Structural Engineering of Mid-Rise Wood-Frame Construction

Anthony Harvey, PE
WoodWorks
Designing a wood building?
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- Allowable Heights/Areas
- Construction Types
- Structural Detailing
- Wood-Framed & Hybrid Systems
- Fire/Acoustic Assemblies
- Lateral System Design
- Alternate Means of Compliance
- Energy-Efficient Detailing
- Building Systems & Technologies

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Outline

Section 1: Mid-Rise Fire and Life Safety

Section 2: Structural Impacts of Mid-Rise

Section 3: Lateral Design of Wood-over-Podium
Wood Mid-Rise Construction

How many stories can be wood framed in the IBC?

Photo credit: Matt Todd & PB Architects
Marselle Condos, Seattle, WA

6 stories for Offices, 5 stories for Residential + Mezzanine + Multi-Story Podium

Photo credit: Matt Todd & PB Architects
Evolution of Mid-Rise

Credit: WoodWorks
Construction Types

New Options in 2021 IBC
Allowable mass timber building size for group B occupancy with NFPA 13 Sprinkler
Mid-Rise Construction Types

Type III
• Exterior walls non-combustible (may be FRTW)
• Interior elements any allowed by code

Type V
• All building elements are any allowed by code

Types III and V can be subdivided to A (protected) or B (unprotected)

Type IV (Heavy Timber)
• Exterior walls non-combustible (may be FRTW)
• Interior elements qualify as Heavy Timber
**Evolution of IBC Mixed-Use Podium**

<table>
<thead>
<tr>
<th></th>
<th></th>
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<td>509.2</td>
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<td>510.2</td>
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<td>Upper Occupancy</td>
<td>A, B, M, R or S</td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>Lower Occupancy</td>
<td>S-2 Parking</td>
<td>A, B, M, R or S-2 Parking</td>
<td>Any Except H</td>
<td></td>
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<td>Podium Height</td>
<td>1 Story</td>
<td></td>
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<tr>
<td>Height</td>
<td>No Restriction</td>
<td></td>
<td></td>
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</table>

IBC Provisions for Mixed-Use podium have been evolving.

*2015 IBC allows multiple podium stories above grade.*
Wood Within Podium Level(s)

FRTW is permitted in non-bearing, non-rated exterior walls in types I & II (IBC 603.1)

Thermal/building envelope benefits, as well as consistent exterior wall detailing

Source: Mahlum Architects
Wood Within Podium Level(s)

2021 IBC allows stairs below the podium to be framed with wood if building above podium is type III, IV or V.

Credit: WoodWorks
Let’s Talk Structure
Structure and Fire & Life Safety

Can’t Live in Separate Bubbles
Structure and Fire & Life Safety

In any project, but particularly wood-frame mid-rise construction, efficiency in structural framing layout, assembly selection and detailing must also account for "architectural" requirements such as:

- Fire-resistance ratings
- Acoustics
- Materials permitted (construction type)

In other words, you’re not just an engineer anymore
Exterior Wall – Bearing vs. Non Bearing

Non loading-bearing exterior walls may have lower fire resistance rating requirements than bearing walls in certain situations. IBC Chapter 2 defines load bearing walls as:

[BUILDING STANDARDS] WALL, LOAD-BEARING. Any wall meeting either of the following classifications:

1. Any metal or wood stud wall that supports more than 100 pounds per linear foot (1459 N/m) of vertical load in addition to its own weight.

[BUILDING STANDARDS] WALL, NONLOAD-BEARING. Any wall that is not a load-bearing wall.
# Exterior Wall – Bearing vs. Non Bearing

Why is this important? **Fire-Resistance Ratings and $**

<table>
<thead>
<tr>
<th>Fire Rating of Structural Elements</th>
<th>IIA</th>
<th>IIB</th>
<th>IIIA</th>
<th>IIIB</th>
<th>IV</th>
<th>VB</th>
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<tr>
<td>IBC Table 601</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>- Exterior bearing walls (hours)</td>
<td>1</td>
<td>0</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>0</td>
</tr>
<tr>
<td>- Interior bearing walls (hours)</td>
<td>1</td>
<td>0</td>
<td>1</td>
<td>0</td>
<td>1</td>
<td>0</td>
</tr>
<tr>
<td>- All other elements (hours)</td>
<td>1</td>
<td>0</td>
<td>1</td>
<td>0</td>
<td>HT</td>
<td>0</td>
</tr>
</tbody>
</table>

| IBC Table 602                     |     |     |      |      |     |    |
| - $X < 10$ feet                   | 1   | 1   | 1    | 1    | 1   | 1  |
| - $10$ ft $\leq X < 30$ feet      | 1   | 0   | 1    | 0    | 1   | 0  |
| - $X \geq 30$ feet                | 0   | 0   | 0    | 0    | HT  | 0  |

**Type III:**
- Exterior Bearing Wall = **2-hours**
- Exterior non-Bearing Wall = varies but often **0-hours**

Credit: WoodWorks
Exterior Walls – Bearing vs. Non-Bearing

If framing parallel to long exterior walls is possible, minimizes area of load bearing exterior walls
Type III Construction - IBC Section 602.3:
Fire-retardant-treated wood framing complying with Section 2303.2 shall be permitted within exterior wall assemblies of a 2-hour rating or less

What does this FRTW requirement include?
- Wall Framing (Studs & Plates) – Yes
- Headers – Yes
- Wall Sheathing – Yes
- Floor sheathing - ?
- Rim Joist- ?
- Floor Joists- ?
Exterior Walls – Intersecting Floors

Does the floor framing & sheathing that extends into the exterior wall need to FRT?
AWC’s DCA3 provides floor to wall intersection detailing options

Addresses both continuity provisions and requirements for FRT elements in exterior wall plane
Exterior Walls – Intersecting Floors

Two-hour fire-resistance-rated exterior wall assembly, rated for exposure from interior side (and from exterior side as required by IBC 705.5)

FRTW wall framing (studs, plates, blocking, etc.)

Untreated wood rim board, designed to support full wall load (with a minimum thickness of 1/8" if wall is required to be rated from exterior per IBC 705.5)

Untreated wood blocking with minimum thickness of 1/4" (Case A), 1/2" (Case B) or 3/4" (Case C). Blocking must be designed to support full wall load if wall is required to be rated from exterior per IBC 705.5.

FRTW wall framing (studs, plates, blocking, etc.)

FRTW sheathing (as required)

Exterior fire protection (as required to achieve fire-resistance rating per IBC 705.5)

Two-hour fire-resistance-rated exterior wall assembly, rated for exposure from interior side (and from exterior side as required by IBC 705.5)

Figure 1A: Example detail for Type III-A exterior wall–floor intersection with rim board and blocking
Type III Exterior Walls – FRT

Structural Impacts of using FRTW
FRT Wood Design Values

**NDS 2.3.4:** Adjusted design values, including adjusted connection design values, for lumber and structural glued laminated timber pressure-treated with fire retardant chemicals shall be obtained from the company providing the treatment and redrying service.
FRT Wood Design Values

FRT manufacturers provide reduction values in literature, ICC ESR’s, etc.

Example FRT manufacturer’s ESR reduction values:

<table>
<thead>
<tr>
<th>PROPERTY</th>
<th>PYRO-GUARD® WALL/FLOOR SERVICE TEMPERATURE TO 100°F/38°C</th>
<th>PYRO-GUARD® ROOF FRAMING, SERVICE TEMPERATURE TO 150°F/66°C,</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Douglas fir</td>
<td>Southern pine</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Extreme fiber stress in bending, $F_b$</td>
<td>0.97</td>
<td>0.91</td>
</tr>
<tr>
<td>Tension parallel to grain $F_t$</td>
<td>0.95</td>
<td>0.88</td>
</tr>
<tr>
<td>Compression parallel to grain $F_c$</td>
<td>1.00</td>
<td>0.94</td>
</tr>
<tr>
<td>Horizontal shear $F_v$</td>
<td>0.96</td>
<td>0.95</td>
</tr>
<tr>
<td>Modulus of elasticity, $E$</td>
<td>0.96</td>
<td>0.95</td>
</tr>
<tr>
<td>Compression perp. to grain $F_{oz}$</td>
<td>0.95</td>
<td>0.95</td>
</tr>
<tr>
<td>Fasteners/ connectors</td>
<td>0.90</td>
<td>0.90</td>
</tr>
</tbody>
</table>
FRT Wood Design Values

Shear wall capacity reduction typically handled by increasing sheathing thickness

When fire-retardant-treated plywood is used in a shear wall, the thickness must be one standard size thicker than that determined in the tabulated allowable shear values contained in Section 4.3 of ANSI/AWC Special Design Provisions for Wind and Seismic (SDPWS) or as shown in the tables referenced in Section 2306.3 of the IBC (2306.4 of the 2009 and 2006 IBC). Thickness to be used for FRT plywood compared to untreated plywood shear walls are shown below:

<table>
<thead>
<tr>
<th>FRT Plywood Thickness (inches)</th>
<th>Untreated Plywood Thickness (inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td>³/₈</td>
<td>⁵/₁₆</td>
</tr>
<tr>
<td>⁷/₁₆</td>
<td>³/₈</td>
</tr>
<tr>
<td>¹⁵/₃₂</td>
<td>⁷/₁₆</td>
</tr>
<tr>
<td>¹/₂</td>
<td>¹⁵/₃₂</td>
</tr>
</tbody>
</table>
Accommodating Wood Shrinkage
2304.3.3 Shrinkage. Wood walls and bearing partitions shall not support more than two floors and a roof unless an analysis satisfactory to the building official shows that **shrinkage of the wood framing will not have adverse effects on the structure or any plumbing, electrical or mechanical systems, or other equipment installed** therein due to excessive shrinkage or differential movements caused by shrinkage. The analysis shall also show that the roof drainage system and the foregoing systems or equipment will not be adversely affected or, as an alternative, such systems shall be designed to accommodate the differential shrinkage or movements.
Wood is orthotropic, meaning it behaves differently in its three orthogonal directions: Longitudinal (L), Radial (R), and Tangential (T)

- Longitudinal shrinkage is negligible
- Can assume avg. of radial & tangential or assume all tangential
Shrinkage occurs in cross-grain, but not longitudinal, wood dimensions

• Primarily in horizontal members
• Wall plates
• Floor/rim joists
• Engineering judgement required when determining what to include in shrinkage zone

• Should Sheathing, I-Joists, Trusses, other products manufactured with low MC be included?

**Shrinkage Calculations – Cross Grain Wood**

**FIGURE 5:** Shrinkage zone in platform-framed detail

- Zone of dimensional stability
- Zone of movement
- Zone of dimensional stability
Minimizing Shrinkage – Detailing

**SHRINKAGE ZONE:**
- 2x SILL PLATE
- 2x BLOCKING
- (2) 2x TOP PLATE
- 15 3/4” TOTAL

**SHRINKAGE ZONE:**
- 2x SILL PLATE
- 2x 12 FLOOR JOIST
- (2) 2x TOP PLATE
- 15 3/4” TOTAL

**SHRINKAGE ZONE:**
- 2x SILL PLATE
- (2) 2x TOP PLATE
- 4 1/2” TOTAL

Images: Schaefer
Minimizing Shrinkage – Detailing

Platform Detail:
15.75” Shrinkage Zone
19% MC Initial
12% EMC

\[ S = (0.0025)(15.75”)(12-19) = 0.28” \]

5-story building: 1.4” total

Semi-Balloon Detail:
4.5” Shrinkage Zone
19% MC Initial
12% EMC

\[ S = (0.0025)(4.5”)(12-19) = 0.08” \]

5-story building: 0.4” total
Minimizing Shrinkage - Detailing

Semi-balloon framing:
- Incorporates floor framing hanging from top plates
- Floor framing/rim joist doesn’t contribute to shrinkage

Non-standard stud lengths and increased hardware requirements should be considered
In parallel chord trusses, only chords contribute to shrinkage, vertical and diagonal webs don’t.
Differential Movement – Veneer Opening

WINDOW, SEE ARCH
SILL FLASHING THICKNESS AS REQUIRED FOR LOADING
PROVIDE GAP BETWEEN TOP OF VENEER AND UNDERSIDE OF FLASHING AS NEEDED TO ACCOMMODATE DIFFERENTIAL MOVEMENT
ATTACHMENT CLIPS FOR SILL FLASHING
WEATHER RESISTANT BARRIER & FLASHING, SEE ARCH
BRICK VENEER, SEE ARCH
EXTERIOR WALL SHEATHING

Image: Schaefer
Shrinkage Resource

Code provisions, detailing options, calculations and more for accommodating differential material movement in wood structures

Free resource at woodworks.org
Stacked Bearing Wall Design
Bearing Wall Studs: Stacking Loads

Options for lower level, stacked bearing wall studs:

• Specify SP or DF plates – up to 40% increase in allowable loads
  – Fc perp= 565 psi to 625psi
• Specify LSL or LVL plates – 75% increase in capacity
• Decrease stud spacing from 16" o.c. to 12” o.c. - 33% increase in capacity
• Double studs – 100% increase in capacity
• Increase the depth of the wall – 2x6 at upper, 2x8 at lower
• Add interior bearing walls at lower levels
Bearing Wall Design

When Floor/Roof Framing and Studs don’t Align
Design of Top Plates in Bending & Shear

Credit: WoodWorks
Bearing Wall Design

• General consensus is to assume two plates act independently. Half load goes to each (equal deflection)
• A 2-2x6 SPF top plate with studs at 16" o.c. has a truss reaction capacity of approximately 1,000 to 1,400 lb depending on load location
Wall Blocking Requirements

Credit: WoodWorks
Wall Blocking Requirements

- Slenderness ratio limits
- Weak axis stud buckling
- Shearwall panel edge blocking
- Fire blocking

NDS Appendix A.11.3:

When stud walls in light-frame construction are adequately sheathed on at least one side, the depth, rather than breadth of the stud, shall be permitted to be taken as the least dimension in calculating the le/d ratio. The sheathing shall be shown by experience to provide lateral support and shall be adequately fastened.

Credit: WoodWorks
Wall Blocking Requirements

NDS Commentary:
“Experience has shown that any code allowed thickness of gypsum board, hardwood plywood, or other interior finish adequately fastened directly to studs will provide adequate lateral support of the stud across its thickness irrespective of the type or thickness of exterior sheathing and/or finish used.”
Whatever you call it, it all comes down to one thing:

**Occupant Comfort**
Acoustical Design

• My interior, acoustically rated wall also needs to be a shearwall (think unit demising wall)
• Can I add wood structural panels to an acoustically tested wall?

Yes, but placement is very important!
Acoustical Design

**FIGURE 6**
Effect of Sheathing Placement on Acoustical Performance (Plan View)

STC 58  
STC 53  
STC 48  
STC 63
Acoustical Design

• For walls with resilient channels, put WSP on opposite side of wall
• For highly loaded shearwalls, can use double layer of sheathing on same side of wall
Acoustical Design

• Staggered stud wall condition:
• Blocking bridges finish on one side of wall to studs on opposite side, defeats purpose.
• Solution: use flat blocking in wall (wide face against WSP)

Credit: WoodWorks
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 into shearwalls.
Multi-Story Wind Design

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

Floor Plan

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

F₅ = 5.2k
F₄ = 3.8k
F₃ = 3.7k
F₂ = 3.6k
F₁ = 3.4k
Fₚ = 1.7k

10’-0” Typ.

29’-0”

Typ. Shear Wall Elevation
Wind Forces Per Story
Components of Shear Wall Design

Typ. Shear Wall Elevation

Accumulated Wind Forces

29'-0"

10'-0" Typ.

29'-0"

Typ. Shear Wall Elevation

Accumulated Wind Forces
Components of Shear Wall Design

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

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

- \( T \) and \( C \) are cumulative at lower stories.
- \( L \) is moment arm, not entire wall length.

Assume \( L = 29\text{ft}-1\text{ft} = 28\text{ft} \).
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
Shear Wall Deflection

SDPWS 2008 Eq 4.3-1
\[ \delta_{sw} = \frac{8v h^3}{E A b} + \frac{v h}{1000 G_b} + \frac{h \Delta_a}{b} \]

SDPWS 2008 Eq. C4.3.2-1
\[ \delta_{sw} = \frac{8v h^3}{E A b} + \frac{v h}{G_y t_v} + 0.75 h e_n + \frac{h}{b} \Delta_a \]

IBC 2000 to 2015 Eq. 23-2
\[ \Delta = \frac{8v h^3}{E A b} + \frac{v h}{G t} + 0.75 h e_n + \frac{d_a h}{b} \]

Rigid Body Rotation
Sole Plate 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
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

\[
\begin{align*}
\text{Eq. 1.0} & \quad f_{c\perp} \leq F_{c\perp 0.02} \text{ in} \\
\Delta &= 0.02 \times \left( \frac{f_{c\perp}}{F_{c\perp 0.02} \text{ in}} \right)
\end{align*}
\]

\[
\begin{align*}
\text{Eq. 2.0} & \quad F_{c\perp 0.02} \text{ in} < f_{c\perp} < F_{c\perp 0.04} \text{ in} \\
\Delta &= 0.04 - 0.02 \times \left( \frac{f_{c\perp}}{F_{c\perp 0.04} \text{ in}} \right) \\
&\quad - \left( \frac{f_{c\perp}}{F_{c\perp 0.04} \text{ in}} \right)^3
\end{align*}
\]

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

$\Delta =$ deformation, in

$f_{c\perp} =$ induced stress, psi

$F_{c\perp 0.04 \text{ in}} = F_{c\perp} =$ reference design value at 0.04 in deformation, psi ($F_{c\perp}$)

$F_{c\perp 0.02 \text{ in}} =$ 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>5&lt;sup&gt;th&lt;/sup&gt; Floor</td>
<td>1.9 k</td>
<td>4.4 in&lt;sup&gt;2&lt;/sup&gt;</td>
<td>(2)-2x4</td>
<td>0.011”</td>
<td>0.057”</td>
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<tr>
<td>4&lt;sup&gt;th&lt;/sup&gt; Floor</td>
<td>5.1 k</td>
<td>11.9 in&lt;sup&gt;2&lt;/sup&gt;</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 in&lt;sup&gt;2&lt;/sup&gt;</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 in&lt;sup&gt;2&lt;/sup&gt;</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 in&lt;sup&gt;2&lt;/sup&gt;</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
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
Overturning Tension

Tension

Compression

Equal and Opposite Forces
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
# 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>5&lt;sup&gt;th&lt;/sup&gt; 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>4&lt;sup&gt;th&lt;/sup&gt; 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>3&lt;sup&gt;rd&lt;/sup&gt; 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>2&lt;sup&gt;nd&lt;/sup&gt; 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>
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<tr>
<td>1&lt;sup&gt;st&lt;/sup&gt; 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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Bearing Plate Crushing
# Bearing Plate Size & Thickness

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<th>Level</th>
<th>W</th>
<th>L</th>
<th>T</th>
<th>Hole Area</th>
<th>A_{brng}</th>
<th>Bearing Load</th>
<th>Allow. Bearing Capacity</th>
<th>Bearing Plate Crush</th>
</tr>
</thead>
<tbody>
<tr>
<td>5th Floor</td>
<td>3 in</td>
<td>3.5 in</td>
<td>3/8”</td>
<td>0.25 in²</td>
<td>10.25 in²</td>
<td>1.9 k</td>
<td>4.4 k</td>
<td>0.012”</td>
</tr>
<tr>
<td>4th Floor</td>
<td>3 in</td>
<td>3.5 in</td>
<td>3/8”</td>
<td>0.518 in²</td>
<td>9.98 in²</td>
<td>3.2 k</td>
<td>4.2 k</td>
<td>0.022”</td>
</tr>
<tr>
<td>3rd Floor</td>
<td>3 in</td>
<td>5.5 in</td>
<td>1/2”</td>
<td>0.518 in²</td>
<td>15.98 in²</td>
<td>4.5 k</td>
<td>6.8 k</td>
<td>0.018”</td>
</tr>
<tr>
<td>2nd Floor</td>
<td>3 in</td>
<td>5.5 in</td>
<td>1/2”</td>
<td>0.69 in²</td>
<td>15.8 in²</td>
<td>5.8 k</td>
<td>6.7 k</td>
<td>0.03”</td>
</tr>
<tr>
<td>1st Floor</td>
<td>3 in</td>
<td>8.5 in</td>
<td>7/8”</td>
<td>0.89 in²</td>
<td>24.6 in²</td>
<td>7.0 k</td>
<td>10.4 k</td>
<td>0.014”</td>
</tr>
</tbody>
</table>
## 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>
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<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

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

SDPWS 2008 Eq 4.3-1

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

SDPWS 2008 Eq. C4.3.2-1

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

IBC 2000 to 2015 Eq. 23-2

Bending of boundary elements
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**

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

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

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

Shear Deformation of Sheathing Panels

&

Slip of nails @ panel to panel connections
Shear Wall Deflection

**SDPWS 2008 Eq 4.3-1**

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

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

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

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

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

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>5&lt;sup&gt;th&lt;/sup&gt; Floor</td>
<td>179 plf</td>
<td>10.5 in&lt;sup&gt;2&lt;/sup&gt;</td>
<td>1400 ksi</td>
<td>10 k/in</td>
<td>0.23”</td>
<td>0.26”</td>
</tr>
<tr>
<td>4&lt;sup&gt;th&lt;/sup&gt; Floor</td>
<td>310 plf</td>
<td>24.5 in&lt;sup&gt;2&lt;/sup&gt;</td>
<td>1400 ksi</td>
<td>10 k/in</td>
<td>0.24”</td>
<td>0.4”</td>
</tr>
<tr>
<td>3&lt;sup&gt;rd&lt;/sup&gt; Floor</td>
<td>438 plf</td>
<td>24.5 in&lt;sup&gt;2&lt;/sup&gt;</td>
<td>1400 ksi</td>
<td>10 k/in</td>
<td>0.43”</td>
<td>0.59”</td>
</tr>
<tr>
<td>2&lt;sup&gt;nd&lt;/sup&gt; Floor</td>
<td>562 plf</td>
<td>36.8 in&lt;sup&gt;2&lt;/sup&gt;</td>
<td>1400 ksi</td>
<td>13 k/in</td>
<td>0.48”</td>
<td>0.6”</td>
</tr>
<tr>
<td>1&lt;sup&gt;st&lt;/sup&gt; Floor</td>
<td>679 plf</td>
<td>49 in&lt;sup&gt;2&lt;/sup&gt;</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

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

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 Design
Calculating Diaphragm Forces

Diaphragm Fastener Schedule

Zone A 12'

Zone B 48'

Zone A 12'

24'

24'

72'
Load: 100 lbs/ft

$V_{\text{max}} = 3,600\text{ lbs}$

$V = 2,400\text{ lbs}$

Diaphragm Fastener Schedule

- **Zone A**: Nailing Pattern 1
- **Zone B**: Nailing Pattern 2
Diaphragm – Bending Member

- Tension edge
- Compression edge

DEFLECTED SHAPE OF DIAPHRAGM

REACTION PROVIDED BY SHEARWALL

R = 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

Figure 1  Simple Beam – Uniformly Distributed Load
Diaphragm Design - Deflection

Assume 7/16” OSB Sheathing with 24/16 Span Rating. Unblocked diaphragm with 8d common nails at 6” o.c. at all panel edges. Spruce Pine Fir trusses spaced 24” o.c.

\[ M_{\text{max}} = 132,829 \text{ lb-ft} \]

\[ T = 3,907 \text{ lb} \]

\[ C = 3,907 \text{ lb} \]

\[ w = 150.6 \text{ plf} \]

\[ V = 6,325 \text{ lb} \]

\[ v_{\text{diaphragm}} = 186 \text{ plf} \]

See SDPWS example C4.2.2-3 & APA L350 for design examples
Diaphragm Design – Deflection

From SDPWS commentary:
The total mid-span deflection of a blocked, uniformly nailed (e.g. same panel edge nailing) wood structural panel diaphragm can be calculated by summing the effects of four sources of deflection:

• Framing bending deflection
• Panel shear deflection
• Deflection from nail slip
• Deflection due to chord splice slip

SDPWS equation C4.2.2-1:

$$\delta_{dia} = \frac{5vL^3}{8EAW} + \frac{vL}{4G_vt_v} + 0.188Le_n + \frac{\sum(x\Delta_c)}{2W} \quad (C4.2.2-1)$$
Diaphragm Design – Deflection

\[
\delta_{\text{dia}} = \frac{5vL^3}{8EAW} + \frac{vL}{4G_v t_v} + 0.188Le_n + \frac{\sum(x\Delta_c)}{2W} \quad (C4.2.2-1)
\]

\( v = \text{max unit shear in diaphragm} \) - plf
\( L = \text{diaphragm length (perpendicular to force)} \) - ft
\( E = \text{modulus of elasticity of diaphragm chords} \) - psi
\( A = \text{area of chord (cross section)} \) - in\(^2\)
\( W = \text{Width of diaphragm in direction of applied force} \) - ft
\( G_v t_v = \text{shear stiffness, lb/in of panel depth} \)
\( x = \text{distance from chord splice to nearest support} \) - ft
\( \Delta_c = \text{diaphragm chord splice slip} \) - in
\( e_n = \text{nail slip} \) - in

Alternate Equation

\[
\delta_{\text{dia}} = \frac{5vL^3}{8EAW} + \frac{0.25vL}{1000G_a} + \frac{\sum(x\Delta_c)}{2W} \quad (C4.2.2-2)
\]
Diaphragm Design – Deflection

\[
\delta_{\text{dia}} = \frac{5vL^3}{8EAW} + \frac{vL}{4G_{t_v}} + 0.188L_{e_n} + \frac{\sum(x\Delta_c)}{2W} \quad (C4.2.2-1)
\]

\[\Delta_{\text{bending}} = \frac{5vL^3}{8EAW} = \frac{5 \times (186 \text{ plf}) \times (84'')^3}{8 \times (1,400,000 \text{ psi}) \times (2 \times 1.5" \times 5.5") \times (34')} = 0.088''\]
Δ_{shear} = \frac{vL}{4G_v t_v}

SDPWS Table C4.2.2A:

\( G_v t_v = 83,500 \text{ lb/in of depth for 7/16” OSB, 24/16 span rating} \)

\[ = \frac{(186 \text{ plf})*(84’)}{[4*83,500]} \]

\[ = 0.047” \]
Δ_{\text{panel nail slip}} = 0.188L_{en}

V_n = \text{load per nail} = 186 \text{ plf} / (12/6”) = 93 \text{ lbs per nail} \hspace{1cm} (\text{panel nails spaced 6” o.c.})

\begin{align*}
e_{\text{n}} &= (V_n / 616)^{3.018} \hspace{1cm} **\text{per footnote in SDPWS Table C4.2.2D, slip needs to be increased by 20% when OSB is not Structural I grade} \\
e_{\text{n}} &= 1.2*(V_n / 616)^{3.018} = 1.2*(93 / 616)^{3.018} = 0.004” \\
\Delta_{\text{panel nail slip}} &= 0.188*84’’*0.004” = 0.063”
\end{align*}
Diaphragm Design – Deflection

\[ \Delta_{\text{chord splice}} = \frac{\sum (x \Delta_c)}{2W} \]

\[ \Delta_c = \frac{2(T \text{ or } C)}{\gamma_n} \text{ (the 2 in the numerator is to account for splice slip on each side of the joint)} \]

\[ \gamma = \text{load/slip modulus for connection} = 180,000(D^{1.5}) \text{ for dowel-type fasteners (wood-to-wood)} \] NDS 11.3.6 \quad D = \text{diameter of dowel-type fastener (16d common)} \]

\[ \gamma = 180,000(0.162^{1.5}) = 11,737 \text{ lb/in/nail} \]

\[ \Delta_{c3} = \frac{2(3,827 \text{ lb})}{(11,737 \times 21)} = 0.031” \quad T_3 = 130,829 \text{ lb-ft} / 34 = 3,827 \text{ lb} \]

\[ \Delta_{c2} = \frac{2(3,189 \text{ lb})}{(11,737 \times 21)} = 0.026” \quad T_2 = 108,432 \text{ lb-ft} / 34 = 3,189 \text{ lb} \]

\[ \Delta_{c1} = \frac{2(1,914 \text{ lb})}{(11,737 \times 21)} = 0.016” \quad T_1 = 65,058 \text{ lb-ft} / 34 = 1,914 \text{ lb} \]
Diaphragm Design – Deflection

\[
\delta_{\text{dia}} = \frac{5vL^3}{8EAW} + \frac{vL}{4G_{t_v}t_v} + 0.188Le_n + \frac{\sum(x\Delta_c)}{2W}
\] (C4.2.2-1)

\[x\Delta_{\text{tension chord}} = 12'\times0.016'' + 24'\times0.026'' + 36'\times0.031'' + 36'\times0.031'' + 24'\times0.026'' + 12'\times0.016''\]

\[x\Delta_{\text{tension chord}} = 3.86 \text{ in-ft}\]
Diaphragm Design – Deflection

From SDPWS: Assuming butt joints in the compression chord are not tight and have a gap that exceeds the splice slip, the tension chord slip calculation is also applicable to the compression chord.

\[ x^*\Delta_{\text{tension chord}} = \frac{x\Delta_{\text{tension chord}}}{2 (34')} = 3.86 \text{ in-ft} \]

\[ \Delta_{\text{chord splice}} = \frac{3.86 + 3.86}{2} = 0.114'' \]

\[ \delta_{\text{dia}} = (0.088 + 0.047 + 0.063 + 0.114) 2.5 \quad (2.5 \text{ to account for unblocked diaphragm}) \]

\[ \delta_{\text{dia}} = 0.78'' \]
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
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..
Changing wall construction does **NOT** impact load to wall line

Hypothetical Flexible Diaphragm Distribution
Hypothetical Rigid Diaphragm Distribution

Changing wall construction impacts load to wall line

Longer, stiffer walls receive more load

Diaphragm assumed to be rigid body.

Narrow, flexible walls receive less load

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
Possible Shear Wall Layouts

Robust Aspect Ratio but only supported on 3 sides...
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.
Five-Story Wood-Frame Structure over Podium Slab

A Design Example of a Cantilever Wood Diaphragm

Developed for WoodWorks by
Douglas S. Thompson, PE, SE, SECB
STB Structural Engineers, Inc.
Lake Forest, CA

Developed for WoodWorks by
R. Torry Malone, PE, SE
Scott Brunsman, PhD, PE, SE

Photos: TDF, Crescent Terminus, architect: Lord Aeck Sargent, engineer: SCA Consulting Engineers, location: Atlanta, GA
NEET: Carbon 12, architect: Path Architecture, engineer: Munoz Structural Engineers, location: Portland, OR
Shear Wall to Podium Slab Interface

- Amplification of seismic forces is required for elements supporting discontinuous walls per ASCE 7-10 12.3.3.3
- Overstrength factor of 3 (may be reduced to 2.5 per footnote g of Table 12.2-1) is required
- Attachment to concrete slab must also conform to ACI 318 Appendix D
- Typically will be transitioning from ASD for wood design to LRFD for concrete design
- Hold down attachments to concrete options: embedded nuts or plates, sleeves through slab, welded studs & reinforcing
Tie Down Attachment to Concrete

Source: Strongtie
Tie Down Bolt with Washer

Source: Strongtie
Tie Down Anchor Chair in Cast Slab

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
Shear Wall to Podium Slab Interface

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.
Offset Shear Wall Overturning Resistance

Source: Strongtie
Offset Shear Wall Overturning Resistance

Source: FEMA 55
Tie Down to Steel Beam Attachment

Source: Strongtie
Tie Down to Steel Beam Attachment
QUESTIONS?

Anthony Harvey, PE
Regional Director
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This concludes The American Institute of Architects Continuing Education Systems Course