Course Description

This presentation highlights 2012 International Building Code (IBC), 2010 Minimum Design Loads for Buildings and Other Structures (ASCE 7-10) and the 2008 Special Design Provisions for Wind and Seismic (SDPWS) requirements applicable to the seismic design of wood structures. Wood-frame shear wall and diaphragm code issues are discussed including deflection equations, detailing requirements, and limitations on the use of wood in seismic design. Changes from previous codes and standards will also be discussed and additional resources will be referenced.

Learning Objectives

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

1. Identify IBC code requirements for seismic design of wood structures.
2. Identify ASCE 7-10 and SDPWS for seismic design of wood structures.
3. Understand the limitations of wood shear walls and diaphragms.
4. Identify special seismic detailing requirements for wood structures.
Overview

- 2012 International Building Code (IBC)
- ASCE 7-10 Seismic Provision for Wood Frame
  - Defined systems, seismic base shear, and allowable story drift limits
- Special Design Provisions for Wind and Seismic (SDPWS)
  - Shear wall and diaphragm construction details, strength, stiffness, and limitations

NeesWood Capstone – six story wood frame
SECTION 2306
ALLOWABLE STRESS DESIGN

2306.1 Allowable stress design. The design and construction of wood elements in structures using allowable stress design shall be in accordance with the following applicable standards:

American Forest & Paper Association.

NDS National Design Specification for Wood Construction

SDPWS Special Design Provisions for Wind and Seismic

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 AF&PA SDPWS.

SECTION 1613
EARTHQUAKE LOADS

1613.1 Scope. Every structure, and portion thereof, including nonstructural components that are permanently attached to structures and their supports and attachments, shall be designed and constructed to resist the effects of earthquake motions in accordance with ASCE 7, excluding Chapter 14 and Appendix 11A. The seismic design category for a structure is permitted to be determined in accordance with Section 1613 or ASCE 7.
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

Lateral Design

12.2 STRUCTURAL SYSTEM SELECTION

12.2.1 Selection and Limitations
The basic lateral and vertical seismic force-resisting system shall conform to one of the types indicated in Table 12.2-1 or a combination of systems as permitted in Sections 12.2.2, 12.2.3, and 12.2.4. Each type is subdivided by the types of vertical elements used to resist lateral seismic forces. The structural system used shall be in accordance with the structural system limitations and the limits on structural height, \( h_w \), contained in Table 12.2-1. The appropriate response modification coefficient, \( R \), overstrength factor, \( \Omega_0 \), and the deflection amplification factor, \( C_d \), indicated in Table 12.2-1 shall be used in determining the base shear, element design forces, and design story drift.

Each selected seismic force-resisting system shall be designed and detailed in accordance with the specific requirements for the system as set forth in the applicable reference document listed in Table 12.2-1 and the additional requirements set forth in Chapter 14.
Lateral Design

- Special Design Provisions for Wind and Seismic, SDPWS 2008

2.1.1 Scope
The provisions of this document cover materials, design and construction of wood members, fasteners, and assemblies to resist wind and seismic forces.

General Overview

- Chapter 1: Flowchart
- Chapter 2: General Design Requirements
- Chapter 3: Members and Connections (wind out-of-plane)
- Chapter 4: Lateral Force Resisting Systems (wind & seismic in-plane)
  - General
  - Wood-Frame Diaphragms
  - Wood-Frame Shear Walls
  - Wood Structural Panels Designed to Resist Combined Shear and Up Lift from Wind
Members and Connections


Overview

- 2012 International Building Code (IBC)
- ASCE 7-10 Seismic Provision for Wood Frame
  - Defined systems, seismic base shear, and allowable story drift limits
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  - Shear wall and diaphragm construction details, strength, stiffness, and limitations
2012 IBC

2306.2.1 Wood-frame structural panel diaphragms. Wood-frame structural panel diaphragms shall be designed and constructed in accordance with AF&PA SDPWS. Where panels are fastened to framing members with staples, requirements and limitations of AF&PA SDPWS shall be met and wood structural panel diaphragms are permitted to resist horizontal forces using the allowable shear capacities set forth in Table 2306.2.1(1) or 2306.2.1(2). The allowable shear capacities in Tables 2306.2.1(1) and 2306.2.1(2) are permitted to be increased 40 percent for wind design.

2306.2.2 Single diagonally sheathed lumber diaphragms. Single diagonally sheathed lumber diaphragms shall be designed and constructed in accordance with AF&PA SDPWS.

2306.2.3 Double diagonally sheathed lumber diaphragms. Double diagonally sheathed lumber diaphragms shall be designed and constructed in accordance with AF&PA SDPWS.

2306.2.4 Gypsum board diaphragm ceilings. Gypsum board diaphragm ceilings shall be in accordance with Section 2508.5.

2009 IBC

- **Diaphragm and Shear wall deflection with staples only**
- SDPWS
- **Allowable shear tables - nails & staples**

Diaphragm Deflection Eq. 23-1
\[
\Delta = \frac{5vL^3}{8Eab} + \frac{vL}{4Gtv} + 0.188Lea + \frac{\sum(\Delta X)}{2b}
\]

Shear Wall Deflection Eq. 23-2
\[
\Delta = \frac{8vh^3}{Eab} + \frac{vh}{Gt} + 0.75he_a + \frac{h}{b}d_a
\]
2012 IBC

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

Diaphragm Deflection Eq. 23-1
\[ \Delta = \frac{5vL^3}{8EAb} + \frac{vL}{4Gt_v} + 0.188Le + \frac{\sum(\Delta X)}{2b} \]

Shear Wall Deflection Eq. 23-2
\[ \Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he + \frac{h}{b} \]

2012 IBC

- Staples,
- 40% increase
- NEW ANSI/APA PRP-210 Plywood Siding
  - Durability
  - Thickness by thickness
  - Siding shear walls
2012 IBC

- IBC LRFD per SDPWS
- Detailing requirements:
  - Spacing < 2” oc
  - 10d 1-1/2 penetration < 3” oc
  - > 700 plf
    - 3X or 2-2X adjoining panel edges

ASCE 7-5

12.3 Diaphragm Flexibility Configuration Irregularities, and Redundancy

12.3.1 Diaphragm Flexibility. The structural analysis shall consider the relative stiffnesses of diaphragms and the vertical elements of the seismic force-resisting system. Unless a diaphragm can be idealized as either flexible or rigid in accordance with Sections 12.3.1.1, 12.3.1.2, or 12.3.1.3, the structural analysis shall explicitly include consideration of the stiffness of the diaphragm (i.e., semirigid modeling assumption).

12.3.1.1 Flexible Diaphragm Condition. Diaphragms constructed of unspaced steel decking or wood structural panels are permitted to be idealized as flexible in structures in which the vertical elements are steel or composite steel and concrete frames, concrete masonry, steel, or composite shear walls. Diaphragms of wood structural panels or unspaced steel decks in one- and two-family residential buildings of light-frame construction shall also be permitted to be idealized as flexible.
Lesson 11 Analysis Methods

2012

IBC 2009

1613.6 Alternative to ASCE 7. The provisions of Section 1613.6 shall be permitted as alternatives to the relevant provisions of ASCE 7.

IBC 2012

1613.6.1 Assumptions or Conformance. See the following text at the end of Section 12.3.1.1 of ASCE 7.

Diaphragms constructed of wood structural panels or untreated steel deckings shall also be permitted to be idealized or flexible, provided all of the following conditions are met:

1. Toppings of concrete or similar materials are not placed over wood structural panel diaphragms except for nonstructural toppings not greater than 1/4 inch (6 mm) thick.

2. Each line of vertical elements of the seismic force-resisting system complies with the allowable story drift of Table 12.12.1.

3. Vertical elements of the seismic force-resisting system are light-frame walls sheathed with wood structural panels used for shear resistance in wood shear walls.

4. Portions of wood structural panel diaphragms that continue beyond the vertical elements of the lateral force-resisting system are designed in accordance with Section 4.3.5.2 of AIA/MSA SDPWS.

ASCE 7

12.3 DIAPHRAGM FLEXIBILITY, CONFIGURATION, IRREGULARITIES, AND REDUNDANCY

12.3.1 Diaphragm Flexibility. The structural analysis shall consider the relative stiffnesses of diaphragms and the vertical elements of the seismic force-resisting system. Unless a diaphragm can be idealized as either flexible or rigid in accordance with Sections 12.3.1.1, 12.3.1.2, or 12.3.1.3, the structural analysis shall explicitly include consideration of the stiffness of the diaphragm (i.e., semirigid modeling assumption).

ASCE 7-05

12.3.1.1 Flexible Diaphragm Condition. Diaphragms constructed of untreated steel decking or wood structural panels are permitted to be idealized as flexible in structures in which the vertical elements are steel, concrete, or composite steel and concrete beams, bridges, or concrete, masonry, steel, or composite shear walls. Diaphragms of wood structural panels or untreated steel decks in one- and two-family residential buildings of light-frame construction shall also be permitted to be idealized as flexible.

ASCE 7-10

12.3.1.1 Flexible Diaphragm Condition

Diaphragms constructed of untreated steel decking or wood structural panels are permitted to be idealized as flexible if any of the following conditions exist:

a. In structures where the vertical elements are steel, concrete, or composite steel and concrete beams, bridges, or concrete, masonry, steel, or composite shear walls.

b. In one- and two-family dwellings.

c. In structures of light-frame construction where all of the following conditions are met:

1. Topping of concrete or similar materials is not placed over wood structural panel diaphragms except for nonstructural topping no greater than 1 1/2 in. (38 mm) thick.

2. Each line of vertical elements of the seismic force-resisting system complies with the allowable story drift of Table 12.12.1.
Overview

- 2012 International Building Code (IBC)
- ASCE 7-10 Seismic Provision for Wood Frame
  - Defined systems, seismic base shear, and allowable story drift limits
- Special Design Provisions for Wind and Seismic (SDPWS)
  - Shear wall and diaphragm construction details, strength, stiffness, and limitations
Seismic Force-Resisting Systems

### Table 12.2-1 Design Coefficients and Factors for Seismic Force-Resisting Systems

<table>
<thead>
<tr>
<th>Seismic Force-Resisting System</th>
<th>ASCE 7 Section Where Detailing Requirements Are Specified</th>
<th>Response Modification Coefficient, $R^a$</th>
<th>Overstrength Factor, $Q^d$</th>
<th>Deflection Amplification Factor, $C^f$</th>
<th>Structural Design Category Limit (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>A. BEARING WALL SYSTEMS</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>13. Light-frame (wood) walls</td>
<td>14.5</td>
<td>6½</td>
<td>3</td>
<td>4</td>
<td>NL NL 65 65 65</td>
</tr>
<tr>
<td>with wood structural panels</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>rated for shear resistance</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>17. Light-frame walls with</td>
<td>14.1 and 14.5</td>
<td>2½</td>
<td>2</td>
<td></td>
<td>NL NL 35 NP NP</td>
</tr>
<tr>
<td>shear panels of all other</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>materials</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>**G. CANTILEVERED COLUMN</td>
<td>6. Timber frames</td>
<td>1½</td>
<td>1½</td>
<td>2½</td>
<td>35 35 35 NP NP</td>
</tr>
<tr>
<td>SYSTEMS**</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Approximate Fundamental Period

- Approximate fundamental period:

$$T_a = C_t h_n^x$$  \quad \text{Eq. 12.8-7}

- where;
  - $T_a = $ approximate fundamental period, s
  - $h_n = $ structural height, ft
  - $C_t = 0.02; $ building period coefficient from Table 12.8-2 for “all other structural systems”
  - $x = 0.75; $ coefficient from Table 12.8-2
Structural Height Defined

ASCE 7-10
Ch. 11 Seismic Design Criteria - NEW

STRUCTURAL HEIGHT: The vertical distance from the base to the highest level of the seismic force-resisting system of the structure. For pitched or sloped roofs, the structural height is from the base to the average height of the roof.

Approximate Fundamental Period

• Approximate fundamental period:

\[ T_a = C_t h_n^x \]

where;
• \( T_a \) = approximate fundamental period, s
• \( h_n \) = structural height, ft
• \( C_t = 0.02 \); building period coefficient from Table 12.8-2 for “all other structural systems”
• \( x = 0.75 \); coefficient from Table 12.8-2

<table>
<thead>
<tr>
<th>( h_n ) (ft)</th>
<th>( T_a ) (s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>15</td>
<td>0.15</td>
</tr>
<tr>
<td>25</td>
<td>0.22</td>
</tr>
<tr>
<td>35</td>
<td>0.29</td>
</tr>
<tr>
<td>45</td>
<td>0.35</td>
</tr>
<tr>
<td>55</td>
<td>0.40</td>
</tr>
<tr>
<td>65</td>
<td>0.46</td>
</tr>
</tbody>
</table>
Seismic Base Shear

- Seismic base shear, $V$:

\[
V = \left(\frac{S_{DS}}{I_e}\right) \frac{W}{R}
\]

From Eq. 12.8-1 and Eq. 12.8-2 where:

- $W$ = effective seismic weight
- $R$ = response modification factor
- $I_e$ = importance factor
- $S_{DS}$ = \(\frac{2}{3}(F_a)(S_s)\)
- $F_a$ = short period site coefficient
- $S_s$ = mapped spectral response acceleration parameter for short periods

Effect of System R on Base Shear

- Example calculation of seismic base shear, $V$

<table>
<thead>
<tr>
<th>BASIC SEISMIC FORCE-RESISTING SYSTEM</th>
<th>RESPONSE MODIFICATION COEFFICIENT $n^*$</th>
<th>BASE SHEAR AS A RATIO OF EFFECTIVE SEISMIC WEIGHT ($I_e=1.0$, $F_a=1.0$, $S_s=1.5$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A. Bearing Wall Systems</td>
<td></td>
<td></td>
</tr>
<tr>
<td>15. Light-frame (wood) walls sheathed with wood structural panels rated for shear resistance</td>
<td>6-1/2</td>
<td>(V = \frac{W}{6.5} = 0.154W)</td>
</tr>
<tr>
<td>17. Light-frame walls with shear panels of all other materials</td>
<td>2</td>
<td>(V = \frac{W}{2.0} = 0.50W)</td>
</tr>
<tr>
<td>G. Cantilevered column systems</td>
<td></td>
<td></td>
</tr>
<tr>
<td>6. Timber frames</td>
<td>1-1/2</td>
<td>(V = \frac{W}{1.5} = 0.67W)</td>
</tr>
</tbody>
</table>
Allowable Story Drift

- Drift Calculation
  \[ \delta_x = \frac{C_d \delta_{xe}}{I_e} \]  
  (Eq. 12.8-15)

  - \( \delta_x \) = drift (see 12.8.2 story drift definition)
  - \( C_d \) = deflection amplification factor
  - \( \delta_{xe} \) = deflection by elastic analysis

- Allowable story drift
  \[ \delta_x < 0.02h_{sx} \]  
  (Table 12.12-1, Risk Category II)

  - \( \delta_x < 0.025h_{sx} \)
  - \( h_{sx} \) = story height

Diaphragm Flexibility

- Idealized as Flexible
  - Diaphragm load is distributed to shear walls based on tributary area (common for wood frame)

- Idealized as Rigid
  - Diaphragm load is distributed to shear walls based on relative wall stiffness

- Semi-rigid
  - Diaphragm load is distributed to shear walls based on relative stiffness of shear walls and diaphragm
12.3.1.1 Flexible Diaphragm Condition. Diaphragms constructed of untopped steel decking or wood structural panels are permitted to be idealized as flexible if any of the following conditions exist:

- a. In structures where the vertical elements are steel braced frames, steel and concrete composite braced frames or concrete, masonry, steel, or steel and concrete composite shear walls.

- b. In one- and two-family dwellings.

- c. In structures of light-frame construction where all of the following conditions are met:
  1. Topping of concrete or similar materials is not placed over wood structural panel diaphragms except for nonstructural topping no greater than 1 1/2 in. (38 mm) thick.
  2. Each line of vertical elements of the seismic force-resisting system complies with the allowable story drift of Table 12.12-1.
Lesson 11 Analysis Methods

Calculated Flexible Diaphragm

\[ \Delta_{\text{MAX DIAPHRAGM}} \geq 2 \times \Delta_{\text{SHEARWALLS}} \]

Prescribed Rigid Diaphragm

- 12.3.1.2 Rigid Diaphragm Condition.
- Idealized as rigid:
  - Concrete slabs or concrete filled metal deck
  - Span-to-depth ratios of 3 or less
  - No horizontal irregularities
Semi-rigid Diaphragm

• Semi-rigid diaphragm behavior
  • Difficult to analyze semi-rigid behavior
  • Envelop approach commonly employed

Exception to Overstrength

• 12.10.2.1 Collector Elements Requiring Load Combinations with Overstrength Factor for Seismic Design Categories C through F...

• EXCEPTIONS:
  1. ....
  2. In structures or portions thereof braced entirely by light-frame shear walls, collector elements and their connections including connections to vertical elements need only be designed to resist forces using the load combinations of Section 12.4.2.3 with seismic forces determined in accordance with Section 12.10.1.1.
### Application of Redundancy

- **12.3.4.2 Redundancy**
  
  Factor, $\rho$, is permitted to be taken as 1.0:

  - **b.** Structures ... consist of at least two bays of seismic force-resisting perimeter framing on each side of the structure ... The number of bays for a shear wall shall be calculated as ... two times the length of shear wall divided by the story height, $h_{sx}$ for light-frame construction.

### Horizontal Combinations

- **12.2.3.3** $R$, $C_{dx}$, and $\Omega_0$ Values for Horizontal Combinations ... $R$ used for design ... shall not be greater than the least value of $R$ for any of the systems utilized in that direction....

  **EXCEPTION:** Least $R$ for each line of resistance permitted where all of the following are met:
  
  - Risk Category I or II building
  - Two stories or less above grade plane
  - Use of light-frame construction or flexible diaphragm
  - $R$ used for diaphragms shall not be greater than the least value of $R$
**Horizontal Combinations and Shear of Dissimilar Materials**

- SDPWS 4.3.3 - Dissimilar materials on opposite sides:
  - Unit shear = larger of 2x smaller unit shear or unit shear of stronger side

**Table. Example Application of ASCE horizontal combination rule to 2-sided walls**

<table>
<thead>
<tr>
<th>Side 1/Side 2</th>
<th>Design capacity(^a)</th>
<th>Bearing wall, R</th>
<th>SDC limit</th>
</tr>
</thead>
<tbody>
<tr>
<td>WSP/GYP</td>
<td>combined WSP/GYP</td>
<td>2</td>
<td>A – D</td>
</tr>
<tr>
<td></td>
<td>WSP only</td>
<td>6.5</td>
<td>A – F</td>
</tr>
<tr>
<td></td>
<td>GYP only</td>
<td>2</td>
<td>A – D</td>
</tr>
</tbody>
</table>

\(^a\) Single-side capacity (WSP only or GYP only) and combined capacity of double-sided wall (combined WSP/GYP) in accordance with SDPWS.

\(^b\) WSP = wood structural panel; GYP = gypsum wallboard.

**Additional Resource**

- Additional resource for application of redundancy and horizontal combinations and dissimilar materials
  - Seismic Requirements for Wood Building Design – Recent Changes to ASCE 7 and IBC, Wood Design Focus, Spring 2006
  - SDPWS Commentary
Two Stage Analysis Procedure

12.2.3.2 Two Stage Analysis Procedure

A two-stage equivalent lateral force procedure is permitted to be used for structures having a flexible upper portion above a rigid lower portion, provided the design of the structure complies with all of the following:

1. Lower portion at least 10x stiffer than upper portion.
2. Period of entire structure not more than 1.1 times period of upper portion.
3. Upper and lower portions designed as separate structures.
4. Upper portion reactions amplified by the ratio of the R/ρ of the upper portion over R/ρ of the lower portion. This ratio >1.0.
5. Upper portion analyzed with the equivalent lateral force or modal response spectrum procedure. Lower portion analyzed with the equivalent lateral force procedure.
**ASCE 7-10**

**509.2  510.2 Horizontal Separation of Buildings**

510.2 Horizontal building separation allowance. A building shall be considered as separate and distinct buildings for the purpose of determining area limitations, continuity of fire walls, limitation of number of stories and type of construction where all of the following conditions are met:

1. The buildings are separated with a horizontal assembly having a fire-resistance rating of not less than 3 hours.
2. The building below the horizontal assembly is not greater than one story above grade plane.
3. The building below the horizontal assembly is of Type IA construction.

**Ch. 2 Definitions**

- **HEIGHT, BUILDING.** The vertical distance from grade plane to the average height of the highest roof surface.

**ASCE 7-10**

Ch. 11 Seismic Design Criteria - NEW

**STRUCTURAL HEIGHT.** The vertical distance from the base to the highest level of the seismic force-resisting system of the structure. For pitched or sloped roofs, the structural height is from the base to the average height of the roof.

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**Additional Resource**

- Additional resources for multi-story wood-frame over podium
  - Five-story Wood-frame Structure over Podium Slab, Developed for WoodWorks by Douglas Thompson
  - APA 110 All-wood Podiums in Mid-Rise Construction
**ASCE 12.14 Simplified Procedure**

- Use of procedure subject to limitations including:
  - Not applicable for SDC E or F
  - Occupancy Category I and II only
  - Other major differences with ELF of ASCE 12.8
    - One, two and three story factor, $F$, for calculation of base shear, $V$
    - Vertical distribution not triangular
    - No required drift check
    - Diaphragm design force based on portion of effective weight at each level

$$V = \frac{F S D W}{R} \quad \text{Eq. 12.14-11}$$

- $F = 1.2$
- $F = 1.1$
- $F = 1.0$

**Diaphragm Anchorage to Concrete and Masonry Structural Walls**

- Section 12.11.2
- Seismic Design Category C and greater
- Continuous ties between chords. Wood structural panel shall not be considered as providing ties. Anchorage shall not be accomplished by use of toenails or nails subject to withdrawal nor shall wood ledgers or framing be used in cross-grain bending or cross-grain tension.
- Sub-diaphragms with aspect ratio of not more than 2.5:1
Additional Resources

• Additional resources for anchorage of wood diaphragms to concrete/masonry structural walls
  • APA Z350 Lateral connections for low-slope roof diaphragms
  • FEMA 547 Techniques for seismic rehabilitation of existing buildings

Overview

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• ASCE 7-10 Seismic Provision for Wood Frame
  • Defined systems, seismic base shear, and allowable story drift limits
• Special Design Provisions for Wind and Seismic (SDPWS)
  • Shear wall and diaphragm construction details, strength, stiffness, and limitations
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
• [http://youtu.be/K1J1dfdZbhI](http://youtu.be/K1J1dfdZbhI)
http://youtu.be/nH4nvIk_umA
Elements of a wood shear wall

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

Connection Behavior

**Balance**
- Strength –
- Ductility -

![Graph showing load vs. displacement with three curves: high strength, poor ductility; good strength, good ductility; low strength, good ductility. The good strength, good ductility curve is marked with a green checkmark.]
Connection Behavior

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

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

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.

---

**Adjustment for Design Level**

Nominal unit shear values adjusted in accordance with 4.3.3 to determine ASD allowable unit shear capacity and LRFD factored unit resistance.

ASD unit shear capacity, $v_5$:

$$v_5 = \frac{510 \text{ plf}}{2.0} = 255 \text{ plf}$$

<table>
<thead>
<tr>
<th>Reference nominal value</th>
<th>ASD reduction factor</th>
</tr>
</thead>
</table>

LRFD unit shear capacity, $v_5$:

$$v_5 = 510 \text{ plf} \times 0.80 = 408 \text{ plf}$$

<table>
<thead>
<tr>
<th>Reference nominal value</th>
<th>LRFD resistance factor</th>
</tr>
</thead>
</table>

---

**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 $R_p$ of 0.80. Any further increase 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
  
  \[
  \text{SG Adjustment Factor} = [1.0 - (0.50 - G)] < 1.0
  \]

• 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 \( 2:1 < h:b_s < 3.5:1 \)

Multiply \( v_s \) by \( 2b_s/h \)
Adjustment for Aspect Ratio

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>8</td>
<td>4</td>
<td>2</td>
<td>1.00</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>3.2</td>
<td>2.5</td>
<td>0.80</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>2.7</td>
<td>3</td>
<td>0.67</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>2.3</td>
<td>3.5</td>
<td>0.57</td>
<td></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, \( v = 510 \text{ plf} \)
- 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)
  
  SG Adjustment Factor = [1.0-(0.50-0.42)] = 0.92

- 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

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

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

\[
\text{SPDWS - Nails}
\]
Shear Wall Deflection

Shear wall deflection, Equation 4.3-1

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

- \( b \) = 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.
- \( \nu \) = induced unit shear, lbs/ft

Non-linear 4-term equation in Commentary
Example Deflection Calculation

Given

- 8 ft x 8 ft Wall
- Apparent shear stiffness, $G_a$: 16 kips/in.
- (2) 2x4 framing as end post area, $A$: (2) x 1.5 x 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?

Solution

Shear wall deflection from Eq. 4.3-1:

- Bending: $\frac{8vh^3}{EAb} = 0.012$ in.
- Shear: $\frac{vh}{1000G_a} = 0.204$ in.
- Anchor slip: $\frac{h}{b} \Delta_a = 0.125$ in.

Total deflection = 0.34 in.
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

- Adjoining panel edge
- 3x framing or blocking
- Adjoining panel edge
- Adjoining panel edges staggered

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>Sheathing to form lateral value per NES (L=4.5' framing)</th>
<th>Fastener spacing (in.) for 2x stud-to-stud connection</th>
</tr>
</thead>
<tbody>
<tr>
<td>6d common, 3/8'' WSP (0.3)</td>
<td>54</td>
<td>12.0</td>
</tr>
<tr>
<td>8d common, 3/8'' WSP (0.4)</td>
<td>71</td>
<td>9.1</td>
</tr>
<tr>
<td>10d common, 7/8'' WSP (0.5)</td>
<td>95</td>
<td>8.8</td>
</tr>
<tr>
<td>10d common, 1 1/8'' WPS (0.42)</td>
<td>95</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 lb in seismic design categories (SDC) D, E, and F. An alternative to single 3x framing, included in SDPWES, and based on principles of mechanics, is the use of 2x-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

Failure Mode?

Small scale test specimen to induce cross grain bending
Foundation Bottom Plate – Testing

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.
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 2h/b.
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+0.05h/b. 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) = 1.09-0.095h/b. 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; bs measured as sheathed width
  - h:bs must not exceed 3.5:1
  - aspect ratio reductions apply for wood structural panel where h:bs 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
Wood Frame Diaphragms

- Diaphragms
  - Elements of a diaphragm
  - Strength and stiffness
  - Detailing and limitations

Wood Frame Diaphragms

- Elements of a diaphragm

![Diagram of diaphragm with labeled parts: Chord, Boundary element, Nailed sheathing, T, C, T.](image)
Diaphragm Test

- Deformation of sheathing nailing in diaphragm test

Wood Structural Panel Diaphragm Cases

Diaphragm Cases 1 through 6
Lesson 11 Analysis Methods

- Example determination of nominal unit shear capacity, $v_s$
  - 15/32 in. Structural I WSP, 10d common Nail, 6 in. Panel Edge/Boundary Fastener Spacing, Framing $G = 0.5$, Framing at 16 in. o.c.
  - Nominal unit shear capacity, $v_s$: 720 plf

- Example determination of apparent shear stiffness, $G_a$
  - 15/32 in. Structural I OSB, 10d common Nail, 6 in. Panel Edge/Boundary Fastener Spacing, Framing $G = 0.5$, Framing at 16 in. o.c.
  - Apparent shear stiffness, $G_a$: 20 kips/in.
Table Footnotes

- 1. Nominal unit shear capacities shall be adjusted in accordance with 4.2.3 to determine ASD allowable unit shear capacity and LRFD factored unit resistance. For general construction requirements see 4.2.6. For specific requirements, see 4.2.7.1 for wood structural panel diaphragms. See Appendix A for common nail dimensions.

- 2. 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.

- 3. Apparent shear stiffness values, Ga, 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 diaphragms constructed with either OSB or 3-ply plywood panels. When 4-ply or 5-ply plywood panels or composite panels are used, Ga values shall be permitted to be increased by 1.2.

- 4. Where moisture content of the framing is greater than 19% at time of fabrication, Ga values shall be multiplied by 0.5.

Adjustment for Design Level

- Nominal unit shear values adjusted in accordance with 4.2.3 to determine ASD allowable unit shear capacity and LRFD factored unit resistance.

- ASD unit shear capacity, vs:
  \[ \text{vs} = \frac{720 \text{ plf}}{2.0} = 360 \text{ plf} \]

- LRFD unit shear capacity, vs:
  \[ \text{vs} = 720 \text{ plf} \times 0.80 = 576 \text{ plf} \]
Adjusted Design Unit Shear

- Example calculation of design unit shear adjusted for framing specific gravity (G=0.42)
- ASD unit shear capacity, vs:
  - \( \text{vs} = \frac{720 \text{ plf}}{2.0 \times 0.92} = 331 \text{ plf} \)
  - SG adjustment factor
  - ASD reduction factor
  - Reference nominal value
- LRFD unit shear capacity, vs:
  - \( \text{vs} = 720 \text{ plf} \times 0.80 \times 0.92 = 530 \text{ plf} \)
  - SG adjustment factor
  - LRFD resistance factor
  - Reference nominal value

Diaphragm Deflection

- Diaphragm deflection, Equation 4.2-1

\[
\Delta = \frac{5vL^3}{8EAW} + \frac{0.25vL}{1000G_a} + \frac{\sum (\Delta_cX)}{2b}
\]

- Non-linear 4-term equation in Commentary

![Comparison of 4-Term and 3-Term Deflection Equations](image)
Diaphragm Deflection

- Diaphragm deflection, Equation 4.2-1

\[
\Delta = \frac{5vL^3}{8EAW} + 0.25vL + \frac{\sum(\Delta cX)}{2b}
\]

- \(L\) = diaphragm length, ft
- \(W\) = diaphragm width, ft
- \(E\) = modulus of elasticity of diaphragm chords, psi
- \(A\) = area of chord cross-section, in\(^2\)
- \(Ga\) = apparent shear wall shear stiffness, kips/in.
- \(X\) = distance from chord splice to nearest support, ft
- \(\Delta c\) = diaphragm chord splice slip, in.
- \(v\) = induced unit shear, lbs/ft

Diaphragm Deflection Example

- Completed example in SDPWS Commentary
Other Wood Structural Panel Diaphragms

- Other wood structural panel diaphragms
  - Unblocked diaphragms
    - Reduced unit shears and stiffness
  - High load diaphragms
    - 2 or 3 lines of fasteners at panel edges
    - 3x or 4x minimum framing
    - Increased unit shears and stiffness
  - Wood structural panel over lumber planking or laminated decking
    - Requirements for attachment to planking or decking
    - Unit shears associated with blocked wood structural panels

Lumber Sheathed Diaphragms

- Strength and stiffness information provided for lumber sheathed diaphragms
  - Horizontal Lumber Sheathing
  - Diagonal Lumber Sheathing
  - Double Diagonal Lumber Sheathing

Table 4.2D Nominal Unit Shear Capacities for Wood-Frame Diaphragms

<table>
<thead>
<tr>
<th>Lumber Diaphragm</th>
<th>Sheathing Material</th>
<th>Nailing at Intermediate and End Bearing Supports</th>
<th>Nailing or Boundary members (type and number of nails per board)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Horizontal</td>
<td>1x5</td>
<td>2-40 common nails (2-80 box nails)</td>
<td>3-60 common nails (3-120 box nails)</td>
</tr>
<tr>
<td>Sheathing</td>
<td>2x6</td>
<td>2-100 common nails (2-200 box nails)</td>
<td>3-150 common nails (3-300 box nails)</td>
</tr>
<tr>
<td>Diagonal</td>
<td>2x6</td>
<td>2-60 common nails (2-120 box nails)</td>
<td>3-120 common nails (3-240 box nails)</td>
</tr>
<tr>
<td>Sheathing</td>
<td>3x6</td>
<td>2-150 common nails (2-300 box nails)</td>
<td>3-180 common nails (3-360 box nails)</td>
</tr>
<tr>
<td>Diagonal</td>
<td>3x6</td>
<td>2-60 common nails (2-120 box nails)</td>
<td>3-120 common nails (3-240 box nails)</td>
</tr>
<tr>
<td>Sheathing</td>
<td>3x6</td>
<td>2-150 common nails (2-300 box nails)</td>
<td>3-180 common nails (3-360 box nails)</td>
</tr>
<tr>
<td>Diagonal</td>
<td>3x6</td>
<td>2-60 common nails (2-120 box nails)</td>
<td>3-120 common nails (3-240 box nails)</td>
</tr>
<tr>
<td>Sheathing</td>
<td>3x6</td>
<td>2-150 common nails (2-300 box nails)</td>
<td>3-180 common nails (3-360 box nails)</td>
</tr>
<tr>
<td>Diagonal</td>
<td>3x6</td>
<td>2-60 common nails (2-120 box nails)</td>
<td>3-120 common nails (3-240 box nails)</td>
</tr>
</tbody>
</table>
Diaphragm Construction Details

- Section 4.2.7.1.1 (3). 3x framing required at adjoining panel edges where:
  - Nail spacing of 2-1/2 in. o.c. or less
  - 10d common nails having penetration of more than 1-1/2 in. at 3 in. o.c. or less

Limitations Overview

- Limitations on use of reference values include:
  - Aspect ratio
  - Open front and cantilever diaphragms
  - Materials and height when resisting forces contributed by masonry and concrete walls
4.2.4 Diaphragm Aspect Ratios

Size and shape of diaphragms shall be limited to the aspect ratios in Table 4.2.4.

<table>
<thead>
<tr>
<th>Diaphragm Sheathing Type</th>
<th>Maximum L/W Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wood structural panel, unblocked</td>
<td>3:1</td>
</tr>
<tr>
<td>Wood structural panel, blocked</td>
<td>4:1</td>
</tr>
<tr>
<td>Single-layer straight lumber sheathing</td>
<td>2:1</td>
</tr>
<tr>
<td>Single-layer diagonal lumber sheathing</td>
<td>3:1</td>
</tr>
<tr>
<td>Double-layer diagonal lumber sheathing</td>
<td>4:1</td>
</tr>
</tbody>
</table>

2015 AWC Standards
2015 NDS/SDPWS

2015 National Design Specifications for Wood Construction

- CLT Provisions

2015 Special Design Provisions for Wind and Seismic

- Design Flexible and Open Front/Cantilever Diaphragms
2015 SDPWS

- Overview:
  - Ch. 2
    - Removes definition of flexible and rigid diaphragms
    - Defines “Open-Front Structure” & “Subdiaphragm”
  - Ch. 4
    - Clarification of concrete and masonry wall anchorage
    - Revised Horizontal Distribution of Shear
    - Clarification of shear wall Aspect ratio adjustments

2015 SDPWS

Figure 4A - Examples of Open Front Structures

(a) Plan View with Shear Wall and Cantilevered Diaphragm
(b) Plan View with Shear Wall, Cantilevered Diaphragm, and Open Front
(c) Plan View with Shear Wall, Cantilevered Diaphragm, and Open Front
(d) Plan View with Shear Wall, Cantilevered Diaphragm, and Open Front
Additional Information

- Diaphragm resources
  - APA 138 Plywood Diaphragms
  - APA L350 Shear Walls and Diaphragms
  - APA Z350 Lateral Load Connections for Low Slope Roof Diaphragms

Additional Information

- American Wood Council - [www.awc.org](http://www.awc.org)
- APA - [www.apawood.org](http://www.apawood.org)
Questions?

AMERICAN WOOD COUNCIL

www.awc.org
info@awc.org