THESIS

SHAKE TABLE TESTING OF HYBRID WOOD SHEAR WALL SYSTEM

Submitted by

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ABSTRACT

SHAKE TABLE TESTING OF HYBRID WOOD SHEAR WALL SYSTEM

Cross-Laminated Timber (CLT) is an engineered, prefabricated mass timber product that has shown excellent structural and mechanical properties. With the growing application of CLT in industry, there have been a number of research projects carried out to introduce CLT in tall buildings located in high seismic regions. The concept of post-tensioning mass timber has been adopted from concrete systems and this led to development of seismically resilient structural systems that can undergo multiple earthquake and continue to re-center. This thesis presents the results of a shake table test program that focused on testing of a one-story full-scale hybrid wood shear wall system comprised of a post-tensioned CLT wall panel with Light-frame wood shear (LiFS) wall panels on each side. The testing was conducted at CSU's Engineering Research Center shake table. The objective of this study was to combine the advantages of the post-tensioned CLT systems with those of LiFS walls. The hybrid shear wall system in the testing had two LiFS walls on either side of a post-tensioned rocking CLT wall panel. Mild steel rods were used as post-tensioning rods in this experiment and the test structure also included gravity frames constructed with wood studs (but no sheathing) and a CLT floor diaphragm to support a seismic weight of 12,000 lbs. The structure was subjected to the 1989 Loma Prieta ground motion record scaled to different intensities. The final test used the original 1994 Northridge ground motion record from the Rinaldi record station, with a slight reduction to be able to be accommodated by the 20 inch shake table actuator stroke. This test was conducted to understand the collapse mechanism of the structure and demonstrated the ability of the post-tensioned CLT to re-center

the structure after 5% inter-story drifts and also the ability of the LiFS walls to act as energy dissipation and lateral force resisting systems.

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CHAPTER 1: INTRODUCTION

1.1 BACKGROUND

Conventional light-frame wood buildings are the most common residential structures in North America. Light-frame wood buildings have been cost effective for residential projects and the ease of construction has seen it become a viable choice for low rise buildings. The major structural components of light-frame wood buildings include horizontal floor diaphragms, gravity walls, shear walls and horizontal or sloped roof systems. These structural elements are typically made of sawn dimension lumber as framing members with sheathing connected using nails. Light-frame wood shear walls (LiFS) typically resist loads in both the horizontal and vertical directions. The gravity loads on the LiFS transferred from the floors and stories above are resisted by the shear wall framing stud members and the lateral loads are taken by the sheathing connected to the framing stud members. The lateral loads in LiFS are resisted by the nail slip that dissipates the energy. Due to its low vertical load carrying capacity as compared to alternative building material choices such as steel and concrete, light-frame buildings have been used predominantly for residential and low-rise commercial buildings, typically one to five stories. This led to researchers developing mass timber products such as Cross Laminated Timber (CLT) and Laminated Veneer Lumber (LVL) which can take greater gravity loads and - shear loads.

CLT was introduced in the 1990's in Europe and gained popularity as a construction material over the last several decades. It is an engineered wood product that was first developed in Austria and Germany. CLT panels are made up of dimension lumber bonded together in layers with structural adhesives in alternating perpendicular layers as shown in Fig. 1.1. CLT has high in-plane strength in both directions, dimensional stability, fire resistance, and good thermal and sound resistance. Due to its rising popularity in North America, there was a demand for standards to be developed for commercial use. FPInnovations, Canada published the Canadian edition of the CLT Handbook (Gagnon and Pirvu, 2011) and the U.S. edition of the CLT Handbook (Gagnon and Pirvu, 2013) was introduced several years later. The ANSI/APA CLT standard committee published Performance Rated Cross-Laminated Timber (ANSI/APA PRG 320) standard. CLT was classified as a mass timber system in the International Building Code (ICC, 2009). In 2015, the National Design Specification for Wood Construction (AWC, 2015) also adopted CLT in the design specification.

Figure 1.1: Illustration of CLT panels (Pei et. al, 2013)

The introduction of CLT led to the development of mid-rise and high-rise timber construction across Europe, North America, Japan, Australia and New Zealand. Treet apartments (14-story) in Bergen, Norway, Forte residential building (10-story) in Melbourne, Australia and Stradthaus residential building (9-story) in London, U.K. are some examples of tall buildings constructed using CLT. Brock Commons – Tallwood House at the University of British Columbia (UBC), Canada is currently the tallest timber building. However all these structures employ concrete or steel in the form of horizontal floor diaphragms or shear walls to take some of the seismic and wind load on the structure. For example, the UBC building has a concrete core with CLT walls and diaphragms.

The use of traditional mechanical holdowns has been one of the key limiting factors for low lateral load resistance in mass timber system. Recent studies on utilizing post-tensioning techniques for mass timber have been studied worldwide (Palermo et al, 2005; Buchanan et. al, 2008; Wanninger and Frangi, 2014). These studies were conducted on Laminated Veneer Lumber (LVL) walls designed to undergo controlled rocking under seismic loading. Even more recent studies in North America were conducted with CLT rocking wall post-tensioned systems (Ganey et al., 2017; Kovacs and Wiebe, 2016). The post-tensioning system acts as a self-centering mechanism by providing a restoring force that brings the structure back to its original position even after large lateral deformations. The post-tensioning system concept evolved originally from unbonded post tensioned concrete walls which are currently accepted in major design codes and guidelines (Priestley, 1991; Priestley and Tao, 1993; Priestley, 1998; Nakaki et. al, 1999; Priestley et. al, 1999).

This research focuses on the behavior of a hybrid shear wall consisting of a post tensioned CLT rocking wall and traditional LiFS walls subjected to dynamic loading. This work primarily reports the results of shake table tests conducted on a one story full-scale hybrid shear wall system with the main objective being to demonstrate re-centering capabilities after excessive inter-story drift. This work is a collaboration with the University of Alabama where wall tests have been carried out subjected to cyclic and dynamic loading and real time models were also developed for a three-story building (Nguyen et. al, 2018).

1.2 MOTIVATION

The rise in popularity of CLT has resulted in an increasing use in the construction industry in North America and many low and mid-rise building have already seen the use of CLT underway in construction projects. Researchers have found that the performance of heavy timber system in seismic regions can be improved with the introduction of a rocking wall system, but there are some complexities associated with the integration of such a system. Post-tensioned LVL rocking systems were developed and studied over the past decade in New Zealand (Palermo et. al, 2005; Palermo et. al, 2006; Newcombe, 2007; Smith et. al, 2008; Pino Merino, 2011). Recently research on post-tensioned CLT rocking wall systems has been conducted in the U.S. and Canada including rocking walls coupled with some form of energy dissipaters (usually internal steel bars and U-shaped Flexural Plates) to enhance the seismic performance of the building structures. The introduction of hybrid CLT walls and LiFS combines the advantage of both systems and can be incorporated in wood building structures, provided compatibility and other issues are eventually addressed (Dao et al, 2018). A rocking CLT wall can provide the re-centering force needed for a structure to remain inhabitable and potentially resilient following a seismic event and the LiFS walls act primarily as energy dissipaters but are damaged. The primary focus of this thesis is to design a relatively simple system that provides a realistic approach to construction of CLT-LiFS hybrid shear wall systems within a building when subjected to seismic loading.

1.3 LITERATURE REVIEW

1.3.1 Light-frame Wood shear walls

The response of light-frame wood shear walls to dynamic loading have been well studied over the years. Hysteretic modeling of typical wood buildings with light-frame wood shear walls have been developed with each shear wall modeled as a single-degree-of-freedom (SDOF) hysteretic oscillator and then assembled into a building system (Bahmani and van de Lindt, 2014). A computer program SAWS (Seismic Analysis of Woodframe Structures) was developed which allowed hysteretic modeling of three-degrees-of-freedom per floor of a woodframe building (Folz and Filiatrault, 2004a,b). In the SAWS model, the actual three-dimensional structure is degenerated into a two-dimensional planar model made of zero-length shear wall spring element that connect the floor diaphragms and the foundation. The CUREE 10-parameter model can be obtained by curve-fitting to obtain the backbone curve for experimental data from a full-scale cyclic wall testing or by using the numerical program CASHEW (Cyclic Analysis of Shear Walls). The model results were verified with experimental results of a shake table test on a two-story building. The CUREE 10-parameter model has been widely used by researchers for modeling wood shear walls.

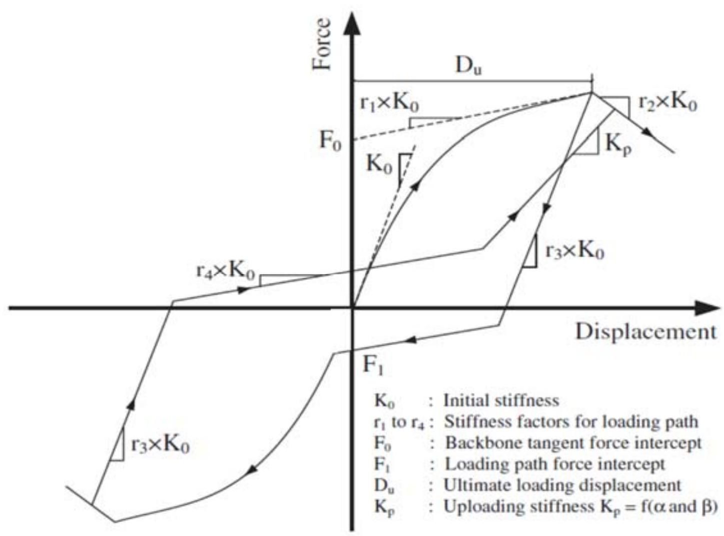


Figure 1.2: CUREE 10-Parameter hysteretic model (Folz and Filiatrault, 2004a,b)

1.3.2 Cross-Laminated Timber

Cross Laminated Timber is a mass timber engineered product that was introduced in Europe in the 1990’s. CLT is a prefabricated, 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 (Pei et. al, 2016). The alternating direction provides strength in both in-plane directions as compared to other engineered wood products like laminated veneer lumber and glulam which have high strength along one major direction. CLT panels are manufactured with varying thickness and the desired number of layers. The number of layers typically ranges between 3 and 7 with a variety of lumber species and lumber dimensions used depending on the country and region. The thickness of each individual layer usually varies between 5/8in. and 2in. CLT panels are usually manufactured in different sizes with the length and width of the panel usually depending on the requirements, size of the manufacturing press and transport limitations. The outer layer of the CLT panel is usually oriented along the major loading direction. The CLT panels used for the walls are oriented with the outer layer along the gravity load direction and the diaphragm panels are oriented with the outer layer spanning across the major span.

Early CLT research was conducted in Central Europe in the early 2000s. In 1990, research activities regarding CLT started at Graz University of Technology (Bradner et. al, 2016). Due to its growing popularity, various projects have been conducted in Europe and North America. The first lateral load testing on CLT wall panels was conducted at University of Ljublijana, Slovenia (Dujic et. al, 2005). Experimental tests on wooden wall elements under constant vertical loads and monotonous and cyclic horizontal loads carried out to study the response to the application of lateral loads. The tests carried out were per EN 594 protocol with three different types of setups; the first case with restraint on only the base (Cantilever mechanism), second case with restraint on one side and restricted translation and rotation on the other side and finally the shear wall mechanism where one end was supported on a firm base and the other end was allowed only to translate in the in-plane direction only. The response of vertical loading and monotonous and cyclic horizontal loading was measured using deflection and plotted to examine/document the influence of loading condition and boundary conditions on the wall panels. It was observed that the response to cyclic loading is lesser than the response to monotonic horizontal loading. The fasteners and anchors used to hold down the wall panels also influences the stiffness of the wall panels to the applied cyclic loads. The boundary conditions affect the response of the wooden panels to the applied loads.

In 2006, shake table tests on a 3-storey building constructed with CLT were investigated by the Trees and Timber Institute of the National Research Council of Italy (CNR-IVALSA) in collaboration with National Institute for Earth Science and Disaster Prevention (NIED), Shizuoka University, Building Research Institute and Center for Better Living, Japan (Ceccotti et. al, 2006). The test was part of a larger program named the SOFIE project. The tests were conducted at the NIED Tsukuba Shake Table facility and it was the first time such a test was conducted on this type of structure. The structure was a three-story building with a total weight of over 110kips. The plan dimensions of the building were 23 ft. by 23 ft. (7 m. by 7 m.) with a total height of 33 ft. (10 m). The structure was subjected to recorded ground motions from Kobe, El Centro and Nocera Umbra scaled to represent a Zone 3 hazard level from the Italian Seismic code. A total of 14 ground motions were used for the test. Further it was also observed that the introduction of openings on the structure did not significantly reduce the strength of the structure.

The SOFIE project was conducted in Italy and was funded by the Trento province of Italy (Ceccotti et. al, 2013). A full-scale seven-story building was designed according to the European seismic standard Euro code 8 and subjected to earthquake loading on a 3D shaking table, E-Defense. The building was designed with a preliminary action reduction factor of three that had been derived from the experimental results of the 3-story building. The test results proved the suitability of using CLT for multi-story buildings in seismic regions. The high stiffness and ductility of the CLT proved advantageous to avoid brittle failures. The building experienced relatively high accelerations at the upper floors which lead to secondary damage. The dimension of the seven-story building was 24.5 ft. by 44 ft. (7.5 m. by 13.5 m.) with an overall height of 77 ft. (23.5 m). The thickness of the walls were reduced at the upper floors to reduce the weight since they were not necessary for gravity support. The overall weight of the structure was 626 kips. Preliminary tests were carried out to study the failure at joints. The joints were critically designed to avoid any failure at the joints. The building was subjected to three earthquake records JMA Kobe, the Italian earthquake of Nocera Umbra and Kashiwazaki R1. A series of 10 tests were conducted with the first six ground motions in one direction and the remaining tests in three dimensions. It was observed that after a series of tests with 10 major earthquakes, the building did not have any residual damage but failures were observed at several of the hold-downs at the first story.

There have been several research tests in North America to examine CLT as a seismic system. A series of quasi-static tests were conducted by FPInnovations-Forintek in Vancouver, BC to study the lateral load resistance of CLT panels (Popovski et. al, 2010). A total of 32 monotonic and cyclic tests were performed on 3 layer CLT panels. 12 different wall assemblies were used in the testing with aspect ratio 1:1 and 1:1.5. The test matrix also included two story wall assemblies. Four different types of bracket with different fasteners were used as connection between the walls and foundation. It was observed from the series of tests that the CLT wall panels behaved almost as rigid bodies during the testing with very small shear deformation. A typical hysteretic force-displacement curve was obtained from each of the cyclic loading tests.

A simplified model approach was created to determine the numerical response of a CLT building (van de Lindt et. al, 2013). The simplified kinematics model as shown in Figure 1.3 were created based on several assumptions including that CLT wall panels behave as in-plane rigid bodies, the panels rotate individually around bottom corner under lateral load, there is no relative slip between the wall and floor panels, the gravity force acts vertically through the center of the CLT wall panels and the panel connectors will undergo deformation during the rocking motion and develop the hysteresis of the wall system.

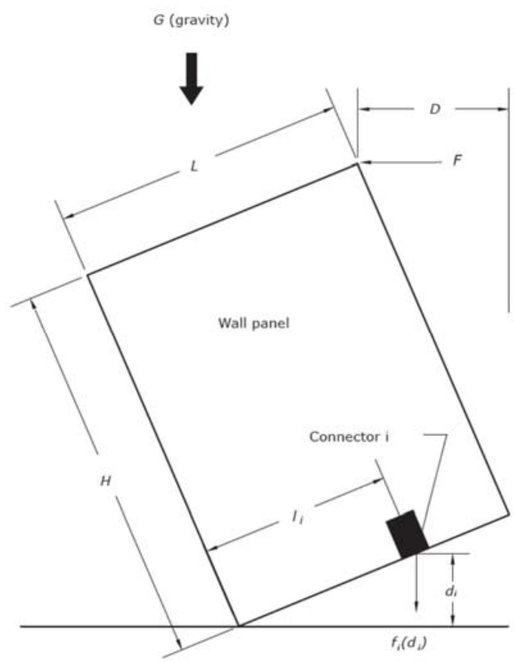


Figure 1.3: Simplified kinematics model (van de Lindt et. al, 2013)

The simplified numerical model developed based on the above assumptions was used to determine the reduction factor for the CLT panels. Four different R-factors of 2,3,4 and 6 were tried and it was found that the performance of the numerical structure for a six-story structure was adequate with a R value of approximately 2 under MCE (Maximum considered Earthquake). It was understood that the building performance factor was set as 3.5% maximum inter-story drift, which is quite low. The limitation of the model included was applicable only for a certain story drift-level and short-wall panel length because of small angle approximation and rotation assumptions in the model.

1.3.3 PreSSS

The concept of a controlled rocking wall for seismic resistance was first developed for precast concrete structural systems in the early 1990s (Priestley, 1991; Priestley et. al, 1999). The PREcast Structural Seismic System (PRESSS) research program focused on developing design procedures for seismic resisting systems for precast concrete frame and precast concrete wall connections. As part of the program a five-story precast concrete structure was experimentally tested, and an analytical model was also developed to predict the response of the structure subjected to dynamic loading. It was concluded from the experimental observations that the walls had very minimal damage (observed only at the base) even after subjected to loads 50% greater than the design loads. It was also observed that the damage to the structure was much lower than reinforced concrete structures subjected to similar drift levels.

1.3.4. Pres-Lam projects

1.3.4.1 Palermo et. al, 2005; Palermo et. al, 2006

The Pres-Lam project was conducted at the University of Canterbury as an extension of the PRESSS research program (Palermo et. al, 2005). The project focused on developing and testing jointed ductile post-tensioned timber system connection for laminated veneer lumber (LVL) systems under seismic loading. The research project involved both numerical and experimental investigation.

As part of the initial phase of the research program, an experimental study was carried out on exterior beam-column subassemblies. Quasi-static cyclic load was applied to the beam-column subassembly and it was found that the performance of the subassembly joint system was satisfactory. Quasi-static cyclic testing was then carried out on wall elements as well (Palermo et. al, 2006). The rocking wall was stressed with two unbonded PT strands. Three tests were conducted of which two had internal and one had external steel bar energy dissipaters. From the test results, it was observed that the hybrid solution allows greater flexibility in the design of connections and both internal and external energy dissipaters performed well and there was minimal damage to the LVL walls.

1.3.4.2 Smith et. al, 2008

Following the subassembly component testing, a 6-story reinforced concrete building located in New Zealand was redesigned with post-tensioned LVL frames in one direction and post-tensioned walls in the other direction (Smith et. al, 2008). The Direct Displacement Based Design approach was used for the design of the building. Drift limits of 2% were set in the frame resisting shear direction and 1% in the wall resisting shear direction. A composite timber-concrete floor system was designed with prefabricated timber panel and in-situ concrete topping which was supported by LVL floor joists. The floor joists were further supported by LVL gravity beams which transferred the gravity load to the supporting gravity columns. The shear force from the floor system was designed to be transferred into the lateral resisting system through coach screws or reinforcing bars connected to fasteners in the solid wall with bolts and threaded couplers. The columns and wall were connected to the foundation using steel dissipating bars with internal epoxied solution. A preliminary cost estimate found that the building cost for the structural system was close to other alternative construction materials (i.e. concrete and steel).

1.3.4.3 Newcombe, 2007

Analytical and numerical models were developed for multi-story timber buildings using the post-tensioned LVL frame system (Newcombe, 2007). A general design philosophy was developed for the post-tensioned timber systems. Analytical models were developed for the post-tensioned LVL frames and walls based on the experimental tests carried out by as part of the Pres-Lam project. The moment-rotation responses of post-tensioned beam-column connection, wall to foundation connection and column to foundation connection were developed based on the experimental test results.

Numerical models were developed for four different frame geometries using the earthquake engineering analysis software RUAUMOKO (Carr, 2008). The frames were either six or 10 stories with 5 or 8 bays with the same total tributary area and same total overall length. Drift limits were set to 2 percent for all frames. A suite of 7 earthquake records were scaled to design basis levels and maximum credible earthquake levels. The connections in the numerical model were developed using rotational springs and the numerical model was also calibrated based on the experimental results. The inter-story drift profile was obtained for the four geometries and the results were compared to the analytical model developed using Direct Displacement design.

1.3.4.4 Pino Merino, 2011

Pino Merino, 2011 carried out shake table experimental tests on 3-story and 5-story post-tensioned timber buildings and also followed it up with quasi-static cyclic tests on 3-story post-tensioned frame. Model frame buildings for the shake table tests were scaled to one-quarter scale and recorded earthquake data and sinusoidal tests were carried out to evaluate the performance of the post-tensioned frame system. Numerical model were also developed using RUAUMOKO and verified with the experimental results. These tests were a follow up to the extensive Pres-Lam research program.

Nelson Pine LVL and 7-wire strands centered inside beams and columns as post-tension for the shake table test. The 5-story structure plan dimension was 16m (52 ft) along the north-south direction and 12m (39 ft) along the east-west direction (shear resisting system direction). The structure has two bays that span for 6m (19.6 ft) and inter-story height of 3.2m (10.5 ft). The building was designed according to Direct Displacement Based Design with interstory drift limit set at 2.5%.

Sinusoidal tests were carried out to determine the natural frequency of the first mode and also evaluate the damping values of the structure. Three levels of amplitude with eleven angular frequency were used to carry out the sinusoidal tests. Six different earthquake records were used and scaled for annual probability of exceedance of 1/500.

It was observed that the drift levels were within the drift limit set at 2.5 percent. It was concluded that the re-centering by the post-tension was effective and have low levels of damage. There were small permanent compressive deformations observed at the internal faces of the columns. It was also found that the damping levels were influenced by the drift levels. Another observation made was the level of post-tensioning had a minor influence on the maximum drift demand.

Following the dynamic test, the 3-story frame was subjected to quasi-static cyclic loading. This test was carried out to compute the static properties of the frame. The cyclic loading protocol was implemented to subject the structure to increments of 0.25% drift up to a maximum of 4.5% drift which is over the MCE value. It was observed that the post-tensioning losses were minimal until the timber crushing occurred. The losses were increased after the timber crushing and a total of 11 per cent of the initial post-tensioning was lost for 4.5% lateral drift.

1.3.5 CLT Rocking wall system

CLT has been proposed as one of the mass timber rocking wall system in North America. Efforts have taken to establish rocking wall CLT system as an alternative in seismic regions. Recently research has been conducted in the United States and Canada on CLT rocking wall systems.

Post-tensioned CLT rocking wall systems were modeled at University of Washington to promote high-rise timber buildings in regions with high seismicity (Ganey, 2015; Ganey et. al, 2017; Pei et. al, 2018a; Pei et. al, 2018b). The objective of the research was to explore the use of CLT rocking wall with post-tensioning connections and supplemental energy dissipation in the form of U-shaped flexural plate (UFP) brackets as a potential ductile CLT lateral force-resisting system. Experiments were conducted to determine the performance of CLT rocking wall subjected to cyclic loading. The experimental results were used to calibrate a numerical model developed in OpenSees (Mazzoni et. al, 2009) to analyze the performance of an 8-story and 14-story structure.

The experimental program included six tests with five of them being carried out on single-wall specimen and one coupled wall system with UFP connection devices. The purpose of the experimental tests was to explore the limit states for the rocking CLT wall system and the relationship between variables and performance. The experimental test used two different PT bar sizes (1.25 in^2 and 1.58 in^2) and three different initial post tensioning stresses. Five of the tests were carried out on steel base beam and one of them had a CLT base to simulate the CLT floor system. The coupled wall system specimen had a UFP brackets at one-third and two-thirds of the total wall height.

Two types of CLT specimens were used in the tests. Five of the experiments used 5-layer CLT panels manufactured with Douglas Fir Larch No. 2 & better lumber boards and one of the test specimens used a Structural composite core for the inner three layers. The PT bars were located at the center of the wall. The walls were subjected to quasi static cyclic loading. The loading protocol included 18 steps with increasing target drift ratios with three fully reversed cycles at each step.

The results of the test reported that all the walls were able to achieve lateral drift ratios of at least 8% except for one of the test which used a reused panel and had splitting failure at 4.5% lateral drift ratio. The hysteretic response for the entire specimen resembled a flag shape as expected. It was observed that toe crushing occurs at the rocking base except for the one specimen which used a CLT base. It was also observed that the CLT base was damaged due to perpendicular to grain loading at the base. The first visible damage for all the other specimens occurred between 1.5% and 5% and damage was increased as the cycles progressed and the final failure mode was crushing of the corners of the CLT wall. It was also observed that the higher initial post-tensioning forces in the larger bars had higher strengths as expected. Also the panel with stronger core demonstrated higher strength as compared to the original specimen. It was also found that the UFP dissipated energy very effectively with the strength of the coupled wall 50% higher than the expected value. The experimental results were used to calibrate the numerical model in OpenSees. The calibrated numerical models were used to develop 8-story and 14-story buildings. The far-field ground motion records compiled by ATC, 2009 were used for the analysis. The ground acceleration records were scaled for three seismic hazard levels were used for each of the building model. It was found that the building model performed well and was able to meet all the performance objectives.

Controlled rocking post-tensioned CLT wall was explored for adaptation in design and construction of buildings in low to moderate seismic hazard regions in Canada (Kovacs and Wiebe, 2016). This study focuses on developing numerical model for accessing the performance of post-tensioned CLT wall in low to moderate hazard regions. A prototype model was designed for low to moderate hazard of Ottawa, Ontario and a numerical model was developed using OpenSees. In order to develop the numerical model, experimental results from a previous test on LVL was used for calibration of the model. A six story structure prototype model was developed and the performance criteria were set as 2.4% maximum inter-story drift as per the building codes. Ground motion records used for the analysis included three each from both near-field and far-field records for eastern Canada and the ground motion were scaled to the Ottawa uniform hazard response spectrum. From the analysis it was found that the inter-story drift was well within 2.5% set by the building codes.

1.3.6 Rocking CLT wall shake table test

A series of shake table test on a two-story building with resilient rocking walls was carried out as part of NHERI TallWood project (Pei et. al, 2018a; Pei et. al, 2018b). This test was done in collaboration with various universities and industry partners. This test was part of a larger on-going research program with the aim of developing and validating a seismic design methodology for tall wood buildings that incorporates high performance structural and non-structural systems.

The test building had a total height of 22 ft. (6.7 m) with first floor at 12 ft. (3.65 m) and second floor height at 10 ft. (3.04 m) The plan dimension of the building was 58 ft. by 20 ft.

(17.67 m by 6.09 m). Two different types of diaphragms were used in the building including a wood only diaphragm at the floor level and concrete-CLT composite diaphragm design at the roof level. Two coupled DR Johnson CLT panels with four Simpson ATS all-threaded rods with 5/8” diameter as post-tension rods for each panel were used in the test building as shear resisting rocking wall system. The building was subjected to a series of 14 ground motions with hazard levels between SLE, DBE and MCE. The final test was run with 120% of the original MCE level.

The building experienced a maximum roof inter-story drift of 5% drift and it was reported that the rocking wall remained elastic for all SLE and DBE level tests and even for some of the MCE level tests. It was also noted that the rocking walls were able to re-center the building perfectly in all SLE and DBE tests and most of the MCE tests. Some losses in the post-tensioning were observed during the largest MCE level tests. The post-tensioning rods did not reach their yield capacity under SLE, DBE or MCE level ground motions. However, the rods did yield in the final test when the ground motion was scaled to 120% of the original MCE level.

1.3.7 Current Research

As mentioned earlier, the research summarized in this thesis is a collaborative effort with the University of Alabama. Some of the experimental work and numerical model on the rocking wall CLT-LiFS shear wall assembly has been published. Ho et. al, 2016 presented the results of a numerical study to examine the behavior of post-tensioned CLT walls under cyclic loadings. The post-tensioned shear wall was designed on the basis of a modified monolithic beam analogy and experimental tests were carried out to validate the numerical model.

The experimental study was conducted on 5.25 in. thick CLT panel with dimensions 2 ft. by 8 ft. (0.6 m by 2.43 m). Grade 270 prestressing tendon with diameter of 0.6 in. was used as post-tensioning steel. The tendon was placed at the center of the panel through a bored hole. Quasi-static cyclic loading protocol was used for the testing. The test results were found to be in good agreement with the numerical model results. The same CLT panel configuration was integrated into a typical LiFS line consisting of three panels with dimensions 4 ft. by 8 ft. (1.2 m by 2.43 m) and an opening of 2 ft. by 4 ft. (0.6 m by 1.2 m)was also included. Six 16d common nails were used as connection between the studs of the LiFS and CLT panel. A cyclic loading protocol with a frequency of 0.05 Hz and a maximum displacement of 3.25 in. was applied to the shear wall system. The experimental hysteretic curve results were compared with numerical hysteretic curve results. Incremental dynamic analysis was performed using a suite of 22 ground motion pairs [PEER NGA (PEER, 2008)] for the LiFS wall and the hybrid shear wall system. The results were compared and it was found that the mean curve for the hybrid shear wall was much steeper than the LiFS wall.

This research served as an initial study for assessment of seismic response of a three story building with the hybrid rocking wall CLT-LiFS system using real time hybrid simulations (Nguyen et al, 2018). The rocking wall CLT-LiFS system described in the previous testing was assumed to be a part of a 3-story building with plan dimension of 18 ft by 36 ft. (5.4 m by 10.97 m) The 3-story model was built numerically with the experimental substructure remaining the same as mentioned in the previous testing. The other walls in the numerical model was assumed to be lightframe wood system and the total mass applied to the numerical structure was 20 psf of dead load and 50 psf of live load at each floor.

In the first phase of the testing, cyclic loading was applied to the physical structure of the building to understand the behavior of the hybrid shear wall system. The CUREE cyclic loading protocol was applied for this test. It was observed that the hybrid shear wall system reached a maximum peak force of 10.5 kips with a total drift of 1.75 in. Also it was observed that the nails connecting the rocking wall and the LiFS wall studs were fractured. Also some of the edge nails were pulled out on the LiFS walls.

This test was followed up with a real time hybrid simulation of three story building. The 6.7 magnitude 1994 Northridge earthquake (Beverly Hills recording) was chosen as the ground motion for the hybrid test. The ground motion was scaled to 44% DBE, DBE and MCE for the three-story earthquake building. A visual check at the end of each test confirmed that there was no damage to the hybrid wall system. It was observed that a peak drift of 0.575 in. and 0.821 in. at DBE and MCE level respectively.

In this thesis, a series of dynamic shake table tests were conducted on the rocking wall CLT-LiFS wall system. In order to further explore the behavior of this wall system, the connection between the rocking CLT wall and LiFS walls were explored with several different options. The numerical model for this test was provided by our collaborators and the primary focus of this thesis was on the shake table experiments.

1.4 THESIS OUTLINE

The organization of the thesis is outlined below:

Chapter 2 describes the design and construction of the experimental test setup with detailed description of every component of the structure. The test matrix is also provided along with the ground motions used in the testing and the instrumentation used to record the data.

Chapter 3 provides the experimental test results and observations. The test observations includes damage inspection and repair works carried out after each test. The test results include the data collected by monitoring throughout each test.

Chapter 4 provides the summary of the experimental program, concluding remarks and recommendations for future study.

CHAPTER 2: TESTING METHODOLOGY

2.1 INTRODUCTION TO EXPERIMENTAL TESTS

The objective of the test program was to better understand the interaction between the CLT rocking wall and LiFS. A series of shake table tests were conducted at the Engineering Research Center, Structures Laboratory at Colorado State University. The structure was designed based on various constraints in the experiment setup. The structure was subjected to scaled ground motion to represent different hazard levels. Based on the observations of the initial testing, few modifications to the design was done in the subsequent tests.

2.2 DESCRIPTION OF THE TEST SETUP

The shake table at the Colorado State University is uniaxial shake table with a plan dimension of 8 ft. by 16 ft. (2.43 m by 4.87 m) equipped with a 35 Kip actuator with a 180 GPM three-stage value. The test structure consisted of different components including two gravity frames, top CLT diaphragm supporting the frame on which the seismic weight was loaded on to the structure, a base CLT for the shear wall assembly to replicate the flooring system in a real building and the rocking CLT-LiFS shear wall assembly. The experimental setup is shown in Figure 2.1.

The plan dimension of the test structure was 6 ft. by 14 ft. (1.82 m by 4.26 m). Two gravity frame stud walls were located 6 ft. (1.82 m) apart on the east and west side of the structure with a total wall length of 14 ft. (4.26 m). The shear resisting CLT-LiFS shear wall assembly was placed between the two gravity frame stud walls. The shear wall assembly includes two LiFS walls with length of 4 ft. (1.21 m) on either side of the 4 ft. (1.21m) long rocking CLT wall panel. To accommodate for the rocking behavior of the rocking CLT panel, a 1 ft. space was provided between the LiFS walls and the rocking walls. Shear keys were provided on either side of the CLT wall panel to prevent sliding. This space could/would be utilized for electrical or ductwork within a typical building as needed. The shear wall assembly and the seismic mass were connected together through a 5-layer Nordic CLT panel floor diaphragm. Further details related to each component is discussed in the following sections.

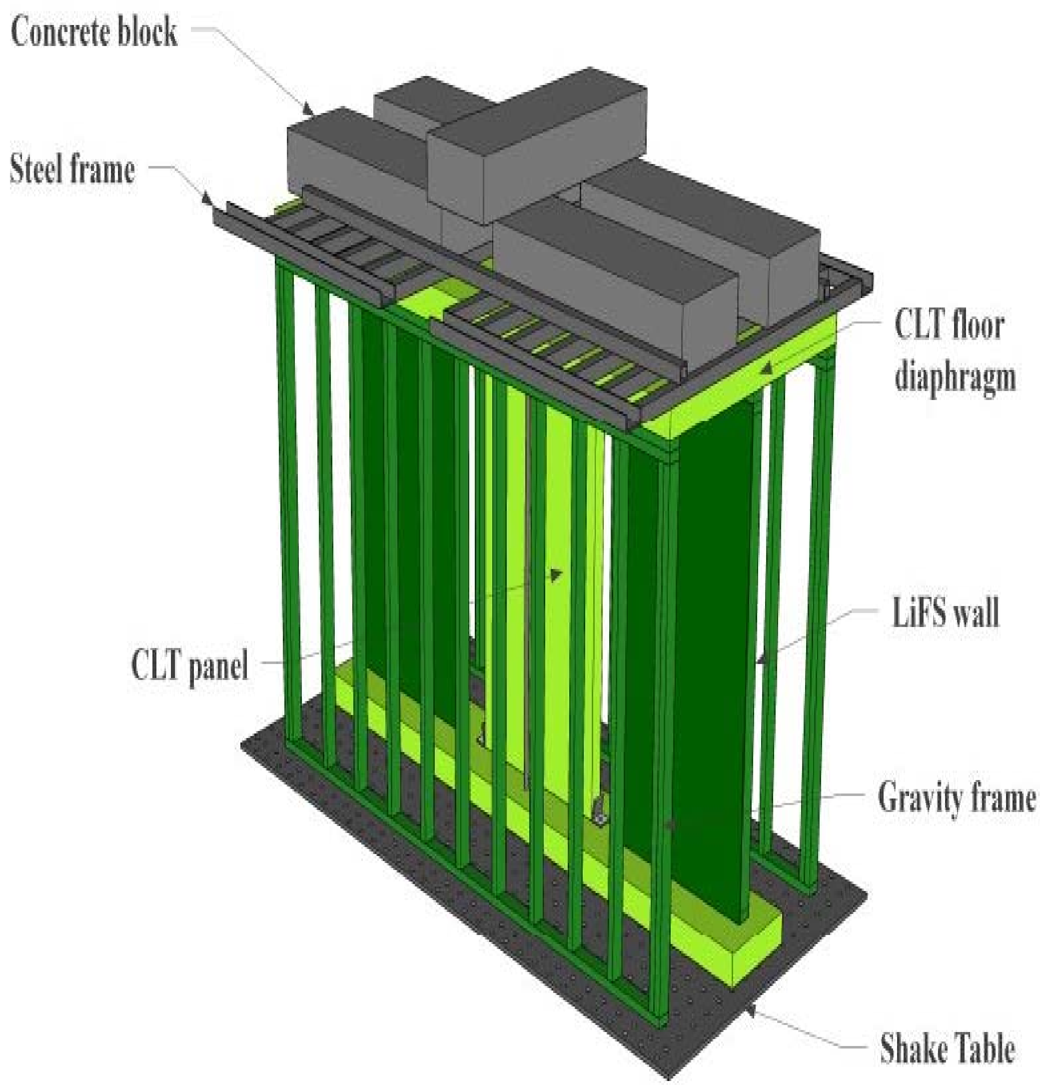


Figure 2.1(a): Schematic drawing showing the experimental setup

Figure 2.1(b): Test setup for the first test with connection between the LiFS walls and the rocking CLT wall panel

Figure 2.1(c): Test setup with connection between the rocking CLT wall panel and the Mass diaphragm

2.2.1 Rocking CLT wall panel

The rocking CLT wall panel used in the test structure was a 3-layer Structurlam panel. The CLT floor panel was a V2 grade panel made of Spruce-Pine-Fir #2&better in major and minor direction layers. The panel width was 4 ft. (1.21 m) and height was 7 ft. 10 in. (2.39 m) with a thickness of 3.89 in. (0.09 m)The panel was not changed throughout the series of the shake table tests since no damage to the CLT panel was observed during inspections between tests. Shear keys were used to prevent sliding of the CLT panel. Each shear key was fabricated using two ½ in. thick steel plate of 6 in. by 6 in. welded together at an angle of 15 degrees to the vertical to allow the rocking motion of the CLT panel.

The post-tensioning was applied externally using two A36 fully threaded steel rods with diameter of 5/8 in. The post-tensioning force was transferred to the wall through a 1 in. thick steel plate. The structure was designed in such a way that the initial post-tensioning force is the only vertical load applied on the CLT panel. The gravity load of the structure was resisted by the gravity frames on either side and the LiFS walls on the shear wall assembly. The total initial post-tensioning force applied to the CLT panel was 14 kips (65% the yielding force) and the initial post-tension force was applied manually to the rods. The forces in the steel rods were monitored throughout the test using strain gages. The post-tension forces were applied just prior to conducting each test to avoid any loss in forces due to relaxation. Mild steel rods were used in this test and were expected to yield at the end of every test and were changed after every test. The material properties for the CLT panel and the post-tension rods used are given in Table 2.1. The connection between the CLT panel and the seismic mass is discussed separately as there were a few modifications after the initial test.

Table 2.1 Material Properties of CLT panel and Post-tensioning rod

CLT	Compressive strength parallel to the grain	1.15 ksi
	Elastic modulus parallel to the grain	1400 ksi
Threaded Rod	Yield Strength	36.3 ksi
	Ultimate Strength	79.8 ksi
	Elastic modulus	29000 ksi

2.2.2 LiFS

Two LiFS walls were used in the shear wall assembly with each wall being 4 ft. wide and 8 ft. high. The LiFS wall frames were constructed using 2 x 4 Douglas-Fir-Larch studs with a stud spacing of 16 in. along with double top plates and double end studs. 16d common nails with shank diameter of 0.162 in. and shank length of 3.5 in. were used in the framing of the LiFS walls. Additionally to resist the overturning moment, Simpson Strong Tie HDU5-SDS2.5 hold-downs were installed on the double end studs. The walls were further connected to the shake table through 5/8 in. all threaded rods which runs through the base CLT and attached to the shake table floor. The seismic mass was tied to the walls through 5/8 in. all threaded rods between the CLT floor diaphragm and the walls. Simpson Strong Tie HGA10 connectors were also used as shear clips between the walls and the seismic mass. The sheathing used in the testing was 15/32 in. thick plywood (sheathing rated) and attached to the framing studs using 8d common nails with edge nail spacing of 4 in. and field nail spacing of 12 in. The sheathing was installed on the east side only. The ten parameters for the panels were obtained from Bahmani and van de Lindt, 2014 and are given in Table 2.2. Since the light-frame wood shear walls are designed to dissipate energy through deformation of the fasteners, there was some damage sustained after every MCE level test. The walls were replaced after every MCE level test. The wall elevation schematic is shown in Figure. 2.2.

Table 2.2 CUREE 10-parameter Hysteresis values

LiFS length	K_0 Kip/in.	F_0 Kip	F_1 Kip	r_1	r_2	r_3	r_4	D_u in.	α	β
4 ft.	7.88	2.52	0.44	0.061	-0.115	1.000	0.016	2.48	0.700	1.100

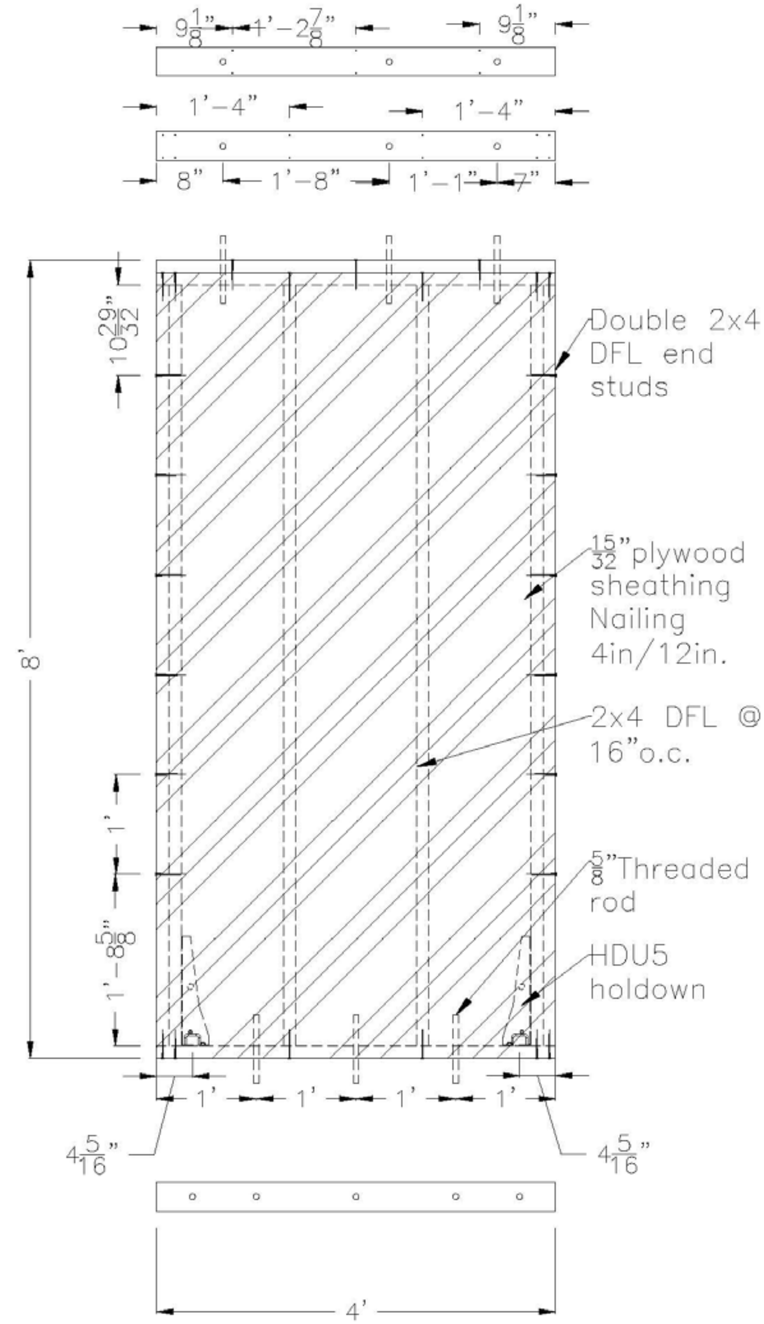


Figure 2.2: Detailed elevation drawing of typical LiFS wall

2.2.3 Gravity Frame stud walls

The gravity frame stud walls were located on the east and west end of the structure. The purpose of the gravity frames was to resist the vertical loads only thereby supporting the seismic mass representative of stories above. The walls were 14 ft. long with a total height of 8 ft. 6 7/8 in. These walls were directly attached to the shake table floor and did not have a CLT base as these were not part of the specimen being tested, but more part of the testing setup itself. The gravity frame stud walls were not changed throughout the test program as they did not sustain any damage and behaved almost as pin-pinned gravity columns.

The frames were constructed from 2 x 4 Douglas-fir-Larch studs with a maximum stud spacing of 16 in. The gravity frame stud walls also had a double end studs with double top plates. 16d common nails were used for the framing of the studs. The frames were connected to the shake table and the top CLT floor diaphragm with 5/8 in. threaded rods.

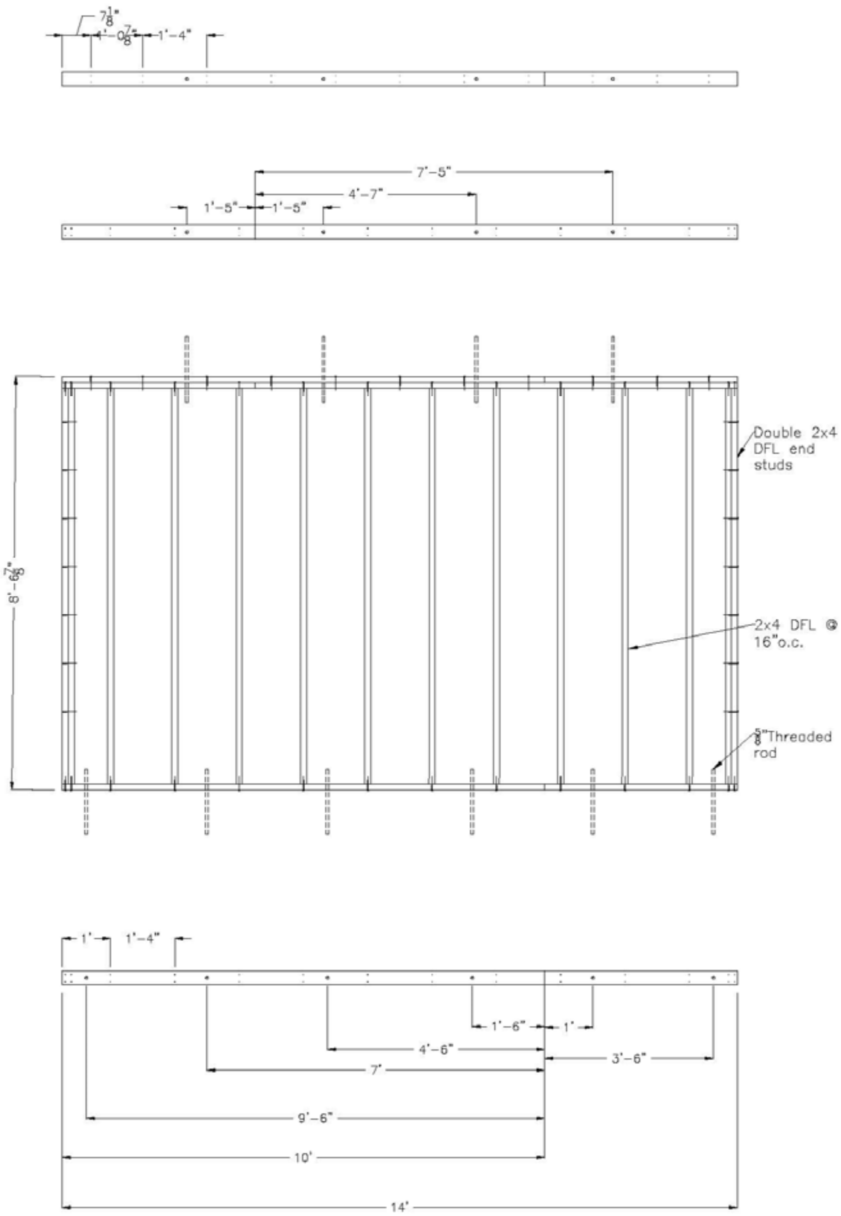


Figure 2.3: Schematic drawing of Gravity frame walls

2.2.4 Connection system for rocking CLT wall panel

The rocking CLT wall panel was designed to undergo rocking without any resisting forces. For this purpose, there was enough space given between the top CLT floor diaphragm and the LiFS walls on either side of the rocking wall panel. An initial connection design included a connection system between the LiFS walls and the rocking wall panel through a bracket designed to allow rocking. However, this was changed after the initial test due to failure of the LiFS studs. Later a connection design for the rocking wall was adopted by modifying a connection system used in another study (Pei et. al 2018a, Pei et. al 2018b). In this section, both the connection details are discussed. The failure of the initial design is discussed in the following chapter.

The initial connection design included a bracket system used to connect the rocking CLT wall panel to the studs of the LiFS walls. It was designed to allow the shear loads to be transferred to the rocking CLT walls through the LiFS walls. The brackets were placed at one third and two third height of the wall on either side of the CLT wall panel. The bracket included slotted holes to allow the rocking of the CLT wall. The brackets were made using ¼ in. thick steel plates. Two slotted holes were cut on the face connecting the bracket to the LiFS walls. Four 5/8 in. threaded rods were used to connect each of the bracket to the LiFS wall studs. In order to allow the rocking movement of the CLT wall panel, the rods were kept snug-tight on the brackets. The bracket was further connected to the CLT wall panel using five 5/8 in. threaded rods. The shearwall elevation including the bracket is shown in Figure. 2.4.

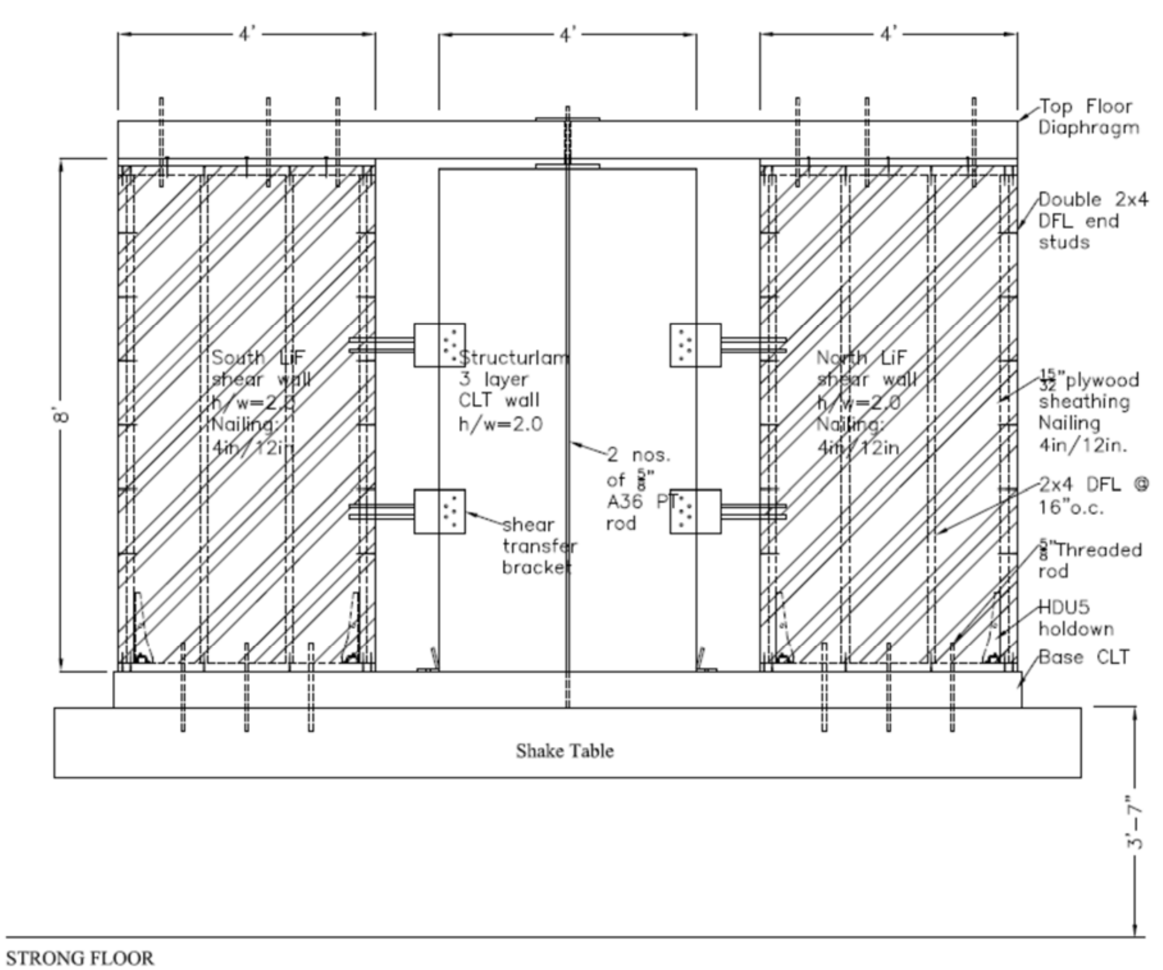


Figure 2.4: Elevation drawing of CLT-LiFS shearwall assembly

After the initial test, these brackets were replaced since the double end studs of the LiFS walls failed due to bending. This happened due to the inability of the threaded rods to slide in the vertical slotted holes. Therefore, a modified connection design was used in subsequent tests. A connection was designed to connect the CLT top floor diaphragm to the rocking CLT wall panel to effectively transfer the shear forces. The bracket was attached in the middle of the panel. A thick 1 ½ in. steel rod was used in the connection between the bracket and the wall panel. The bracket allowed vertical uplift of the wall through slotted hole connection. Further the bracket was connected to the CLT top floor diaphragm through 16 Simpson Strong Tie SDS 25600 (1/4 in. by 6 in.) screws. The bracket was made using ¼ in. thick steel plate for the face connecting the bracket to the CLT top floor diaphragm and ½ in. thick steel plates for connection between the wall panel and the bracket. Additionally, a slot was cut on the 1 in. steel plate transferring the post tension forces to the CLT panel in order to accommodate for the bracket. The dimensions of the bracket are shown in Figure. 2.5. Also the shear wall elevation with the modified shear transfer design is illustrated in Figure. 2.6.

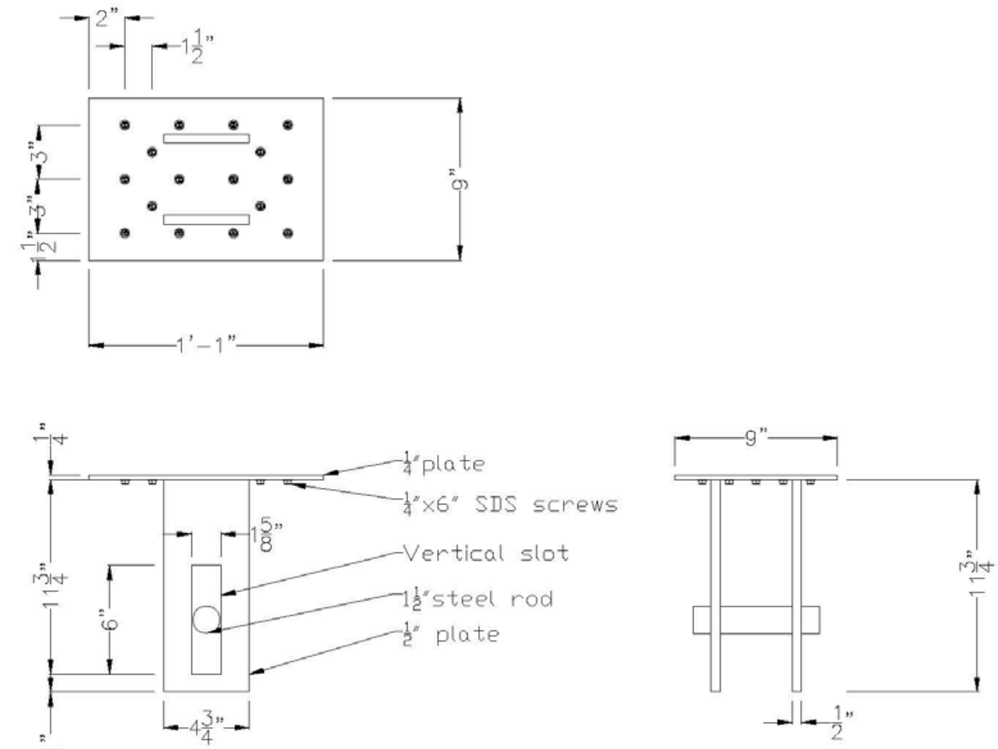


Figure 2.5: Detailed drawing for shear transfer connection for Rocking CLT wall panel

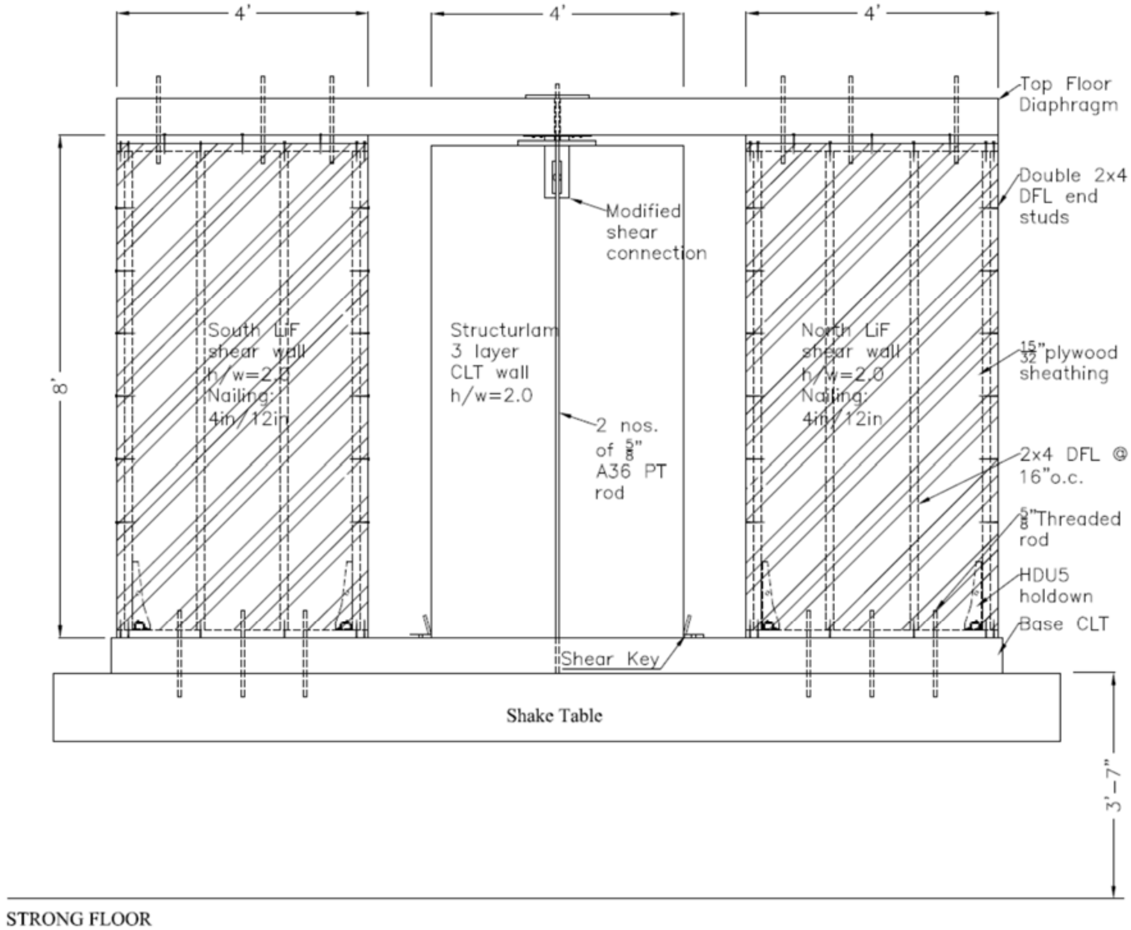


Figure 2.6: Elevation drawing of CLT-LiFS shearwall assembly with modified shear connection design

2.2.5 Seismic mass

A separate hydraulic system was installed to support the seismic mass of the structure while repair work was carried out on the walls. Four hydraulic cylinders were installed on existing columns around the shake table. Two W14X61 beams of length 15 ft. 1 ½ in. were mounted on the hydraulic cylinders along the North-South direction and were used to support the framing system carrying the seismic load. The frame holding the load was made of HSS 3X3X3/8 spaced at 1 ft. along the north-south direction and held together with two long pieces spanning across the entire length. The frame was attached to the CLT top floor diaphragm with angle brackets welded to the frame and SDS 25600 screws to connect to the CLT. Further, M6X12 sections welded on each end to the HSS tubes were used. The frame was also a part of the seismic mass during the test.

As part of the seismic mass for the structure, five concrete blocks were installed and tied to the CLT floor diaphragm through 1 in. threaded rods. Each block was 6 ft. by 2 ft. by 1 ft and each block weighed 1800 lbs. The layout of the blocks is shown in Figure 2.7 and the blocks were secured for safety. The total seismic mass acting on the structure including the weight of the floor diaphragm, the steel frame and concrete blocks was calculated to be 12.5 kips. The W14X61 beams were pinned to the hydraulic cylinders and were fixed to the columns at about 6 in. from the frame through bolts during the test for safety. Safety straps were also installed for additional safety. The safety straps were given enough slack so that the straps were engaged only when the structure completely fails and thus would not interfere during the actual test.

Additionally a modification was found to be necessary after torsion was induced during the MCE test. In order to resist torsion, four rollers were installed to prevent torsion. The rollers were attached to the W14X61 beams and were allowed to slide between the column flanges. The rollers were 13 in. long and were allowed to guide the C-section on the frame to resist the torsion load.

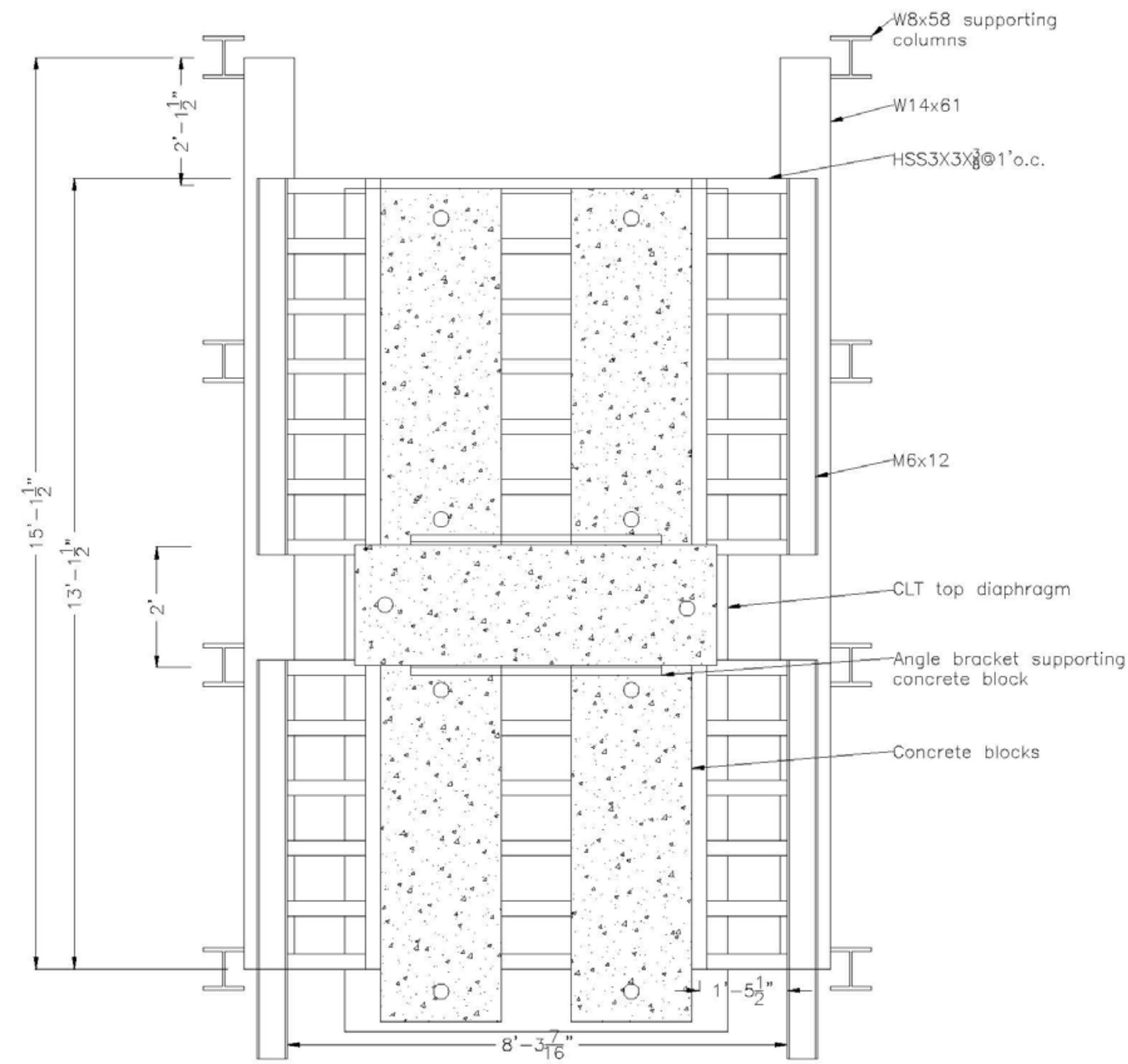


Figure 2.7: Plan view layout of the seismic mass counter-weight

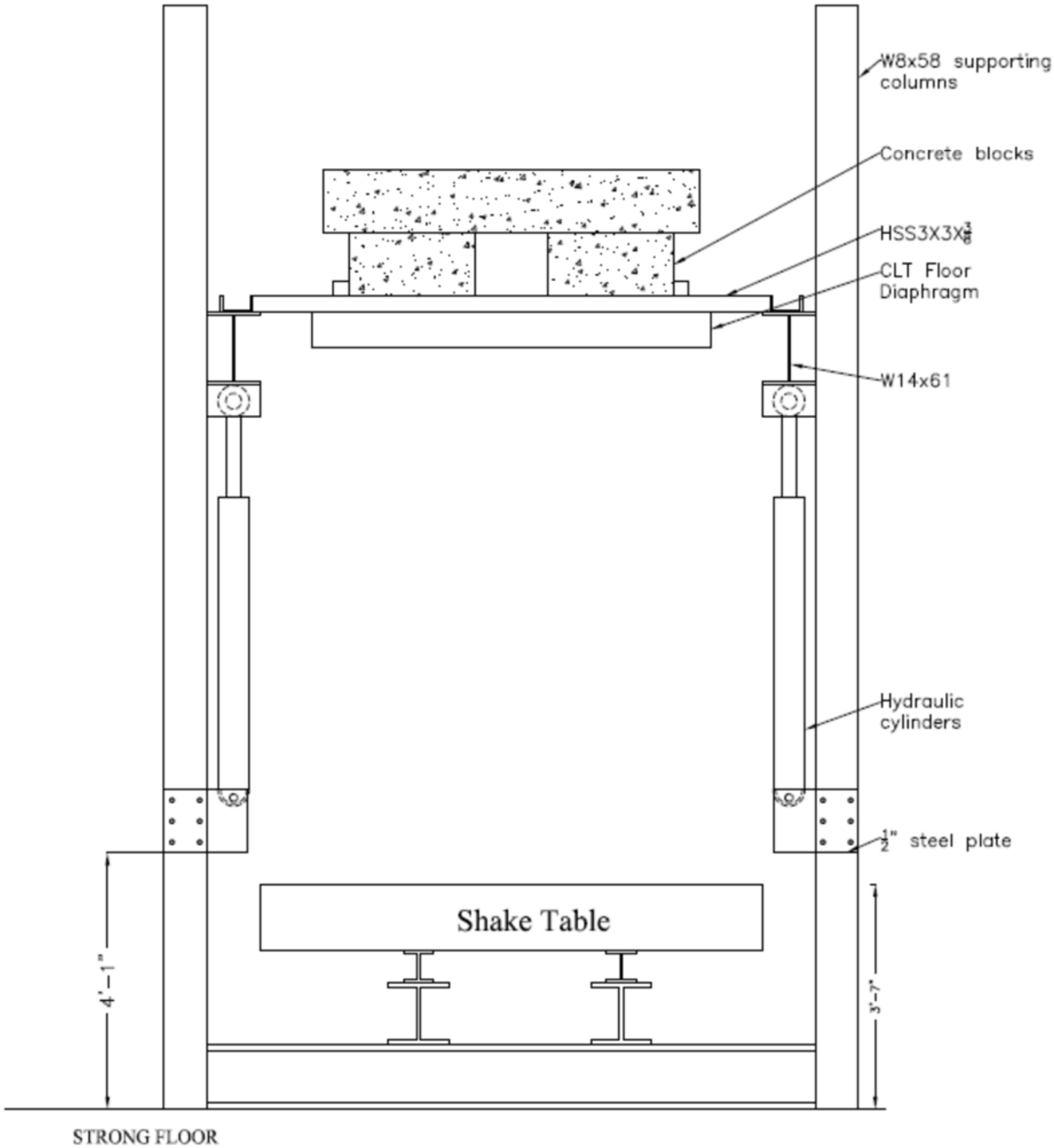


Figure 2.8: Elevation view of hydraulic support system for seismic counter-weight

2.2.6 Test Setup Construction

There were a few space and equipment constraints in order to construct the experimental setup. Thus a separate hydraulic system was installed in order to hold the seismic mass and provide assistance while carrying out repair works under the structure. The construction sequence for the test set up is described in detail in this section.

2.2.6.1 Installation of hydraulic system

A separate hydraulic system comprising of 4 hydraulic cylinders were installed around the shake table on the four columns located around the shake table. Two ½" thick steel plates were used to attach the hydraulic cylinders to the flange of the columns. The steel plates were bolted to the flange using six A307 bolts and a 1 ½" diameter pin material was used to pin the cylinders to the steel plates. Two W14x61 beams spanned in the North-South direction over the hydraulic columns to support the seismic mass. The hydraulic cylinders with the beams were first erected in place. The hydraulic cylinders were operated using a small hydraulic pump unit with controllers to operate the hydraulic cylinders. The controller allows operation of one hydraulic cylinder at a time. The seismic mass was raised and pinned to the column while repair work was carried out. During the testing the hydraulic cylinders and the beams were detached from the seismic mass and pinned 6 inches below the seismic mass for additional safety.

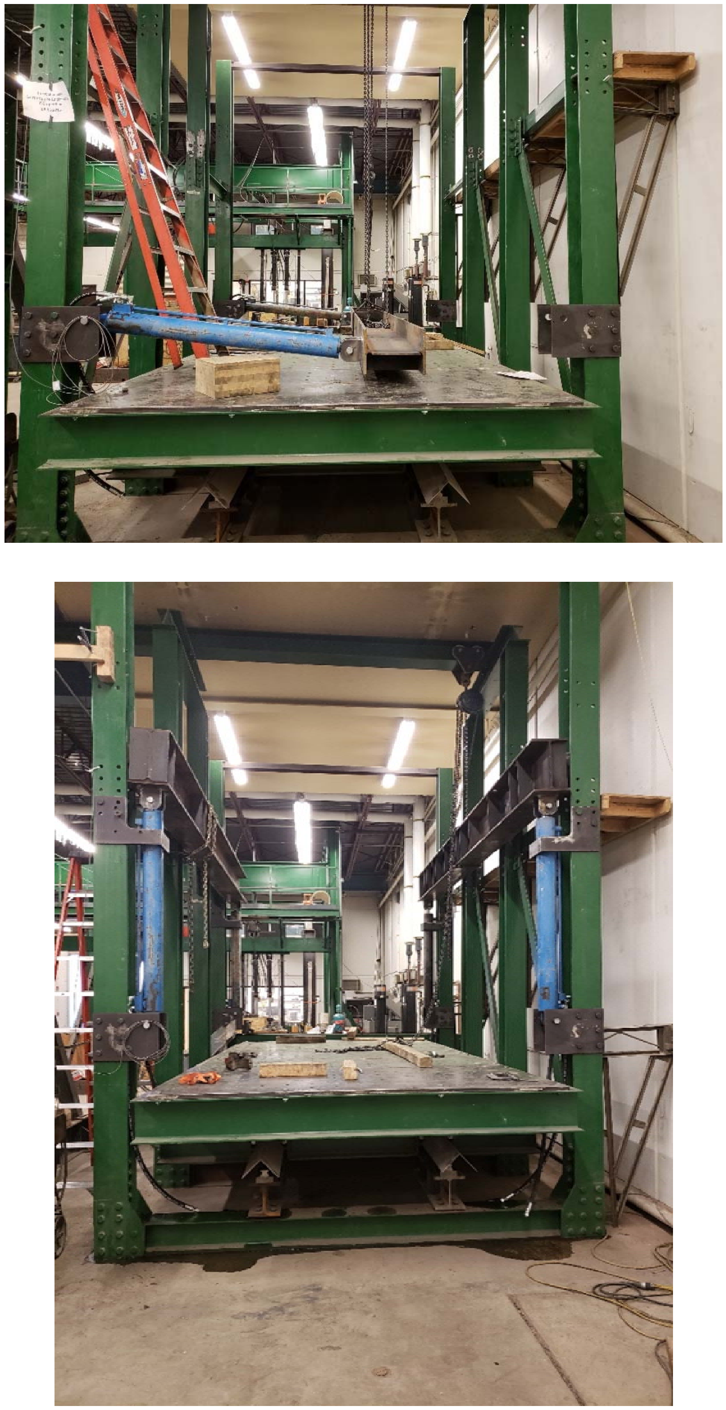


Figure 2.9 (a): Installation of hydraulic system

2.2.6.2 Seismic mass support

A frame was built in order to support the seismic mass. The frame was constructed using HSS3x3x3/8 steel sections as described in section 2.2.5. The CLT diaphragm was attached to the frame using eight angle brackets. The frame and the CLT diaphragm was placed over the beams using a chain hoist and fork lift and then the frame was attached to the beams using four small bolts to hold it in place on the beams.

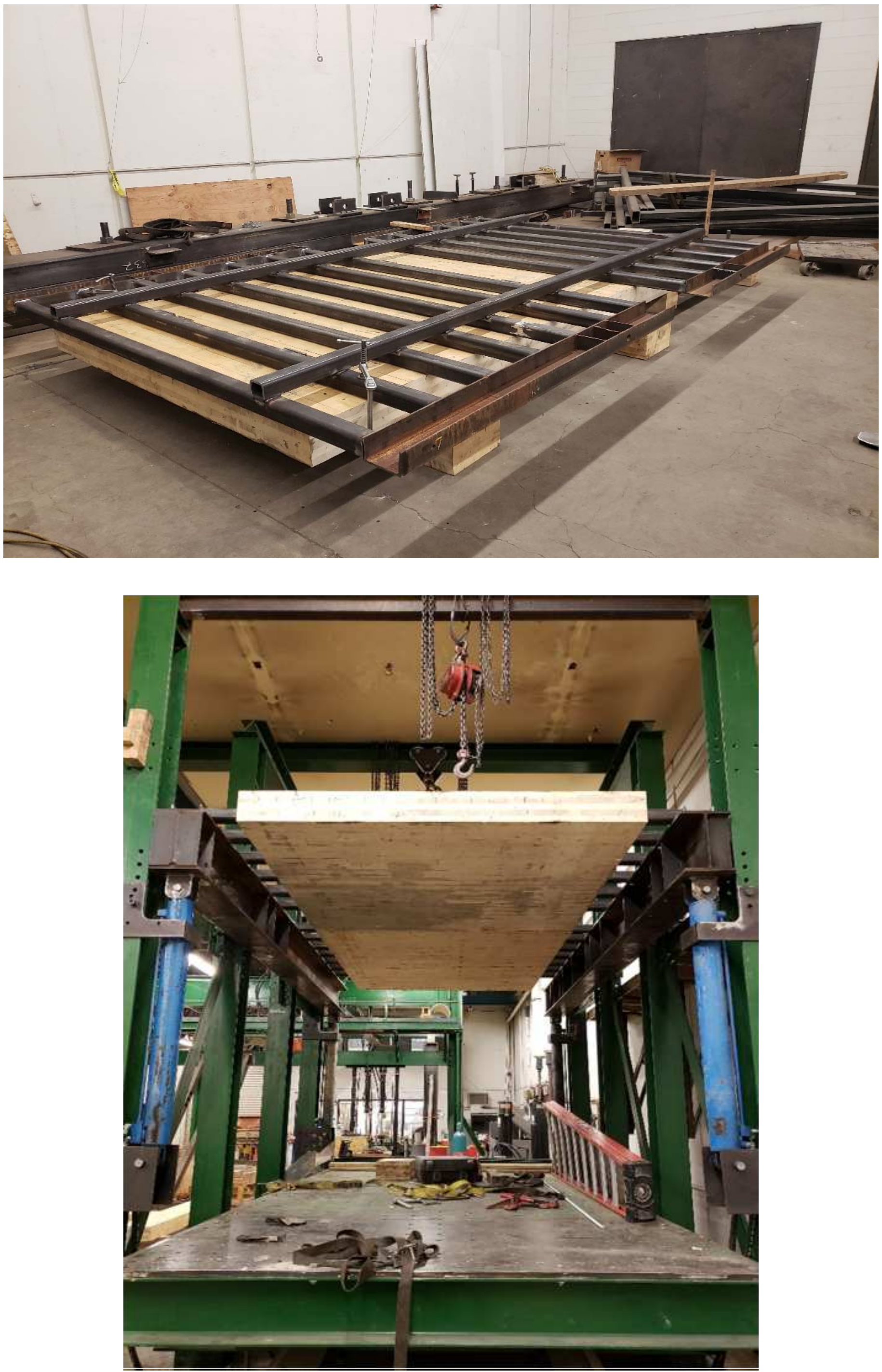


Figure 2.9 (b): Steel frame and CLT diaphragm before and after erection

2.2.6.3 Seismic mass

Once the frame of the seismic mass had been erected in place, the seismic mass was loaded to the frame. Five concrete blocks of dimensions 6 ft. x 2 ft. x 1 ft were used as seismic mass for the test. The last block was placed over the other four blocks due to lack of space and each of the four blocks were attached to the CLT diaphragm using two 1” threaded rods. The final block was secured in place using two angle brackets on either side of the block. Additional lifting (safety) straps were used to fasten the block to the angle brackets. The blocks were placed in such a way that the mass was uniformly distributed.

Figure 2.9 (c): Concrete blocks placed for seismic mass

2.2.6.4 Construction of walls

Once the seismic mass was placed in position over the frame, the seismic mass was lifted up approximately 2 ft. over the height of the walls and pinned to the columns. The LiFS walls and gravity frames were constructed first before installation. The gravity wall frames were then placed in position and then the base CLT for the hybrid shear wall system was installed in place. The rocking CLT wall was then placed over the base CLT followed by the installation of the LiFS walls on either side of the CLT wall panel. Once the walls were installed in place the seismic mass was lowered over the walls. The seismic mass was attached to the double top plates of stud walls using 5/8" threaded rods. The rods were attached to the CLT diaphragm to which the seismic mass was attached. The hydraulic system was detached and lowered from the seismic mass once the mass was set over the walls.

Figure 2.9 (d): Installation of walls and complete test setup

2.3 GROUND MOTION

The 1989 Loma Prieta ground motion recorded at Capitola Recording Station was used and scaled to various seismic intensities for the test program. The site location for the structure was chosen to be in California with a stiff soil class D. The mapped MCE spectral response acceleration parameter at short period S_s as 2g and period at 1s, S_1 as 1g were determined using the seismic maps specified by ASCE/SEI, 2010. The site coefficient for the short period, F_a was 1 and at period 1s, F_v was 1.5. The ground motion was scaled to match the response spectrum value at the approximate time period of the structure which was found to be 0.09s for the structure. The ground motion was scaled to three different intensities namely SLE (50% probability of exceedance in 50 years), DBE (10% probability of exceedance in 50 years) and MCE (2% probability of exceedance in 50 years).

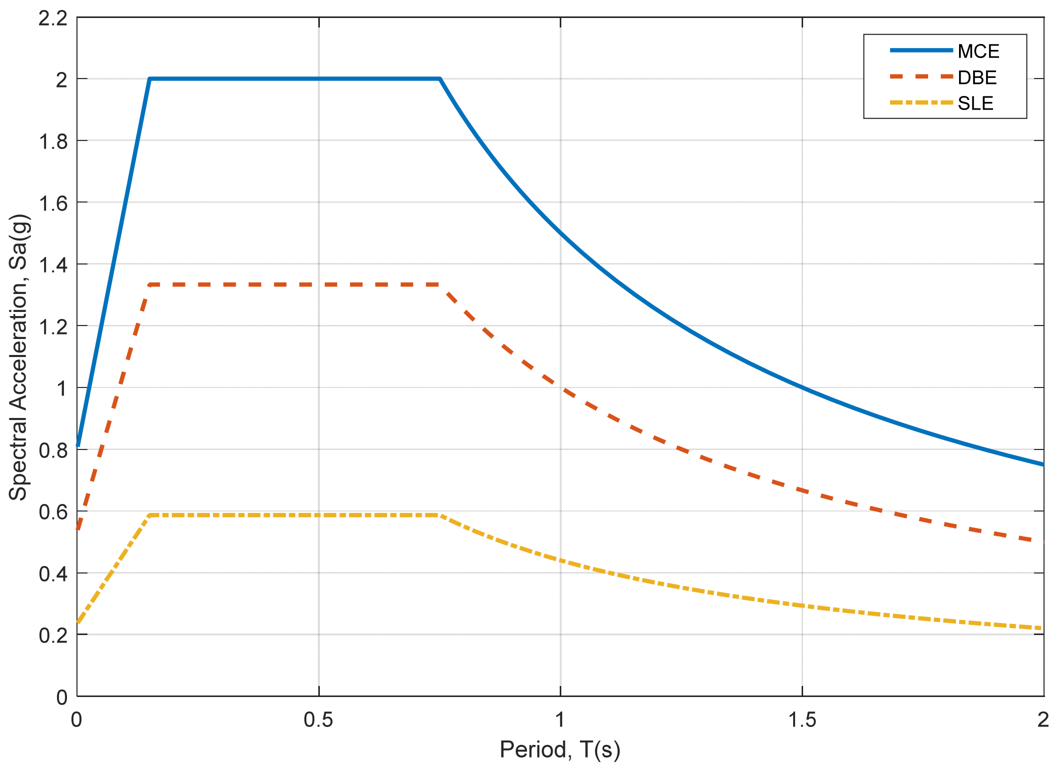


Figure 2.10: Design acceleration response spectra

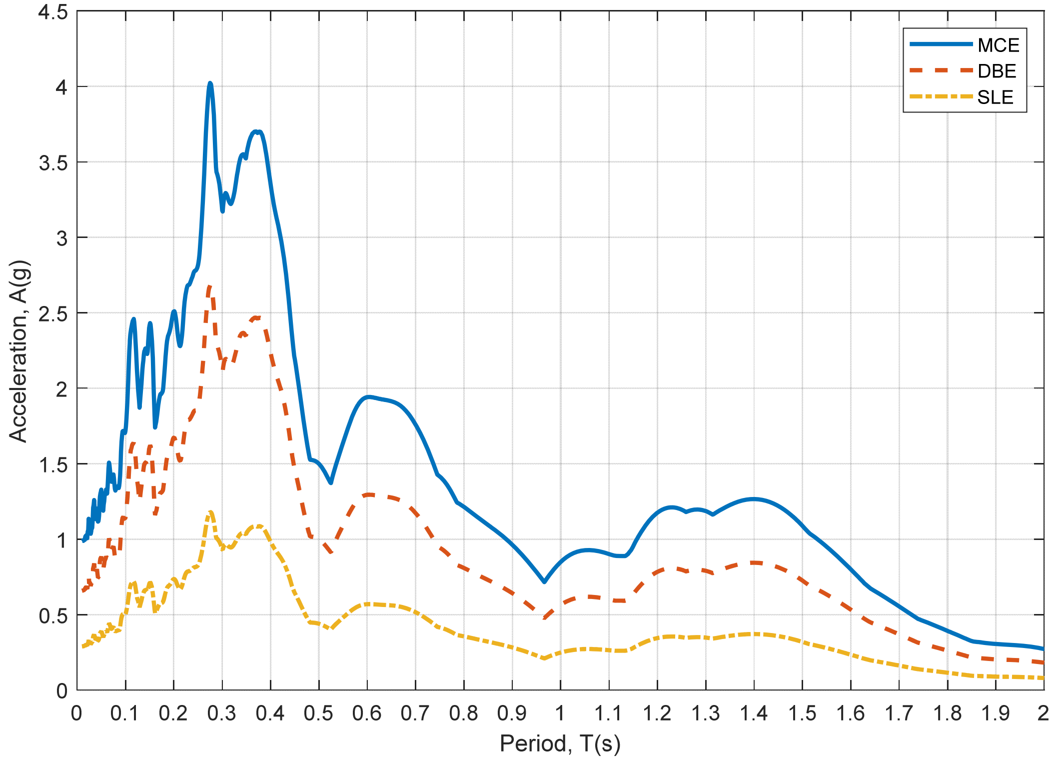


Figure 2.11: Spectral acceleration for Loma Prieta scaled to SLE, DBE and MCE levels

The final test carried out was an attempt to completely collapse the structure in order to observe and document the collapse mechanism for the specimen. For this test, the Northridge earthquake recorded at the Rinaldi station was used for this test. The original earthquake record was used and since the allowable stroke length of the shake table was 20 in. (0.508 m) the seismic intensity of the record was reduced to 85 per cent of the original record to be able to reproduce the motion. The response spectrum for this record is shown in Figure 2.12.

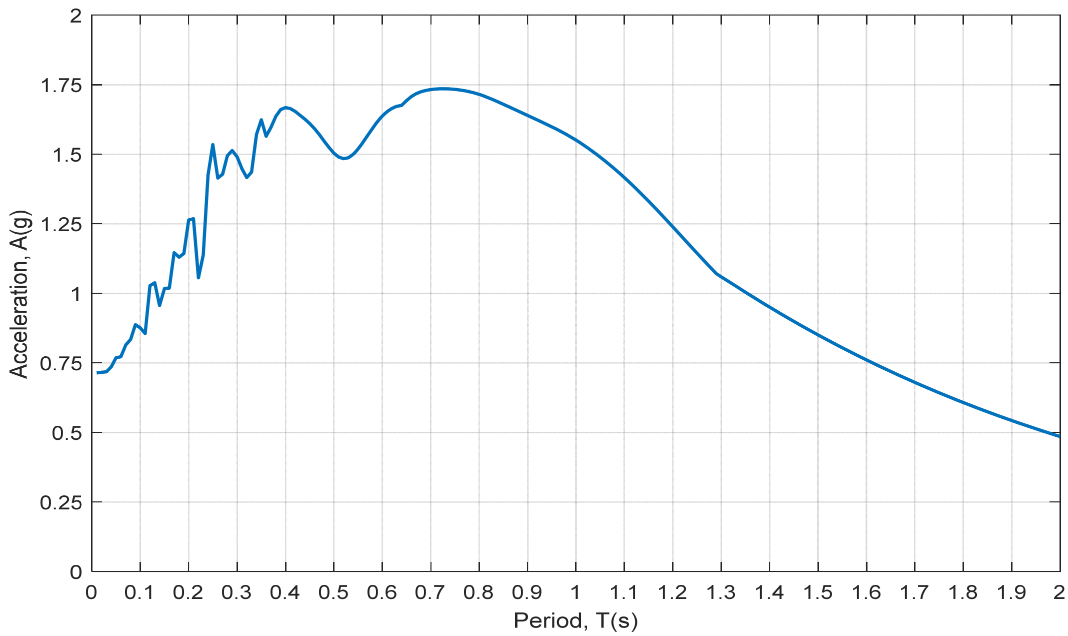


Figure 2.12: Spectral acceleration for Northridge Rinaldi station ground motion

2.4 INSTRUMENTATION

The response of the structure was measured using string potentiometers, linear potentiometers, spring potentiometers and strain gages. A total of 15 instruments were used in each of the test to record the measurements of the response throughout the testing process. The instrumentation plan for the structure is shown in Figure 2.13.

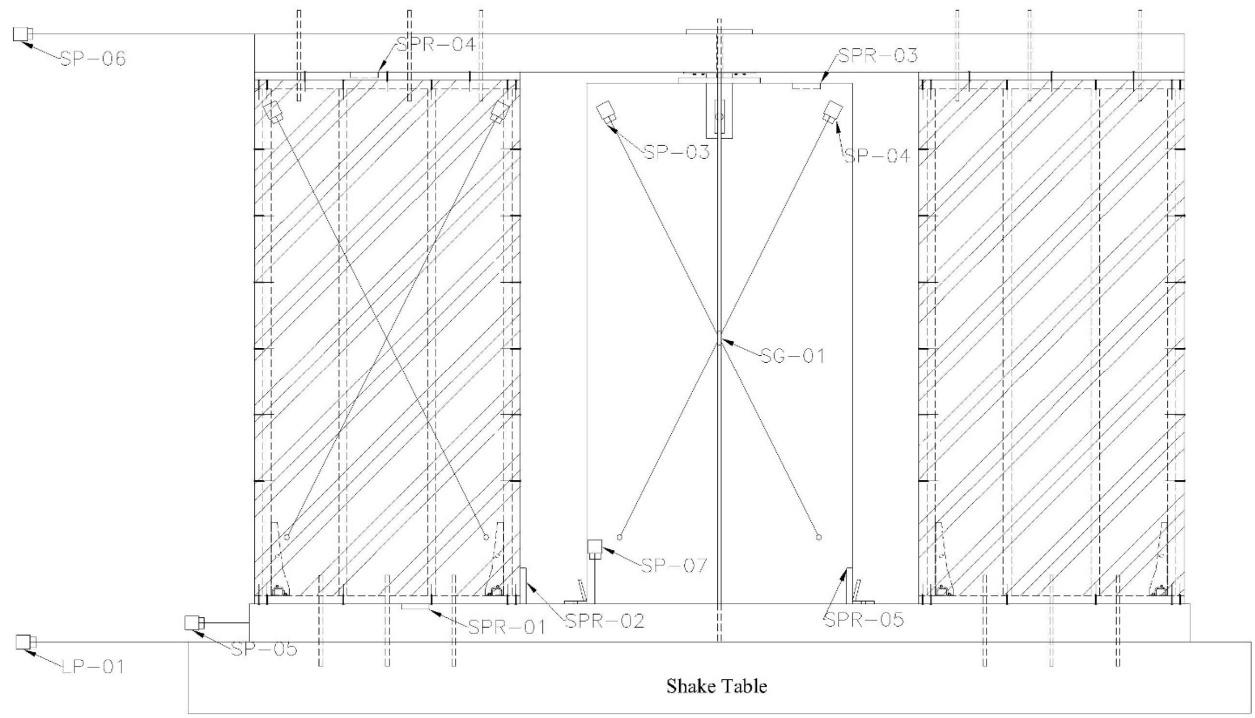


Figure 2.13: Instrumentation plan

Instrumentation was only installed on the shear wall assembly and no instrumentation was installed on the gravity frames. Since the structure plan was symmetric it was assumed that the LiFS walls on either side of the rocking CLT wall responded similarly and hence only the South LiFS wall was instrumented. A string potentiometer was used to record the top mass displacement and a linear potentiometer was used to measure the motion of the shake table. A string potentiometer was also used to check if there was any racking on the CLT wall panel or sheathing of the LiFS wall. The loads on the post-tensioned rods were recorded using strain gages. CLT wall uplift was recorded using a spring potentiometer. Spring loaded potentiometers were also used to record if there was any sliding on the LiFS walls and uplift of the LiFS wall. Table 2.3 describes the location of each instrument.

Table 2.3: Instrumentation List

Designation no.	Instrumentation no.	DAQ No.	Channel no.	Location
SPR01	SPR01	DAQ 2	CH01	South LiF sliding
SPR02	SPR02	DAQ 2	CH02	South LiF uplift
SPR03	SPR03	DAQ 2	CH03	CLT sliding (Top mass)
SPR14	SPR14	DAQ 2	CH14	South LiF sliding (Top mass)
SPR15	SPR15	DAQ 2	CH15	CLT uplift
SP01	SP09	DAQ 1	CH09	South LiF racking
SP02	SP06	DAQ 1	CH06	South LiF racking
SP03	SP10	DAQ 1	CH10	CLT racking
SP04	SP08	DAQ 1	CH13	CLT racking
SP05	SP05	DAQ 1	CH05	Base CLT
SP06	SP04	DAQ 1	CH04	Top displacement
SP07	SP12	DAQ 1	CH12	CLT uplift
LP01	LP01	DAQ 1	CH07	Table displacement
SG01	SG01	DAQ 1	CH1	East PT
SG02	SG02	DAQ 1	CH2	West PT

2.5 TEST PROGRAM

The test program included a series of tests with the scaled ground motion applied to the structure. The following table describes the test matrix.

Table 2.4 Test matrix

Test Number	Grnd Motion	Grnd Motion scaling	Duration (sec)	S_a (g)
1a	Loma Prieta	SLE	39.995	0.44
1b	Loma Prieta	DBE	39.995	1.01
1c	Loma Prieta	MCE	39.995	1.52
2a	Loma Prieta	SLE	39.995	0.44
2b	Loma Prieta	DBE	39.995	1.01
2c	Loma Prieta	MCE	39.995	1.52
3	Loma Prieta	MCE	39.995	1.52
4a,b,c	Northridge	Original	14.945	0.88

Test 1:

The first test was conducted with the bracket connection between CLT panel and LiFS walls as shown earlier in Figure 2.4. As mentioned, due to failure of the connection system these brackets were changed after the initial test. The test results are further discussed in the following chapter. The tests 1a, 1b and 1c used the same specimens with repair works carried out between the tests. The specimens were not replaced since no significant damage occurred to the structure.

Test 2:

The connection for shear transfer for the rocking CLT wall panel was modified as shown in Figure 2.6. This connection design performed well allowing the rocking motion for the CLT without any hindrance. The structure was subjected to the ground motions specified in Table 2.4 with no repair work carried out between tests. However, the structure experienced significant torsion during the MCE ground motion and the test was aborted before the completion of the ground motion.

Test 3:

To counter the effect of torsion, a roller system was installed to prevent the structure from undergoing torsion. Four rollers were installed in the columns on the East and West side of the structure and the mass was allowed to slide on it thus attempting to prevent torsion. The structure was then subjected to MCE ground motion only. The structure performed as expected with no interference from torsion. Test1 and Test2 served as learning and refinement tests in order to enable the test setup to work correctly.

Test 4:

This test was conducted with the objective of studying the collapse mechanism of the CLT-LiFS shear wall system. The ground motion used for this test was the Northridge Rinaldi station earthquake ground motion recording from the Rinaldi station. Three MCE ground motions were run one after the other with no repair to the structure in between the tests. The structure was completely damaged during the third test with the CLT post-tension bars fractured. However due to failure of instruments the complete results of the collapse mechanism was not captured. The detailed results are discussed in the next chapter.

CHAPTER 3: EXPERIMENTAL RESULTS AND DISCUSSIONS

3.1 INTRODUCTION

In this chapter, the experimental observations and test results are discussed. As mentioned in the previous chapter, the initial tests were part of a learning process for test design and there were several modifications on the subsequent tests. Although this chapter reports the test results of all the tests conducted, Test 1 and Test 2 should be considered as a learning step as the failure mechanism was not ideal due to the test setup and helped in modifying the system for the development of Test 3 and Test 4.

The damage inspection observations carried out after every ground motion is also reported. Every component of the structure was inspected after every ground motion to identify if there is any need for repair/replacement of component. Based on these observations, changes in design were carried out to obtain the successful test methodology. The gravity frames and the rocking CLT wall panel were not changed throughout the test program since there was no damage to these components as expected. The LiFS walls were replaced after every MCE event.

The objective of Test 4 was to understand and document the collapse mechanism of the system. For this purpose, three ground motions were run one after the other without any repairs between the tests. However, due to instrumentation failure during the second test, complete test data was not available for the last test. The test observation for these tests are discussed qualitatively to examine the failure mechanism of the system.

3.2 EXPERIMENTAL OBSERVATION

Test 1a:

The first test set up included shear wall system shown in Figure 2.4. The structure was initially subjected to Loma Prieta ground motion scaled to SLE. As expected the rocking CLT wall panel and the LiFS walls experienced no damage and the structure was able to re-center completely. The CLT wall uplift was very low. However, there was some amount of sliding observed in the connection between the top mass and the LiFS walls. Thus in order to prevent sliding, Simpson Strong-Tie HGA10's were used as additional shear transfer connectors between the top mass and the LiFS walls.

Test 1b:

The post-tension rods were tensioned again before the test. As there was no damage observed from previous test, there were no repairs carried out. The structure was subjected to the Loma Prieta ground motion scaled to DBE intensity levels. It was observed there were very few sheathing edge nails starting to withdrawal. However no significant damage was observed on the structure and the same specimen was used for the next test.

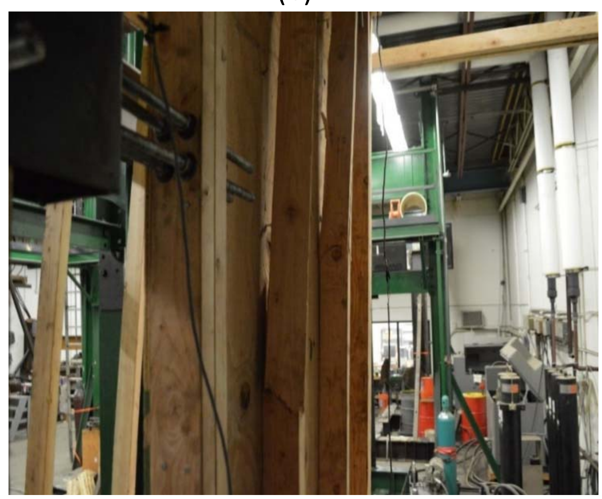
Test 1c:

The post-tensioned rods were tensioned again to the initial post-tension force before the test. The same rods were used since the rods had not yielded in the previous tests. The structure was subjected to Loma Prieta ground motion scaled to MCE hazard levels. During the test, bending failure was inflicted to the studs to which the shear transfer brackets were attached. Also the structure experienced torsion which was not expected. Since the failure of the LiFS wall studs was not desired, the brackets connecting the LiFS walls and rocking wall CLT panel were redesigned for use in the subsequent tests.

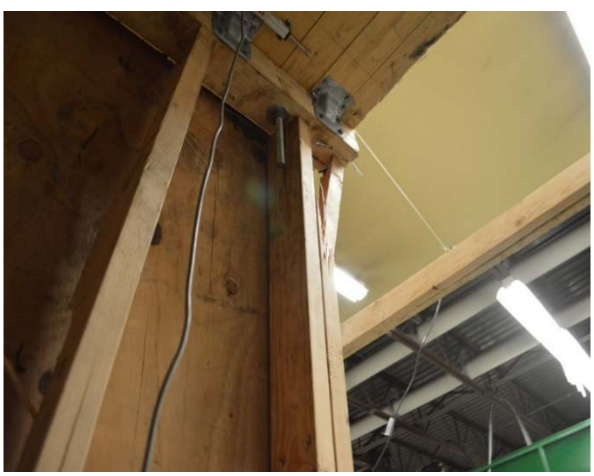
(a)

(b)

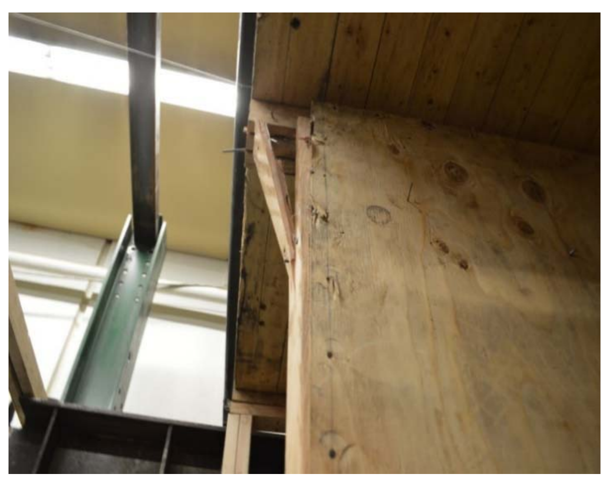
(c)



(d)



(e)



(f)

Figure 3.1: (a) Overall structure after the MCE event; (b), (c): Damage to the studs of the LiFS wall; (d): The failure of studs on LiFS walls; (e), (f): Damage to the connection between studs and top plate

Test 2:

The test setup was modified as shown in Figure 2.6. In this test, the Loma Prieta ground motion scaled to SLE, DBE and MCE and were run one after the other with no repair works carried out between the tests. However, inspections were carried out between the tests. As expected the structure experienced no damage after the SLE ground motion. The sheathing of the LiFS walls was slightly shifted after the DBE ground motion, but no significant damage was observed after the DBE ground motion. The structure was then subjected to the MCE ground motion. However due to significant torsion the test was not completed and the ground motion was stopped without completion.

It was observed that the connection modification for the shear transfer system between the top mass and the rocking CLT wall panel performed well under the MCE intensity shake until the point of the test stoppage. The rocking CLT wall panel was rotated out of plane due to torsion caused by the test setup since the rocking CLT wall panel was attached to the top mass and there was no resistance to the out of plane rotation. The torsion was induced even though there was even distribution of weight on the structure eventually resulting in the installation of roller support to prevent torsion in subsequent tests.

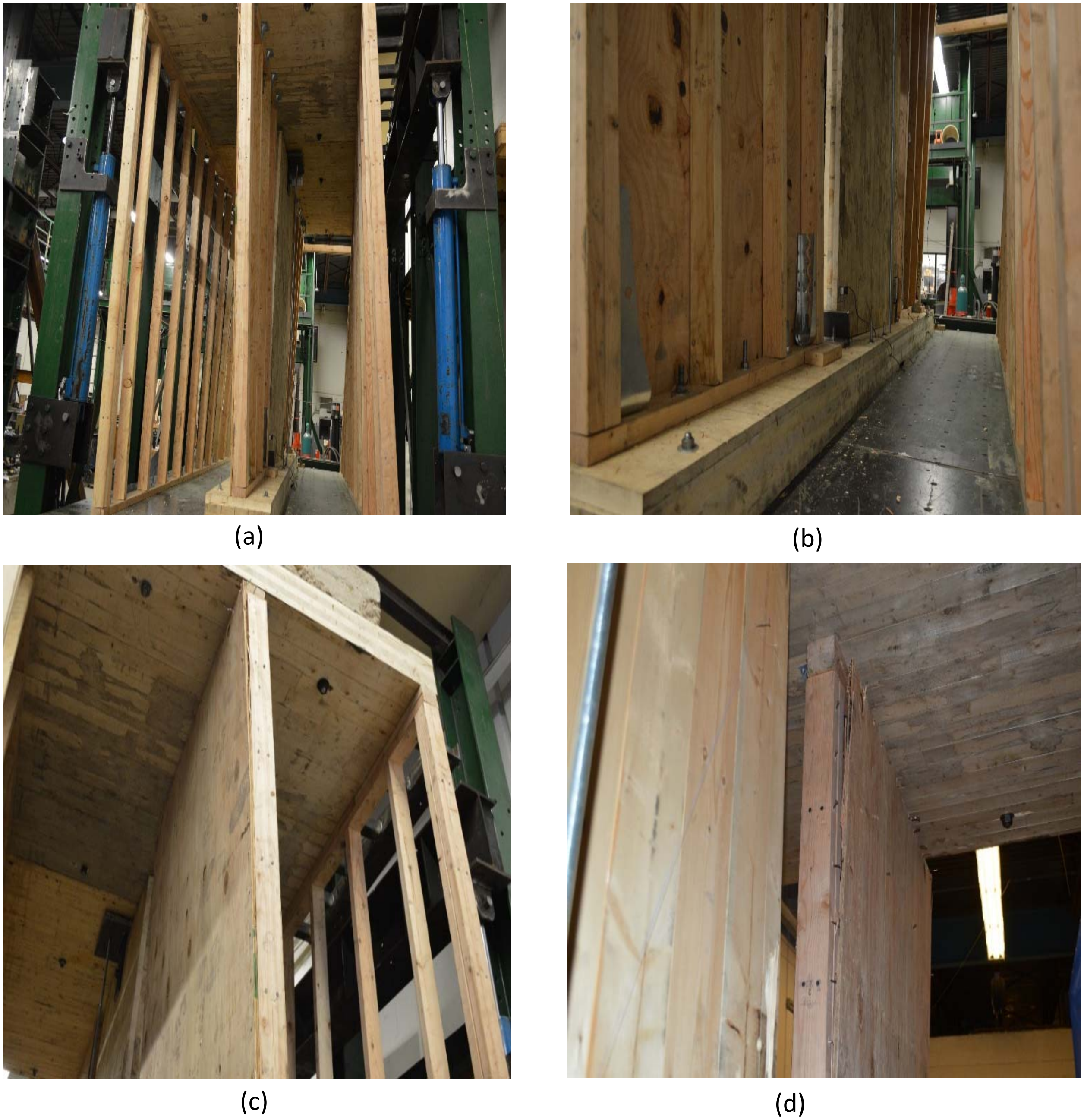


Figure 3.2: (a): Overall structure after the MCE event; (b): The shift of the CLT wall panel due to torsion; (c), (d): Damage to the sheathing and withdrawal of nails in the LiFS walls

Test 3:

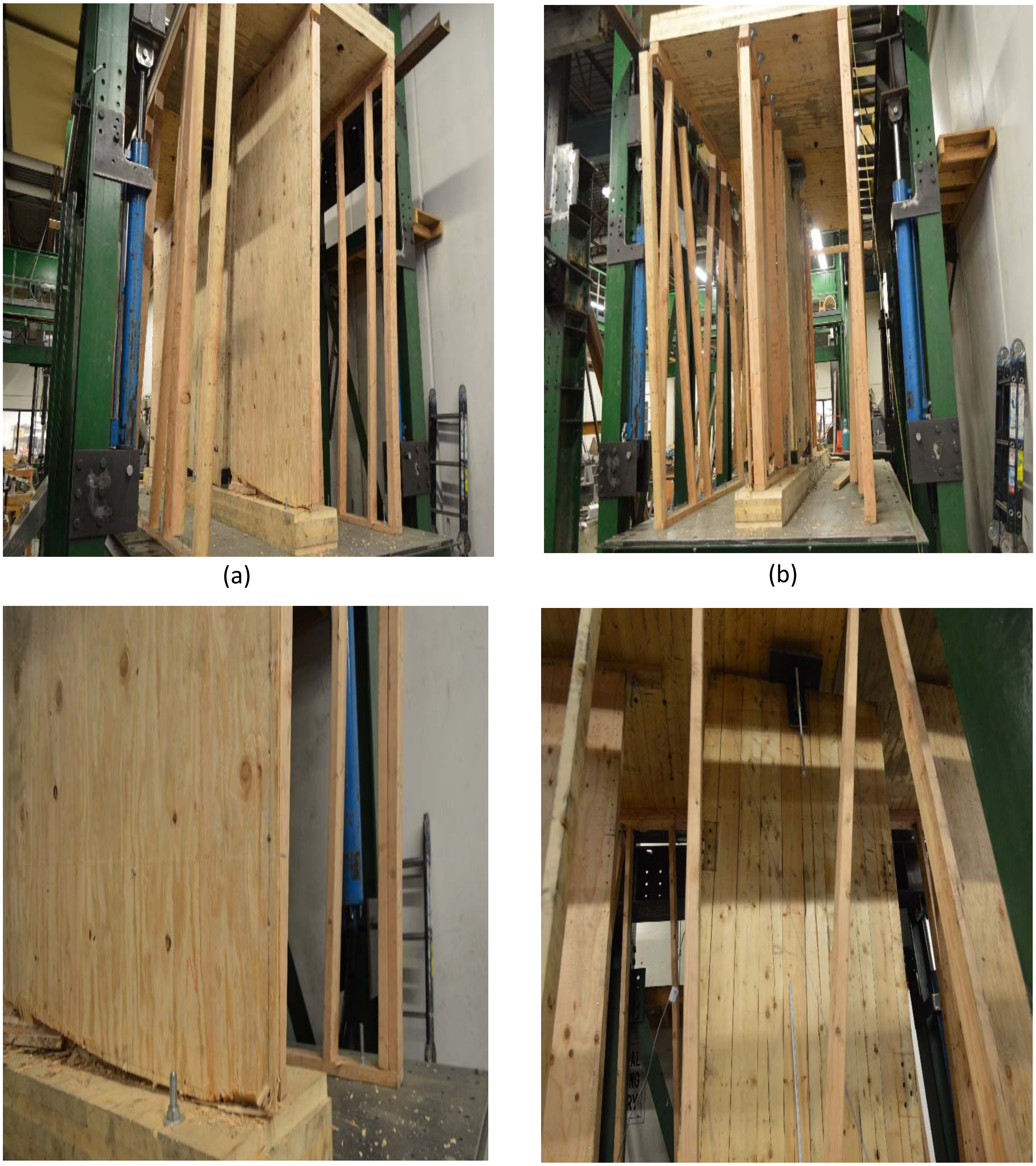
This test was carried out after adding a roller system to prevent the torsion that occurred in the previous test. In this test, the structure was subjected to only MCE ground motion. The performance of the test structure was as expected with no accidental torsion from the test setup. The structure was able to completely re-center after the MCE intensity shake. The LiFS wall sheathing was damaged along the edges with edge nails partially withdrawing. There was no damage to the studs of the LiFS walls and the gravity frames of the structure.

(a) (b) (c) (d)

Figure 3.3: (a): Overall structure after the MCE event; (b): Damage to the sheathing of the LiFS wall; (c): Edge nail withdrawal on the sheathing of the LiFS wall; (d): Damage to the sheathing of the LiFS wall

Test 4:

The Northridge ground motion record from the Rinaldi station was used for the final test. The structure was subjected to three ground motions one after the other with no repair works carried out between the tests. The structure was able to completely re-center after the first ground motion and there was some withdrawal of nails on the LiFS walls sheathing. The structure was checked for safety before the second ground motion. There was some significant damage to the LiFS structure after the structure was subjected to the second ground motion. Most of the sheathing nails on the LiFS walls were either withdrawn or the sheathing was damaged itself. The holdown on the South end of the South LiFS was damaged indicating some amount of damage was sustained by studs in the LiFS. The structure was once again checked for safety before running the third ground motion. The structure was completely damaged at the end of the third ground motion with the top plates of the LiFS walls completely sheared off the studs. The rocking CLT wall panel was not able to re-center the system due to fracture of the PT bars. The gravity frames on the east and west side of the hybrid shear wall system was completely damaged at the end of the third ground motion. The seismic mass was secured by the safety straps which engaged during the ground motion. The string pot measuring the displacement at the top of the system failed during the second ground motion and therefore the lateral displacement data was not recorded for the second and third ground motions. However, this test was useful to understand the collapse mechanism of a rocking CLT wall panels from a qualitative perspective.



(a) (b)

(c) (d)

Figure 3.4: (a), (b): Damage to the overall structure; (c): Damage to the sheathing of the LiFS wall; (d): PT bar fracture and damage to the studs of the gravity wall

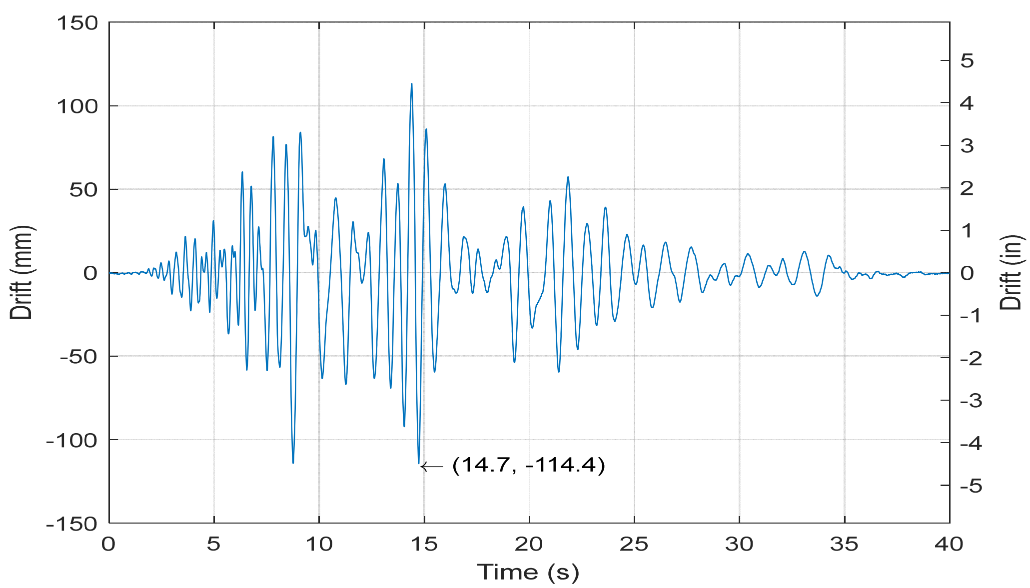
Figure 3.4: (e), (f): Rocking CLT wall panel after the test; (g), (h): Damage to the studs of the LiFS wall and holdown of the LiFS wall

3.3 EXPERIMENTAL RESULTS

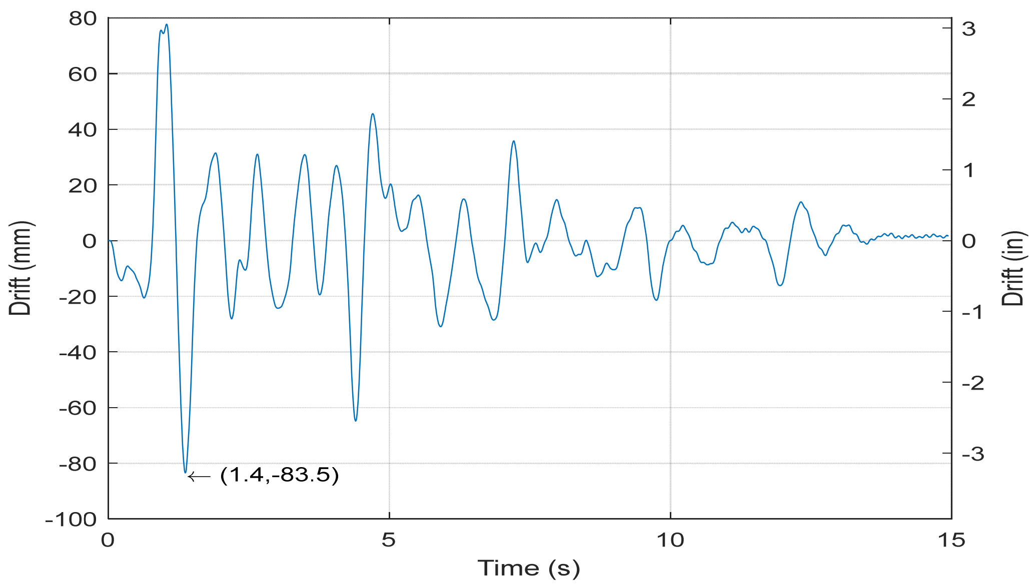
The following section presents the test results from each of the shake table tests. It should be noted that the shake table results from the Test 1 and Test 2 are not reported, these results were only used for test setup planning purposes. Test 3 and Test 4 results are discussed in detail below.

3.3.1 Inter-story Drift

The inter-story drift for the test structure was calculated by taking the difference between the string potentiometer measuring the displacement at the roof diaphragm and the table displacement recorded by the linear potentiometer. From the test results for Test 3 and 4a, it can be seen that the test structure was able to completely re-center after a major seismic event. Figure 3.5 shows the time history plot from test 3 and test 4a.



(a)

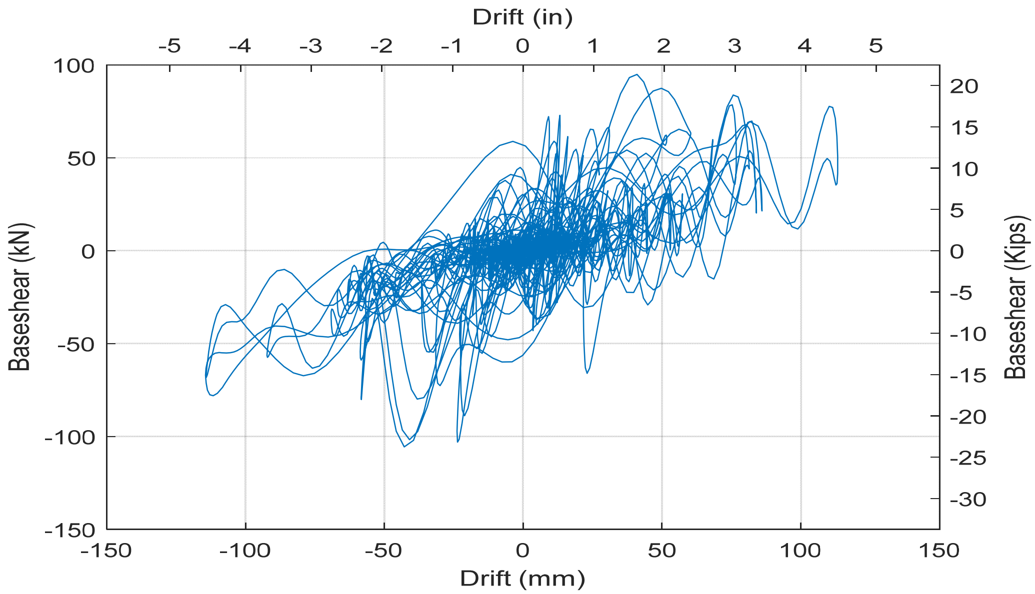


(b)

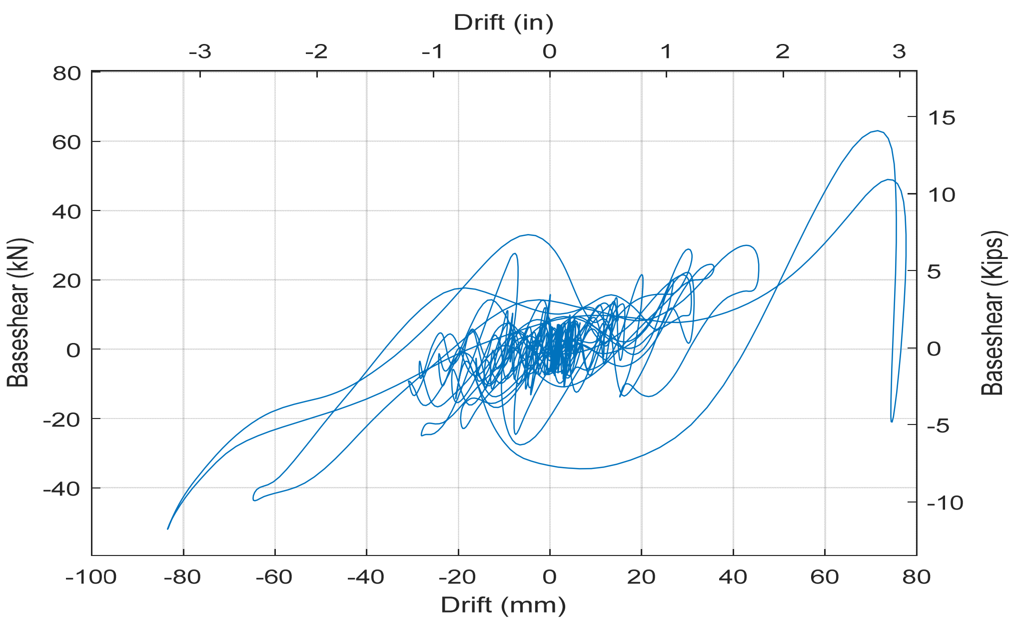
Figure 3.5: Interstory drift time-history: (a) Test 3; (b): Test 4a

3.3.2 Global Hysteresis

The base shear force was estimated from the acceleration of the seismic mass using Newton's second law of motion. The acceleration was obtained from inter-story drift data recorded through double differentiation and noise filtering. The hysteresis plot for Test 3 and Test 4a are shown in Figure. 3.6. The maximum base shear force calculated for the Loma Prieta ground motion at MCE and the Northridge Rinaldi ground motion was found to be 23.7 Kips and 14 Kips, respectively. The hysteretic curve was not the ideal curve one observes from modeling since the energy dissipation may depend on factors such as the energy dissipated by the sheathing nails of the LiFS walls, the rocking of the CLT wall panel and frictional losses between any moving parts.



(a)

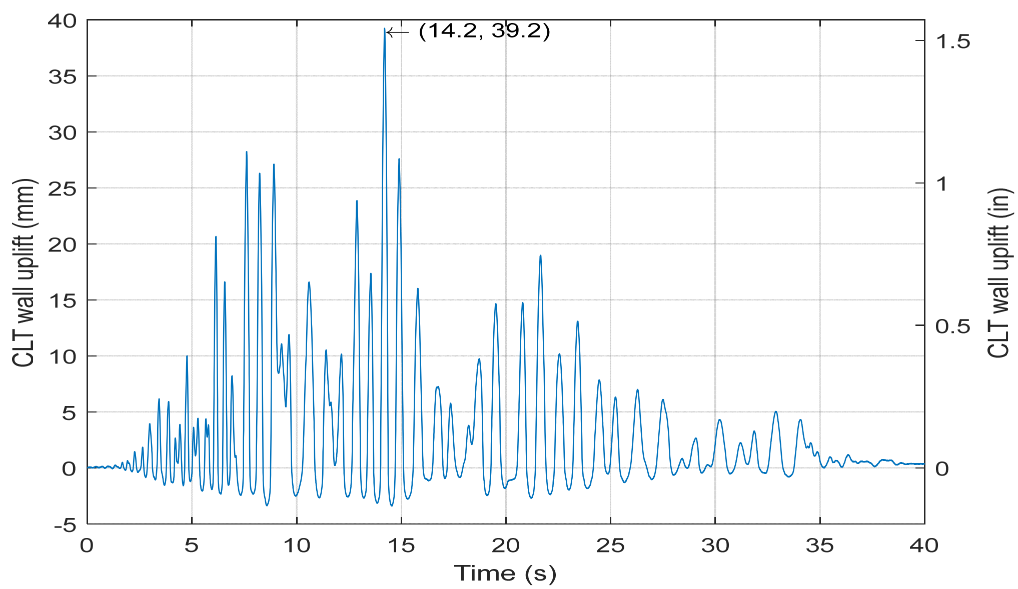


(b)

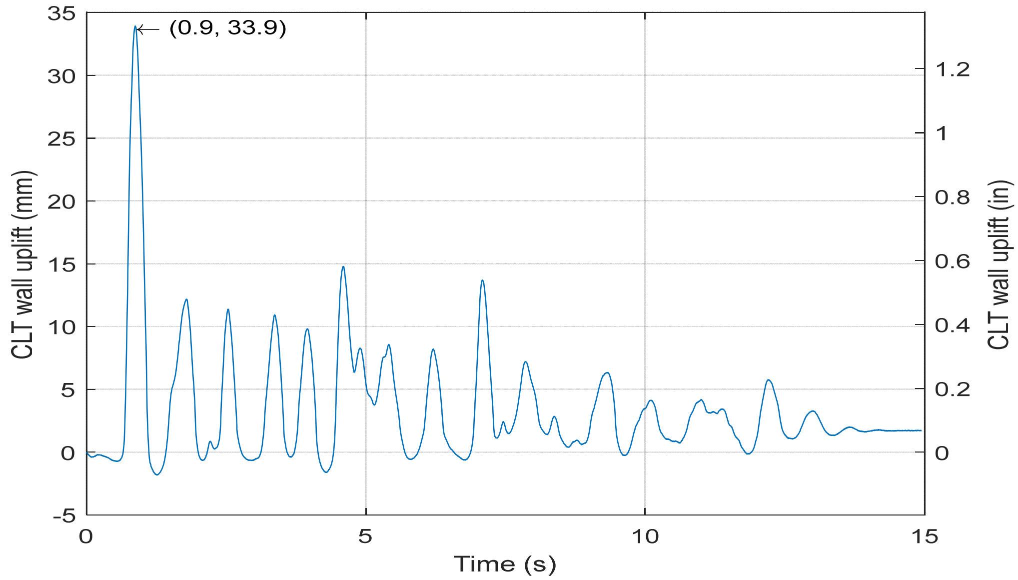
Figure 3.6: Global hysteresis: (a) Test 3; (b): Test 4a

3.3.3 Rocking CLT wall uplift

The uplift of the rocking CLT wall panel was recorded using linear potentiometers at the corners of the CLT wall panel to study the response behavior of the rocking CLT wall panel. The maximum uplift recorded for Test 3 and Test 4a are shown in Figure 3.7. The rocking CLT wall panel was allowed to rock freely on its base and it can be seen on the uplift displacement measured. The shear transfer connection designed for the rocking CLT wall panel was effective in transferring the shear and allowing free rocking of the panel.



(a)

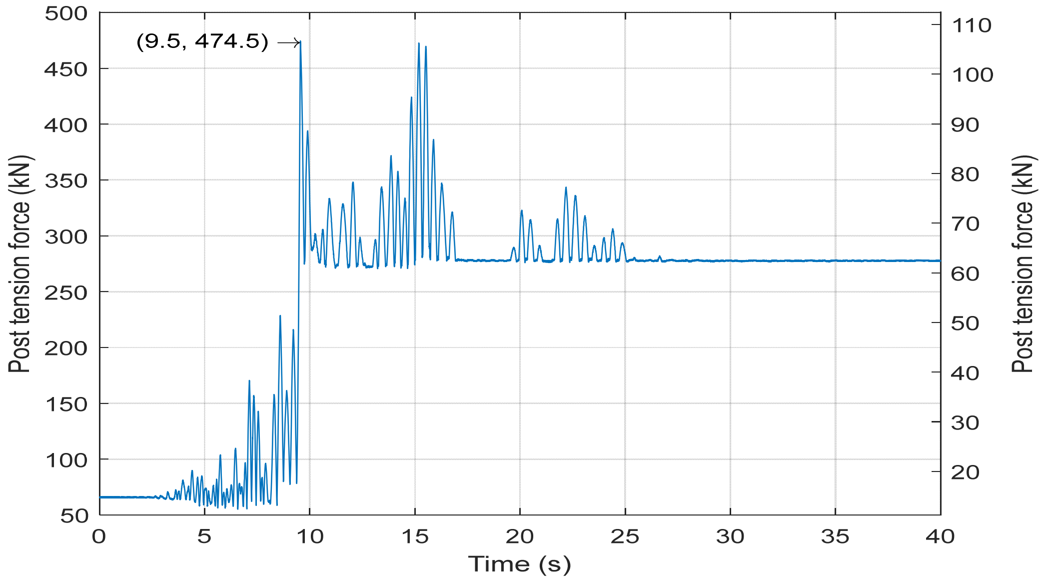


(b)

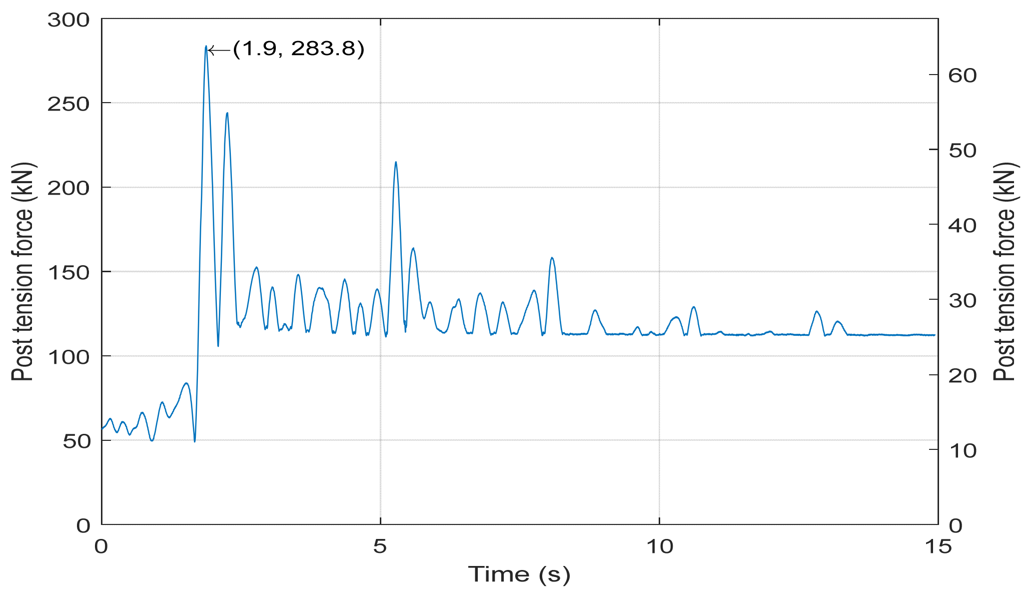
Figure 3.7: CLT Wall uplift time-history: (a) Test 3; (b): Test 4a

3.3.4 Post-tensioned bar forces

The post-tensioned bar forces were monitored throughout the test. The resultant PT force of the two bars on either side was assumed to act at the center of the wall panel. Since mild steel rods were used as PT bars for the experiments, it was observed that the rods yielded after every MCE level test and were replaced at the end of the test. Though the rods reached the theoretical yielding force, the structure was able to re-center after these large ground motions. Time-history plots for Test 3 and Test 4a are shown in Figure 3.8.



(a)



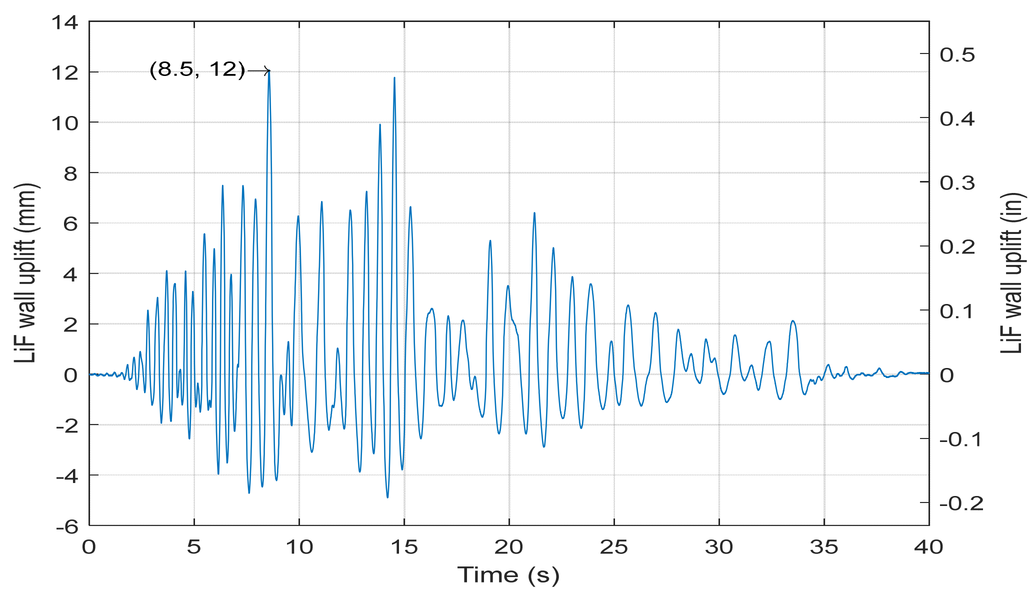
(b)

Figure 3.8: Total PT force Time-history: (a) Test 3; (b): Test 4a

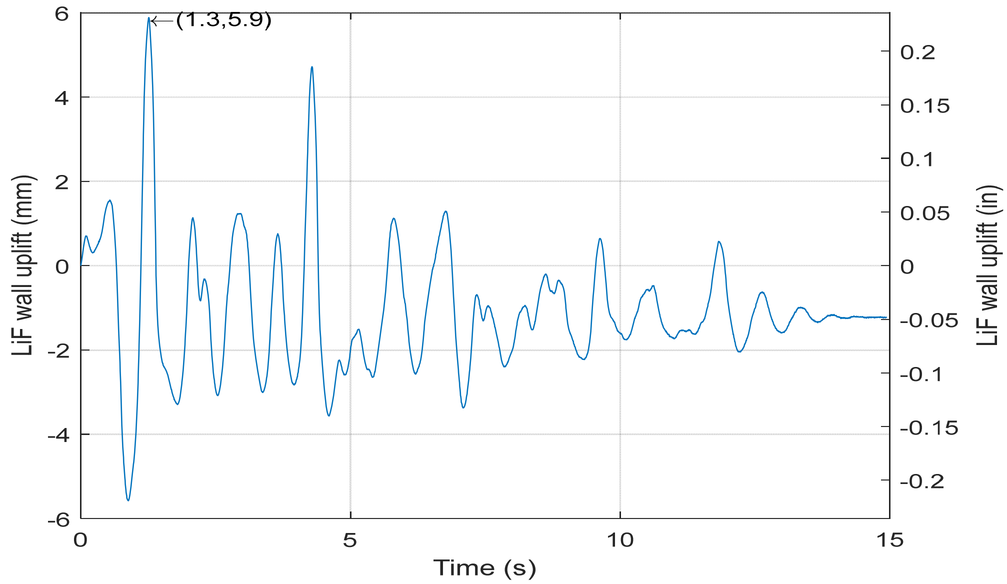
3.3.5 LiFS wall uplift and sliding

The LiFS walls used Simpson's HDU5-SDS2.5 hold downs designed to resist overturning moment and uplift. The walls were monitored for any significant uplift experienced during the tests. The uplift was measured at one corner of the South LiFS walls and time-history plots for the LiFS wall uplift for Test 3 and Test 4a are shown in Figure 3.9. Based on the recorded measurements, it can be seen that there was no significant uplift as expected, i.e. less than a half of an inch.

LiFS walls may also slide between the LiFS walls and the base CLT and therefore instrumentation between the bottom CLT and LiFS walls was included. This was also monitored between the top of the LiFS and the upper CLT roof system. Due to considerable amount of sliding observed between the LiFS wall and the top CLT after Test 1b, additional shear key connectors were added. The addition of these shear keys was found to perform well and it was observed that no significant sliding was recorded in the following tests. Time-history plots for sliding between the base CLT and South LiFS wall are shown in Figure 3.10 which confirms that there was no significant sliding.

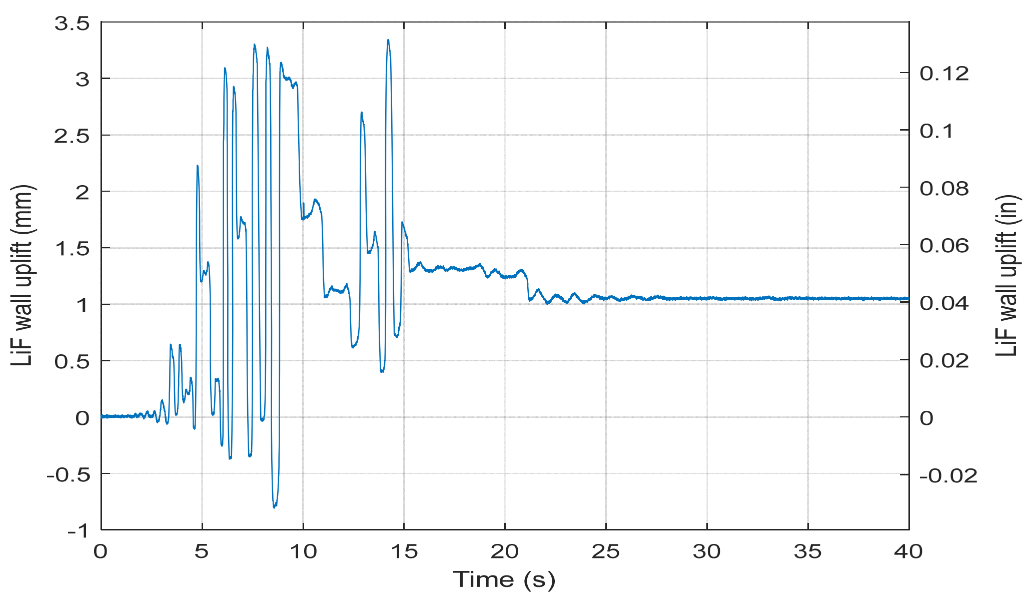


(a)

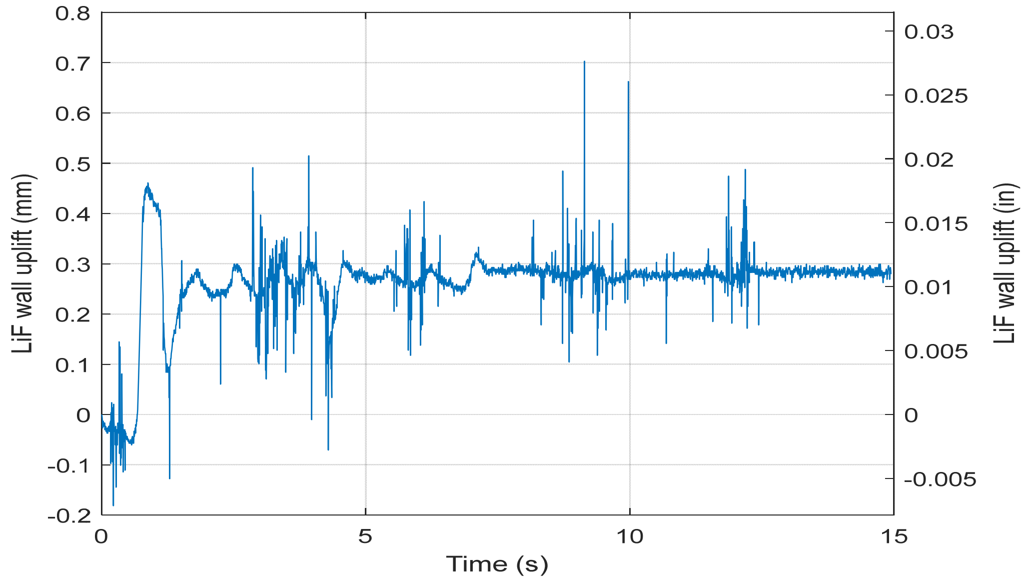


(b)

Figure 3.9: LiFS wall uplift time-history: (a) Test 3; (b): Test 4a



(a)



(b)

Figure 3.10: LiFS wall sliding time-history: (a) Test 3; (b): Test 4a

CHAPTER 4: SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

A full-scale shake table test of a hybrid shear wall system made up of a post-tensioned rocking CLT wall and LiFS walls was presented in this thesis. Two different ground motions were used during the test program, namely the 1989 Loma Prieta ground motion record from the Capitola station scaled to intensities corresponding to SLE, DBE and MCE levels and 1994 Northridge ground motion record from the Rinaldi station. The hybrid shear wall system was able to completely re-center following an MCE intensity shake with some moderate damage to the sheathing of the LiFS walls.

The test observations showed that the hybrid shear wall system was able to re-center as a result of the post-tensioned CLT rocking wall and the LiFS sustained moderate damage but would be repairable after an MCE intensity shake. The LiFS walls on either side of the post-tensioned rocking CLT wall acted as the energy dissipaters with withdrawal of sheathing nails. The post-tensioned rocking CLT wall re-centered the structure completely as described above. In this test, mild steel rods were used as post-tensioning rods. It was observed that the rods experienced tensile stress greater than the theoretical yield stress of the mild steel rods during the MCE level ground motion and would need to be replaced after such an event. The rod replacement can be avoided by using a higher strength steel for the post-tension rod material. However, there might be loss in post-tension force after a large earthquake for which minor repair work may need to be carried out. Also , although the LiFS walls were replaced after every MCE intensity shake for test purposes, it was also observed that the walls were not completely damaged and some repair work may be sufficient, i.e. just replacement of the sheathing but not the studs.

In order to understand the collapse mechanism of a post-tensioned rocking CLT wall system, three Northridge Rinaldi station ground motions were used without any repair between tests. Although the complete test results were not obtained due to failure of the instrumentation, test observations were useful to understand the failure of the rocking CLT wall system. The PT bar fractured which further resulted in the inability of the structure to re-center thus resulting in collapse of the system. Finally, the test results from the program provided valuable data for development and refinement of numerical models for designing structures with these hybrid shear wall system.

The development of rocking CLT walls in seismic resistant structures is relatively new and the idea of a hybrid shear wall system incorporating stud walls and rocking CLT walls can prove to be advantageous and thus there are several recommendations that can be considered for future studies:

1.) Different aspect ratio CLT wall panels should be considered as the aspect ratio can affect the performance of the shear wall system, i.e. rocking versus sliding behavior.
2.) The shear transfer mechanism for the rocking CLT wall in this test was effective. However, this can likely be further improved to allow for greater rocking of the CLT walls.
3.) Mild steel bars were used in this test for post-tensioning bars as this is readily available. Though mild steel bars can be efficient in low-rise buildings, high strength steel rods may be more efficient for mid- and high-rise buildings which should be further explored.
4.) Finally, since the complete test results were not available for study of the collapse mechanism, due to failure of the string potentiometer, this could be further explored in future studies to allow numerical model calibration.

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