THESIS

SHAKE TABLE TESTING OF A TWO-STORY CLT PLATFORM BUILDING

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ABSTRACT

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Cross Laminated Timber (CLT) is an engineered, prefabricated, wood product that is well established in the European construction market, and has seen increasing usage in North America. In the U.S., CLT's application has been mostly limited to low seismic regions due to its exclusion from current seismic design standards, requiring designers to apply alternative design methods, and ultimately undermining its economic competiveness in some cases. This thesis presents the method and results from a full-scale two-story CLT platform building test conducted at the NHERI@UCSD Shake Table in San Diego, California. The testing was divided into three phases, with each phase testing a different wall configuration. The first two phases investigated the effects of different CLT panel aspect ratios (height:width) on the performance and behavior of the structure, with aspect ratios of 3.5:1 and 2.1:1 being tested respectively. The third phase used the same 3.5:1 aspect ratio CLT panels as the first phase, but introduced transverse walls to document the behavior of a more realistic building system. The Equivalent Lateral Force (ELF) procedure was used in the design of the stacked CLT shear walls, with the assumption that shear was resisted entirely by generic angle brackets, and the overturning moment was resisted by tie-down rods on either end of each shear wall. The structure was subjected to several different intensities of the 1989 Loma Prieta ground motion record, with the largest motion having a return period of approximately 2500 years. Life-safety is the primary objective of current seismic design in the U.S., and all three phases of testing showed no risk of collapse. The test results provided information on the dynamic behavior of platform style CLT construction with stacked shear walls.

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CHAPTER 1– AN INTRODUCTION

Background Information

Timber, as a construction material, has a history that stretches back thousands of years. However, in U.S. modern history, timber has been viewed as a cost-effective material for low-rise and light frame construction. This categorization has begun to shift over the last several decades with the development of mass timber. Mass timber is a general term that includes both new methods of producing and implementing highly engineered timber products in high performance situations on various scales. Cross Laminated Timber (CLT) is a mass timber product first developed in Central Europe several decades ago. CLT is a prefabricated, engineered, solid wood panel that is produced by utilizing kiln-dried dimension lumber (1x or 2x are typical) and glue lamination to create lamination layers that are then stacked with long wood grains in alternating, orthogonal, directions to create panels, resulting in a non-homogenous, anisotropic material [20]. The number of layers in a panel is variable and depends on the application and required properties, but it is typical to have an odd number of layers (3 to 7 is common), creating major and minor flexural and shear axes. The dimensions of a CLT panel are normally rectangular (with the major axes typically in the long direction), with the width limited by the size of the press, and the length by the mode of transport to the project site. In addition, using precision routers, it is possible to cut holes in the panels in a variety of shapes and sizes for such things as doors, windows, etc. as well as alter the rectangular shape of the panel to fit various architectural and design constraints. CLT manufacturing methods lend themselves well to panelization, allowing for the building segments to be prefabricated at the manufacturer, transported, and assembled on site. This, combined with its high strength to weight ratio, low environmental impact, and other performance characteristics were the incentives for investigations into CLT as an alternative to conventional materials (concrete, steel, etc.) for several building applications. Specifically, mid-rise buildings were identified as an ideal area for CLT to be competitive, but quickly implementation challenges presented themselves. Panelized CLT buildings perform well resisting gravity loads due to the panel's rigid properties, however, it was discovered that the behavior of CLT panels under lateral load was not well understood. While the ridged properties of the panels were an asset in bearing, they didn't allow for CLT wall systems to deflect laterally as well as its light frame counterparts, limiting applications of CLT in moderate to high seismic regions. More ductility or an energy dissipating mechanism was needed in the design to reduce the high acceleration amplitudes and overturning moments experienced by rigid CLT wall systems. The most advantageous place to introduce the required properties were the connections, which allowed for the CLT panels themselves to remain unchanged while achieving the required performance. The potential of CLT as a competitive alternative to concrete and steel for mid-rise buildings, even in seismic areas, was the impetus of investigations into CLT lateral force resisting systems or more specifically to earthquake engineering, seismic force resisting systems (SFRS). A review of notable applicable research with CLT as a seismic force resisting system is presented in the following with the summary of relevant researched organized both geographically, and chronologically.

European Developments

The history of heavy timber construction in Central Europe created an economy in which mass timber companies could expand. By the early 2000s, little over a decade since its inception, several mass timber companies were producing CLT panels, and even complete panelized CLT buildings. However, most of these applications were limited to single family homes and low-rise buildings, due to the gaps in contemporary research demonstrating CLT as an effective lateral force resisting system, with wind specifically limiting mid-rise applications in non-seismic regions. Furthermore, in some regions of Central Europe (Italy, Slovenia, etc.) as well as Southern

Europe (Portugal, Greece, Turkey), high seismicity was an obstacle, providing additional incentive to use CLT as not just a lateral force resisting system for wind loading, but for seismic resistance as well.

Early Research

Slovenia was situated at the crux of the limitations of CLT in the early 2000s, part of Central Europe and the heavy timber traditions, several mass timber companies producing panelized CLT buildings existed in the country, however seismicity in the region limited the market to Northern Europe. Demand to incorporate the local market as well as expand the use of CLT to mid-rise applications provided motivation for further research. Published in 2004, Dujic et. al (2004) [9] was one of the first studies conducted with the intent of demonstrating CLT as a lateral force resisting system. The testing took place at the University of Ljubljana in Slovenia, financed in part by the Slovenian Ministry of Education, Science, and Technology, RIKO HISE Ltd. (a local panelized CLT building company), and KLH Massivholz GmbH (an Austrian CLT Manufacturer). RIKO HISE Ltd. Produced mostly single-family panelized CLT homes and desired to expand into the local market, preferably utilizing mostly existing designs. The building designs were connected to the foundation using steel anchorages, and as mentioned previously, even prior to this study, it was identified that connectors and anchorages were the most advantageous component of a CLT wall system to introduce ductility or energy dissipation. Therefore, the study focused on testing the existing anchorage design for several modifications with the goal of determining their mechanical properties. To do this, a CLT panel cantilever test set-up was designed, in which a CLT panel was connected to a steel frame along the top of the panel, and to a horizontally displaceable mechanism controlled by an actuator along the base. A constant vertical load was then applied to the panel, and displacement-controlled horizontal loading was applied to rack the panel. Three different vertical load magnitudes were used along with three different horizontal loading protocols, utilizing a combination of monotonic and cycling loading, a first for CLT panel testing. Three anchorage configurations were tested, leading to a total of 15 tests. The test results revealed that the anchorage strength and local wood failures were the controlling factors in the capacity of a CLT lateral force resisting system. This was already suspected due to the rigid nature of the CLT panels and their limited ability to deflect laterally, forcing the anchorages and connectors to be the main means of energy dissipation. In addition, similar to lateral force resisting systems using other materials, CLT wall systems see improved lateral performance when the panels are simultaneously engaged in bearing, especially in cases when the anchorages were weak. This study took the first initial steps towards building the necessary knowledge to incorporate CLT wall systems as lateral force resisting systems, resulting in further studies, and laying the groundwork for a variety of studies in Europe and beyond.

While designing and conduction the testing in Dujic et. al (2004) [9], several deficiencies in the application of contemporary European testing protocol to CLT became apparent. The testing protocol EN594, which was the most current at the time, was only designed to estimate the racking strength of timer frame walls with sheathing plates and fully restrained frame studs as well as only accounting for monotonic racking loads [8]. In the case of partially anchored studs and minimal vertical load as well as high seismic regions, the aforementioned testing protocol overestimates the bearing capacity of the wall. To address the need for updated testing protocols for realistic boundary conditions and seismic prone regions, an investigation was launched by Dujic et. al (2006) [8] with the objective of demonstrating a variety of testing possibilities using adaptable boundary conditions and various loading protocols. Three boundary cases were identified as most likely to occur in reality, and were designated Case A, B, and C respectively. Case A included a shear cantilever mechanism, where the base of one edge of the CLT panel is fixed, and the other is free to rotate and translate. This is most commonly seen with flexible roof systems, or narrow, slender CLT panels. Case B, featured a restricted rocking mechanism, where one edge of the panel is fixed, and the other can translate and rotate, but only as much as a ballast restricted to vertical translation, typical for walls carrying floor load, allows. The final boundary condition, Case C, is a shear wall mechanism, where one edge of the panel is fixed, and the other is only allowed to translate parallel to the fixed edge. This case usually occurs with the infill of a stiff surrounding frame. Case A and B had a constant vertical load throughout the testing, while Case C had a variable vertical load due to the need to increase the load during testing, preventing panel uplift. The testing incorporated both timber-framed walls, and CLT wall panels subjected to both monotonic and cyclic loading, using the aforementioned boundary condition cases, as well as three levels of vertical loads. The results showed that boundary conditions can significantly impact the measured load bearing capacity of a sample. For the CLT panels, Case C had a capacity of almost double that of Case A, leading the authors to concluded that additional loading protocols needed to be developed, and that designing a CLT shear wall using Case C alone was not recommended. This study quantified the suspected deficiencies in contemporary loading protocol for timber structures particularly CLT panels. It also emphasized the need for additional development of loading protocols using cyclic loading for CLT shear walls located in seismic regions.

The culmination of the multi-year project funded by the Slovenian government and various commercial partners was shake table tests conducted at the Dynamic Testing Laboratory of the Institute of Earthquake and Engineering Seismology (IZIIS) in Skopje, Macedonia [10]. The primary objective of these tests was to correlate the shake table results with the quasi-static test results from the earlier study, ultimately providing the necessary information to design and construct seismically resistant CLT buildings. The test specimens were single story boxes that consisted of parallel CLT wall panels with perpendicular panels used for support only. During the testing, both single and double panel configurations were tested, and subjected to earthquake motion records with frequencies ranging from 5 Hz-7.5 Hz. The tests showed that the wall configurations exhibited a non-linear behavior with the connectors deforming and dissipating energy, while the CLT panels acted as a rigid, linear elastic body. These tests were the first shake table tests performed on CLT lateral force resisting systems, and provided valuable insight into the behavior of the systems under seismic loading, as well as identifying needs for future research. The authors of the study specifically described the need for new protocols and design limits (interstory drift, base shear, etc.) to be developed for CLT structures located in seismic areas, as well as the need for further testing to develop a behavior factor "q" for CLT for use in Eurocode. The "q" factor is similar to the R factor used in U.S. seismic design.

SOFIE Project

Other seismic regions of Europe were also extremely interested in the potential application of CLT as a lateral force resisting system. Specifically, beginning in 2005, a comprehensive, landmark study investigating all components of a prefabricated CLT building's behavior, including mechanical properties, building physics, acoustics, fire, durability, and of particular relevance to this thesis, seismic performance [17]. The project was funded by the Trento Province in Northern Italy, and its components were coordinated and conducted by the Italian National Research Council – Trees and Timber Institute (CNR-IVALSA). The generalized approach of the study was to start with smaller scale quasi-static experiments on connectors and wall panels, eventually leading to multiple full-scale shake table tests.

The first stage of the project began in the mid-2000s with Ceccotti (2008) [6]. The study conducted monotonic and cyclic tests performed on four different CLT wall panel configurations, and also a one-story pseudo-dynamic test. The wall panel configurations tested different connectors, configurations, and interstory connections. Configuration A was based on a real first story CLT wall system with commercial connectors and hold-downs, and a constant vertical load. There were three steel angle brackets spaced across the bottom edge of the panel, and a hold-down on either end, with the number and strength of nails variable by tests. This configurations was tested a total of seven times using a combination of monotonic and cyclic loading. The tests were conducted to gather data on various failure types (i.e. hold-down vs connector), and the number of nails used to secure the hold-down to the CLT wall panel was changed to influence the failure mode. Configuration B, similar to Configuration A, was based on a real CLT wall system, in this case a typical second floor shear wall. The configuration consists of the same basic components of A, however there are only two shear connectors across the base of the panel instead of three. Configuration B was tested in a similar way to Configuration A with both monotonic and cyclic test being conducted, however the number of nails securing the shear connector to the panels was varied instead to achieve different failure modes. Configuration C was also a first story CLT wall system, however there were no shear connectors and only hold-downs located at each of the four corners of the panel. The testing of this configuration consisted of only one monotonic and one cyclic test with no variation in the configuration. Configuration D was unique among the four configurations due to the presences of an opening in the form of a doorway, otherwise the connectors and hold-downs were similar to Configuration B. The testing program was similar to that of Configuration C, however, the size of the opening was increased between the monotonic and cyclic test. The tests provided more evidence that the connectors were the ductile part of the CLT wall system, with the panels themselves exhibiting rigid behavior. This localized displacement and force concentration was discovered to potentially cause material failure in the CLT panel itself if defects (such as an error in the lamination) were present. The tests also produced an average viscous damping of 14%, demonstrating the suitability of CLT wall systems for seismic regions. After the individual CLT wall systems were tested, a larger scale one-story CLT wall assembly with three different configurations was tested. A pseudo-dynamic testing approach was taken where the forces on the structure are calculated using the weight of the structure and the acceleration values of a real ground motion record. This method is limited in its ability to accurately simulate the velocity of an earthquake, and is usually considered as a precursor or proof of concept test before a full scale shake table test. The first test configuration was a symmetric layout with external openings, and one smaller internal opening, with the goal of simulating a real structure. The second configuration was identical to the first except the internal opening was increased to the same size as the external openings. The final configurations was an asymmetric layout with one external opening being approximately 1.8 times larger than the other openings. It should be noted that cyclic tests were carried out in between the pseudo-dynamic tests to establish the stiffness of the structure. In general, it was observed that the configurations were very stiff, but still have sufficient ductility (mostly provided by the connections). This was observed in other testing, and this result reinforces the observations of earlier tests. What was a surprising and consequential discovery was the similarity in performance and stiffness of the second and third configurations. The third configuration featured a much larger opening than the second configuration, and the results imply that the lateral stiffness is determined by the connections, not the wall panels, for low magnitude shear forces. In general, this test further reinforced the importance of the connections in determining seismic behavior (and capacity) and in particular demonstrated that connector strength can be increased to a point approaching the material without compromising ductility. This revealed the clear potential of CLT wall systems as a SFRS and provided the necessary data and proof of concept needed for larger scale tests.

The next step in the SOFIE project was to progress from pseudo-dynamic testing to full scale shake table testing, in order to determine an over strength factor for Eurocode design. This was done in two phases, with the first phase consisting of testing a three-story CLT building on the National Institute for Earth Science and Disaster Prevention's (NIED) Tsukuba Shake Table in Japan [6]. The building was a full scale 7m x 7m (23 ft x 23 ft) three-story CLT building with a total height of 10 m (33ft). The panels were made of spruce imported from Italy, and the building was constructed on a steel frame connected directly to the shake table. The building itself was connected to the steel frame using commercial connectors and anchors, and consisted of four outer walls, one inner wall, floor diaphragms, and roof panels, all composed of CLT panels. Three different configuration were tested, following a similar pattern to the one-story test with each configuration using a different sized opening on the ground floor of the structure. Additional weight was added on each floor of the building to account for the insulation and finishing to the building as well as a percentage of the design live load as stipulated by the European and Italian seismic code. The building itself was designed assuming all of the ductility was provided by the connectors. The connecters were then designed to ensure that energy dissipation (deformation and pull out) occurred first in the inter-panel joints between panels, then the shear connectors between the walls and the floors, and finally the hold-downs resisting overturning moment. This approach was taken to maximize energy dissipation, and was based on results from the panel and connection tests previously discussed. Instrumentation including accelerometers and displacement transducers were placed on each story of the structure to measure acceleration, relative displacements, interstory drift, and uplift. Load cells were also used to measure the force in the hold-down rods at the base of the structure. The three story CLT building then was subjected to a total of 15 shake table tests with peak ground accelerations (PGA) ranging from 0.5 g to 1.2 g.

Between tests, only minor repairs were made to the structure, and the building performed well and did not sustain any damage that was not felt to be repairable. There was however, a significant amount of damage observed in the connections that would need to be repaired, but which was expected since the connections had been established by this point to provide the energy dissipation. In conjunction with the seismic tests, a numerical model was developed to model the behavior of the building. The numerical model used a nonlinear time history approach that modeled CLT panels as rigid braced frames interconnected with nonlinear connector springs. The purpose of the model was to incorporate previous test findings and shake table tests to produce a model capable of accurately predicting the response of CLT buildings for seismic loads. The developed numerical model incorporated the over strength factor calculated from the responses of the three story CLT building, attempting to recreated the test numerically. The results of the efforts were in good agreement with the experimental results. The testing was not only the first shake table test on a realistically sized CLT building, it also provided information necessary to suggest an over strength factor necessary for design using Eurocode, as well as to calibrate a powerful numerical model capable of other applications such as design and response predictions.

The SOFIE project culminated in a seven-story full scale shake table test conducted in 2007 at NIED's shake table in Miki, Japan (the largest shake table in the world). The test was the largest 3D shake table test ever conducted up to that point on a full scale structure composed of any material. The structure itself was designed using the simplified lateral force method in Eurocode 8, using knowledge from other parts of the project such as the over strength factor from the three story tests. The test is documented in Ceccotti et al. (2013) [7], and similar to the three-story tests, the structure was constructed using CLT produced using forests in Northern Italy. The building consisted of a 7.5 m x 13.5 m (24.5 ft x 44ft) floor plan with seven stories, resulting in a total height of 23.5 m (77ft) with the thickness of the wall varied per story due to decreasing structural demand on the higher floors. The connections were designed according to the shear demand of each floor, and consisted of the same commercial connectors tested in the earlier phases of the project. The structure was heavily instrumented with over 260 sensors, and accelerations, uplift, relative displacements, interstory drifts, and slip between floor and wall panels were measured. The structure was subjected to more than 10 shakes with a maximum PGA of 0.82 g. After the tests, no residual displacement was observed in the structure, but damage was observed on the hold-downs on the first floor, confirming that the combination of the stiff behavior seen in previous tests with a slender profile would produce large overturning moments. Large accelerations were also observed (over 4 g) at the upper levels of the building. Additionally, it was determined that uplift, slip, and interstory drift were not critical. The test emphasized the need to introduce more ductility in the system for mid-rise buildings during large earthquakes, as well as to expect large overturning moments. Overall, the test and the SOFIE project as a whole demonstrated the suitability of CLT for used in low and mid-rise structures in seismic areas in a very thorough and highly visible way that could be used to convince government agencies as well as the general public. In addition, the project also left a clear path forward for research, improvements in the contemporary design philosophy were still very much possible.

Other European Efforts

In a follow up of sorts to the SOFIE project, Gavric et al. (2011) [13] summarizes a multipart project investigating the some of the areas identified as in need of further research during the course of the SOFIE project. In particular, the study focused on the performance of connections in CLT wall panels, with the ultimate objective of developing analytical models to predict strength and stiffness properties of connectors, allowing for the development of over strength factors. Gavric et al. (2011) [13] was the third and final phase of the investigation with the previous two parts testing hold-down connections and angle bracket connection tests [15]. The final phase of the investigation tested over 20 different configurations of panel connectors with screws as fasteners. Monotonic and cyclic loading were used in all three components of the testing, and the results of the project provided valuable information on the design of CLT connections where limited damage under seismic load is desired. The study proposed over strength factors for design ranging from 1.2 to 1.9, depending on design and applications.

North American Developments

These early CLT studies, and with the construction materials growing popularity in Europe, improved the research and practicing engineering communities understanding of its properties and applications, and its competitiveness with concrete and steel began to spread to other places around the world. In North America, Canada with its large timber industry, has long had significant economic incentives to make timber as versatile as possible. It is no surprise then that they should take the leading role for introducing CLT into the North American market. Similar to Europe however, moderate seismicity in British Columbia limited its application there, and in response, FPInnovations commenced a series of studies on the seismic design of CLT systems. Popovski et al. (2010) [27] conducted 32 monotonic and cyclic tests using CLT panels imported from KLH Massivholz in Austria. Twelve different wall configurations were tested with a wide variety of aspect ratios, connector types, connector configurations, boundary conditions, and openings. In addition to the wall configurations, a two-story panel test was conducted. These relatively comprehensive tests provided valuable information on a wide variety of wall configurations, with the results agreeing well with previous European studies. The interpanel joints and shear connectors were the main source of ductility in the tested CLT seismic systems, and while this was expected, the tests began a large and extensive research effort in North America to further research

CLT for use in seismic design. In a follow up study, Popovski et al. (2012) [25] attempted to build on previous European studies quantifying the behavioral factor of CLT for Eurocode 8. This included estimating the seismic modification factors (R-factors) for CLT wall systems for the National Building Code of Canada (NBCC). The effort consisted of three distinct methods: comparison to existing timber systems in NBCC, using a similar methodology to previously conducted European studies, and following the equivalency approach that is recommended by the International Code Council 2013. The recommended AC130 approach was conducted using the test results from Popovski et al. (2010) [27]. The final recommended factors for R_0 and R_d were 1.5 and 2.0 respectively. This study represented a first attempt to estimate these factors and would provide a basis for further studies, including the project presented in this thesis.

A summary of contemporary knowledge of CLT systems in North America was presented in Popovski et al. (2011) [26], which was then incorporated into FPInnovations' CLT Design Handbook, Canadian edition. Similarly in the U.S., through a collaborative effort including FPInnovations, the American Wood Council, APA-The Engineered Wood Association, Woodworks U.S., and the U.S. Forest Products Laboratory, developed a corresponding US edition of the CLT Handbook. Both editions present a wide collection of information on CLT including manufacturing techniques, design methodology, connection design, lateral performance of CLT wall systems, fire performance, etc. The handbooks provided a comprehensive compilation of contemporary CLT research for use in the design and construction of CLT buildings in North America.

Previous European studies had demonstrated the capability of CLT for use in performance based seismic design. However, a system for quantifying damage levels for CLT had not yet been developed. Schneider et al. (2012) [28] attempted to fill this gap using test data from previous

North American studies. The approach attempted to quantify damage using an energy based index first developed by Kratzig et al. (1989) [16]. The different failure modes for the connectors were identified, and five damage categories were developed: None, Minor, Moderate, Severe, and Collapse. The energy damage method was compared with a visual damage assessment over the course of the testing. The correlation between the calculated and observed damage was good, and represented the wall behavior satisfactorily. The establishment of these damage categories were an essential component of the implementation of CLT into performance based design, and laid the groundwork for further development.

With a satisfactory number of preliminary 2-D investigations conducted in North America, Popovski et al. (2016) [24] investigated the 3-D performance of a CLT structure subjected to lateral loading. Documenting the global behavior of the structure, its deformation capacity, and the frequency response of the structure before and after each test were the primary objectives. A 6.0 m x 4.8 m (19.75 ft x 15.75 ft) house was subjected to five quasi-static tests (one push-over and four cyclic), with each direction being tested sequentially. During the different tests, the connector's configurations as well as the loading was varied. This included the direction of loading, quantity of hold-downs, and connections between the CLT panels. This test demonstrated that sliding and rocking are caused by bracket failures and represent the majority of the ductility of the CLT panel. The testing also demonstrated the ability of CLT panels to exhibit rigid body motion (i.e. rocking). In addition, the study particularly stressed the need of further research into the effect of different aspect ratios on the behavior of the panels. One of the objectives of the larger research project of which this thesis is a part, was to determine the effects of various aspect ratios on the performance and behavior of CLT shear wall systems.

An effort to estimate suitable seismic design factors (R-factors) for CLT buildings in the U.S. was undertaken by Pei et al. (2013) [19] because the R-factors are a necessary component of force-based seismic codes in the U.S. To that end, a nonlinear, load-resistance model was developed for CLT shear walls based on previous reverse cyclic test data. Possible R-factors were estimated by designing a 6-story CLT apartment building, then using peak interstory drifts, and selecting R-factors that achieve the desired performance. An R-factor of 4.5 was suggested thereby providing a probability of non-exceedance of 80% for a 4% interstory drift. This study was one of the earlier attempts to estimate an R factor, however it was limited by scope and methodology. This estimate only used one building configuration, to be more rigorous, a number of archetypes should be developed and used instead. A methodology to develop an R-factor in a comprehensive and rigorous way is presented in FEMA P695 (2009), and it is suggested in the study that this be used for future R-factor development.

Current State of the CLT industry in North America and CLT code in the U.S.

Since its introduction, CLT has seen steady growth in North America, with numerous manufactures such as Structurlam and Nordic structures in Canada, and Smartlam (located in Montana) in the U.S appearing since 2010 [20]. Another region seeing a growing interest in CLT is the Pacific Northwest in the U.S. The region has high interest in CLT due to the regional timber industry. DR Johnson Lumber Co. was the first to begin CLT production in 2015. Sterling Lumber Company is another example of the growing presence of CLT and began production in the spring of 2016 outside of Chicago. Several research efforts funded by various private and government agencies have been conducted to identify local wood species ideal for CLT production, as well as to develop the most economical production chain for CLT. The general idea is to create an efficient industry capable of using low grade hardwoods and other low-value sources to produce CLT with satisfactory strength and other material properties. One of the first steps to introduce a new engineering product is to develop product standards. In North America, the first standard was developed by a collaboration of APA-The Engineered Wood Association and FPInnovations [20]. The PRG 320 standard was then published and recognized by the American National Standards Institute (ANSI). This standard, along with a dedicated chapter for CLT design was added into the 2015 version of the National Design Specification® (NDS®). The International Building Code (IBC) then adopted CLT construction by referencing the NDS®. CLT was also recognized as a heavy timber material, permitted in Type IV construction. This was due in most part to the efforts of the American Wood Council (AWC) and their sponsorship of an ASTM E119 fire endurance test. There still remains a need to gain acceptance of CLT in seismic building codes by developing a seismic design methodology and demonstrating its effectiveness.

Overview of Colorado State University Project to Quantify Seismic Performance Factors for CLT

To help facilitate the implementation of CLT in seismic regions in the U.S. Colorado State University undertook a multi-year project funded by the Forest Products Laboratory to identify seismic performance factors for CLT using the procedure laid out in FEMA P695 (2009), with the ultimate goal of incorporating CLT into US seismic design codes and standards. FEMA P695 [11] is a methodology to develop seismic performance factors including the system overstrength factor, the deflection amplification factor, and response modification factor (R-factor). The methodology calculates the margin of collapse of a system using an iterative process in which a suite of archetypes is developed and evaluated using non-linear dynamic analysis [1].

The project was divided into several sub levels: (1) Component and assembly level testing, (2) Design methodology development and calibration based on test data, (3) Developing and calibrating numerical model, (4) Representing the design space with the development of archetypes, and (5) Extensive analysis to identify seismic performance factors. In the summer of 2017 there was an opportunity to build onto an existing full-scale shake table test being planned and test several wall configurations designed using the CLT design methodology with the equivalent lateral force procedure. However, at the time of the test the exact factors were not known so a best estimate was used which were not the final values now being proposed.

As demonstrated by numerous previous research efforts, the connector placement and behavior in a CLT wall system greatly affects the performance of the system under lateral loading. The first stage of the project involved extensive testing of connectors, specifically angle bracket connectors and inter-panel connectors [1]. Generic sheet steel was used in the testing to allow for connector manufacturers to perform an equivalency analysis, thus allowing use of their products in the U.S. The connectors were subjected to shear and uplift testing with 10 different tests being conducted using the CUREE loading protocol [16], which used a combination of monotonic and cyclic loading. Two different grades of CLT (E1 and V2) were also tested to investigate their effect on connector performance. The connectors performed as expected, with nail withdrawal being the primary deformation behavior. CLT wall tests were the next stage in the assembly level testing, and investigated the effects of a variety of parameters, including: boundary conditions of CLT shear walls imposed by CLT diaphragms, gravity loading, connector type, connecter plate thickness, CLT grade, CLT panel aspect ratio, panel thickness, and inter-panel connector presence [1]. A total of 26 tests were conducted varying all of the previously mentioned components of the wall system to develop test data representing a wide variety of possible scenarios. This data, along with the connector data was used to develop a numerical model.

For development of the numerical model, information compiled in the U.S. Edition of the CLT Handbook was used extensively [2]. The connectors were modeled using a 10 parameter

CUREE model that was then reverse calibrated using the procedure set forth in the CLT Handbook, and the compiled connector assembly test data. The CLT walls were modeled to simulate behavior presented in the CLT Handbook, and calibrated using the wall assembly test data. Finally, the building level modeling was done using SAPWood software [21], which was developed as part of the NEESWood project analyzing light-frame wood structures. The model created was capable of performing nonlinear static analysis as set forth in Section 3.3.3 of ASCE/SEI 41-06 (2007), with the purpose of determining period based ductility and over-strength factors [2]. The software was also set up to perform Incremental Dynamic Analysis (IDA) as required in FEMA P695, which utilized a set of large-magnitude predefined ground motion records, known as “Far-Field” earthquakes.

With the model in place, a suite of archetypes was required in order to comply with the FEMA P695 methodology [2]. This critical component of the process defined the design space and therefore the range of applicability of the design factors under development. For CLT, single-family dwellings, multi-family dwellings, and mid-rise commercial (as well as mixed use) archetypes were considered. Each archetype was developed considering several variables, such as number of stories, design categories, story height, interior and exterior wall finishes, and CLT shear wall aspect ratios. With the archetypes and numerical model in place, it was possible to perform extensive analyses to determine the seismic performance factors as set forth by FEMA P695 for CLT.

As a supplemental step to the multi-year project developing the seismic performance factors for CLT, a full scale two-story CLT platform building test served as an opportunity to gather experimental data on the performance of CLT and the effects of different CLT panel aspect ratios on the performance of the system. As mentioned earlier, the seismic performance factors that were to be proposed from the multi-year project were not known at the time of the shake table testing, thus best estimates which were eventually determined as un-conservative were used. However, a number of important conclusions which will be highlighted later were able to be drawn. This thesis documents the methods and results of this testing, with the building layout, instrumentation, and testing program discussed in detail in Chapter 2; the results of the testing including the overall global and local performance of the structure and CLT shear walls are presented in Chapter 3; and finally, a discussion on the overall performance of the structure and its implications for CLT implementation in future seismic code is included in Chapter 4.

CHAPTER 2 – TESTING METHODOLGY

Building Layout and Shake Table

Full-scale shake table tests are a large, expensive undertaking which usually require a significant amount of time to plan and execute. Therefore, it is often beneficial to collaborate on large shake table projects in order to distribute the cost and planning workload. To that end, the two-story CLT platform shake table test presented here was one part of a multi-stage test program investigating CLT as both a traditional and resilient seismic force resisting system (SFRS), requiring collaboration from a multitude of different institutions and universities. The test program was split into three distinct stages, with the first and second stages focusing on resilient SFRS in the form of two-story tall rocking walls [22], and the third stage investigating a SFRS using platform construction with generic shear connections and tie-down rods designed using Equivalent Lateral Force (ELF) procedure. The test building (gravity frame and floor/roof diaphragm) was designed to facilitate the needs of all three stages of the three month test program, however, this paper focuses exclusively on the third stage of testing and its three sub phases, which each had a different shear wall configuration. Therefore, only a brief description of the relevant information about the gravity frame and floor/roof diaphragms will be presented and with further information available in [4, 5, 22]. The testing structure was a composite two-story glulam and CLT platform structure, as can be seen in Figures 1 and 2, with a height, width, and length of 6.7 m (22 ft.), 6.1 m (20 ft.), and 17.7 m (58 ft.) respectively. The structure itself consisted of three sub-systems, the gravity frame, the diaphragm, and the SRFS. Figure 1 shows a photograph of the test structure with parallel shear wall stacks comprised of 3.5:1 aspect ratio CLT panels. Figure 2 (a) shows the gravity frame, which consisted of the glulam beams and columns that supported the dead (gravity) loads of the structure, and can be divided into the first story and the roof. The column system on

Figure 1. Test structure with 3.5:1 aspect ratio CLT shear walls (Photo credit: NHERI TallWood Team).

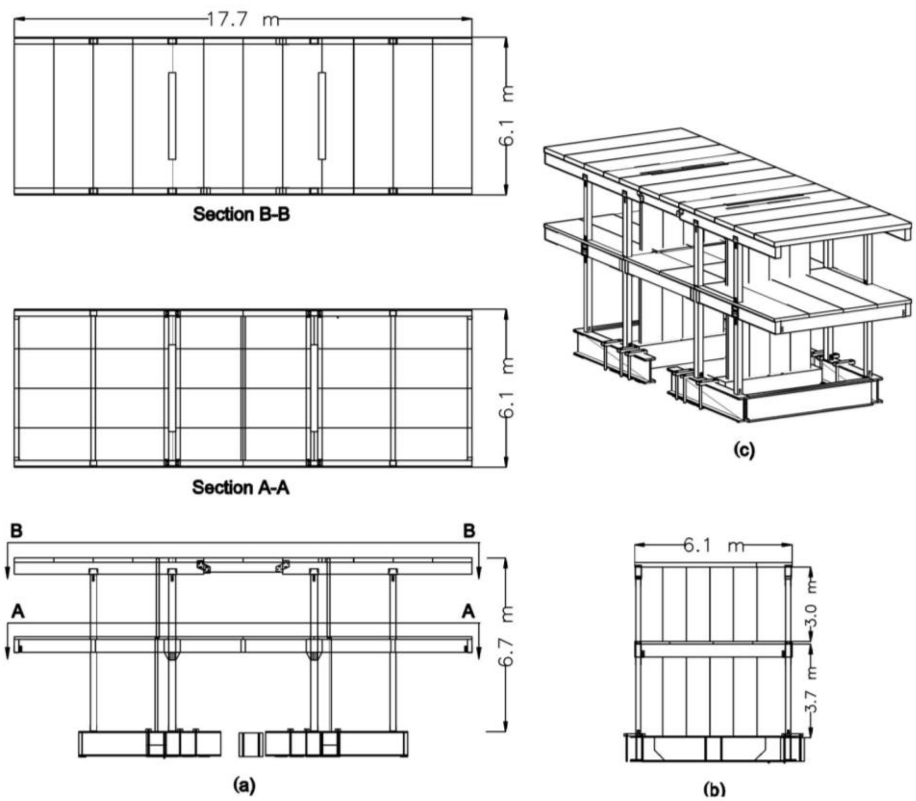


Figure 2. Test Building: (a) Front elevation and plan view; (b) Side elevation view; (c) Isometric view

the first story consists of 10 total glulam columns, all grade L2, with four columns being continuous to the roof. The first story includes four grade 24F-V8 longitudinal beams, and nine grade 24F-V4 lateral beams with beams and columns connected using commercial connectors [29], which can be seen in Figure 3. The roof of the structure has a similar column layout to the first story with 10 total glulam columns, also grade L2, with the four continuous columns terminating at the roof, and eight discontinuous columns located above their respective first story counterparts. The roof beam system was significantly different than the first story with only longitudinal beams being used. There are a total of six beams, four grade 24F-V8, and two grade 24F-V4 which are also connected using similar connectors as the first story.

Figure 3. First story gravity frame (Photo credit: NHERI TallWood Team).

Similar to the gravity frame, the diaphragms for the first story and the roof were quite different with the objective of investigating two different diaphragm systems throughout the course of the testing [4]. The first story panels were all 3-ply grade V1 CLT panels, although the size of the panels varied. The diaphragm can be thought of as two parts, the interior and exterior diaphragm panels. The exterior diaphragm panels acted as cantilevers on the outside edge of either shear wall stack, while the interior diaphragm panels were located in between the two shear walls. A total of eight 1.5 m x 6 m (5 ft x 20 ft) panels were used for the exterior of the diaphragm, while eight 1.5 m x 2.75 m (5 ft x 9 ft) panels were used for the interior. The exterior panels were oriented longitudinally, while the interior panels were oriented laterally as seen in Figure 2 (a). The roof diaphragm was a composite consisting of CLT panels with a light weight concrete topping slab. The CLT panels were 1.5 m x 6 m (5 ft x 20 ft) [with the exception of two 1.2 m x 6 m (4 ft x 20 ft) panels] 5-ply grade V1 panels oriented laterally along the length of the structure as seen in Figure 2 (a). The concrete layer was also a 70 mm (2 ¾ in) light weight concrete topping on the CLT panels with composite action being achieved with 45-degree anchors installed into the CLT panels. Figure 4 shows the roof diaphragm before the concrete topping was poured

Figure 4. Roof diaphragm prior to pouring concrete (Photo credit: NHERI TallWood Team).

The SFRS system used for the testing varied for each phase, but retained the same core components, namely CLT panels connected with metal hardware to make up a longer wall at each story level. The SFRS system consisted of two parallel shear wall system stacks spanning the entire height and located equidistant from the center of the structure for symmetry (see Fig 2). The system was designed assuming that the tie-down rods would resist the tension caused by the overturning moment, while inter-panel connectors and generic angle brackets (both using nails) would transfer shear [2]. The assumption of pure shear in the base connector brackets described later is not perfect, but is a design assumption being embedded into the design approach for platform CLT construction standards in the U.S. for simplicity of design. The loading used in the design was derived using ELF procedure as described in ASCE 7 [3], and design spectrum values of 1.5g for S_{DS} and 1.0g for S_{D1} obtained for a San Francisco, California location were used.

The NHERI@UCSD Shake Table is a 7.6m x 12.2 m (25 ft x 40 ft) uniaxial table equipped with two actuators with a maximum payload of 20,000 kN (4,496 kips). This location has been used for many previous tests, and the full details of the shake table can be found in [18]. The footprint of the test structure was larger than the table in the direction perpendicular to the actuator (direction perpendicular to shaking), so large steel outrigging beams were used as a way to extend the table to the appropriate dimensions. In order to ensure safety, two steel towers were installed in front of the control building. A third tower was installed on the east side of the structure with safety straps wrapping around the center of the structure, with the straps remaining slack unless the structure became unstable and began to collapse, again for additional safety. This extra measure was implemented due to concerns that the structural elements of the test structure could have been damaged from the previous stages' motions. Figure 5 shows the orientation of the structure on the shake table and the location of the safety towers.

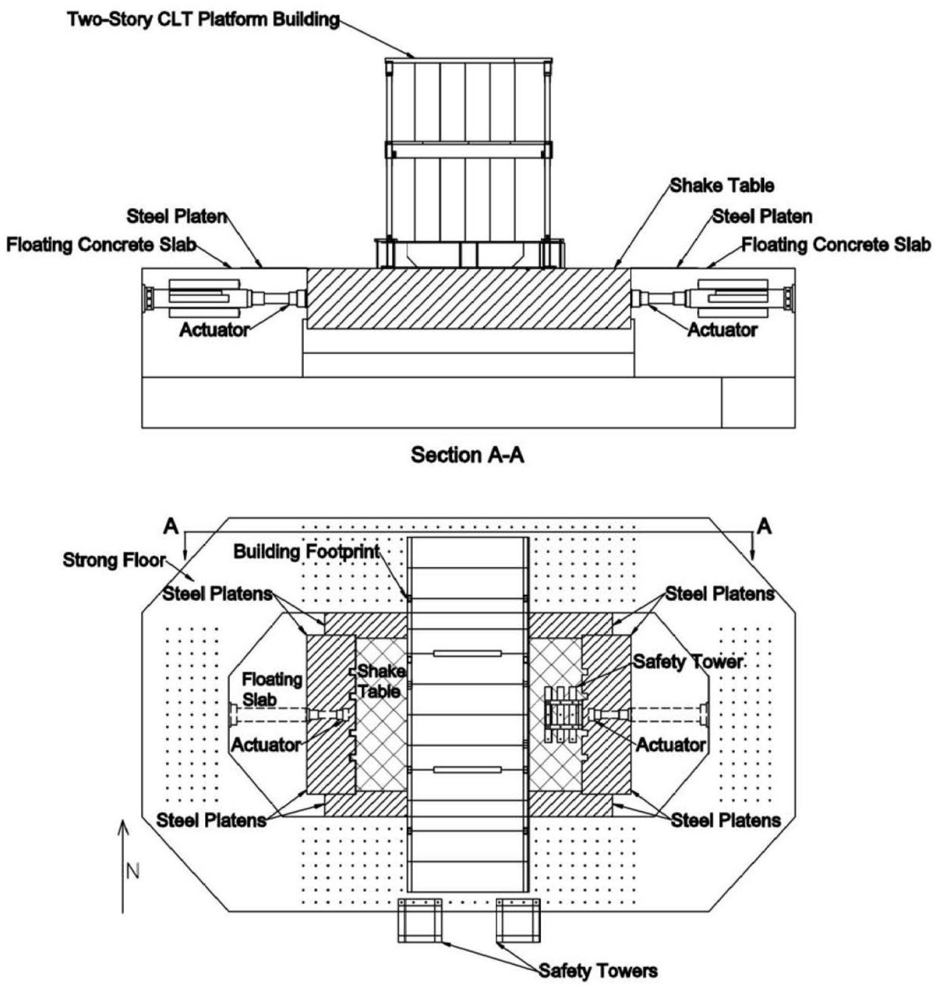
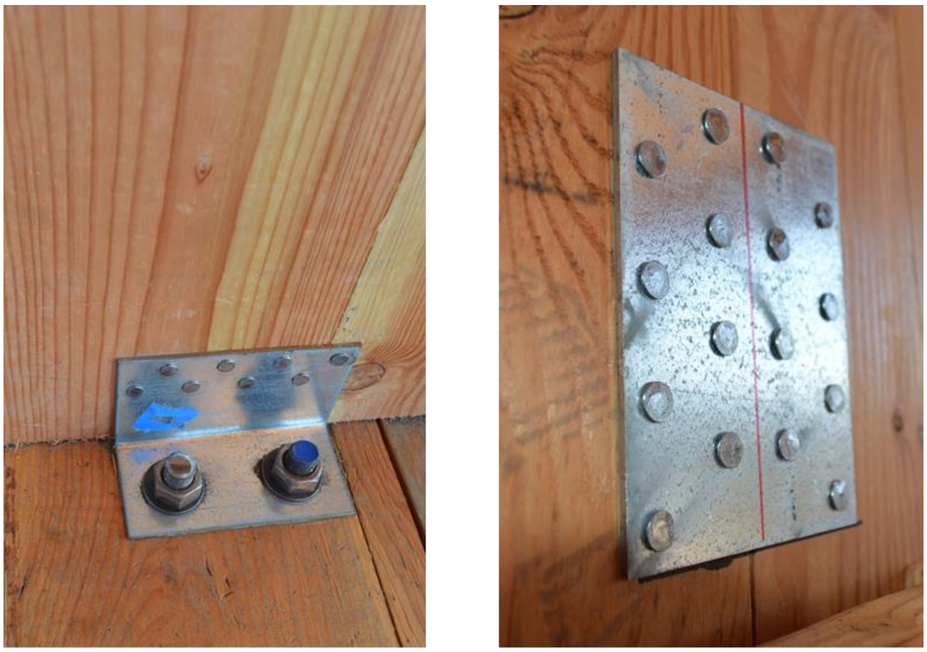


Figure 5. Position of the two-story CLT Platform Building on the shake table

Phase 1 Description

The purpose of dividing the testing into three different phases was to be able to test and compare three different wall configurations. Each configuration consisted of a north and south wall system, with a first-floor wall height of 3.2 m (126 in) and roof wall with a height of 2.8 m (110 in), as well as both shear connectors at the top and bottom of the wall and overturning restraint. The Phase 1 walls were comprised of four 0.9 m x 3.2 m (36 in x 126 in) CLT panels with a thickness of 105 mm (4-1/8 in), resulting in a total wall length of 3.7 m (144 in), which is shown in the schematic of Figure 6. The shear resistance for the wall consisted of inter-panel vertical shear connectors and base/top shear connectors. The base/top shear connectors were generic 76mm x 57mm x 3 mm (3 in x 2-1/4 in x 3/25 in) L angle brackets with a length of 121 mm (4-3/4 in) with the number varying between the first and roof based on shear demand from the design approach. Figures 6 (a and b) show a typical inter-panel connector and angle bracket respectively. On the first story, the brackets were placed to ensure that each CLT panel was secured



(a) (b)

Figure 6. Connectors: (a) Generic angle bracket; (b) inter-panel connector (Photo credit: NHERI TallWood Team).

by brackets on each side of the wall, resulting in a total of eight brackets per panel and 32 per wall on the first story. The brackets on the roof walls were only located on the outside face of each wall and were spaced to ensure three brackets per panel on both the base and top, resulting in a total of 6 brackets per panel and 24 per wall. The brackets were secured to each panel using 16 16D nails (4.2 mm diameter x 89 mm) and to the diaphragm by two 19 mm (3/4 in) diameter fully threaded, A36 lag bolts. It should be noted that the brackets at the base of the first-floor walls were connected to a steel beam secured to the shake table instead of a CLT diaphragm. The inter-panel shear resistance consists of inter-panel connectors vertically spaced at 406 mm (16 in) on center. These connectors transfer the shear between each individual panel and are secured by eight of the same 16d nails described earlier per panel. These inter-panel connectors are placed on either side of each of the three inter-panel joints in the walls, resulting in a total of 14 per vertical joint on the first story (total of 42) and 10 per vertical joint on the roof (total of 30). The overturning moment resisting system was comprised of vertical steel rods the full height of the building. The tie down rods were located at the end of each wall on both sides, for a total of four per wall and eight total and were allowed to reach at each floor (second flood diaphragm and roof level in this case) using a bearing plate to distribute the load. The diameter of the rods was not constant throughout the height of the building, with a coupler located above the first story bearing plate reducing the diameter from 31.8 mm (1-1/4 in) at the base to 19 mm (3/4 in) at the roof. The tie-down rods were not designed for compression loading, only tension, so to prevent buckling the tie-down rods were allowed to slip through the oversized holes at the diaphragm when the corresponding side of the shear wall was in compression. The SFRS configuration for Phase 1 is presented in the schematic of Figure 7 and a photograph of a second floor wall with the connectors and tie-down rods installed (including bearing plates) can be seen in Figure 8.

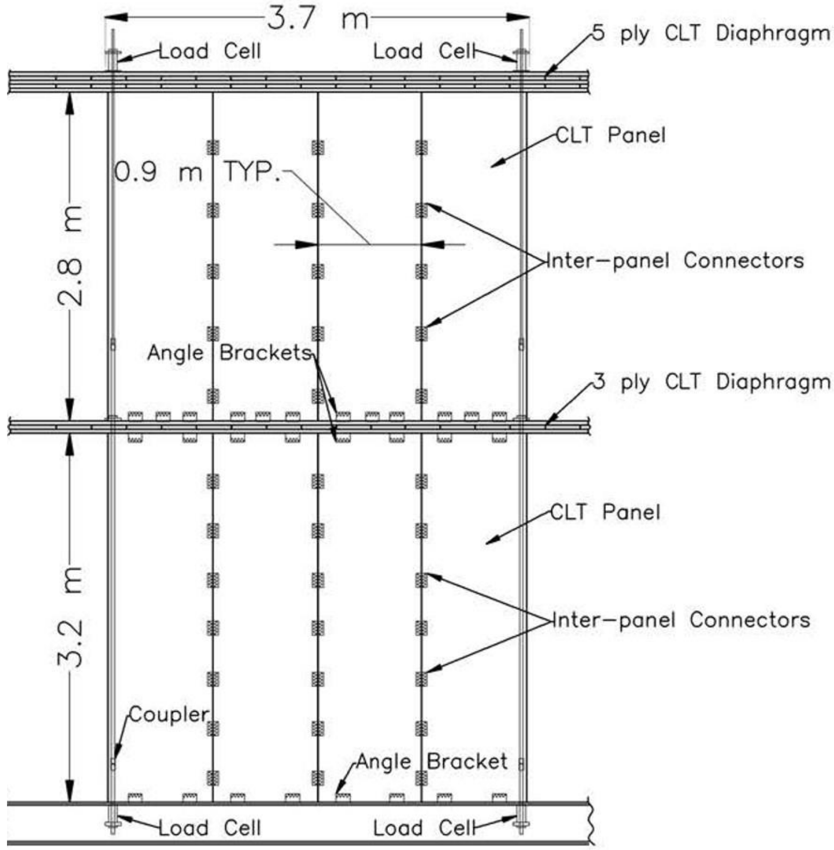


Figure 7. Phase 1: 3.5:1 aspect ratio panels shear wall stack layout

Figure 8. Phase 1: 3.5:1 aspect ratio second floor shear wall (Photo credit: NHERI TallWood Team).

Phase 2 Description

The Phase 2 wall system was similar in composition to the Phase 1 wall system, consisting of two walls and general shear and overturning restraint systems. The difference from Phase 1 lies in the CLT wall panels and their effect on the size and behavior of the systems. The wall configuration for Phase 2 consisted of two 1.52 m x 3 m (60 in x 126in) CLT panels with a thickness of 105 mm (4-1/8 in), resulting in a total wall width of 3 m (120 in) and an aspect ratio of 2.1:1, which can be seen in Figure 4. The shear force was resisted in a similar fashion to Phase 1, with angle brackets to transfer shear to the diaphragm (or steel beam in the case of the base) and inter-panel connectors to transfer shear between panels. Similar to Phase 1, the number of brackets per panel varied between the first and roof. The first story had 16 angle brackets per panel with four located on the base and top of the panel on both sides. The second consisted of 12 brackets per panel with three brackets on the base and top of the panel as well as on both sides. The brackets were secured to the panels and diaphragm in a similar way to Phase 1, using the same nails and bolts. The number of inter-panel connectors also varied between each story, with the first story having eight inter-panel connectors on each side of the joint between panels (total of 16) and six per joint on the roof (total of 12). The overturning moment resisting system consisted of the same basic components as Phase 1, with the tie-down rods transferring tension to the bearing plates on each floor and a coupler reducing the size of the rod after the bearing plate on the first story as seen in Figure 9. However, the rod was reduced to a 22.2 mm (7/8 in) diameter tie-down rod instead of a 19 mm (3/4 in) tie-down rod due to larger expected overturning moment, i.e. a function of aspect ratio of the panels making up the wall system for Phase 2 as shown in Figure 10.

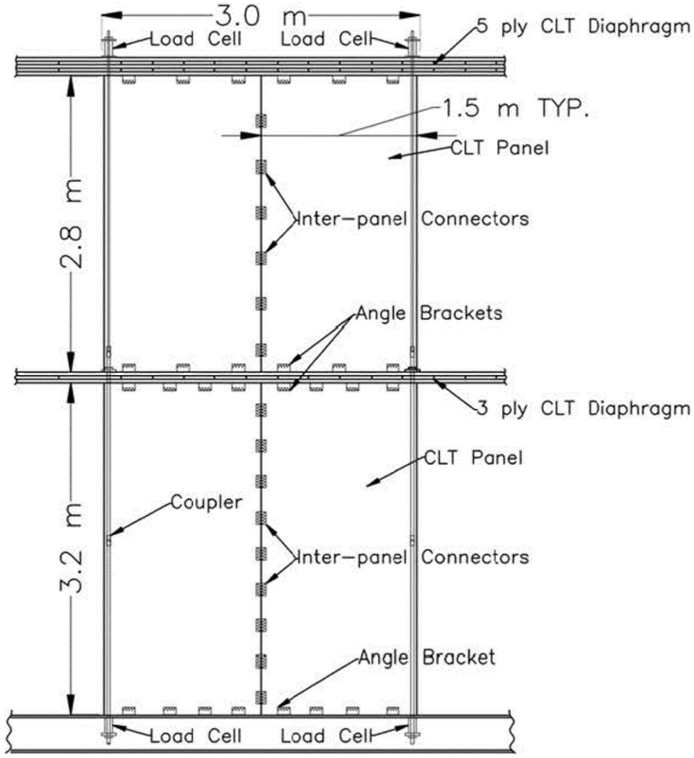


Figure 9. Phase 2: 2.1:1 aspect ratio panels shear wall stack layout

Figure 10. Phase 2: 2.1:1 aspect ratio first story shear wall layout (Photo credit: NHERI TallWood Team).

Phase 3 Description

The Phase 3 wall system was identical to the Phase 1 wall system except for the addition of transverse CLT wall panels added to both end of each wall stack as seen in Figure 5 (a). The transverse panels were added to examine the effect of these transverse walls on seismic performance. Two different dimensions of transverse panels were used, one for each story, primarily because of constructability issues with the diaphragm, which recall was already in place from different test programs earlier. The first story transverse walls were CLT panels with the same dimensions of the shear panels, but located on either end of the wall configuration, for a total of four transverse walls on the first story. It should be noted that due to existing gravity frame beams, a 533 mm x 483 mm (21 in x 19 in) section was removed from the top corner of each panel to fit around the beams. The walls were secured to the diaphragm on the top and the steel beam at the base by six angle brackets with three for the top and base respectively. The roof transverse walls were CLT panels identical to the shear wall panels, but located on either end of the wall configuration. These panels were secured to the top and base diaphragms by three angle brackets each. The Phase 3 wall system can be seen in Figures 11 and 12.

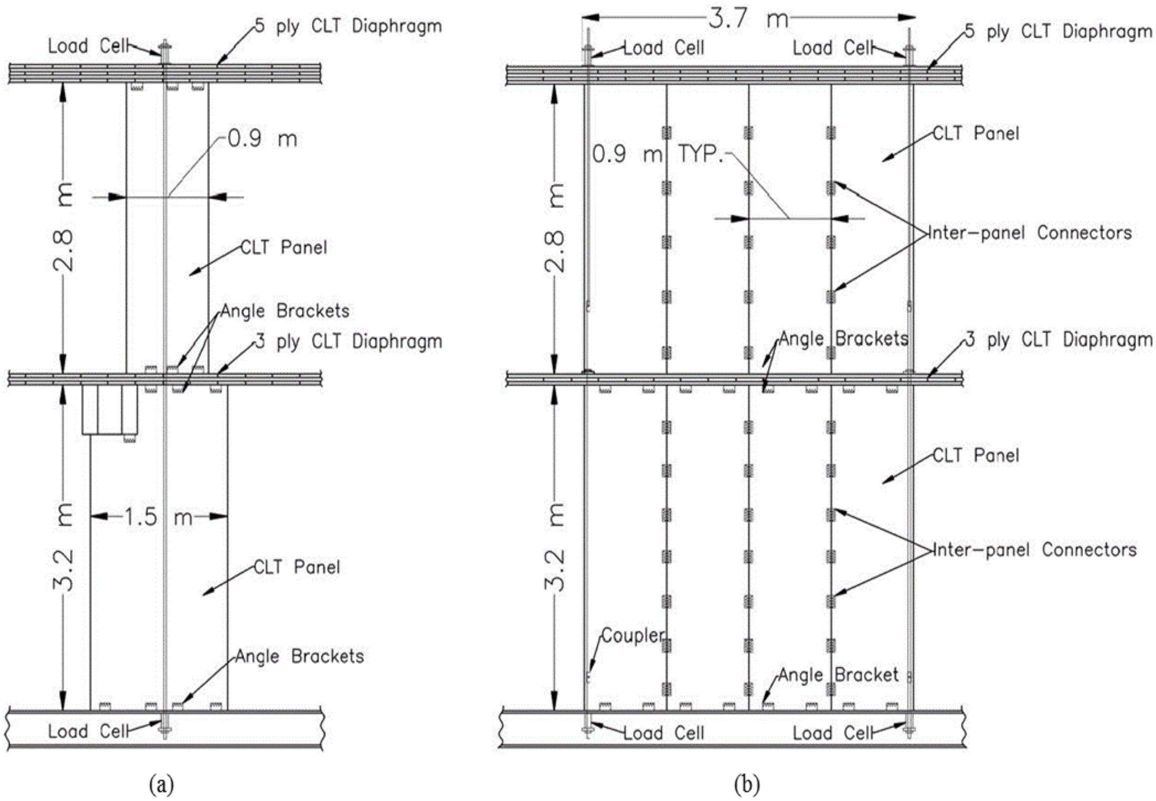


Figure 11. Phase 3: (a) Transverse CLT walls; (b) 3.5:1 aspect ratio panels shear wall stack layout

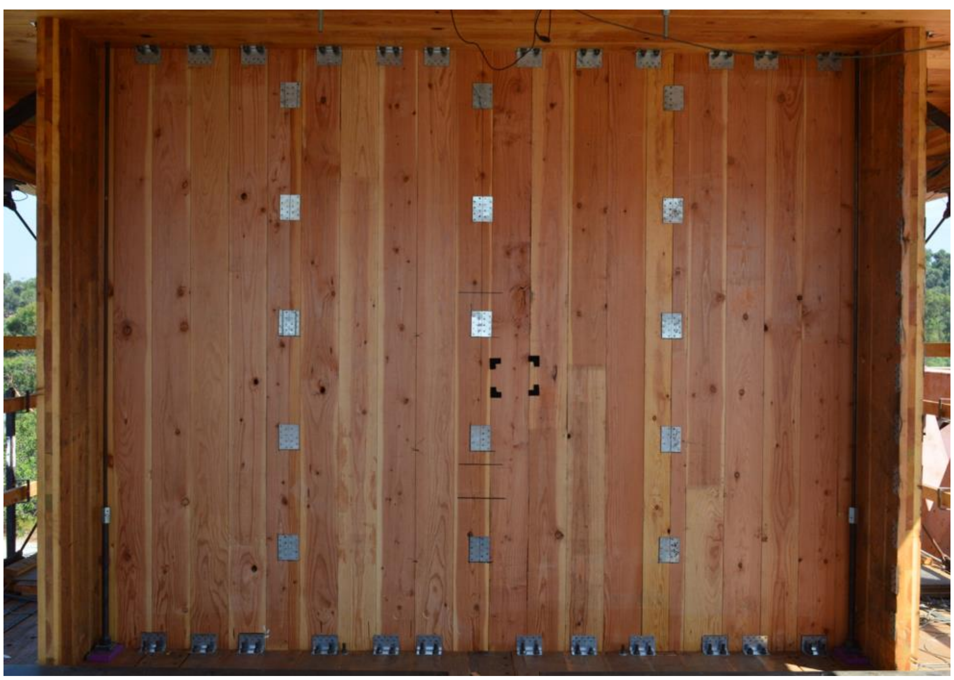


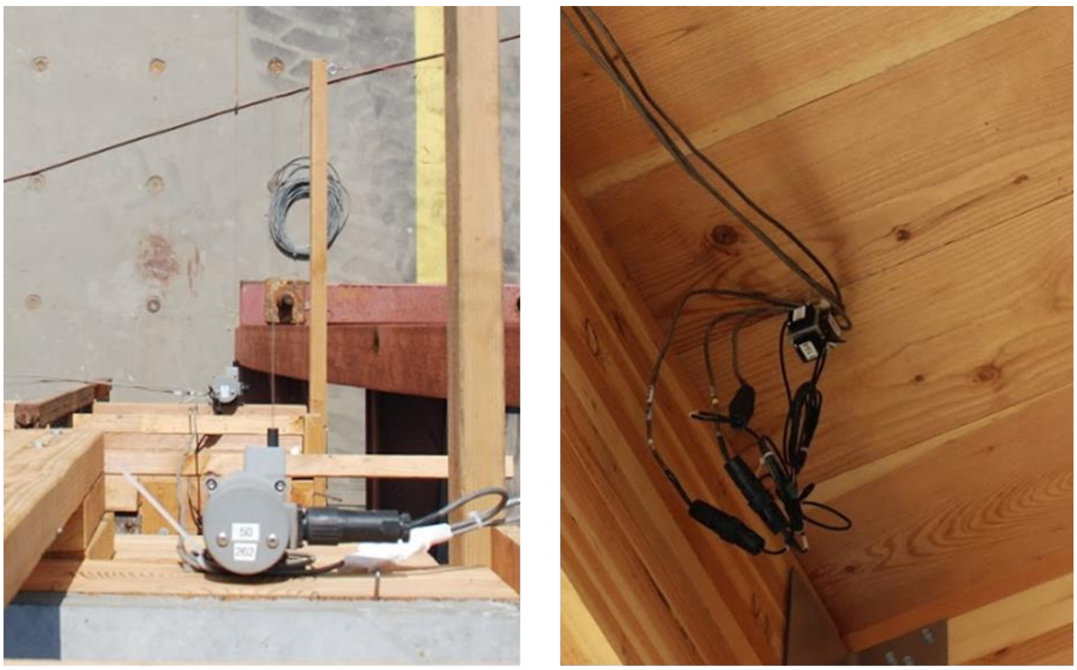
Figure 12. Phase 1: 3.5:1 aspect ratio with transverse walls, second floor shear wall layout (Photo credit: NHERI TallWood Team).

Instrumentation

The response of the building during shaking was recorded by over 300 sensors placed in strategic locations throughout the building. As mentioned previously, the structure was part of a collaborative test program with three stages and throughout the testing, the sensors installed to measure the response of the gravity frame and diaphragms remained constant. However, the sensors installed on the SFRS varied from each stage and in some cases within the stage. The diaphragms and gravity frame had 274 sensors installed throughout both floors of the building, and the quantity of each type of sensor can be seen in Table 1. Strain gauges were used to measure the deformation in the chord splices on the diaphragms as well as the rebar installed in the concrete of the composite roof. Linear potentiometers were used to measure the relative displacement between various components of the structure such as: CLT panels and the diaphragm, the diaphragm and gravity frame, and between the concrete and CLT panels in the composite roof diaphragm. String potentiometers were installed in the center of the diaphragm as well as on each corner of both floors to measure the global displacement of the building, a typical installation can be seen in Figure 13 (a). They were also used to measure the relative vertical displacement between floors and the relative vertical and horizontal displacement of the two farthest corners of the structure. Three directional accelerometers units were installed to measure the acceleration in the X, Y, and Z directions, and were installed in a similar fashion on each floor with one located at each corner of the diaphragm, as well as at quarter points along the centerline of the diaphragm. A typical installation of accelerometer block can be seen in Figure 13 (b).

Table 1. Instrumentation on gravity frame and diaphragm

Instrument	Quantity
Strain Gauge	133
Linear Potentiometers	63
String Potentiometers	42
Accelerometer	36



(a) (b)

Figure 13. Typical installation: (a) diaphragm to safety tower linear potentiometer; (b) diaphragm accelerometer (Photo credit: NHERI TallWood Team).

Figure 14 shows the location and details of the typical Phase 1 wall instrumentation. A total of 72 sensors were installed for Phase 1 (see Table 2) on both the north and south walls systems to capture the response behavior as well as to confirm the symmetric nature of the structure. Linear potentiometers were installed horizontally on the base and top of the first and second stories on both the north and south wall systems in order to capture any sliding motion as shown in Figure 15 (a). They were also installed vertically at each corner on the first and second stories on the south wall system to capture the uplift behavior of the walls. However, due to constraints on the quantity of sensors, the north wall system only had vertical linear potentiometers on two corners of the walls on both the first and roof to demonstrate the symmetry of the system. String potentiometers were installed in the horizontal and vertical directions on two of the three joints between the CLT panels on each of the four walls that comprised the north and south wall systems, as shown in Figure 15 (b) with the purpose to measure any relative panel displacement in either the vertical or horizontal directions. Two string potentiometers were also installed diagonally in opposing directions on both a first and roof wall on the north wall system to measure any panel deformation as shown in Figure 15 (c). A total of 16 load cells were installed under the anchor bolts on the base and top of each tie-down rod [as seen in Figure 15 (d)] on both the north and south wall system in order to measure the tension in the rods caused by the overturning moment. Strain gauges were also installed strategically along the lengths of the rods as a redundant measurement to calculate the tension in the tie down rods in case of load cell failure.

Table 2. Instrumentation on shear walls.

	Quantity for each phase		
Instrument	1	2	3
Strain Gauge	16	4	3
Load Cell	16	16	16
Linear Potentiometer	20	20	20
String Potentiometer	20	20	20
Accelerometer	0	6	0

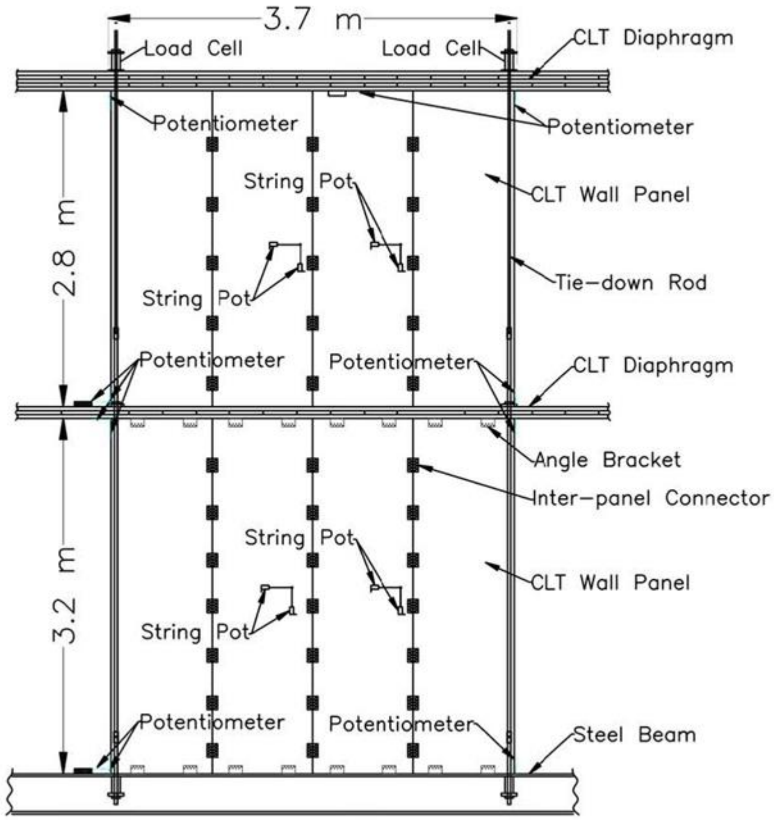


Figure 14. Phase 1 instrumentation on south face of south shear wall (typ. across phases)

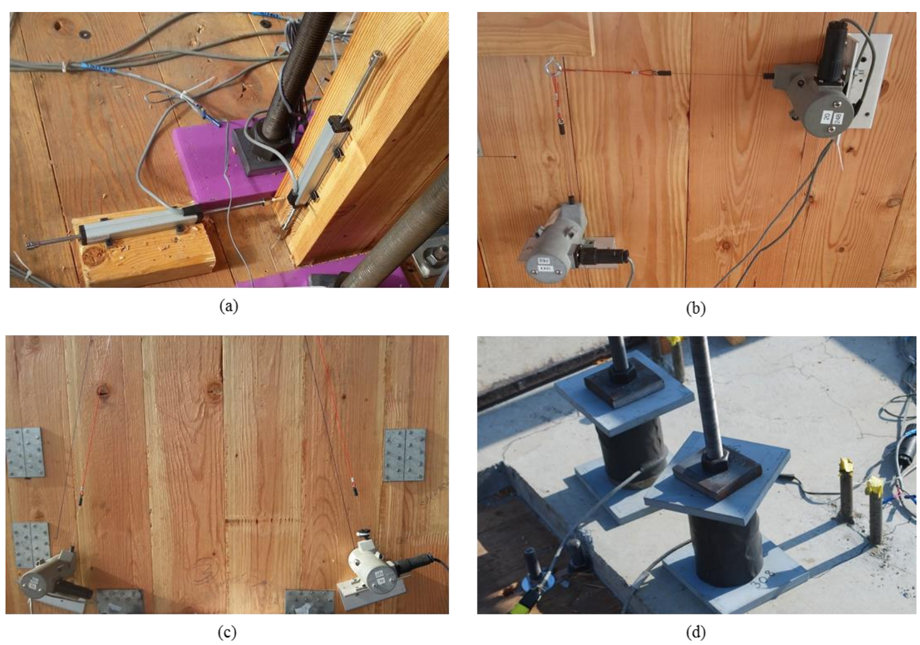


Figure 15. Typical installation: (a) linear potentiometers; (b) string potentiometers relative displacement; (c) string potentiometers panel deformation; (d) load cells (Photo credit: NHERI TallWood Team).

The Phase 2 SFRS sensors were installed in a similar fashion to Phase 1, however there were some minor differences due to the smaller wall dimensions and fewer CLT panels making up each wall. With only one inter-panel joint in each wall, there were several unused string potentiometers previously utilized in Phase 1 for relative panel deformation. These sensors were instead used to measure panel deformation on additional panels on both the north and south wall systems. All four panels comprising the two-story north wall system were measured for panel deformation as well as one panel from each floor on the south wall system. The total quantity of sensors for Phase 2 can be seen in Table 2.

The Phase 3 string potentiometers, load cells, and strain gauges were installed in the same locations as Phase 1, however the locations of some of the linear potentiometers changed due to the addition of transverse walls on either end of all four walls comprising the north and south wall systems. Some of the sensors measuring sliding were moved from the end of the walls towards the center due to the previous locations being blocked by the installed transverse walls. One potentiometer from each floor on the south wall system was moved to the end of the transverse wall on their respective floors to capture any potential uplift. The total quantity of sensors for Phase 3 can also be seen in Table 2.

Ground Motion and Testing Program

The 1989 Loma Prieta earthquake record was used and scaled to various intensities for all testing. The scaling of the motion was done according to the methodology in FEMA P695 for a location in San Francisco, California. Intensities were selected based on levels corresponding to a service level earthquake (SLE), design base earthquake (DBE), and maximum considered earthquake (MCE), with mean return periods of 72 years, 475 years, and 2475 years respectively. For the MCE motion, this led to a S_{DS} and S_{D1} of 1.5 g and 1.0 g respectively, and the scaled response spectra is presented in Figure 16, while the spectral accelerations for each phase shown in Table 3. Due to time constraints for repairs, Phase 2 and 3 testing used only the SLE and MCE scaled ground motions, which optimized the amount of data collected with a minimal amount of repair.

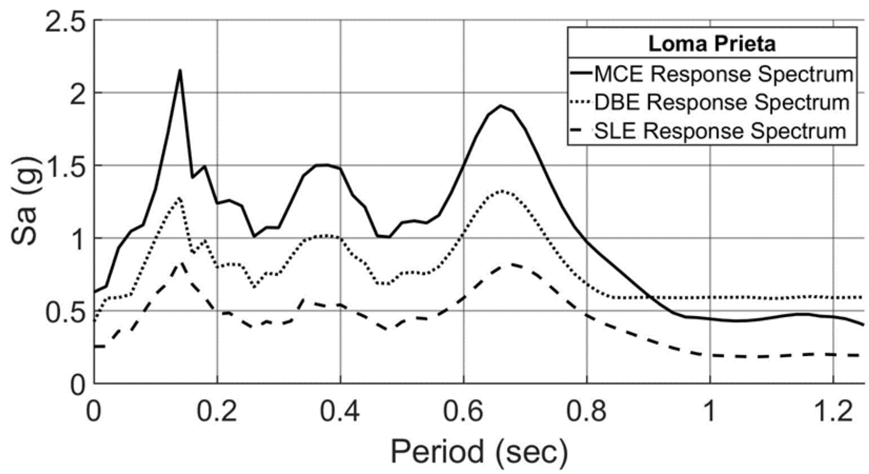


Figure 16. Spectral accelerations for Loma Prieta scaled to SLE, DBE, and MCE levels respectively

Table 3. Test Sequencing and Global Story Response for Each Phase

				Average Peak Displacement[c,d] (mm)		Peak Average Interstory Drift[e] (%)		Peak Story Shear[f,g] (kN)	
Phase[a]	Test[b]	S_a (g)	PGA (g)	Story 1	Story 2	Story 1	Story 2	Story 1	Story 2
1	SLE	0.525	0.25	18	29	0.49	0.42	306	206
	DBE	0.92	0.42	51	94	1.40	1.40	582	456
	MCE	1.36	0.63	99	158	2.70	1.95	765	579
2	SLE	0.54	0.24	17	32	0.47	0.54	308	219
	MCE	1.29	0.66	93	159	2.54	2.15	873	643
3	SLE	0.68	0.265	40	48	1.08	0.59	301	190
	MCE	1.49	0.68	89	151	2.42	2.06	873	645

[a]Phase 1 (4:1 aspect ratio CLT shear walls), Phase 2 (2:1 aspect ratio), Phase 3 (4:1 aspect ratio with additional transverse walls).

[b]Loma Prieta earthquake ground motion was used for all tests, and scaled appropriately to SLE, DBE, and MCE levels respectively.

[c]Average of the displacements recorded at the center, and both ends of each story.

[d]1 in=25.4 mm.

[e]3,663 mm (144 in) and 3,043 mm (120 in) were used for the height of the first and second story respectively.

[f]342.5 kN (77 kips) and 422.5 kN (95 kips) were used for the weight of the first and second story respectively.

[g]1 kip=4.448 kN.

The natural period of the test structure varied between tests due to damage sustained to the gravity frame, repairs conducted on the shear wall stacks, and different wall configurations between phases, and because of this, white noise tests were conducted before and after every test in order to determine the natural period. Figure 17 presents a plot of the change in the natural period of the structure over the platform CLT testing program. Initially with Phase 1 (3.5:1 aspect ratio shear walls), the test structure had a natural period of approximately 0.38 seconds, which increased following testing with the peak natural period following Phase 1 being approximately 0.89 seconds. The installation of the Phase 2 shear walls, which had an aspect ratio of 2.1:1, returned the natural period of the structure to approximately 0.41 seconds. The peak natural period following Phase 2 testing was 0.73 seconds after the MCE level test. Phase 3, with the return to the 3.5:1 aspect ratio shear walls, and the addition of transverse CLT walls, reduced the natural period of the structure to 0.17 seconds. This decrease is most likely due to the additional stiffness provided by the transverse CLT walls, including some potential "flanging" action during smaller deformations in and near the elastic range. The peak natural period for the structure following Phase 3 testing after the MCE level test was the shortest of the phases at 0.35 seconds. This indicates that the least amount of softening occurred in Phase 3 with the transverse walls in place.

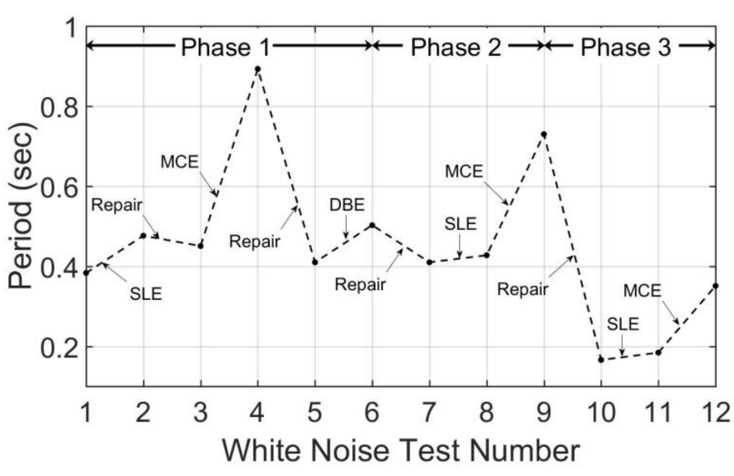


Figure 17. Fundamental building period and the effect of repairs and different wall configuration

CHAPTER 3 – RESULTS

Displacement Profile

Figure 18 presents the displacement profile of the test structure for each test, with the profile constructed using the average maximum displacement of the story relative to the shake table. The horizontal displacement was measured at the north, south, and center of the diaphragm on each story. Across all of the phases, primarily a first mode response was observed, with the peak interstory drift for each story occurring simultaneously to the maximum displacement. Phase 1 and Phase 2 had similar peak MCE level average displacements of 158 mm (6.22 in.) and 159 mm (6.26 in) respectively, and Phase 3 had the smallest peak average displacement of 151 mm (5.94 in.). As mentioned previously, the objective of the testing was to demonstrate the performance of CLT shear walls using typical design techniques, in this case ELF was used. In the design, 2/3 of the MCE level spectral acceleration is reduced by a seismic reduction factor (R), developed over the course of the project, to account for the nonlinear response of the structure, specifically through its deformation capacity and ability to dissipate energy. The nonlinear response is confirmed by the results in Figure 18, as the MCE level motion is 1.5 times greater than the DBE level motion, but the maximum displacement for the Phase 1 MCE test (158 mm) is greater than 1.5 times the maximum displacement for the Phase 1 DBE test (94 mm). It was theorized in Popovski et al. (2016) [24] that the addition of walls transverse to the shear walls would improve the performance of the structure, and as can be seen in Table 3 and Figure 9, even though Phase 3 MCE had the largest peak ground acceleration (PGA) recoded by the table, its maximum story displacements were less than Phase 1 and Phase 2 on both floors.

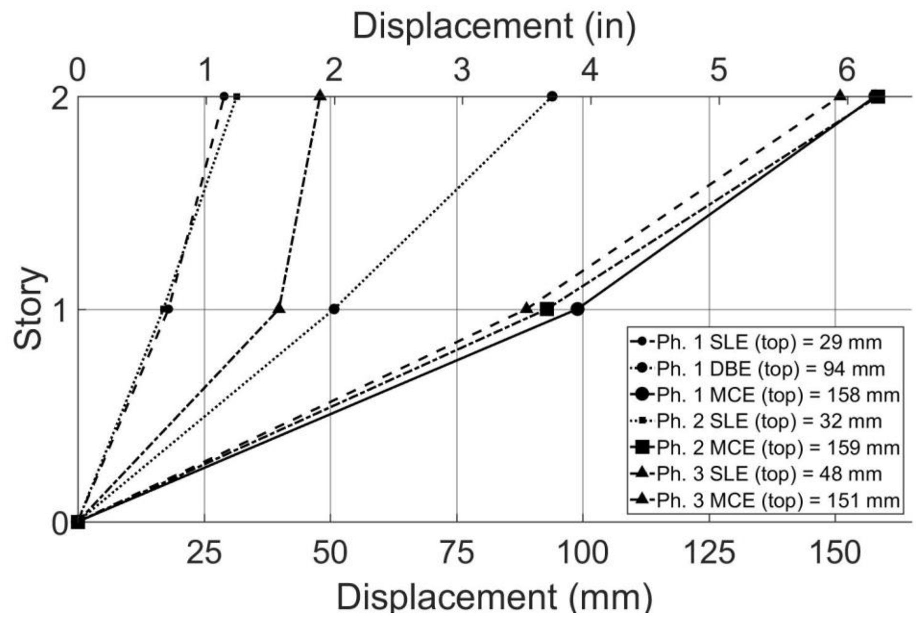


Figure 18. Average displacement of first and second stories for each test

Interstory Drift

The interstory drifts (ISDs) of the test structure were obtained by finding the average relative displacement of each story. For the first story this was done by taking the difference of the string potentiometers on the first story diaphragm and the table displacement feedback data, then averaging the results. The roof relative displacement was calculated in a similar way, except that instead the difference between the first and roof string potentiometers was used. The peak interstory drift was then divided by the relevant story height, 3663 mm (144 in) and 3043 mm (120 in) for the first and second stories respectively, to obtain drift as a percentage of the story height. Table 3 shows the ISDs for all of the tests, and it can be seen that the first story ISDs were larger than the roof, which may imply slight soft story behavior of the structure. This is particularly evident in the Phase 1 MCE level ISD results, the first story has an ISD 0.75% greater than the roof. It can also be seen in Figure 18 that the addition of transverse walls in Phase 3 improved the performance significantly of the first story even though the PGA was greater than in Phase 1.

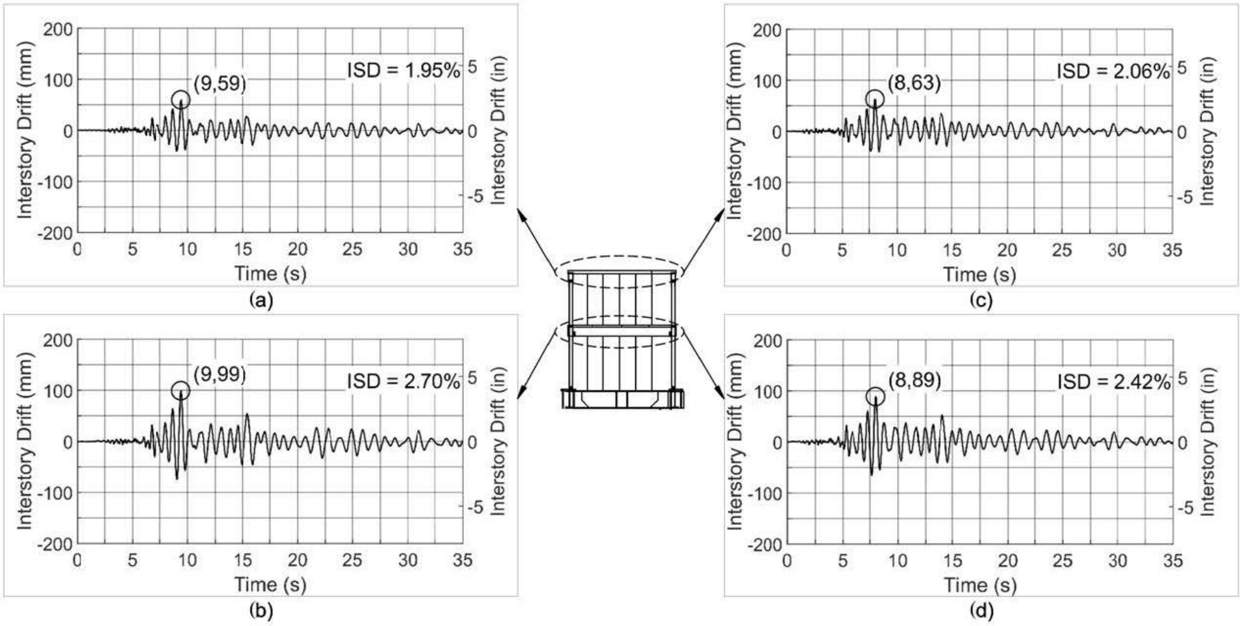


Figure 19. Interstory drift: (a) Phase 1 MCE roof; (b) Phase 1 MCE first story; (c) Phase 3 MCE roof; (d) Phase 3 MCE first story

Global Hysteresis

Newton's second law was used to calculate the inertial force of each story by using the average of acceleration time histories recorded on both floors at each corner and along the center of the diaphragm at the north edge, center, and south edge. The shear force for each story was then calculated accordingly, and Table 2 presents the shear force for all seismic tests and both stories. It can be seen that the largest shear force of 873 kN (196 kips) occurred during Phase 3 and Phase 2 MCE. Figure 20 compares plots of the floor displacements versus the story shear for Phase 3 MCE and Phase 1 MCE. Phase 1 and Phase 3 both had 3.5:1 aspect ratio panels, but Phase 3 also included the addition of transverse walls. This was done to simulate a real structure where there would be shear walls in both directions and investigate their effect on performance. As can be seen, while the story shear is higher in Phase 3, the displacements are either similar or less than Phase 1. Since the PGA was larger in Phase 3, this implies that the transverse walls had no negative effect on performance, and in fact improved performance of the structure, which was anticipated.

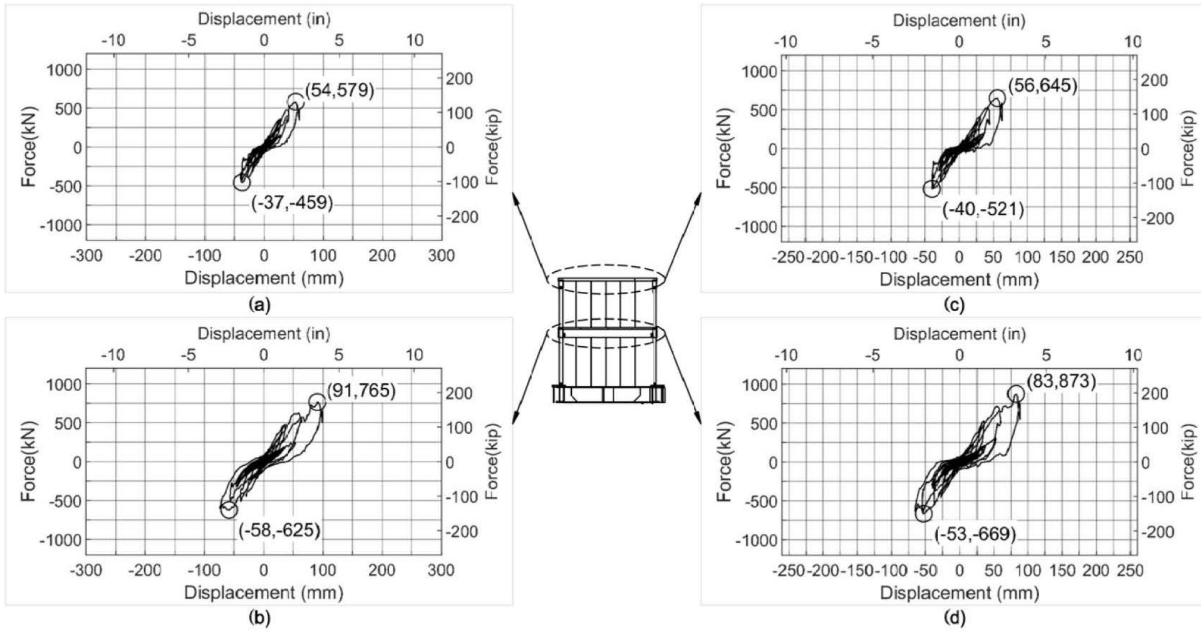


Figure 20. Global hysteresis curves: (a) Phase 1 MCE roof; (b) Phase 1 MCE first story; (c) Phase 3 MCE roof; (d) Phase 3 MCE first story

Torsion

As mentioned previously, the test structure was subjected to a large number of tests over three month period, with over 30 tests being conducted during the three stages, with several exceeding the MCE level presented in this paper. It is likely because of the number of repeated MCE and larger shakes, that torsion caused by some softening in the diaphragm was evident. Figure 21 shows the deformed shape of the diaphragm indicating some torsion, with the south end of the roof displacing 35 mm (1.38 in) more than the center of the structure. It can also be seen that the torsion on the first story was not as large (25 mm), it was still however present and it should be noted that the north end of the structure did not experience much torsion across either story. The torsion experienced by the structure increased further into the testing program, but despite this, the CLT shear wall stacks still performed well.

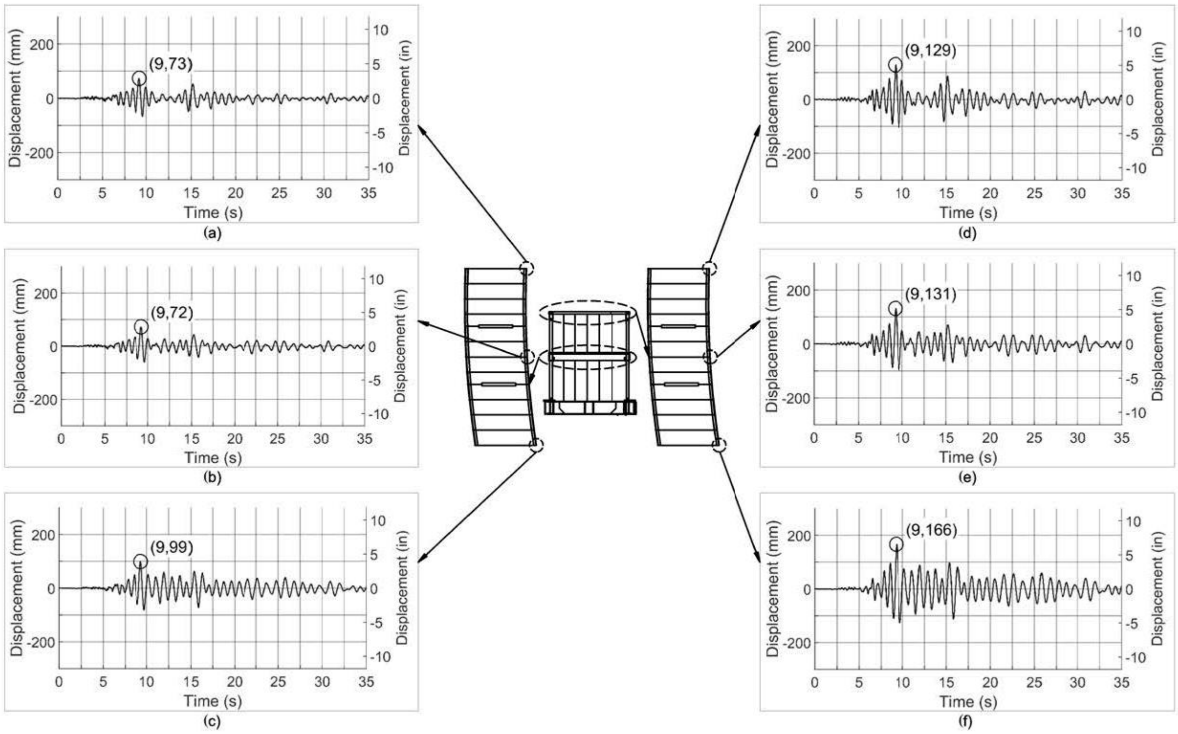


Figure 21. 3.1 MCE Displacement: (a) first story northeast corner; (b) first story center east side; (c) first story southeast corner; (d) roof northeast corner; (e) roof center east side; (f) roof southeast corner

CLT Panel Uplift

One of the principal objectives in the testing was to observe individual CLT panel behavior and investigate the effect of aspect ratio and in the case of Phase 3, the addition of transverse walls on that seismic behavior. To capture any potential uplift of the panels, linear potentiometers were used at the four corners of the shear wall on both the first and second stories, and Table 4 summarizes the results. In addition, for Phase 3, linear potentiometers were installed on the transverse walls to capture any potential uplift.   Figure 22 shows the uplift recorded on the west side of the south shear wall stack during Phase 1 MCE, and it can be seen that the bottom of the panels experienced the most rocking, with the first story having the largest uplift values. The top of the panels experienced smaller uplift and hence less rocking, which was expected. Figure 22 shows the uplift recorded during Phase 3 MCE on both floors of the south shear wall stack as well as the uplift recorded at the base of the transverse walls. There was concern that the addition of transverse walls on either end of the shear wall stack could inhibit the ability of the CLT panels to rock. Figures 23 (a and b) demonstrate that the CLT panels were still able to exhibit rocking behavior even with transverse walls present, and the behavior was similar to Phase 1. It was expected that the transverse walls would experience minimal uplift during testing, and Figures 23 (c and d) show that is indeed the case. The largest uplifts recorded during testing occurred during the Phase 2 MCE test, with a peak uplift of 24.4 mm (0.96 in) recorded at the bottom of the first story, but it should be noted that this measurement could have been affected by the nail shearing observed on the first story and even though the uplift was the largest in Phase 2, the test structure still had no risk of collapse. The uplift in Phase 2 followed a similar pattern to the other phases with the bottom of the CLT panel experiencing the most rocking behavior on both the first and second stories as shown in Figure 24.

Table 4. Test Sequencing and Local Story Response for Each Phase

		Peak Uplift[a] (mm)		Peak Vertical Relative Panel Displacement[b] (mm)		Peak Sliding[c] (mm)		Peak ATS Rod Force[d] (kN)	
Phase	Test	Story 1	Story 2	Story 1	Story 2	Story 1	Story 2	Story 1	Story 2
1	SLE	4.7	3.3	2.6	1.2	1.7	2.4	44.5	9.8
	DBE	13.2	12.5	7.8	4.7	8.1	9.0	105.1	31.3
	MCE	21.7	16.7	19.6	10.0	12.0	9.4	170.0	66.3
2	SLE	4.2	2.7	1.9	1.0	1.1	1.7	45.8	12.7
	MCE	26.1	12.2	13.6	9.9	56.1	32.7	237.6	74.4
3	SLE	3.7	2.7	2.1	1.4	2.6	1.1	31.6	10.2
	MCE	14.5	9.4	15.7	13.6	7.2	6.5	--	--

[a]Uplift was measure at the upper and lower corners of each wall on both the first and second story

[b]Vertical Relative Panel Displacement is the vertical displacement measured at the inter-panel joint between CLT panels

[c]Sliding was measured and the top and base of each wall on both the first and second story

[d]Load cells were placed on the top and base of each ATS rod.

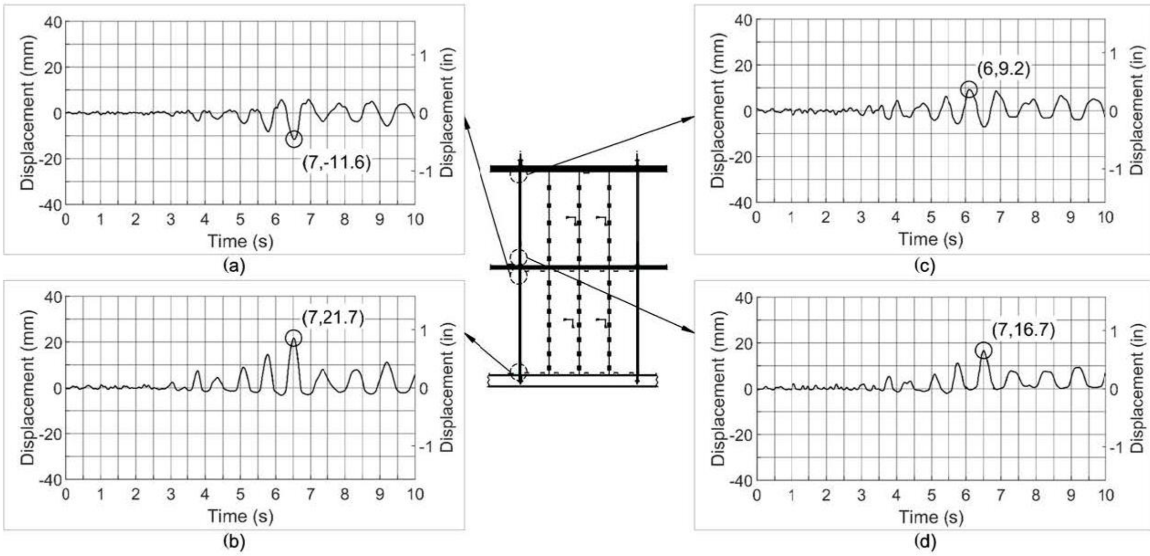


Figure 22. Phase 1 MCE Wall Uplift: (a) First story shear wall top west corner; (b) First story shear wall bottom west corner; (c) Roof shear wall top west corner; (d) Roof shear wall bottom west corner

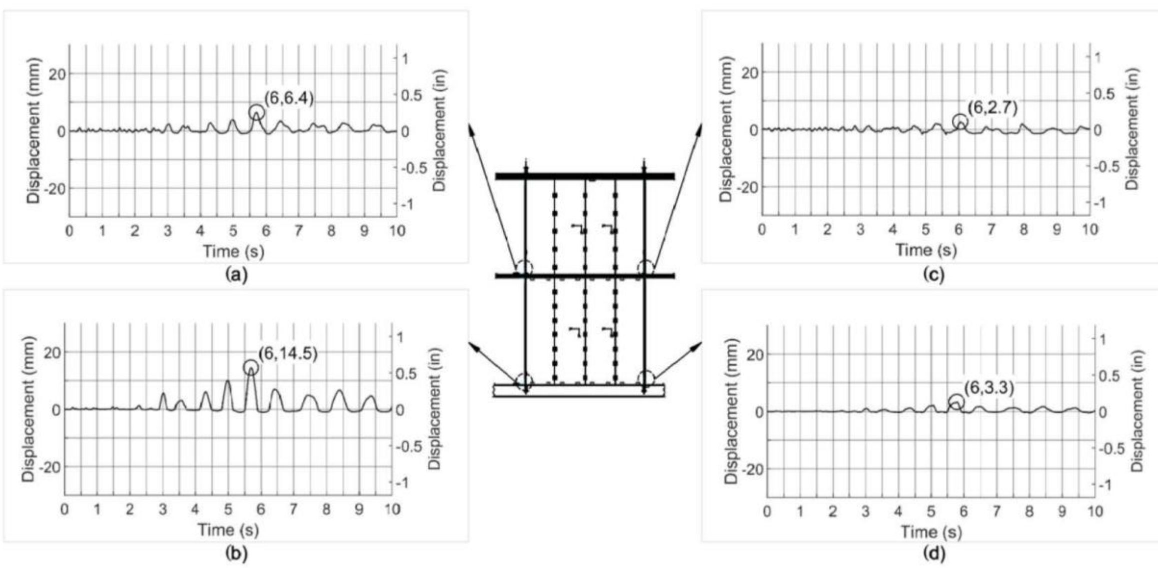


Figure 23. Phase 3 MCE Uplift: (a) Roof shear wall bottom west corner; (b) First story shear wall bottom west corner; (c) Roof transverse wall top south corner; (d) First story shear wall bottom south corner

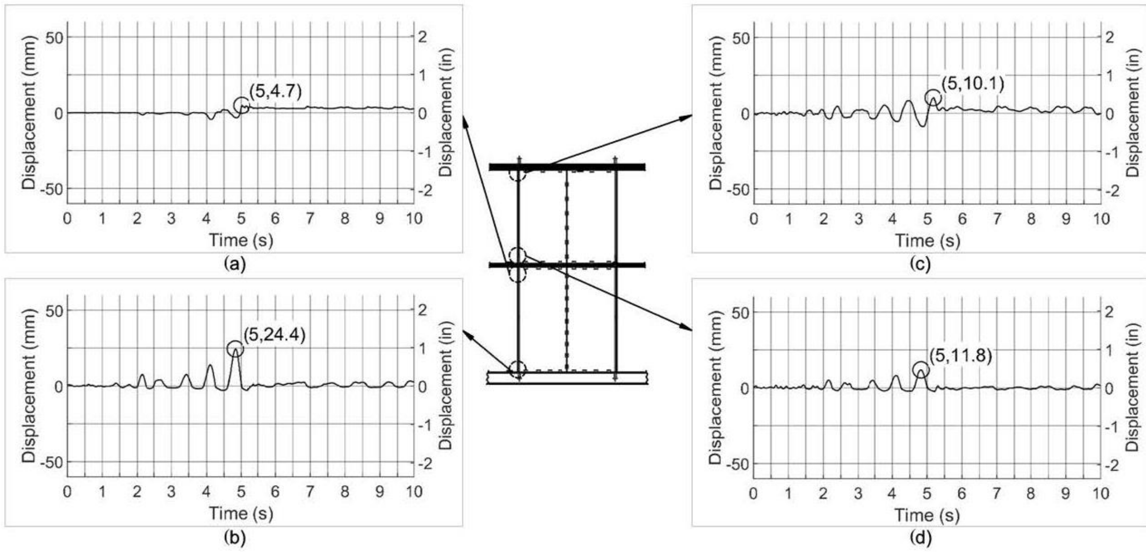


Figure 24. Phase 2 MCE Uplift: (a) First story shear wall top west corner; (b) First story shear wall bottom west corner; (c) Roof shear wall top west corner; (d) Roof shear wall bottom west corner

Sliding

The sliding of the shear wall panels was recorded using linear potentiometers positioned horizontally at the base and top of each shear wall, and Table 4 summarizes the results. The panel sliding followed a similar pattern to the uplift, with the largest sliding occurring at the base of the panel at each story, and this can be seen in Figure 25 for Phase 1 MCE. The sliding behavior can be partially explained by nail withdrawal as the deformation of the angle brackets work the nails from the CLT. This was especially present for the Phase 2 MCE test, where nail shear was observed on the first story, resulting in a sliding of 56.1 mm (2.2 in) as seen in Figure 26, but although the sliding was large, the test structure was in no danger of collapsing with the tie down rods providing uplift restraint and likely some level of shear through bearing on the hole they passed through in the CLT diaphragm and, in the case of the lower shear walls, the steel support beam acting as a foundation into the shake table. An example of nail withdrawal from Phase 1 MCE and nail shear from Phase 2 MCE can be seen in Figures 27 (a an b) respectively.

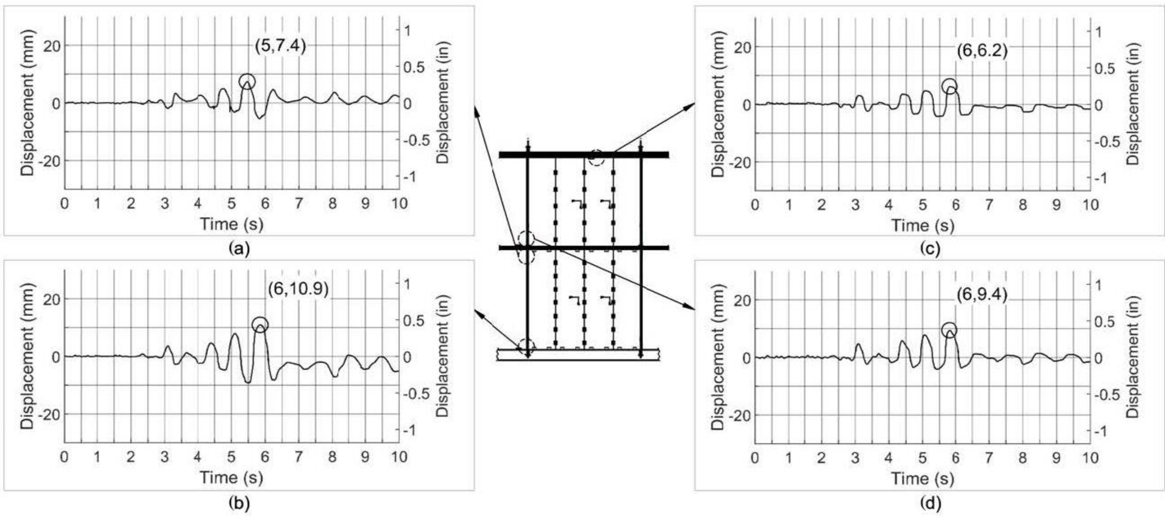


Figure 25. Phase 1 MCE Sliding: (a) First story shear wall top west corner; (b) First story shear wall bottom west corner; (c) Roof shear wall top west corner; (d) Roof shear wall bottom west corner

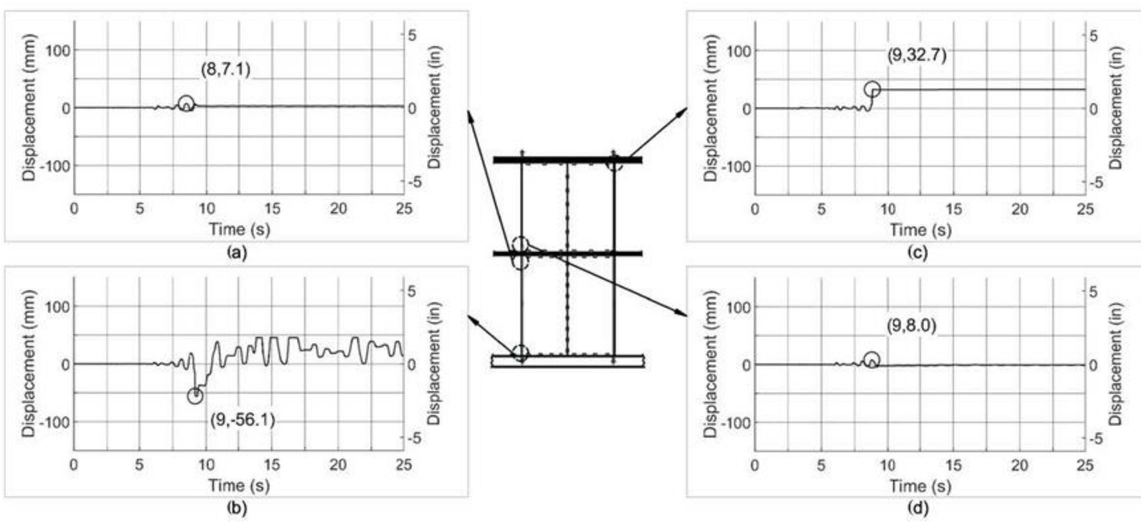
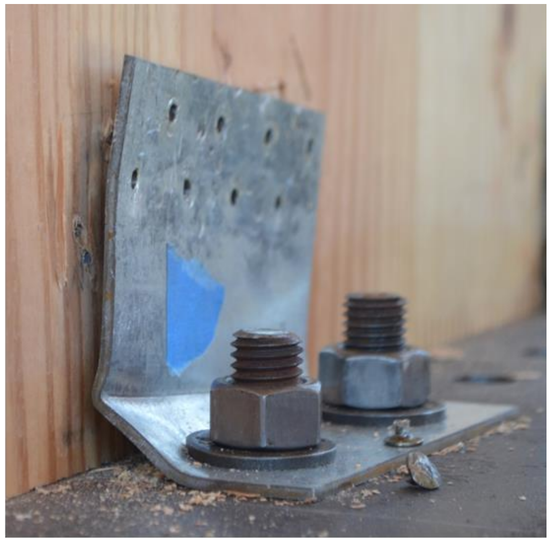


Figure 26. Phase 2 MCE Sliding: (a) First story shear wall top west corner; (b) First story shear wall bottom west corner (nail shear failure occurred in brackets); (c) Roof shear wall top west corner; (d) Roof shear wall bottom west corner

(a)



(b)

Figure 27. MCE angle bracket inspection: (a) Phase 1 first floor; (b) Phase 2 first floor (Photo credit: NHERI TallWood Team).

Relative Panel Displacement

Relative panel displacement was measured in the horizontal and vertical direction using string potentiometers at the inter-panel joints of each CLT panel for all tests and phases. No significant relative displacement was recorded or observed in the horizontal direction, so the vertical direction will be the primary focus, and Table 4 summarizes the results across the phases. Figure 28 presents the Phase 1 MCE results, and as can be seen, the first story experienced more relative panel displacement than the roof, which was expected given it deformed more overall. The displacement was also relatively consistent across the story with only a 3 mm (0.11 in) difference between the two recorded joints for Phase 1 MCE. The relative panel displacements for the 2.1:1 shear wall aspect ratio followed a similar trend to the 3.5:1 aspect ratio walls in Phase 1, the largest displacement occurred on the first story, this can be seen in Figure 29. Relative displacement between the panels was observed to cause nail withdrawal, and sometimes nail shear in the inter-panel connectors and thus dissipating energy. Examples of this can be seen if Figures 30 (a and b) for Phase 1 and Phase 2 respectively.

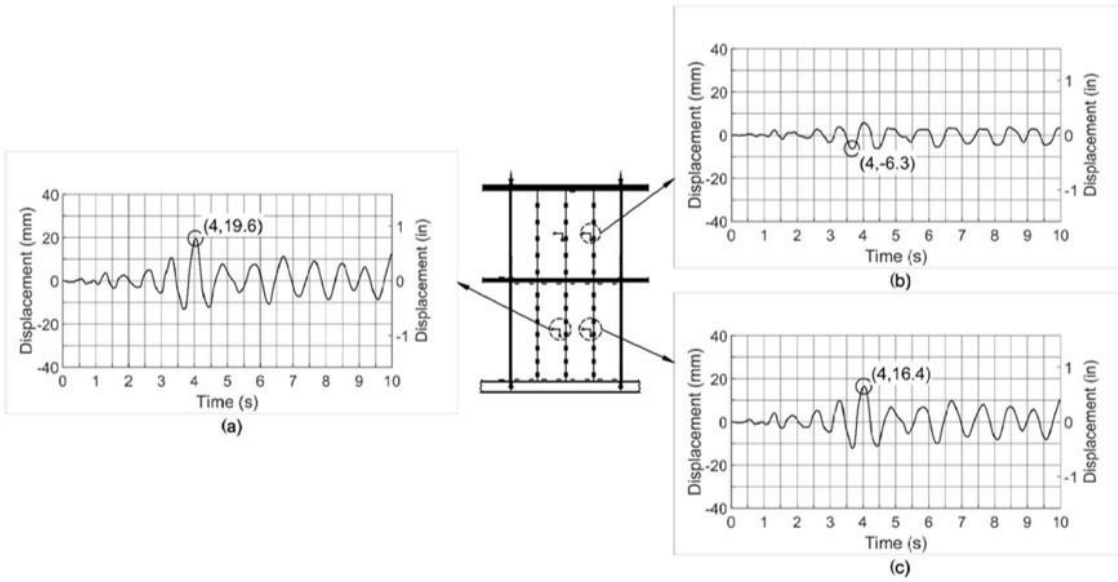


Figure 28. Phase 1 MCE Vertical Relative Panel Displacement: (a) First story shear wall top west corner; (b) First story shear wall bottom west corner; (c) Roof shear wall top west corner; (d) Roof shear wall bottom west corner

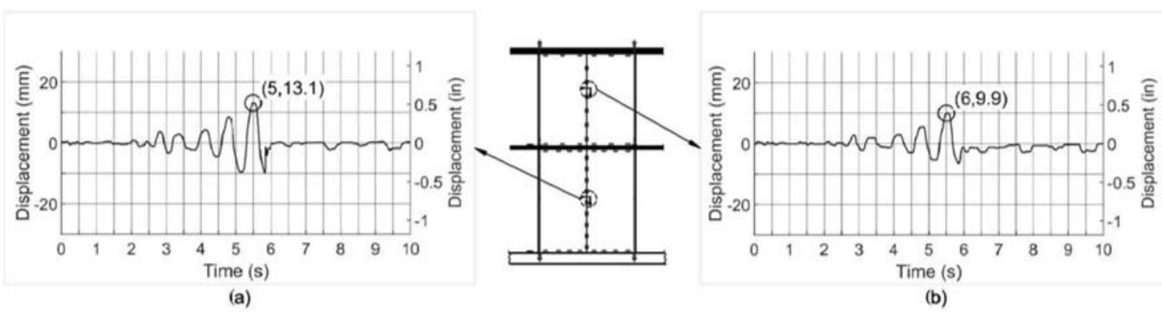


Figure 29. Phase 2 MCE Vertical Relative Panel Displacement: (a) First story shear wall top west corner; (b) First story shear wall bottom west corner; (c) Roof shear wall top west corner; (d) Roof shear wall bottom west corner

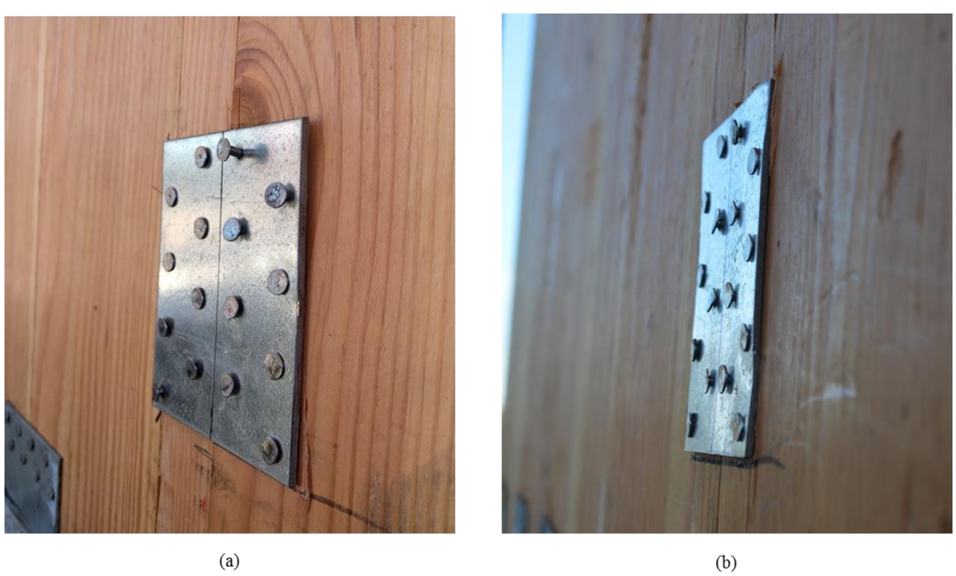


Figure 30. MCE inter-panel connector inspection: (a) Phase 1 first floor; (b) Phase 2 first floor (Photo credit: NHERI TallWood Team).

Forces in Tie-Down Rods

Tie-down rods were installed in the SFRS, with a rod on each end of the wall and both faces, with a bearing plate providing a reaction point on each story, similar to standard construction for light-frame wood buildings. This was consistent throughout all the phases, and in the design of the SFRS, it was assumed that the CLT wall panels worked as a continuous segment with the inter-panel connectors transferring shear, and because of this, tie-down rods were only required at the ends of the walls. The tie-down rods are designed to absorb tension from the overturning moment present in the structure, while the CLT panels and brackets transfer shear. The rods therefore are not designed to take any compression force, and are thus allowed to slide through at the bottom of the structure, allowing the CLT panels to resist the compression load. The summary of the recorded forces in the rods can be seen in Table 4. No load cell data was available for the Phase 3 MCE level test, so data from strain gauges placed on the tie-down rods was used instead. The largest tension force in the rods was 237.6 kN (53 kips) recorded at the base on the first story during the Phase 2 MCE test. This test included the 2:1 aspect ratio walls, and it most likely due to the nail failure in the shear brackets, essentially forcing the hold downs to do 100% of the resistance in uplift. Although the shear brackets are assumed to only resist shear in the design approach for the platform CLT stacked walls herein, they do resist some overturning as the CLT panels rack. A large tension force of 170 kN (38 kips) was also recorded during the Phase 1 MCE level test, and the tension both in this test and the Phase 2 MCE test were enough to cause some yielding in the A36 tie-down rods. Figure 31 presents the average tension force across both wall faces on the CLT panels from Phase 1 MCE level test at both the base and roof of the structure. It can clearly be seen that the loading was not homogenous across the structure, and the east side received more load than the west side. This is a result of the torsion discussed earlier, and both Phase 2 and Phase 3 showed a similar trend. An objective of the test program was to determine if the addition of transverse walls in Phase 3 would decrease the loading in the tie-down rods, unfortunately, load cell data for Phase 3 was unavailable due to technical difficulties. However, strain gauges were placed in strategic locations along the tie-down rods to act as a backup in such an event. Table 5 presents the strain gauge data for one of the tie-down rods for both the Phase 1MCE level test and Phase 3 MCE level test. It should be noted that in Phase 1 MCE level test the maximum recorded force (approx. 167 kN) was approaching the yield point of the tie-down rod, and due to this, it was not possible to convert the strain to force with a satisfactory degree of accuracy. However, as can be seen in the table, the difference in strains between the tests are rather significant, with the Phase 3 MCE level test experiencing strain an order of magnitude smaller, thus making the conversions to force unnecessary to analyze the performance of the structure. As anticipated, the addition of transverse walls reduced the strain in the tie-down rods, implying reduced force, and ultimately, reduced overturning moment and improved performance.

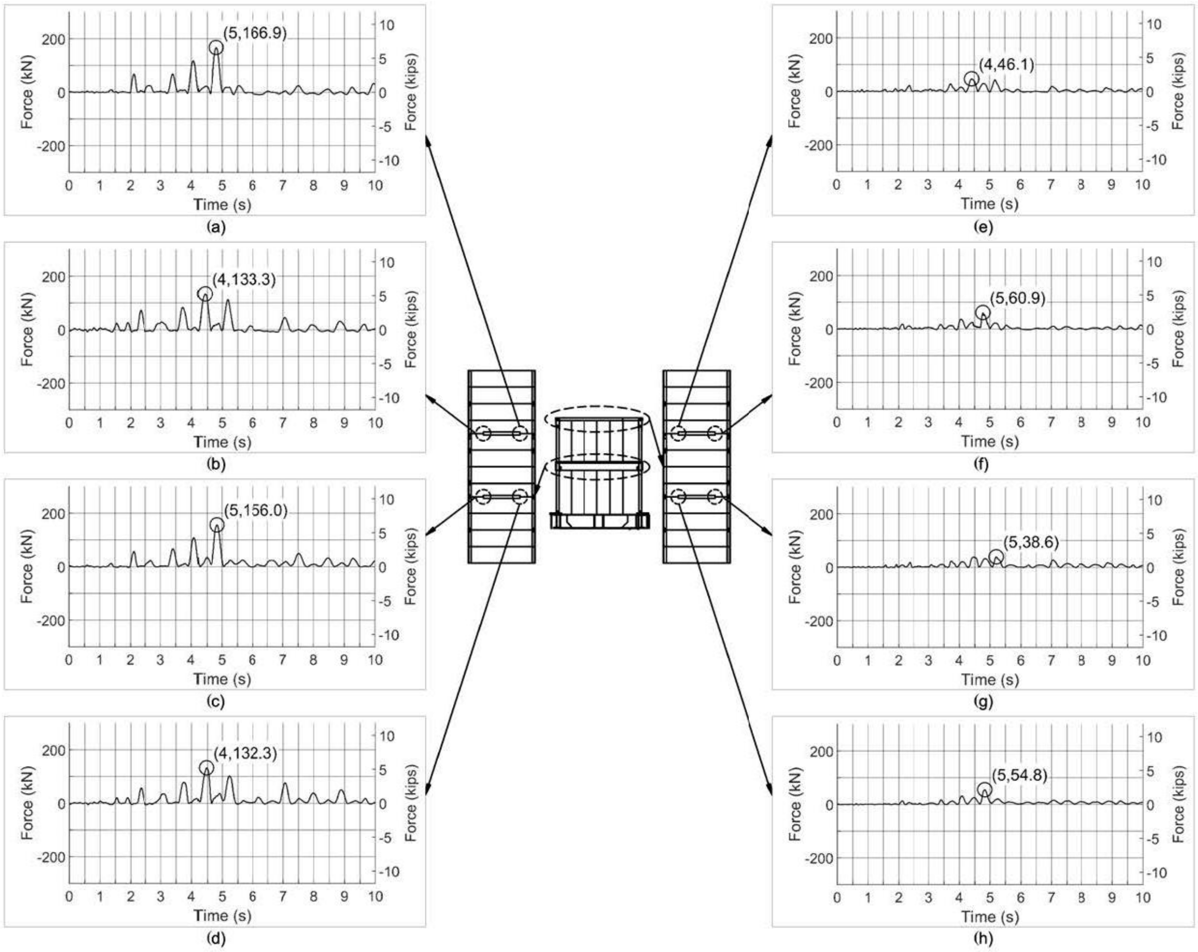


Figure 31. Phase 1 ATS Rod Load Cells: (a) First story north wall east side ; (b) First story north wall west side; (c) First story south wall west side; (d) First story south wall east side; (e) Roof north wall west side; (f) Roof north wall east side; (g) Roof south wall east side; (h) Roof south wall west side.

Table 5. MCE Tie-down Strain

Phase	First Floor Strain	Second Floor Strain
1	7.15e-03	7.13e-03
3	6.83e-04	6.92e-04

CHAPTER 4 – SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

A full-scale two-story CLT platform building with two CLT panel shear wall stacks with tie-downs rods with three different shear wall configurations each tested, namely, 3.5:1 aspect ratio panels, 2.1:1 aspect ratio panels, and 3.5:1 aspect ratio panels with transverse walls installed on each end, being tested. All of the configurations were subjected to the 1989 Loma Prieta ground motion scaled to intensities corresponding to SLE, DBE, and MCE levels respectively, with spectral accelerations ranging from 0.52 g to 1.5 g. The design of the shear wall stacks used the ELF procedure with the intent of providing life safety to would-be occupants.

In all three test phases summarized in this thesis, each of which utilized several different ground motion intensities, the structure provided life-safety had there been occupants in the building. Phase 1 and Phase 3 CLT panels both clearly were governed by rocking behavior as had been demonstrated to occur in higher aspect ratio panels. Partial nail pull out and some steel angle bracket deformation was observed during Phase 1 and Phase 3 MCE level tests, but this was expected and demonstrated the connectors behaved as intended. It was also observed that the transverse walls installed in Phase 3 did not significantly affect the ability of the CLT panels to rock or the connections to perform as designed. Phase 2 and the lower aspect ratio panels experienced significant sliding and nail shearing during the MCE level test, resulting in failure of the shear brackets at the base. Although this is far from an ideal behavior, it should be noted that the design used in this test was less than 2/3 the capacity eventually to be proposed for design of platform CLT systems in the U.S.; and also it is important to note that the stability of the structure was never in jeopardy. The tie-down rods experienced some yielding effects in both the Phase 1 and Phase 2 MCE level tests, but still performed as designed and resisted the overturning moment. In Phase 3, there was no yielding observed due to the improvement in performance provided by the addition of the transverse walls. At SLE and DBE level tests, there was no observable damage in the connections in the shear wall stacks, and no yielding recorded in the tie-down rods. Towards the end of the testing, torsion in structure began to become more pronounced, and introduced some asymmetric loading into the structure, but the SFRS still performed such that there was never risk of collapse at MCE level shaking.

Each of the wall configurations throughout testing met the life-safety requirements of current U.S. seismic codes. Recall that the wall configurations were designed to a capacity of less than 2/3 of what is to eventually be proposed for inclusion in U.S. codes, implying greater capacity in future systems designed using the seismic performance factors developed based on the FEMA P695 methodology. It was confirmed that the behavior of the higher 3.5:1 aspect ratio wall were governed by a rocking failure mechanism, and while the lower 2.1:1 aspect ratio walls experienced significant sliding, due to the nail shear failure in the brackets, no conclusion into the failure mode of the panels can be drawn. The testing also confirmed hypotheses that the addition of transverse walls on either end of the shear walls do not inhibit the rocking of the panels, while simultaneously improving the performance of the tie-down rods in resisting overturning moment. Finally, the experimental results from the testing provide valuable data for the further refinement of numerical models for designing CLT SFRS using equivalent lateral force procedure in the U.S..

The testing was a success and accomplished the objectives of the project, providing valuable data for the further development and refinement of the design methodology. However, there are several things that should be considered in the future. First, additional testing on smaller aspect ratio panels is needed. This is due to the large nail shearing observed on the 2.1:1 CLT panel testing, preventing some conclusions on the behavior of the system from being drawn. Also, the method of installation for the sliding linear potentiometers could be improved. The way they were installed during the testing left them susceptible to getting caught between the CLT panel and diaphragm during the rocking motion of the panel, which occurred during several tests, resulting in no measurements. Finally, while contingency plans existed for some instrumentation failure during testing, better procedures for evaluating instruments prior to testing should be implemented. This was particularly the case for the Phase 3 MCE level test, where load cell data was unavailable, and the contingency strain gauges had to be used.

While providing life-safety is the primary objective of current U.S. seismic code, there has been growing interest in more resilient seismic designs. CLT has also shown promise in this area of seismic design, and further research into its use resilient seismic designs has been growing. Projects such as the NHERI TallWood project, which is a multi-year project to design and validate a resilient-based design methodology for tall wood buildings, have been pioneering CLT resilient-based seismic design in the U.S., but there are many more research opportunities in this field, and these represent the shifting focus of earthquake engineering beyond life-safety.

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