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PRESCRIPTIVE SEISMIC DESIGN PROCEDURE FOR POST-TENSIONED MASS TIMBER ROCKING WALLS IN THE UNITED STATES

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ABSTRACT: This paper describes a prescriptive seismic design procedure for a lateral force-resisting system composed of post-tensioned mass timber rocking walls. This procedure utilizes techniques and analysis methods routinely adopted by industry and adheres to force-based approaches found in the current U.S. seismic loading standard. Unlike performance-based design approaches that are more complex and require peer review, this design procedure targets future adoption into model codes by providing a basis for the prescriptive design of mass timber rocking wall lateral force-resisting systems. To illustrate this procedure, a series of example buildings were designed using the methods described in this paper.

KEYWORDS: Tall wood building, Seismic design, Mass timber rocking wall, Cross-laminated timber

1 INTRODUCTION

With the recent growth and popularity of mass timber in the United States, the number of mass timber building projects is expected to multiply. Although there has been increased traction for mass timber lateral force-resisting systems (LFRSs)—which utilize mass timber panels such as cross-laminated timber (CLT), laminated veneer lumber (LVL), and mass plywood panels—in research and development, the U.S. building code and its reference design standards provide limited options for mass timber seismic force-resisting systems as of the end of 2022. The one currently permitted system comprises platform-framed, conventionally connected CLT walls.

Out of the many potential mass timber LFRSs, post-tensioned mass timber rocking walls are thought to be the competitive choice for mass timber building projects in regions with high seismicity, particularly because they are considered a low-damage lateral system. The mechanism of this system is very similar to that of a precast concrete rocking wall system, which was heavily researched and tested in the 1990s and then codified via ACI ITG-5.1-07 and ITG-5.2-09 [1,2]. Researchers in New Zealand conceptualized the first timber version of this system, using LVL walls, and continued the development with several real building project applications [3]. The U.S. also adopted this system and has had real projects that were either permitted [4] or built [5,6]. However, all of these existing designs and analyses were conducted using advanced tools and procedures (performance-based design and nonlinear response history analysis), which are not typically used in average design offices. This situation calls for a prescriptive design method for post-tensioned mass timber rocking wall systems.

2 MASS TIMBER ROCKING WALL SYSTEM

The design procedure described in this paper is suitable for post-tensioned mass timber rocking wall LFRSs like those shown in Figure 1.

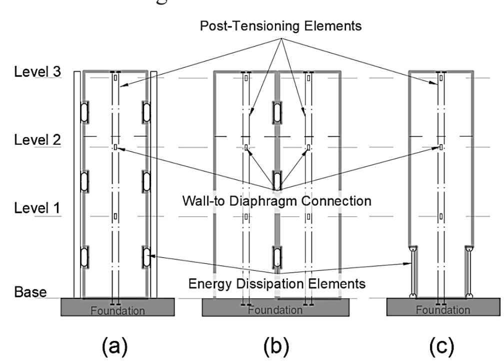

Figure 1: *(a) Conceptual configuration of a post-tensioned mass timber rocking wall with bounding columns; (b) conceptual configuration of coupled post-tensioned mass timber*

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rocking walls; and (c) conceptual configuration of an uncoupled post-tensioned mass timber rocking wall.

The walls consist of mass timber panels attached to the floor diaphragms in a balloon-framing configuration, and the panels can either be coupled or uncoupled. The material of the wall panels can be of the engineer's choice, given they meet the requirements of the National Design Specification (NDS) for Wood Construction's Chapter 8 for structural composite lumber or Chapter 10 for cross-laminated timber [7]. The wall panels are post-tensioned (P/T) at the center of the wall with high-strength steel strands or threaded rods. Energy dissipation (ED) elements, which can be yielding or frictional devices or specialized friction dampers, must also be part of the system to help absorb the earthquake input energy during the rocking motion. These damping elements can include, but are not limited to, U-shaped flexural plates (UFPs) or buckling restrained devices (BRBs). Connections between the walls and the diaphragms, and between the wall and the foundation must also be detailed to complete the lateral load path.

3 SEISMIC DESIGN APPROACH

This design methodology was created to be used by practicing engineers in the United States to design post-tensioned mass timber rocking walls as a lateral force-resisting system [8]. To accomplish this, a prescriptive design method using either the Equivalent Lateral Force (ELF) method or Modal Response Spectrum Analysis (MRSA) from ASCE 7 Chapter 12 [9] was created. The overall design is completed at the design basis earthquake (DBE) with some additional checks at the maximum considered earthquake (MCE_R). The approach can be broken into the five major steps described below:

3.1 PRELIMINARY DESIGN

This section contains the architectural design and the creation of a linear analysis model of the building.

3.2 DRIFT ANALYSIS

This section calls for the calculation of the building drifts and checking them against the allowable story drifts in ASCE 7 Chapter 12 [9].

3.3 ROTATIONAL DEMAND AT THE ROCKING INTERFACE

This section addresses the calculation of the gap opening at both DBE and MCE_R levels. This is done for each of the hazard levels by following Equations 1 and 2, respectively:

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

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

Where δ_x is the elastic roof displacement from a fixed-base, elastic model, C_1 is a factor that relates displacements from a linear model to expected inelastic displacements, $\delta_{xe,y}$ is displacement due to the elastic flexibility of the wall panel itself, and h_w is the height of the wall.

3.4 SYSTEM LEVEL DESIGN AND CHECKS

This section contains the determination of the system moment and shear demands from the fixed-based model and their scaling, when appropriate. In addition, the calculation of the system capacity using sectional analysis is also determined. Following the determination of the demands and capacities, a check is performed to ensure the strength of the system is sufficient. The moment capacity of the system shall be determined at the center of the wall for a single wall panel, and at the center of the system for a coupled rocking wall. The moment check can be determined using Equation 3.

$$M_{u,rm} \leq \phi_{rm} * M_{n,rm} \tag{3}$$

Where $M_{u,\ rm}$ is the moment demand of the rocking mechanism from ASCE 7 LRFD factored load combinations, excluding overstrength load combinations, φ_{rm} is the strength reduction factor for the rocking mechanism, and $M_{n,rm}$ is the nominal moment capacity of the rocking mechanism and can be calculated following Equation 4a for single wall panels and Equation 4b for coupled wall panels.

$$M_n = M_{ed,comp} + M_{wood,comp1} + M_{wood,comp2} + M_{ed,tens} \tag{4a}$$

$$M_n = M_{ed,comp} + M_{wood,comp1} + M_{wood,comp2} + M_{ed,tens} + M_{PT\Delta} \tag{4b}$$

Where $M_{ed.comp}$ is the moment due to the ED elements at the wall toe (i.e., compression end of the wall), $M_{wood,comp1}$ is the moment due to the compression of the wood in the linearly varying stress portion, $M_{wood,comp2}$ is the moment due to the compression of the wood in the constant stress portion, $M_{ed,tens}$ is the moment due to the ED elements at the wall heel (i.e., tension end of the wall), and $M_{PT\Delta}$ is the moment due to the incremental force from elongation of the P/T elements during rocking (i.e., total force in P/T elements minus initial P/T force). Each of these moments is defined further in Equations 5 through 9. It should be noted here that the following equations are derived for when the moment is being taken about the center of the wall. When the moment is being taken about a different point (e.g., in a coupled wall system), adjustment to the equations is required. In addition, Equation 9 is only needed for the coupled wall as indicated in Equation 4b.

$$M_{ed,comp} = \frac{F_{ed,comp} * -L_w}{2} \tag{5}$$

$$M_{wood,comp1} = F_{wood,comp1}\left(-\frac{L_w}{2} + c_2 + \frac{c_1}{3}\right) \tag{6}$$

$$M_{wood,comp2} = F_{wood,comp2}\left(-\frac{L_w}{2} + \frac{c_2}{2}\right) \tag{7}$$

$$M_{ed,tens} \ \frac{F_{ed,tens} * L_w}{2} \tag{8}$$

$$M_{PT\Delta} = F_{PT\Delta} * \frac{L_w}{2} \tag{9}$$

Where $F_{ed,\,comp}$ is the force due to the ED elements at the wall toe (i.e., compression end of the wall), $F_{wood,comp1}$ is the force in the wood in the region of linearly varying stress, $F_{wood,comp2}$ is the force in the wood in the region of constant compressive stress, $F_{ed,tems}$ is the force due to the ED elements at the wall heel (i.e., tension end of the wall), $F_{PT\Delta}$ is the force due to the elongation of the P/T elements during rocking (i.e., total force in P/T elements minus initial P/T force), L_w is the length of the wall, c_1 is the length of the linearly varying stress portion in the wall toe, and c_2 is the length of the constant compressive stress in the wall toe. It should be noted that c_1 and c_2 summed together will give c, the neutral axis depth of the wall. These forces are illustrated in the free body diagram in Figure 2 and can be calculated using sectional analysis. The probable moment of the system will be calculated using the same process but using the gap opening rotation at the MCE_R and expected/ultimate material properties.

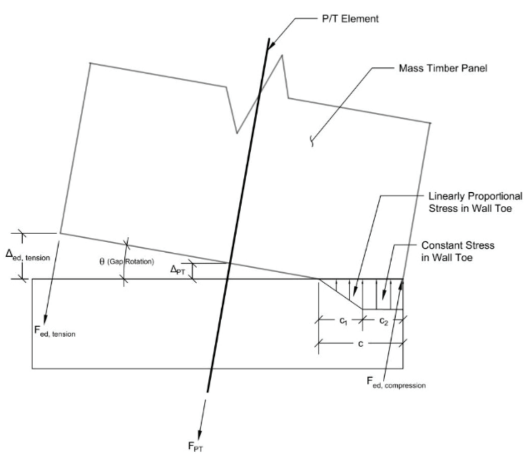

Figure 2: **Free body diagram of mass timber panel at the rocking interface.**

The shear check can be determined using Equation 10 which is also found in the NDS [7].

$$F'_v \geq f_v \tag{10}$$

Where F'_v is the adjusted shear design capacity determined per the NDS and f_v is the in-plane shear capacity of the mass timber panel. It should be noted that dynamic shear amplification may need to be accounted for due to the reduction of yielding in modes other than the fundamental mode for the wall.

3.5 LOCAL COMPONENT DESIGN AND CHECKS

The steps in this section are responsible for the determination of the wall hardware including the P/T and ED elements. Following the determination of these elements, several local checks—detailed below—are completed.

3.5.1 Restoring Ratio

This ensures the rocking wall will re-center after shaking up to and including the design basis earthquake. This is done by comparing the resisting force provided by the initial P/T force plus the additional dead load to the force provided by the ED components on the tension end of the wall at their ultimate capacity. This can be done following Equation 11 below.

$$F_{PTi} + 0.9 * P_u > 2 * F_u * A_{ed,tens} \tag{11}$$

Where F_{PTi} is the initial post-tensioning force in the P/T components, P_u is the vertical dead load in the wall at its base including the wall self-weight, F_u is the ultimate strength of the energy dissipation elements, and $A_{ed,tens}$ is the area of the energy dissipation elements on the tension end of the wall.

3.5.2 Energy Dissipation Ratio

This ratio enforces that there will be some energy dissipation from the system. To calculate this, a ratio of the moment capacity resulting from the energy dissipation devices to the total moment capacity is compared to a minimum limit. This ratio can be calculated using Equation 12 detailed below.

$$\frac{(M_n - M_{noed})}{M_n} \geq 0.25 \tag{12}$$

Where M_n is the nominal moment capacity of the wall and M_{noed} is the nominal moment capacity of the wall if there were no energy dissipation elements in the system.

3.5.3 Limited Wall-Toe Crushing at DBE

This ensures minimal damage occurs at the toe of the rocking wall for shaking up to and including the design basis earthquake and is completed by checking the strains in the wood. This can be calculated using Equation 13 detailed below.

$$\varepsilon_{wood,compr} \leq 0.9 * \varepsilon_{mtmax} \tag{13}$$

Where $\varepsilon_{wood,compr}$ is the strain in the extreme compression fiber and ε_{mtmax} is the maximum useable strain of the mass timber panel. The maximum useable strain shall not be taken greater than the strain at which the post-peak stress reaches 80 percent of the peak compression stress determined from an edgewise compression stress-strain curve.

3.5.4 No P/T Yielding at DBE

This is another check to ensure that re-centering will occur after shaking up to and including the design basis earthquake since P/T yielding compromises re-centering capability. This check is done by comparing the demand to yield stress of the P/T and can be calculated using Equation 14 detailed below.

$$f_{PT} \leq 0.9 * f_{yPT} \tag{14}$$

Where f_{PT} is the stress demand in the P/T elements at DBE and f_{yPT} is the yield stress.

3.5.5 No P/T Failure at MCE_R

This check is to ensure the P/T system will not fail at MCE_R. This is determined by calculating the strain in the P/T elements using Equation 15 detailed below.

$$\varepsilon_{PTf} \leq 0.85 * \varepsilon_{PT,u} \tag{15}$$

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Where ε_{PTf} is the strain demand in the P/T elements at the MCE_R and $\varepsilon_{PT,u}$ is the ultimate strain capacity of the P/T element.

3.5.6 No ED Failure at MCE_R

This check is to ensure the ED elements will not fail at MCE_R. This is determined by calculating the strain or deformation in the ED elements using Equations 16a or 16b detailed below.

$$\varepsilon_{ed,tens} \leq 0.85 * \varepsilon_{ed,u} \qquad (16a)$$

$$\Delta_{ed,tens} \leq 0.85 * \Delta_{ed,u} \qquad (16b)$$

Where $\varepsilon_{ed,tens}$ or $\Delta_{ed,tens}$ is the strain or deformation of the ED elements in tension due to elongation, and $\varepsilon_{ed,u}$ or $\Delta_{ed,u}$ is the ultimate strain or deformation capacity of the ED elements.

4 DESIGN EXAMPLES

To illustrate the use of the proposed design approach, two design examples will be presented. The first example is a six-story mixed-use building featuring post-tensioned mass timber rocking walls with bounding columns. The second example features a three-story academic building with coupled post-tensioned mass timber rocking walls.

4.1 SIX-STORY EXAMPLE

4.1.1 Preliminary Design

This example features an office building located in Seattle, WA on Site Class C and a Seismic Design Category of D. The building features a 6.1m first story with the remaining stories being 3.7m in height. The floor plan of the building is shown below in Figure 3.

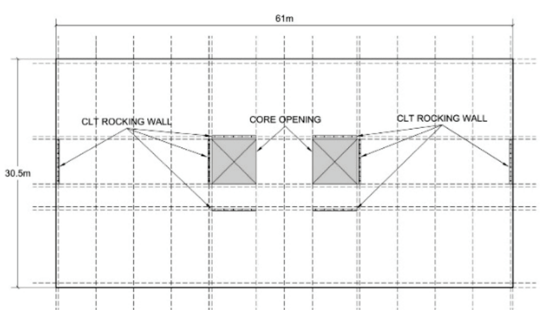

Figure 3: *Floor plan of the 6-story example building.*

The lateral system is composed of eight, 5.9m post-tensioned mass timber rocking walls consisting of 7-ply E1M1 CLT in a bounding column configuration. The wall components for this example are shown in Figure 4. Since this system's response modification factor was not yet available, it was assumed to be equivalent to that of a special reinforced concrete shear wall, giving an R = 6. Additionally, C_d was taken equal to R based on the recommendations made by Uang et. al [10]. Furthermore, the CLT panel material is isotropic and therefore needs to be modeled with appropriate stiffness parameters.

The linear elastic model for this example was built in ETABS with the seismic ground motion parameters listed in Table 1, which were determined using the ATC Hazards by Location Tool [11,12].

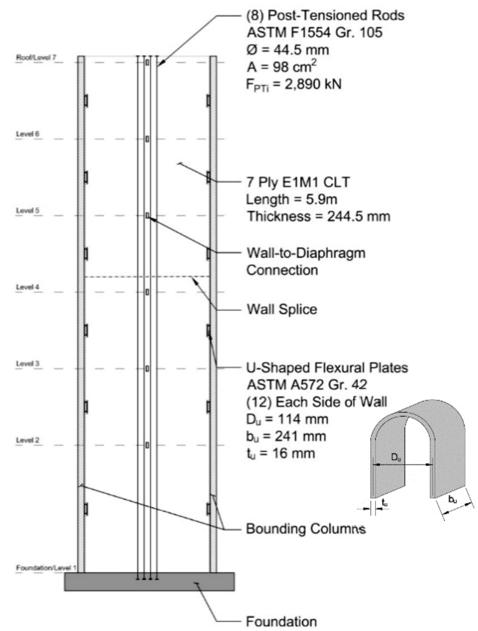

Figure 4: **Description of wall components, including UFP dimensions, for the 6-story example.**

Table 1: *List of seismic ground parameters for the 6-story example.*

Seismic Ground Motion Parameters	
The mapped MCE_R spectral response acceleration parameter at short periods, S_s	1.38
The mapped MCE_R spectral response acceleration parameter at a period of 1s, S_1	0.48
Short-Period Site Coefficient, F_a	1.2
Long-Period Site Coefficient, F_v	1.5
The MCE_R spectral response acceleration parameters for short periods, S_{MS}	1.65
The MCE_R spectral response acceleration parameters at a period of 1s, S_{M1}	0.72
The design spectral response acceleration parameter at short periods, S_{DS}	1.10
The design spectral response acceleration parameter at a period of 1s, S_{D1}	0.48

4.1.2 Drift Analysis

The drift of the building was determined using the linear elastic ETABS model and is shown in Figure 5. ASCE 7-16 Table 12.12-1 gives the Allowable Story Drift [9]. With the classification of "All other structures" and a Risk Category of II, the allowable story drift is 2%. Based on the data presented in Figure 5, it is evident that the drift in both directions does not exceed the 2% limit, therefore validating the current building layout and allowing for the design of the wall to continue.

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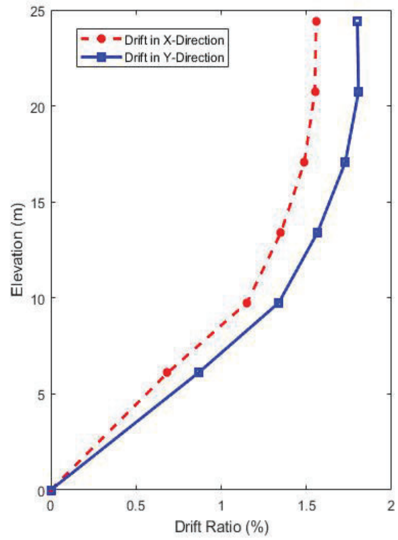

Figure 5: *Drift responses at each story for the 6-story example.*

4.1.3 Rotational Demand at the Rocking Interface

Using the analysis from the linear structural model created in the previous step, the deflections needed to calculate the rotations at the rocking interface can be determined. The design rotation at the rocking interface was given by Equation 1 and can be calculated as follows:

$$\theta_{design} = \frac{1.0*0.273m-0.093m}{24.4m} = 0.007\ rad$$

The maximum rotation at the rocking interface was given by Equation 2 and can be calculated as follows:

$$\theta_{max} = \frac{1.5*1.0*0.273m-0.093m}{24.4m} = 0.013\ rad$$

4.1.4 System Level Design and Checks

The nominal moment capacity of the wall system is calculated using Equation 4a, and subsequently Equations 5 through 8. The forces needed to calculate the moment capacity of the wall were determined using sectional analysis and are listed in Table 2.

Table 2: *Parameters needed to calculate moment capacity for the 6-story example.*

Parameter	Value
$F_{ed,\ comp}$	924 kN
$F_{wood,comp1}$	-472 kN
$F_{wood,comp2}$	-3513 kN
$F_{ed,tems}$	924 kN
c_1	0.22m
c_2	0.81m

Using the values populated in the table above, the individual moment capacities can be computed following Equations 5 through 8.

$$M_{ed,comp} = \frac{-924\ kN*-5.9m}{2} = 2{,}747\ kN*m$$

$$M_{wood,comp1} = -472\ kN\left(-\frac{5.9m}{2} + 0.81m + \frac{0.22m}{3}\right) = 986\ kN*m$$

$$M_{wood,comp2} = -3513\ kN\left(-\frac{5.9m}{2} + \frac{0.81m}{2}\right) = 9017\ kN*m$$

$$M_{ed,tens} = \frac{924\ kN*5.9m}{2} = 2{,}747\ kN*m$$

Following the individual moment capacity calculations, the nominal moment for the wall system can be calculated following Equation 4 as the following:

$$M_n = 2{,}747\ kN*m + 986\ kN*m + 9{,}017\ kN*m + 2{,}747\ kN*m = 15{,}497\ kN*m$$

The probable moment for this system would also be computed using the same equations, but with the maximum gap opening rotation and expected/ultimate material properties. For this example, the probable moment was calculated to be 19,421 kN*m.

Following the nominal moment capacity calculation, Equation 3 can be used to determine if the capacity is sufficient for the demand taken from the ETABS model.

$$13{,}367\ kN*m \le\ 0.9*15{,}497\ kN*m = 13{,}947\ kN*m$$

Based on the calculation above, the strength of the rocking mechanism is satisfied.

The shear check for this example can be computed following Equation 10 and is detailed below.

$$4.0\ MPa \ge 2.9\ MPa$$

Based on the calculation above, the in-plane shear capacity of the wall is sufficient. It should be noted that the in-plane shear at any section of the mass timber panel shall be calculated using the wall shear demand from ASCE 7-16 factored load combinations (excluding overstrength load combinations) [9]. For this example, the shear demand was determined considering amplification due to flexural overstrength and higher mode effects as reported elsewhere [8].

4.1.5 Local Component Design and Checks

4.1.5.1 Restoring Ratio

Based on Equation 11 and the wall components, the restoring ratio can be determined as the following:

$$2891\ kN + 0.9*0 > 2*\frac{110\ kN}{UFP}*12\ UFPs$$

This then gives the following:

$$2891\ kN > 2640\ kN$$

It should be noted that since this wall is in a bounding column configuration there is no additional dead load from the floors. Furthermore, the self-weight of the wall is conservatively neglected. Based on the calculation above, the system will be able to re-center.

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4.1.5.2 Energy Dissipation Ratio

Based on Equation 12 and the wall components and the previous calculations, the energy dissipation ratio can be determined as the following:

$$\frac{(15{,}497 kN*m - 10{,}002\ kN*m)}{15{,}497\ kN*m} = 0.35 \geq 0.25$$

Based on the calculation above, the energy dissipation of the system is sufficient.

4.1.5.3 Limited Wall-Toe Crushing at DBE

Based on Equation 13 and the wall components, the following can be computed:

$$0.0118 \leq 0.9 * 0.015 = 0.0135$$

Based on the calculation above, the strain of the extreme compression fiber of the mass timber panel is satisfied.

4.1.5.4 No P/T Yielding at DBE

Based on Equation 14 and the wall components, the following can be computed:

$$407\ MPa \leq 0.9 * 724 MPa = 652\ MPa$$

Based on the calculation above, there is no yielding of the P/T bars during DBE.

4.1.5.5 No P/T Fracture at MCE$_R$

Based on Equation 15 and the wall components, the following can be computed:

$$0.0024 \leq 0.85 * 0.05 = 0.043$$

Based on the calculation above, there is no fracturing of the P/T bars at the MCE$_R$.

4.1.5.6 No ED Fracture at MCE$_R$

For UFPs, the critical design property is the distance from the start of the 180-degree bend to the nearest attachment point (e.g., weld or bolt) on the straight portion. Based on Equation 16b and the wall components the following can be computed:

$$71.8\ mm \leq 0.85 * 127\ mm = 108\ mm$$

Based on the calculation above, there is no ED fracture at the MCE$_R$.

Since all the system and local level component checks are satisfied, the overall design of the wall is complete.

4.2 THREE-STORY EXAMPLE

4.2.1 Preliminary Design

This example features an academic building located in Seattle, WA on Site Class C and a Seismic Design Category of D. The building features story heights of 3.7m. The floor plan of the building is shown below in Figure 6.

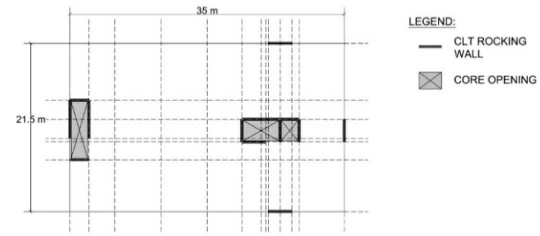

Figure 6: *Floor plan of the 3-story example building.*

The lateral system is composed of eight, 2.44m post-tensioned mass timber rocking walls consisting of 9-ply E1M1 CLT in a coupled wall configuration (not all walls in the floorplan are coupled). The wall components for this example are shown in Figure 7. The response modification factor for this system, since not yet available, was assumed to be equivalent to that of a special reinforced concrete shear wall, giving an R = 6. Additionally, C_d was taken equal to R based on the recommendations made by Uang et. al [10]. Furthermore, the CLT panel material is isotropic and therefore needs to be modeled with appropriate stiffness parameters.

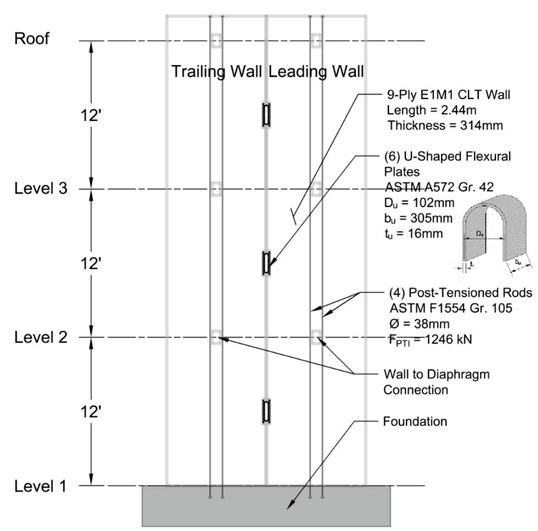

Figure 7: *Description of wall components, including UFP dimensions, for the 3-story example.*

The linear elastic model for this example was built in ETABS with the seismic ground parameters listed in Table 1.

4.2.2 Drift Analysis

The drift of the building was determined using the linear elastic model and is shown in Figure 8. ASCE 7-16 Table 12.12-1 gives the Allowable Story Drift [9]. With the classification of "All other structures" and a Risk Category of III, the allowable story drift is 1.5%. Based on the data presented in Figure 8, it is evident that the drift in both directions does not exceed the 1.5% limit, therefore validating the current building layout and allowing for the design of the wall to continue.

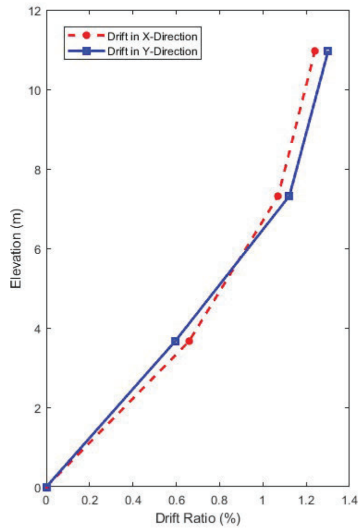

Figure 8: *Drift responses at each story for the 3-story example.*

4.2.3 Rotational Demand at the Rocking Interface

Using the analysis from the linear structural model created in the previous step, the deflections needed to calculate the rotations at the rocking interface can be determined. Since the coupled wall is symmetric in the linear analysis model, the design rotations at the rocking interface will be the same for both walls. The design rotation at the rocking interface was given by Equation 1 and can be calculated as follows:

$$\theta_{design} = \frac{1.1*0.094m-0.028m}{11.0m} = 0.007\ rad$$

The maximum rotation at the rocking interface of each wall was given by Equation 2 and can be calculated as follows:

$$\theta_{max} = \frac{1.5*1.1*0.094m-0.028m}{11.0m} = 0.013\ rad$$

Note that it is a coincidence that the design and maximum rotations at the rocking interface for this example match those of the 6-story example presented in Section 4.1.3.

4.2.4 System Level Design and Checks

For a coupled wall system, the subsequent calculations are broken down into the values for the trailing wall (subscript Tw) and leading wall (subscript Lw). The orientation of these walls is illustrated in Figure 7. The nominal moment capacity of each wall is calculated using Equation 4b, and subsequently Equations 5 through 9, with some modifications to the moment arm as mentioned previously and detailed below. Following the calculation of each wall's nominal moment capacity, the moment for the wall system will be determined following the summation from each wall. The forces needed to calculate the moment capacity of each wall were determined using sectional analysis and are listed in Table 3.

Table 3: *Parameters needed to calculate moment capacity for the 3-story example.*

Parameter	Value
Trailing Wall	
$F_{ed,\ comp}$	-657 kN
$F_{wood,comp1}$	-590 kN
$F_{wood,comp2}$	-368 kN
$F_{ed,tens}$	N/A
$F_{PT\Delta}$	411 kN
c_1	0.22 m
c_2	0.069 m
Leading Wall	
$F_{ed,\ comp}$	N/A
$F_{wood,comp1}$	-590 kN
$F_{wood,comp2}$	-1583 kN
$F_{ed,tens}$	657 kN
$F_{PT\Delta}$	312 kN
c_1	0.22 m
c_2	0.29 m

Using the values populated in the table above, the individual moment capacities can be computed following Equations 5 through 9. As previously mentioned, these equations need modifications to reflect the moment arm for summing moments about the center of the coupled wall system (rather than the center of each respective wall) and are reflected below:

$$M_{ed,comp\_TW} = F_{ed,comp\_TW} * 0 = -657\ kN * 0\ m = 0\ kN * m$$

$$M_{wood,comp1\_TW} = F_{wood,comp1\_TW} * \left(c_{2_{TW}} + \frac{c_{1_{TW}}}{3}\right) = -590\ kN\left(0.069\ m + \frac{0.218\ m}{3}\right) = -83.4\ kN * m$$

$$M_{wood,comp2\_TW} = F_{wood,comp2\_TW} * \left(\frac{c_{2_{TW}}}{2}\right) = -368\ kN\left(\frac{0.069\ m}{2}\right) = -12.6\ kN * m$$

$$M_{PT\Delta\_TW} = F_{PT\Delta} * \frac{L_w}{2} = 411\ kN * \frac{2.44\ m}{2} = 501.1\ kN * m$$

$$M_{ed,tens\_TW} = N/A$$

$$M_{ed,comp\_LW} = N/A$$

$$M_{wood,comp1\_LW} = -F_{wood,comp1\_LW} * \left(L_w - (c_{2_{LW}} + \frac{c_{1_{LW}}}{3})\right) = 590\ kN\left(2.44\ m - \left(0.29\ m + \frac{0.218\ m}{3}\right)\right) = 1{,}223\ kN * m$$

$$M_{wood,comp2\_LW} = -F_{wood,comp2\_LW} * \left(L_w - \frac{c_{2_{LW}}}{2}\right) = 1583\ kN\left(2.44m - \frac{0.29\ m}{2}\right) = 3{,}628.1\ kN * m$$

$$M_{PT\Delta\_TW} = -F_{PT\Delta} * \frac{L_w}{2} = 312.3\ kN * \frac{2.44\ m}{2} = -380.7\ kN * m$$

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$$M_{ed,tens\_LW} = F_{ed,tens\_LW} * 0 = 657\ kN * 0\ m = 0\ kN * m$$

Based on the calculations above, the moment capacity for each wall can be computed following Equation 4b as the following:

$$M_{n\_TW} = 0\ kN * m - 83.4\ kN * m - 12.6\ kN * m + 501.1\ kN * m = 405.1\ kN * m$$

$$M_{n_{LW}} = 1{,}223\ kN * m + 3{,}628.1\ kN * m - 380.7\ kN * m + 0\ kN * m = 4{,}470.4\ kN * m$$

$$M_{n\_tot} = 405.1\ kN * m + 4{,}470.4\ kN * m = 4{,}875.5\ kN * m$$

The probable moment for this system would also be computed using the same equations, but with the maximum design values. For this example, the probable moment was calculated to be 6,070 kN*m.

Following the determination of the moment capacity of the wall system, Equation 3 can be used to determine if the capacity is sufficient compared to the demand determined by the linear analysis model.

$$3{,}510.4\ kN * m \leq 0.9 * 4{,}875.5\ kN * m = 4{,}388\ kN * m$$

Based on the calculation above the strength of the rocking mechanism is satisfied.

The shear check for this example can be computed following Equation 10 and is detailed below.

$$4.0\ MPa \geq 1.2\ MPa$$

Based on the calculation above the in-plane shear capacity of the wall is sufficient. It should be noted that the in-plane shear at any section of the mass timber panel shall be calculated using the wall shear demand from ASCE 7-16 factored load combinations (excluding overstrength load combinations).

4.2.5 Local Component Design and Checks

The restoring and energy dissipation ratios will be checked at the system level for the coupled walls. However, all other local component checks will be broken down by the individual wall panel.

4.2.5.1 Restoring Ratio

Equation 11 will be used to determine the restoring ratio for the wall system. Since there are two walls in the system, the restoring force provided by each wall will need to be considered in the calculation. This can be done by summing the two forces as follows:

$$(1245.5\ kN + 0.9 * 42.3\ kN) + (1245.5\ kN + 0.9 * 42.3\ kN) > 2 * \frac{156.4\ kN}{UFP} * 6\ UFPs$$

This then gives the following:

$$2567.1\ kN > 1876.8\ kN$$

Based on the calculation above, the system will be able to re-center.

4.2.5.2 Energy Dissipation Ratio

Equation 12 will be used to determine the energy dissipation ratio for the wall system. Since there are two walls in the system, the moment contribution from both walls will need to be considered in the calculation. This can be done by summing the moments as follows:

$$\frac{(405.1\ kN*m - 204.4\ kN*m) + (4470.4\ kN*m - 3138.4\ kN*m)}{405.1\ kN*m + 4470.4\ kN*m} = 0.314 \geq 0.25$$

Based on the calculation above, the energy dissipation of the system is sufficient.

4.2.5.3 Limited Wall-Toe Crushing at DBE

The strain in the extreme compression fiber of the trailing wall can be calculated following Equation 13. This check is calculated below:

$$0.0032 \leq 0.9 * 0.015 = 0.0135$$

The strain in the extreme compression fiber of the leading wall can be calculated following Equation 13. This check is calculated below:

$$0.0058 \leq 0.9 * 0.015 = 0.0135$$

Based on the calculations above, the strain in the extreme compression fiber of both mass timber panels is satisfied.

4.2.5.4 No P/T Yielding at DBE

The stress in the P/T components of the trailing wall can be computed following Equation 14 and is shown below:

$$481\ MPa \leq 0.9 * 724MPa = 652\ MPa$$

The stress in the P/T components of the leading wall can be computed following Equation 14 and is shown below:

$$451\ MPa \leq 0.9 * 724MPa = 652\ MPa$$

Based on the calculations above, there is no yielding of the P/T bars during DBE in either wall panel.

4.2.5.5 No P/T Fracture at MCE_R

The strain in the P/T elements for the trailing wall can be calculated following Equation 15 and is shown below:

$$0.00295 \leq 0.85 * 0.05 = 0.043$$

The strain in the P/T elements for the leading wall can be calculated following Equation 15 and is shown below:

$$0.00259 \leq 0.85 * 0.05 = 0.043$$

Based on the calculations above there is no fracturing of the P/T elements during MCE_R in either wall panel.

4.2.5.6 No ED Fracture at MCE_R

The strain in ED elements between the trailing and leading wall can be calculated following Equation 16 and is shown below:

$$33.5mm \leq 0.85 * 127mm = 108mm$$

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Based on the calculations above, there is no fracturing of the ED elements during MCE_R.

Since all the system and local-level component checks are satisfied, the overall design of the wall system is complete.

5 CONCLUSIONS

This paper describes a prescriptive design procedure for post-tensioned mass timber rocking walls. Based on the existing design procedure for precast concrete walls, this design procedure was developed with a prescriptive format so that it can be integrated into the provisions of ASCE 7 [8] and the National Design Specification (NDS) for Wood Construction [9]. The design procedure includes an assessment of drift limits and an estimation of gap openings. It also checks the wall's ability to re-center, the minimum energy dissipation, and several local element checks. In addition to the ideology of the checks and calculations, two full building examples highlighting different wall configurations are also presented.

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