Description

Per the International Building Code (IBC), structures using wood shear walls and diaphragms to resist wind and seismic lateral loads shall be designed and constructed in accordance with AWC’s Special Design Provisions for Wind and Seismic (SDPWS). This course will discuss the 2015 SDPWS which is a dual format document with both allowable stress design (ASD) and load and resistance factor design (LRFD). In this course, participants will learn about format of the SDPWS and how to apply design provisions to shear walls and diaphragms as well as changes from previous editions.
Learning Objectives

On completion of this course, participants will:

• Be able to understand load path basics and how it applies to wood structural design
• Be familiar with the significant changes between the 2008 and 2015 SDPWS
• Be able to identify lateral resisting systems and understand where to obtain design specifications for these systems
• Be able to analyze format and content within the 2015 SDPWS

Polling Question

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

- Lateral Load Basics
- Code Acceptance
- 2008/2015 SDPWS Overview
- 2015 SDPWS Summary

Load Path

- Wind
- Seismic (Earthquake) Motion
- IBC Section 1604.4 requires complete load path

Image courtesy APA – The Engineered Wood Association 2003
Lateral Loads

- Wind Load Path
- Diaphragm SUPPORTS out-of-plane walls
- In-plane walls SUPPORT diaphragm

General Lateral Load Path

Roof (horizontal diaphragm) carries load to end walls.

Wind load, \( F \) (lb per sq ft)

Side wall carries load to roof diaphragm at top, and to foundation at bottom.

End wall (vertical diaphragm or shear wall) carries load to foundation.

\[
\begin{align*}
v &= \frac{wL}{2b} \\
w &= \frac{Fh}{2} \\
T &= C = vh
\end{align*}
\]
Load Path

From Diaphragm to Shear Walls

- Seismic Design Category (C, D, E, & F)
- Diaphragms shall be provided with continuous ties or struts between diaphragm chords to distribute these anchorage forces into the diaphragms. (ASCE 7 - 10 sec. 12.11.2.2.1)
Lateral Structural Elements

- **Diaphragm**
- **Collector beam (or drag strut)**
- **Shear wall**
- **Drag strut (or collector)**

Rigid v. Flexible Diaphragm

- **Importance**
  - Distribution of load to shear wall

- **Flexible**
  - Diaphragm load is distributed to shear walls by tributary area

- **Rigid**
  - Diaphragm load is distributed to shear walls by wall stiffness and torsional component
Flexible v. Rigid

\[
\begin{array}{ccc}
\text{Stiffness} & 2K & 2K \\
\text{Flexible} & 0.25wL & 0.25wL \\
\text{Rigid (no Torsion)} & 0.40wL & 0.40wL \\
\end{array}
\]

Drag Struts and Collectors

- "Collects" diaphragm load and "drags" it back to a shear wall
- Occur most frequently at junction of diaphragm and shear wall
Collector Beam in Floor

• For “wall continuity”

Drag Strut in Wall

• Usually the top plate serves as collector and diaphragm chord
• Might be window or door header
Shear Wall - Parts

- Five parts of a shear wall

1. wood frame
2. wood structural panels
3. nails
4. plate anchors
5. hold downs

Shear Wall Test

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

Shear Wall - Types

Individual Full-Height Wall Segments
• No openings within individual full-height segments

Force Transfer Shear Walls
• With openings, but framing members, blocking, and connections around openings are designed for force-transfer

Perforated Shear Walls
• With openings, but reduced shear strength is used based on size of openings
Code Acceptance of Standard

• 2012 IBC
  • References 2008 SDPWS in Section 2305 for lateral design and construction
• 2015 IBC
  • References 2015 SDPWS in Section 2305 for lateral design and construction

General Overview

• Scope
  • “The provisions of this document cover materials, design and construction of wood members, fasteners, and assemblies to resist wind and seismic forces.”
  • ASD and LRFD
General Overview

Outline

• Chapter 1: Flowchart
• Chapter 2: General Design Requirements
• Chapter 3: Members and Connections
• Chapter 4: Lateral Force Resisting Systems

Chapter 1 – Designer Flowchart
Chapter 1 – Designer Flowchart

Chapter 2 – General Requirements

GENERAL DESIGN REQUIREMENTS

2.1 General 4
2.2 Terminology 4
2.3 Notation 6
Chapter 2 – General Requirements

• General

2.1.3 Sizes

Wood product sizes are stated in terms of standard nominal, standard net, or special sizes. For wood structural panels produced in accordance with PS 1 or PS 2, use of the term “nominal panel thickness” in this standard refers to the “Performance Category” value for these products.
Chapter 2 – General Requirements

• Terminology

New **Definitions**

**OPEN FRONT STRUCTURE.** A structure in which any diaphragm edge cantilevers beyond vertical elements of the lateral force-resisting system.

**SUBDIAPHRAGM.** A portion of a diaphragm used to transfer wall anchorage forces to diaphragm cross ties.

• Flexible and Rigid Diaphragm removed

Chapter 2 – Design Methodologies

2.1.2.1 Allowable Stress Design: Allowable stress design (ASD) shall be in accordance with the National Design Specification® (NDS®) for Wood Construction (ANSL/ACI NDS) and provisions of this document.

2.1.2.2 Strength Design: Load and resistance factor design (LRFD) of wood structures shall be in accordance with the National Design Specification® (NDS®) for Wood Construction (ANSL/ACI NDS) and provisions of this document.
Chapter 3 - Members and Connections

• Framing
• Sheathing
• Connections

• Covers out-of-plane wind load resistance of shear walls and diaphragms
Chapter 3 - Members and Connections

- Framing – walls
  - Accounts for composite action
    - Strength and Stiffness
    - Applies now to EI
  - Up to 24” oc
  - Extension of 1.15 repetitive member factor, $C_r$

<table>
<thead>
<tr>
<th>Stud Size</th>
<th>System Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>2x4</td>
<td>1.50</td>
</tr>
<tr>
<td>2x6</td>
<td>1.35</td>
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<td>2x8</td>
<td>1.25</td>
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<tr>
<td>2x10</td>
<td>1.20</td>
</tr>
<tr>
<td>2x12</td>
<td>1.15</td>
</tr>
</tbody>
</table>

Table 3.1.1.1 Wall Stud Repetitive Member Factors

Chapter 3 - Members and Connections

- Sheathing capacities - walls

Revised

Table 3.2.1 Nominal Uniform Load Capacities (psf) for Wall Sheathing Resisting Out-of-Plane Wind Loads

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Chapter 3 - Members and Connections

• Sheathing capacities – roof

Revised

Table 3.3.2  Nominal Uniform Load Capacities (psf) for Roof Sheathing Resisting Out-of-Plane Wind Loads

<table>
<thead>
<tr>
<th>Sheathing Type</th>
<th>Span Rating per Grains</th>
<th>Minimum Thickness (in.)</th>
<th>Strength-Axis Applied Perpendicular to Supports</th>
<th>Strength-Axis Applied Parallel to Supports</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wood Structural Finsars (Sheathing Grades: C, C+, C++, C++)</td>
<td>24/16</td>
<td>3/8</td>
<td>435</td>
<td>540</td>
</tr>
<tr>
<td>Wood Structural Finsars (Sheathing Grades: C, C+, C++, C++)</td>
<td>24/16</td>
<td>7/16</td>
<td>5/8</td>
<td>305</td>
</tr>
<tr>
<td>Wood Structural Finsars (Sheathing Grades: C, C+, C++, C++)</td>
<td>16/16</td>
<td>7/16</td>
<td>5/8</td>
<td>305</td>
</tr>
</tbody>
</table>

Chapter 3 - Members and Connections

• Uplift Resisting Systems

New

3.4 Uplift Force Resisting Systems

3.4.1 General

The proportioning, design, and detailing of engineered wood systems, members, and connections resisting wind uplift shall be in accordance with the reference documents in 3.1.2 and in accordance with 3.4.2, which are continuous load path, or paths, with adequate strength and stiffness to transfer all forces from the point of application to the final point of resistance.

3.4.2 Design Requirements

Uplift force resisting systems shall comply with the following:

1. Metal connectors, continuous tie rods, or other similar connection devices used in the wind uplift load path shall be of adequate strength and stiffness to transfer induced forces to supporting elements.

2. The design strength and stiffness of wood members and connections used in combination with metal connectors, continuous tie rods, or other similar connection devices shall be determined in accordance with 3.3.

3. Where wind uplift load path connections are not aligned from point of load application to point of resistance, additional forces and deflections resulting from such eccentricities shall be accounted for in the design of supporting load path elements.

Exception: Walls sheathed with wood structural panel sheathing or siding that are designed to resist uplift from wind, or combined shear and uplift from wind shall be in accordance with 4.4.
Chapter 3 - Members and Connections

3.4 Uplift Force
Resisting Systems

New

Chapter 4 - Lateral Force-Resisting Systems
Chapter 4 - Lateral Force-Resisting Systems

• General
• Wood Diaphragms
• Wood Shear Walls

• Covers in-plane wind and seismic load resistance of shear walls and diaphragms
4.1.5.1 Anchorage of Concrete or Masonry Walls to Diaphragm

- SDC C, D, E, or F

New

4.1.5.1 Anchorage of Concrete or Masonry Structural Walls to Diaphragms: In Seismic Design Categories C, D, E, or F, diaphragms shall be provided with continuous ties or struts between diaphragm chords to distribute concrete or masonry structural wall anchorage forces in accordance with Section 12.11.2 of ASCE 7 into the diaphragm. Subdiaphragms shall be permitted to be used to transmit the anchorage forces to the main continuous cross-ties. The maximum length-to-width ratio of the structural subdiaphragm shall be 2.5:1. Connections and anchorages capable of resisting the prescribed forces shall be provided between the diaphragm and the attached components.

4.1.5.1.1 Anchorages shall not be accomplished by use of nails subject to withdrawal or toe-nails nor shall wood ledgers or framing be used in cross-grain bending or cross-grain tension.

4.1.5.1.2 The diaphragm sheathing shall not be considered effective as providing the ties or struts required by this section.
Chapter 4 – Anchorage of Concrete or Masonry Walls

- Pre-fabricated roof sections lifted into place
- Safety, cost, speed

Two common types
- All wood
- Hybrid

Source: WoodWorks
Chapter 4 - Lateral Force-Resisting Systems

- Wood Diaphragms

Polling Question

2. The 2015 SDPWS provisions for anchoring concrete or masonry walls to wood diaphragms:
   a. Require use of continuous ties or subdiaphragms to maintain load path
   b. Apply to Seismic Design Categories A through F
   c. Are not referenced in the IBC
   d. None of the above
Chapter 4 - Lateral Force-Resisting Systems

• Wood Diaphragms

4.2.2 Deflection

Calculations of diaphragm deflection shall account for bending and shear deflections, fastener deformation, chord splice slip, and other contributing sources of deflection.

The diaphragm deflection, $\delta_{d,e}$, shall be permitted to be calculated by use of the following equation:

$$\delta_{d,e} = \frac{5vL^3}{8EAW} + \frac{0.25vL}{1000G_y} + \sum \frac{(x\Delta_y)}{2W}$$  \hspace{1cm} (4.2-1)

Chapter 4 - Lateral Force-Resisting Systems

• Wood Diaphragms

$$G_y = \frac{v}{G_{y,t} + 0.75\varepsilon_y}$$  \hspace{1cm} [3]

where:
- $G_{y,t}$ = panel shear stiffness, lb/inch of panel depth;
- $\varepsilon_y$ = nail slip, inches; and
- $v = 1.4$ times the ASD unit shear value of the shear wall or diaphragm, plf.

Figure C4.3.2 Comparison of 4-Term and 3-Term Deflection Equations
Chapter 4 - Lateral Force-Resisting Systems

- Wood Diaphragms
  - Deflection example
  - 2008 SDPWS Commentary

EXAMPLE C4.2.2-3 Calculate Mid-Span Diaphragm Deflection

Chapter 4 – Nominal Design Value

- Wind nominal unit shear capacity $v_w$
  Legacy IBC allowable stress design value x 2.8
- Seismic nominal unit shear capacity $v_s$
  $v_s = v_w / 1.4$

Table 4.2A Nominal Unit Shear Capacities for Wood-Frame Diaphragms
Chapter 4 – Design Value Format

Nominal design values tabulated for diaphragms

- **ASD**
  - reduction factor (2.0)

- **LRFD**
  - resistance factor $\phi$ (0.80)

4.2.3 Unit Shear Capacities

Tabulated nominal unit shear capacities for seismic design are provided in Column A of Tables 4.2A, 4.2B, 4.2C, and 4.2D, and for wind design in Column B of Tables 4.2A, 4.2B, 4.2C, and 4.2D. The ASD allowable unit shear capacity shall be determined by dividing the tabulated nominal unit shear capacity, modified 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 0.80. No further increases shall be permitted.

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Chapter 4 - Lateral Force-Resisting Systems

Wood Diaphragms

<table>
<thead>
<tr>
<th>Table 4.2.4 Maximum Diaphragm Aspect Ratios</th>
</tr>
</thead>
<tbody>
<tr>
<td>(Horizontal or Sloped Diaphragms)</td>
</tr>
<tr>
<td>Diaphragm Sheathing Type</td>
</tr>
<tr>
<td>--------------------------</td>
</tr>
<tr>
<td>Wood structural panel, unblocked</td>
</tr>
<tr>
<td>Wood structural panel, blocked</td>
</tr>
<tr>
<td>Single-layer straight lumber sheathing</td>
</tr>
<tr>
<td>Single-layer diagonal lumber sheathing</td>
</tr>
<tr>
<td>Double-layer diagonal lumber sheathing</td>
</tr>
</tbody>
</table>
Table 4.2C Nominal Unit Shear Capacities for Wood-Frame Diaphragms

<table>
<thead>
<tr>
<th>Sheathing Grade</th>
<th>Common Nail Size</th>
<th>Minimum Nailing Pattern (in./panel (m))</th>
<th>Minimum Nominal Panel Thickness (in.)</th>
<th>Minimum Nominal Weight of Panel (lb/ft²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td></td>
<td>0d 1-10</td>
<td>510 9.0 6.8</td>
<td>520 1.2 5.0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>7/16 2</td>
<td>510 9.0 6.8</td>
<td>520 1.2 5.0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1/16 3</td>
<td>510 9.0 6.8</td>
<td>520 1.2 5.0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1/16 3</td>
<td>510 9.0 6.8</td>
<td>520 1.2 5.0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1/16 3</td>
<td>510 9.0 6.8</td>
<td>520 1.2 5.0</td>
</tr>
</tbody>
</table>

Chapter 4 – Nominal Design Value

- Diaphragm Configuration Figures
  - Direction with respect to load of
    - Continuous panel joints
    - Framing members
  - Independent of panel orientation

Cases 1 & 3: Continuous Panel Joints Perpendicular to Framing
Cases 2 & 4: Continuous Panel Joints Parallel to Framing
Cases 5 & 6: Continuous Panel Joints Perpendicular and Parallel to Framing

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Chapter 4 – Horizontal Distribution

4.2.5 Horizontal Distribution of Shear

- Idealized as Flexible
  - ASCE 12.3.1.1, or
  - $\Delta_{\text{DIAPHRAGM}} > 2 \times \Delta_{\text{SHEARWALLS}}$
  - tributary area
- Idealized as Rigid
  - ASCE 12.3.1.2, or
  - $\Delta_{\text{DIAPHRAGM}} \leq 2 \times \Delta_{\text{SHEARWALLS}}$
  - relative lateral stiffness of vertical LFRS
- Semi-rigid – complex analysis or “envelope” (NEW)

Chapter 4 - Lateral Force-Resisting Systems

4.2.5.1 Torsional Irregularity

- SDC A - Exempt
- Rigid or Semi-rigid
- WSP diaphragms L/W < 1.5:1
- Diagonal Lumber (single or double layer) L/W < 1:1
4.2.5.1 Torsional Irregularity

- Torsional Irregular
- Story Drift $\Delta_{A_{max}} > 1.2 \Delta_{A \& B \, \text{Average}}$

\[ \Delta_{A_{max}} = \frac{\Delta_A + \Delta_B}{2} ; \quad \Delta_{A \& B \, \text{Average}} = \frac{\Delta_A + \Delta_B}{2} \]

Figure 12.8-1 Torsional Amplification Factor, $A_t$
4.2.5.2 Open Front Structures

- Not Torsionally Irregular
  - WSP diaphragms L’/W’ < 1.5:1
  - Diagonal Lumber (single or double layer) L’/W’ < 1:1

- Torsionally Irregular
  - > 1-story L’/W’ < 0.67:1
  - 1-story L’/W’ < 1:1

Load parallel to opening - model as semi-rigid or rigid

\[ \Delta_{A_{\text{max}}} < \text{ASCE 7 allowable story drift} \]

- \( L' \leq 35' \)

Exception: Cantilever < 6' beyond nearest vertical LFRS need not comply to 4.2.5.2.
Chapter 4 – Open Front Structures

Revised

4.2.5.2.1 Open Front Structures - 1 story
- \( L' \leq 25' \)
- \( L'/W' \leq 1:1 \)
- Idealized as rigid - distribution of torsional shear

\[ \text{Plan Views} \]

\[ \text{Sheet Wall} \]

\[ \text{Cantilevered Diaphragm} \]

\[ \text{Open Front} \]

\[ L' \]

\[ \text{w' 40'} \]

\[ \text{w' 20'} \]

Chapter 4 – High Load Diaphragms

- Consistent with IBC 2006
- Includes apparent shear stiffness \((G_a)\)
- Table 4.2B Nominal Unit Shear Capacities for Blocked Wood-Frame Diaphragms Utilizing Multiple Rows of Fasteners (High Load Diaphragms)

---

### Table 4.2B Nominal Unit Shear Capacities for Wood-Frame Diaphragms

<table>
<thead>
<tr>
<th>Sheathing Type</th>
<th>Minimum Fastener Spacing (inches)</th>
<th>Number of Fasteners (inches)</th>
<th>Width (B_w) (inches)</th>
<th>Shear (V_{u,n}) (inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Structural</td>
<td>5/8</td>
<td>5/8</td>
<td>L' x W'</td>
<td></td>
</tr>
</tbody>
</table>
Chapter 4 – High Load Diaphragms

Blocked Diaphragm Configuration Figures

- Direction with respect to load of
  - Continuous panel joints
  - Framing members
  - Independent of panel orientation

High Load Diaphragms

- 4.2.7.1.2 rules for construction
- Need 3" or greater nominal members
- Requires Special Inspection
4.2.7.1.2 High Load Blocked Diaphragms

4. The depth of framing members and blocking into which the nail penetrates shall be 3" nominal or greater.

4–5. The width of the nailed face of framing members and blocking at boundaries and adjoining panel edges shall be 3" nominal or greater. The width of the nailed face not located at boundaries or adjoining panel edges shall be 2" nominal or greater.
Chapter 4 – High Load Diaphragms

Avoid Nail Splitting

Polling Question

3. Open front structures are limited to a maximum diaphragm length of _____ in the 2015 SDPWS.
   a. 25 feet
   b. 35 feet
   c. 50 feet
   d. Open front structures are not allowed per SDPWS
Chapter 4 - Lateral Force-Resisting Systems

- Wood Shear Walls

4.3.2 Deflection

Calculations of shear wall deflection shall account for bending and shear deflections, fastener deformation, anchorage slip, and other contributing sources of deflection.

The shear wall deflection, $\delta_{sw}$, shall be permitted to be calculated by use of the following equation:

$$\delta_{sw} = \frac{8vfh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b} \quad (4.3-1)$$
Chapter 4 - Lateral Force-Resisting Systems

Structural Fiberboard Shear Walls

NEW

4.3.2.3 Deflection of Structural Fiberboard Shear Walls: For a structural fiberboard shear wall with an aspect ratio \((h/b_r)\) greater than 1.0, the deflection obtained from equation 4.3-1 shall be multiplied by \((h/b_r)^{1.2}\).

Chapter 4 – Nominal Design Value

- Wind nominal unit shear capacity \(v_w\)
  - Legacy IBC allowable stress design value \(\times 2.8\)
- Seismic nominal unit shear capacity \(v_s\)
  - \(v_s = v_w / 1.4\)

Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls1,3,5,7

<table>
<thead>
<tr>
<th>Sheathing Material</th>
<th>Minimum Panel Thickness (in)</th>
<th>Minimum Panel Span (in)</th>
<th>Factor Type &amp; Grade</th>
<th>Panel Edge Factor (Option 1)</th>
<th>Panel Edge Factor (Option 2)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wood-based Panels</td>
<td>1/8</td>
<td>16</td>
<td>A</td>
<td>6.5</td>
<td>6.5</td>
</tr>
<tr>
<td></td>
<td>3/32</td>
<td>16</td>
<td>B</td>
<td>3.5</td>
<td>3.5</td>
</tr>
<tr>
<td></td>
<td>7/64</td>
<td>16</td>
<td>C</td>
<td>2.6</td>
<td>2.6</td>
</tr>
</tbody>
</table>

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Chapter 4 – Design Value Format

- Nominal design values tabulated for shear walls
  - ASD
    - reduction factor (2.0)
  - LRFD
    - resistance factor $\phi$ (0.80)

4.3.3 Unit Shear Capacities

The ASD allowable unit shear capacity shall be determined by dividing the tabulated nominal unit shear capacity, modified 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 0.80. No further increases shall be permitted.

Chapter 4 – Nominal Unit Shear Capacities

- ASD
  - Wind $v_w / 2.0$
  - Seismic $v_s / 2.0$

- LRFD
  - $\phi = 0.8$
    - Wind $v_w \times 0.8$
    - Seismic $v_s \times 0.8 = v_w \times 0.8 / 1.4 = v_w \times 0.57$

- LRFD has up to 12% strength benefit for seismic design for shear when using the following IBC load factors
  - LRFD: 1.0E or 1.6W
  - ASD: 0.7E or 1.0W
Ch. 4 - Shear Walls Sheathed on 2 Sides

• Provisions for shear walls sheathed on two sides
  • Table 4.3A Footnote 6

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

<table>
<thead>
<tr>
<th>Wood-based Panel*</th>
<th>Wood Framing Type &amp; Store</th>
<th>Panel Edge/Framing Spacing (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>6</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Left</td>
</tr>
</tbody>
</table>
| Nominal Core 
Type & Thickness | Minimum | Minimum | Maximum | Maximum | Maximum | Maximum |
| 3/4" x 1/2" HPL | 1.0 | 1.5 | 1.2 | 1.7 | 1.0 | 1.5 |
| 3/4" x 1/2" OSB | 1.0 | 1.5 | 1.2 | 1.7 | 1.0 | 1.5 |
| 1-1/2" OSB | 1.0 | 1.5 | 1.2 | 1.7 | 1.0 | 1.5 |

6. Where panels are applied on both faces of a shear wall and nail spacing is less than 6" 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" nominal or greater at adjoining panel edges and nails at all panel edges shall be staggered.
Section 4.3.3.2
- Nails 6" panel edge spacing
- Up to 2:1 aspect ratio
- 16' height limit
- Based on cyclic testing
- Shear capacity reduction

### Table 4.3.3.2 Unblocked Shear Wall Adjustment Factor, $C_{ub}$

<table>
<thead>
<tr>
<th>Nail Spacing (in.)</th>
<th>Stud Spacing (in.)</th>
</tr>
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<tbody>
<tr>
<td>Supported Edges</td>
<td>Intermediate Framing</td>
</tr>
<tr>
<td>6</td>
<td>6</td>
</tr>
<tr>
<td>6</td>
<td>12</td>
</tr>
</tbody>
</table>

Chapter 4 - Unblocked Shear Walls

**Deflection (4.3.2.2)**
- Less stiffness
- Deflection amplified by $C_{ub}$

Fig. 3. Shear wall test setup
Chapter 4 - Lateral Force-Resisting Systems

• Summing Shear Capacities - Dissimilar Materials on Opposite Sides

• 4.3.3.3.2 ...combined nominal unit shear capacity, \( (v_{sc}) \) or \( (v_{wc}) \), shall be either two times the smaller nominal unit shear capacity or the larger nominal unit shear capacity, whichever is greater.
• Exception: combined unit shear capacity for wind is additive: WSP + Gypsum

Chapter 4 - Lateral Force-Resisting Systems

4.3.3.4 Shears Walls in a Line: same materials and construction

4.3.3.4.1 - Individual full height shear walls provide all same deflection, \( \delta_{sw} \)
4.3.3.4 Shears Walls in a Line: same materials and construction

4.3.3.4.1 - Individual full height shear walls provide all same deflection, $\delta_{sw}$

Exception:
- WSP $h/b_w > 2:1$ $v_s \times 2b_s/h$
- Fiberboard $h/b_w > 1:1$ $v_s \times (0.1 + 0.9b_s/h)$
- Shear distribution proportional to capacities
- Shear capacity reduction not combined with aspect ratio adjustment (4.3.4.2)

<table>
<thead>
<tr>
<th>$h/b_w$</th>
<th>$2:1$ unless $v_s = 2(b_s/h)$</th>
</tr>
</thead>
<tbody>
<tr>
<td>2:1</td>
<td>3:1</td>
</tr>
<tr>
<td>1.00</td>
<td>0.67</td>
</tr>
<tr>
<td>3½:1</td>
<td>0.57</td>
</tr>
</tbody>
</table>

Table 4.3.4 Maximum Shear Wall Aspect Ratios

<table>
<thead>
<tr>
<th>Shear Wall Sheathing Type</th>
<th>Maximum $h/b_w$ 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 Wall having aspect ratios exceeding 3:5:1 shall be blocked shear walls.
Chapter 4 – Aspect Ratios & Capacity Adjustments

4.3.4.2 – Shear Wall Aspect Ratio Factors

- \( \frac{h}{b_s} > 2:1 \) WSP
  - \( v_s \times (1.25 - 0.125\frac{h}{b_s}) \)
- \( \frac{h}{b_s} > 1:1 \) Struct. Fiberboard
  - \( v_s \times (1.09 - 0.09\frac{h}{b_s}) \)

<table>
<thead>
<tr>
<th>Aspect Ratio</th>
<th>Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>2:1</td>
<td>1.00</td>
</tr>
<tr>
<td>3:1</td>
<td>0.875</td>
</tr>
<tr>
<td>3½:1</td>
<td>0.813</td>
</tr>
</tbody>
</table>

2:1 unless \( v_a = (1.25 - 0.125(h/b_s)) \)

4.3.4.3 – Perforated Shear Walls

- \( \frac{h}{b_s} > 3.5:1 \) Not considered
- \( \frac{h}{b_s} > 2:1 \) \( L_i = L(2b_s/h) \)
- Aspect Ratio Factors (4.3.4.2) do not apply
- Shear distribution exceptions (4.3.3.4.1) do not apply

Note: \( b_s \) is the minimum shear wall segment length, \( b \) in the perforated shear wall.
Chapter 4 – Aspect Ratios & Capacity Adjustments

4.3.4.4 – Force-transfer Shear Walls
• \( h/b_s > 3.5:1 \) Not considered

Chapter 4 – Perforated Shear Walls

• Wood Perforated Shear Walls

Table 4.3.3.4 Shear Capacity Adjustment Factor, \( C \)

<table>
<thead>
<tr>
<th>Wall Height, ( h )</th>
<th>( h/3 )</th>
<th>Maximum Opening Height</th>
<th>( h/2 )</th>
<th>( h )</th>
<th>( h/6 )</th>
<th>( h )</th>
</tr>
</thead>
<tbody>
<tr>
<td>8' Wall</td>
<td>2' - 8'</td>
<td>3' - 4'</td>
<td>5' - 6'</td>
<td>8' - 8'</td>
<td>10' - 10'</td>
<td></td>
</tr>
<tr>
<td>10' Wall</td>
<td>3' - 4'</td>
<td>5' - 6'</td>
<td>6' - 8'</td>
<td>8' - 4'</td>
<td>10' - 6'</td>
<td></td>
</tr>
</tbody>
</table>

Percent Full-Length Sheathing

<table>
<thead>
<tr>
<th>Effective Shear Capacity Ratio</th>
<th>0%</th>
<th>10%</th>
<th>20%</th>
<th>30%</th>
<th>40%</th>
<th>50%</th>
<th>60%</th>
<th>70%</th>
<th>80%</th>
<th>90%</th>
<th>100%</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.00</td>
<td>1.00</td>
<td>0.99</td>
<td>0.95</td>
<td>0.93</td>
<td>0.91</td>
<td>0.87</td>
<td>0.83</td>
<td>0.80</td>
<td>0.77</td>
<td>0.74</td>
<td>0.71</td>
</tr>
</tbody>
</table>

1. The maximum opening height shall be taken as the maximum opening height in a performed shear wall. Where areas above and below openings in the same vertical plane shall be different, the shall be taken as the maximum height of the opening plus the maximum opening.
2. The area of the openings in the performed shear wall shall be added to the total length of the performed shear wall.
Chapter C4 – Perforated Shear Walls

- Commentary Section 4.3.3.5

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

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

- Alternative to tabulated values – Section 4.3.3.5
  - Allows more efficient designs
  - Actual area of openings
  - Table requires maximum opening size
  - Example: 1 door & 2 windows
  - Table assumes windows are same height as door
  - Takes away panel shear area
Chapter 4 - Perforated Shear Walls

Revised

4.3.5.3 Perforated Shear Walls
- Clarifies WSP permitted one or both sides
- Clarifies WSP + Gypsum on opposite side permitted

Source: APA

Polling Question

4. Maximum allowable shear wall aspect ratios:
   a. Are not included in the 2015 SDPWS
   b. Vary depending on the wall sheathing material
   c. Are independent of wall sheathing material
   d. Vary depending upon shear wall anchorage
Chapter 4 – Construction Requirements

NEW

4.3.6.1.1 Common Framing Members
• 2-2x permitted to replace 3x
  • Fastened together per NDS
  • Spacing <4” o.c. shall be staggered
• Applies broadly to all framing

4.3.7 Shear Wall Systems
• 2-2x permitted to replace 3x
  • Wood Structural Panels (4.3.7.1(5))
  • Particleboard (4.3.7.3(5))

Chapter 4 - Lateral Force-Resisting Systems

4.3.6.3.1 Adhesives: Adhesive Attachment of Shear Wall Sheathing

Table 1.—Adhesive attachment of shear wall sheathing.

<table>
<thead>
<tr>
<th>Seismic design category</th>
<th>Seismic design coefficient</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>SDPWS</td>
</tr>
<tr>
<td>A, B, and C</td>
<td>R = 1.5, Ω₀ = 2.5</td>
</tr>
<tr>
<td>D</td>
<td>Not permitted</td>
</tr>
</tbody>
</table>

SDPWS 4.3.6.3.1 Exception...
Panel Widths for Shear Walls

4.3.7.1...Panels shall not be less than 4’x8’, except at boundaries and changes in framing where 24” is allowed unless framing members or blocking is provided at the edges of all panels.

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
Chapter 4 - Shear Wall Anchorage – 3” x 3” Default

Shear wall anchorage provisions at foundation – Section 4.3.6.4.3

- 3” x 3” x 0.229” steel
- slotted hole permitted
- placed within ½” of sheathing material
- automatically satisfied for 2x4 plate

Chapter 4 - Shear Wall Anchorage – 3” x 3” Default

Shear wall anchorage provisions at foundation – Section 4.3.6.4.3

- Exception: Standard cut washers permitted
- Anchor bolts designed to resist shear only
- Hold downs designed for uplift neglecting DL
- Aspect ratio ≤ 2:1
- Limited nominal shear wall capacities
  - ≤ 980 plf seismic
  - ≤ 1370 plf wind
Chapter 4 - WSP Over Gypsum Shear Walls

• Section 4.3.7.2
• Shear wall with fire resistance

Chapter 4 - Combined Wind Uplift & Shear - WSP

• Wood structural panels (WSP)
  • Resist combined wind uplift and shear
  • Resist tension only from wind uplift
  • Alternate to metal straps

---

### Table 4.38 Nominal Unit Shear Capacities for Wood-Frame Shear Walls

<table>
<thead>
<tr>
<th>Sheathing Material</th>
<th>Wood Structural Panels Applied over 3/8&quot; or 5/8&quot; Gypsum Wallboard or Gypsum Sheathing Board</th>
<th>Wood Structural Panels Applied over 3/8&quot; or 5/8&quot; Gypsum Wallboard or Gypsum Sheathing Board</th>
</tr>
</thead>
<tbody>
<tr>
<td>Standing Seam</td>
<td>0.02</td>
<td>0.03</td>
</tr>
<tr>
<td>Metal Straps</td>
<td>0.02</td>
<td>0.03</td>
</tr>
<tr>
<td>Alternate to metal straps</td>
<td>0.02</td>
<td>0.03</td>
</tr>
</tbody>
</table>

---

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Chapter 4 - Combined Wind Uplift & Shear - WSP

- Table 4.4.1 Shear & Uplift
  - Amount of available uplift capacity beyond shear capacity

<table>
<thead>
<tr>
<th>Table 4.4.1 Nominal Uplift Capacity of 7/16&quot; (Nominal) Minimum Wood Structural Panel Sheathing or Siding When Used for Both Shear Walls and Wind Uplift Simultaneously over Framing with a Specific Gravity of 0.42 or Greater</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>Nails- Single Edge</td>
</tr>
<tr>
<td>Nails- Double Row</td>
</tr>
</tbody>
</table>

Chapter 4 - Combined Wind Uplift & Shear - WSP

- Minimum panel thickness = 7/16"
- Vertical sheathing
- Minimum spacing of fasteners in a row = 3"
- Horizontal blocking
- All shear wall types
  - Individual full-height
  - Perforated
  - Force-transfer
Panel Orientation

4.4.1.2 Panels: Panels shall have a minimum nominal panel thickness of 7/16" and shall be installed with the strength axis parallel or perpendicular to the studs.

• Critical details
  • Note minimum edge distance is 1/2"
Chapter 4 - Combined Wind Uplift & Shear - WSP

- Sheathing tension splice

![Diagram of Sheathing Splice Plate (Alternate Detail)]

(Deals with tension perp)

---

**Chapter 4 - Combined Wind Uplift & Shear - WSP**

- Sheathing Splices per 4.4.1.7

<table>
<thead>
<tr>
<th>Number of Stories</th>
<th>No Horizontal Sheathing Joint over Studs</th>
<th>Horizontal Sheathing Joint over Studs</th>
<th>Sheathing Tension Splice over Studs</th>
</tr>
</thead>
<tbody>
<tr>
<td>One-Story</td>
<td>No splice required</td>
<td>• Blocking to resist shear (4.4.1.3)</td>
<td>Resists both shear and uplift (4.4.1.7 Exception)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Stud nailing to resist uplift (4.4.1.7 (2))</td>
<td></td>
</tr>
<tr>
<td>Multi-Story</td>
<td>Nail spacing at common horizontal framing ≥ 3” single row or ≥ 6” double row (4.4.1.7 (1) and Fig. 4H)</td>
<td>• Blocking to resist shear (4.4.1.3)</td>
<td>Resists both shear and uplift (4.4.1.7 Exception)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Stud nailing to resist uplift (4.4.1.7 (2))</td>
<td></td>
</tr>
</tbody>
</table>
Chapter 4 - Combined Wind Uplift & Shear - WSP

• Multi-story – sheets splice at band joist

![Figure 4H Panel Splice Occurring over Horizontal Framing Member]

- Combined
- Uplift
- Shear

Nail spacing limited to address tension perp

6" o.c. minimum
3" o.c. minimum

Chapter 4 - Combined Wind Uplift & Shear - WSP

• Multi-story – sheets splice at stud mid-height

![Figure 4I Panel Splice Occurring across Studs]
Chapter 4 - Combined Wind Uplift & Shear - WSP

- Example: Splice over studs
  - 110 MPH Exposure B
  - Uplift to resist: 277 plf
  - Use (2.0 x 277 = 554 plf) and Table 4.4.1 below

| Table 4.4.1 Nominal Uplift Capacity of 7/16" (Nominal) Minimum Wood Structural Panel Sheathing or Siding When Used for Both Shear Walls and Wind Uplift Simultaneously over Framing with a Specific Gravity of 0.42 or Greater 1 |
|---------------------------------|---------------------------------|---------------------------------|---------------------------------|---------------------------------|---------------------------------|
| Nail Spacing Required for Shear Wall Design | Nail Spacing Required for Shear Wall Design | Nail Spacing Required for Shear Wall Design | Nail Spacing Required for Shear Wall Design | Nail Spacing Required for Shear Wall Design | Nail Spacing Required for Shear Wall Design |
| 6d Common Nail 6d Common Nail 6d Common Nail 6d Common Nail 6d Common Nail | 6d Common Nail 6d Common Nail 6d Common Nail 6d Common Nail 6d Common Nail | 6d Common Nail 6d Common Nail 6d Common Nail 6d Common Nail 6d Common Nail | 6d Common Nail 6d Common Nail 6d Common Nail 6d Common Nail 6d Common Nail | 6d Common Nail 6d Common Nail 6d Common Nail 6d Common Nail 6d Common Nail | 6d Common Nail 6d Common Nail 6d Common Nail 6d Common Nail 6d Common Nail |
| 8" panel edge spacing 9" field spacing 9" field spacing 12" field spacing 12" field spacing | 8" panel edge spacing 9" field spacing 9" field spacing 12" field spacing 12" field spacing | 8" panel edge spacing 9" field spacing 9" field spacing 12" field spacing 12" field spacing | 8" panel edge spacing 9" field spacing 9" field spacing 12" field spacing 12" field spacing | 8" panel edge spacing 9" field spacing 9" field spacing 12" field spacing 12" field spacing | 8" panel edge spacing 9" field spacing 9" field spacing 12" field spacing 12" field spacing |
| Alternate Nail Spacing at Top and Bottom Plate Edges | Alternate Nail Spacing at Top and Bottom Plate Edges | Alternate Nail Spacing at Top and Bottom Plate Edges | Alternate Nail Spacing at Top and Bottom Plate Edges | Alternate Nail Spacing at Top and Bottom Plate Edges | Alternate Nail Spacing at Top and Bottom Plate Edges |
| G" | E" | D" | C" | B" | A" | G" | E" | D" | C" | B" | A" | G" | E" | D" | C" | B" | A" |
| Uplift Capacity (psi) of Wood Structural Panel Sheathing or Siding 1 |
| Single Row 1 | 0 | 180 | 336 | 0 | 216 | 432 | NA | 0 | 216 | 432 | 0 | 262 | 524 |
| Double Row 1 | 336 | 672 | 1008 | 432 | 864 | 1296 | 216 | 648 | 1008 | 524 | 1048 | 1572 |

Chapter 4 - Combined Wind Uplift & Shear - WSP

- Example: Splice over studs
  - Uplift too high to allow multi-story splice to occur over band joist, so design splice over studs

4.4.1.7 Sheathing Splices
1. In multi-story applications where the upper story and lower story sheathing adjoin over a common horizontal framing member, the nail spacing shall not be less than 3" o.c. for a single row or 6" o.c. for a double row in Table 4.4.1 (see Figure 4.11).

| Table 4.4.1 Nominal Uplift Capacity of 7/16" (Nominal) Minimum Wood Structural Panel Sheathing or Siding When Used for Both Shear Walls and Wind Uplift Simultaneously over Framing with a Specific Gravity of 0.42 or Greater 1 |
|---------------------------------|---------------------------------|---------------------------------|---------------------------------|---------------------------------|---------------------------------|
| Nail Spacing Required for Shear Wall Design | Nail Spacing Required for Shear Wall Design | Nail Spacing Required for Shear Wall Design | Nail Spacing Required for Shear Wall Design | Nail Spacing Required for Shear Wall Design | Nail Spacing Required for Shear Wall Design |
| 6d Common Nail 6d Common Nail 6d Common Nail 6d Common Nail 6d Common Nail | 6d Common Nail 6d Common Nail 6d Common Nail 6d Common Nail 6d Common Nail | 6d Common Nail 6d Common Nail 6d Common Nail 6d Common Nail 6d Common Nail | 6d Common Nail 6d Common Nail 6d Common Nail 6d Common Nail 6d Common Nail | 6d Common Nail 6d Common Nail 6d Common Nail 6d Common Nail 6d Common Nail | 6d Common Nail 6d Common Nail 6d Common Nail 6d Common Nail 6d Common Nail |
| 8" panel edge spacing 9" field spacing 9" field spacing 12" field spacing 12" field spacing | 8" panel edge spacing 9" field spacing 9" field spacing 12" field spacing 12" field spacing | 8" panel edge spacing 9" field spacing 9" field spacing 12" field spacing 12" field spacing | 8" panel edge spacing 9" field spacing 9" field spacing 12" field spacing 12" field spacing | 8" panel edge spacing 9" field spacing 9" field spacing 12" field spacing 12" field spacing | 8" panel edge spacing 9" field spacing 9" field spacing 12" field spacing 12" field spacing |
| Alternate Nail Spacing at Top and Bottom Plate Edges | Alternate Nail Spacing at Top and Bottom Plate Edges | Alternate Nail Spacing at Top and Bottom Plate Edges | Alternate Nail Spacing at Top and Bottom Plate Edges | Alternate Nail Spacing at Top and Bottom Plate Edges | Alternate Nail Spacing at Top and Bottom Plate Edges |
| G" | E" | D" | C" | B" | A" | G" | E" | D" | C" | B" | A" | G" | E" | D" | C" | B" | A" |
| Uplift Capacity (psi) of Wood Structural Panel Sheathing or Siding 1 |
| Single Row 1 | 0 | 180 | 336 | 0 | 216 | 432 | NA | 0 | 216 | 432 | 0 | 262 | 524 |
| Double Row 1 | 336 | 672 | 1008 | 432 | 864 | 1296 | 216 | 648 | 1008 | 524 | 1048 | 1572 |
Chapter 4 - Combined Wind Uplift & Shear - WSP

• Example: Splice over studs

- Double row
  8d nails @ 4” o.c.

- Single row
  8d nails @ 4” o.c.

See 4.4.1.7 (1)
### Chapter 4 - Combined Wind Uplift & Shear - WSP

**Example: Splice over studs**

Additional nails (n) required above and below joint in studs to resist uplift capacity (4.4.1.7(2))

For 8d @ 4" & 12" ASD Capacity = 324 plf > 277 plf OK

\[
n = \frac{[324 \times 16/12]}{[67 \times 1.6]} = 4.03 \text{ use 4 nails per stud}
\]

<table>
<thead>
<tr>
<th>Nail Spacing Required for Shear Walls/Visors</th>
<th>Nail Spacing Required for Shear Walls/Visors</th>
<th>Nail Spacing Required for Shear Walls/Visors</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stud Common Nail 6&quot; panel edge spacing 5/16&quot; field spacing</td>
<td>Stud Common Nail 6&quot; panel edge spacing 5/16&quot; field spacing</td>
<td>Stud Common Nail 6&quot; panel edge spacing 5/16&quot; field spacing</td>
</tr>
<tr>
<td>Alternate Nail Spacing at Top and Bottom Panel Edges</td>
<td>Alternate Nail Spacing at Top and Bottom Panel Edges</td>
<td>Alternate Nail Spacing at Top and Bottom Panel Edges</td>
</tr>
<tr>
<td>Use Capacity (plf) of Wood Structural Panel Sheathing on Site</td>
<td></td>
<td></td>
</tr>
<tr>
<td>#Nails, Single Row</td>
<td>#Nails, Double Row</td>
<td>#Nails, Double Row</td>
</tr>
<tr>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>336</td>
<td>336</td>
<td>336</td>
</tr>
<tr>
<td>1080</td>
<td>1080</td>
<td>1080</td>
</tr>
</tbody>
</table>

Check field nailing requirements for stud supporting 4 x 12 panel below the rim joist:

Min. stud length on 4 x 12 panel below rim = 31.875”

#nails/stud = 31.875”/12” + 1 = 3.65 nails = 4 nails

Total # nails required/stud = 4 + 4

= 8 nails

Spacing: 31.875”/(8-1) = 4.55”

Specify 4” o.c. in field
Chapter 4 - Wind Uplift - WSP

Uplift only case
• Single or double row of fasteners
• Tension not shear
• Test verified

Chapter 4 - Combined Wind Uplift & Shear - WSP

• Default anchorage (4.4.1.6)
  • Anchor bolts with 3" x 3" x 0.229" steel plate washer
  • Plate washer within ½" of sheathing face that provides uplift
  • Resist cross grain bending of bottom plate
  • Spacing of anchor bolts per Table 4.4.1.6 New
Chapter 4 - Combined Wind Uplift & Shear - WSP

- New Anchor Bolt Table

Table 4.4.1.6 Maximum Anchor Bolt Spacing (inches) for Combined Shear and Wind Uplift

<table>
<thead>
<tr>
<th>Nail Size</th>
<th>Nominal Uplift Capacity (p/ft)</th>
<th>G-0.50</th>
<th>0</th>
<th>16</th>
<th>32</th>
<th>64</th>
<th>128</th>
<th>256</th>
<th>512</th>
<th>1024</th>
<th>2048</th>
</tr>
</thead>
<tbody>
<tr>
<td>6d common 0.394&quot; (4&quot;)</td>
<td>0</td>
<td>0</td>
<td>40</td>
<td>62</td>
<td>66</td>
<td>92</td>
<td>124</td>
<td>248</td>
<td>248</td>
<td>192</td>
<td>16</td>
</tr>
<tr>
<td>7d common 0.394&quot; (4&quot;)</td>
<td>0</td>
<td>0</td>
<td>40</td>
<td>62</td>
<td>66</td>
<td>92</td>
<td>124</td>
<td>248</td>
<td>248</td>
<td>192</td>
<td>16</td>
</tr>
</tbody>
</table>

- Specific Gravity of framing members
- Not permitted

1. The minimum nominal panel thickness of wall sheathing shall be in accordance with Section 4.4.1.3.
2. Tabulated anchor bolt spacings are for minimum 1/2" diameter "fully threaded" bolts (see NDS Appendix Table L1)
3. This anchor bolt spacing is provided for interpolation purposes.

Chapter 4 - Combined Wind Uplift & Shear - WSP

NEW

4.4.1.6 Sheathing Extending to Bottom Plate or Sill Plate

4.4.1.6(3)
- Anchor bolts end of each plate with minimum end distance
  - 7d end distance
  - < 1/2 tabulated value (Table 4.4.1.6)
  - < 12"
- Exception: Hold-down present
Appendix A

Standard Nails and Cut Washers

Polling Question

5. Shear wall foundation anchor bolt washers must be:
   a. Not less than 0.229"x3"x3" in size
   b. Placed within 1" of the sheathing material
   c. Both a) and b)
   d. None of the above
2015 Wind & Seismic Standard

Top Ten Changes!
1. Repetitive member factor on EI
2. New section on uplift force resisting systems
3. New section on anchorage of concrete and masonry walls to wood diaphragms
4. Updated diaphragm flexibility terminology terms consistent with ASCE 7-10
5. Consolidated provisions for the design of open front structures and cantilevered diaphragms
6. High load diaphragms include minimum depth for framing and blocking
7. Clarified substitution of 2-2x for 3x framing
8. Updated provisions for distribution of shear to shear walls in a line to clearly address stiffness compatibility
9. New reduction factor for high aspect ratio walls
10. New method to account for high aspect ratio for perforated shear walls
11. New anchor bolt table for combined shear and wind uplift

Wind & Seismic Standards

- More details on changes
- Wood Design Focus papers
  - 2008 Special Design Provisions for Wind and Seismic
  - Use of Wood Structural Panels to Resist Combined Shear and Uplift from Wind
- Structure Magazine
  - 2015 SDPWS July 2015
- Download free at www.awc.org
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