Shearwall Basics Using SDPWS

American Wood Council
This presentation is protected by US and International Copyright laws. Reproduction, distribution, display and use of the presentation without written permission is prohibited.

© American Wood Council 2015
SDPWS and IBC

2008 SDPWS is referenced in 2012 IBC
SECTION 2305 GENERAL DESIGN REQUIREMENTS FOR LATERAL FORCE-RESISTING SYSTEMS

2305.1 General. Structures using wood-frame shear walls or wood-frame diaphragms to resist wind, seismic or other lateral loads shall be designed and constructed in accordance with AF&PA SDPWS and the applicable provisions of Sections 2305, 2306 and 2307.
Significant Changes to 2012 IBC

2306.3 Wood structural panel shear walls. Wood-frame shear walls. 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.

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**

2305.3 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.

$$\Delta = \frac{8vH^3}{EA_b} + \frac{vh}{G_t} + 0.75 \cdot h \cdot e + d \cdot \frac{h}{a_b}$$ (Equation 23-2)
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 plf 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. When fasteners connecting the two framing members are spaced less than 4" on center, they shall be staggered.
SDPWS

2008 SDPWS

• Engineered
• Res and Non-Res
• ASD & LRFD
• Efficiencies in designs
• Shear wall provisions
  • Segmented
  • Perforated
  • Force Transfer Around Openings
Chapter 4 – Nominal Design Value

- Wind nominal unit shear capacity \( v_w \)
- 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 Walls\(^1,3\)

<table>
<thead>
<tr>
<th>Panel Edge Fastener Spacing (in.)</th>
<th>6</th>
<th>4</th>
<th>3</th>
<th>2</th>
</tr>
</thead>
<tbody>
<tr>
<td>( V_s ) (kips/in.)</td>
<td>40</td>
<td>13.0</td>
<td>500</td>
<td>18.0</td>
</tr>
<tr>
<td>( V_s ) (kips/in.)</td>
<td>500</td>
<td>840</td>
<td>1090</td>
<td>1430</td>
</tr>
</tbody>
</table>

- 5/16" H 1-1/4"
- 5/8" Z 1-3/8"
- 15/32" 1/2"

### Wood-based Panels (Excluding Plywood for \( G_a \))\(^4\)

<table>
<thead>
<tr>
<th>Sheathing Material</th>
<th>Minimum Nominal Panel Thickness (in.)</th>
<th>Minimum Fastener Penetration in Framing (in.)</th>
<th>Fastener Type &amp; Size</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wood Structural Panels</td>
<td>5/16&quot; H 1-1/4&quot;</td>
<td>5/8&quot; Z 1-3/8&quot;</td>
<td>15/32&quot; 1/2&quot;</td>
</tr>
<tr>
<td>Structural (^4)</td>
<td>15/32&quot; 1/2&quot;</td>
<td>15/32&quot; 1/2&quot;</td>
<td>15/32&quot; 1/2&quot;</td>
</tr>
</tbody>
</table>

---

\(^1\) Seismic design requirements only.
\(^2\) Nominal values for G_a.
\(^3\) Data derived from AISC Building code and ASCE Building code.
\(^4\) Wood Structural Panel = Wood Structural Panel. Structural = Structural Panel.
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, \( v_s \):**

\[ v_s = \frac{510 \text{ plf}}{2.0} = 255 \text{ plf} \]

- ASD reduction factor
- Reference nominal value

**LRFD unit shear capacity, \( v_s \):**

\[ v_s = 510 \text{ plf} \times 0.80 = 408 \text{ plf} \]

- LRFD resistance factor
- Reference nominal value

---

**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, \( \phi_D \), of 0.80. 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>
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

Table 4.3.4 Maximum Shear Wall Aspect Ratios

<table>
<thead>
<tr>
<th>Shear Wall Type</th>
<th>Maximum h/b_s 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(^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(^2)</td>
</tr>
<tr>
<td>Portland cement plaster</td>
<td>2:1(^3)</td>
</tr>
<tr>
<td>Structural Fiberboard</td>
<td>3.5:1(^3)</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_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+0.9b_s/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) = 1.09-0.09h/b_s. The value of the Aspect Ratio Factor (Wind) shall not be greater than 1.0.
## 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></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>
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.
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
- Adjoining panel edges not staggered

a. Adjoining panel edges staggered
b. Adjoining panel edges not 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 frame lateral value per NDS (G=0.5 framing)</th>
<th>Fastener spacing (in.) for 2x stud-to-2x stud connection (10d common nail, Z = 118 lbf)</th>
<th>Panel edge nail spacing (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>6d common, 3/8&quot; WSP (G=0.5)</td>
<td>54</td>
<td>12.0 8.0 6.0 4.0</td>
<td>6 4 3 2</td>
</tr>
<tr>
<td>8d common, 3/8&quot; WSP (G=0.5)</td>
<td>71</td>
<td>9.1 6.1 4.5 3.0</td>
<td></td>
</tr>
<tr>
<td>8d common, 7/16&quot; WSP (G=0.5)</td>
<td>73</td>
<td>8.8 5.9 4.4 2.9</td>
<td></td>
</tr>
<tr>
<td>10d common, 19/32&quot; WSP (G=0.42)</td>
<td>95</td>
<td>6.8 4.5 3.4 2.3</td>
<td></td>
</tr>
</tbody>
</table>

* Spacing based on 8' wall and assuming only 87.5" of stud height available for stud-to-stud fastening.
**Table. 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 frame lateral value per NDS (G = 0.5 framing)</th>
<th>Fastener spacing (in.) for 2x stud-to-2x stud connection (10d common nail, Z = 118 lbf)</th>
<th>Fastener spacing (in.) for 2x stud-to-2x stud connection (SDS 1/4 x 3, Z = 280 lbf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>6d common, 3/8&quot; WSP (G=0.5)</td>
<td>54</td>
<td>12.0 8.0 6.0 4.0 28.4 18.9 14.2 9.5</td>
<td></td>
</tr>
<tr>
<td>8d common, 3/8&quot; WSP (G=0.5)</td>
<td>71</td>
<td>9.1 6.1 4.5 3.0 21.6 14.4 10.8 7.2</td>
<td></td>
</tr>
<tr>
<td>8d common, 7/16&quot; WSP (G=0.5)</td>
<td>73</td>
<td>8.8 5.9 4.4 2.9 21.0 14.0 10.5 7.0</td>
<td></td>
</tr>
<tr>
<td>10d common, 19/32&quot; WSP (G=0.42)</td>
<td>95</td>
<td>6.8 4.5 3.4 2.3 16.1 10.7 8.1 5.4</td>
<td></td>
</tr>
</tbody>
</table>

*Spacing based on 8' wall height and assuming only 87.5" of stud height available for stud-to-stud fastening.*
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 to single 3x 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
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”
Questions?

www.awc.org
info@awc.org
Shear Wall Design for Non-Residential and Multi-Family Buildings
Shear Wall

- A wall designed to resist lateral forces parallel to the plane of wall (IBC Sec. 2302.1)
Load Path Components

3. Shear Walls
Shear Wall Design: Loads

- Nailing from panels to framing resists shear in wall
Failure Mode

Racking
Shear Wall Design

- Specific nail size and spacing requirements
- Wood structural panel - specific grade and thickness
- Specific stud species
- Hold-down anchors
- Anchor bolts
- Specific nail size and spacing requirements
Anchor bolts resist base shear
Foundation Anchorage

Hurricane Katrina
Shear Wall Design: Loads

- Hold downs resist overturning
Shearwall Hold-Down Anchors
Hold-down Anchor
Hold-down Anchor

- Low-slip fasteners
- Pre-deformed base
- A plus in seismic loading
Hold-down Anchor

- Multi-story apps.
- Self-tightening
- A plus in taller structures
Typ. Failure Modes – Edge Tear, Nail Yield, Nail Pull Through
Typ. Failure Modes – Nails Worked in Lumber
Typ. Failure Modes – Nails Yield
End Post (Chord) Failure
End Post (Chord) Failure
Design Methods (SDPWS)

1. Segmented Shear Walls

2. Shear Walls with Openings
   - a. force transfer around openings
   - b. perforated shear walls
Shear Wall Design

Segmented
1. Aspect Ratio for seismic 2:1
2. Aspect ratio up to 3.5:1, if allowable shear is reduced by 2w/h

SDPWS 4.3.5.1

Force Transfer
1. Code does not provide guidance for this method
2. Different approaches using rational analysis could be used

SDPWS 4.3.5.2

Perforated
1. Code provides specific requirements
2. The capacity is determined based on empirical equations and tables

SDPWS 4.3.5.3
Shearwall Minimum Aspect Ratios $h/b_s$

Minimum width:

$$b_s = h/2$$

exception: 3.5:1 can be used

$$b_s = h/3.5$$

with penalty $(2b_s/h)$
Site Built Portal Frame
Reference: APA Report TT-100

Outside Elevation

3" x 11-1/4" min. header (built-up, solid sawn, glulam, etc.)

6' to 18'

Max. 10'

16" or 24"

Fasten sheathing to header with 8d common nails in 3" grid pattern as shown and 3" o.c. in all framing (studs and sills) typ.

For frames over 8', panel splice (if needed) shall occur within 24" of mid-height. Blocking is required. If 2x blocking is used, then it must be stitch nailed with one row of 16d sinkers at 3" o.c.

4200 lb hold down straps applied over sheathing installed in concrete per manufacturer's instruction

Side Elevation

Sheathing filler if needed

16d sinkers at 3" o.c.

1000 lb header strap

3/8" minimum thickness wood structural panel

4200 lb hold down straps applied over sheathing
• Only full height segments are considered
• Max aspect ratio
  ▪ 2:1 – for seismic
  ▪ 3.5:1 – for wind
• Current Code design values based on data dating back to 1950’s.
Shear Wall With Opening – Force Transfer Around Openings (SDPWS 4.3.5.2)

- Openings accounted for by strapping or framing
  - “based on a rational analysis”
- H/w ratio defined by wall pier

Aspect ratio applies to wall pier segment (dotted)
Shear Wall With Opening –
Force Transfer Around Opening

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

Aspect ratio applies to wall pier segment (dotted)
Shear Wall With Opening – Perforated Shear Wall (SDPWS 4.3.5.3)

- Openings accounted for by empirical adjustment factor
- Hold-downs only at ends
- Uplift between hold downs, $t$, at full height segments is also required

Aspect ratio applies to full height segment (dotted)
Wood Shear Wall Capacity

- For:
  - 8d com. @ 4” edge nail spacing
  - 7/16” OSB Sheathing Grade

- Go to Table 4.3A SDPWS
## Wood Shear Wall Capacity

<table>
<thead>
<tr>
<th>Sheathing Material</th>
<th>Minimum Nominal Panel Thickness (in.)</th>
<th>Minimum Fastener Penetration in Framing Member or Blocking (in.)</th>
<th>Fastener Type &amp; Size</th>
<th>Panel Edge Fastener Spacing (in.)</th>
<th>B WIND</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>vₑ (plf) vₑ (plf) vₑ (plf) vₑ (plf)</td>
<td></td>
</tr>
<tr>
<td>Wood Structural Panels - Structural</td>
<td>5/16</td>
<td>1-1/4</td>
<td>6d</td>
<td>560</td>
<td>6 4 3 2</td>
</tr>
<tr>
<td></td>
<td>3/8</td>
<td>1-3/8</td>
<td>8d</td>
<td>645</td>
<td>560 645</td>
</tr>
<tr>
<td></td>
<td>7/16</td>
<td>1-3/8</td>
<td></td>
<td>715</td>
<td>715 715</td>
</tr>
<tr>
<td></td>
<td>15/32</td>
<td>1-1/2</td>
<td>10d</td>
<td>785</td>
<td>785 785</td>
</tr>
<tr>
<td></td>
<td>15/32</td>
<td></td>
<td></td>
<td>950</td>
<td>950 950</td>
</tr>
<tr>
<td>Wood Structural Panels – Sheathing</td>
<td>5/16</td>
<td>1-1/4</td>
<td>6d</td>
<td>505</td>
<td>505 505</td>
</tr>
<tr>
<td></td>
<td>3/8</td>
<td></td>
<td></td>
<td>560</td>
<td>560 560</td>
</tr>
<tr>
<td></td>
<td>7/16</td>
<td>1-3/8</td>
<td>8d</td>
<td>615</td>
<td>615 615</td>
</tr>
<tr>
<td></td>
<td>15/32</td>
<td></td>
<td></td>
<td>730</td>
<td>730 730</td>
</tr>
<tr>
<td></td>
<td>15/32</td>
<td></td>
<td></td>
<td>870</td>
<td>870 870</td>
</tr>
<tr>
<td></td>
<td>19/32</td>
<td></td>
<td></td>
<td>950</td>
<td>950 950</td>
</tr>
</tbody>
</table>
Wood Shear Wall Capacity

- For:
  - 8d com. @ 4” edge nail spacing
  - 7/16” OSB Sheathing Grade

- From Table 4.3A (SDPWS)

- Wall capacity = 980 plf / 2 = 490 plf

For a wall length of 8-ft, the total capacity = 3920 lbs
Overturning: Sizing the Hold Down (no DL)

- **Sum moment about o:**
  - $V \cdot h = H \cdot L$

- **Rearranging:**
  - $H = \frac{V \cdot h}{L}$

- **Letting $v=V/L$:**
  - $H = v \cdot h$

Is $L$ the true effective length?
Sizing the Hold Down (no DL)

- Let $x$ represent the distance from edge of wall to hold down rod.

- Summing moments simplifies to:
  - $H = \frac{Vh}{L-x}$ or
  - $H = \frac{vhL}{L-x}$

Is this reduced length precision accurate?
Sizing the Hold Down (no DL)

- APA testing (1:1 aspect ratio) shows using $H = v \cdot h$ matches reality better than “reduced length”

- The dead load assumed to counteract $H$ is probably at least an equally important consideration
Don’t Forget to Check the Chord Size and Strength!

4x4 shear wall end post failed in tension
Sizing the Chords (no DL)

- **Tension chords**
  - The end studs
  - Sized as hold down forces are
  - Designed as tension members
- **Compression chords** should include DL

Chord forces can become quite large in highly loaded stacked shear walls!
Sizing Hold Down Example (no DL)

- Given 350 plf ASD capacity wall

- What’s overturning force required?

\[ H = 350 \text{ plf} \times 8\text{-ft} = 2800 \text{ lbf} \]
Overturning Forces

- Every structure shall be designed to resist overturning effects (IBC 1604.4)
Only 0.6 x design dead load can be used to resist overturning from wind or earthquake (IBC 1605.3, ASCE 7 Sec. 2.4)
The amount of dead load available to resist overturning depends on:

- Rational analysis
- Framing system and configuration
- Engineering judgment
Wind produces uplift and shear at the same time.
The load combinations and analysis of shear, uplift and dead load can be complex.

Breyer et al. has design examples.
Height to width ratio (SDPWS 4.3.4.1)

- For shear walls and perforated shear walls
- h:w must not exceed 2:1 (seismic) or 3.5:1 (wind) ratio
Height to width ratio (SDPWS 4.3.4.2)

- For force transfer around opening shear walls
  - h:w must not exceed 2:1 (seismic) or 3.5:1 (wind) ratio
Shear Wall 3x Requirements

At adjoining panel edges

Sill plate
Shear Walls: 3x’s

- **3x’s at adjoining panels required when:**
  - Allowable shear > 700 plf in SDC D-F (SDPWS 4.3.7.1(4))
  - Double sided walls do not have panels offset (SDPWS Table 4.3A footnote 6)
  - Nails are spaced 2” o.c. (SDPWS 4.3.7.1(4))
  - 10d nails are spaced 3” o.c. and have penetration >1.5” (SDPWS 4.3.7.1(4))

- **See footnotes to shear wall tables!**
Framing at Adjoining Panel Edges

3/8” Min
Typ. 1/8” Gap Typ.

5/16” 13/16” 1-1/16”

2x_ framing 3x_ framing 2x_ framing

1-1/2 2-1/2

1-1/16”

3”

Two 2x stitch nailed per SDPWS 4.3.7.1(4)) Exception
Summing Shear Capacities

- Two sides sheathed = twice the strength (perforated: nominal unit capacity = 2435 plf max for wind, SDPWS 4.3.5.3)

- For wind design:
  - Gypsum shear wall strength can be added to wood shear wall strength (SDPWS 4.3.3.3.2)
Shear Walls: Wind v. Seismic

**Wind Design:**
- 40% increased capacity
- Gypsum strength can be added
- 3.5:1 max. aspect ratio

**Seismic Design:**
- Requires 3x framing more often (SDC D-F)
- 2:1 max. aspect ratio without penalty
- 3.5:1 permitted with penalty (2w/h)
Shear Walls: Wind v. Seismic

**Given:**
- 7/16” OSB
- 8d common
- 3”/ 6” edge/field nail spacing
- Gypsum on opposite face
Shear Walls: Wind v. Seismic

- **Wind Capacity:**
  - \( V = (630 \text{ plf}) \times 2.25' = 1418 \text{ lb} \)
    - Length of wall
    - From table

- **Seismic Capacity:**
  - \( V = 450 \text{ plf} \times 2(2.25')/8' \times 2.25' = 570 \text{ lb} \)
    - When less than 2:1 aspect ratio, 2w/h adjustment
Shear Walls: Wind v. Seismic

**Given:**
- 7/16” OSB
- 8d common
- 3”/ 6” edge/field nail spacing
- Gypsum on opposite face
Shear Walls: Wind v. Seismic

- **Wind Capacity:**
  - \( V = (630 \text{ plf} + 100 \text{ plf}) \times 5.33' = 3891 \text{ lb} \)
  - Length of wall
  - For gypsum from table
  - From table

- **Seismic Capacity:**
  - \( V = 450 \text{ plf} \times 5.33' = 2399 \text{ lb} \)
High Load Shear Walls
Maximum ASD Capacity

For two sides sheathed with Wood Structural Panels:

- Wind maximum  = 1740 plf x 1.4 = 2436 plf
- Earthquake maximum  = 1740 plf
High Load Shear Walls
Boundary Elements

- Hold down and chord forces due to lateral load only:

  - $H = 2435 \text{ plf} \times 8' = 19,480 \text{ lbs}$

- Hold down and chord forces can get very large!
Designing Shear Walls With Openings SDPWS 4.3.5

- Force transfer around openings
- Perforated
Reducing Hold-Down Anchorage

Segmented Shearwalls

Continuous Shearwalls
Force Transfer Around Openings

Design shall be based on rational analysis

The following method is described in detail in Design of Wood Structures

From Kelly Cobeen, S.E. used with permission
One-Story Wall Design: Transfer Around Openings - Concept
One-Story Wall Design:
Transfer Around Openings - Analysis

T = C = 3420 x 9' / 14' = 2200 #
One-Story Wall Design: Transfer Around Openings - Analysis

B & G: HORIZ v = 3420 # / (3.5' + 3.5') = 489 plf
D & E: VERT v = 2200 # / (1.5' + 2.5') = 550 plf
One-Story Wall Design: Transfer Around Openings - Analysis

F & H: HORIZ v = (1925 # / 3.5') - 489 plf = 61 plf
A & C: HORIZ v = 61 plf by symmetry
One-Story Wall Design: Transfer Around Openings - Aspect Ratio

Pier aspect ratio
= pier height / pier width = 5' / 3.5' = 1.43

For aspect ratio < 2, no allowable shear adjustment required
One-Story Wall Design: Transfer Around Openings - Boundary Members

Double top plates serve as collector

Double studs or posts serve as shear wall chords (vertical boundary members) carrying tension and compression

T = C = 2200 #

Edge nail sheathing for full height of studs or posts

Straps, studs and blocking on Lines 2, 3, 6 & 7 act as boundary members. Provide edge nailing for the full length of these lines.
Perforated Shear Wall Design

Definition SDPWS 4.3.5.3

Perforated Shear Wall – a wood structural panel sheathed shear wall with openings that has not been specifically designed and detailed for force transfer around the openings
Perforated Shear Wall Design

Definition SDPWS 4.3.4.1

Perforated Shear Wall segment – full height segment meeting aspect ratio limits
Perforated Shear Wall Design

Code sets specific limitations on the use of this method

- Limitations

- (SDPWS 4.3.5.3)
Limitations

1. Perforated Shear Wall segment required at each end of perforated shear wall
Perforated Shear Wall Design

Limitations

1. Openings are allowed beyond the ends of the perforated shear wall, but should not be included in the width of perforated shear wall.
Limitations

2. Nominal unit shear capacity for wind shall not exceed 2435 plf
Limitations

3. Out-of-plane offsets occur, walls shall be considered as separate perforated shear walls
Limitations

4. Collectors for shear transfer shall be provided through the full length of the perforated shear wall
Perforated Shear Wall Design

Limitations

5. A perforated wall shall have uniform top of wall and bottom of wall elevation. (otherwise use different method)

Perforated Shear Wall OK

Use other methods
Limitations

6. Maximum Perforated Wall height is 20 ft
Perforated Shear Wall Design

- Perforated Shear Wall Resistance
- (SDPWS 4.3.3.5)
Perforated Shear Wall Design

Resistance

1. Calculating percentage (%) of full-height sheathing

\[ \% = \frac{a_1 + a_2 + a_3 + a_4}{L} \]
2. The maximum opening height is the maximum opening clear height

- $5H/6 = 6'8''$
- $H/3 = 2'8''$

opening height
Perforated Shear Wall Design

Resistance

3. The unadjusted shear resistance shall be the allowable shear set in Table 2306.4.1 for h/w ratio of any perforated shear wall segments that do not exceed 2:1 for seismic forces and 3.5:1 for other forces.

- h/w = 3:1
- h/w = 2:1
Perforated Shear Wall Design

Maximum h/w ratio requirements for Perforated Shear Walls

<table>
<thead>
<tr>
<th>Load</th>
<th>Maximum h/w Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Seismic</td>
<td>2:1</td>
</tr>
<tr>
<td>Seismic (shear values in table adjusted by 2w/h)</td>
<td>2:1 &lt; h/w &lt; 3.5:1</td>
</tr>
<tr>
<td>Other than seismic</td>
<td>3.5 : 1</td>
</tr>
</tbody>
</table>

Based on SDPWS Table 4.3.4

Maximum Shear Wall Aspect Ratios
Perforated Shear Wall Design

Resistance

4. The adjusted shear resistance shall be calculated by multiplying the unadjusted shear resistance by the shear resistance adjustment factors of SDPWS Table 4.3.3.5 (interpolations are allowed)
# Perforated Shear Wall Design

## SDPWS Table 4.3.3.5 Shear Resistance Adjustment Factor, $C_o$

<table>
<thead>
<tr>
<th>WALL HEIGHT (h)</th>
<th>MAXIMUM OPENING HEIGHT RATIO$^a$ AND HEIGHT</th>
<th>SHEAR CAPACITY ADJUSTMENT FACTOR</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>h/3</td>
<td>h/2</td>
</tr>
<tr>
<td>8'-0&quot;</td>
<td>2'-8&quot;</td>
<td>4'-0&quot;</td>
</tr>
<tr>
<td>10'-0&quot;</td>
<td>3'-4&quot;</td>
<td>5'-0&quot;</td>
</tr>
<tr>
<td>Percent Full-Height Sheathing$^b$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>10%</td>
<td>1.00</td>
<td>0.69</td>
</tr>
<tr>
<td>20%</td>
<td>1.00</td>
<td>0.71</td>
</tr>
<tr>
<td>30%</td>
<td>1.00</td>
<td>0.74</td>
</tr>
<tr>
<td>40%</td>
<td>1.00</td>
<td>0.77</td>
</tr>
<tr>
<td>50%</td>
<td>1.00</td>
<td>0.80</td>
</tr>
<tr>
<td>60%</td>
<td>1.00</td>
<td>0.83</td>
</tr>
<tr>
<td>70%</td>
<td>1.00</td>
<td>0.87</td>
</tr>
<tr>
<td>80%</td>
<td>1.00</td>
<td>0.91</td>
</tr>
<tr>
<td>90%</td>
<td>1.00</td>
<td>0.95</td>
</tr>
<tr>
<td>100%</td>
<td>1.00</td>
<td>1.00</td>
</tr>
</tbody>
</table>
# Shear Capacity Adjustment

<table>
<thead>
<tr>
<th>Wall Height, h</th>
<th>Maximum Opening Height¹</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>h/3</td>
<td>h/2</td>
</tr>
<tr>
<td>8' Wall</td>
<td>2'-8&quot;</td>
<td>4'-0&quot;</td>
</tr>
<tr>
<td>10' Wall</td>
<td>3'-4&quot;</td>
<td>5'-0&quot;</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Percent Full-Height Sheathing²</th>
<th>Effective Shear Capacity Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>10%</td>
<td>1.00</td>
</tr>
<tr>
<td>20%</td>
<td>1.00</td>
</tr>
<tr>
<td>30%</td>
<td>1.00</td>
</tr>
<tr>
<td>40%</td>
<td>1.00</td>
</tr>
<tr>
<td>50%</td>
<td>1.00</td>
</tr>
<tr>
<td>60%</td>
<td>1.00</td>
</tr>
<tr>
<td>70%</td>
<td>1.00</td>
</tr>
<tr>
<td>80%</td>
<td>1.00</td>
</tr>
<tr>
<td>90%</td>
<td>1.00</td>
</tr>
<tr>
<td>100%</td>
<td>1.00</td>
</tr>
</tbody>
</table>

¹ The maximum opening height shall be taken as the maximum opening clear height in a perforated shear wall. Where areas above and/or below an opening remain unsheathed, the height of each opening shall be defined as the clear height of the opening plus the unsheathed areas.

² The sum of the perforated shear wall segment lengths, $\sum L$, divided by the total length of the perforated shear wall, $L_{w}$. Lengths of perforated shear wall segments with aspect ratios greater than 2:1 shall be adjusted in accordance with Section 4.3.4.3.
Shear Capacity Adjustment

Equation for Perforated Shearwalls

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

\[ r = \frac{1}{1 + \frac{A_o}{h \sum L_i}} \]
Shear Capacity Adjustment

Equation for Perforated Shearwalls

- 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
Perforated Shear Wall Design

Resistance

5. The perforated shear wall resistance shall be equal to the shear resistance times the sum of the width of the perforated shear wall segments.

\[ V = (v_{\text{allowable}}) \times (2w/h) \times (C_0) \times (a_1+a_2+a_3+a_4) \]
Perforated Shear Wall Design

- Uplift Anchorage

- Should either comply with the additional prescriptive code requirements or calculated using principles of mechanics.
4.3.6.4.2 Uplift Anchorage at Shear Wall Ends: Where the dead load stabilizing moment is not sufficient to prevent uplift due to overturning moments on the wall (from 4.3.6.1.1 or 4.3.6.1.2), an anchoring device shall be provided at the end of each shear wall.

Shear in Perforated Shear Wall

Shear Resistance Adjustment Factor

\[ T = \frac{Vh}{C_0 \Sigma L_i} \]

Sum of width of perforated wall segments
Perforated Shear Wall Design

4.3.6.4.1.1 In-plane Shear Anchorage for Perforated Shear Walls: The maximum induced unit shear force, $v_{\text{max}}$, transmitted into the top of a perforated shear wall, out of the base of the perforated shear wall at full height sheathing, and into collectors connecting shear wall segments, shall be calculated in accordance with the following:

$$v = \frac{V}{C_0 \Sigma L_i}$$

Shear Resistance Adjustment Factor

Sum of width of perforated wall segments
Perforated Shear Wall Design

4.3.6.4.2.1 Uplift Anchorage for Perforated Shear Walls: In addition to the requirements of 4.3.6.4.2, perforated shear wall bottom plates at full height sheathing shall be anchored for a uniform uplift force, \( t \), equal to the unit shear force, \( v_{\text{max}} \), determined in 4.3.6.4.1.1, or calculated by rational analysis.

\[
v = \frac{V}{C_o \Sigma L_i}
\]
Summary

- Prescribed forces for shear and uplift connections ensure that the capacity of the wall is governed by the sheathing to framing attachment (shear wall nailing) and not bottom plate attachment for shear and/or uplift.
Method Comparison Summary

- Which method works “best” will depend
Reference:

Examples of:
Shear Wall Design
Deflection Calculations
APA Publications and Website

Free APA publications
www.APAnwood.org

Diaphragms and Shear Walls

APA Designers Circle
APAwood.org/DesignersCircle
Questions?

Bryan Readling, P.E.
bryan.readling@apawood.org

APA Help Desk
help@apawood.org
253-620-7400
Alternate Wood Frame and Hybrid Lateral Force Resisting Systems

October, 2015
Engineered Shear Wall Systems w/ WSP

Stapled Shear Walls

• Capacities in IBC 2306

Fig. 1. Cross section of typical standard shear wall and midply wall

Source: Journal of Structural Engineering, 2007

Source: nees.org
Gypsum Shearwalls

- Lower capacities than WSP Shearwalls (about 1/3 capacity)
- SDPWS Table 4.3C, Section 4.3.7.5 provides capacities & requirements
- Not permitted in SDC E or F
Non-WSP Engineered Shear Wall Systems

Proprietary Trussed Shear Walls

Manufacturer Provides Capacities & Stamped Design

Source: smartcomponents.us
Non-WSP Engineered Shear Wall Systems

Horizontal & Diagonal Board Sheathing

Capacities in AWC’s SDPWS Table 4.3D

Source: firstdayonpei

Source: johnotvos
Open Front & Narrow Walls
Using Prefab Shearwalls

Considerations:

• Engineered Narrow Wall Section
• Proprietary
• Large Hold-down forces
• Deflections
• Manufacturer Provides Wall Capacity
Proprietary Portal Frame Systems

Source: strongtie.com
Prefab Shearwall Anchorage

SLAB ON GRADE FOUNDATION

CURB OR STEMWALL FOUNDATION

Source: strongtie.com
**Portal Frame Systems**

**Allowable Design Shear Values**

<table>
<thead>
<tr>
<th>Min. Width (in.)</th>
<th>Max. Height (ft.)</th>
<th>Shear (lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td>16</td>
<td>8</td>
<td>850</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>625</td>
</tr>
<tr>
<td>24</td>
<td>8</td>
<td>1,675</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>1,125</td>
</tr>
</tbody>
</table>

*Figure 1. Construction Details for APA Portal-Frame Design with Hold Downs*

- Extent of header with double portal frames (two braced wall panels)
- Extent of header with single portal frame (one braced wall panel)
- 2' to 18' rough width of opening for single or double portal
- Min. 3" x 11-1/4" net header steel header not allowed
- Fasten sheathing to header with 8d common or galvanized box nails at 3" grid pattern as shown
- Header to jack-stud strap per wind design. Min 1000 lb on both sides of opening opposite side of sheathing.
- Min. double 2x4 framing covered with min. 3/8" thick wood structural panel sheathing with 8d common or galvanized box nails at 3" o.c. in all framing (studs, blocking, and sill) typ.
- Min length of panel per table 1
- Min. 1500 lb hold-downs (embedded into concrete and nailed into framing)
- Min reinforcing of foundation, one #4 bar top and bottom of footing. Lap bars 15' min.
- Min footing size under opening is 12' x 12'. A turned-down slab shall be permitted at door openings.
- Min [1] 5/8" diameter anchor bolt installed per IRC R403.1.6 - with 2" x 2" x 3/16" plate washer

**APA Report TT-100F**
Hybrid Wood/Steel Prefab Shearwalls

Source: hardyframe.com
Hybrid Wood/Steel Prefab Shearwalls

Source: hardyframe.com
Post Frame Buildings – Lateral Options

- Kickers/Knee Braces
- Sheathed Walls/Roof
- Steel Rod X-Bracing
- Others

Source: newenglandbarn.com
Heavy Timber Braced Frames (HTBF)

Heavy timber braced frames are becoming a preferred alternative vertical/lateral resisting system due to cost, performance and aesthetics.
Hybrid Wood/Steel Braced Frames

The Bullitt Center
Architect: The Miller Hull Partnership
Photo: John Stamets
Questions?

It’s Lunch Time!

Visit www.woodworks.org for more educational materials, case studies, design examples, a project gallery, and more