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FULL-SCALE SHAKE TABLE TEST OF RESILIENT SIX-STORY HYBRID MASS TIMBER AND STEEL STRUCTURE

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ABSTRACT: Mass timber solutions are becoming more and more viable for high-seismic regions while remaining sustainable, efficient, and affordable. The industry is driving innovation leading to the development of resilient hybrid steel-mass timber solutions that can minimize post-earthquake losses and downtime. A resilient six-story hybrid mass timber structure with: [i] laminated veneer lumber (LVL) beams and columns, [ii] a cross-laminated timber (CLT) self-centering rocking wall (SCRW) in one direction, and [iii] a steel moment frame/concentric braced frame (MF/CBF) in the other direction was tested at the University of California, San Diego (UCSD) large high-performance outdoor shake-table facility. The dynamic testing included uni-, bi-, and tri-directional ground motion time histories applied at increasing intensities, including 43- and 225-year hazard levels, design earthquake (DE) level, and risk-targeted maximum considered earthquake (MCE_R) level per ASCE 7-16 for a location in Seattle, Washington. Four (4) design earthquakes and two (2) risk-targeted maximum considered tri-directional earthquakes were applied to the structure. Testing resulted in peak story drift ratios of 2.4% and 1.4% in the SCRW and MF/CBF directions, respectively. Even at MCE_R levels of shaking, the performance-based seismic design allowed for (1) the CLT-SCRW to remain essentially undamaged and (2) the MF to remain essentially elastic, providing elastic restoring forces, while the CBF provided stable and controlled hysteretic energy dissipation. After testing, residual drifts were smaller than 1.6 mm (1/16 inch) at the roof, indicating that resilient hybrid mass timber-steel structures are viable. This paper presents the specimen design and summarizes the preliminary results from the shake-table testing.

KEYWORDS: hybrid system, mass timber, resilient, shake-table, seismic

1 – INTRODUCTION

The Natural Hazards Engineering Research Infrastructure (NHERI) Converging Design (CD) project included a multi-phase experimental test protocol of a six-story mass timber building to assist the development of a new design paradigm for designing resilient, sustainable, and affordable mass timber buildings [1]. A full-scale six-story mass timber gravity system with steel connections was constructed on the Large High-Performance Outdoor Shake Table (LHPOST) at the NHERI Englekirk Structural Engineering Center at University of California, San Diego (UCSD). This building structure included the opportunistic reuse of the NHERI 10-story Tallwood project [2]. In the NHERI CD project, three different lateral force-resisting systems (LFRS) were tested alongside the gravity frame in three different phases.

The Phase 1 LFRS consisted of self-centering rocking walls (SCRW) with U-shaped flexural plates (UFPs) distributed across the height of the structure for energy dissipation and post-tensioned threaded steel rods for recentering in both principal directions of the structure [3]. The mass timber wall panels in one direction were fabricated using cross-laminated timber (CLT), while the other direction consisted of mass-ply panels (MPP). Phase 2 replaced the MPP SCRW and UFPs with a new MPP SCRW with buckling-restrained boundary elements (BRBs) which concentrated energy dissipation

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to the first story [4]. The orthogonal direction utilized the same wall from Phase 1 (CLT SCRW with UFPs). Finally, Phase 3 replaced the MPP SCRW with BRBs walls with a steel moment frame / concentrically braced frame (MF/CBF) utilizing steel yield-link connectors for energy dissipation [5, 6]. Similar to Phase 2, the orthogonal direction utilized the CLT SCRW with UFPs used in Phase 1 and Phase 2.

Performance objectives for Phase 3 were established for the structure, including avoiding story mechanisms, implementing explicit design provisions that mitigated residual drifts, concentrating damage to ductile replaceable elements, and meeting drift requirements per ASCE 7-16 [7]. This paper provides an overview of Phase 3 of the NHERI CD project, including the test specimen, shake table test protocols, and engineering results from the shake table testing program.

2 – PROJECT DESCRIPTION

The design objectives for Phase 3 focused on developing resilient behavior of the mass timber-steel hybrid systems. First, the primary objective of the design was to limit residual drifts to less than 0.25% at the MCE_R shaking intensity and meet all ASCE 7 transient drift requirements. However, secondary design objectives were also established, including (1) the steel frame and braces exhibiting no damage, and (2) limiting inelastic deformations to the yield-link brace connections (YLBCs). Additionally, (3) the SCRW would see minimal damage at the design earthquake (DE) shaking intensity and limited repairable damage at MCE_R. Finally, (4) no damage to the gravity frame should occur throughout the entire test protocol.

To meet these objectives each component of the SCRW was designed with a direct displacement-based design (DDBD) methodology [8, 9, 10]. The seismic demands were estimated by displacing the building to a specified displacement profile proportional to the first vibrational mode of the building. Next, a factor of 2.1 was applied to increase shear force demands to account for higher mode effects [3]. Individual components were checked with these design forces and nonlinear response history analysis (NLRHA) was conducted to verify performance.

The MF/CBF fuses were sized according to the ICC-ES evaluation report 4342 [6] and equivalent lateral force method per ASCE 7-16 [7]. The MF system was sized using a modal pushover analysis procedure that assumes modes are weakly coupled in the nonlinear range [11, 12, 13]. This allows for the peak response in each mode to be combined despite theoretically violating modal orthogonality. These demands were used to size columns, but NLRHA was utilized to confirm and iterate on column size for until desired residual drift values were minimized. The preliminary design using equivalent lateral force method only resulted in an expected residual drift ratio of 0.19%, while the final design resulted in an expected residual drift ratio of 0.11%.

The structure had a total height of 20.73 m (68 feet) spread across six stories, with the first story measuring 3.96 m (13 feet) and all other stories measuring 3.35 m (11 feet). Each floor had dimensions of 10.47 m by 10.52 m (34.4 feet by 34.5 feet). Figure 1 shows a picture of the building prior to Phase 3 testing.

Diaphragms at each story were constructed with four mass timber products. The second- and third-floor level diaphragms comprised visually graded European Spruce 5-ply CLT (Grade C24, European Technical Assessment-09/0036) [14]. The fourth and fifth-floor diaphragms consisted of European Spruce glue-laminated timber (Grade GL24h) [15] topped with 16 mm (5/8 inch) thick structural-I graded plywood. The floor slab at level six consisted of No. 2 spruce-pine-fir (SPF) [16] nail laminated timber. Level seven (roof) consisted of dowel laminated timber using the same SPF sawn lumber. Both the sixth floor and roof diaphragms were topped with 16 mm (5/8 inch) thick structural-I graded plywood. Figure 2 shows the typical plan view of the diaphragms.

Beams and columns of the gravity system were constructed with grade 1.8E 2650 laminated veneer lumber (LVL) [16]. Gravity connections comprised an internal steel double knife plate designed to allow for rotations up to 0.05 radians without damage [17], which allowed inter-story drift ratios beyond those expected at the risk-targeted maximum considered earthquake (MCE_R). Similarly, columns were connected to the foundation with a bi-directional pin consisting of three specially steel fabricated parts [17]. Additionally, column splice plates were used at some locations to connect LVL columns across the height of the structure. Figure 3 shows a picture of the gravity system.

Two mass timber SCRW constructed with CLT panels resisted lateral forces in the East-West direction of the structure. The wall panels were custom fabricated E-rated 9-ply CLT panels with a major strength direction of MSR rated 2400F-2.0E Southern Pine and a minor strength direction of visually graded No. 1 Southern

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Pine [18]. A splice connection was also necessary at mid-height of the fourth story due to shipping and manufacturing constraints. To provide self-centering capabilities, four post-tensioned steel threaded rods with 31.8-millimeter (1-1/4 inch) diameter ran the full height of each wall with a 104 kN (23.4 kips) post-tensioning force in each. Additionally, for the distributed energy dissipation, ASTM A572 steel UFPs were added along the height of the structure. Each UFP was 203 mm (8 inches) wide by 10 mm (3/8 inches) thick with an internal bend diameter of 162 mm (6-3/8 inches). A total of 14 UFPs were included on each wall, with two at mid-height of each story and four at the first story. Figure 4 shows a picture of one of North SCRW.

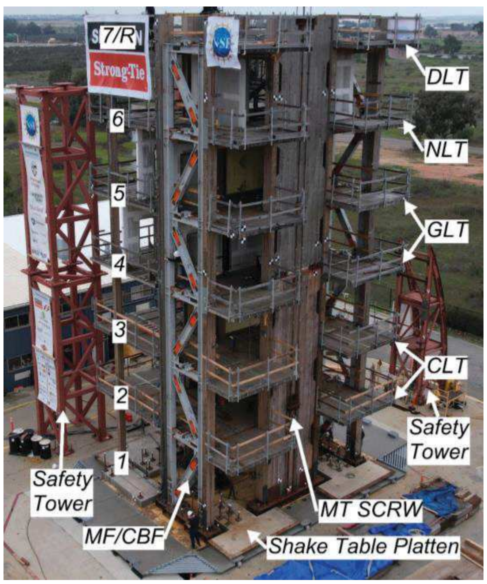

Figure 1. Image of NHERI Converging Design six-story building prior to Phase 3 testing.

In the orthogonal direction of the SCRW, two steel MF/CBF were constructed to resist seismic demands in the North-South direction. The bottom three stories of the MF/CBF consist of W14x82 columns with two (2) YLBC-8-30-1 plates at both ends of a W14x53 brace [5, 6, 19]. The top three stories utilize a smaller column size, W14x74, and fewer fuse plates, one (1) YLBC-8-30-1, at both ends of the same W14x53 braces. Steel beams were W12x40 and had one YLMC 4-4-10 assembly at alternating ends. A column splice of the W14x82 and W14x74 columns was located at the lower third point of the fourth story. Figure 5 shows a picture of one of the MF/CBFs.

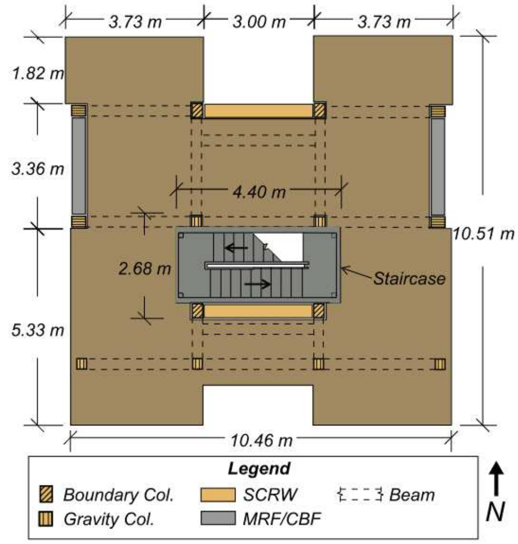

Figure 2. Plan view drawing of typical diaphragm geometry.

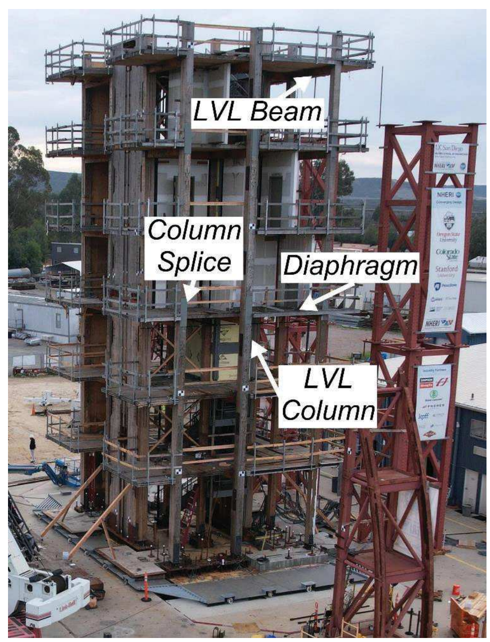

Figure 3. Picture of gravity system during construction of MF/CBFs.

Additional connections were also developed for the structure, including steel shear key for connecting diaphragms to LFRS in only the principal direction of the system, out-of-plane braces to keep the LFRS from disengaging from the structure, and non-structural connections. Detail drawings of the entire structure,

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including these connections, can be found in the DesignSafe repository of the project [1].

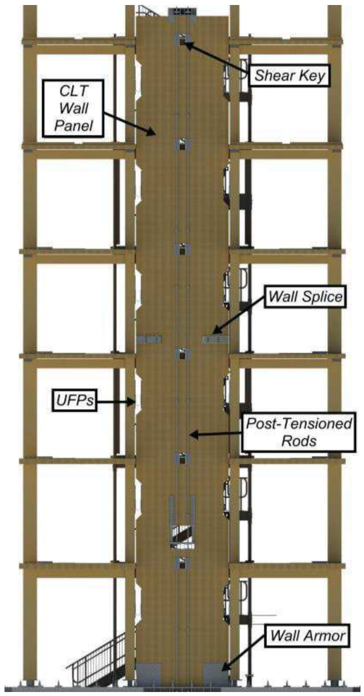

Figure 4. Diagram of mass timber SCRW.

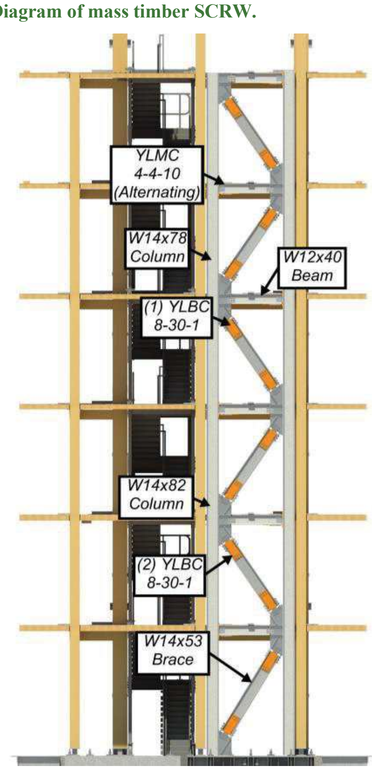

Figure 5. Diagram of MF/CBF system.

3 – EXPERIMENTAL PROCEDURES

This section details the test procedures and results from testing. The test procedure applied to the hybrid structure consisted of 20 ground motions and 20 white noise (WN) input shake table tests. The WN inputs had a root-mean-square amplitude of 0.04g with three standard deviations of 0.12g. A cut-in frequency of 0.125 Hz and cut-off frequency of 50 Hz were used to help identify the fundamental periods of the building and any shifts from high intensity shaking tests. Ground motions were first selected by matching a suite of historical ground motions maximum direction spectra (RotD100) to the design level response spectrum obtained for the selected building site location per ASCE 7-16 amplitude scaling method specifications. Then, for the testing program, two records were selected, one from the 2010 Ferndale earthquake and one from the 1989 Loma Prieta earthquake [20, 21], representing a subduction type and crustal type earthquakes, respectively. These two ground motions were scaled to four intensity levels: 43-year return period, 225-year return period, ASCE 7-16 design earthquake (DE), and ASCE 7-16 risk-targeted maximum considered earthquake (MCE_R) intensity levels. These two ground motions were run in combinations of just a uni-directional horizontal components, bi-directional horizontal components, and tri-directional ground motion components of shaking. The complete test schedule and instrumentation details can be found under the DesignSafe NHERI CD Phase 3 repository [1].

4 – RESULTS

Figure 6 shows time history responses of the structure in both directions during the Loma Prieta MCE_R shaking. The MF/CBF experienced a lower peak relative roof displacement of 176 mm (7.0 inches), while the SCRW experienced a peak relative roof displacement of 289 cm (11.4 inches).

Figure 7 shows the envelope of inter-story drift ratios (IDR) response for the Loma Prieta ground motion records scaled to four intensity levels. Figure 5a shows the IDR in the direction of the SCRW with a peak value of 1.65%, while Figure 7b shows IDR in the direction of the MF/CBF with a peak value of 1.04%. The peak values obtained were within similar magnitudes for the Ferndale ground motions. Results indicate that the peak IDR was consistently higher for the SCRW system versus the MF/CBF. A near vertical IDR profile was experienced at all intensity levels, with differences

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primarily attributed to sensor noise at lower intensity levels. Additionally, IDR for lower intensities (43-year and 225-year) were below 0.5% IDR, while the DE and MCE_R IDRs experienced were below the transient drift design thresholds of 2% and 3%, respectively.

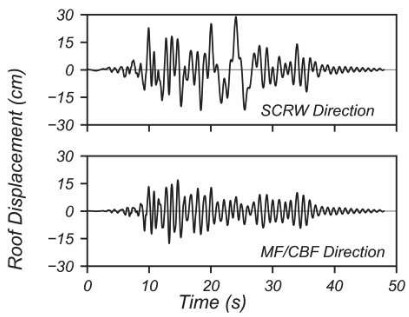

Figure 6. Time history of relative roof drift in SCRW and MF/CBF directions during Loma Prieta MCE_R shaking.

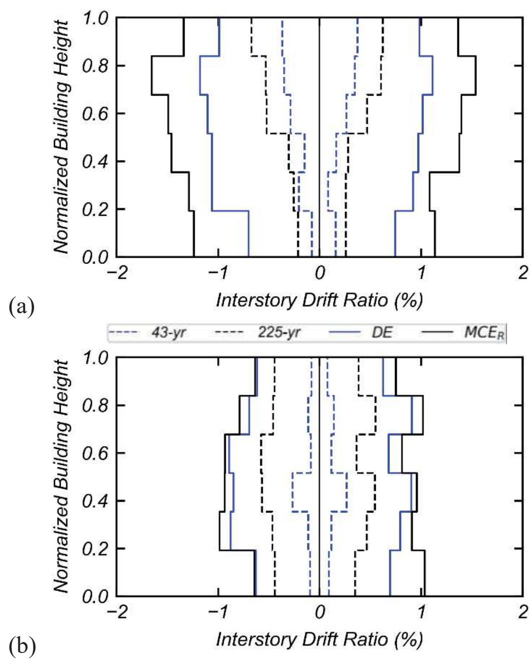

Figure 7. Inter-story drift ratios of (a) SCRW and (b) MF/CBF during four intensity levels of Loma Prieta ground motion.

Figure 8a and Figure 8b show the envelope of absolute floor acceleration during the Ferndale ground motion shaking at four intensity levels for the SCRW and MF/CBF, respectively. Absolute floor accelerations shown correspond to the mean of the envelope of multiple accelerometer measurements at each floor level. Based on the observed responses, it can be seen that both systems experienced higher mode effects, demonstrated by the decrease in the magnitude of accelerations at approximately 80% of the building height, following a node of a typical second mode of vibration. The effect of higher modes is not as evident for lower intensity shaking, as can be seen for both the MF/CBF and SCRW at 43-year intensity of shaking.

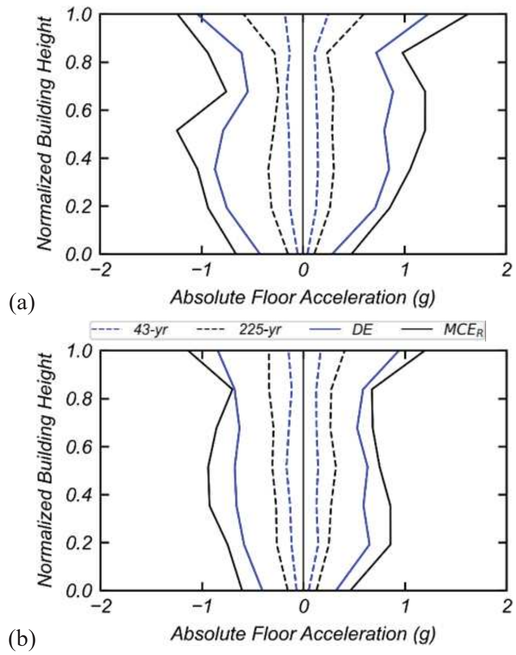

Figure 8. Absolute floor acceleration of (a) SCRW and (b) MF/CBF during four intensity levels of Ferndale ground motion.

Figure 9 shows the uplift of the walls during the Loma Prieta ground motion at four intensity levels. Each level contains two lines, showing the instant of maximum uplift in each direction. Negligible uplift was measured at the 43-year return period. displacements below zero indicate compression of the wall panel that was experienced at the 225-year, DE and MCE_R intensities. At the completion of the test program, visual observation of the wall panels confirmed that crushing occurred only at the most extreme fibers of the wall cross-section, for a length of only 6.25 mm (1/4 inch) at either wall edge. However, this crushing could have occurred from any of the three phases of the six-story project or the 150 shake table tests during the NHERI Tallwood project (from which the NHERI CD specimen was reused). Finally, the UFPs exhibited steel mill scaling, indicating yielding, but the post-tensioned rods did not reach yield strain throughout the test program.

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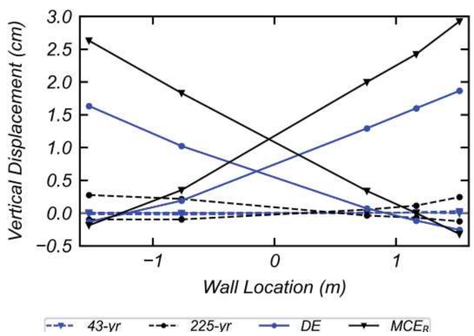
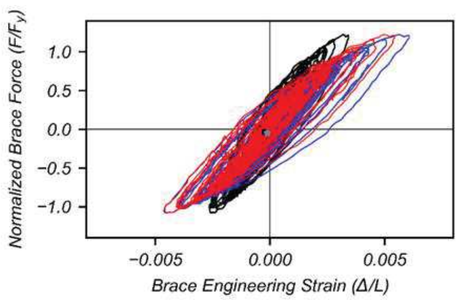
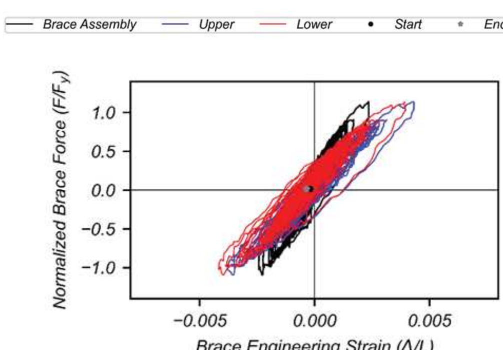


Figure 9. Wall uplift during four intensity levels of Loma Prieta ground motion.

Figure 10a and Figure 10b show the response of the first- and fourth-story YLBCs during the shake table shaking for the Loma Prieta ground motion. The first-story YLBC experienced approximately seven inelastic cycles, while the fourth-story YLBC experienced three cycles. However, in both cases, the fuses had minimal residual drift at the end of testing. Overall, the YLBCs responded as expected and allowed the rest of the frame to remain essentially elastic through the entire test program.



(a)



(b)

Figure 10. Brace components and assembly force versus engineering strain responses for the MCE$_R$ Loma Prieta ground motion: (a) first-story brace, and (b) fourth-story brace.

Overall, the structure met the design performance objectives set for the project. No story mechanisms were recorded during testing, with IDRs meeting ASCE 7-16 transient drift requirements. Inelasticities were isolated to the UFPs and YLBC fuses for the SCRW and MF/CBF, respectively. Visual inspection of the gravity system showed no inelasticities and rotations remained below the 0.05 radians defined for design. At the end of the testing program, residual drifts at the roof were measured at less than 1.6 mm (1/16 inches).

In summary, the testing program demonstrated that (1) shake-table specimen reuse was not only viable, but that it allowed the NHERI Converging Design 6-story shake table program to study structural resilient lateral force-resisting systems that were not possible to be tested in the NHERI 10-story TallWood project; (2) sustainable and economical resilient designs are possible with both mass timber only structures and also with hybrid mass timber-steel structures.

5 – CONCLUSION

A full-scale shake table test of a resilient six-story hybrid mass timber and steel structure was designed, constructed, and tested at the NHERI@UCSD LHPOST in 2024. The six-story mass timber gravity frame consisted connections that allowed for deformation compatibility up to 0.05 radians without experiencing any damage. Two innovative LFRS were tested in orthogonal directions (1) a mass timber SCRW with UFPs in one direction, and (2) a resilient steel MF/CBF system. The mass timber SCRW isolated inelasticities to the UFPs and avoided significant crushing damage to the wall panels at the foundation. Post-tensioned rods for self-centering remained elastic through the testing program. Similarly, the MF/CBF primary framing members remained essentially elastic and isolated inelasticities to the YLBC components on either end of each brace, especially braces at the first two stories. The performance objectives set forth for the project were all met, showing that a resilient mass timber structure can be affordably designed for resilient behavior under extreme seismic excitations.

6 – AKNOWLEGEMENTS

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