are Applied to Both Faces of the Wall
3x at Adjoining Panel Edge

- Section 4.3.7.1(4). 3x framing also required at adjoining panel edges where:
  - Nail spacing of 2 in. o.c.
  - 10d common nails having penetration of more than 1-1/2 in. at 3 in. o.c. or less
  - Nominal unit shear capacity on either side exceeds 700 plf in SDC D, E, or F.
- Exception: (2) 2x framing permitted in lieu of (1) 3x where fastened in accordance with the NDS to transfer the induced shear between members.

(2) 2x At Adjoining Panel Edge

Approximate stud to stud connection spacing for wood structural panel (WSP) walls sheathed on one side.

<table>
<thead>
<tr>
<th>Nail size and sheathing</th>
<th>Stud to frame lateral force per NDS (G=0.5 framing)</th>
<th>Fastener spacing (in.) for 2x stud-to-2x stud connection</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Z (0)</td>
<td>6</td>
</tr>
<tr>
<td>8d common, 3/8&quot; WSP (G=0.5)</td>
<td>9.4</td>
<td>12.0</td>
</tr>
<tr>
<td>8d common, 3/8&quot; WSP (G=0.5)</td>
<td>7.1</td>
<td>9.1</td>
</tr>
<tr>
<td>8d common, 7/16&quot; WSP (G=0.5)</td>
<td>7.3</td>
<td>8.8</td>
</tr>
<tr>
<td>10d common, 1/2&quot; WSP (G=0.42)</td>
<td>5.5</td>
<td>6.8</td>
</tr>
</tbody>
</table>

* Spacing based on 8' wall and assuming only 87.5" of stud height available for stud-to-stud fastening.
(2) 2x at Adjoining Panel Edges

C4.3.7.1(4). A single 3x framing member is specified at adjoining panel edges for cases prone to splitting and where nominal unit shear capacity exceeds 700 plf in seismic design categories (SDC) D, E, and F. An alternative is single 8x framing, included in SDPWS, and based on principles of mechanics, is the use of 2-2x “stitched” members adequately fastened together. Cyclic tests of shear walls confirm that use of 2-2x members nailed (22, 25, and 30) or screwed (33) together results in shear wall performance that is comparable to that obtained by use of a single 3x member at the adjoining panel edge. Attachment of the 2-2x members to each other is required to equal or exceed design unit shear forces in the shear wall. As an alternative, a capacity-based design approach can be used where the connection between the 2-2x members equals or exceeds the capacity of the sheathing to framing attachment. Where fastener spacing in the “stitched” members at adjoining panel edges is closer than 4” on center, staggered placement is required.

Foundation Bottom Plate

- Plate washer
  - Must extend to within ½ in. of sheathed edge of bottom plate
- Exceptions
  - Lower capacity sheathing materials (nominal unit shear is 400 plf or less)
  - Hold-downs are sized for full overturning – neglecting dead load
Foundation Bottom Plate – Testing

Failure Mode?

Small scale test specimen to induce cross grain bending

Mode I and Mode II observed in small specimen testing
Foundation Bottom Plate – Testing

Failure Mode?

Small scale test specimen to induce cross grain bending

Figure 2. Shearwall assembly in test fixture

View of bottom plate after test.
Foundation Bottom Plate – Testing

Shear wall assembly in test fixture

View of bottom plate after test.

Polling Question

6. Plate washers (3”x3”) at the foundation sill plate need to be installed within:
   a. 1/2” of the sheathed edge
   b. 1” of the sheathed edge
   c. 1-1/2” of the sheathed edge
   d. None of the above
Minimum Panel Width

- Blocked wood structural panel shear wall
  - no minimum panel width
  - SDPWS 4.3.7.1
  - “Panels shall not be less than 4 ft x 8 ft, except at boundaries and changes in framing. All edges of all panels shall be supported by and fastened to members or blocking”

Minimum Panel Width

- Blocked wood structural panel shear wall
  - Average 6 in. width: Peak load = 10,092 lbf; Deflection = 2.48 in.
  - Average 24 in. width: Peak load = 10,350 lbf; Deflection = 2.59 in.
Limitations Overview

- Limitations on use of reference values include:
  - Adhesive attachment associated with reduced R and SDC limits
  - Maximum shear wall aspect ratio
  - Resisting forces contributed by masonry and concrete walls

Adhesive Attachment of Sheathing

- R=1.5 and Seismic Design Category C limitation applies where adhesive attachment of sheathing is used:

  4.3.6.3.1 Adhesives: Adhesive attachment of shear wall sheathing shall not be used alone, or in combination with mechanical fasteners.

  **Exception:** Approved adhesive attachment systems shall be permitted for wind and seismic design in Seismic Design Categories A, B, and C where R = 1.5 and \( \Omega_c = 2.5 \), unless other values are approved.

- Limited data shows increased shear strength but reduced deformation capacity.
Adhesive Attachment of Sheathing

• Static and dynamic tests of timber shear walls fastened with nails and wood adhesive, Filiatrault and Foschi, CJCE-18-1991.

Shear Wall Aspect Ratios

• SDPWS Table 4.3.4 – Maximum Shear Wall Aspect Ratios

<table>
<thead>
<tr>
<th>Shear Wall Sheathing Type</th>
<th>Maximum h/b Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wood structural panels, unblocked</td>
<td>2:1</td>
</tr>
<tr>
<td>Wood structural panels, blocked</td>
<td>3.5:1</td>
</tr>
<tr>
<td>Particleboard, blocked</td>
<td>2:1</td>
</tr>
<tr>
<td>Diagonal sheathing, conventional</td>
<td>2:1</td>
</tr>
<tr>
<td>Gypsum wallboard</td>
<td>2:1</td>
</tr>
<tr>
<td>Portland cement plaster</td>
<td>2:1</td>
</tr>
<tr>
<td>Structural Fiberboard</td>
<td>3.5:1</td>
</tr>
</tbody>
</table>

1 For design to resist seismic forces, the shear wall aspect ratio shall not exceed 2:1 unless the nominal unit shear capacity is multiplied by 2b/h.
2 Walls having aspect ratios exceeding 1.5:1 shall be blocked shear walls.
3 For design to resist seismic forces, the shear wall aspect ratio shall not exceed 1:1 unless the nominal unit shear capacity is multiplied by the Aspect Ratio Factor (Seismic) = 0.1+0.9b/h. The value of the Aspect Ratio Factor (Seismic) shall not be greater than 1.0. For design to resist wind forces, the shear wall aspect ratio shall not exceed 1:1 unless the nominal unit shear capacity is multiplied by the Aspect Ratio Factor (Wind) = 1.09-0.09b/h. The value of the Aspect Ratio Factor (Wind) shall not be greater than 1.0.
Individual Full-Height Shear Walls

- SDPWS Figure 4E – Typical Individual Full-Height Wall Segments Height-to-Width Ratio
  - $h$ measured as length of full height segment; $b_s$ measured as sheathed width
  - $h:b_s$ must not exceed 3.5:1
  - aspect ratio reductions apply for wood structural panel where $h:b_s$ exceeds 2:1

Force Transfer Around Openings

- SDPWS Figure 4F – Typical Shear Wall Height-to-Width Ratio for Shear Walls Designed for Force Transfer Around Openings
  - $h$ measured as clear height of segment adjacent to opening; $b_s$ measured as sheathed width
  - $h:b_s$ must not exceed 3.5:1
  - aspect ratio reductions apply for wood structural panel where $h:b_s$ exceeds 2:1
Perforated Shear Walls

- SDPWS Figure 4G – Typical Shear Wall Height-to-Width Ratio for Perforated Shear Walls
  - \( h \) measured as length of full height segment; \( b_s \) measured as sheathed width
  - \( h:b_s \) must not exceed 3.5:1
  - aspect ratio reductions apply for perforated shear wall where \( h:b_s \) exceeds 2:1

Resisting Concrete and Masonry Wall Forces

- 4.1.5 Wood-frame shear walls, wood-frame diaphragms, trusses, and other wood members and systems shall not be used to resist seismic forces contributed by masonry or concrete walls in structures over one story in height.

  Exceptions:
  - 1) Wood floor and roof members shall be permitted to be used in diaphragms and horizontal trusses to resist horizontal seismic forces contributed by masonry or concrete walls provided such forces do not result in torsional force distribution through the diaphragm or truss.
  - 2) Vertical wood structural panel sheathed shear walls shall be permitted to be used to provide resistance to seismic forces contributed by masonry or concrete walls in two-story structures, provided the following requirements are met:
Resisting Concrete and Masonry Wall Forces

• (Cont.)...provided the following requirements are met:
  • a. Story-to-story wall heights shall not exceed 12 ft.
  • b. Diaphragms shall not be considered to transmit lateral forces by torsional force distribution or cantilever past the outermost supporting shear wall.
  • c. Combined deflections of diaphragms and shear walls shall not permit design story drift of supported masonry or concrete walls to exceed the allowable story drift in accordance with Section 12.12.1 of ASCE 7.
  • d. Wood structural panel diaphragms shall be blocked diaphragms.
  • e. Wood structural panel shear walls shall be blocked shear walls and, for the lower story, the sheathing shall have a minimum thickness of 15/32 in.
  • f. There shall be no out-of-plane horizontal offsets between the first and second stories of wood structural panel shear walls.

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### Outline & Schedule

**Wednesday, September 24, 2014**

| 1:00 pm - 2:00 pm | Introduction and Chapter Overview of the 2012 WFCM |
| 2:00 pm - 2:15 pm | Break |
| 2:15 pm - 5:00 pm | Introduction and Chapter Overview continued, and Design of Wood Frame Buildings for High Wind, Snow, and Seismic Loads (WFCM Workbook) continued |

**Thursday, September 25, 2014**

| 8:00 am - 10:00 am | Design of Wood Frame Buildings for High Wind, Snow, and Seismic Loads (WFCM Workbook) continued |
| 10:00 am - 10:15 am | Break |
| 10:15 am - 11:00 am | Shear Wall Design Examples |
| 11:00 am - 11:45 am | Lunch at Hotel Roanoke |
| 12:00 pm - 1:00 pm | Design for High Wind, Snow, and Seismic Loads continued |
| 1:00 pm - 1:15 pm | Lunch at Hotel Roanoke |
| 1:15 pm - 3:15 pm | Break |
| 3:15 pm - 5:00 pm | Design for High Wind, Snow, and Seismic Loads continued |

**Friday, September 26, 2014**

| 8:00 am - 9:00 am | Permanent Bracing for Residential Metal-Roof-Connected Wood Trusses |
| 9:00 am - 9:45 am | Break |
| 9:45 am - 10:30 am | Specifying and Constructing Floors to Accommodate Ceramic Tile and Stone |
| 10:30 am - 11:45 am | New Prescriptive Residential Wood Deck Construction Guide (DCA 6 - 2012 IRC Version) |
| 11:45 am - 12:00 pm | Course Evaluation and Presentation of Certificates |

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