Lateral Design of Mid-Rise Wood Structures

Presented by Ricky McLain, MS, PE, SE
Technical Director – WoodWorks
Texas Workshops – December, 2016
FOLLOW THE LOAD
FOLLOWING THE LOAD...
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
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 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

Floor Plan

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

Shearwall Layout

Shearwall design we’ll look at
Multi-Story Wind Design

Shearwall Layout

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

- $F_5 = 5.2k$
- $F_4 = 3.8k$
- $F_3 = 3.7k$
- $F_2 = 3.6k$
- $F_1 = 3.4k$

Typ. Shear Wall Elevation
Wind Forces Per Story

29’-0”
10’-0” Typ.
Components of Shear Wall Design

Typ. Shear Wall Elevation
Accumulated Wind Forces

- $F = 5.2k$
- $F = 9k$
- $F = 12.7k$
- $F = 16.3k$
- $F = 19.7k$

29’-0”
10’-0” Typ.
Components of Shear Wall Design

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

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

$$T = C = F \times \frac{h}{L}$$

$T$ & $C$ are cumulative at lower stories

$L$ is moment arm, not entire wall length
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

\begin{align*}
\text{Eq. 1.0} & \quad 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)
\end{align*}

\begin{align*}
\text{Eq. 2.0} & \quad F_{c\perp0.02 \text{ in}} < f_{c\perp} < F_{c\perp0.04 \text{ in}} \\
& \quad 1- \left( \frac{f_{c\perp}}{F_{c\perp0.04 \text{ in}}} \right) \\
\Delta &= 0.04 - 0.02 \times \frac{f_{c\perp}}{0.27 \text{ in}}
\end{align*}

\begin{align*}
\text{Eq. 3.0} & \quad f_{c\perp} > F_{c\perp0.04 \text{ in}} \\
& \quad \Delta &= 0.04 \times \left( \frac{f_{c\perp}}{F_{c\perp0.04 \text{ in}}} \right)^3
\end{align*}

$\Delta$ = 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 ($0.73 F_{c\perp}$)
## Compression Post Size & Sole Plate Crush

<table>
<thead>
<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>
</tr>
</thead>
<tbody>
<tr>
<td>5th 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>4th 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>3rd 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>2nd 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>1st Floor</td>
<td>22.5 k</td>
<td>39.8 in²</td>
<td>(4)-4x4</td>
<td>0.026”</td>
<td>0.13”</td>
</tr>
</tbody>
</table>

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).
Rim Joist Buckling & Crushing
Increasing Compression Post Size

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

Compression

Equal and Opposite Forces

Tension
Using Dead Load to Resist Overturning

Load Combinations of ASCE 7-10:

\[ 0.6D + 0.6W \]

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

Continuous Rod Holdown System
Threaded Rod Tie Down w/Take Up Device

Source: Strongtie

Source: hardyframe.com
Threaded Rod Tie Down w/o Take Up Device
## Tie Down Rod Size & Elongation

<table>
<thead>
<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>
</tr>
</thead>
<tbody>
<tr>
<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>
</tr>
</tbody>
</table>
Bearing Plate Crushing
# Bearing Plate Size & Thickness

<table>
<thead>
<tr>
<th>Level</th>
<th>Bearing Plate</th>
<th>Bearing Load</th>
<th>Allow. Bearing Capacity</th>
<th>Bearing Plate Crush</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>W</td>
<td>L</td>
<td>T</td>
<td>Hole Area</td>
</tr>
<tr>
<td>5&lt;sup&gt;th&lt;/sup&gt; Floor</td>
<td>3 in</td>
<td>3.5 in</td>
<td>3/8”</td>
<td>0.25 in$^2$</td>
</tr>
<tr>
<td>4&lt;sup&gt;th&lt;/sup&gt; Floor</td>
<td>3 in</td>
<td>3.5 in</td>
<td>3/8”</td>
<td>0.518 in$^2$</td>
</tr>
<tr>
<td>3&lt;sup&gt;rd&lt;/sup&gt; Floor</td>
<td>3 in</td>
<td>5.5 in</td>
<td>1/2”</td>
<td>0.518 in$^2$</td>
</tr>
<tr>
<td>2&lt;sup&gt;nd&lt;/sup&gt; Floor</td>
<td>3 in</td>
<td>5.5 in</td>
<td>1/2”</td>
<td>0.69 in$^2$</td>
</tr>
<tr>
<td>1&lt;sup&gt;st&lt;/sup&gt; Floor</td>
<td>3 in</td>
<td>8.5 in</td>
<td>7/8”</td>
<td>0.89 in$^2$</td>
</tr>
</tbody>
</table>
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>
</tr>
</thead>
<tbody>
<tr>
<td>5th 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>4th 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>3rd 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>2nd 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>1st 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>
Shear Wall Deflection

**SDPWS 2008 Eq 4.3-1**

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

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

\[ \delta_{sw} = \frac{8v h^3}{EAb} + \frac{\nu h}{G_v I_v} + 0.75 h e_n + \frac{h}{b} \Delta_a \]

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

\[ \Delta = \frac{8v h^3}{EAb} + \frac{\nu h}{Gt} + 0.75 h e_n + d_a \frac{h}{b} \]

Bending of boundary elements
Shear Wall Deflection

SDPWS 2008 Eq 4.3-1
\[ \delta_{sw} = \frac{8\nu h^3}{EAb} + \frac{\nu h}{1000G_a} \cdot \frac{h\Delta_a}{b} \]

SDPWS 2008 Eq. C4.3.2-1
\[ \delta_{sw} = \frac{8\nu h^3}{EAb} + \frac{\nu h}{G_v t_v} + 0.75h e_n + \frac{h}{b} \Delta_a \]

IBC 2000 to 2015 Eq. 23-2
\[ \Delta = \frac{8\nu h^3}{EAb} + \frac{\nu h}{G_t} + 0.75h e_n + d_a \frac{h}{b} \]

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

**SDPWS 2008 Eq 4.3-1**

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

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

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

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

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

Rigid Body Rotation
<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 in$^2$</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 in$^2$</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 in$^2$</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 in$^2$</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 in$^2$</td>
<td>1400 ksi</td>
<td>13 k/in</td>
<td>0.4”</td>
<td>0.67”</td>
</tr>
</tbody>
</table>
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

Robust Diaphragm Aspect Ratio

Not using all shared walls for Shear
Diaphragm Modeling Methods

Possible Shear Wall Layouts

Robust Diaphragm Aspect Ratio

But maybe not much wall available on exterior
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.
Changing wall construction does **NOT** impact load to wall line.
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.
Rigid Diaphragm Analysis

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
Robust Aspect Ratio but only supported on 3 sides...

Possible Shear Wall Layouts
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, \text{ one story}
\leq 2/3, \text{ 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

- $L_c \leq 25$ ft
- $L_c/W \leq 2/3$
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
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.
Wind Load Distribution to Shearwalls
Tie Down Attachment to Concrete

Source: Strongtie
Tie Down Bolt with Washer

Source: Strongtie
Tie Down Bolt with Washer - Reinforcing

Source: Strongtie
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
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

Ricky McLain, MS, PE, SE
WoodWorks
Ricky.McLain@WoodWorks.org
(802)498-3310

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

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

© The Wood Products Council 2016