Overview

- Diaphragms
- Shearwalls
Diaphragm Design
Wind Load Distribution to Diaphragm

WIND INTO DIAPHRAGMS

WIND SURFACE LOADS ON WALLS
Wind Load Paths

Wind into diaphragms as uniform linear loads
Wind Load Paths

Diaphragms span between shearwalls as concentrated loads.
Stud to Diaphragm

DIAPHRAGM SHEATHING

WIND LOAD

Floor/Roof framing perpendicular to walls
Stud to Diaphragm

Floor/Roof framing parallel to walls (add blocking)
Calculating Diaphragm Forces

**Figure 1** Simple Beam – Uniformly Distributed Load

\[
R = V = \frac{w\ell}{2}
\]

\[
V_x = w\left(\frac{\ell}{2} - x\right)
\]

\[
M_{\text{max}} \text{ (at center)} = \frac{w\ell^2}{8}
\]

\[
M_x = \frac{wx}{2}(\ell - x)
\]

Max Shear at Ends

Max Moment at Mid-Span
Calculating Diaphragm Forces

Diaphragm Shear:
- Max Shear = Diaphragm Reaction at Shearwall
- Diaphragm Unit Shear = Reaction / Length of Diaphragm = plf
Unblocked Diaphragm
Unblocked Diaphragm Capacity

- Capacities in SDPWS are **Nominal** values. Not ASD
  
  *Divide Nominal Values by 2.0 for ASD Capacity*
  
  *Multiply Nominal Values by 0.8 for LRFD Capacity*

- Capacity is reduced for species with Specific Gravity < 0.5
- For Spruce Pine Fir multiply by 0.92

**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 Fastener Penetration in Framing (in.)</th>
<th>Minimum Nominal Panel Thickness (in.)</th>
<th>Minimum Nominal Width of Nailed Face at Supported Edges and Boundaries (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Structural I</td>
<td>6d</td>
<td>1-1/4</td>
<td>5/16</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>8d</td>
<td>1-3/8</td>
<td>3/8</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>10d</td>
<td>1-1/2</td>
<td>15/32</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>6d</td>
<td>1-1/4</td>
<td>3/8</td>
<td>2</td>
</tr>
</tbody>
</table>

**A SEISMIC**

<p>| 6 in. Nail Spacing at diaphragm boundaries and supported panel edges |
|-------------------------------------------------|-----------------|-----------------|-----------------|-----------------|-----------------|-----------------|-----------------|</p>
<table>
<thead>
<tr>
<th>Case 1</th>
<th>Cases 2,3,4,5,6</th>
<th>v_x (plf)</th>
<th>G_s (kips/in.)</th>
<th>v_x (plf)</th>
<th>G_s (kips/in.)</th>
<th>v_x (plf)</th>
<th>G_s (kips/in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>OSB</td>
<td>OSB</td>
<td>330</td>
<td>9.0</td>
<td>370</td>
<td>7.0</td>
<td>480</td>
<td>8.5</td>
</tr>
<tr>
<td>PLY</td>
<td>PLY</td>
<td>7.0</td>
<td>6.0</td>
<td>280</td>
<td>4.5</td>
<td>530</td>
<td>7.5</td>
</tr>
<tr>
<td>OSB</td>
<td>OSB</td>
<td>280</td>
<td>4.5</td>
<td>570</td>
<td>9.0</td>
<td>640</td>
<td>8.0</td>
</tr>
<tr>
<td>PLY</td>
<td>PLY</td>
<td>6.0</td>
<td>4.0</td>
<td>430</td>
<td>9.5</td>
<td>480</td>
<td>6.0</td>
</tr>
<tr>
<td>OSB</td>
<td>OSB</td>
<td>220</td>
<td>6.0</td>
<td>300</td>
<td>9.0</td>
<td>340</td>
<td>7.0</td>
</tr>
<tr>
<td>PLY</td>
<td>PLY</td>
<td>6.0</td>
<td>4.0</td>
<td>250</td>
<td>5.0</td>
<td>330</td>
<td>7.5</td>
</tr>
<tr>
<td>OSB</td>
<td>OSB</td>
<td>240</td>
<td>4.0</td>
<td>250</td>
<td>5.0</td>
<td>370</td>
<td>8.0</td>
</tr>
<tr>
<td>PLY</td>
<td>PLY</td>
<td>4.0</td>
<td>3.0</td>
<td>240</td>
<td>6.0</td>
<td>390</td>
<td>4.0</td>
</tr>
</tbody>
</table>

**B WIND**

<table>
<thead>
<tr>
<th>6 in. Nail Spacing at diaphragm boundaries and supported panel edges</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case 1</td>
</tr>
<tr>
<td>---------</td>
</tr>
<tr>
<td>460</td>
</tr>
<tr>
<td>520</td>
</tr>
<tr>
<td>670</td>
</tr>
<tr>
<td>740</td>
</tr>
<tr>
<td>895</td>
</tr>
<tr>
<td>420</td>
</tr>
<tr>
<td>475</td>
</tr>
<tr>
<td>520</td>
</tr>
</tbody>
</table>
Blocked Diaphragm
Blocked Diaphragm Capacity

- Capacities in SDPWS are **Nominal** values. Not ASD
  
  *Divide Nominal Values by 2.0 for ASD Capacity*
  *Multiply Nominal Values by 0.8 for LRFD Capacity*

- Capacity is reduced for species with Specific Gravity < 0.5
- For Spruce Pine Fir multiply by 0.92

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

<table>
<thead>
<tr>
<th>Sheathing Grade</th>
<th>Common Nail Size</th>
<th>Minimum Fastener Penetration in Framing Member or Blocking (in.)</th>
<th>Minimum Nominal Panel Thickness (in.)</th>
<th>Minimum Nominal Width of Nailed Face at Adjoining Panel Edges and Boundaries (in.)</th>
<th>Seismic Nail Spacing (in.) at diaphragm boundaries (all cases), at continuous panel edges parallel to load (Cases 3 &amp; 4), and at all panel edges (Cases 5 &amp; 6)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Structural 1</td>
<td>6d</td>
<td>1-1/4</td>
<td>5/16</td>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td>8d</td>
<td>1-3/8</td>
<td>3/8</td>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td>10d</td>
<td>1-1/2</td>
<td>15/32</td>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td>6d</td>
<td>1-1/4</td>
<td>5/16</td>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td>3/8</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sheathing</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Shear Wall Capacities in AWC SDPWS

<table>
<thead>
<tr>
<th>B</th>
<th>WIND</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Nail Spacing (in.) at diaphragm boundaries (all cases), at continuous panel edges parallel to load (Cases 3 &amp; 4), and at all panel edges (Cases 5 &amp; 6)</td>
</tr>
<tr>
<td></td>
<td>6</td>
</tr>
<tr>
<td></td>
<td>Nail Spacing (in.) at other panel edges (Cases 1, 2, 3, &amp; 4)</td>
</tr>
<tr>
<td></td>
<td>6</td>
</tr>
<tr>
<td>$v_w$ (plf)</td>
<td>$v_w$ (plf)</td>
</tr>
<tr>
<td>520</td>
<td>700</td>
</tr>
<tr>
<td>590</td>
<td>785</td>
</tr>
<tr>
<td>755</td>
<td>1010</td>
</tr>
<tr>
<td>840</td>
<td>1120</td>
</tr>
<tr>
<td>895</td>
<td>1190</td>
</tr>
<tr>
<td>1010</td>
<td>1345</td>
</tr>
<tr>
<td>475</td>
<td>630</td>
</tr>
<tr>
<td>530</td>
<td>700</td>
</tr>
<tr>
<td>520</td>
<td>700</td>
</tr>
<tr>
<td>590</td>
<td>785</td>
</tr>
<tr>
<td>670</td>
<td>895</td>
</tr>
<tr>
<td>755</td>
<td>1010</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>B</th>
<th>WIND</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>6 in. Nail Spacing at diaphragm boundaries and supporting members</td>
</tr>
<tr>
<td></td>
<td>Case 1</td>
</tr>
<tr>
<td>$v_w$ (plf)</td>
<td>$v_w$ (plf)</td>
</tr>
<tr>
<td>460</td>
<td>350</td>
</tr>
<tr>
<td>520</td>
<td>390</td>
</tr>
<tr>
<td>670</td>
<td>505</td>
</tr>
<tr>
<td>740</td>
<td>560</td>
</tr>
<tr>
<td>800</td>
<td>600</td>
</tr>
<tr>
<td>895</td>
<td>670</td>
</tr>
<tr>
<td>420</td>
<td>310</td>
</tr>
<tr>
<td>475</td>
<td>350</td>
</tr>
<tr>
<td>460</td>
<td>350</td>
</tr>
<tr>
<td>520</td>
<td>390</td>
</tr>
<tr>
<td>600</td>
<td>450</td>
</tr>
<tr>
<td>670</td>
<td>505</td>
</tr>
</tbody>
</table>
Diaphragm Types

CASE 1 DIAPHRAGM
• Higher Shear Values
• Panels perpendicular to floor framing for improved performance

CASES 2-6 May be preferred for low shear demand where changing framing direction helps
• HVAC runs
• Fire Blocking/Draft Stopping

N-S
4x8 sheathing
Roof Trusses
Diaphragm Types

SDPWS Tables 4.2A & B
### Diaphragm Aspect Ratio

**Table 4.2.4** Maximum Diaphragm Aspect Ratios

(Horizontal or Sloped Diaphragms)

<table>
<thead>
<tr>
<th>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>

SDPWS Table 4.2.4
Calculating Diaphragm Forces

Diaphragm Fastener Schedule

Zone A 12'  Zone B 48'  Zone A 12'

72'  24'
Diaphragm – Bending Member

Tension edge

Compression edge

DEFLECTED SHAPE OF DIAPHRAGM

REACTION PROVIDED BY SHEARWALL

UF = TRANSVERSE LATERAL FORCE

END SHEARWALL
Diaphragm Chord Forces:

- Max Chord Force Occurs at Location of Max Moment
- Chord Force = T or C
- Chord Force = $\frac{M_{\text{MAX}}}{\text{Diaphragm Depth}}$
- Chord Unit Shear = Chord Force / Length of Diaphragm = plf
Diaphragm Chords

Wall Top Plates Typically Function as Both Diaphragm Chords and Drag Struts
Diaphragm Boundary

Reaction = 200 plf * 24'/2 = 2400 lbs
Diaphragm Only at Shearwall = 2400 lbs / 16' = 150 plf
Diaphragm Boundary

Does this mean that no drag struts are required?
Diaphragm Boundary

All edges of a diaphragm shall be supported by a boundary element. (ASCE 7-10 Section 11.2)

• **Diaphragm Boundary Elements:**
  • Chords, drag struts, collectors, Shear walls, frames
  • Boundary member locations:
    • Diaphragm and shear wall perimeters
    • Interior openings
    • Areas of discontinuity
    • Re-entrant corners.
Example: Retail Restaurant

Assume Basic Wind Speed = 115 mph Ultimate Exposure B

Diaphragm Design
  • Capacity

Shearwall Design
  • Conventional
  • Force Transfer Around Opening
  • Perforated Shearwall
Retail Restaurant – Diaphragm Design

Critical Shearwall at front of building

Check Diaphragm for wind loads on 84’ wall
## Diaphragm Aspect Ratios

**SDPWS TABLE 4.2.4**

**TYPE - MAXIMUM LENGTH/WIDTH RATIO**

<table>
<thead>
<tr>
<th>Type</th>
<th>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>

For an 84 x 34 diaphragm the aspect ratio is 2.5 < 3. Diaphragm aspect ratio is OK.
Calculating MWFRS Wind Loads

Calculate wind pressure using Directional Method (ASCE 7 Chpt 27)

\[ p = q_h[(G_{cpf})-(G_{cpi})] \]

\[ q_h = 0.00256 \times 0.57 \times 1.0 \times 0.85 \times 115^2 \times 1 = 16.4 \text{ psf} \]

\[ G_{cpf} = 0.85 \times [0.8 - (-0.3)] = 0.935 \]

\[ G_{cpi} = 0.18 - 0.18 = 0 \]

\[ p = (16.4 \text{ psf})(0.935) = 15.34 \text{ psf} \]

\[ 0.6*W = 0.6 \times 15.34 = 9.2 \text{ psf on walls} \]

Use min 9.6 psf per ASCE 27.1.5

<table>
<thead>
<tr>
<th>Surface</th>
<th>L/B</th>
<th>( C_p )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Windward Wall</td>
<td>All values</td>
<td>0.8</td>
</tr>
<tr>
<td></td>
<td>0-1</td>
<td>-0.5</td>
</tr>
<tr>
<td>Leeward Wall</td>
<td>2</td>
<td>-0.3</td>
</tr>
<tr>
<td></td>
<td>( \geq 4 )</td>
<td>-0.2</td>
</tr>
</tbody>
</table>

ASCE 7-10 Figure 27.4-1
At parapets windward and leeward pressures occur on each parapet.

**Section 27.4.5:**  \( P_p = q(GC_{pn}) \)

- **Windward parapet, \( GC_{pn} = 1.5 \)**:  \( 16.4 \times 1.5 \times 0.6 = 14.76 \text{ psf} \)
- **Leeward parapet, \( GC_{pn} = -1.0 \)**:  \( 16.4 \times 1.0 \times 0.6 = 9.84 \text{ psf} \)

**Net Parapet** = 14.76 + 9.84 = 24.6 psf
Retail Restaurant – Diaphragm Design

W = (9.6 psf * (5’+3’)+(24.6) * 3’) = 150.6 plf
V = (150.6 plf) * (84’/2) = 6,325 lb
M = (150.6 plf) * (84’^2)/8 = 132,829 lb*ft
T = C = (132,829 lb*ft)/(34 ft) = 3,907 lb

ν_{diaphragm} = 6,325 lb/34’ = 186 plf
ν_{diaphragm} = 3,907 lb/84’ = 47 plf
Diaphragm Capacity: SDPWS Table 4.2C

<table>
<thead>
<tr>
<th>PANEL GRADE</th>
<th>COMMON NAIL SIZE OR STAPLE† LENGTH AND GAGE</th>
<th>MINIMUM FASTENER PENETRATION IN FRAMING</th>
<th>MINIMUM NOMINAL WIDTH OF FRAMING MEMBERS AT ADJOINING PANEL EDGES AND BOUNDARIES§</th>
<th>NAIL SPACING AT ALL PANEL EDGES</th>
<th>Case 1 (No unblocked edges or continuous joints parallel to load)</th>
<th>All other configurations (Cases 2, 3, 4, 5 and 6)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sheathing &amp; single floor</td>
<td>8d (2½ “ x 0.131”)</td>
<td>1 3/8”</td>
<td>2 IN.</td>
<td>6 IN.</td>
<td>460 (Seismic) 645 (Wind)</td>
<td>340 (Seismic) 475 (Wind)</td>
</tr>
<tr>
<td></td>
<td>7/16”</td>
<td>3IN.</td>
<td>6 IN.</td>
<td>510 (Seismic) 715 (Wind)</td>
<td>380 (Seismic) 530 (Wind)</td>
<td></td>
</tr>
</tbody>
</table>

Capacity is reduced for species with Specific Gravity < 0.5. For Spruce Pine Fir multiply by 0.92

**Capacity** = (645 plf)(0.92)/2 = 297 plf

297 plf > 186 plf, diaphragm is adequate with sheathing & fastening as shown above
Small Openings in Diaphragms

Accounting for openings in shear panels (diaphragms and shear walls) is a code requirement (IBC 2305.1.1)

No code path for checking minimum size opening limit (other than prescriptive design – IBC 2308.4.4.1 & 2308.7.6.1)

Do you need to account for a 12” square opening in a diaphragm?

Small Openings in Diaphragms

FPInnovations method for checking small holes in diaphragms:

Recommend running an analysis of the opening’s effects on the diaphragm unless the following conditions are met.

3. It is strongly recommended that analysis for a diaphragm with an opening should be carried out except where all four of the following items are satisfied:

   a. Opening depth no greater than 15% of diaphragm depth;
   b. Opening length no greater than 15% of diaphragm length;
   c. Distance from diaphragm edge to the nearest opening edge is a minimum of 3 times the larger opening dimension; and
   d. The diaphragm portion between opening and diaphragm edge satisfies the maximum aspect ratio requirement.
Overview

- Diaphragms
- Shearwalls
Shearwall Functions

Wind Loads create shear (sliding) and racking forces on a structure

Sliding resisted by shearwall base anchorage
Racking resisted by shear panel & fasteners
Components of Shear Wall Design

Collector & Drag Design

Shear Wall Construction

Shear Transfer Detailing

Shear Resistance
Shear Wall Configuration Options

Solid or Segmented Walls

Perforated Walls

Maximum ASD Capacity of 870 plf (Seismic) 1217 plf (Wind)

Useful, If Necessary.

Force Transfer Around Openings Walls
Shearwalls

WOOD STUDS

HOLD-DOWN

WSP SHEATHING

ANCHOR BOLTS
Racked Shearwall

EDGE NAILING PROVIDES RACKING RESISTANCE
Panel Fasteners
Shearwalls

ANCHOR BOLTS TO FOUNDATION

PREVENT SLIDING!
Hold-Downs Resist End Uplift
Shear Wall Requirements in AWC SDPWS

3:5:1 max aspect ratio for blocked Wood Structural Panel Shear Wall. Reduction in Capacity when greater than 2:1
# Shearwall Aspect Ratio

## NDS SDPWS TABLE 4.3.4

### MAXIMUM SHEAR WALL DIMENSION RATIOS

<table>
<thead>
<tr>
<th>Material</th>
<th>For other than seismic:</th>
<th>For seismic:</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wood structural panels, blocked</td>
<td>3½:1&lt;sup&gt;1&lt;/sup&gt;</td>
<td>2:1&lt;sup&gt;1&lt;/sup&gt;</td>
</tr>
<tr>
<td>Wood structural panels, unblocked</td>
<td></td>
<td>2:1</td>
</tr>
<tr>
<td>Diagonal sheathing, single</td>
<td></td>
<td>2:1</td>
</tr>
<tr>
<td>Structural Fiberboard</td>
<td></td>
<td>3½:1&lt;sup&gt;3&lt;/sup&gt;</td>
</tr>
<tr>
<td>Gypsum board, portland cement plaster</td>
<td></td>
<td>2:1&lt;sup&gt;2&lt;/sup&gt;</td>
</tr>
</tbody>
</table>

See SDPWS Table 4.3.4 for footnotes
WSP Shearwall Capacity

- Capacities listed in AWC’s Special Design Provisions for Wind and Seismic (SDPWS)
- Sheathed shear walls most common. Can also use horizontal and diagonal board sheathing, gypsum panels, fiberboard, lath and plaster, and others
- Blocked shear walls most common. SDPWS has reduction factors for unblocked shear walls
- Note that capacities are given as nominal: must be adjusted by a reduction or resistance factor to determine allowable unit shear capacity (ASD) or factored unit shear resistance (LRFD)
### Shearwall Capacity - SDPWS Chpt 4

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

<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>Panel Edge Fastener Spacing (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Wood Structural Panels - Structural r5</td>
<td>3/8</td>
<td>1-3/8</td>
<td>8d</td>
<td>460 19 14 720 24 17</td>
<td>645 1010 1290 1710</td>
</tr>
<tr>
<td>Wood Structural Panels - Sheathing r5</td>
<td>3/8</td>
<td>1-3/8</td>
<td>8d</td>
<td>480 15 11 700 22 14</td>
<td>715 1105 1415 1875</td>
</tr>
<tr>
<td>Plywood Siding</td>
<td>5/16</td>
<td>1-3/8</td>
<td>6d</td>
<td>360 13 9.5 540 18 12</td>
<td>560 755 980 1260</td>
</tr>
<tr>
<td>Particleboard Sheathing - M5 &quot;Exterior Glue&quot; and M2 &quot;Exterior Glue&quot;</td>
<td>3/8</td>
<td>1/2</td>
<td>6d</td>
<td>240 15 360 17</td>
<td>335 505 645 840</td>
</tr>
<tr>
<td>Structural Fiberboard Sheathing</td>
<td>1/2</td>
<td>11 ga. galv. roofing nail (0.120&quot; x 1-1/2&quot; long x 7/16&quot; head)</td>
<td>11 ga. galv. roofing nail (0.120&quot; x 1-3/4&quot; long x 3/8&quot; head)</td>
<td>340 4.0 460 5.0 520 5.5</td>
<td>475 645 730</td>
</tr>
<tr>
<td>Sheathing Material</td>
<td>Minimum Nominal Panel Thickness (in.)</td>
<td>Minimum Fastener Penetration in Framing Member or Blocking (in.)</td>
<td>Fastener Type &amp; Size</td>
<td>Panel Edge Fastener Spacing (in.)</td>
<td></td>
</tr>
<tr>
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<td>B WIND</td>
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<td>⎯ ⎯ ⎯ ⎯ ⎯ ⎯ ⎯ ⎯ ⎯ ⎯ ⎯ ⎯</td>
<td></td>
</tr>
<tr>
<td>Wood Structural Panels - Structural</td>
<td>5/16</td>
<td>1-1/4</td>
<td>6d</td>
<td>645 840 1090 1430</td>
<td></td>
</tr>
<tr>
<td>7/16²</td>
<td>3/8²</td>
<td>1-3/8</td>
<td>8d</td>
<td>715 1105 1415 1875</td>
<td></td>
</tr>
<tr>
<td>7/16²</td>
<td>15/32</td>
<td>1-3/8</td>
<td>8d</td>
<td>785 1205 1540 2045</td>
<td></td>
</tr>
<tr>
<td>15/32</td>
<td></td>
<td>1-1/2</td>
<td>10d</td>
<td>950 1430 1860 2435</td>
<td></td>
</tr>
<tr>
<td>Wood Structural Panels - Sheathing</td>
<td>5/16</td>
<td>1-1/4</td>
<td>6d</td>
<td>505 755 980 1260</td>
<td></td>
</tr>
<tr>
<td>3/8</td>
<td></td>
<td></td>
<td></td>
<td>560 840 1090 1430</td>
<td></td>
</tr>
<tr>
<td>3/8²</td>
<td></td>
<td></td>
<td></td>
<td>615 895 1150 1485</td>
<td></td>
</tr>
<tr>
<td>7/16²</td>
<td>15/32</td>
<td>1-3/8</td>
<td>8d</td>
<td>670 980 1260 1640</td>
<td></td>
</tr>
<tr>
<td>15/32</td>
<td></td>
<td>1-1/2</td>
<td>10d</td>
<td>730 1065 1370 1790</td>
<td></td>
</tr>
<tr>
<td>19/32</td>
<td></td>
<td></td>
<td></td>
<td>870 1290 1680 2155</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>950 1430 1860 2435</td>
<td></td>
</tr>
</tbody>
</table>
Shearwall Capacity - SDPWS Chpt 4

Capacity based on blocked shearwall. Reduce capacities for unblocked

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>
<th>Supported Edges</th>
<th>Intermediate Framing</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>12</td>
<td>16</td>
<td>20</td>
</tr>
<tr>
<td>Supported Edges</td>
<td>6</td>
<td>6</td>
<td>1.0</td>
</tr>
<tr>
<td>Intermediate Framing</td>
<td>6</td>
<td>12</td>
<td>0.8</td>
</tr>
</tbody>
</table>
Components of Shear Wall Design

- Holdown
- Tension
- Compression
- Boundary Posts
- Anchorage
Shearwall Hold Downs

Source: strongtie.com
Shearwall Hold Downs

Bucket Style

Source: DartDesignInc.com

Source: strongtie.com
Components of Shear Wall Design

- Overturning restrained connection at bottom of story
- Accumulated tension from framing to hardware to framing at each floor level

Discrete Holdown Systems

- 3 kips
- 2 kips
- 1 kip

Dimensions:
- 10 ft, typ
- 20 ft

Instructions:
- Framing to hardware
- Hardware to framing
- Hardware to concrete
Components of Shear Wall Design

- Collector & Drag Design
- Shear Wall Construction
- Shear Transfer Detailing

Shear Resistance
Shear Transfer Detailing

Figure 5A. Typical Platform Floor Framing at Wall Using Sawn Joists

Source: WoodWorks Five-Story Wood-Frame Structure over Podium Slab Design Example
Shear Transfer Detailing

Figure 5. Typical Floor Framing at Wall

Design a complete load path
Components of Shear Wall Design

- Collector & Drag Design
- Shear Wall Construction
- Shear Transfer Detailing
- Shear Resistance
Collector & Drag (Diaphragm) Design

Load Path, Load Path, Load Path...

**IBC 1604.4 Analysis.** Load effects on structural members and their connections shall be determined by methods of structural analysis that take into account equilibrium, general stability, geometric compatibility and both short- and long-term material properties.

[...]

Any system or method of construction to be used shall be based on a rational analysis in accordance with well-established principles of mechanics. **Such analysis shall result in a system that provides a complete load path capable of transferring loads from their point of origin to the load-resisting elements.**

Resources:

**The Analysis of Irregular Shaped Diaphragms.** Whitepaper by R. Terry Malone


**The Analysis of Irregular Shaped Structures.** Textbook by the R. Terry Malone


Retail Restaurant – Shearwall Design

$P = 6,325 \text{ lb} – \text{from diaphragm calcs using Directional Method}$

Let’s see what happens when we use Envelope Method to calculate MWFRS loads to front shearwall.
Calculating MWFRS Wind Loads

Calculate wind pressure using Envelope Method (ASCE 7 Chpt 28)

\[ p = q_h [(GC_{pf}) - (GC_{pi})] \]

\[ q_h = 0.00256 * 0.70 * 1.0 * 0.85 * 115^2 * 1 = 20.14 \text{ psf} \]

\[ GC_{pf} (Zones \ 1 \ & \ 4) = 0.4 - (-0.29) = 0.69 \] (ASCE 7 Fig. 28.4-1)

\[ GC_{pf} (Zones \ 1E \ & \ 4E) = 0.61 - (-0.43) = 1.04 \] (ASCE 7 Fig. 28.4-1)

\[ GC_{pi} = 0.18 - 0.18 = 0 \]

\[ P_{1&4} = (20.14 \text{ psf})(0.69) = 13.9 \text{ psf}; 0.6*W = 0.6*13.9 = 8.3 \text{ psf walls typ.} \]

\[ P_{1E&4E} = (20.14 \text{ psf})(1.04) = 20.9 \text{ psf}; 0.6*W = 0.6*20.9 = 12.5 \text{ psf walls crnr} \]
Calculating MWFRS Wind Loads

\[ a = \text{Lesser of:} \]

- \[ 10\% \text{ least horizontal dimension (LHD)} \times 0.1 = 3.4’ \]
- \[ 0.4h = 0.4 \times 13’ = 5.2’ \]

But not less than:

- \[ 0.04 \text{ LHD}=1.4’ \text{ or } 3’ \]

Use \( a = 3.4’ \) for zones 1E & 4E

\[ 2a = 3.4’ \times 2 = 6.8’ \]
Parapet Design – Section 28.4.2

Section 28.4.2:  \( P_p = q(GC_{pn}) \)

\( GC_{pn} = 1.5 \) Windward parapet, -1.0 Leeward parapet

Windward Parapet  \( GC_{pf} \) is 1.5:  \( 20.14 \times 1.5 \times 0.6 = 18.12 \text{ psf} \)

Leeward Parapet  \( GC_{pf} \) is 1.0:  \( 20.14 \times 1.0 \times 0.6 = 12.08 \text{ psf} \)

Net Parapet = 18.12 + 12.08 = 30.2 psf
Retail Restaurant – Shearwall Design

P = (8.3 psf * (5'+3') + (30.2) * 3') * (84'/2) + ((12.5 psf - 8.3 psf) * (5'+3')) * 6.8' * (77.2'/84') = **6,804 lb**

(for comparison: Directional method gave us 6,325 lb)
Shearwall Aspect Ratios

- Check Aspect Ratios: Assume blocked WSP Shearwall
- 10’/2’ = 5 > 3.5; Inadequate
- 10’/6’ = 1.67 < 3.5; OK

Front Wall Elevation
Shearwall Aspect Ratios

- Check Aspect Ratios: Assume blocked WSP Shearwall
- 10’/2’ = 5 > 3.5; Inadequate
- 10’/6’ = 1.67 < 3.5; OK

\[ \nu_{\text{shearwall}} = \frac{6,804 \text{ lb}}{12'} = 567 \text{ plf} \]
Conventional Shearwall Capacities

\[ \nu_{\text{shearwall}} = 567 \text{ plf} \]

Assume 15/32”, Structural I sheathing attached with 8d nails

Nominal Tabulated Capacity = 1540 plf
Adjusted ASD Capacity = \((1370 \text{ plf})(0.92)/2 = 630 \text{ plf}\)
630 plf > 567 plf, OK
8d nails at 3” o.c. acceptable

<table>
<thead>
<tr>
<th>PANEL GRADE</th>
<th>FASTENER TYPE &amp; SIZE</th>
<th>MINIMUM PANEL THICKNESS</th>
<th>MINIMUM FASTENER PENETRATION IN FRAMING</th>
<th>NAIL SPACING AT ALL PANEL EDGES</th>
<th>PANEL EDGE FASTENER SPACING</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wood Structural Panels – Sheathing</td>
<td>8d (2½ “ x 0.131”)</td>
<td>15/32”</td>
<td>1 3/8”</td>
<td>3 IN.</td>
<td>980 (Seismic) 1370 (Wind)</td>
</tr>
</tbody>
</table>

SDPWS Table 4.3A
Conventional Shearwall Overturning

\[ \nu_{\text{shearwall}} = 567 \text{ plf} \]

Hold downs required at shearwalls

\[ T = \nu h \]

\[ T = 567 \text{ plf} \times 10' = 5,670 \text{ lb} \]

Hold down capacity = 7,045 lb

Many available prefabricated hold downs with capacities listed by manufacturers
Hold-Down Anchor
Conventional Shearwall Overturning

\[ \nu_{\text{shearwall}} = 567 \text{ plf} \]

Posts are also required at ends of the wall to resist compression forces

\[ C = T = \nu h \]

\[ C = 567 \text{ plf} \times 10' = 5,670 \text{ lb} \]

Size post for bearing on wall sole plate

Assume 2x6 wall,

Required post width =

\[ \frac{5,670 \text{ lb}}{(565 \text{ psi})(5.5 \text{ in})} = 1.8 \text{ in} \]

Use **2-2x6 post** min.
Conventional Shearwall Base Anchorage

- $\nu_{\text{shearwall}} = 567$ plf
- $\frac{1}{2}''$ Anchor Bolt capacity for wood bearing = $680 \text{ lb} \times 1.6 = 1,088 \text{ lb}$ per NDS Table 11E
- Spacing = $1088 \text{ lb}/567 \text{ plf} = 1'-11''$ o.c. max.

---

Table 11E BOLTS: Reference Lateral Design Values, Z, for Single Shear (two member) Connections

<table>
<thead>
<tr>
<th>Thickness</th>
<th>Embodiment Depth in Concrete</th>
<th>Side Member</th>
<th>Bolt Diameter</th>
<th>G=0.67 Red Oak</th>
<th>G=0.55 Mixed Maple</th>
<th>G=0.55 Southern Pine</th>
<th>G=0.50 Douglas Fir-Larch</th>
<th>G=0.49 Douglas Fir-Larch(N)</th>
<th>G=0.46 Douglas Fir(S) Hem Fir(N)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-1/2</td>
<td>1/2</td>
<td>770</td>
<td>480</td>
<td>660</td>
<td>310</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>5/8</td>
<td>1070</td>
<td>660</td>
<td>510</td>
<td>250</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>3/4</td>
<td>1530</td>
<td>960</td>
<td>580</td>
<td>290</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>7/8</td>
<td>1750</td>
<td>960</td>
<td>580</td>
<td>290</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>1</td>
<td>2100</td>
<td>770</td>
<td>410</td>
<td>210</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1-3/4</td>
<td>1/2</td>
<td>830</td>
<td>510</td>
<td>340</td>
<td>170</td>
<td></td>
<td></td>
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</tr>
<tr>
<td>1</td>
<td>5/8</td>
<td>1160</td>
<td>680</td>
<td>340</td>
<td>170</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>3/4</td>
<td>1530</td>
<td>960</td>
<td>580</td>
<td>290</td>
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<td></td>
<td></td>
<td></td>
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<tr>
<td>1</td>
<td>7/8</td>
<td>1750</td>
<td>960</td>
<td>580</td>
<td>290</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>1</td>
<td>2100</td>
<td>770</td>
<td>410</td>
<td>210</td>
<td></td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>6.0 and greater</td>
<td>1/2</td>
<td>830</td>
<td>510</td>
<td>340</td>
<td>170</td>
<td></td>
<td></td>
<td></td>
<td></td>
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<tr>
<td></td>
<td>5/8</td>
<td>1160</td>
<td>680</td>
<td>340</td>
<td>170</td>
<td></td>
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<tr>
<td></td>
<td>3/4</td>
<td>1530</td>
<td>960</td>
<td>580</td>
<td>290</td>
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<tr>
<td></td>
<td>7/8</td>
<td>1750</td>
<td>960</td>
<td>580</td>
<td>290</td>
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<tr>
<td></td>
<td>1</td>
<td>2100</td>
<td>770</td>
<td>410</td>
<td>210</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Hold-Downs: Segmented v. Perforated

Segmented Shearwall

Perforated Shearwall
Perforated Shear Wall Method

Fewer hold downs required, shear capacity is reduced

Uniform uplift at base of wall required – magnitude = shear force – SDPWS 4.3.6.4.2.1
Perforated Shearwall Design

- Check Aspect Ratios: Assume blocked WSP Shearwall
- 10’/2’ = 5 > 3.5; Inadequate
- 10’/6’ = 1.67 < 3.5; OK

Use only full height sheathed sections to resist shear

\[ \nu_{\text{shearwall}} = 6,804 \text{ lb/}12' = 567 \text{ plf} \]
Perforated Shearwall Capacity

Wall has 12’/18’ = 67% full height sheathing, max. opening H = 6’-8”

Multiply capacity by 0.75 for opening 2H/3

Reduced capacity is 630 plf*0.75 = 473 plf < 567 plf, Inadequate

SDPWS Table 4.3.3.5

---

**Table 4.3.3.5 Shear Capacity Adjustment Factor, C_o**

<table>
<thead>
<tr>
<th>Wall Height, h</th>
<th>Maximum Opening Height</th>
<th>Percent Full-Height Sheathing</th>
<th>Effective Shear Capacity Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>8' Wall</td>
<td>h/3</td>
<td>10%</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>h/2</td>
<td>20%</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>2h/3</td>
<td>30%</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>5h/6</td>
<td>40%</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>h</td>
<td>50%</td>
<td>1.00</td>
</tr>
<tr>
<td>10' Wall</td>
<td>h/3</td>
<td>60%</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>h/2</td>
<td>70%</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, \( \sum L_i \), divided by the total length of the perforated shear wall.
Perforated Shearwall Capacity

\[ v_{\text{shearwall}} = 567 \text{ plf} \]

Try reducing nail spacing to 2” with 8d nails – **will require 3x framing**

Nominal Tabulated Capacity = 1790 plf  
Adjusted ASD Capacity = \((1790 \text{ plf})(0.92)(0.75)/2 = 618 \text{ plf}\)  
618 plf > 567 plf, OK  
**8d nails at 2” o.c. acceptable for perforated wall**

<table>
<thead>
<tr>
<th>PANEL GRADE</th>
<th>FASTENER TYPE &amp; SIZE</th>
<th>MINIMUM PANEL THICKNESS</th>
<th>MINIMUM FASTENER PENETRATION IN FRAMING</th>
<th>NAIL SPACING AT ALL PANEL EDGES</th>
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<td>1 3/8”</td>
<td>2 IN.</td>
<td>1280 (Seismic) 1790 (Wind)</td>
</tr>
</tbody>
</table>

SDPWS Table 4.3A
Perforated Shearwall Overturning

\[ v_{\text{shearwall}} = 567 \text{ plf} \]

Hold downs required at ends of perforated wall

\[ T = \frac{v_h}{C_o} \]

\[ T = 567 \text{ plf} \times \frac{10'}{0.75} = 7,560 \text{ lb} \]

Hold down capacity from segmented wall option = 7,045 lb, Inadequate – need to select higher capacity hold down
Perforated Shearwall Uplift

\[ v_{\text{shearwall}} = \frac{567 \text{ plf}}{0.75} = 756 \text{ plf}, \text{ use same magnitude for uniform uplift at full height segments} \]

One option is to use anchor bolts with large washers to resist uplift in bearing.

If net washer area = 8 in\(^2\), can resist \((565 \text{ psi})(8 \text{ in}^2) = 4,520 \text{ lb in uplift}\)

- Max. anchor bolt spacing = \(4,520 \text{ lb}/756 \text{ plf} = 5’-11” \text{ o.c.}\)
- Will also need to check shear loads on anchor bolts for controlling case.
Force Transfer Around Opening (FTAO)
FTAO Shearwalls Methodologies

- Shearwall design methodology which accounts for sheathed portions of wall above and below openings (perforated neglects)
- Openings accounted for by reinforcing edges using strapping or framing
- SDPWS 4.3.5.2 provides specific requirements
  - H/L ratio defined by wall pier
  - Min. wall pier width = 2’-0”
- Reduced number of hold downs (only at ends of total wall)

- There are 3 main methods of FTAO analysis; SDPWS does not require one particular method be used, only that design is “based on a rational analysis”
  - Drag Strut, Cantilever Beam, & Diekmann Design Options
Why Use Force Transfer Around Openings?

Full height wall piers do not meet max 3.5:1 Ratio

10 feet tall
2 feet wide
10/2 = 5 > 3.5
Not A Shear wall!
Why Use Force Transfer Around Openings?

Shorter Constrained piers do meet 3.5:1 max aspect ratio

5 feet tall
2 feet wide
5/2 = 2.5 < 3.5
Can be a Shear Wall!
References for FTAO Design

APA Authored SEAOC Paper

SEAOC Structural/Seismic Design Manual, Volume 2
Provides narrative and worked out example

Design of Wood Structures
Textbook by Breyer et al.
Double-Sided Shearwalls

High-strength wood shear walls can be double-sided with WSP sheathing on each side:

**SDPWS 4.3.3.3 Summing Shear Capacities:** For shear walls sheathed with the same construction and materials on opposite sides of the same wall, the combined nominal unit shear capacity shall be permitted to be taken as twice the nominal unit shear capacity for an equivalent shear wall sheathed on one side (4.3.5.3 has max capacities for double-sided perforated walls)
Double-Sheathed Shearwalls

There is also an option to have a single sided, double sheathed shearwall.

Testing and report by APA conclude that it is permissible to use the capacity of the wall the same as if there was one layer of WSP on each side of the wall provided that a number of criteria are met including:

- Framing members at panel joints are 3x or 2-2x
- Minimum nail spacing is 4"
- Others
Questions?

This concludes The American Institute of Architects Continuing Education Systems Course

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