Prescriptive Seismic Design Procedure for Post-Tensioned Mass Timber Rocking Walls

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Abstract: In this study, a prescriptive seismic design procedure for post-tensioned mass timber rocking wall lateral force–resisting systems is proposed. Unlike performance-based design approaches that employ nonlinear analysis, this procedure utilizes techniques and analysis procedures that are routinely applied in design industry practice as well as adhering to traditional approaches contained in current US standards. The design procedure targets providing a basis for prescriptive design of mass timber rocking wall lateral force–resisting systems and their future adoption into model codes. For illustration, the design approach is applied to an example building, with the building's performance validated through a nonlinear numerical model simulation. **DOI: 10.1061/(ASCE)ST.1943-541X.0003240.** *© 2021 American Society of Civil Engineers.*

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Background

With the growing popularity of mass timber construction and proposed building code changes in the US, the number of mass timber building projects is expected to increase. Although most of these buildings can be erected using a concrete or steel lateral force–resisting system (LFRS), researchers have also been pursuing lateral system solutions using mass timber materials, such as panelized cross-laminated timber (CLT) shear walls (Amini et al. 2018; van de Lindt et al. 2020) and post-tensioned rocking wall systems (Iqbal et al. 2015). Although mass timber LFRS are gaining traction in research and development through testing and in constructed projects through performance-based design applications, there are no mass timber LFRS that are fully recognized in the US building code and its reference design standards as of the end of 2020. Prescriptive design procedures will make such systems more accessible to the wider design community in the US.

Among many potential mass timber LFRS, post-tensioned rocking walls are viewed by the research and design community to be a low-damage lateral system that can be competitive for mass timber building projects in regions with high seismicity. This system is an adaptation of the precast concrete rocking wall system, heavily researched and tested in the 1990s and codified via American Concrete Institute (ACI) standards ACI ITG 5.1-07 and 5.2-09 (ACI 2007, 2009). The timber version of this system was originally conceptualized in New Zealand using laminated veneer lumber (LVL) walls with several real project applications (Palermo et al. 2012). Post-Tensioned rocking walls made of CLT panels were tested in the US both at the component (Ganey et al. 2017; Akbas et al. 2017) and system levels (Pei et al. 2019; Blomgren et al. 2019). The system was also adopted in real projects in the US that were either permitted (Zimmerman and McDonnell 2018) or built (Sarti et al. 2017b; Baas et al. 2019).

All of these existing designs and analyses were conducted using fairly advanced tools and procedures (such as performance-based design and nonlinear response history analysis), which are more sophisticated than those currently used in the average design office. These pioneer research activities and actual projects generated a wealth of data and experience on the behavior and design of post-tensioned mass timber rocking wall LFRS. This has enabled the development of a prescriptive design approach to mirror that of post-tensioned precast concrete rocking walls, which is the focus of this study.

In the following sections, seismic design limit states for post-tensioned mass timber rocking walls are listed. The procedure and design calculations to conduct these limit-state checks are described. An example design of a mass timber rocking wall LFRS in a 6-story office building is also presented. The procedures presented here are part of an effort seeking to introduce mass timber rocking wall LFRS into the *Special Design Provisions for Wind and Seismic* standard (AWC 2015a), which is the building code reference for design of wood LFRS in the US via the International Building Code (ICC 2018).

Design recommendations related to mass timber rocking walls subjected to wind loads are not presented in this study. Performance expectations and analysis and design procedures for wind loading, although necessary to satisfy code requirements, are distinct from those for seismic loading (e.g., postyield behavior is generally not acceptable for wind loading) and are the subject of separate research. However, engineers should be aware that wind loading,

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particularly service-level wind drifts and accelerations, may govern the design of taller mass timber rocking wall buildings.

Mass Timber Rocking Wall System

The design procedure developed in this study is suitable for the post-tensioned mass timber rocking wall LFRS shown in Fig. 1. The system consists of either uncoupled or coupled mass timber panels that can be attached to floor diaphragms of a multistory building in a balloon framing configuration. The rocking wall is post-tensioned with high-strength steel strands or threaded bars [post-tensioning (PT) elements] from the top of the wall, typically placed near the center of the wall length. The PT elements can be distributed away from the center, but it is recommended that, except for unusual cases, the combined PT force align with the center of the wall for symmetric response. The system also must include energy-dissipation elements that will absorb the earthquake input

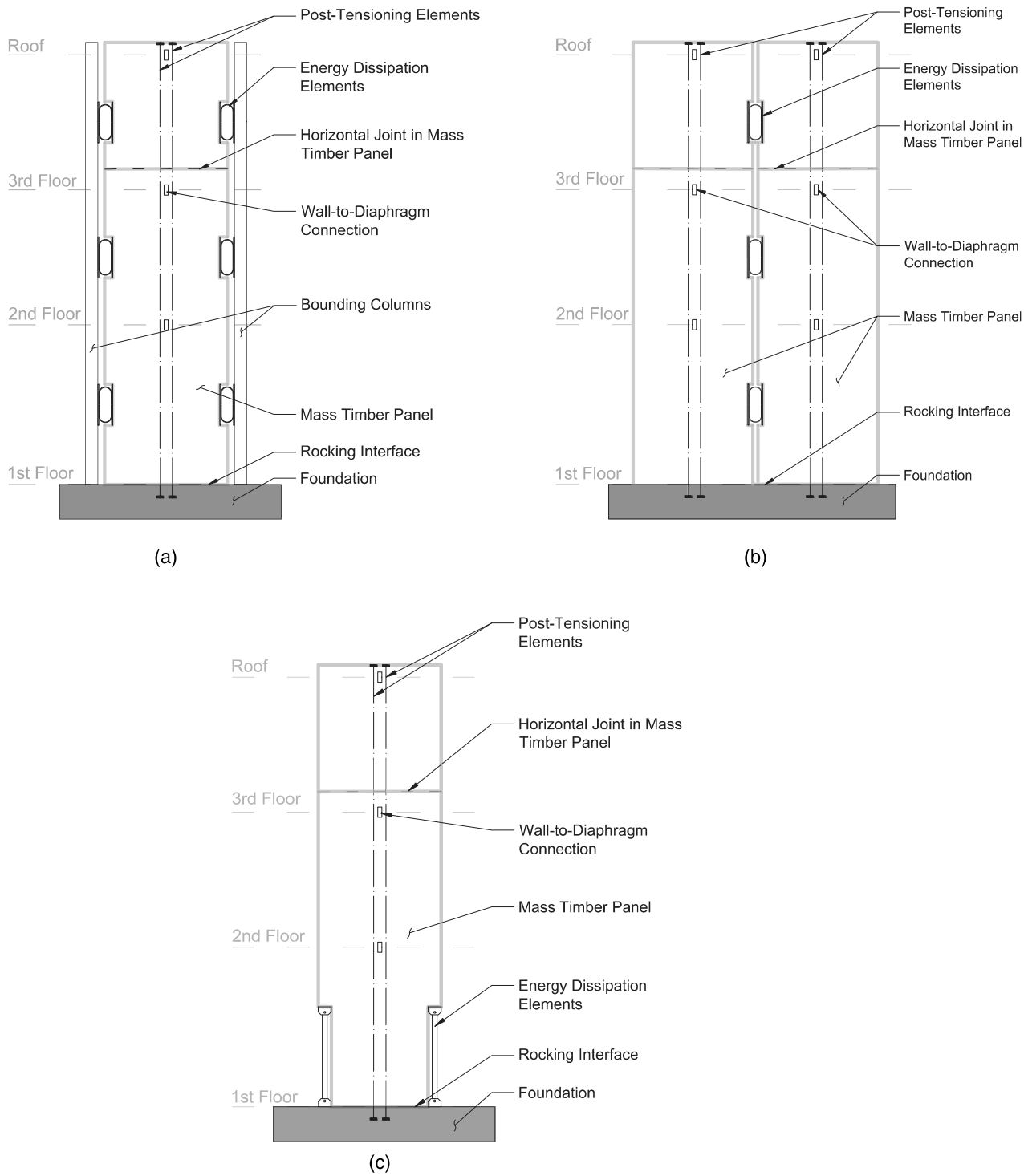

Fig. 1. (a) Conceptual configuration of a post-tensioned mass timber rocking wall with bounding columns; (b) conceptual configuration of coupled post-tensioned mass timber rocking walls; and (c) conceptual configuration of an uncoupled post-tensioned mass timber rocking wall.

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energy during the rocking motion. These can be yielding or frictional devices ranging from U-shaped flexural plates (UFPs) or buckling-restrained braces (BRBs) to specialized friction dampers.

Depending on the system configuration, they can be anchored to the foundation (Uncoupled Wall Configuration), attached at the edge of the mass timber panel to an adjacent panel (Coupled Wall Configuration), or fixed to a bounding column (wall with bounding column configuration). Connection of the energy-dissipation elements to wood is typically made through dowel-type fasteners in shear or withdrawal (e.g., self-tapping screws), epoxied steel rods or plates, or wood bearing. The connections need to be designed for the overstrength capacity of the dissipation elements. The rocking wall must also have a connection at the base to transfer shear loads to the building foundations as well as a connection between wall and diaphragm.

Seismic Design Requirements

The seismic design requirements for post-tensioned mass timber rocking walls are taken to meet, at a minimum, those for prescriptive LFRS already recognized in ASCE 7 (ASCE 2016) as well as the requirements contained in the *National Design Specification (NDS) for Wood Construction* (AWC 2015b). In general, this includes both strength and deformation limits for the global system under the design-basis earthquake (DBE) per ASCE 7 and strength limits for individual components per the NDS. Global deformation limits in accordance with ASCE 7 primarily include allowable drift ratios for each story. Global strength limits relate to the minimum base shear and corresponding floor forces, story shears, and overturning moments as prescribed by ASCE 7. When these global strength and deformation demands are distributed to the individual components of the LFRS, they must then satisfy local limits in accordance with the NDS (e.g., flexure and shear in a mass timber panel). In addition to meeting the requirements of ASCE 7 and the NDS, the design procedure proposed here includes several other criteria, which are described in the following subsections.

Achieving Recentering after the DBE

Although not required by the International Building Code, recentering is targeted for post-tensioned mass timber rocking walls as a specific, desired aspect of this lateral system. It is expected that the ability to recenter will be a deciding factor in the adoption of post-tensioned mass timber rocking walls over other prescriptive lateral force–resisting systems. The design procedure proposed here intends to produce a design with negligible residual displacement after a DBE event.

Establishing Minimum Required Energy Dissipation

The strength and deformation limits of ASCE 7 and the NDS can technically be met by an elastic system. A mass timber rocking wall with only post-tensioning and no energy-dissipation elements is an example of such a nonlinear elastic system. However, because this design procedure is partially modeled off that contained in ACI ITG 5.2-09, which specifies a minimum energy dissipation; that minimum energy dissipation, corresponding to the energy-dissipation elements providing at least 25% of the total moment capacity, is adopted here as well.

Allowing Crushing of the Wall Toe at DBE

Plastic deformation of materials and elements for earthquake demands is acceptable in prescriptively designed LFRS composed of steel, concrete, masonry, and light-frame wood. This acceptance of plastic deformation is extended to post-tensioned mass timber rocking walls by permitting inelastic action in the energy-dissipation elements and also, notably, in the mass timber panel at the wall toe. The current edition of the NDS does not address plastic deformation in mass timber panels for current panel configurations. Therefore, quantitative acceptance limits for crushing of the wall toe are introduced specifically for mass timber panels used in post-tensioned rocking walls as discussed further in the section entitled "Seismic Design Approach."

Requiring Several Local Checks at MCE_R

Design per ASCE 7 is almost exclusively performed at the DBE level whereas performance at the Risk-Targeted Maximum Considered Earthquake MCE_R) is assumed to be satisfied indirectly. The requirements for post-tensioned mass timber rocking walls follow this precedent of design at the DBE but additionally include several explicit checks that must be performed at the MCE_R.

Seismic Design Approach

A seismic design approach for post-tensioned mass timber rocking walls should be easy to use by engineers in practice. In the context of the International Building Code, this essentially means that it must be a force-based procedure to align with the linear analysis methods in ASCE 7 (i.e., equivalent lateral force and modal response spectrum analysis). Although displacement-based procedures and nonlinear analysis are suitable to the design of post-tensioned mass timber rocking walls (i.e., performance-based design approaches), the lack of familiarity by practicing engineers in the US and/or lack of simplicity make them less preferable.

Similar to seismic design procedures for other lateral force–resisting systems, the process starts by developing a trial design based on experience and given architectural constraints. A set of prescriptive steps are then followed to ensure all force and deformation limit states are satisfied. The proposed procedure includes five major steps illustrated in the flowchart shown in Fig. 2. The steps shown in the flowchart are explained in detail in the following subsections. It should be noted that a complete seismic design of a rocking mass timber wall LFRS requires other design checks (e.g., diaphragms, chords, collectors, diaphragm-to-wall connections, wall-to-foundation connections, and others). However, these checks can be performed similar to other lateral force–resisting systems and are therefore not included in this paper.

Step 1: Preliminary Design

Based on the given architectural design, the location and preliminary dimensions (i.e., thicknesses, lengths, and locations) of the post-tensioned mass timber rocking walls are determined. The engineer will complete a linear analysis of the building with the given floor plan using mass timber material properties and the preliminary rocking wall dimensions. This step can be completed using any commercial finite-element software; ETABS version 18 (CSI 2013) was used in this study. At this stage, the rocking walls should be modeled as fixed to the foundation (essentially acting as a cantilever). Although rocking is expected at the interface between the walls and the foundation, this is incorporated indirectly in Step 3 because modeling of a gap element to permit rocking is not part of a linear analysis.

It is typically acceptable to neglect the contribution to stiffness of the energy-dissipation elements in either the Uncoupled Wall Configuration or Wall with Bounding Column Configuration

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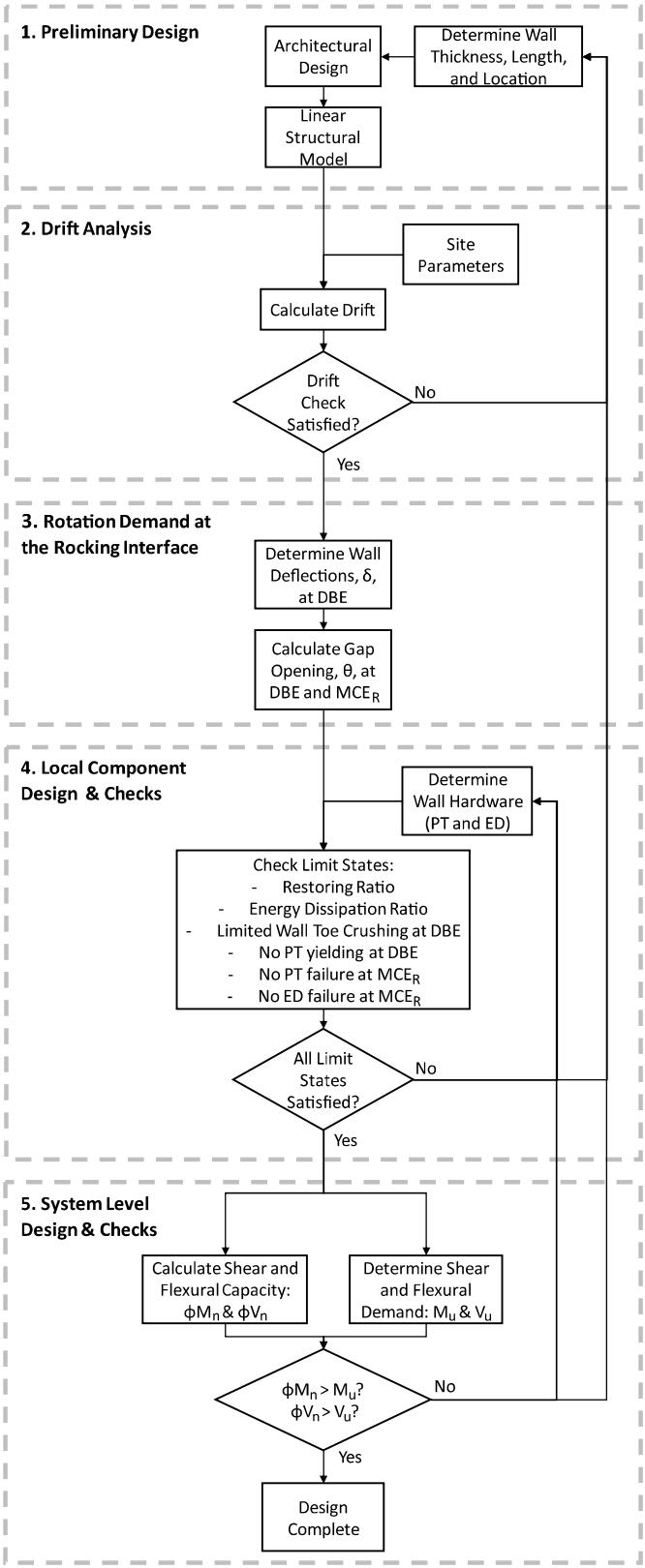

Fig. 2. Flowchart illustrating the design procedure.

(Fig. 1) because the additional stiffness contribution from the energy-dissipation elements is often small compared with that of the mass timber panel. In some cases (e.g., very stiff energy-dissipation elements and a high level of coupling between walls), the energy-dissipation elements should be included in the linear analysis model for the Coupled Wall Configuration. It is almost always acceptable to neglect the stiffness contribution from the PT elements in the linear analysis model.

Because seismic design parameters have not yet been established for post-tensioned mass timber rocking walls in ASCE 7, the response modification coefficient (R) is taken per ASCE 7-16 Table 12.2-1 for building frame systems' special reinforced concrete shear walls. The appropriateness of using $R = 6$ for special reinforced concrete shear walls in the design of post-tensioned mass timber rocking walls can be justified by observing that post-tensioned precast rocking walls designed in accordance with ACI ITG 5.1 and 5.2 are permitted to use that same R per ACI 318 (ACI 2019). Because the design procedure proposed here closely follows ACI ITG 5.2, and because the system response of a rocking precast concrete panel versus a rocking mass timber panel are similar, post-tensioned mass timber rocking walls can be seen as a system that emulates post-tensioned precast concrete rocking walls (Zimmerman and McDonnell 2017).

Furthermore, a $R = 6$ has been adopted in mass timber rocking wall preliminary designs for past studies and testing projects (Pei et al. 2019; Zimmerman and McDonnell 2018) and has proven to be reasonable for these albeit limited, cases. Furthermore, although a complete FEMA P-695 (FEMA 2009) study for mass timber rocking walls is ultimately required to allow the system to be designed prescriptively via ASCE 7 (ASCE 2016), recent research suggests that $R = 7$ for a post-tensioned mass timber wall system is appropriate (Sarti et al. 2017a). In this paper, $R = 6$ is adopted.

Step 2: Drift Analysis

An equivalent lateral force (ELF) or modal response spectrum analysis (MRSA) procedure is conducted using the linear model and DBE demands. The deflection amplification factor C_d is recommended to be set equal to R rather than being taken less than R, as would be the case if adopting the C_d for special reinforced concrete shear walls. This recommendation is based on Uang and Maarouf (1994).

The story drifts in both directions of the building are checked and compared with the maximum allowable story drifts of 2% of the story heights (values prescribed by ASCE 7-16 Table 12.12-1). If the design does not satisfy the drift requirements, the design should iterate back to Step 1 until the drift requirements are met.

Step 3: Rotation Demand at the Rocking Interface

With the drift limits satisfied, the rotation demand at the rocking interface can be calculated. Before determining the rotation demand, the story shear of the building must be calculated using either the ELF or MRSA procedure in accordance with ASCE 7-16 Section 12.8 or 12.9.1, respectively. This involves reducing the seismic demand by $R = 6$. When the MRSA procedure is used, the base shear must be scaled to match the base shear obtained from the ELF analysis in the specified direction as prescribed by ASCE 7-16 Section 12.9.1.4.1.

The rocking wall rotation demands at DBE and MCE$_R$ intensity levels can be calculated following Eqs. (1) and (2), respectively, as illustrated in Fig. 3

$$\theta_{\text{design}} = \frac{C_1 \times \delta_x - \delta_{xe,y}}{h_w} \tag{1}$$

$$\theta_{\text{maximum}} = \frac{1.5 \times C_1 \times \delta_x - \delta_{xe,y}}{h_w} \tag{2}$$

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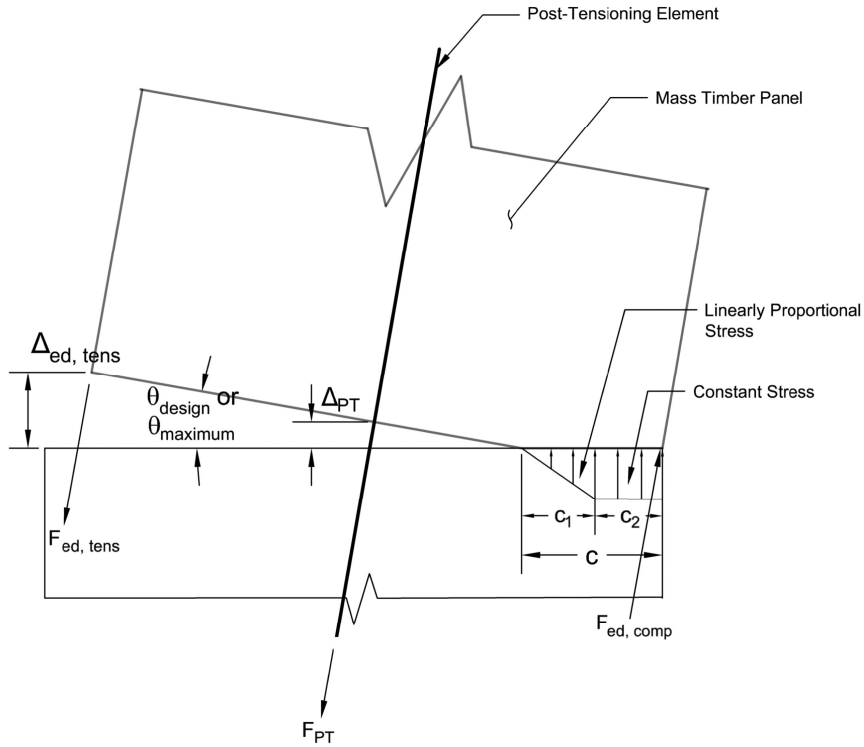

Fig. 3. Statics and kinematics of the rocking interface.

where C_1 = inelastic displacement amplification factor taken in accordance with ASCE 41-17 Equation 7.4.1.3.1; δ_x = deflection at the top of the wall from the linear drift analysis in Step 2; $\delta_{xe,y}$ = deflection at the top of the wall when subjected to the floor forces that produce the ELF or MRSA procedure base shear; and h_w = height of the wall. Eq. (1) is derived by equating the total deflection at the top of the wall, after approximately adjusting/amplifying for inelastic response (i.e., $C_1 \times \delta_x$), to the sum of the elastic deflection of the wall at yield of the system (i.e., $\delta_{xe,y}$) and the inelastic rotation due to gap opening at the base (i.e., $\theta_{\text{design}} \times h_w$). Eq. (2) uses the relationship that the MCE_R is 1.5 times the DBE. These calculated rotational demands will subsequently be used in the limit-state calculations for steel and wood components checks.

Step 4: Local Component Design and Checks

Once the rotational demands are calculated, a trial selection of PT elements (bars or strands) and energy-dissipation elements should be made. With these, the following limit states can be checked. If not all satisfied, iteration of the design occurs until they are.

Restoring Ratio

The restoring ratio of the rocking wall is defined as the ratio between the approximate resisting force provided by the initial stress in the PT elements plus additional dead load (i.e., $F_{\text{PT}i} + 0.9 \times P_u$) and that provided by the energy-dissipation elements on the tension side at their ultimate capacity (i.e., $F_u \times A_{\text{edt}}$). It must exceed 1.0 [Eq. (3)]. This ratio is necessary to ensure that there is sufficient elastic restoring force to overcome the resistance from energy-dissipating elements in order to allow the system to recenter under the DBE event

$$\frac{F_{\text{PT}i} + 0.9 \times P_u}{2 \times F_u \times A_{ed,\text{tens}}} > 1.0 \tag{3}$$

where $F_{\text{PT}i}$ = initial post-tensioning force within the PT elements; P_u = vertical dead load in the wall at its base including the wall self-weight; F_u = ultimate strength of the energy-dissipation elements; and $A_{ed,\text{tens}}$ = area of the energy-dissipation elements on the tension side of the wall. Eq. (3) can be derived by idealizing all of the compression force to act at the wall toe during rocking and summing moments about that location. The moment contribution from the restoring forces (i.e., $F_{\text{PT}i}$ and $0.9 \times P_u$) acts at the center of the wall length (i.e., $L_w/2$), whereas the tension-side energy-dissipation contribution acts at L_w. Ensuring that the restoring moment exceeds the moment contribution from the energy-dissipation elements results in Eq. (3). The initial post-tensioning force, $F_{\text{PT}i}$, should consider post-tensioning losses over the lifetime of the structure, including post-tensioning components seating at transfer, force transfer through energy-dissipation components, shortening due to creep and shrinkage of the mass timber panels, and relaxation of post-tensioning components.

Energy-Dissipation Ratio

The energy-dissipation ratio is defined as the ratio between the resisting moment provided by the energy-dissipation elements and the total resisting moment (i.e., provided by the combination of the energy-dissipation elements, post-tensioning elements, and dead load). It must exceed 0.25. Eq. (4) is derived for the case where the energy-dissipation elements in tension and compression are at the extreme edge of the wall in accordance with Fig. 3. This requirement is adopted from ACI ITG 5.2-09, which specifies a minimum amount of energy dissipation to prevent the design of an undamped or inadequately damped system based on the work of Kurama (2002)

$$\frac{(|F_{ed,\text{tens}}| + |F_{ed,\text{compr}}|) \times L_w/2}{M_n} > 0.25 \tag{4}$$

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$$F_{ed,\text{tens}} = A_{ed,\text{tens}} \times \text{MIN}\begin{cases} f_{\text{yed}} \\ E_{ed} \times \varepsilon_{ed,\text{tens}} \end{cases} \tag{5}$$

$$\varepsilon_{ed,\text{tens}} = \frac{\Delta_{ed,\text{tens}}}{L_{ed}} \tag{6}$$

$$\Delta_{ed,\text{tens}} = \theta_{\text{design}}(L_w - c) \tag{7}$$

$$F_{ed,\text{compr}} = A_{ed,\text{compr}} \times \text{MAX}\begin{cases} -f_{yed} \\ E_{ed} \times \varepsilon_{ed,\text{compr}} \end{cases} \tag{8}$$

$$\varepsilon_{ed,\text{compr}} = \frac{\Delta_{ed,\text{compr}}}{L_{ed}} \tag{9}$$

$$\Delta_{ed,\text{compr}} = -\theta_{\text{design}} \times c \tag{10}$$

where $F_{ed,\text{tens}}$ = force due to the energy-dissipation elements in tension; $F_{ed,\text{compr}}$ = force due to the energy-dissipation elements in compression; L_w = length of the wall; M_n = nominal moment capacity (this will be calculated in Step 5, and iteration may be needed here for this limit state); f_{yed} = yield strength of the energy-dissipation elements; E_{ed} = elastic modulus of the energy-dissipation elements; $\varepsilon_{ed,\text{tens}}$ = strain in the energy-dissipation elements in tension; $\varepsilon_{ed,\text{compr}}$ = strain in the energy-dissipation elements in compression; $\Delta_{ed,\text{tens}}$ = elongation of the energy-dissipation elements in tension; $\Delta_{ed,\text{compr}}$ = elongation of the energy-dissipation elements in compression; L_{ed} = length of the energy-dissipation elements; and c = neutral axis depth. Where significant flexibility exists in the connections of the energy-dissipation elements, this flexibility would need to be included in Eqs. (6) and (9) and would reduce the strain in the energy-dissipation elements. Where significant axial elongation or shortening of the tension or compression side, respectively, of the wall above the base occurs, this displacement would need to be included in Eqs. (7) and (10).

Limited Wall Toe Crushing at DBE

Acceptance of wood crushing at the wall toe is checked against an allowable wood strain. In the spirit of load and resistance factor design (LRFD), a resistance factor (phi) of 0.9 is used similar to that for wood compression parallel to grain in the NDS. Generally, $\varepsilon_{\text{wood},u}$ is taken to exceed the elastic strain of the mass timber panel in compression (i.e., inelastic deformation is permitted). It typically must be determined by project-specific testing or provided by the mass timber panel manufacturer based on their testing

$$\frac{\varepsilon_{\text{wood,compr}}}{(\phi = 0.9) \times \varepsilon_{\text{wood},u}} \leq 1.0 \tag{11}$$

$$\varepsilon_{\text{wood,compr}} = \phi_p \times c \tag{12}$$

$$\phi_p = \frac{\theta_{\text{design}}}{H_p} \tag{13}$$

where $\varepsilon_{\text{wood,compr}}$ = strain in the extreme compression fiber; $\varepsilon_{\text{wood},u}$ = ultimate strain capacity in the wood; ϕ_p = curvature of the section in the plastic hinge; and H_p = height of the plastic hinge in the mass timber panel. The concept of a plastic hinge is adopted from that used for reinforced concrete walls. H_p may be determined by reviewing existing testing of mass timber panels or by engineering judgment. To date, there is no simple estimate of the plastic hinge length for mass timber panels like that which exists for concrete (e.g., $0.06 \times h_w$ per ACI ITG 5.1).

Prevent PT Element Yielding at DBE

In addition to checking the restoring ratio, PT elements must not yield for the wall to recenter. In Eq. (14), $\phi = 0.9$ is per AISC 360 (AISC 2016) Section D2. Eq. (17) is derived for a wall with the PT elements placed at the wall center in accordance with Fig. 3. This requirement is only enforced for the DBE. The design procedure does not target nor assure recentering at the MCE_R

$$\frac{F_{\text{PT}i} + F_{\text{PT}\Delta}}{(\phi = 0.9) \times f_{y\text{PT}} \times A_{\text{PT}}} \leq 1.0 \tag{14}$$

$$F_{\text{PT}\Delta} = E_{\text{PT}} \times \varepsilon_{\text{PT}\Delta} \times A_{\text{PT}} \tag{15}$$

$$\varepsilon_{\text{PT}\Delta} = \frac{\Delta_{\text{PT}}}{L_{\text{PT}}} \tag{16}$$

$$\Delta_{\text{PT}} = \theta_{\text{design}}\left(\frac{L_w}{2} - c\right) \tag{17}$$

where $F_{\text{PT}\Delta}$ = force due to the elongation of the PT elements at DBE; $f_{y\text{PT}}$ = yield strength of the PT elements; A_{PT} = total area of the PT elements considering reduction due to threads; E_{PT} = elastic modulus of the PT elements; $\varepsilon_{\text{PT}\Delta}$ = strain in the PT elements due to elongation at DBE; Δ_{PT} = elongation of the PT elements at DBE; and L_{PT} = length of the PT elements. Where PT elements are spaced significantly away from the center of the wall, the $L_w/2$ term in Eq. (17) should be modified to be the actual distance from toe of wall to the location of the PT elements.

PT Element Failure at MCE_R

Although the PT elements are permitted to yield beyond the DBE, they must not fail under the MCE_R

$$\frac{\varepsilon_{\text{PT}f}}{\varepsilon_{\text{PT},u}} \leq 1.0 \tag{18}$$

$$\varepsilon_{\text{PT}f} = \frac{F_{\text{PT}i}}{E_{\text{PT}} \times A_{\text{PT}}} + \varepsilon_{\text{PT}\Delta} \tag{19}$$

where $\varepsilon_{\text{PT}f}$ = final strain in the PT elements at MCE_R; $\varepsilon_{\text{PT},u}$ = ultimate strain capacity of the PT elements; $\varepsilon_{\text{PT}\Delta}$ = strain in the PT elements due to elongation at MCE_R; and Δ_{PT} = elongation of the PT elements at MCE_R. Low-cycle fatigue effects should be considered in the establishment of $\varepsilon_{\text{PT},u}$. Furthermore, $\varepsilon_{\text{PT}\Delta}$ in Eq. (19) must be calculated in accordance with Eqs. (16) and (17) but using θ_{maximum} instead of θ_{design}.

Energy-Dissipation Element Failure at MCE_R

The energy-dissipation elements must not failure under the MCE_R

$$\frac{\varepsilon_{ed,\text{tens}}}{\varepsilon_{ed,u}} \leq 1.0 \tag{20}$$

where $\varepsilon_{ed,u}$ = ultimate strain capacity of the energy-dissipation elements. Low-cycle fatigue effects should be considered in the establishment of $\varepsilon_{ed,u}$. Furthermore, $\varepsilon_{ed,\text{tens}}$ in Eq. (20) must be calculated in accordance with Eqs. (6) and (7) but using θ_{maximum} instead of θ_{design}.

If any of these limit states are not satisfied, the designer must go back to the beginning of this step and modify the selection of PT or energy-dissipation elements. When such wall-level adjustment is not able to produce satisfactory design, it may also be necessary to go back to the preliminary design step and adjust wall thickness, lengths, and/or locations.

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Step 5: System Design and Checks

With the local component checks satisfied, the moment capacity of the rocking mechanism should be checked against the LRFD moment demand at the wall base from the ELF or MRSA procedure in accordance with Eq. (21)

$$M_u \leq \phi M_n \tag{21}$$

where M_n is calculated by summing the forces and stresses shown in Fig. 3 about the wall center. In Eq. (22), this has, for simplicity, been separated into the contribution from the energy-dissipation elements in tension and compression and from the wood compressive zone

$$\phi M_n = \phi(M_{ed,\text{comp}} + M_{\text{wood,comp}} + M_{ed,\text{tens}}) \tag{22}$$

where ϕ = strength reduction factor equal to 0.9 for compression parallel to grain in accordance with the NDS; $M_{ed,\text{comp}}$ = moment due to the energy-dissipation elements in compression; $M_{\text{wood,comp}}$ = moment due to the wood in compression; and $M_{ed,\text{tens}}$ = moment due to the energy-dissipation elements in tension.

The following equations are relevant to this calculation and are based on the free-body diagram presented in Fig. 3, including the assumption of arrangement of the post-tensioning and energy-dissipation elements in that figure:

$$M_{ed,\text{comp}} = \frac{F_{ed,\text{comp}} \times -L_w}{2} \tag{23}$$

The value of $M_{ed,\text{tens}}$ is computed similarly.

$M_{\text{wood,comp}}$ is further broken down into two parts corresponding to the linearly varying stress portion and the constant stress portion. This elastic-perfectly plastic representation of the wood is a simplification. Where the complete stress-strain curve is available, it may be used instead

$$M_{\text{wood,comp1}} = F_{\text{wood,comp1}}\left(-\frac{L_w}{2} + c_2 + \frac{c_1}{3}\right) \tag{24}$$

$$F_{\text{wood,comp1}} = 0.5 \times c_1 \times b_w \times \text{MAX}\begin{cases} E_{\text{wood}} \times \varepsilon_{\text{wood,comp}} \\ -K_F \times \lambda \times F'_c \end{cases} \tag{25}$$

$$c_1 = \text{MIN}\begin{cases} \dfrac{\varepsilon_y}{\phi_p} \\ c \end{cases} \tag{26}$$

$$\varepsilon_y = \frac{K_F \times \lambda \times F'_c}{E_{\text{wood}}} \tag{27}$$

$$c_2 = \text{MAX}\begin{cases} c - c_1 \\ 0 \end{cases} \tag{28}$$

where $F_{\text{wood,comp1}}$ = force in the wood in the region of linearly varying stress; c_1 = length of the wall that has linearly varying compressive stress; c_2 = length of the wall that has constant compressive stress; b_w = total width of the vertical laminations; E_{wood} = elastic modulus of the wood for the vertical laminations; $K_F \times \lambda \times F'_c$ = LRFD compression strength of the wood; and ε_y = yield strain of the wood

$$M_{\text{wood,comp2}} = F_{\text{wood,comp2}}\left(-\frac{L_w}{2} + \frac{c_2}{2}\right) \tag{29}$$

$$F_{\text{wood,comp2}} = \begin{cases} 0, c_2 = 0 \\ -K_F \times \lambda \times F'_c \times c_2 \times b_w, c_2 \neq 0 \end{cases} \tag{30}$$

where $F_{\text{wood,comp2}}$ = force in the wood in the region of constant compressive stress.

The shear capacity of the wall should be checked against the LRFD shear demand at the wall base from the ELF or MRSA procedure in accordance with Eq. (31)

$$V_{ua} \leq \phi V_n \tag{31}$$

The shear demand can be determined by taking the ELF or MRSA procedure shear and further amplifying it using Eq. (32)

$$V_{uA} = 1.5 \times \Omega_v \times \alpha_v \times V_u \leq 4.5 \times V_u \tag{32}$$

$$\Omega_v = \frac{M_{pr}}{M_u} \tag{33}$$

where V_{uA} = amplified shear; Ω_v = overstrength factor of the wall at the critical section; M_{pr} = probable moment capacity; and M_u = moment demand. The overstrength factor, Ω_v, increases the shear demand over the full height of the wall due to the overstrength in flexure at the rocking interface. Overstrength in flexure comes from several sources, including accounting for strength-reduction factors, strain hardening, and expected material properties of the energy-dissipation reinforcement, and overdesign. The parameter α_v is the dynamic shear amplification factor, which is required to be calculated using a rational analysis. Amplification of shear is needed to account for higher modes which, in contrast to the first mode, tend to respond elastically in rocking wall structures and are not reduced by the base rocking mechanism. The factor of 1.5 is to account for the parabolic distribution of shear in a rectangular cross section and V_u is the shear demand from the ELF or MRSA procedure. Then, ϕV_n is calculated in accordance with Eq. (34)

$$\phi V_n = \phi \times K_F \times \lambda \times F'_v \times L_w \tag{34}$$

where ϕ = strength-reduction factor; K_F = NDS format conversion factor; λ = time effect factor; and F'_v = in-plane shear capacity provided by the manufacturer.

If any of these limit states are not satisfied, the designer should either go back to the wall hardware selection and adjust the PT or energy-dissipation elements or to the preliminary design step and adjust wall thickness, lengths, and/or locations. Once all of the limit-state checks are satisfied, the design of the post-tensioned mass timber rocking wall is considered complete.

Design Example

In order to illustrate the proposed approach, the design process of a 6-story building with a post-tensioned mass timber rocking wall LFRS is presented here. This example building features a typical 30 m(100 ft) × 61 m(200 ft)office floor plan located in Seattle, Washington

Preliminary Design

The example building has six stories above grade with story heights of 3.7 m (12 ft), except for the first story, which has a story height of 6.1 m (20 ft). The building has a total seismic weight of 340 kg/m^2 (70 psf) per floor. The lateral system is composed of eight post-tensioned mass timber rocking walls made from 7-ply CLT, CLT diaphragms, and two core openings. In this paper, the design

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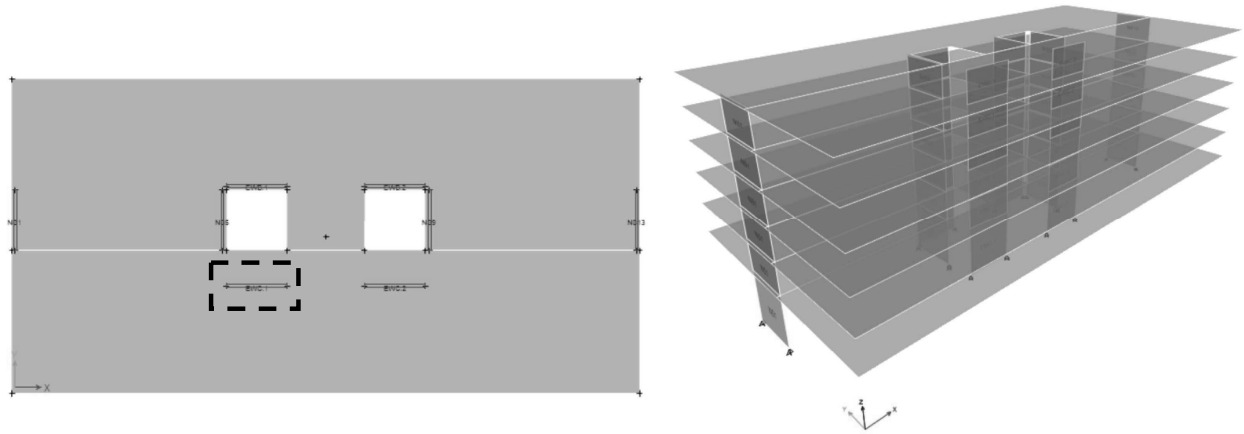

Fig. 4. Building layout with wall selection.

process for one of the walls in the long direction (x-direction, left–right on page) is presented. In practice, this process would need to be repeated for all walls in both directions.

Based on architectural considerations, it is common for a commercial multistory building to locate lateral force–resisting elements in a core configuration around stair, elevator, and mechanical shafts. Additionally, a hallway from the stair/elevator lobby often exists and is used to place the lower long direction walls in this example. Thus, the post-tensioned mass timber rocking walls in the long direction of the building are set to be 5.9 m (19.5 ft) in length. In the short direction, the far-left and far-right rocking walls are placed to reduce diaphragm cantilever distances and provide torsional resistance. The height of the wall is equal to the height of the building, 24.4 m (80 ft). In this study, the building is modeled using ETABS. Fig. 4 shows the linear model of the building with the example wall highlighted.

The site seismic hazard parameters were obtained from the ATC Hazards by Location tool (ATC 2016) with reference to ASCE 7-16 for a site in Seattle, Washington. The seismic hazard and key design parameters used in this example are listed in Table 1.

Drift Analysis

In this example, the MRSA procedure is used. Based on results from the MRSA in ETABS using $R = 6$ and $C_d = 6$, the distribution of the story drift ratios under the DBE event are shown in Fig. 5. The deformed shape of the rocking walls is also shown in Fig. 5. The maximum drift responses in both directions do not exceed the 2% limit set by ASCE 7 Table 12.12-1 for All Other Structures for a Risk category II building.

Rotation Demand at the Rocking Interface

The gap opening of the wall at the DBE and MCE_R can be calculated using Eqs. (1) and (2). Here, δ_x is the roof displacement from the drift analysis in Step 2 (i.e., $R = 6$ and $C_d = 6$), and $\delta_{xe,y}$ is the roof displacement under the forces corresponding to the design base shear (i.e., $R = 6$ plus scaling of the base shear to not be less than the ELF procedure).Because the fundamental period of the building exceeds 1.0 s, C_1 is equal to 1.0 (i.e., equal displacement rule)

$$\theta_{\text{design}} = \frac{1.0 \times 272\ \text{mm} - 93.5\ \text{mm}}{24{,}400\ \text{mm}} = 0.007\ \text{rad} \quad \text{(SI units)}$$

$$\theta_{\text{design}} = \frac{1.0 \times 10.7\ \text{in.} - 3.68\ \text{in.}}{960\ \text{in.}} = 0.007\ \text{rad} \quad \text{(US units)}$$

Table 1. Key design parameters

Parameters	Value
Mapped acceleration parameters	
S_s	$1.38g$
S_1	$0.48g$
Site class	C
Short-period and long-period site coefficients	
F_a	1.2
F_v	1.5
Design spectral acceleration parameters	
$S_{\text{MS}} = F_a \times S_s$	$1.65g$
$S_{M1} = F_v \times S_1$	$0.72g$
Design peak ground acceleration = 0.4 S_{DS}	$0.44g$
$S_{\text{DS}} = 2/3\ S_{\text{MS}}$	$1.10g$
$S_{D1} = 2/3\ S_{M1}$	$0.48g$
Risk category	
Risk category	II (primary occupancy will be office)
Importance factor	
Importance factor	1.0
Seismic design category	
SDC	D
Design parameters	
R	6
C_d	6 (set equal to R)
Modeling properties for 7-ply CLT	
Elastic modulus parallel to grain	12,400 MPa (1,800 ksi)
LRFD compression strength of wood parallel to grain = $K_F \times \lambda \times F'_c$	31,000 kPa (4,500 psi)
Yield strain of wood	0.0025
Ultimate strain of wood	0.015[a]

Note: SDC = seismic design category.
[a]Ultimate strain of wood for this example is based on limited testing (Zimmerman and McDonnell 2018) and engineering judgment. Value provided here is not intended to be used as a standard value.

$$\theta_{\text{maximum}} = \frac{1.5 \times 1.0 \times 272\ \text{mm} - 93.5\ \text{mm}}{24{,}400\ \text{mm}} = 0.013\ \text{rad} \quad \text{(SI units)}$$

$$\theta_{\text{maximum}} = \frac{1.5 \times 1.0 \times 10.7\ \text{in.} - 3.68\ \text{in.}}{960\ \text{in.}} = 0.013\ \text{rad} \quad \text{(US units)}$$

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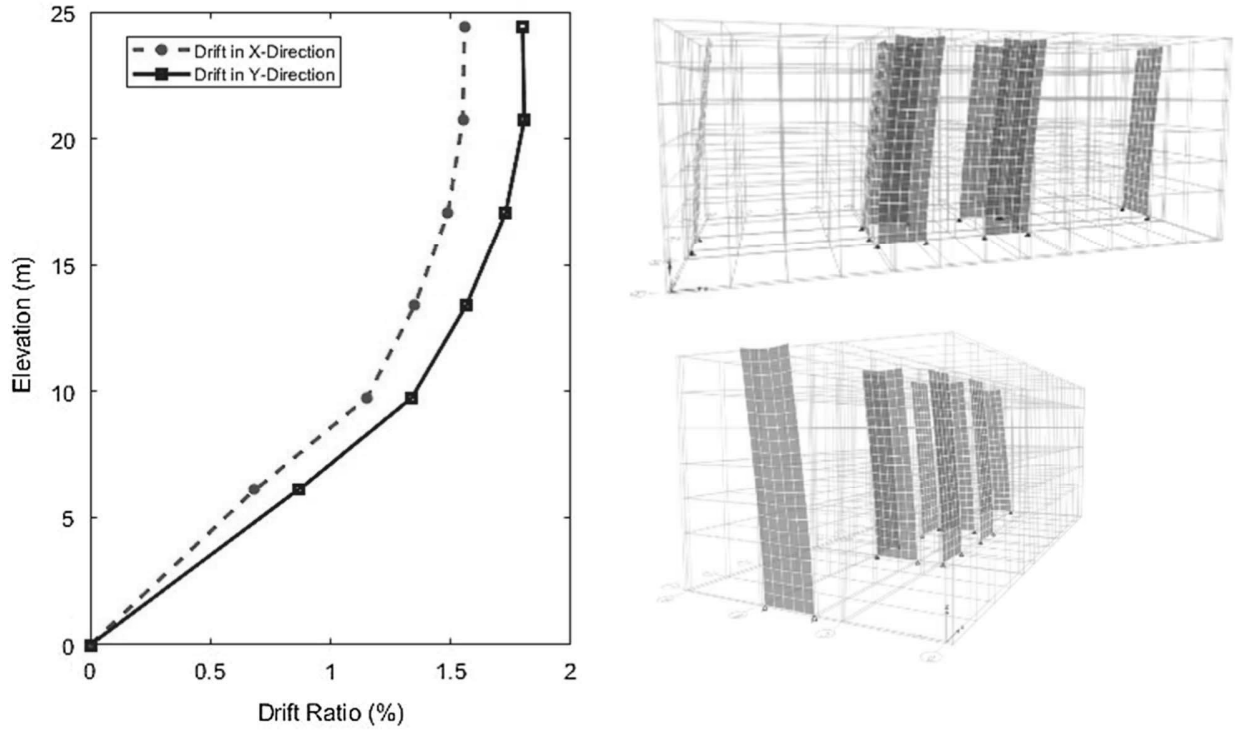

Fig. 5. Story drifts and deformed shapes.

Local Component Design and Checks

Following the determination of the rotation demands, a trial selection of energy dissipation and PT elements can be made. The PT elements are selected to be eight threaded rods (ASTM F1554 Grade 105) with a 44.5-mm (1.75-in.) diameter, a yield strength of 725 MPa (105 ksi), an ultimate strength of 860 MPa (125 ksi), and are initially post-tensioned to a total force of 2,890 kN (650 kips). The initial post-tensioning force is typically selected through a trial-and-error approach until all of the local component checks (e.g., restoring ratio, energy-dissipation ratio, and others) are satisfied. Post-tensioning can be achieved through temporary hydraulic jacks placed at the top of the wall after the full-height rocking wall panels are erected. The energy-dissipation elements consist of U-shaped flexural plates (UFPs). ASTM A572 Grade 42 steel is used, and the shape and capacity (Baird et al. 2014 gives full details) of the selected UFP device is shown in Fig. 6. The total number of UFPs is set to be 24 on each side of the rocking wall. Fig. 7(a) illustrates the configuration of the wall with the PT and energy-dissipation elements.

The component-level limit-state requirements listed in Step 4 were checked for the example design as follows, and the values used in these checks are illustrated in Figs. 7(b and c):

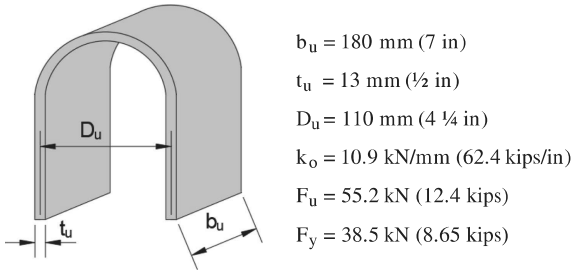

Fig. 6. U-shaped flexural plate with dimensions and capacity.

- 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

$$\text{Restoring ratio} = \frac{2{,}890\ \text{kN} + 0.9 \times 0}{2 \times 55.2\ \text{kN} \times 24} = 1.10 > 1.0 \quad (SI\ \text{units})$$

$$\text{Restoring ratio} = \frac{650\ \text{kips} + 0.9 \times 0}{2 \times 12.4\ \text{kips} \times 24} = 1.10 > 1.0 \quad (US\ \text{units})$$

- Energy dissipation [Eqs. (4)–(10)]

$$\text{Energy-dissipation ratio} = \frac{(925\ \text{kN} + 925\ \text{kN}) \times 5.94\ \text{m}/2}{15{,}500\ \text{kN} \cdot \text{m}} = 0.35 > 0.25 \quad (SI\ \text{units})$$

$$\text{Energy-dissipation ratio} = \frac{(208\ \text{kips} + 208\ \text{kips}) \times 234\ \text{in.}/2}{137{,}100\ \text{kip} \cdot \text{in.}} = 0.35 > 0.25 \quad (US\ \text{units})$$

- Limited wall toe crushing at DBE [Eqs. (11)–(13)] with the plastic hinge height, H_p, taken as 61 cm (24 in.)

$$\text{DCR}_{\text{wood,compstrain}} = \frac{-0.0118}{-0.9 \times 0.015} = 0.87 \leq 1.0$$

- No PT yielding at DBE [Eqs. (14)–(17)]

$$\text{DCR}_{\text{PT,yieldforce}} = \frac{2{,}890\ \text{kN} + 1{,}090\ \text{kN}}{0.9 \times 724\ \text{MPa} \times 98\ \text{cm}^2} = 0.62 \leq 1.0 \quad (\text{SI units})$$

$$\text{DCR}_{\text{PT,yieldforce}} = \frac{650\ \text{kips} + 246\ \text{kips}}{0.9 \times 105\ \text{ksi} \times 15.2\ \text{sq in.}} = 0.62 \leq 1.0 \quad (\text{US units})$$

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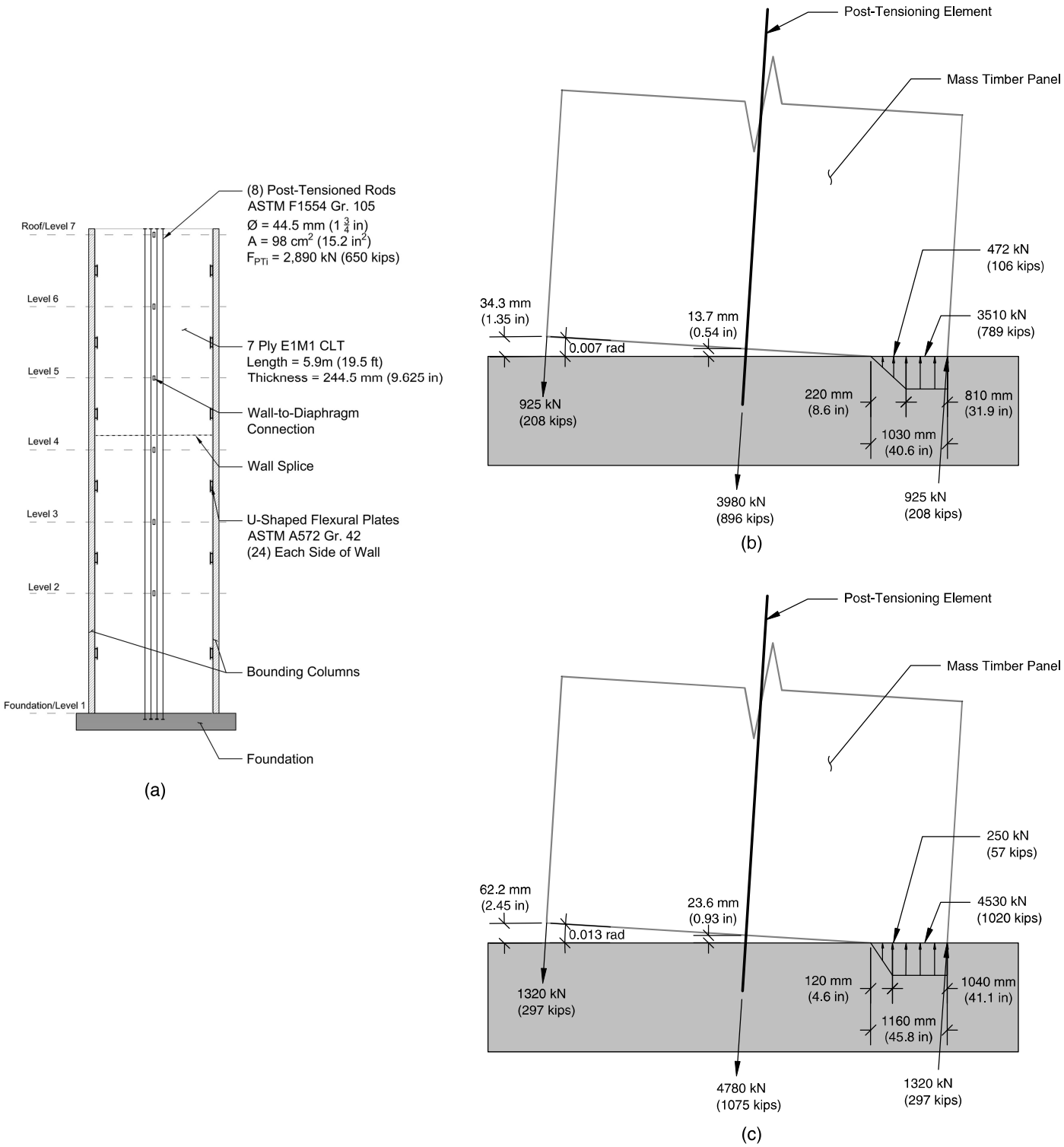

Fig. 7. (a) Configuration of the 6-story example wall; (b) free-body diagram of the 6-story example at the rocking interface for DBE; and (c) free-body diagram of the 6-story example at the rocking interface for MCE_R.

- No PT failure at MCE_R [Eqs. (18) and (19)]

$$\mathrm{DCR}_{\mathrm{PT,ultimatestrain}} = \frac{0.0024}{0.05} = 0.05 \leq 1.0$$

- No energy-dissipation failure at MCE_R. 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, this length would need to exceed $\Delta_{ed,\mathrm{tens}} = 62$ mm (2.45 in.) per Fig. 7(c) plus any elastic elongation of the tension side of the CLT wall panel at the location of the top story UFPs.

As shown in the preceding paragraphs, all the limit-state checks are satisfied.

LRFD Check at System Level

Once all the component-level limit states are checked and satisfied, the wall's shear and flexural capacity can be checked. In order to calculate the story shear demands, two analyses were conducted in ETABS in accordance with the ELF and MRSA procedures. In this example $C_u T_a = 0.75$ s, and the final ELF procedure base shear (for the entire building) is 4,100 kN (920 kips). The MRSA procedure base shear is 2,000 kN (450 kips). From this, the MSRA procedure base shear was scaled to match the ELF base shear, resulting in a scale factor approximately equal to 2.05.

The shear demand tributary to the wall of interest in this example is $V_u = 930$ 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

$$V_{uA} = 1.5 \times 1.5 \times 1.25 \times 930 \text{ kN} = 2{,}620 \text{ kN} \quad \text{(SI units)}$$

$$V_{uA} = 1.5 \times 1.5 \times 1.25 \times 210 \text{ kips} = 590 \text{ kips} \quad \text{(US units)}$$

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

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

The shear capacity mentioned in Eq. (34) in Step 5 is as follows, where the reference in-plane shear capacity is taken per the CLT manufacturer's literature:

$$\begin{aligned}\phi V_n &= 0.75 \times 2.88 \times 1.0 \times 5.94 \text{ m} \times 457 \text{ kN/m} \\ &= 5{,}870 \text{ kN} \quad \text{(SI units)}\end{aligned}$$

$$\begin{aligned}\phi V_n &= 0.75 \times 2.88 \times 1.0 \times 19.5 \text{ ft} \times 31.3 \text{ kip/ft} \\ &= 1{,}320 \text{ kips} \quad \text{(US units)}\end{aligned}$$

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

$$\phi M_{n,\text{design}} = 0.9 \times 15{,}500 \text{ kN} \cdot \text{m} = 13{,}900 \text{ kN} \cdot \text{m} \quad \text{(SI units)}$$

$$\begin{aligned}\phi M_{n,\text{design}} &= 0.9 \times 137{,}100 \text{ kip} \cdot \text{in.} \\ &= 123{,}400 \text{ kip} \cdot \text{in.} \quad \text{(US units)}\end{aligned}$$

The relevant values used to calculate the moment capacity are listed in Table 2.

These values will then be check against the demand values that were previously determined

$$V_{uA} = 2{,}620 \text{ kN} \leq 5{,}870 \text{ kN} = \phi V_{n,\text{design}} \quad \text{(SI units)}$$

$$V_{uA} = 590 \text{ kips} \leq 1{,}320 \text{ kips} = \phi V_{n,\text{design}} \quad \text{(US units)}$$

$$M_u = 13{,}400 \text{ kN} \cdot \text{m} \leq 13{,}900 \text{ kN} \cdot \text{m} = \phi M_{n,\text{design}} \quad \text{(SI units)}$$

$$\begin{aligned}M_u &= 118{,}300 \text{ kip} \cdot \text{in.} \leq 123{,}400 \text{ kip} \cdot \text{in.} \\ &= \phi M_{n,\text{design}} \quad \text{(US units)}\end{aligned}$$

Because they are both satisfied, the design is now considered complete. Although not shown in this example, the mass timber wall panel above the rocking interface would also need to be checked for combined bending and axial loading. Additionally, for a design where rocking wall panels need to be spliced with multiple panels up the height, a similar LRFD capacity check should be conducted for these splices following the NDS. The engineer should consider overstrength of the rocking mechanism and higher-mode amplification when performing checks of the wall panel for combined bending and axial loading, as well as the wall panel splices for shear and for combined bending and axial loading.

Table 2. Moment capacity calculation values at DBE

Parameter	Value
	$M_{\text{steel,compr}} = 2{,}740$ kN-m (24,290 kip-in.)
$F_{\text{steel,compr}}$	−925 kN (−208 kips)
$\Delta_{\text{steel,compr}}$	−7.1 mm (−0.28 in.)
	$M_{\text{wood,compr1}} = 990$ kN-m (8,720 kip-in.)
$F_{\text{wood,compr1}}$	−472 kN (−106 kips)
c_1	220 mm (8.6 in.)
ε_y	0.0025
	$M_{\text{wood,compr2}} = 9{,}010$ kN-m (79, 800 kip-in.)
$F_{\text{wood,compr2}}$	−3,510 kN (−789 kips)
c_2	810 mm (31.9 in.)
	$M_{\text{steel,tens}} = 2{,}750$ kN-m (24, 300 kip-in.)
$F_{\text{steel,tens}}$	925 kN (208 kips)
$\Delta_{\text{steel,tens}}$	34.3 mm (1.35 in.)

As-Designed Building Performance

In order to assess the performance of the final design, a simple numerical model for the designed rocking wall system was established with the design tributary mass at each floor. The model utilizes linear beam elements to capture the elastic deformation of the rocking wall panel, and implements a nonlinear rotational spring at the base to simulate rocking responses. This model has been validated using seismic test data from a full-scale 2-story mass timber building with PT rocking walls (Pei et al. 2019).

The numerical model was subjected to 10 earthquake ground motions adopted from the CUREE project (Folz and Filiatrault 2004). The design response spectrum value at the estimated building period was used to scale the ground motion suite. The response spectrum of the 10 ground motions scaled to DBE level is shown in Fig. 8.

The input parameters of the example building can be summarized in Table 3.

Fig. 9 shows the CDF distribution of the interstory drift of the building based on the time-history results from the simple numerical model. For SLE ground motions, the building is capable of controlling the peak interstory drift ratio under 1% with 100% confidence. For DBE ground motions, the building has 100% probability of controlling the peak interstory drift ratio under 2%. For MCE_R ground motions, the building has 90% probability of controlling the peak interstory drift ratio under 2.5%. The simulation results from the numerical model further reveal that the proposed design meets the strength and deformation limit states.

Conclusion and Future Work

In this study, a prescriptive design procedure for post-tensioned mass timber rocking walls is developed. This design approach is partially based on an existing design procedure for precast concrete rocking walls and can be integrated into the provisions of ASCE 7

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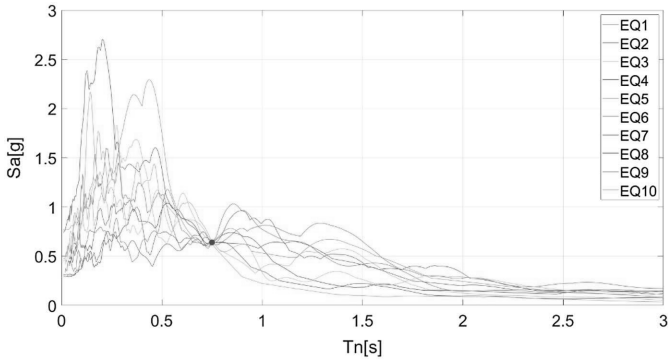

Fig. 8. Scaled response spectrum for DBE-level ground motions.

Table 3. Input parameters of the example building

Parameter	Value
Building information	
Story 1 height	6.1 m (240 in.)
Story 2–6 height (each story)	3.7 m (144 in.)
Story weight (each story)	6,230 kN (1,400 kips)
Seven-ply CLT rocking wall parameters	
E_{CLT}	12,400 MPa (1,800 ksi)
b_w	140 mm (5.5 in.)
L_w	5.9 m (234 in.)
h_w	24.4 m (80 ft)
PT properties	
Length of PT	24.4 m (80 ft)
Area (single PT bar)	12.3 cm^2 (1.90 sq in.)
Location	Center of wall
Elastic modulus, E_{pt}	200 GPa (29,000 ksi)
Initial force (single PT Bar)	360 kN (81 kips)
Yield stress	725 MPa (105 ksi)
Yielding deformation	76 mm (3.5 in.)
Number	8
UFP properties	
Number (per side)	24
Elastic stiffness (single UFP)	10.9 kN/mm (62.4 kip/in.)
Yield stiffness (single UFP)	0.11 kN/mm (0.62 kip/in.)
Yield deformation	3.6 mm (0.14 in.)

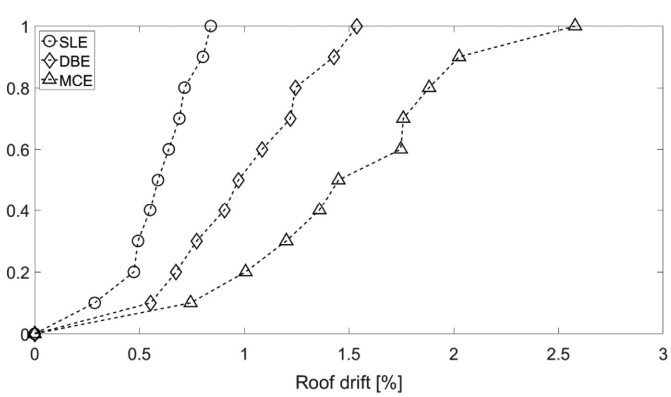

Fig. 9. CDF distribution of peak roof drift.

and *National Design Specification (NDS) for Wood Construction*. It includes assessment of drift limits, estimation of gap opening rotations, ensuring recentering, establishing minimum required energy dissipation, and several local element checks such as limiting toe crushing in the mass timber panel. Unlike performance-based design approaches that require nonlinear analysis, the proposed procedure only relies on standard modeling tools and analysis procedures available in a typical structural engineering office. The proposed procedure is demonstrated through a step-by-step design of a rocking wall system for a theoretical 6-story building in Seattle, Washington, and is validated through nonlinear response history analysis of a simplified numerical model.

Post-tensioned mass timber rocking wall systems are still relatively new and, as a result, it is difficult to gather performance data on buildings utilizing this system. Although the design method proposed here is based on data generated from recent research projects on rocking wall components and system tests, more thorough validation through nonlinear simulation and experimental testing should be conducted. Furthermore, additional analysis and testing (such as a complete FEMA P-695 study) need to be conducted in order to develop seismic design parameters for this system. Until this occurs, this design procedure can serve as well-developed starting point for projects in the US, which ultimately use performance-based design and nonlinear response history analysis to validate the design.

Data Availability Statement

All data, models, and code that support the findings of the study are available from the corresponding author upon reasonable request.

Acknowledgments

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