Mass Timber Rocking Wall Systems and Design

Reid Zimmerman, PE, SE
Technical Director | KPFF Portland

WoodWorks Seminar: State-of-Art Mass Timber Seismic Design and 10-Story NHERI Shake Table Test
May 19, 2023

Disclaimer: This presentation was developed by a third party and is not funded by WoodWorks or the Softwood Lumber Board.

Outline

• Introduction to mass timber rocking walls
• Current state of research and project applications
• Design procedures and examples
• What comes next?
Introduction to Mass Timber Rocking Walls

Left figure from Framework project
Introduction to Mass Timber Rocking Walls

Left figure from Framework project

Figures from Busch et al (2022)
Introduction to Mass Timber Rocking Walls

Current State of Research and Project Applications
Current State of Research

TallWood Planning
Project Testing

2013 2015 2017 2019 2021 2023

Figures from Ganey et al. (2017)

Current State of Research

Framework Testing

2013 2015 2017 2019 2021 2023

Photos from Framework testing at Oregon State University and Portland State University
Current State of Research

TallWood 2-Story Building Specimen Testing

2013 2015 2017 2019 2021 2023

Current State of Research

TallWood Biaxial Testing

2013 2015 2017 2019 2021 2023

Figures from Amer (2019)
Current State of Research

OSU 3-story Building Specimen Testing

TallWood 10-story Building Specimen Testing

Left figures courtesy of LEVER Architecture
Current State of Research

[Images of research projects]

2013 2015 2017 2019 2021 2023

Current Project Applications in the U.S.

Oregon State University
Peavy Hall, Corvallis, OR

Killingsworth
Portland, OR

Framework
Portland, OR

Center and right renderings courtesy of LEVER Architecture
Design Procedures and Examples
Prescriptive Design Procedure

- Either Equivalent Lateral Force or Modal Response Spectrum Analysis acceptable in state-of-the-practice software
- Majority of checks performed at DBE; several additional checks at MCE_R
- Based on proposed AWC SDPWS Appendix
Prescriptive Design Procedure

Figures from Busch et al (2022)
Prescriptive Design Procedure

From ASCE 7-16 Chapter 12

Table 12.12-1 Allowable Story Drift, $\Delta_y^{\text{a,d}}$

| Structure, other than masonry shear walls, structures, four stories or less above the base, or defined, in Section 12.2, with moment walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts | Allowable Story Drift $\Delta_y^{\text{a,d}}$ |
|---|---|---|
| Masonry shear wall systems | $0.025\Delta_y$ | $0.035\Delta_y$ | $0.015\Delta_y$ |
| All other structures | $0.030\Delta_y$ | $0.035\Delta_y$ | $0.015\Delta_y$ |

Note: In the story height before drift $\Delta_y$, the seismic force resisting system is comprised of moment frames in Seismic Design Categories D, E, F, the allowable story drift shall comply with the requirements of Section 12.12.1.2.

Figures from Busch et al (2022)
Prescriptive Design Procedure

\[ \theta_{design} = \frac{C_1 \cdot \delta_x - \delta_{se,Y}}{h_w} \]
Prescriptive Design Procedure

> 0 to ensure recentering

\[
\sum_{j=1}^{n} \left( F_{pti,j} + F_{grav,j} \right) \leq \frac{w_{j}}{2} \geq \sum_{j=1}^{n} F_{edu,wh,j} * w_{j}
\]

Prescriptive Design Procedure

4. Local Component Design & Checks

Check Limit States:
- Restoring Ratio
- Energy Dissipation Ratio
  - Limited Wall Toe Crushing at DBE
  - No PT yielding at DBE
  - No PT failure at MCE
  - No ED failure at MCE

Determine Wall Hardware (PT and ED)

All Limit States Satisfied?

Yes

No
Prescriptive Design Procedure

Set minimum ratio of areas to ensure sufficient damping

\[ \sum_{j=1}^{n} M_{ed,rm,j} \geq \frac{1}{4} \sum_{j=1}^{n} M_{n,rm,j} \]

Prescriptive Design Procedure

Figures from Busch et al (2022)
Prescriptive Design Procedure

Limit Wall Toe Crushing at DBE

No Post-Tensioning Yielding at DBE

Prescriptive Design Procedure

4. Local Component Design & Checks

Check Limit States:
- Restoring Ratio
- Energy Dissipation Ratio
- Limited Wall Toe Crushing at DBE
- No PT yielding at DBE
- No PT failure at MCE
- No ED failure at MCE

All Limit States Satisfied?

Yes

No
Prescriptive Design Procedure

No Post-Tensioning Failure at MCE$_R$

No Energy Dissipation Failure at MCE$_R$

\[ \theta_{max} = 1.5 \times \frac{C_1 \times \delta_x - \delta_{xy}}{h_w} \]

Prescriptive Design Procedure

4. Local Component Design & Checks

- Determine Wall Hardware (PT and ED)
- Check Limit States:
  - Restoring Ratio
  - Energy Dissipation Ratio
  - Limited Wall Toe Crushing at DBE
  - No PT yielding at DBE
  - No PT failure at MCE$_R$
  - No ED failure at MCE$_R$

All Limit States Satisfied?

Yes

No
Prescriptive Design Procedure

5. System Level Design & Checks

- Calculate Shear and Flexural Capacity: $\phi M_n$ & $\phi V_n$
- Determine Shear and Flexural Demand: $M_u$ & $V_u$

- $\phi M_n > M_u$?
  - No
  - Design Complete

- $\phi V_n > V_u$?
  - Yes
  - Design Complete

Figures from Busch et al (2022)

Prescriptive Design Procedure

Left and right figure from Busch et al (2022)
Prescriptive Design Procedure

From ACI 318-19

18.10.3.1 The design shear force \( V \) shall be calculated by:

\[ P_{0} d_{v} + \alpha L_{v} \leq V \leq P_{0} d_{v} + \alpha L_{v} \]

where \( P_{0} \), \( d_{v} \), and \( L_{v} \) are defined in 18.10.3.1.1, 18.10.3.1.2, and 18.10.3.1.3, respectively.

18.10.3.1.1 \( P_{0} \) is the shear force obtained from the lateral load analysis with factored load combinations.

18.10.3.1.2 \( d_{v} \) shall be in accordance with Table 18.10.3.1.2.

Table 18.10.3.1.2: Overstrength factor \( \alpha \) at critical section.

<table>
<thead>
<tr>
<th>Condition</th>
<th>( d_{v} )</th>
<th>( \alpha )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( d_{v} &lt; 1.5 )</td>
<td>( \alpha )</td>
<td>( 1.2 )</td>
</tr>
<tr>
<td>( d_{v} = 1.5 )</td>
<td>( \alpha )</td>
<td>( 1.0 )</td>
</tr>
<tr>
<td>( d_{v} &gt; 1.5 )</td>
<td>( \alpha )</td>
<td>( 0.6 )</td>
</tr>
</tbody>
</table>

For the load combinations producing the highest value of \( V \), unless a more ductile analysis is demonstrated as a feasible design.

18.10.3.1.3 For walls with \( h_{0}/d_{v} < 2.0\), \( \alpha \) shall be taken as 1.0. Otherwise, \( \alpha \) shall be calculated as:

\[ \alpha = 0.9 + \frac{h_{0}}{d_{v}} \quad \text{for} \quad \frac{h_{0}}{d_{v}} < 5.0 \]

\[ \alpha = 1.3 + \frac{h_{0}}{d_{v}} \quad \text{for} \quad \frac{h_{0}}{d_{v}} > 5.0 \]

From prescriptive design provisions

C.6.3 Amplification of Forces and Moments

Amplification of forces and moments, \( \alpha \), in Sections C.6.4 through C.6.9 shall be determined using a rational analysis that accounts for flexural overstrength of the rocking mechanism in the fundamental mode and reduced nonlinearity in modes other than the fundamental mode. The value of \( \alpha \) shall be calculated for each force or moment of interest and shall not be taken less than \( \omega_{B} \) and need not be taken greater than 3.
Prescriptive Design Procedure

\[ f_v = 1.5 \alpha \frac{V}{t_p \cdot l_w} \]
**Design Examples**

3-Story

6-Story

12-Story

---

**Design Examples**

6-Story example for hypothetical site in Seattle, WA

[Diagram showing building design examples with specifications and details.]

Center and right figures from Busch et al (2022)
Performance-Based Design

FRAMEWORK – INNOVATION IN RE-CENTERING MASS TIMBER WALL BUILDINGS

R.R. Zimmerman and E. McDonnell

ABSTRACT

Performance-based design is a concept that emphasizes the importance of incorporating a thorough understanding of performance criteria into the design process. By considering the potential for damage and failure under various loading conditions, performance-based design allows for the development of more resilient and sustainable buildings.

What Comes Next?

[Diagram of performance-based design framework]
### What Comes Next?

From ASCE 7-16

| Table 12.3-5 Design Coefficients and Factors for Seismic Force-Resisting Systems |
|---------------------------------|-----------------|-----------------|-----------------|-----------------|-----------------|-----------------|-----------------|-----------------|-----------------|
| Seismic Force-Resisting System  | ASCE 10.3 Frame Classifying | Height Limitation | Response Modification Coefficients | Characteristic Forces | Deflected Base Isolation | Seismic Design Category |
| 6.2.7 Post-Tensioned Mass Timber Rocking Walls | 14.1 and 14.5 | 6 | 2½ | 5 | NL | NL | 160 | 160 | 100 | 100 |

---

### Thank You

Reid Zimmerman, PE, SE
Technical Director | KPFF Portland
reid.zimmerman@kpff.com
References


Design Examples

6-Story example for hypothetical site in Seattle, WA

From prescriptive design provisions

\[ k_{ufp} = \frac{16 \times E_{ufp} \times b_{ufp}}{27\pi} \left( \frac{t_{ufp}}{D_{ufp}} \right)^3 \]

\[ V_{n,ufp} = \frac{f_{ufp} \times b_{ufp} \times t_{ufp}^2}{2 \times D_{ufp}} \]

\[ V_{pr,ufp} = \frac{R_t \times f_{ufp} \times b_{ufp} \times t_{ufp}^2}{2 \times D_{ufp}} \]

Center and right figures from Busch et al (2022)

- \( b_{ufp} = 180 \text{ mm (7 in)} \)
- \( t_{ufp} = 13 \text{ mm (1/2 in)} \)
- \( D_{ufp} = 110 \text{ mm (4 3/4 in)} \)
- \( k_t = 10.9 \text{ kN/mm (62.4 kips/in)} \)
- \( F_u = 55.2 \text{ kN (12.4 kips)} \)
- \( F_y = 38.5 \text{ kN (8.65 kips)} \)
Design Examples

- **Restoring ratio**: Eq. (3): No floor dead load acts on the wall due to the wall with Bounding Column Configuration, and the self-weight of the wall itself is conservatively neglected.
  
  Restoring ratio = \frac{2,890 \text{ kN} + 925 \text{ kN}}{2 \times 55.2 \text{ kN} \times 24} = 1.10 > 1.0 \text{ (SI units)}

  Restoring ratio = \frac{650 \text{ kips} + 925 \text{ kips}}{2 \times 12.4 \text{ kips} \times 24} = 1.10 > 1.0 \text{ (US units)}

- **Energy dissipation**: Eqs. (4)-(10).
  
  Energy-dissipation ratio = \frac{0.925 \text{ kN} \times 5.94 \text{ m}^2}{15,500 \text{ kN} \cdot \text{m}} = 0.35 > 0.25 \text{ (SI units)}

  Energy-dissipation ratio = \frac{208 \text{ kips} \times 208 \text{ kips}}{137,100 \text{ kip} \cdot \text{in.}} = 0.35 > 0.25 \text{ (US units)}

- **Limited wall toe crushing at DBE**: Eqs. (11)-(13) with the elastic range target, \( \Delta_{c,el} \), taken as 0.01 cm (4 in.).
  
  \[ \frac{0.0118}{-0.9 \times 0.015} = 0.87 \leq 1.0 \]

  \[ \frac{0.87}{1.0} = 0.87 \leq 1.0 \]

- **No PT yielding at DBE**: Eqs. (14)-(17).
  
  \[ \frac{2,890 \text{ kN} + 1,090 \text{ kN} \cdot \text{m}}{0.9 \times 724 \text{ MPa} \times 98 \text{ cm}^2} = 0.62 \leq 1.0 \text{ (SI units)} \]

  \[ \frac{2,890 \text{ kN} + 1,090 \text{ kN} \cdot \text{m}}{0.9 \times 105 \text{ ksi} \times 15.2 \text{ sq.in.}} = 0.62 \leq 1.0 \text{ (US units)} \]

- **No PT failure at MCE**: Eqs. (18) and (19).
  
  \[ \frac{0.0024}{0.05} = 0.05 \leq 1.0 \]

- **No energy-dissipation failure at MCE**: For UFPs, the critical design property is the distance from the start of the 180° bend and the nearest attachment point (e.g., weld or bolt) on the straight portion. Therefore, the length would need to exceed \( \Delta_{c,el} = 62 \text{ mm} (2.45 \text{ in.}) \) per Fig. 7(a) plus any elastic elongation of the tension side of the CLT wall panel at the location of the top story UFPs.

**Figures from Busch et al. (2022)**

---

Design Examples

**Demands:**

The shear demand tributary to the wall of interest in this example is \( V_u = 930 \text{ kN} \) (210 kips) based on the scaled MRSA from the ETABS analysis (which provides a shear demand for each wall). Alternatively, the shear demand for the wall of interest can be determined from the base shear and diaphragm analysis deemed appropriate for the building. As mentioned previously, further modifications were applied to the shear based on Eq. (32) in Step 5 to provide the final amplified shear demand to account for higher mode effects.

\[
V_{u,ax} = 1.5 \times 1.5 \times 1.25 \times 930 \text{ kN} = 2,620 \text{ kN} \text{ (SI units)}
\]

\[
V_{u,ax} = 1.5 \times 1.5 \times 1.25 \times 930 \text{ kips} = 590 \text{ kips} \text{ (US units)}
\]

The moment demand of the wall was taken directly from the ETABS model under the scaled MRSA in the x-direction.

\[
M_u = 13,400 \text{ kN} \cdot \text{m}(118,300 \text{ kip} \cdot \text{in.})
\]

**Capacities:**

The shear capacity mentioned in Eq. (34) is where the reference in-plane shear capacity, \( V_{u,design} \), is based on manufacturer’s literature:

\[
\phi V_{u,design} = 0.75 \times 2.88 \times 1.0 \times 5.94 \text{ m} \times 457 \text{ kN/m} = 5,870 \text{ kN} \text{ (SI units)}
\]

\[
\phi V_{u,design} = 0.75 \times 2.88 \times 1.0 \times 19.5 \text{ ft} \times 31.3 \text{ kip/ft} = 1,320 \text{ kips} \text{ (US units)}
\]

The moment capacity mentioned in Eq. (22) in Step 5 will be as follows:

\[
\phi M_{u,design} = 0.9 \times 137,100 \text{ kip} \cdot \text{in.} = 123,400 \text{ kip} \cdot \text{in.} \text{ (US units)}
\]

**Check:**

\[
\begin{align*}
V_{u,ax} &= 2,620 \text{ kN} \leq 5,870 \text{ kN} = \phi V_{u,design} \text{ (SI units)} \\
V_{u,ax} &= 590 \text{ kips} \leq 1,320 \text{ kips} = \phi V_{u,design} \text{ (US units)} \\
M_u &= 13,400 \text{ kN} \cdot \text{m} \leq 13,000 \text{ kN} \cdot \text{m} = \phi M_{u,design} \text{ (SI units)} \\
M_u &= 118,300 \text{ kip} \cdot \text{in.} \leq 123,400 \text{ kip} \cdot \text{in.} = \phi M_{u,design} \text{ (US units)}
\end{align*}
\]

**Figures from Busch et al. (2022)**

51