From Testing to Codification: Post-tensioned Cross Laminated Timber Rocking Walls

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1 Introduction

Wood buildings in North America have been predominantly constructed using light-framed wood systems since the mid 1900's, with the exception of heavy timber construction in some non-residential applications. This situation is likely to change in the future with the growing acceptance of mass timber construction in the region. In fact, a number of mass timber buildings have been constructed in recent years in the U.S. and Canada, including low- to mid-rise mixed-use buildings (e.g. University of Massachusetts Amherst Student Center, T3 building in Minneapolis, MN) and tall towers (e.g. Brocks Commons at University of British Columbia). Most of these buildings utilized cross laminated timber (CLT) or nail laminated timber (NLT) floors and heavy timber framing systems to support gravity loads, and a non-wood lateral system such as concrete shear walls or steel braced frames to resist wind and seismic loads. Although CLT material and glulam products have been recognized in the U.S. and Canada (IBC (2018) and NBCC (2015)), there is currently no mass timber lateral system in the U.S. that is recognized by the building codes. As a result, special design procedures and review/approval processes must be followed for any building intended to use a mass timber lateral system. At the time of this paper, there has been only limited on-going effort to codify mass timber lateral systems in the U.S., including one project to develop seismic design parameters for panelized CLT shear walls (Amini et al., 2014) following the FEMA P695 procedure. Another lateral system that has attracted significant attention in research is the post-tensioned mass timber rocking wall system, which has potential applicable to balloon framed low- to high-rise wood buildings. This paper will focus on recent research development on mass timber rocking wall system in the U.S., especially with the use of CLT wall panels, and the effort to develop a seismic design procedure for this system for inclusion in the Special Design Provisions for Wind and Seismic (SPDWS)(2015).

2 Post-tensioned Rocking Wall System

The post-tensioned rocking wall concept was first formalized in concrete rocking walls. By applying a post-tensioning force at the center of a precast concrete panel, the panel was able to alternatively rock about each end under lateral loads but would always re-center as long as the post-tensioning strand or rod did not yield. The concrete rocking wall system has been extensively tested (e.g., Lu, et al. 2018 and Marriott, et al., 2008) and applied in real building projects (Suncoast Post-Tension, 2017; SEAOC, 2016). A design procedure for precast rocking walls already exists in the building code in the U.S. (ACI, 2007). However, no such similar design procedure has been codified for mass timber rocking walls. Mechanistically though, the rocking wall concept is applicable to panels made of any solid material, thus giving rise to the idea of a wood-based rocking wall system.

Combining large engineered wood members with post-tensioning techniques, researchers in New Zealand pioneered the development of low-damage and self-centering wood lateral force resisting systems and studied a variation of this concept starting in the early 2000's (Buchanan et al. 2008, Palermo et al. 2005, 2006). These earlier studies experimented with post-tensioning techniques on wood-frame moment connections and LVL walls (Buchanan et al. 2008, Iqbal et al. 2015, Iqbal et al. 2016a, Iqbal et al. 2016b). Some of these systems were used in real building projects in New Zealand (Palermo et al. 2012, Holden et al. 2012). In the U.S., reversed cyclic load tests of a number of full-scale post-tensioned CLT rocking walls was conducted by Ganey et al. (2017). Calibrated modelling parameters for CLT rocking walls were derived from the test data by Akbas et al. (2017). The understanding of rocking timber lateral system from these earlier efforts was applied to the design of a full-scale, two-story building that the authors tested on the UCSD shake table in 2017 as part of the NHERI TallWood Project (Pei et al. 2019). During this test program, the full-scale building with CLT rocking wall system survived 14 seismic excitations with only minimal damage. In fact, the structural system remained damage-free during all design-basis earthquake (DBE) events. Damage was only observed during the maximum considered earthquake (MCE) events. Following this two-story building test, two parallel efforts are currently on-going to further the design of post-tensioned rocking mass timber wall system in the U.S. The first effort is to develop a design methodology for mass timber rocking walls for consideration in SDPWS adoption (funded by U.S. Forest Services). The second effort is the planning (also by the NHERI TallWood Project team) of a full-scale 10-story wood building shake table test with non-structural components in 2021. This large test is aimed at validating the resilience-based design methodology proposed, thus referred to as the NHERI TallWood Validation Test in this paper. It is envisioned that through these research and development efforts, post-tensioned mass timber rocking walls will become a well-validated and accepted lateral option for multi-story mass timber buildings.

3 NHERI TallWood Project: Vision and Current Results

NHERI TallWood Project is a six-university collaborative research project funded by the National Science Foundation (NSF) spanning a period of 5 years from 2016 to 2021. Its ultimate goal is to develop and validate a seismic design methodology for tall wood buildings that incorporates high-performance structural and non-structural systems to achieve a resilience objective following major earthquakes. The project mainly includes four research components, namely the tall wood archetype development, a holistic modelling approach, a resilience-based seismic design methodology, and a final validation test.

3.1 Tall Building Archetypes:

During the first year of the project, the research team collaborated with Lever Architecture and KPFF Consulting Engineers and developed a group of tall wood building archetypes intended for mixed-use applications (residential and commercial). These archetypes are all based on a 100 x 200 ft lot size which is typical for urban areas in the U.S. with population density suitable for this type of buildings. Three building heights, namely 6, 12, and 18 story-buildings, were considered (see Figure 1). All of the archetypes utilized a glulam beam and column gravity system to enable an open floor plan that can be reconfigured to different uses (see Figure 1). Different internal floor plans (including non-structural partition and typical contents arrangement) were also developed for residential, office, and commercial usage. These non-structural components will play a major role in building resilience assessment and be tested in the final full-scale validation test.

3.2 Investigative testing of a two-story mass timber building

In order to generate building system level dynamic response data for model calibration, a 2-story mass timber building was built and tested as an investigative test in 2017. The concept for the 2-story test building specimen is shown in Figure 2, with an open floor plan. The building had a relatively high aspect ratio diaphragm via diaphragm cantilevering to specifically study the lateral responses of the CLT diaphragm. Two sets of coupled CLT rocking walls were inserted into the diaphragm and connected using shear-transfer-only slotted connections (see Figure 3). In order to accommodate the expected large inter-story drift, the gravity framing connections

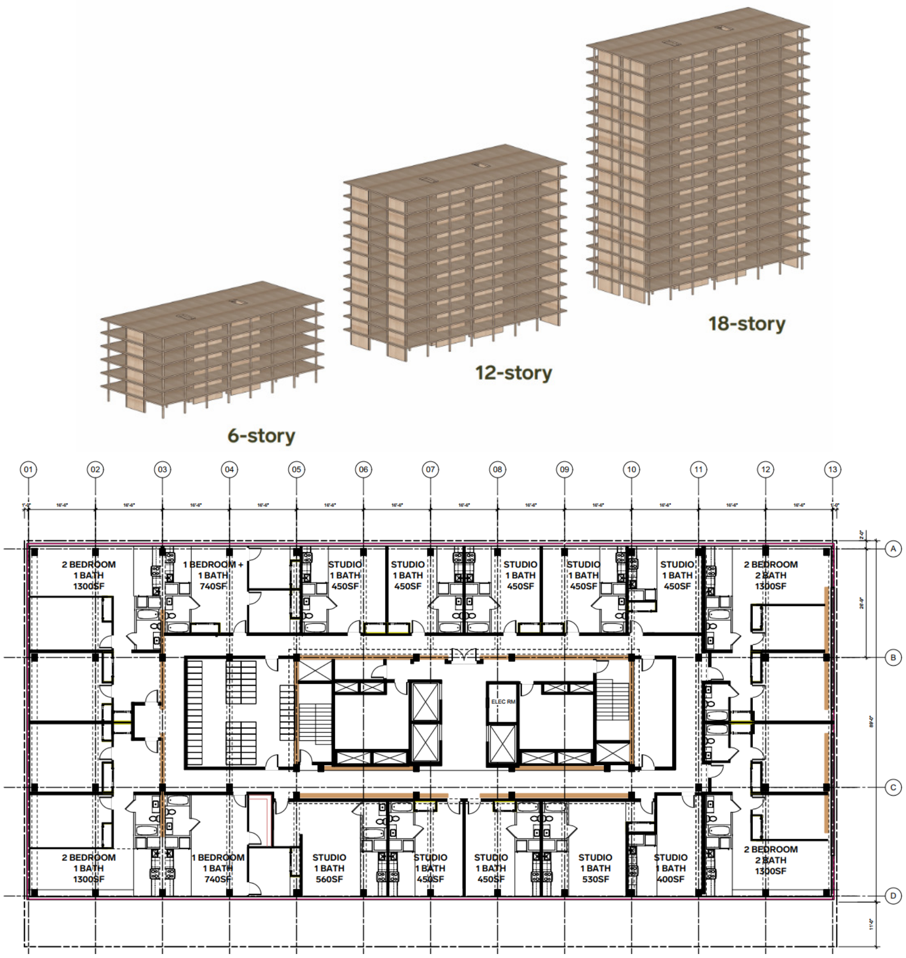
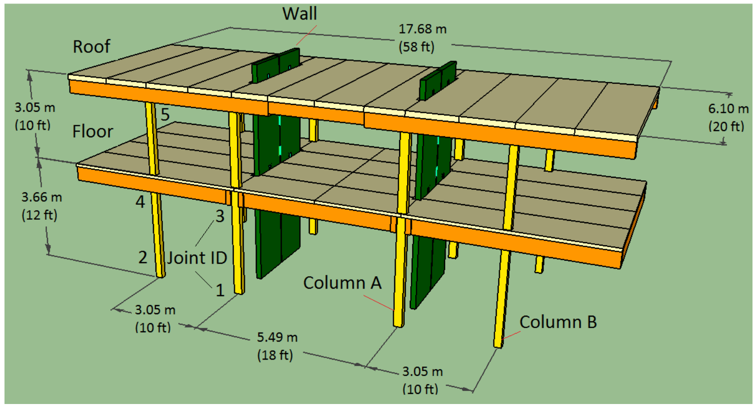


Figure 1: Archetypes and example floor plan for NHERI Tallwood Project

were designed to rock at the beam-to-column interface (i.e. the gravity frame was not part of lateral force-resisting system). The overall seismic mass of the frame and CLT floor/roof was approximately 321 kg/m^2 (64 psf) for the floor and 386kg/m^2 (79 psf) for the roof. The CLT panel joints were constructed following typical CLT top spline floor splice details with pre-routed panel edges covered with plywood strips. The floor and roof diaphragms were designed to remain elastic under the planned seismic excitations. A shear demand calculation was conducted to determine the number of structural screws (Simpson Strong-Tie SDS screws) needed for shear transfer across the panel splices. The chord tensile forces in the diaphragm were carried over panel joints using custom sized metal plates installed with screws.

The CLT rocking walls were coupled using steel U-shaped flexural plate (UFP) energy dissipaters. Similar energy dissipaters have been used in concrete rocking wall systems (Priestley et al. 1999, Johnston et al. 2014), as well as the CLT rocking walls tested by Ganey et al. (2017). The walls and UFPs were initially sized using approximate demands calculated using ASCE 7-10 (ASCE 2010) for a Class B soil site in San Francisco with an assumed seismic force reduction factor, R, of 6. Detailed description of the design configuration of the test building can be found in Pei et al. (2019).

The installation of the test building was completed in two weeks, and the major construction stages illustrated in Figure 4.



(a)

(b)

Figure 2: Investigative testing of a full-scale two-story mass timber building with rocking walls. (a) schematic drawing showing dimensions, (b) specimen completed and ready for testing.

Figure 3: Post-tensioned CLT rocking walls coupled with UFP connectors and attached to the CLT diaphragm with shear-transfer-only connectors (viewed from the 2nd floor)

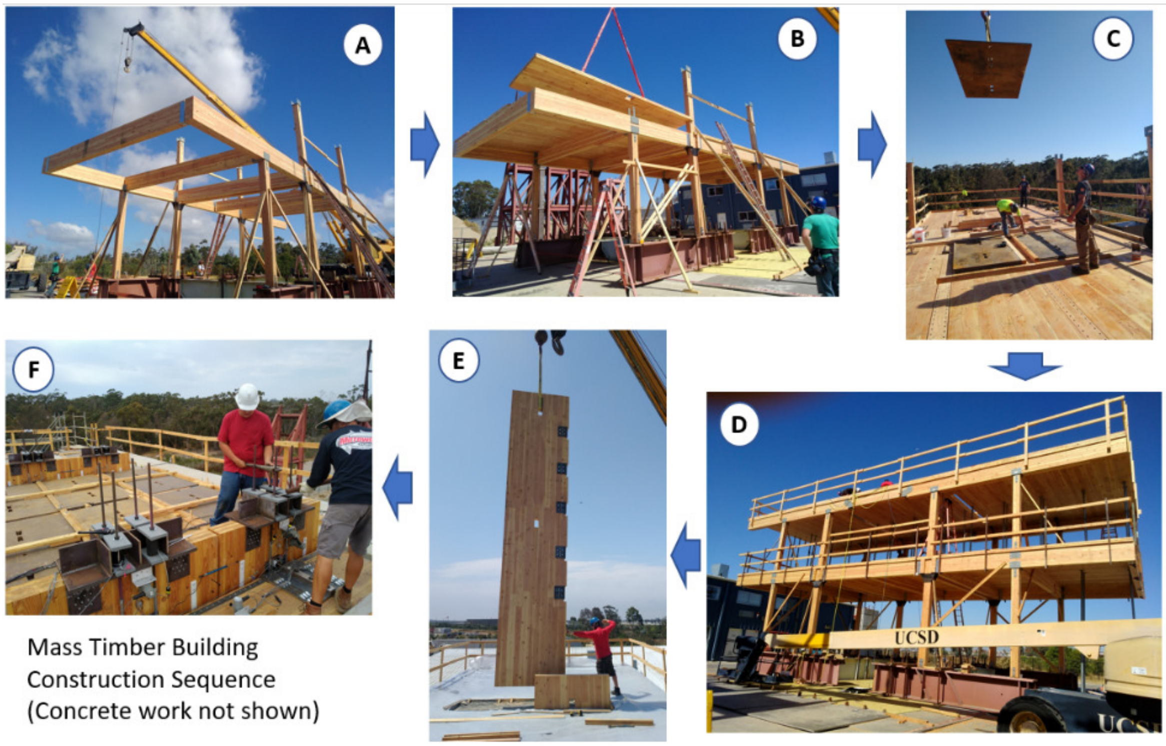


Figure 4: Construction process of the two-story test building. (A) installation of the gravity force resisting system, (B) instalation of the CLT diaphragms, (C) Installation of Intertial mass, (D) Completed gravity system with floors, (E) Installation of Rocking Shear Walls, (F) Post-tensioning of rocking wall.

A total of fourteen (14) seismic tests with different historical records and various intensity levels were conducted with over 350 channels of data measurement installed on the building. A variety of sensors were installed on the test building, providing measurement of force, displacement, strain, and acceleration. The 14 earthquake excitations were selected to represent three hazard levels: (1) Service Level Earthquake (SLE) (i.e., 50% probability of exceedance in 50 years), (2) Design Basis Earthquake (DBE) (i.e., 10% probability of exceedance in 50 years), and (3) Maximum Considered Earthquake (MCE) (i.e., 2% probability of exceedance in 50 years) for the San Francisco site considered in the design. Ground motion records from historic California earthquakes were used with different scaling factors. During the public tests (Tests 6 and 8) which themselves occurred on different days, the unscaled ground motion from the Northridge earthquake (Canoga Park Station record) was run twice without stopping in between. The objective of such particular tests was to illustrate the ability of the building to withstand multiple consecutive strong earthquakes without the need for repair in between. All ground motions were applied uni-axially in the short direction of the building (i.e., along the direction of the CLT rocking walls). The response spectrum of the measured table ground motions are plotted in Figure 5, which represents the actual seismic excitation experienced by the test building.

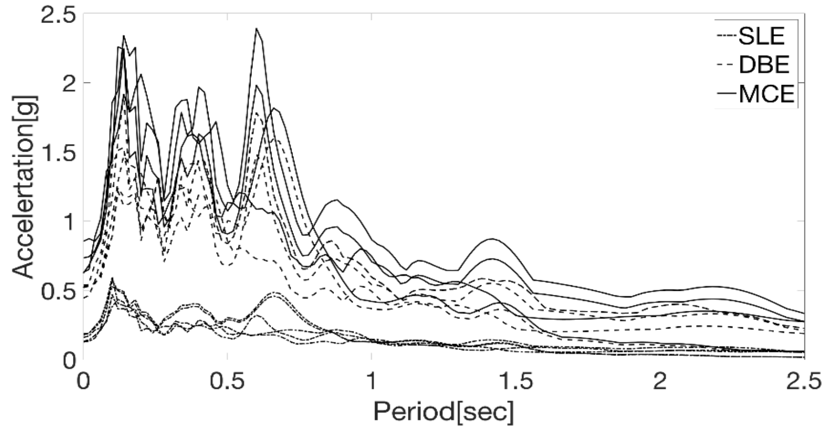


Figure 5: Response spectra of all 14 shake table test inputs

The peak building displacements at the roof and floor levels (relative to the shake table) were presented in Figure 6. One can see that the maximum roof response among all seismic tests was about 350 mm (14 in), which corresponds to approximately 5% overall building drift ratio. Throughout these tests, the building had negligible residual deformations at all levels of shaking. Test 14 was conducted using a ground motion scaled beyond the MCE level intensity with the intent to induce yielding in the PT bars.

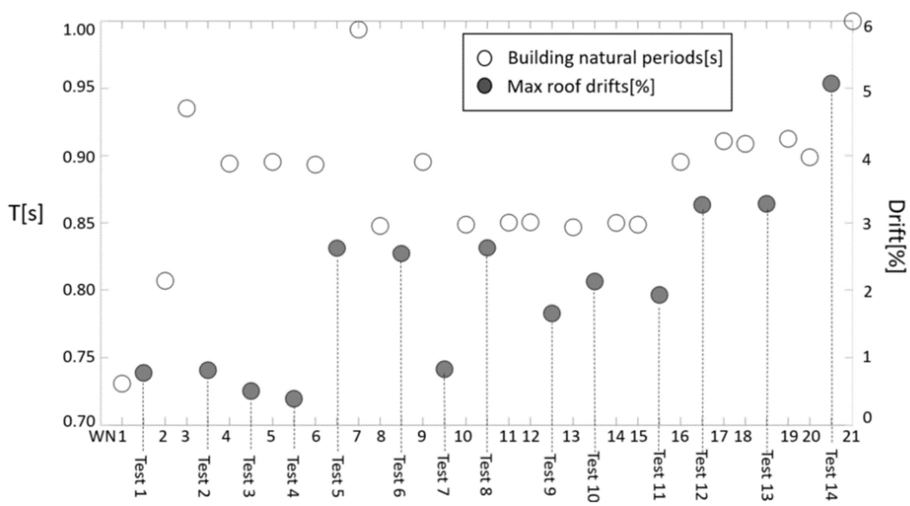


Figure 6: Maximum building responses and natural periods

The PT forces where monitored during all tests using load cells. The maximum, minimum, and residual (RES) PT forces during each test are plotted in the upper left plot in Figure 7. As examples for different intensity levels, time-history plots of selected PT bars are also shown. For some PT bars, the post-tension loss was significant during large earthquakes. The MCE-plus test (Test 14) resulted in tension force loss of about 37 kN (8.4 kips), so the residual force was approximately 16 kN (3.6 kips) measured following the test, (the initial PT force was 53.4 kN (12 kips)) This was mainly caused by the yielding of the bar itself (i.e. the maximum PT force recorded by the load cell was about 150 kN (33.7 kips), while the theoretical yielding force of the PT bar is only

134 kN(30.1 kips)). Tension force loss was found for only a few PT bars, while most did not experience significant loss. Thus, the building was able to re-center with negligible residual drift even for these large events (with partial PT yielding).

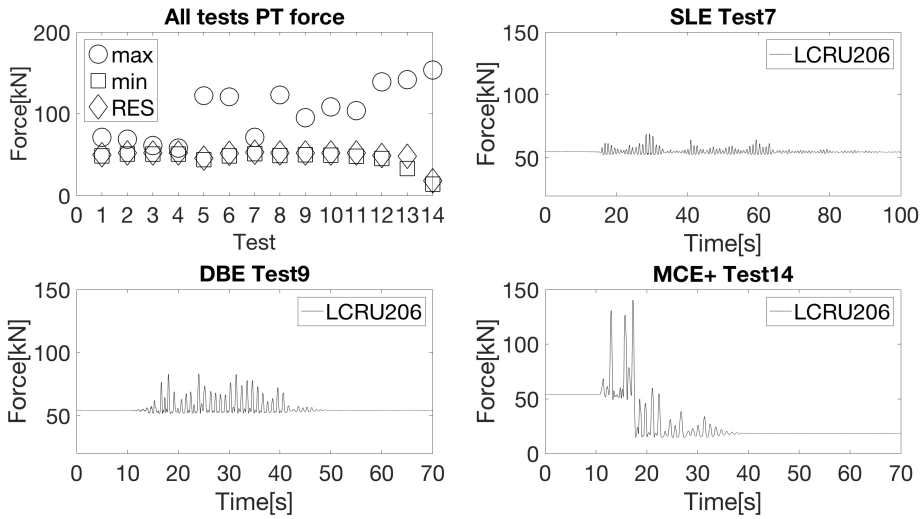


Figure 7: Peak PT forces and example PT time history during tests

The diaphragms were designed to remain elastic during most of the ground motions and very limited deformation (maximum measured deflection, at roof diaphragm in Test 14, was only 24 mm (0.94 inch) over a 17.7m (58 ft) diaphragm span) occurred during the tests.

The specimen was inspected for damage after each test at DBE or MCE level. The structural system was designed to achieve a resilient performance objective, and there was no significant damage to the building at any time during the entire testing program. The only visible damage was found at the bottom corners of the rocking wall panels after large DBE and MCE ground motions. The damage was relatively minor (e.g., splitting of the outside wood fiber and slight deformation of the toe, see

Slight compression deformation at the rocking wall corner

Chipping of wood at the rocking wall corner

Figure 8: Minimal structural member damage observed after 14 earthquake tests

Figure 8) and did not warrant structural repair. It should also be noted that the building was intentionally pushed to very large drift levels, beyond what is required by current building codes. At the code-specified drift levels, the structural system was essentially damage-free.

3.3 Holistic performance modelling and RBSD:

A number of numerical models have been developed in order to simulate seismic response of wood buildings with a CLT rocking wall system. These models include linear finite element models constructed using commercial software packages such as ETABS (CSI, 2018) for preliminary design and demand calculations, detailed nonlinear dynamic models built in OpenSees that utilized fiber elements, nonlinear spring elements, and contact elements, and simplified nonlinear mechanistic models with limited degrees of freedom that are designed for reliability simulations. Some of these models are currently still under development, but many have already been validated using full-scale test data. An example is the simple lumped mass rocking wall model which simulates the rocking wall as an elastic beam with a nonlinear rotational spring foundation and which was shown to accurately estimate the response of the 2-story test (see Figure 9).

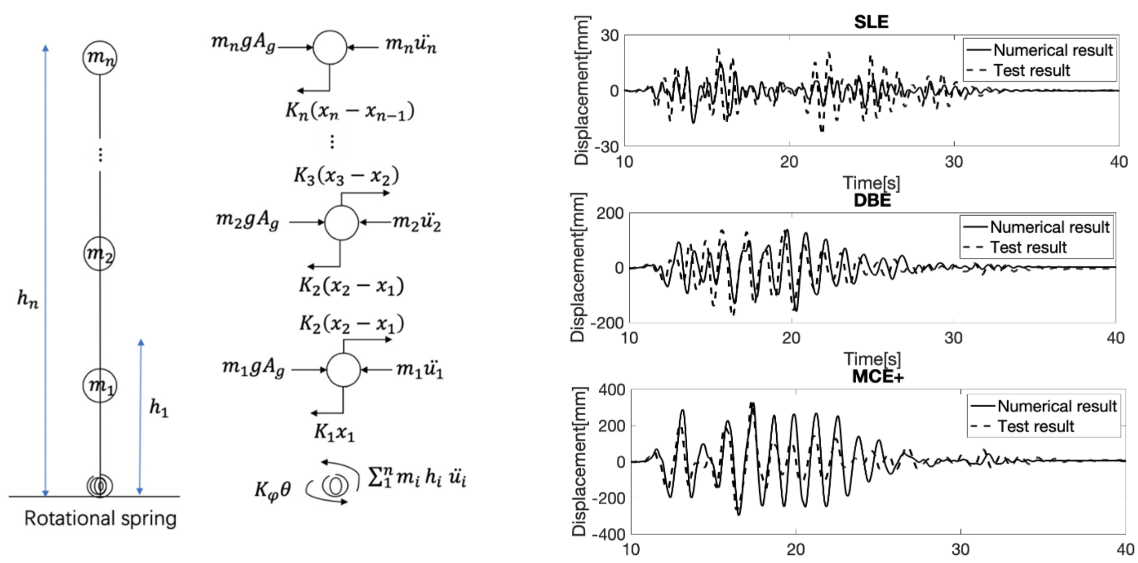


Figure 9: One of the numerical prediction models used by the project team

Once a set of satisfactory simulation models for mechanical responses of the building are obtained, a fragility-based approach will be used to produce building damage and recovery status using engineering demand parameters such as inter-story drift and floor acceleration. These metrics will further be integrated into the down-time of the building after an earthquake. An iterative process will then be followed to design a tall wood building given a target resilience level. Finally, this procedure will be formalized into a resilience-based seismic design approach for tall wood buildings. These research tasks are currently on-going and expected to be used for the design of the 10-story validation test structure.

3.4 Ten-story Validation Test

While tests result from the two-story building validated the structural robustness of the proposed mass timber system, the resilience of tall wood buildings is closely tied with the damage to non-structural components within the building. Additionally, the dynamic responses of a tall rocking wall may include significant higher-mode interactions that could increase structural demands (mostly force demands on wood components and connections). In order to truly validate the numerical model and design methodology developed in this study, a full-scale, 10-story building will be tested at the end of the project in 2021. The test will, similar to the 2-story, be conducted at UCSD's large outdoor shake table. Currently, the gravity design and floor plan layout for the 10-story test building is completed considering the geometric limitations of the shake table. The gravity design considered a Dead Load of 3.3 kN/m^2 (70 lb/sq.ft.) for all floors and roof, plus a Live Load of 3.1 kN/m^2 (65 lb/sq.ft.) for all floors. All columns and beams were designed with sacrificial layers ensuring 2-hour fire rating (@ 4 cm/hr (1.6 inch/hr) char rate on all exposed surfaces) in order to truly represent the practical condition (i.e., exposed) where those members would be used in an actual building. A 4.0 m (13 ft) floor height was assigned to the bottom floor while other floors use a 3.4 m (11 ft) height. Allowable Stress Design provision from NDS 2015 were used to consider the member residual strengths after fire. The resulting test building configuration is shown in Figure 10 on the shake table. A more detailed floor plan for beam-column grid on the shake table is shown in Figure 11.

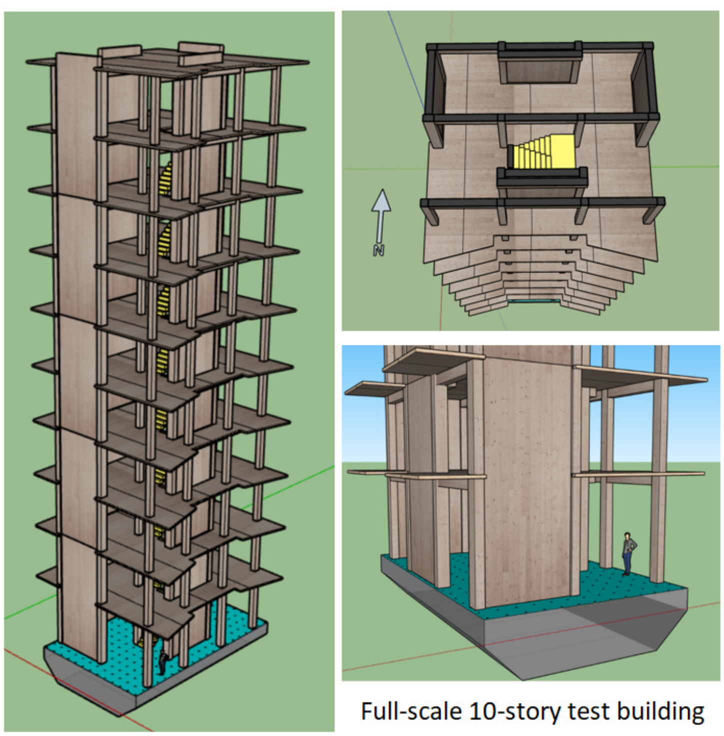


Figure 10: 3D model of the planned 10-story test building on the UCSD shake table

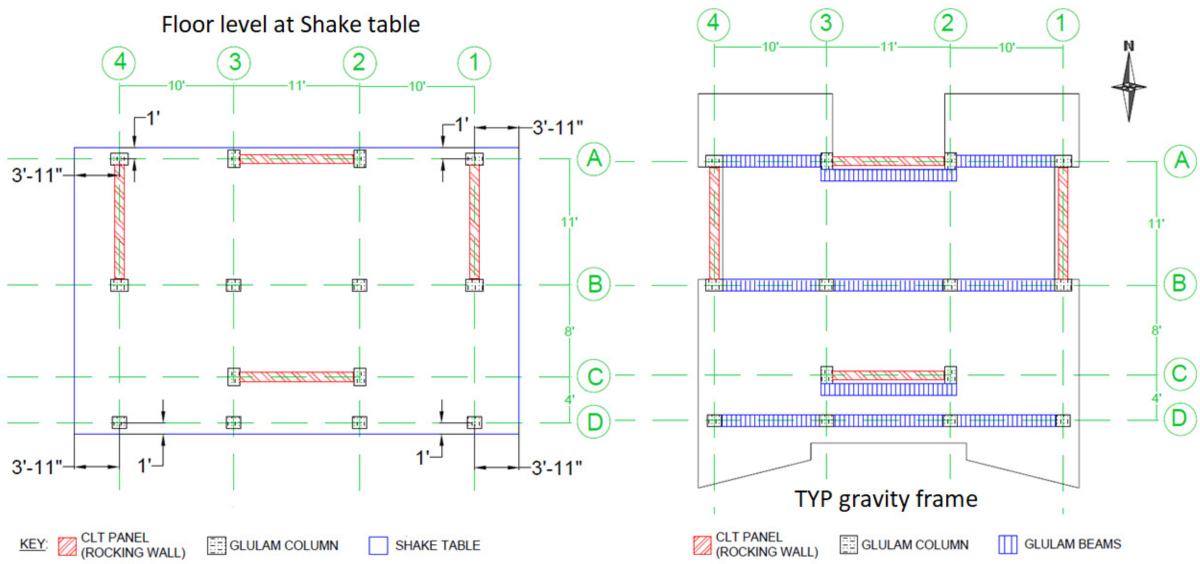


Figure 11: Floor plan of the gravity framing relative to the shake table.

The lateral design of the test building is currently being completed, together with the design of building envelop and interior non-structural configurations. The research team is working to secure material and funding support for the test through industry and academic partners. The building floor plan and dimensions are published on the project website: http://nheritallwood.mines.edu/ to serve as an informational basis for potential donors (of building products) and collaborators (for pay-load testing ideas). The project team is scheduled to complete the building design by early 2020, finalize construction documents by mid-2020, and start construction early 2021. The UCSD shake table is also being upgraded to enable 3D seismic excitation capacity. The upgrade is scheduled to be completed by March 2021. The test building will be constructed on the shake table afterwards. This validation test program is scheduled to be completed by the end of 2021.

4 Design Guide and Provisions for Post-Tensioned CLT Rocking Walls

As the NHERI TallWood Project focuses on developing "above-code" resilience design methodology, another on-going project was funded by the U.S. Forest Services to develop a design guide for post-tensioned mass timber rocking walls within the ASCE 7 and SDPWS framework. This project is led by a research team at Colorado School of Mines and KPFF Consulting Engineers. Through collaboration with AWC and FPInnovations, the ultimate deliverables from the project include a design code provision package (including commentary and examples) that can potentially be considered for adoption into the SPDWS.

The proposed design procedure for post-tensioned mass timber rocking walls follows closely the design provisions already codified for post-tensioned rocking concrete walls (ACI, 2007). For demand calculations, the design procedure does not require nonlinear response history analysis but rather is suitable with either the equivalent lateral force or modal response spectrum procedures of ASCE 7 (ASCE, 2016). Where nonlinear response history analysis is pursued, however, the design procedure also permits its use. The equivalent lateral force and modal response spectrum procedures in ASCE 7 rely on what is known as the response modification coefficient, R (i.e., a ratio of inelastic to elastic force demands). Since a response modification coefficient for mass timber rocking walls has not yet been established through a formal FEMA P695 (FEMA, 2009) study, which would be necessary for final code adoption in the U.S., the design procedure provides commentary suggesting that a R equal to 6 is likely appropriate. This stems from consideration of the similarity in behaviour between a rocking concrete and a rocking mass timber wall. In other words, a rocking mass timber wall can be seen as an emulative system to a rocking concrete wall to support the use of the same response modification coefficient. Rocking concrete walls are given a R equal to 6 in ASCE 7 as special reinforced concrete walls.

For capacity calculations, the design procedure outlines a set of provisions which establish the capacity of the rocking mechanism for each wall or, where coupling of walls is used such as in the 2-story NHERI test specimen, line of coupled walls. Once the capacity of the rocking mechanism has been established and shown to exceed the flexural rocking demand, all remaining components of the lateral force-resisting system are force-protected using capacity design principles. For example, the shear strength of a mass timber wall is compared to the shear corresponding to the probable flexural rocking capacity of that wall, considering both material overstrength and higher mode amplification of shear.

Additional provisions require checking that stresses and strains in the post-tensioning and energy dissipation elements are within those specified by other material standards (e.g., the American Institute of Steel Construction for steel elements). A chapter in the provisions also provides criteria to ensure sufficient initial post-tensioning force to achieve re-centering, sufficient energy dissipation capacity for energy absorption, and that wall configurations meet prescriptive limits.

The project was initiated at the end of 2018 and t a first draft of the design guide for the rocking wall system has been written. The research team is currently working through design examples and the final design provisions, commentary, and examples will be complete by the end of 2020.

5 What the future holds

In summary, a combination of experimental, analytical, and developmental work has been conducted to date in the U.S. to better understand the dynamic behaviour of post-tensioned CLT rocking walls and their potential incorporation into the design code. This body of work was built upon earlier pioneering studies of post-tensioned low-damage concrete and wood systems originating in New Zealand. Although most of the work discussed in this article is still on-going, intermediate results and validations have shown great promises. With the scheduled completion of the mass timber rocking wall design guidelines and the validation of the resilience-based seismic design methodology through full-scale testing, it is envisioned that post-tensioned rocking mass timber walls will play an important role in the expansion of mass timber construction in regions with high seismicity around the world. Once the technical know-how and public perception gaps are filled through research and demonstration, this easy-to-design/install and highly resilient lateral system will provide cost competitive solutions for both new construction and structural retrofit. Tall wood buildings will become one of the integrated components of future earthquake resilient cities world-wide in the next a few decades.

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