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NUMERICAL MODELING OF THE SEISMIC PERFORMANCE OF A 10-STORY MASS TIMBER BUILDING

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ABSTRACT: Mass timber buildings with post-tensioned rocking wall lateral force-resisting systems are now possible in areas of high seismicity and offer benefits such as fast construction, architectural uniqueness, reduced carbon footprint, and the potential for design for deconstruction. Additionally, these systems provide an opportunity to improve upon the usual collapse prevention performance for buildings in large earthquakes by developing design methods that enable resilient performance while maintaining an efficient design, creating a competitive lateral force-resisting system. To better understand the behavior and performance of seismically resilient mass timber buildings with mass timber rocking wall lateral force-resisting systems, the Natural Hazards Engineering Research Infrastructure (NHERI) TallWood Project designed and tested a 10-story building specimen at the NHERI outdoor shake table (LHPOST) at the University of California, San Deigo. The walls were initially designed using a linear force-based procedure and were then validated using nonlinear response history analysis in OpenSees. This presentation will compare the nonlinear numerical analysis predictions of seismic performance with the experimental results. Results indicate that the nonlinear modeling methods provide a good prediction of seismic performance while also highlighting areas that can be improved.

KEYWORDS: seismic, resilience, mass timber, shake table experiment, tall building

1 – INTRODUCTION

The NHERI TallWood Project team tested a full-scale 10-story mass timber building with a mass timber lateral force-resisting system. The prior work on the two-story test specimen (Wichman et al. 2022), such as the modeling work and validation of the design procedure, provided valuable insight for work on the 10-story specimen and the development of design recommendations for future design of tall timber buildings. The work presented here focuses on the design, modeling, and analysis of the mass timber lateral force-resisting system in the 10-story specimen. The other components of the test such as the gravity system, diaphragms, and nonstructural elements have been the focus of other collaborators. From a lateral force-resisting system prospective, the overall objective of this test was to (1) study the feasibility of designing and constructing a tall, resilient, fully mass timber building with a post-tensioned rocking wall lateral system for a high seismic region and (2) study the seismic performance of the structure and validate the design and modeling methodologies for use in future buildings and aid in the codification of this lateral system. Note, additional detail on the lateral system design, analysis, and experimental performance may be found in Wichman (2023) and forthcoming journal papers.

2 – NUMERICAL MODEL DEVELOPMENT AND VALIDATION WITH PREVIOUS TESTING

Description of Previous Testing

Pei et al. (2019) described testing of a two-story mass timber building on the NHERI@UCSD shake table atthe University of California San Diego. The test specimen is shown schematically in Figure 1. The specimen featured two rocking post-tensioned mass timber CLT shear walls and an extended cantilevered diaphragm. Each wall consisted of two CLT planels connected by U-shaped flexural plates (UFPs) that were designed to provide energy dissipation. Each wall was connected to steel foundation beams that were then attached to the shake table. The specimen was designed for the seismic hazard corresponding to a class B site in San Francisico, CA using demands computed with ASCE/SEI 7-10 (ASCE, 2010). Details of the specimen design can be found in Pei et al. (2019), Wichman (2018) and Wichman et al. (2022).

The specimen was tested using 1D ground motions selected and scaled to represent service level, design basis, and maximum considered earthquake shaking. The ground motions were applied in the in-plane direction of the CLT rocking shear walls. Experimental results showed essentially zero damage to the structure, even at

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the corners of the rocking wall. It was found that the flanges fo the foundation beams deformed slightly under the rocking walls and may have limited the damage to the wall corners. As described below, the were included in the nonlinear numerical model.

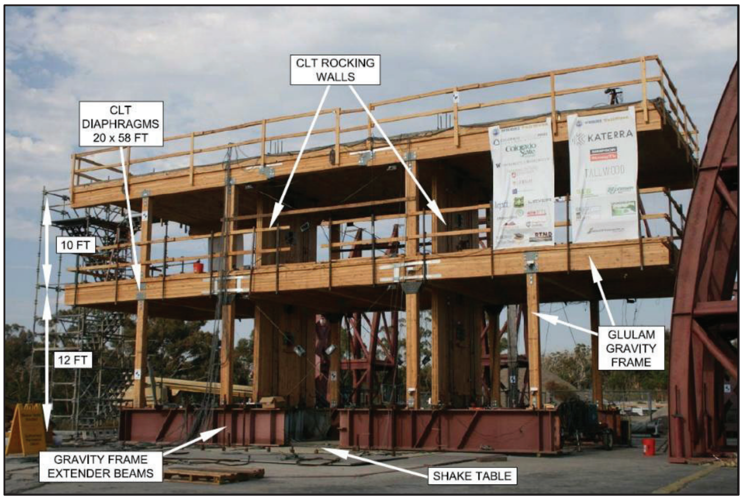

Figure 1. Two-Story Rocking Post-Tensioned CLT Wall Shake Table Specimen

Numerical Modeling of Previous Testing

A nonlinear numerical model of the two-story test was developed in OpenSees (Mazzoni et al. 2009) and is decribed in detail in Wichman et al (2022). The model is shown schematically in Figure 2. The model was 2D, modeling the in-plane response of the rocking walls. The walls were modeled with linear elastic beam-column eleemnts with stiff eleemnts exending to the location of the UFPs, which provided energy dissipation. Zero-length nonlinear springs represented the UFPs and connected the two wall panels togeter. A series of zero-length nonlinear springs were used at the wall base to allow uplift and represent wall the compressive behavior of the CLT. Additional springs were placed in series with those to repsent the deformation of the flanges of the foundation beams.

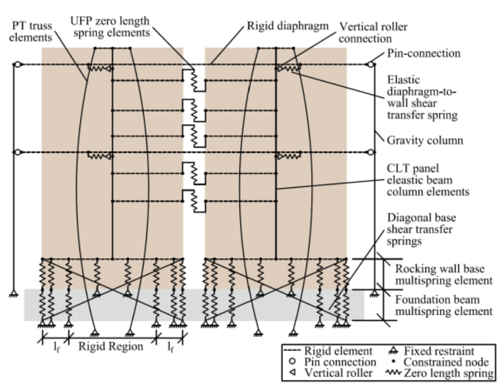

Figure 2. Two-Story Rocking Post-Tensioned CLT Wall Shake Numerical Model

Comparison of Experimental and Numerical Results

Figure 3 shows a comparison of the numeircal model and experimental resutls for three different motions (one at each hazard level) in terms of the time history of roof drift respose. Results are shown for numerical models that include and exclude the springs representing the foundation deformation (Num. Flex and Num. Rigid, respectively). As shown, when foundation deformations are included the model represents the global respsonse well. Figure 4 shows spectral acceleation of the ground motion at the building's fundamental period versus peak roof drift from all lthe gound motions used in the experiments and the corresponding numerical model resutls with and without foundation deformations. Again, when the foundation deformations are included, the specimen response is represented well with the numerical model.

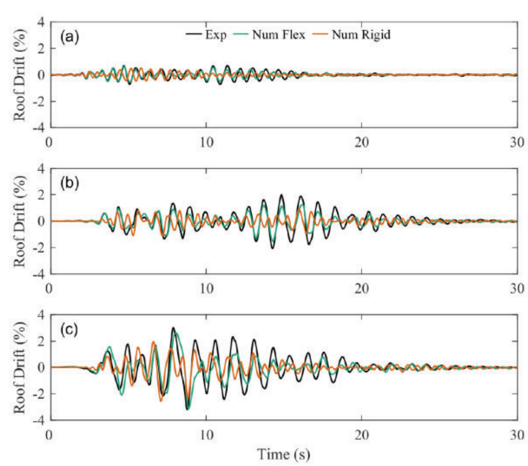

Figure 3. Comparison of the roof drift time history response from experimental results,the flexible foundation numerical model, and the rigid foundation numerical model for (a) SLE (b) DBE, and (c) MCER

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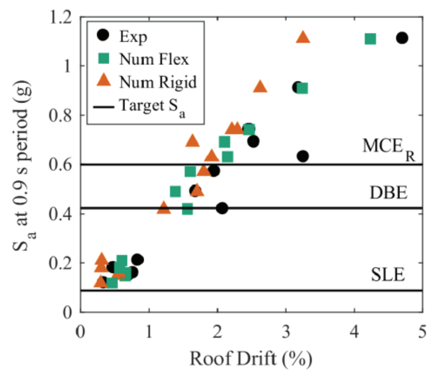

Figure 4. Comparison of Nonlinear Model and Experimental Peak Roof Drift for Two-Story Test for All Ground Motions

3 – 10-STORY BUILDING OVERVIEW

The full-scale 10-story NHERI TallWood test specimen is a fully mass timber building with resilient post-tensioned mass timber rocking walls as the lateral force-resisting system. The specimen is representative of a partial footprint of a building. Figure 5 shows a photo of the test specimen.

Figure 5. Photo of the 10-Story Building Shake Table Specimen

As shown in the 10-story elevation view in Figure 6, the first story is 3.96m (13-feet) and all other stories are 3.35m (11-feet), resulting in a roof elevation of 34.14m (112 feet). Figure 5 also shows a typical floor plan with the locations of the lateral system, beams, and columns called out. While the exact details of each floor diaphragm varied floor to floor, the geometry and general structural layout was the same on all floors. The lateral force-resisting system is symmetric and consists of post-tensioned rocking mass timber shear walls with two lines of resistance in each direction. The walls are composed of CLT in the east-west direction and MPP in the north-south direction. Two different mass timber products were chosen for the walls to study the performance of both materials. Each wall is post-tensioned with external threaded rods that run the full height and are positioned near the center of each panel. At each story, UFPs connect each side of the wall panels to laminated veneer lumber (LVL) boundary columns which support part of the gravity loads. The reminder of the gravity framing system consists of LVL gravity columns and beams. At the base of the walls, a shear transfer mechanism exists to transfer shear forces from the walls into the foundation. A system of steel and concrete elements were bolted to the shake table and serve as the foundation for the structure. A variety of mass timber panel products make up the different floor diaphragm systems. Detailed material properties and design calculations for the lateral system can be found in Wichman (2023). Detailed information on the design of the gravity framing systems can be found in Busch (2023). The building design and experimental perforamnce is also described in Pei et al. (2024).

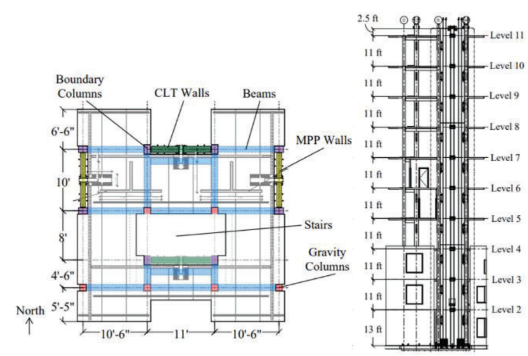

Figure 6. Plan and Elevation of Schematics of the 10-Story Building Shake Table Specimen

The building specimen was designed for a location in the Capitol Hill neighborhood of Seattle, Washington (Coordinates: 47.6156, -122.3197) with site class C soil. Because the specimen was designed to represent a mixed-use building, it was classified as a risk Category II building. This location was chosen because, at the time of this test, the Pacific Northwest is a hot spot for mass timber construction and the Capital Hill neighborhood is a growing area where mid-rise mixed-use buildings are in demand. The site seismic parameters were obtained from the Applied Technology Council (ATC) Hazards by Location tool (ATC, 2016) with reference to ASCE/SEI 7-16.

The building was designed using performance-based seismic design methodologies and nonlinear response history analysis. Thus in accordance with ASCE/SEI 7-16, the design was finalized at MCER, but additional earthquake hazard levels were also selected to assess performance. The earthquake hazard levels identified for this project were as follows:

• 43-year return period (50% in 30 years)

• 225-year return period (20% in 50 years)

• 475-year return period (10% in 50 years)

• 975-year return period (5% in 50 years)

• 2475-year return period (2% in 50 years)

• 4975-year return period (1% in 50 years)

• Site specific Risk-Targeted Maximum Considered Earthquake (MCE_R)

Response spectra for the various hazard levels are shown in Figure 7. For nonlinear analysis, ground motions were selected and scaled to each hazard level. The motions were selected to be consistent with the Seattle hazard and included both crustal and subduction zone earthquake records.

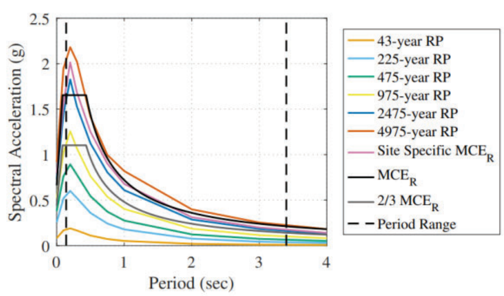

Figure 7. Response Spectra Used for Design and Ground Motion Selection and Scaling for the 10-Story Building

4 – 10-STORY BUILDING NONLINEAR MODEL

The model of the 10-story building was developed in OpenSees [Mazzoni et al., 2009] and includes a detailed representation of the post-tensioned rocking wall lateral system and the lateral connections. It assumes a rigid diaphragm and includes the boundary and gravity columns. The building was designed such that the response is controlled by the post-tensioned rocking walls and contributions from the stairs and nonstructural elements are minimal. Thus, they were not included in this model. While this model focused primarily on the lateral system and its design, force and deformation demands results were also used to inform and aid in the design of other key building components including the diaphragms, stairs, and nonstructural walls. This model uses 2% Rayleigh damping on the 1st and 9th mode and uses the elements' current stiffness matrix. The modelling methodologies presented in this section built upon two-dimensional techniques presented above and in Ganey (2015), Ganey et al. (2017) and Wichman et al. (2022) which validated the techniques using cyclic and dynamic loading tests, respectively. This model was not developed to study the response of vertical input motions.

A schematic of the post-tensioned rocking wall OpenSees model is shown in Figure 8. With these lateral systems, the walls remain elastic up the height of the wall and all inelastic behavior of the wall system occurs at the base through uplift and compressive deformations. The elastic portion of the rocking wall is modeled using a series of force beam column elements with a rectangular fiber cross section.

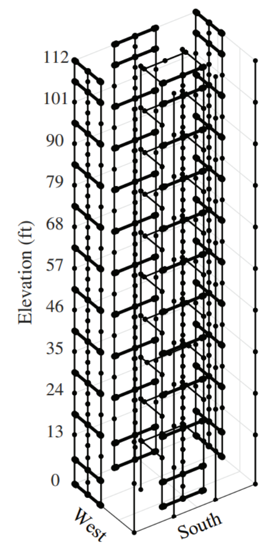

Figure 8. Schematic of the 10-Story Building Nonlinear Building Model

The inelastic compressive deformation of the wall base and the rocking behavior was modeling using a multispring contact element, initially developed by Spieth et al. (2004) for prestressed concrete structures and utilized in two-dimensions by Ganey (2015), Wichman et al. (2022), and others. For the 10-story model, this multispring element methodology was extrapolated to develop a three-dimensional multispring element that can capture rocking in both the in-plane direction and out-of-plane direction. Figure 9 shows selected details of the key model components.

Other aspects of the model were consistent with those developed to model the two-story test described above except that the foundaiton springs were not included as the the 10-story building used a foundation that did not deform.

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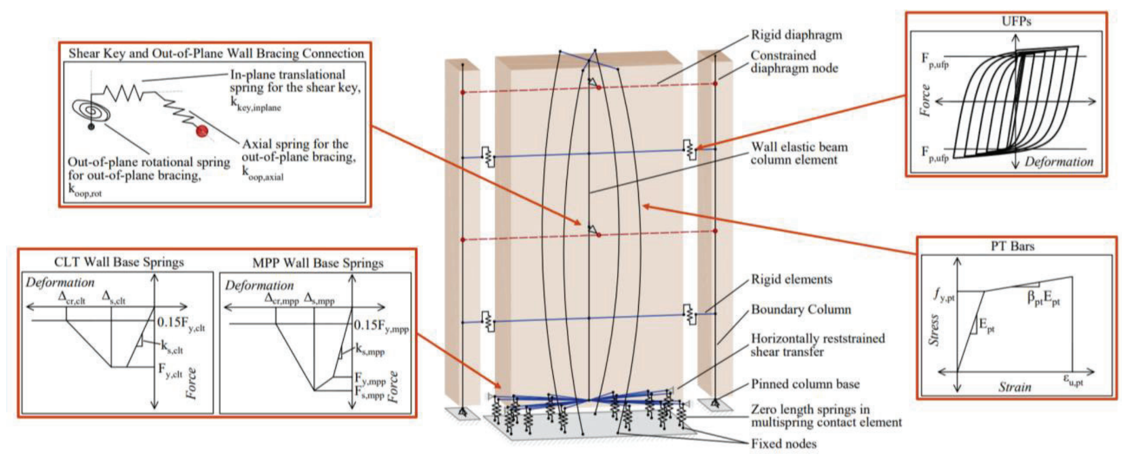

Figure 9. Key Components of Nonlinear Model of the 10-Story Building

5 – COMPARISON OF NONLINEAR MODEL AND EXPERIMENTAL RESULTS FOR THE 10-STORY BUILDING

The original model used for design of the test specimen used 2% damping, applied through a Raliegh damping model. The first stage of somparing the numerical model and experimental reuslts demonstrated that the damping in the test was considerably larger. Figure 10 shows a comparison of the nonlinear model results with the 2% damping and also with 5% damping for the two directions for a ground motion at the MCEr hazard level. The fiugre indicates that 5% damping provides a better estimate of global response. As such, 5% damping is used for the other comparisons. Additional work that will be published later in a journal paper refines the damping further.

Figure 11 shows the story drift profile from the nonlinear model for two different MCEr gound motions versus the test results. For the test results drift are opbtained two ways: (i) using string potentiometers, and (ii) using accelerometer data that is filtered and double integrated. The figure shows that the numerical model is resonably able to predict the distribution of drift over the hieght of the structure. Note that the second gound motion shown was applied only in the EW direction of the building. Due to noise in the accelerometers there is a some dirft shown in the NS direction that is likely not actually there.

Figure 12 shows a comparison of the nonlinear model predicted peak roof drift in the two orthogonal directions of the building versus the experimental results. As shown, the model predicts roof drift reasonably well.

Figure 13 shows the peak story drift predicted from the nonlinear model versus the experimental results. As shown, the experimental peak story drift is larger than predicted from the numerical model for large ground motions. This observation has resulted in refinement of the model to adjust the damping. A future journal publication will detail those changes and provide updated results.

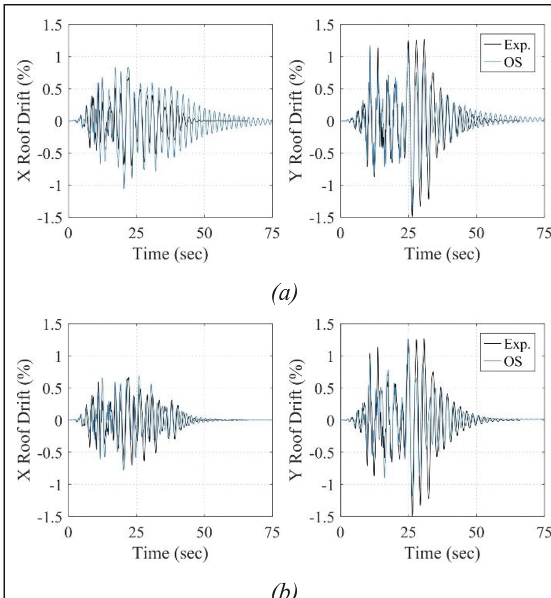

Figure 10. Comparison of 10-Story Numerical Model Roof Drift and Experimental Roof Drift (a) 2% Damping in the Nonlinear Model and (b) 5% Damping in the Nonlinear Model

6 – Conclusions

The NHERI TallWood 10-story test building was designed using performance-based design principles and utilized a nonlinear numerical model that was analysed for site-specific ground motions consistent with current design practices for tall buildings on the west coast of the US. Analyses indicated the building was expected to have only minor damage in even the large MCE_R level ground motions. This was validated by the observed and measured results of shake table testing of the full-scale 10-story building. The experimental results were also used to validate the numerical modeling methodology

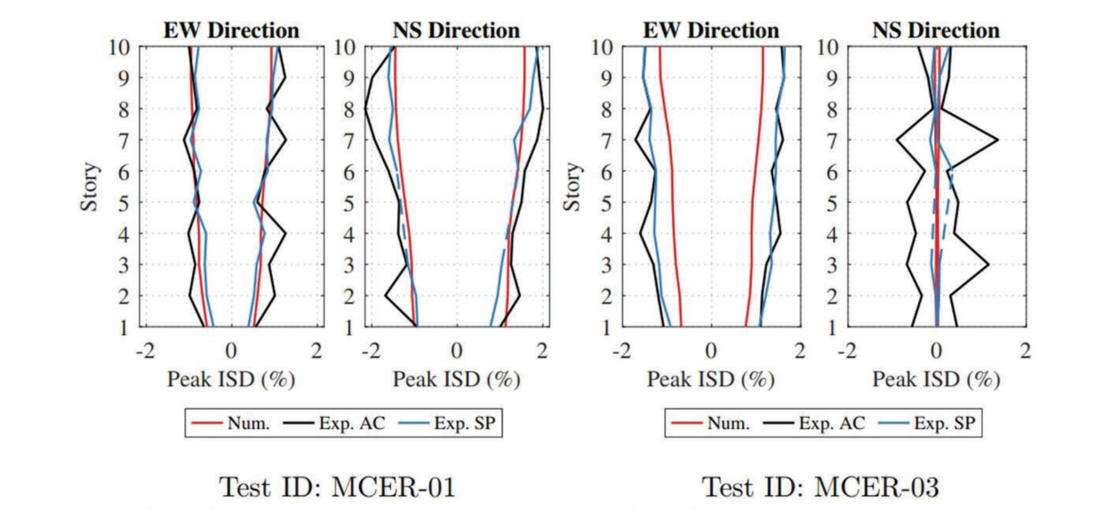

Figure 11. Comparison of Story Drift from Nonlinear Model with Eaprimetnal Results for Two Ground Modtions

and provide confidence of using such models for performance prediction of tall mass timber buildings with rocking mass timber wall systems for seismic load resistance.

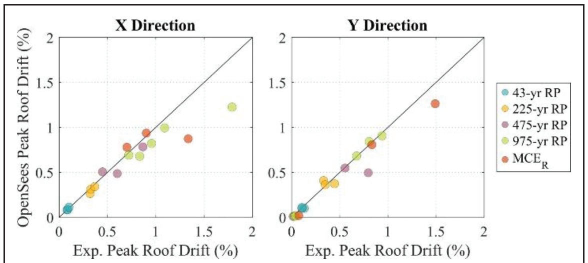

Figure 12. Peak Roof Drift fron Nonlinear Analysis Versus Experimental Results for the 10-Story Building

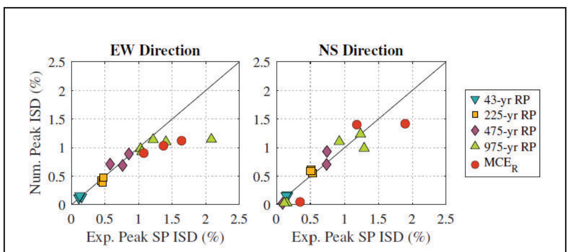

Figure 13. Peak Story Drift fron Nonlinear Analysis Versus Experimental Results for the 10-Story Building

As shown, the nonlinear model was able to predict key performance parameters with good accuracy, especially for engineering design purposes.

7 – Acknowledgements

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8 – References

ASCE (2010). Minimum Design Loads for Buildings and Other Structures. ASCE/SEI 7-10.American Society of Civil Engineering, Reston, VA.

ASCE (2016). Minimum Design Loads for Buildings and Other Structures. ASCE/SEI 7-16. American Society of Civil Engineering, Reston, VA.

Applied Technology Council (ATC) (2016). "Hazards by location, <https://hazards.atcouncil.org>. Accessed on February 19, 2019.

Busch, A. (2023). "Design and construction of tall mass timber buildings with resilient posttensioned mass timber rocking walls." Ph.D. thesis, Colorado School of Mines, Golden, CO.

Ganey, R. S. (2015). "Seismic design and testing of rocking CLT walls." M.S. thesis, University of Washington, Seattle, WA.

Ganey, R., Berman, J., Akbas, T., Loftus, S., Dolan, J. D., Sause, R., Ricles, J., Pei, S., van de Lindt, J., and Blomgren, H.-E. (2017). "Experimental investigation of self-centering cross-laminated timber walls." Journal of Structural Engineering, 143(10), 04017135.

Mazzoni, S., McKenna, F., Scott, M., and Fenves, G. (2009). Open Systems for Earthquake Engineering

https://doi.org/10.52202/080513-0430

Simulation User Command-Language Manual - OpenSees version-2.0. Pacific Earthquake Engineering Research Center, University of California, Berkeley, Berkeley, CA.

Pei, S., van de Lindt, J. W., Barbosa, A. R., Berman, J. W., McDonnell, E., Dolan, J. D., Blomgren, H.-E., Zimmerman, R. B., Huang, D., and Wichman, S. (2019). "Experimental seismic response of a resilient 2-story mass-timber building with post-tensioned rocking walls." Journal of Structural Engineering, 145(11), 04019120.

Spieth, H. A., Carr, A. J., Pampanin, S., Murahidy, A. G., and Mander, J. (2004). "Modeling of precast prestressed concrete frame structures with rocking beam-column connections." Report No. 2004-01, University of Canterbury. Christchurch, New Zealand.

Wichman, S., Berman, J. W., and Pei, S. (2022). "Experimental investigation and numerical modeling of rocking cross laminated timber walls on a flexible foundation." Earthquake Engineering & Structural Dynamics, 51(7), 1697–1717.

Wichman, S. (2018). "Large-scale dynamic testing of rocking cross laminated timber walls." M.S. thesis, University of Washington, Seattle, WA.

Wichman, S. (2023). Seismic Behavior of Tall Rocking Mass Timber Walls, Ph.D Dissertation, University of Washington, Seattle, WA, USA.