Lateral Design of Mid-Rise Wood Structures

Presented by Ricky McLain, MS, PE, SE
Technical Director – WoodWorks
Chicago Area Workshops – April, 2017
FOLLOW
THE
LOAD
FOLLOWING THE LOAD...

A

B

A

B
Load Path Continuity
Multi-Story Considerations

- Wind Load Paths
- Multi-Story Stacked Shear Wall Effects
- Accumulation of Overturning Loads
- Shear Wall Deflection
- Diaphragm Modeling
- Discontinuous Shear Walls
Wind Load Distribution to Shearwalls
Wind Load Distribution to Shearwalls

Photo credit: Matt Todd & PB Architects
Multi-Story Wind Load Design

Design Principles are the Same

Remember to:
FOLLOW THE LOAD!
Multi-Story Wind Load Design

WIND SURFACE LOADS ON WALLS
Multi-Story Wind Load Design

WIND INTO DIAPHRAGMS AS UNIFORM LINEAR LOADS
Multi-Story Wind Load Design

DIAPHRAGMS SPAN BETWEEN SHEARWALLS

WIND INTO SHEARWALLS AS CONCENTRATED LOADS
Multi-Story Wind Load Design

DIAPHRAGM WIND FORCES DO NOT ACCUMULATE - THEY ARE ISOLATED AT EACH LEVEL

SHEARWALL WIND FORCES DO ACCUMULATE - UPPER LEVEL FORCES ADD TO LOWER LEVEL FORCES
Design Example: Five Over One Wood Frame

Free download at woodworks.org
Multi-Story Wind Design

Source: WoodWorks Five-Story Wood-Frame Structure over Podium Slab Design Example
Multi-Story Wind Design

Source: WoodWorks Five-Story Wood-Frame Structure over Podium Slab Design Example
Multi-Story Wind Design

Shearwall Layout

Source: WoodWorks Five-Story Wood-Frame Structure over Podium Slab Design Example
Multi-Story Wind Design

Shearwall Layout

Source: WoodWorks Five-Story Wood-Frame Structure over Podium Slab Design Example
Components of Shear Wall Design

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

Shear Resistance
Components of Shear Wall Design

Typ. Shear Wall Elevation

Wind Forces Per Story

$F_5 = 5.2k$

$F_4 = 3.8k$

$F_3 = 3.7k$

$F_2 = 3.6k$

$F_1 = 3.4k$
Components of Shear Wall Design

Accumulated Wind Forces

Typ. Shear Wall Elevation

29’-0”

10’-0” Typ.

F = 5.2k

F = 9k

F = 12.7k

F = 16.3k

F = 19.7k

Typ. Shear Wall Elevation
Accumulated Wind Forces
Multi-Story Shear Accumulation

- Shear forces are additive from floor to floor
- “Base shear” at the bottom or base of a structure is equal to the sum of all story shears
- Sole plate attachment of each wall must adequately transfer accumulative shear forces to the wall/foundation below

Typical wall to wall attachment:
- Fasteners (nails, screws, etc.), angles, sheathing

Typical wall to foundation attachment:
- Anchor bolts
Multi-Story Shear Accumulation

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

Source: WoodWorks Five-Story Wood-Frame Structure over Podium Slab Design Example
Multi-Story Shear Accumulation

**Figure 5. Typical Floor Framing at Wall**

Design a complete load path

Source: WoodWorks Five-Story Wood-Frame Structure over Podium Slab Design Example
Multi-Story Shear Accumulation

NDS Table 11L & 11N: Nail/Screw Capacity; SPF Sole plate, Shear Parallel to Grain (Along Shear Wall)

16d Common: $(120 \text{ lb}) \times (1.6) = 192 \text{ lb}$

12 Ga. Wood Screw”: $(125 \text{ lb}) \times (1.6) = 200 \text{ lb}$
Multi-Story Shear Accumulation
Multi-Story Shear Accumulation

NDS Table 11E: Anchor Bolt Capacity; SYP Sole plate, Shear Parallel to Grain (Along Shear Wall)

1/2”: $(680 \text{ lb}) \times (1.6) = 1088 \text{ lb}$

5/8”: $(970 \text{ lb}) \times (1.6) = 1552 \text{ lb}$

3/4”: $(1330 \text{ lb}) \times (1.6) = 2128 \text{ lb}$

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<th>Thickness</th>
<th>Bolt Diameter</th>
<th>G=0.67</th>
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<td>2060</td>
<td>1930</td>
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# Shearwall Nailing

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<tr>
<th>Level</th>
<th>Accum Shear</th>
<th>ASD Shear</th>
<th>ASD Unit Shear</th>
<th>Wall Sheathed</th>
<th>Fastener Edge Spacing</th>
<th>Allow. Shear</th>
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<tr>
<td>5th Floor</td>
<td>5.2 k</td>
<td>3.1 k</td>
<td>107 plf</td>
<td>1 side</td>
<td>6”</td>
<td>336 plf</td>
</tr>
<tr>
<td>4th Floor</td>
<td>9 k</td>
<td>5.4 k</td>
<td>186 plf</td>
<td>1 side</td>
<td>6”</td>
<td>336 plf</td>
</tr>
<tr>
<td>3rd Floor</td>
<td>12.7 k</td>
<td>7.6 k</td>
<td>262 plf</td>
<td>1 side</td>
<td>6”</td>
<td>336 plf</td>
</tr>
<tr>
<td>2nd Floor</td>
<td>16.3 k</td>
<td>9.8 k</td>
<td>338 plf</td>
<td>1 side</td>
<td>4”</td>
<td>490 plf</td>
</tr>
<tr>
<td>1st Floor</td>
<td>19.7 k</td>
<td>11.8 k</td>
<td>407 plf</td>
<td>1 side</td>
<td>4”</td>
<td>490 plf</td>
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</table>

Assumes 15/32” Rated Sheathing, 8d nails, SPF Framing
# Shearwall Shear Anchorage

<table>
<thead>
<tr>
<th>Level</th>
<th>Accum Shear</th>
<th>ASD Shear</th>
<th>ASD Unit Shear</th>
<th>Fastener</th>
<th>Fastener Spacing</th>
<th>Allow. Shear</th>
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<tr>
<td>5th Floor</td>
<td>5.2 k</td>
<td>3.1 k</td>
<td>107 plf</td>
<td>16d</td>
<td>2 @ 16”</td>
<td>288 plf</td>
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<tr>
<td>4th Floor</td>
<td>9 k</td>
<td>5.4 k</td>
<td>186 plf</td>
<td>16d</td>
<td>2 @ 16”</td>
<td>288 plf</td>
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<tr>
<td>3rd Floor</td>
<td>12.7 k</td>
<td>7.6 k</td>
<td>262 plf</td>
<td>16d</td>
<td>2 @ 16”</td>
<td>288 plf</td>
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<tr>
<td>2nd Floor</td>
<td>16.3 k</td>
<td>9.8 k</td>
<td>338 plf</td>
<td>16d</td>
<td>3 @ 16”</td>
<td>432 plf</td>
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<td>1st Floor</td>
<td>19.7 k</td>
<td>11.8 k</td>
<td>407 plf</td>
<td>½” A.B.</td>
<td>2’-8”</td>
<td>408 plf</td>
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Making Buildings Safe - Wind
Components of Shear Wall Design

- Holdown
- Boundary Posts
- Anchorage
- Overturning Resistance
- Compression
- Tension
Overturning Force Calculation

\[ T = C = F \times \frac{h}{L} \]

\[ T \text{ and } C \text{ are cumulative at lower stories} \]

\[ L \text{ is moment arm, not entire wall length} \]

Assume \( L = 29\text{ft}-1\text{ft} = 28\text{ft} \)

\begin{align*}
F &= 5.2k \\
F &= 9k \\
F &= 12.7k \\
F &= 16.3k \\
F &= 19.7k \\
1.9k & \quad 5.1k \\
9.6k & \quad 15.4k \\
22.5k &
\end{align*}
Sole Plate Crushing
Sole Plate Crushing

Compression forces perpendicular to grain can cause localized wood crushing. NDS values for $F_{c\perp}$ with metal plate bearing on wood result in a maximum wood crushing of 0.04”. Relationship is non-linear.

\[ Eq. 1.0 \]
\[ f_{c\perp} \leq F_{c\perp0.02 \text{ in}} \]
\[ \Delta = 0.02 \times \left( \frac{f_{c\perp}}{F_{c\perp0.02 \text{ in}}} \right) \]

\[ Eq. 2.0 \]
\[ F_{c\perp0.02 \text{ in}} < f_{c\perp} < F_{c\perp0.04 \text{ in}} \]
\[ \Delta = 0.04 - 0.02 \times \frac{f_{c\perp}}{0.27 \text{ in}} \]

\[ Eq. 3.0 \]
\[ f_{c\perp} > F_{c\perp0.04 \text{ in}} \]
\[ \Delta = 0.04 \times \left( \frac{f_{c\perp}}{F_{c\perp0.04 \text{ in}}} \right)^3 \]

\( \Delta = \) deformation, in
\( f_{c\perp} = \) induced stress, psi
\( F_{c\perp0.04 \text{ in}} = F_{c\perp} = \) reference design value at 0.04 in deformation, psi (\( F_{c\perp} \))
\( F_{c\perp0.02 \text{ in}} = \) reference design value at 0.02 in deformation, psi (0.73 \( F_{c\perp} \))
Sole Plate Crushing

NDS Commentary C4.2.6: when a joint is made of two wood members and both are loaded perpendicular to grain, the amount of deformation will be approximately 2.5 times that of a metal plate to wood joint.

<table>
<thead>
<tr>
<th>Bearing Condition</th>
<th>Deformation Adjustment Factor</th>
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<tr>
<td>1. Wood-to-wood (both perpendicular to grain)</td>
<td>2.5</td>
</tr>
<tr>
<td>2. Wood-to-wood (one parallel to grain and one perpendicular to grain)</td>
<td>1.75</td>
</tr>
<tr>
<td>3. Metal-to-wood (wood loaded perpendicular to grain)</td>
<td>1.0</td>
</tr>
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Source: WoodWorks  Five-Story Wood-Frame Structure over Podium Slab Design Example
# Compression Post Size & Sole Plate Crush

<table>
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<tr>
<th>Level</th>
<th>Compression</th>
<th>Required Bearing Area</th>
<th>Post Size</th>
<th>Story Sole Plate Crush</th>
<th>5x Sole Plate Crush</th>
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<tbody>
<tr>
<td>5&lt;sup&gt;th&lt;/sup&gt; Floor</td>
<td>1.9 k</td>
<td>4.4 in²</td>
<td>(2)-2x4</td>
<td>0.011”</td>
<td>0.057”</td>
</tr>
<tr>
<td>4&lt;sup&gt;th&lt;/sup&gt; Floor</td>
<td>5.1 k</td>
<td>11.9 in²</td>
<td>(2)-4x4</td>
<td>0.013”</td>
<td>0.067”</td>
</tr>
<tr>
<td>3&lt;sup&gt;rd&lt;/sup&gt; Floor</td>
<td>9.6 k</td>
<td>22.6 in²</td>
<td>(2)-4x4</td>
<td>0.034”</td>
<td>0.171”</td>
</tr>
<tr>
<td>2&lt;sup&gt;nd&lt;/sup&gt; Floor</td>
<td>15.4 k</td>
<td>36.3 in²</td>
<td>(3)-4x4</td>
<td>0.039”</td>
<td>0.195”</td>
</tr>
<tr>
<td>1&lt;sup&gt;st&lt;/sup&gt; Floor</td>
<td>22.5 k</td>
<td>39.8 in²</td>
<td>(4)-4x4</td>
<td>0.026”</td>
<td>0.13”</td>
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Floors 2-5 use S-P-F #2 Sole Plate, $F_{cperp} = 425$ psi
Floor 1 use SYP #2 Sole Plate, $F_{cperp} = 565$ psi
Story to Story Compression Force Transfer

Figure 13. Load Transfer from Compression Posts to Compression Posts

Notes for Figure 13:
Detail A (at platform framed) may have a single block with a drilled hole for the tie-down rod (see Figure 15).

Source: WoodWorks Five-Story Wood-Frame Structure over Podium Slab Design Example
Rim Joist Buckling & Crushing
Increasing Compression Post Size

**Figure 10. Example Plan Section at Boundary Members**

- TIE-DOWN ROD
- EDGE NAILING TO COMPRESSION POSTS
- CENTER OF BOUNDARY MEMBERS FOR COMPRESSION

Source: WoodWorks Five-Story Wood-Frame Structure over Podium Slab Design Example
Overturning Tension

Equal and Opposite Forces

Tension

Compression
Using Dead Load to Resist Overturning

Dead load from above (Wall, Floor, Roof) can be used to resist some or all overturning forces, depending on magnitude.

Load Combinations of ASCE 7-10: 06.D + 0.6W

Source: Strongtie
Shear Wall Holdown Options

- Strap Holdown Installation
  - 6+ kip story to story capacities

- Standard Holdown Installation
  - 13+ kip capacities

- Continuous Rod Tiedown Systems
  - 100+ kip capacities
  - 20+ kips/level
Components of Shear Wall Design

Overturning restraint at bearing plate at top of story

Tension accumulates in rod. Bearing plates see local overturning only. Tension zone boundary framing in compression!
Threaded Rod Tie Down w/Take Up Device

Source: Strongtie

Source: hardyframe.com
Threaded Rod Tie Down w/o Take Up Device
Overturning Resistance vs. Uplift Resistance

Key Differences:
- Rod location
- Load direction
- Framing Requirements
- Load path
- Shrinkage/compression location

Shear Wall Overturning Resistance

Uplift Resistance

Graphics Source: Strongtie
Tie Downs: Skipped Floors vs. All Floors

Skipped Floor System:
- Increased cost for posts and rods
- Increased drift
- Lack of vertical redundancy
- Inefficient load path
- Shrinkage not accommodated at each floor
- Lack of construction stability

All Floors Tied-Off System:
- Cost savings on posts and rods
- Reduced drift
- System redundancy
- Efficient load path
- Shrinkage accommodated at each floor
- Construction stability

Source: Strongtie
# Tie Down Rod Size & Elongation

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<tr>
<th>Level</th>
<th>Plate Hght</th>
<th>Tension</th>
<th>Rod Dia.</th>
<th>Steel</th>
<th>Rod Capacity</th>
<th>Rod Elong.</th>
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<td>5th Floor</td>
<td>10 ft</td>
<td>1.9 k</td>
<td>3/8”</td>
<td>A36</td>
<td>2.4 k</td>
<td>0.10”</td>
</tr>
<tr>
<td>4th Floor</td>
<td>10 ft</td>
<td>5.1 k</td>
<td>5/8”</td>
<td>A36</td>
<td>6.7 k</td>
<td>0.09”</td>
</tr>
<tr>
<td>3rd Floor</td>
<td>10 ft</td>
<td>9.6 k</td>
<td>5/8”</td>
<td>A193</td>
<td>14.4 k</td>
<td>0.18”</td>
</tr>
<tr>
<td>2nd Floor</td>
<td>10 ft</td>
<td>15.4 k</td>
<td>3/4”</td>
<td>A193</td>
<td>20.7 k</td>
<td>0.19”</td>
</tr>
<tr>
<td>1st Floor</td>
<td>10 ft</td>
<td>22.5 k</td>
<td>7/8”</td>
<td>A193</td>
<td>28.2 k</td>
<td>0.2”</td>
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Threaded Rod Tie Down - Couplers

Reducing Coupler

Source: Strongtie
Specifying Continuous Tie-down Systems

**Performance Route:** Engineer Specifies
- Holdown Locations
- Tension and Compression Loads
- Compression Framing Type
- Expected building shrinkage per story
- Allowable rod deformation limits (if required)

Vendor develops designs to meet the requirements and submits stamped designs and shop drawings.

**Detailed Design Route.** Engineer Specifies it all
- Rod size and material
- Bearing plates, couplers, shrinkage compensators
- Compression Posts
- Approved alternatives, etc.

WABO/SEAW White Paper
## Bearing Plate Size & Thickness

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<th>Level</th>
<th>Bearing Plate</th>
<th>Bearing Load</th>
<th>Allow. Bearing Capacity</th>
<th>Bearing Plate Crush</th>
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<td>W</td>
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<td>T</td>
<td>Hole Area</td>
<td>$A_{brng}$</td>
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<tr>
<td><strong>5th Floor</strong></td>
<td>3 in</td>
<td>3.5 in</td>
<td>3/8”</td>
<td>0.25 in$^2$</td>
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<tr>
<td><strong>4th Floor</strong></td>
<td>3 in</td>
<td>3.5 in</td>
<td>3/8”</td>
<td>0.518 in$^2$</td>
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<tr>
<td><strong>3rd Floor</strong></td>
<td>3 in</td>
<td>5.5 in</td>
<td>1/2”</td>
<td>0.518 in$^2$</td>
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<tr>
<td><strong>2nd Floor</strong></td>
<td>3 in</td>
<td>5.5 in</td>
<td>1/2”</td>
<td>0.69 in$^2$</td>
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<tr>
<td><strong>1st Floor</strong></td>
<td>3 in</td>
<td>8.5 in</td>
<td>7/8”</td>
<td>0.89 in$^2$</td>
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Shearwall Deformation – System Stretch

Total system stretch includes:

• Rod Elongation
• Take-up device displacement
• Bearing Plate Crushing
• Sole Plate Crushing

Source: WoodWorks Five-Story Wood-Frame Structure over Podium Slab Design Example
# Accumulative Movement

## With Shrinkage Compensating Devices

<table>
<thead>
<tr>
<th>Level</th>
<th>Rod Elong.</th>
<th>Shrinkage</th>
<th>Sole Plate Crush</th>
<th>Bearing Plate Crush</th>
<th>Take Up Deflect. Elong.</th>
<th>Total Displac.</th>
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</thead>
<tbody>
<tr>
<td>5&lt;sup&gt;th&lt;/sup&gt; Floor</td>
<td>0.1”</td>
<td>0.03”</td>
<td>0.057”</td>
<td>0.012”</td>
<td>0.03”</td>
<td>0.23”</td>
</tr>
<tr>
<td>4&lt;sup&gt;th&lt;/sup&gt; Floor</td>
<td>0.09”</td>
<td>0.03”</td>
<td>0.067”</td>
<td>0.022”</td>
<td>0.03”</td>
<td>0.24”</td>
</tr>
<tr>
<td>3&lt;sup&gt;rd&lt;/sup&gt; Floor</td>
<td>0.18”</td>
<td>0.03”</td>
<td>0.171”</td>
<td>0.018”</td>
<td>0.03”</td>
<td>0.43”</td>
</tr>
<tr>
<td>2&lt;sup&gt;nd&lt;/sup&gt; Floor</td>
<td>0.19”</td>
<td>0.03”</td>
<td>0.195”</td>
<td>0.03”</td>
<td>0.03”</td>
<td>0.48”</td>
</tr>
<tr>
<td>1&lt;sup&gt;st&lt;/sup&gt; Floor</td>
<td>0.2”</td>
<td>0.03”</td>
<td>0.13”</td>
<td>0.014”</td>
<td>0.03”</td>
<td>0.4”</td>
</tr>
</tbody>
</table>
Shearwall Tie Down Elongation

SDPWS Definition of $\Delta_a$: “Total vertical elongation of wall anchorage system (including fastener slip, device elongation, rod elongation, etc.) at the induced unit shear in the wall.”

**Figure 11. Effect of $\Delta_a$ on Drift**

$$\Delta = d_a \frac{h}{b}$$

Notes for Figure 11:
Where: $h$ = floor-to-floor height
$b$ = the out-to-out dimension of the shear wall

Source: WoodWorks Five-Story Wood-Frame Structure over Podium Slab Design Example
Shear Wall Deflection

**SDPWS 2008 Eq. 4.3-1**

\[
\delta_{sw} = \frac{8\nu h^3}{E Ab} + \frac{\nu h}{1000 G_a} + \frac{h \Delta_a}{b}
\]

**SDPWS 2008 Eq. C4.3.2-1**

\[
\delta_{sw} = \frac{8\nu h^3}{E Ab} + \frac{\nu h}{G_v t_v} + 0.75 h e_n + \frac{h \Delta_a}{b}
\]

**IBC 2000 to 2015 Eq. 23-2**

\[
\Delta = \frac{8\nu h^3}{E Ab} + \frac{\nu h}{G t} + 0.75 h e_n + \frac{d_a h}{b}
\]

Take care as symbols are not consistent between standards

Displacement ≠ Drift ≠ Deflection?
Shear Wall Deflection

**SDPWS 2008 Eq. 4.3-1**

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

**SDPWS 2008 Eq. C4.3.2-1**

\[ \delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{G_vt_v} + 0.75he_n + \frac{h}{b}\Delta_a \]

**IBC 2000 to 2015 Eq. 23-2**

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

Bending of boundary elements
Shear Wall Deflection

**SDPWS 2008 Eq 4.3-1**

\[
\delta_{sw} = \frac{8v h^3}{E A b} + \frac{v h}{1000 G_a} \cdot \frac{h \Delta_a}{b}
\]

**SDPWS 2008 Eq. C4.3.2-1**

\[
\delta_{sw} = \frac{8v h^3}{E A b} + \frac{v h}{G_v t_v} + 0.75 h e_n + \frac{h}{b} \Delta_a
\]

**IBC 2000 to 2015 Eq. 23-2**

\[
\Delta = \frac{8v h^3}{E A b} + \frac{v h}{G_t} + 0.75 h e_n + d_a \frac{h}{b}
\]

Shear Deformation of Sheathing Panels
&
Slip of nails @ panel to panel connections
Shear Wall Deflection

SDPWS 2008 Eq 4.3-1

\[
\delta_{sw} = \frac{8v h^3}{E A b} + \frac{v h}{1000 G_a} + \frac{h \Delta_a}{b}
\]

SDPWS 2008 Eq. C4.3.2-1

\[
\delta_{sw} = \frac{8v h^3}{E A b} + \frac{v h}{G_v t_v} + 0.75 h e_n + \frac{h \Delta_a}{b}
\]

IBC 2000 to 2015 Eq. 23-2

\[
\Delta = \frac{8v h^3}{E A b} + \frac{v h}{G t} + 0.75 h e_n + \frac{d_a h}{b}
\]

Rigid Body Rotation
### Shearwall Deflection

<table>
<thead>
<tr>
<th>Level</th>
<th>Unit Shear</th>
<th>End Post A</th>
<th>End Post E</th>
<th>Ga</th>
<th>Total Displace.</th>
<th>Deflection</th>
</tr>
</thead>
<tbody>
<tr>
<td>5th Floor</td>
<td>179 plf</td>
<td>10.5 in²</td>
<td>1400 ksi</td>
<td>10 k/in</td>
<td>0.23”</td>
<td>0.26”</td>
</tr>
<tr>
<td>4th Floor</td>
<td>310 plf</td>
<td>24.5 in²</td>
<td>1400 ksi</td>
<td>10 k/in</td>
<td>0.24”</td>
<td>0.4”</td>
</tr>
<tr>
<td>3rd Floor</td>
<td>438 plf</td>
<td>24.5 in²</td>
<td>1400 ksi</td>
<td>10 k/in</td>
<td>0.43”</td>
<td>0.59”</td>
</tr>
<tr>
<td>2nd Floor</td>
<td>562 plf</td>
<td>36.8 in²</td>
<td>1400 ksi</td>
<td>13 k/in</td>
<td>0.48”</td>
<td>0.6”</td>
</tr>
<tr>
<td>1st Floor</td>
<td>679 plf</td>
<td>49 in²</td>
<td>1400 ksi</td>
<td>13 k/in</td>
<td>0.4”</td>
<td>0.67”</td>
</tr>
</tbody>
</table>
Shear Wall Deflections In Practice

2D/3D Modeling of all the components of a shear wall system not practical in design.

Approaches used in Mid Rise include:
- Spreadsheets
- Equivalent cantilever columns models in commercial analysis software
- Equivalent FEM wall models in commercial analysis software
- Proprietary calculations by system manufacturer

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

SDPWS 2008 Eq 4.3-1
Shearwall Deflection Methods

Multiple methods for calculating accumulative shearwall deflection exist

Mechanics Based Approach:
- Uses single story deflection equation at each floor
- Includes rotational & crushing effects
- Uses SDPWS 3 part equation

Other methods exist which use alternate deflection equations, FEM
Shearwall Deflection Criteria for Wind

Unlike seismic, no code information exists on deflection/drift criteria of structures due to wind loads.

Serviceability check to minimize damage to cladding and nonstructural walls.

**ASCE 7-10:**

*C.2.2 Drift of Walls and Frames. Lateral deflection or drift of structures and deformation of horizontal diaphragms and bracing systems due to wind effects shall not impair the serviceability of the structure.*

What wind force should be used?
What drift criteria should be applied?
Shearwall Deflection Criteria for Wind

Wind Forces
Consensus is that ASD design level forces are too conservative for building/frame drift check due to wind
• Commentary to ASCE 7-10 Appendix C suggests that some recommend using 10 year return period wind forces:
  • ~ 70% of 700 return period wind (ultimate wind speed for risk category II buildings)
• Others (AISC Design Guide 3) recommend using 75% of 50 year return period forces

Drift Criteria
Can vary widely with brittleness of finishes but generally recommendations are in the range of H/240 to H/600
Diaphragm Modeling Methods

Possible Shear Wall Layouts

Not using all shared walls for Shear

Robust Diaphragm Aspect Ratio
Diaphragm Modeling Methods

Possible Shear Wall Layouts

Robust Diaphragm Aspect Ratio

But maybe not much wall available on exterior
Rigid or Flexible Diaphragm?

Light Frame Wood Diaphragms often default to Flexible Diaphragms

Code Basis: ASCE 7-10 26.2 Definitions (Wind)

Diaphragms constructed of wood structural panels are permitted to be idealized as flexible

Code Basis: ASCE 7-10 12.3.1.1 (Seismic)

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

[...]

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. thick.
2. Each line of vertical elements of the seismic force resisting system complies with the allowable story drift of Table 12.12-1.
Hypothetical Flexible Diaphragm Distribution

Changing wall construction does NOT impact load to wall line

Large portion of load on little wall

Area tributary to corridor wall line

Area tributary to exterior wall line

Hypothetical Flexible Diaphragm Distribution
Hypothetical Rigid Diaphragm Distribution

Changing wall construction impacts load to wall line

Longer, stiffer walls receive more load

Narrow, flexible walls receive less load

Diaphragm assumed to be rigid body.

Hypothetical Rigid Diaphragm Distribution
Can a Rigid Diaphragm be Justified?

ASCE 7-10 12.3.1.3 (Seismic)

[Diaphragms] are permitted to be idealized as flexible where the computed maximum in-plane deflection of the diaphragm under lateral load is more than two times the average story drift of adjoining vertical elements of the seismic force-resisting system of the associated story under equivalent tributary lateral load as shown in Fig. 12.3-1.

IBC 2012 Chapter 2 Definition (Wind & Seismic)

A diaphragm is rigid for the purpose of distribution of story shear and torsional moment when the lateral deformation of the diaphragm is less than or equal to two times the average story drift.
Some Advantages of Rigid Diaphragm

• More load (plf) to longer interior/corridor walls
• Less load (plf) to narrow walls where overturning restraint is tougher
• Can tune loads to walls and wall lines by changing stiffness of walls

Some Disadvantages of Rigid Diaphragm

• Considerations of torsional loading necessary
• More complicated calculations to distribute load to shear walls
• May underestimate “Real” loads to narrow exterior walls
• Justification of rigid assumption
Two More Diaphragm Approaches

Semi-Rigid Diaphragm Analysis

• Neither idealized flexible nor idealized rigid
• Explicit modeling of diaphragm deformations with shear wall deformations to distribute lateral loads
• Not easy.

Enveloping Method

• Idealized as BOTH flexible and rigid.
• Individual components designed for worst case from each approach
• Been around a while, officially recognized in the 2015 SDPWS
Possible Shear Wall Layouts

The Cantilever Diaphragm Option
Possible Shear Wall Layouts

Robust Aspect Ratio but only supported on 3 sides...
Cantilevered Diaphragms in SDPWS 2008

Open Front Structure

Cantilever Diaphragm

AWC SDPWS 2008 Figure 4A

AWC SDPWS 2008 Figure 4B
Cantilevered Diaphragms in SDPWS 2008

Open Front Structure

SDPWS 4.2.5.1.1

$L \leq 25 \text{ ft}$

$L/W \leq 1$, one story

$\leq 2/3$, multi-story

Exception: Where calculations show the diaphragm deflections can be tolerated, the length, $L$, can be increased to $L/W \leq 1.5$ for WSP sheathed diaphragms.
Cantilevered Diaphragms in SDPWS 2008

Cantilevered Diaphragm

SDPWS 4.2.5.2

Lc \leq 25 \text{ ft}

Lc/W \leq 2/3
Possible Shear Wall Layouts

Open Front Structure or Cantilevered Diaphragm?
Cantilevered Diaphragms in SDPWS 2015

Open Front Structure with a Cantilevered Diaphragm

AWC SDPWS 2015 Figure 4A
Open Front Structure & Cantilevered Diaphragms in SDPWS 2015

Cantilevered Diaphragm

SDPWS 4.2.5.2

$L'/W' \leq 1.5$

When Torsionally Irregular

$L'/W' \leq 1$, one story

$2/3$, multi-story

$L' \leq 35$ ft

Provided diaphragms modelled as rigid or semi-rigid and for seismic, the story drift at each edge of the structure within allowable story drift of ASCE 7. Story drifts include torsion and accidental torsional loads and deformations of the diaphragm.
Open Front Structure & Cantilevered Diaphragms in SDPWS 2015

Cantilevered Diaphragm

SDPWS 4.2.5.2

\[ L'/W' \leq 1.5 \]

When Torsionally Irregular

\[ L'/W' \leq 1, \text{ one story} \]

\[ \leq 2/3, \text{ multi-story} \]

\[ L' \leq 35 \text{ ft} \]

Exception: If \( L' \leq 6 \text{ ft} \), section doesn’t apply.
Tie Down Attachment to Concrete

Source: Strongtie
Tie Down Bolt with Washer

Source: Strongtie
Tie Down Bolt with Washer

Source: Strongtie
Tie Down Bolt with Washer - Reinforcing

Source: Strongtie
Tie Down Anchor Chair in Cast Slab

Source: Earthbound Anchors
Reinforcing Around Anchor Chairs

Source: Earthbound Anchors
Embedded Steel Plates – Weld on Rods

Fig. RD.5.2.9—Anchor reinforcement for tension.
Tie Down Anchors – Precast Through Bolt
Tie Down Anchors – Through Podium
Discontinuous Shear Walls

Photo credit: Matt Todd & PB Architects

Karuna I
Holst Architecture

Photo: Terry Malone
Lateral Load Path Continuity: Wall Elevation

Header distributes upper shear wall end post concentrated load to wall below.

Header also distributes upper shear wall shear to wall below.

Note: any member supporting a discontinuous wall must be designed for the over-strength factor under ASCE 7-10 Section 12.3.3.3, for SDC B-F.

Posts in lower wall transfer upper wall end post concentrated loads to foundation.

Wall plates act as drag struts to transfer shear loads from upper wall to lower wall.
Offset Shear Wall Overturning Resistance

Source: FEMA 55
Offset Shear Wall Overturning Resistance

Source: Strongtie
Tie Down to Steel Beam Attachment

Source: Strongtie
Tie Down to Steel Beam Attachment
ASCE 7-10 Section 12.3.3.3 and Commentary C12.3.3.3 provides guidance on seismic load requirements for various elements supporting discontinuous shear walls.
Recap

- Wind Load Paths
- Multi-Story Stacked Shear Wall Effects
- Accumulation of Overturning Loads
- Shear Wall Deflection
- Diaphragm Modeling
- Discontinuous Shear Walls
Questions?

This concludes The American Institute of Architects Continuing Education Systems Course

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