Shake Table Testing of a 10-story Mass Timber Building with Nonstructural Components

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ABSTRACT

To advance the wood products market, new design solutions for tall wood buildings using mass timber products are being developed. In particular, post-tensioned cross-laminated timber (CLT) rocking walls have been proposed as a seismic resilient lateral system. To advance the seismically resilient CLT rocking walls for tall buildings, a comprehensive shake table test of a 10-story building with CLT rocking walls is planned for 2021 on the NHERI@UC San Diego outdoor shaking table. An essential aspect of building resilience is assurance that nonstructural components (NCSs) sustain minimal damage or are easily repairable; however, a number of aspects specific to rocking wall systems and their impact on NCSs are not well understood. This test is the culmination of the NHERI TallWood Project that aims to develop seismic design methodology for resilient timber buildings. An essential aspect of building resilience is assurance that NCSs sustain minimal damage or are easily repairable; however, a number of aspects specific to rocking wall systems and their impact on NCSs are not well understood.

In this paper, preliminary plans for a seismic shake table test of a 10-story mass timber building are described, and a vision for incorporating NCSs into the test program is presented. Due to the lack of shake table test data of exterior skin systems with glazing, a key objective is to characterize the response of a variety of curtain wall and light-framed window wall type systems, and to validate industry standard racking tests as predictors of seismic performance. In addition, detailing improvements for stairs, interior partition walls, and suspended ceilings are proposed. The project team is actively seeking industry and academic collaborators for testing of various NCSs to capitalize on this opportunity.

1.0 INTRODUCTION

Typical light frame wood stud walls sheathed with wood structural panels are limited to low or mid-rise construction. To expand the wood products market, industry aims to develop and promote the application of new tall wood buildings built from mass timber products. The greatest challenge to taller wood construction is the development of a reliable lateral load system that can produce predictable response and minimal damage when subjected to strong seismic shaking. Recent research aims to enhance the seismic performance of these buildings through use of a cross-laminated timber (CLT) rocking wall lateral system that accommodates the seismic deformation demands by rocking about the base. These promising CLT systems use post-tensioning rods anchored from the top of the wall to the foundation for restoring force and recentering capability, with external energy dissipation devices to reduce drift demands.

To advance seismically resilient CLT rocking wall systems for tall buildings, a landmark shake table test of a 10-story building with CLT rocking walls is planned for 2021 on the NHERI@UC San Diego outdoor

shaking table. This test is the culmination of the NHERI TallWood Project that aims to develop a seismic design methodology for resilient timber buildings. An essential aspect of building resilience is assurance that nonstructural components (NCSs) sustain minimal damage or are easily repairable; however, a number of aspects specific to rocking wall systems and their impact on NCSs are not well understood. The NHERI@UC San Diego shake table facility is in the process of being upgraded to allow 6-degree-of-freedom (DOF) input motions. The planned CLT tall building test program in 2021 will be the first tall building tested on the newly upgraded facility.

Uniquely, the 10-story building specimen will incorporate a variety of NCSs, such as facades and egress (stairs). Architects and engineers envision mass timber buildings to be clad with lightweight façades, such as curtain wall, storefront, and light-framed window wall type systems. Glass facades have not been evaluated under rigorous shake table testing. The inclusion of several different systems through industry stakeholder participation is envisioned. The height of the test building allows testing of stairs with a variety of connection details, including expansion joints that better accommodate floor-to-floor interstory movement. Additional NCSs will be strategically located in the building so that they are subjected to variable demands; for example, NCSs at lower levels are subjected to larger drifts while NCSs at upper levels are subjected to larger accelerations. The objectives of this paper are to discuss the basic configuration and preliminary gravity and lateral design of the 10-story building specimen, explain the test objectives and test plan, and present a vision for the inclusion of NCSs. The project presents an opportunity for collaboration with other academic and industry partners.

2.0 DESCRIPTION OF 10-STORY TEST BUILDING

2.1 Configuration

The preliminary design of the 10-story building was completed by the second author in collaboration with structural engineers and architects from KPFF, Holmes Structures, and Lever Architecture. Several considerations dictated the plan configuration of the 10-story test building. First, the structural base is required to sit completely on the shake table with its 25' x 40' footprint, to avoid the need to build an outrigger platform (Fig. 1(a)). However, to maximize the floor plan area, cantilevered floor slabs are utilized on all floors above the base (Fig. 1(b)). Story heights are assumed as 13' for the first story and 11' for all other floors.

The inclusion of realistic architectural layouts was a high priority. Based on the fact that real buildings are rarely designed to be perfectly rectangular, the plan incorporates some insets and angled walls. (Fig. 1(b)). Further to this goal, beams, columns and walls are arranged based on a vision for the building architectural layout with mixed-use occupancy. As frequently seen in practice, usable floor space surrounds a centrally located stair core. Figure 2 illustrates a concept for how the test building floor plan might be used on an office floor (Floors 3-6, Fig. 2(a)) and a residential floor with 2 units (Floors 7-9, Fig. 2(b)). Floors 1 and 2 are envisioned as open lobby, while Floor 10 is envisioned as a penthouse unit.

The main lateral system is composed of two post-tensioned CLT rocking walls in each direction. The central stair core is utilized for the placement of one of the walls. The other three walls are located on the building perimeter, which increases torsional resistance, and allows for the rocking walls to be part of the building architecture and on display from the street level. The wall placement then determined the placement of the gravity system composed of glulam beams and columns. All rocking walls have boundary columns on each side; thus columns are spaced 11' o.c. when used as rocking walls. The remaining column spacing is
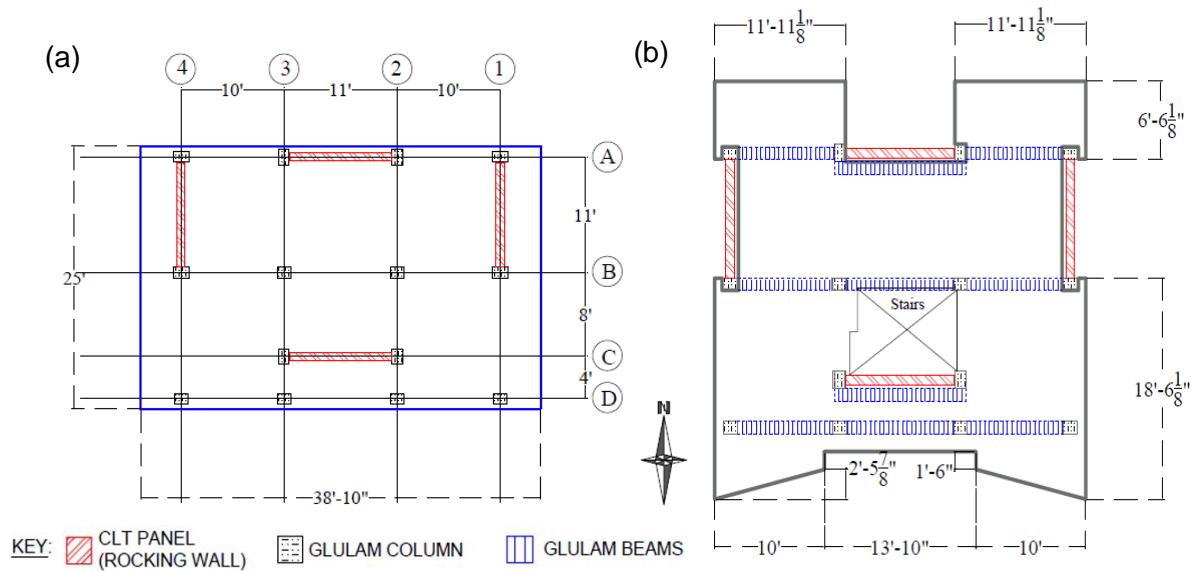
Figure 1. Structural plan layout of 10-story test building: (a) base level on shake table (table outline in blue), (b) typical floor plan (floor panel edge boundary in gray).
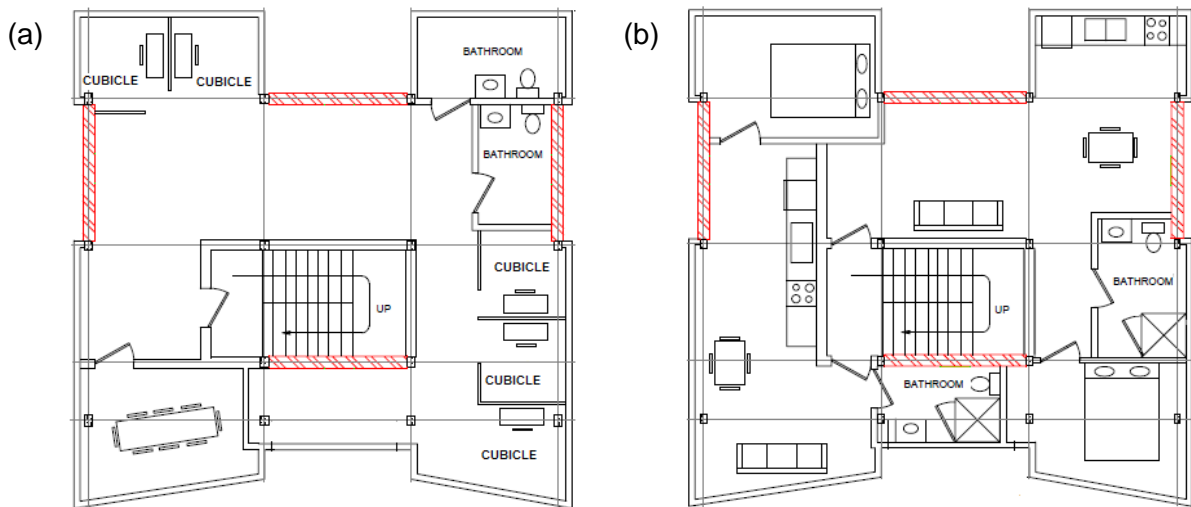

Figure 2. Representative architectural floor plan layout for (a) lower story office occupancy, and (b) upper story residential occupancy.

dictated by the geometric constraints (Fig. 1(a)). Glulam beams are utilized in the E-W direction only, between all columns except offset as needed at rocking wall locations (Fig. 1(b)). A 3D rendering of the structural system is shown in Fig. 3.

2.2 Gravity Design

The gravity system was designed according to the National Design Specification for Wood Construction (AWC, 2015), using Allowable Stress Design. The gravity members are designed for 60 psf dead load for all floors and 50 psf live load for office occupancy or 40 psf live load for residential occupancy, plus 15 psf partition load, for all floors excluding the roof. The seismic mass to be applied during testing is 70 psf,

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which will be achieved by adding steel plates. In addition, all columns and beams are designed with sacrificial layers ensuring 2 hour fire rating (@ 3.6 inch/hr char rate on all exposed surfaces).
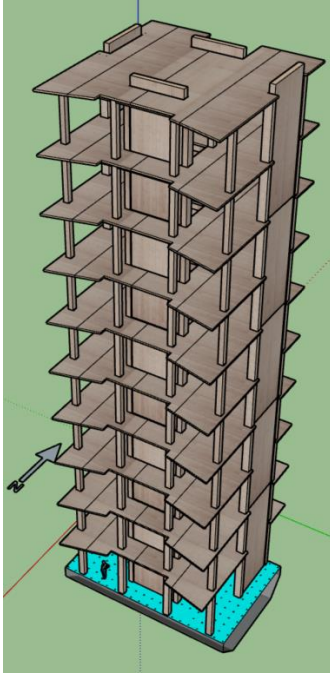

Figure 3. 3D rendering of 10-story building structural system

Beams and columns are assumed to be glulam Western Species 24F-1.8E stress class or equivalent. The floor diaphragms are 3 Ply CLT (4.125") PRG320 – compliant panels. The 3-ply floor panels satisfy the requirements for gravity design, but the team is anticipating upgrading to 5-ply to accommodate exterior skins hung from the cantilevered edges. The CLT floor panels will be fabricated in 6 pieces per floor with strategic geometry that will allow them to be moved into place around the columns.

Table 1. Structural system member dimensions

Member	Cross-Section Dimension or Thickness (in)
Columns (Floor 1-2)	12.25 x 15
Columns (Floor 3-6)	12.25 x 13.5
Columns (Floor 7-10)	12.25 x 12
Rocking wall boundary columns (All Floors)	12.25 x 18
Beam (All)	12.25 x 13.5
CLT floor panel (3-ply)	~4-1/8
Rocking wall Panel (9-ply)	~12-3/8

2.3 Lateral Design

As mentioned previously, the lateral resistance is dominated by two post-tensioned CLT rocking walls in each direction. The walls are designed to rock up off the base, while each wall is post-tensioned through the center to provide a restoring force. The post-tensioning rods are designed to resist the moment that develops at the base of the wall. In addition, each wall is encapsulated by boundary columns. The wall and columns are connected by U-shaped flexural plates (UFPs) at every other story that dissipate energy when subjected to relative motion between the wall and the columns. The UFPs are designed to transmit the vertical shear that develops from the relative vertical displacement between the rocking wall and the bounding columns. Sample connection details for post-tensioning rods and UFP dissipaters are shown in Figs. 4(a) and 4(b), respectively. These details were utilized in a prior shake table test of a 2-story building with CLT rocking walls and performed adequately (Pei et al., 2019a). A floor-to-wall tongue connection at the center of the wall is utilized (Fig. 4(c)). Walls are attached to the floor diaphragms by a vertically slotted shear key that allows movement of the wall relative to the diaphragm. This detail, used in the 2-story building test (Pei et al., 2019a) was effective to prevent diaphragm distortion during wall uplift.

The preliminary CLT panel design has been validated by response spectrum analysis using SAP 2000, and based on ASCE 7-16 specifications. A design spectrum and maximum considered earthquake risk-targeted (MCE$_R$) spectrum assuming a location in downtown Seattle are utilized. The seismic weight is estimated assuming an area dead load of 70 psf on each floor that includes the floor diaphragm, and self-weight of the structural framing. For this analysis, the CLT walls are modeled with thin shell elements and CLT E1M5 structural laminated wall panel material. The preliminary design for drift is based on an equivalent linear elastic analysis of a fixed base shear wall subjected to unreduced forces. A strength design check is also
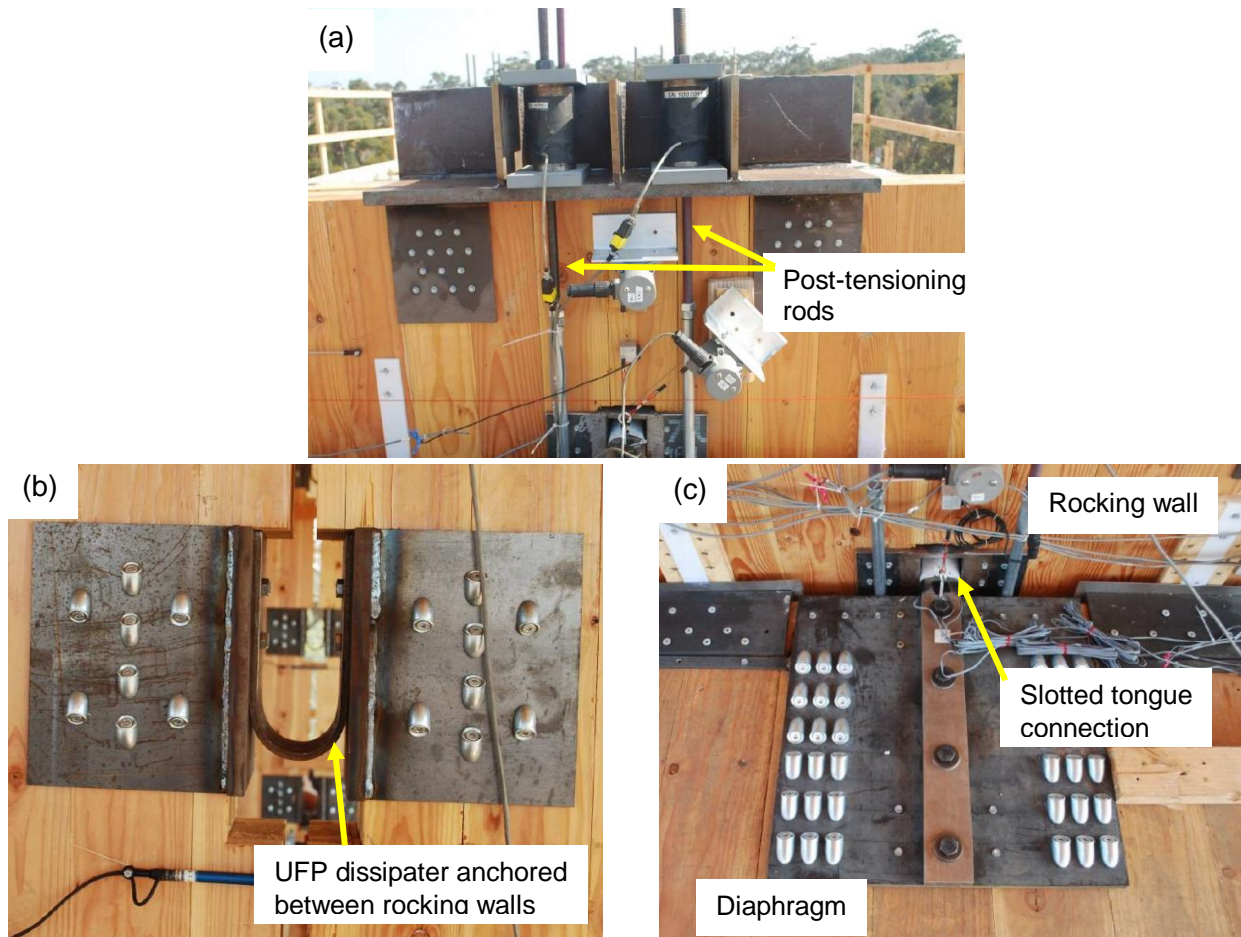
Figure 4. Details of CLT rocking wall lateral system: (a) post-tensioning rod connection at top of wall, (b) U-shaped flexural dissipater between rocking walls, and (c) floor-to-wall slotted connection at wall center. (Pei et al., 2019b).

performed, using the story shear envelope and base overturning moments evaluated via elastic analysis. The forces are calculated based on a design spectrum reduced by $R = 6$, and scaled up to meet the code minimum design base shear coefficient. These preliminary forces will be used in the design of the post-tensioning rods and UFPs, and the design will be refined by inelastic response history analysis.

3.0 OBJECTIVES AND TEST PLAN

As mentioned earlier, the TallWood project seeks to validate the resilience-based seismic design (RBSD) methodology for tall wood buildings using the 10-story test. Although final testing program is still in development over the next year or so, it is envisioned that the building will be constructed with a variety of NCSs, including different facade types and egress (stairs) in particular, and subjected to different levels of seismic ground motions considered in design. The building performance will be evaluated according to the following qualitative performance target metrics.

- The building and its components should remain damage-free for small earthquakes.
- The building and components should see very limited damage during design basis earthquakes (DBE), which is an event with a 475 year return period. The level of damage during DBE events should be limited to minor NCS damage that can be repaired quickly, resulting in little loss of

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functionality after earthquakes. In other words, the minor repair work can be completed without significant interruption to occupants.

- At the MCE level, which is associated with a 2475 year return period event, the building is expected to experience some repairable structural damage (such as yielding of post tensioning bars) and moderate NCS damage, which may correspond to a relatively short downtime for the entire building to be inspected and repaired.

The research team will compare the actual test performance to the intended resilience design targets used in the design of the building. The central idea of the validation is to achieve controllable downtime at different intensity levels through RBSD. Furthermore, the test serves as a great opportunity to expand the database for development of fragilities for NCSs, and to evaluate the drift compatibility and resilience of NCSs that use both traditional and improved details.

The program will also include a number of small intensity tests for building dynamic characterization, followed by a few DBE level tests to demonstrate resilience of the structural system and possibly very limited NCS damage. The research team will collect data along the way to help develop damage and repair fragilities for NCSs installed in the building. Finally the test will be concluded with 1~3 MCE level tests to examine the building performance in rare earthquakes. It is expected that the test program will take advantage of the 3D excitation capacity of the UCSD shake table to be added in 2020.

4.0 VISION FOR NCSs

4.1 Facades

Architects and engineers envision mass timber buildings to be clad with lightweight façades with a high degree of transparency. The architectural vision is to display the rocking walls and much of the structural wood so that the building is perceived as a timber high rise from the street level. Examples of suitable facades are curtain wall and storefront, as well as light-gage steel or timber framed with metal panel skin, and a high percentage of window opening (e.g. window wall system).

The design of the Framework Building in Portland, Oregon exemplifies the architectural vision for tall mass timber buildings (Fig. 5). Framework was conceived as a mixed-use building that combines retail, office space and affordable housing, and utilized CLT rocking walls for lateral resistance. The primary skin is an aluminum composite metal (ACM) with fiberglass windows, but storefront was utilized in the lower lobby space, and curtain wall in the middle of the transverse edge up the elevation of the building displays the rocking wall to viewers from the street (Fig. 5). A shift in ACM panel geometry at mid-height differentiates lower office floors from upper residential floors. Unfortunately, the Framework Building project was cancelled due to permitting and funding issues.

Figure 5. 3D rendering of Framework Building that utilizes several different types of exterior skins. (Source: Lever Architecture, 2016).

4.1.1 Research Needs and Objectives

Buildings in earthquake zones generally absorb the earthquake accelerations by relative movement of the structural frame between the stories; exterior skin systems are also subjected to this interstory movement, and thus must "go along for the ride". Design codes require demonstration (e.g. racking tests of exterior skin mockups) that curtain walls and other glass-infill systems can sustain the design lateral interstory drift without glass fallout. However, such systems are routinely damaged (Evans et al. 1988; Filiatrault et al. 2001; Miranda et al. 2012; Baird et al 2012). For example, glass damage was extensive in the 1994 Northridge Earthquake, despite other NCS damage being limited (Reitherman and Sabol 1995). Notably, rubber gaskets in mechanically captured systems were dislodged and caused a consistent breach in water tightness in buildings (Harter 1994). Recently, following the 2011 Christchurch earthquake, 25% of curtain wall systems were non-operational (Baird et al. 2012). Reconnaissance data does not elucidate why glass damage is so common in spite of the robustness of modern designs. The majority of failures may be in vulnerable systems or affected by boundary details with adjacent systems, however, details of individual failures are not known or well understood.

Industry standard in-plane static and dynamic racking tests of full-size curtain wall and storefront mock-up specimens are used to validate seismic drift capacity. This static test (AAMA 501.4) is the industry standard for serviceability, and specifies a sequence of tests including air and water penetration before and after 3 cycles of static loading (racking) to a target drift level, e.g. 1% (AAMA 2018a). The displacement sustained without glass fallout, or $\Delta_{fallout}$, can be experimentally determined using the AAMA 501.6 dynamic racking protocol (AAMA 2018b). ASCE 7-16 (2016) specifies that for any glazed system, $\Delta_{fallout} \geq 1.25 D_{pl} I_e$, where D_{pl} = design relative story displacement and I_e = importance factor. Given the high rate of glass cracking and failure in the field, the AAMA testing protocol may not represent the seismic response of curtain walls and other glazed systems in field conditions. Seismic performance of curtain walls may be affected by: interaction between in-plane and out-of-plane movement, interaction between panels, flexure or vertical vibration at structural attachment points, high frequency vibrations or diverse frequency content of motion inherent in a multi-modal system, and boundary conditions between different façade types. Large-scale 3D shake table testing offers the opportunity to incorporate some of these more realistic field conditions, but shake table tests of curtain walls and glazed systems are extremely limited (Ma and Yan 1998, Wang et al. 2002, Lu et al. 2010). Thus, a performance comparison of similar specimens tested under the AAMA 501.6 loading protocol and shake table testing is needed. Equally important, ASCE 7-16 does not have a serviceability requirement, but California Building Code Section 2410 (CBSC 2016), used for critical emergency response buildings, requires that structurally glazed systems demonstrate serviceability at the design drift. Confirming that an AAMA 501.6 compliant design can maintain serviceability after a shake table test is important for continued functionality in the proposed new immediate occupancy performance objective (Sattar et al., 2018).

The research objectives related to façade testing in the present program are: (1) Evaluate the seismic performance of glazed facades in the context of the design methodology and performance targets to achieve seismic resilience; (2) evaluate the relative performance of glazed systems with different details, including any proposed detailing improvements, to identify systems best suited to meeting performance targets for seismic resilience; (3) validate AAMA 501.4 and 501.6 racking protocols for system qualification to represent the seismic performance of curtain walls and other glazed systems.

4.1.2 Preliminary Plans

The vision for this project is to clad portions of the building with a variety of different skin systems provided by different industry suppliers, i.e. the "patchwork quilt" approach. Suppliers may select a portion of the

exterior to install their systems. Suppliers are encouraged to put forth innovative details that may improve the seismic performance. As a priority objective is to understand and evaluate the performance of individual systems/details, interactions between varying skin systems will be avoided. Thus, the building will not be fully clad and will not need boundary joints between different skin systems. Based on story height, footprint dimensions and rocking wall locations, the building will have about 10,000 sf of usable exterior skin area. Individual subassemblies on the order of 500–2000 sf are considered desirable. Figure 6 illustrates different possible configurations for individual skin subassemblies that vary from about 200-250 sf per story.
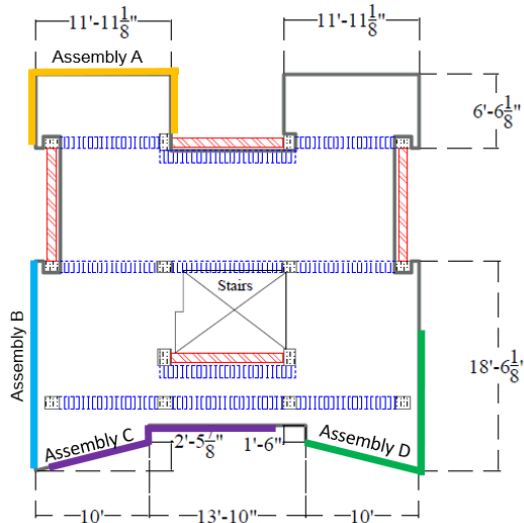

Figure 6. Potential subassembly configurations for testing different skin systems

Table 2 provides a list of general type of skin systems and detailing variations that are desired to be incorporated into the test program. The desired facades can be classified into one of two basic types of systems: glazed curtain wall, and light-frame systems with windows. With regard to curtain walls, stick-built and unitized are constructed distinctly and have the distinct mechanisms (rack versus sway) for accommodating the inter-story drift; thus comparing their performance is of great interest. Among light-frames systems, steel stud framing with metal panel finish are of great interest, with recognition of the myriad of options available on the market for metal panels. One of the unique aspects of this project is assessing the performance of the glass during 3D shaking, including the ability of the system to resist glass cracking or fallout under intermediate and large drift levels. As such, the proposed glass variations with regard to aspect ratio, glass treatment and glass type can be pursued across both curtain walls and light-framing with window openings. The research team is actively pursuing material donations and other in-kind contributions to support investigations of each of these types of promising systems.

The plan sketches (Figs. 1 and 6) show that the CLT floor beams are set back from the plan edge and that floor diaphragms are cantilevered over the edge of the floor beams for the skin subassembly regions denoted in Fig. 6. Thus, the CLT floor diaphragms will be utilized for anchoring the exterior skins. As mentioned in Section 2.2, 5-ply (6.875" thick) CLT is anticipated for this purpose. A top of slab lag bolt anchor is envisioned for hanging the curtain walls. Platform framed systems are easily supported between the floor diaphragms, while a balloon framed system could be supported by installation of perimeter joists.

Table 2. Skins systems and detailing variations desired for the test program

Basic Skin System	Class of Variation	Variations Considered
Storefront	Glazing method	Mechanically captured vs structurally glazed
Curtain wall w/ glazing	Glass aspect ratio	Varied from 1:2 to 2:1
Stick built curtain wall	Glass treatment	Heat strengthened vs. fully annealed
Unitized curtain wall	Glass type	Laminated and insulating glass units (IGU)
Light-framed with windows	Framing style	Balloon vs platform framed
Light gage steel stud framing	Finish material	Metal panel, wood shingle, and stucco
Wood stud framing	Window type	Fixed or operable, variable size
	Window framing	Metal or wood framed
	Glass variations	Same variables as for curtain walls may be applied

4.2 Stairs

In the event of power loss during an earthquake, stair systems serve as the sole means of human egress and ingress. Moreover, the continued function of the stairs during the shake table test program will be critical to support access to all floors of the building to inspect damage after each test. The proposed building layout (Fig. 2) includes one scissor staircase per floor, stacked up on all floors of the building. The architectural vision for the tall mass timber buildings would utilize lightweight prefabricated stairs built from steel, wood, or a hybrid combination of the two for an aesthetic match to the building architecture.

4.2.1   Research Needs and Objectives

Scissor stairs are susceptible to twisting as their structural form rises in elevation. Steel stairs tend to be lighter and more deformable than other commonly used reinforced concrete stairs. Although stairs are assigned an importance factor $I_p = 1.5$, catastrophic behavior has been documented in past earthquakes (e.g. Roha et al. 1982; Bull 2011; Kam and Pampanin 2011; Li and Mosalam 2013). For example, collapse of stair systems in New Zealand during the 2011 Christchurch earthquake precluded occupant safe egress from three high rise buildings in the business district. The most prominent mechanisms that have led to stair failure are described in Bull (2011). Appreciable interstory drift along the strong axis of the stair system may lead to: 1) compression or shortening of a straight stair system resulting in an opening 'knee failure' or 2) extension, which can lead to stairs sliding off their support if insufficient width is provided. Interstory drift transverse to the stair primary axis causes bending and shear in the plane of the stairs, which also has led to significant damage observations in the field. Finally, scissor stairs are especially susceptible to twisting and torsional displacements due to their mid-height landing and high asymmetry. The experience in Christchurch prompted immediate changes to New Zealand design provisions for stairs (SESOC 2011) that require fixing a stair's top connection, and allowing a sliding base connection or sliding at mid-height in the case of scissor stairs, where sliding displacements are to be designed for 1.5 times the ultimate limit displacements. These modern design and detailing guidelines have not been tested to validate their effectiveness.

Numerical studies have been focused on quantifying the influence of stair systems on the overall building response (e.g. Cosenza et al. 2008; Tegos et al. 2013), while coupled numerical analyses involved comparisons of response with and without stairs to elicit the impact of the presence of stairs. Such simulations have been limited to considering elastic response of the stairs, though inelastic response could occur under large earthquakes. A few experimental programs have been conducted on stair subsystems (Higgins 2009; Black et al. 2017). In the latter test program, stairs fixed at one end, and partial freedom at the opposing end were tested and demonstrated promise to minimize drift induced damage. While these tests do offer understanding of the stair subsystem performance, they do not elicit interaction with their supporting structure. However, a few full-scale building tests have incorporated stairs. In a previous large scale building test, prefabricated steel stairs were damaged at interstory drift levels well below the building design performance targets (~0.75%), including weld fracture at flight-support locations (Wang et al. 2015). The design details in these tests were intended to provide ductility, but rather demonstrated inadequate load path continuity precipitated by premature weld failure.

The research objectives related to stairs testing in the present program are: (1) Evaluate the seismic performance of stairs in the context of the design methodology and performance targets to achieve seismic resilience; and (2) further develop and validate approaches to protect stairs from drift-induced damage. Evolving solutions isolate the stairs from the drift demands through a free connection at one floor level.

4.2.2   Preliminary Plans

Donations from industry are sought to incorporate a variety of lightweight stairs built from different materials and with different support conditions. Desired stair support systems utilize wood carriages and stringers or steel stringers. The treads may be finish wood flooring over plywood, steel pan treads with concrete fill, flat plate treads with textured top surfaces, or bar grating treads; paired with wood or steel risers. Open riser configurations are also of interest.

Solutions that are under development for stairs aim to isolate the stairs from the drift demands through a free (slotted or sliding) connection at one floor level. To test these solutions, a fixed connection may be utilized for the stair to top landing connection, while a free or deformation compatible connection may be incorporated at the stair to bottom-landing connection. Thus, it is envisioned that new deformation compatible bottom landing connections (fixed-free) are compared with traditional fixed-fixed connections. These configurations should be dispersed throughout the building to capitalize on the variation of drift and acceleration demands. Table 3 presents a possible selection matrix to combine the different possible variations for the stairs. Fixed-free connections are utilized on the stories with the largest expected drift demands (predicted to be the lowest stories) to minimize damage and preserve building access throughout the test program.

Table 3. Selection matrix for stair detail variations

Story	Stair Support	Tread and Riser	Boundary Conditions
1	steel	steel treads with concrete-fill	fix-free
2	steel	wood	fix-free
3	wood	wood	fix-free
4	steel	steel treads with concrete-fill	fix-fix
5	steel	wood	fix-fix
6	wood	wood	fix-free
7	steel	steel bar grating treads	fix-fix
8	steel	steel flat plate treads	fix-free
9	steel	wood, open riser	fix-fix
10	steel	wood, open riser	fix-free

4.3 Other NCSs

Also of interest to the team are interior partition walls, suspended ceilings, and piping/mechanical equipment. The performance of these NCSs is impacted by their interactions. Unlike stairs and facades, extensive experimental and analytical research has already been conducted on many of these NCSs, such that their performance has been rigorously quantified, and the major vulnerabilities have been identified. Thus, this test is an opportunity to test detailing improvements that have potential to eliminate or reduce some sources of component damage in small to moderate level shaking.

4.3.1   Research Needs and Objectives

Interior partition walls are drift sensitive and incur significant damage at relatively low levels of drift (Retamales et al. 2013). One study (Almufti and Willford 2013) concluded that preventing damage in non-load bearing walls is the key to resilience for low-to-mid seismic hazard levels. Damage-reducing details have been developed, such as including the gaps at wall boundaries, sliding "slip-track" or friction connections, and enhanced strength connections for weaker structural systems (Araya-Letelier and

Miranda, 2012, Retamales et al. 2013, Tasligedik et al. 2013, Swenson et al. 2015). Even walls with the slip track detailing incur damage at intersecting walls, because the attached intersecting wall cannot slip in the out-of-plane direction, or at wall ends when sheathing over the slip-plane. Fire-rated detailing, which is an important design consideration in tall timber buildings, generally requires that exposed steel and timber be covered. This requirement, unfortunately, is at odds with many of the current ideas for resilient nonstructural walls that aim to somewhat isolate the partition wall from the interstory drift as well as accommodate some deformation in the wall through gapping.

Nonstructural walls with improved details were recently tested in bidirectional subassembly tests at Lehigh University. In the first test, a planar wall with double deflection or telescoping slip track (nested top tracks with a vertical gap usually used for accommodating vertical deflection) was compared to an identical wall with conventional slip track detailing (Fig. 7(a) and (b)). In the second test, the response of two C-shaped walls with different gap details were evaluated: a corner gap detail at the intersecting walls, and a distributed expansion gap detail (Figs. 7(c)). The gaps were intended to reduce damage at the wall intersections. While the results are still being analyzed, preliminary observations suggest the following. The telescoping detail improves performance over the traditional slip track, especially at wall ends where the studs and track can become disconnected under deflection. Both the distributed gap and corner gap details delayed the onset of damage to about 1% drift (with the exception of one distributed gap expansion joint that suffered some premature cosmetic damage). With the distributed gap detail, the small gaps directly adjacent to the intersection are the most effective at absorbing relative drift, thus, lengthening these gaps might further delay the onset of damage. One drawback of the gap strategy is the risk that the gaps open completely at large drifts, thus allowing for portions of the wall to separate. This could lead to isolated wall segments being at risk of collapse under large drifts, which could outweigh the gains of delayed damage onset.
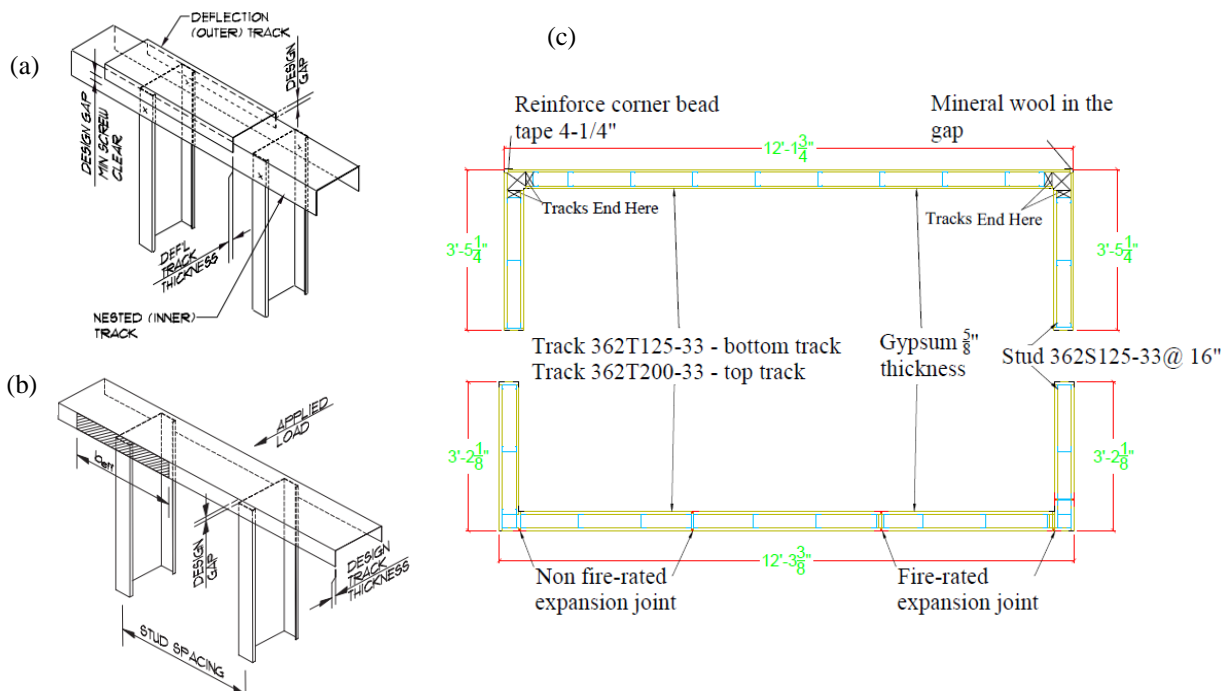

Figure 7. (a) Telescoping connection detail, (b) slip track connection detail, (c) C-shaped wall with corner gaps (top) and distributed gaps (bottom).

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Fire protection of the CLT rocking walls is another issue of importance. Building code regulations will typically require that some CLT walls be sheathed with gypsum wall board (GWB) for fire protection, to limit the total amount of exposed wood. In this case, standard architectural detailing would involve sheathing directly over the wall and boundary columns with three or four layers of GWB. However, this detail would not accommodate wall uplift and rocking, and relative movement between the walls and boundary columns. It is expected that the GWB would tear apart and induce significant damage. The project team has discussed other fire protection detailing, such as introducing a gap with a fire retardant impregnated foam.

Suspended ceilings are frequently used in commercial buildings to provide soundproofing, fire protection, and cover mechanical equipment, wiring and plumbing in the plenum region. Suspended ceilings consist of a horizontal grid system of main tees and cross tees with lay-in tiles, and supported at select locations by vertical hanger wires. In the U.S., suspended ceilings covering more than 1000 sf area between boundary walls are required to have seismic bracing, which consists of compression posts and diagonal splay wires. As typical boundary conditions, the ceiling is fixed to the perimeter walls along two adjacent edges, and floating with a small gap on the opposite two edges.

Typical damage to suspended ceilings is discussed in Soroushian et al. (2016), as determined from 3 major experiments. During large intensity shaking, ceiling panels are routinely dislodged and some panels may fall to the floor. As shaking intensity increases, several grid connections may fail in a localized area causing portions of the ceiling to collapse. This behavior is often instigated by pounding of the ceiling system at the wall boundaries. Pop rivets and seismic clips installed at the boundaries suffer failures at relatively low levels of shaking. In addition, a shake table test at E-Defense led to the conclusion that suspended ceilings with seismic bracing are very susceptible to damage from the vertical component of shaking, because the rigid bracing drives the ceiling grid system to move with the floor above (Soroushian et al. 2015).

To improve the resilience of suspended ceilings, Pourali et al. (2017) have proposed the concept of a floating ceiling. Compared to the conventional fixed-floating boundary, the floating ceiling is installed with a sizable gap between the ceiling edge and perimeter walls on all sides, and no seismic bracing so that the ceiling is a fully suspended system that vibrates as a pendulum, with vibration frequency determined by the length of the hanger wires. A series of tests on floating ceilings were conducted at University of Canterbury (Pourali et al. 2017), and the specimens were subjected to both sinusoidal and earthquake loading. It was shown that maintaining a completely floating ceiling is unrealistic due to large displacements that led to perimeter gap closure, and subsequent significant acceleration amplifications of the ceiling. As a result, another configuration with an acoustic isolation foam placed between the ceiling and the boundary walls was tested. The foam was effective to mitigate the effect of pounding and reduce the seismic displacements, but acceleration amplification actually increased. The concept may have some potential for a resilient ceiling system if a balance can be achieved between utilizing the pendulum behavior while controlling the displacement demands.

The project team proposes another low damage solution for ceiling systems for consideration in the test program. Dislodged ceiling tiles are generally observed at relatively low shaking intensities, before any other damage. While dislodged tiles are easily restored, the temporary dislodgement disrupts the integrity of the suspended ceiling system, leaving the grid framing susceptible to other damage states. One idea is to explore the use of magnets to secure the ceiling tiles to the grid members as a defense against dislodging. Also, as mentioned above, the authors observed that under current detailing (Fig. 8(a)), rigid seismic bracing caused the ceiling system to move rigidly with the floor in the vertical direction, which led to dislodgement
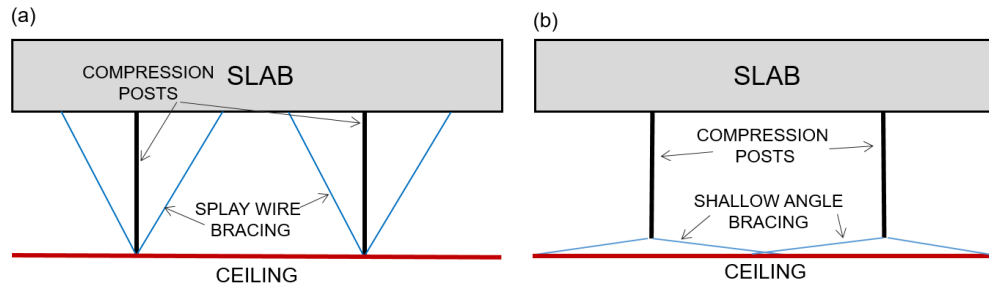
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Figure 8. (a) Current and (b) proposed seismic bracing detail for suspended ceilings (Soroushian, 2019)

of numerous ceiling tiles. A new detail for seismic bracing is proposed that uses shallow angle splay wires to connect the compression posts to the ceiling grid (Fig. 8(b)) (Souroushian 2019). This detail is envisioned to maintain the function of the bracing in the horizontal direction while separating the suspended ceiling from the floor above in the vertical direction, and thus reducing the vertical demands to the system.

Several tests have been conducted on suspended ceilings integrated with sprinkler piping (Soroushian et al. 2015; Jenkins et al. 2017). However, a knowledge gap exists with regard to mechanical equipment in the plenum region. Dynamic response of above-anchored equipment such as air handlers should be quantified under realistic 3D shaking. Furthermore, displacement compatibility of main equipment units and connecting HVAC ducts should be evaluated. These lightweight ducts are often installed with very limited bracing. Finally, the compatibility of the seismic response of the mechanical equipment and suspended ceiling system should be experimentally evaluated for undesirable interactions, leading potentially to new solutions that consider the integrated design of the systems.

4.3.2 Preliminary Plans

Preliminary plans are to fabricate and test partition walls on a minimum of two floors and suspended ceilings on a minimum of two floors. This would permit some of the detailing variations discussed above to be explored while limiting the required resources for testing. Two options will be considered: 1) integrated partition walls and suspended ceilings on the same floors, or 2) partition walls and suspended ceilings installed on separate floors. The decision depends on the test objectives for each system. For instance, if the main focus of the suspended ceiling tests is to evaluate new detailing, omitting partition walls to allow for larger open plan area best supports this objective. However, partition walls should be installed around the plan boundary in the absence of exterior skins. Under option 1 (integrated partition walls and ceilings), it is proposed to install these NCSs on one lower level (subjected to higher drift demands) and one upper level (subjected to higher acceleration demands). Under option 2 (separate partition walls and ceilings), it is proposed to install the partition walls in adjacent lower floors (subjected to similar and larger drift demands) and the ceilings in adjacent upper level floors (subjected to similar and larger acceleration demands).

Piping and/or mechanical equipment may be added as resources permit. Again, these systems may be integrated with suspended ceilings or installed in isolation depending on the determined objectives. New details for suspended ceilings or equipment installation should first be tested in isolation. However, an integrated design and test to evaluate interaction between components is desirable if the response of the components in isolation is well-established.

5.0 SUMMARY

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Preliminary plans for a seismic shake table test of a 10-story mass timber building have been described. The building is constructed with glulam beams and columns, CLT floor diaphragms, and post-tensioned CLT rocking walls. The overall test objective is to validate the design methodology for seismically resilient timber buildings. The response of NCSs are critical to resilience, and the timber specimen offers a testbed to evaluate existing and improved detailing strategies for many types of NCSs. This is an exciting opportunity to advance the state-of-the-art in NCS analysis and design. A vision for NCS testing has been presented; however, payload projects to advance alternative objectives will be entertained.

The testing will take place at the NHERI@UC San Diego outdoor shaking table after its upgrade to allow input of 6 degree-of-freedom input motions. Construction of the specimen will commence in spring of 2021, and testing is planned over the summer of 2021. The project team is actively seeking industry and academic collaborators for testing of various NCSs. Currently, the following types of contributions are needed: material donations; fabrication and disassembly; and in-kind engineering design, analysis and data synthesis.

6.0 ACKNOWLEDGEMENT

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