EXPERIMENTAL SEISMIC BEHAVIOUR OF A TWO-STOREY CLT PLATFORM BUILDING: DESIGN AND SHAKE TABLE TESTING

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ABSTRACT

Since its introduction in Europe more than two decades ago, Cross Laminated Timber (CLT) is viewed as a new-generation of engineered wood products and has found its way into the US construction market. Recent research efforts have demonstrated that CLT can be effectively utilized as a seismic force resisting system. This paper presents the results of part of a study conducted at Colorado State University to systematically investigate seismic behaviour of CLT shear wall systems in regions of high seismicity for use in the United States. Specifically, the results of a full-scale shake table test of a two-storey 223 m² building. The CLT special shear walls in the building were designed based on a design methodology that resulted from connector and reverse cyclic testing of isolated CLT shear walls using a generic connector. The main design assumption for these walls is that all overturning was resisted by the overturning anchor (tie-down rod or hold-downs) at the wall ends and the shear is resisted by the generic angle brackets connected with nails. The shake table tests were performed in three phases with each phase consisting of a different CLT wall configuration. Phase 1 and 2 were multi-panel configurations with 4:1 (h/b) and 2:1 aspect ratio panels, respectively. Phase 3 was another 4:1 aspect ratio wall with return walls added in the transverse direction to examine their effect on the overall response. The structure was subjected to several ground motions including one scaled to the maximum credible earthquake (MCE), equivalent to a 2475 year return period.

Keywords: Cross Laminated Timber; Seismic Force Resisting System; FEMA P-695 Methodology; Shake Table Testing

1. INTRODUCTION

Cross Laminated Timber (CLT) has been available as an engineered wood product in Europe for many years but was only recently introduced into the US construction market. The use of CLT increased after its introduction, but many applications were in regions of low to moderate seismicity. Interest in using it as a seismic force resisting system (SFRS) increased with much of the research from Europe. One of the first major experimental studies was conducted by Dujic et al. (2004) where they demonstrated the
capability of CLT as a SFRS, and concluded that the capacity of the CLT shear walls is controlled by local failures in the wood material and by the connector/anchorage strength. The SOFIE project, a landmark comprehensive CLT study funded by the Trento Province in Italy, consisted of a multitude of tests including connection tests, CLT shear wall tests, and several full-scale building systems including its culmination with a 7-storey shake table test at Japan’s E-Defense Cecotti et al. (2013). The results, summarized in Gavric et al. (2012) provided the information essential for designing CLT connections that remained undamaged under seismic loading. Research in CLT as a SFRS is not limited to Europe, with a more recent body of work developing in Japan, e.g. Miura et al. (2016). Another major Japanese contribution is Okabe et al. (2012), in which the authors concluded that the predominant behaviour in CLT shear walls during seismic loading was rocking, and that the inter-panel connections would fail in shear. Another study conducted by Popovski et al. (2015) investigated the performance of CLT wall assemblies using quasi-static monotonic and cyclic testing. The results of the tests showed that most inter-storey drift is caused by the rocking and sliding of CLT panels, and that CLT diaphragms can act as rigid bodies. The testing also emphasized the need for further testing using different aspect ratios for CLT wall panels.

Interest has been increasing worldwide in developing application of CLT as a SFRS, but in the United States there are no provisions allowing for the use of CLT in seismic regions, forcing any design applications to utilize the alternative means and methods approach within governing design standards. A multi-year effort is in its final stages in the U.S. to obtain the experimental data necessary to formulate a seismic design approach for the use of CLT in high seismic areas without the need to utilize alternative methods within the American Society of Civil Engineers Standard 7 (ASCE 7). The results presented in this paper are the results of a full-scale shake table test program of a two-storey 223 m² platform type building, which incorporated two parallel CLT shear wall stacks.

2. STRUCTURE INFORMATION AND CONSTRUCTION

2.1 Structure and Testing Description

The testing was conducted at the NHERI@UCSD Shake Table from June until September 2017 included three distinct phases, each a collaboration of institutions and universities, investigating different Seismic Force Resisting Systems (SFRS), but all from CLT. Specifically, Phase I and Phase II investigated resilient SFRS, while Phase III focused on the SFRS designed using FEMA P-695 methodology. This paper focuses exclusively on Phase III of testing, which included three sub-phases, and the processes by which they were conducted.

The test structure is a combination of three distinct components, the gravity frame, the diaphragm, and the shear walls. Throughout the testing the gravity frame and diaphragm remained constant, while the shear walls, being the main variables in the testing, had a different design for each phase with the goal of answering different research questions for each sub-phase. Figure 1 shows elevation views of the structure, and the location of the shear walls within the structure can be seen in Figure 1. Figure 2 shows the floor plans for the first and second floor of the structure.
2.2 Gravity Frame and Diaphragm

The gravity frame component of the test specimen consisted of glulam beams and columns, with Simpson Strong Tie connectors. The completed first floor of the gravity frame can be seen in Figure 3 (b). The columns in the gravity frame were a mixture of continuous and non-continuous columns, all of which were grade L2 glulam, with the continuous columns located at the same alignment as the cavities for the shear walls. Two different cantilever glulam beam system designs were used in the test specimen, one for each floor. The main difference between the systems occurs in the lateral bracing system for the exterior, longitudinal beams. The first-floor system had lateral beams located at either end of the specimen, the centre, and around the wall cavities. In contrast, the second floor did not have any lateral beams along its length, and the longitudinal beam size was smaller between the wall cavities. The CLT panels making up the diaphragm varied in orientation and thickness with the first-floor diaphragm consisting of two sizes of CLT panels. On either end of the structure, four 1524 mm x 6096 mm x 105 mm CLT panels were oriented with the longer side parallel with the exterior glulam beams. The eight interior CLT panels were oriented similarly to the exterior panels, but were smaller with dimensions of 1524 mm x 2743 mm x 104.78 mm. The panels and their orientations can be seen in Figure 2 (a). The second-floor diaphragm also used 1524 mm x 6096 mm x 175 mm CLT panels oriented with the longer side perpendicular to the exterior glulam beams. The second-floor diaphragm also included a 57 mm thick concrete topping with #3 rebar mesh. The orientations of these panels can be seen in Figure 2 (b).

2.3 Shear Force Resisting System

It should be noted that a typical wall configuration would have a more conventional platform style placement of the walls with multiple walls throughout the entire floor, but due to the limitations of the testing and structure, two stacked parallel systems were tested. This configuration can be thought of as a worst-case scenario where two wall stacks alone resist the entire inertial forces generated by the earthquake without typical load sharing observed in these types of systems. The CLT panel wall system in each of phases was designed using the Equivalent Lateral Force (ELF) method in accordance with ASCE 7 (2010). The design spectrum values used were obtained from a San Francisco, California location with 1.5g and 1.0g \( S_{05} \) and \( S_{01} \) respectively. The design was done under the assumption that the tie-down rods located on either end of the wall system resist overturning moment, and the
connections provided by the generic steel angle brackets and nails solely resist shear (Amini et al, 2016). Inter-panel connectors consisting of nails and steel plates were added at the vertical connection between CLT panels. Similar to light frame wood construction, the tie-down rods provide for overturning restraint with bearing area provided at each floor. The compression side of the wall is governed by allowable compressive stresses perpendicular to grain, often governing the CLT wall thickness selection in the design method.

As mentioned previously, the test structure was utilized in three different phases of testing, with this paper focusing on Phase III. Phase III consisted of three different sub-phases, which in this paper will be referred to as Phase One (1), Two (2), and Three (3) respectively, each with varying panel and wall configurations. Phase 1 wall configuration was comprised of four 910 mm CLT wall panels forming a 3650 mm long wall representing a 4:1 aspect ratio. Phase 2 was a 3280 mm wall consisting of two 2640 mm panels that represented a 2:1 aspect ratio. The purpose of testing different aspect ratios was to investigate the effect that different aspect ratios had on the behaviour of the CLT walls stacks within the full-scale shake table test. Phase 3 was the same as the Phase 1 configuration but with transverse walls added in an attempt to identify any adverse effects on the performance of the wall system or angle bracket connections with the addition of the transverse wall. In this paper the results focus on the Phase 1 configuration since this was felt to be the most typical configuration striking a balance between strength and deformation capacity.

2.4 Construction

The test specimen is a composite of three distinct components, the gravity frame, the diaphragm, and the shear walls, the construction of which occurred in different phases taking place over the course of several weeks. The entire building was constructed on several steel beams post-tensioned to the table that served as the transfer connection between the test building and the shake table surface. These beams, which also allowed the specimen to be wider that the short direction of the shake table as cantilevers, can be seen in Figure 3(a).

The construction of the gravity frame and diaphragm occurred as platform style construction, with the gravity frame being erected first, followed by the installation of the diaphragm. The gravity system for the first-floor consisted of glulam columns and beams connected using a variety of Simpson Strong Tie connectors as seen in Figure 3(b). After the gravity frame and the diaphragm for the first floor were installed, seismic mass in the form of steel trench plates, were fastened to the diaphragm to replicate the design weights for non-structural and other components not included in the test specimen during the test. After the seismic mass was placed, the second-floor gravity frame and diaphragm were installed using a similar approach to the first-floor, which is shown in Figures 3(c) and 3(d). The second-floor diaphragm was intended to be a composite of CLT and concrete, so after the CLT and gravity frame erection was complete, angled anchors and rebar were installed as seen in Figure 3(e). The angled anchors provide the shear transfer mechanism between the concrete and CLT. A topping of concrete was poured, then additional seismic mass was added to the second-floor as seen in Figure 3(f). For diaphragm details and the comparison between the wood-only and the composite diaphragm the interested reader is referred to Barbosa et al. (2018).

This test specimen was part of a large collaboration of universities and institutions, and it was used for other tests prior to the test program presented in this paper, affecting the available methods for installing the shear walls. Ideally, they would have been installed as each floor was completed with gravity load included, but this was not possible since the gravity system was in place already, so instead they were cut to size and installed after the gravity frame and diaphragms. They were wedged into place and likely had substantial (but not all) gravity load on the CLT shear walls during testing. The installation approach is shown in the photos of Figures 3(g) and 3(h).
Figure 3. Construction sequence of two-storey test structure and Phase 3 wall installation: (a) Steel beam cantilever; (b) First floor gravity frame; (c) Installation of seismic mass; (d) Completed gravity frame and diaphragm; (e) Installation of rebar; (f) Finished composite diaphragm; (g) Installation of ground floor shear walls; (h) Installation of first floor shear walls (Photo credit: NHERI TallWood Team).
3. INSTRUMENTATION AND GROUND MOTIONS

3.1 Test Motions

For the testing of Phase III the 1989 Loma Prieta earthquake record was selected with three varying intensities. Scaling was done using a scaling methodology consistent with FEMA P695 (2009) for San Francisco, California, with a 1.5 g and 1.0 g $S_{DS}$ and $S_{D1}$, respectively for the largest motion. The selected intensities were intended to correspond to a frequent earthquake, design earthquake (DBE), and maximum considered earthquake (MCE), each with a mean return period of 72 years, 475 years, and 2475 years, respectively. In this paper, the results focus on the DBE and the MCE because of their controlling effect on the design of the structure. During the DBE, the structure experienced an acceleration of 0.92 g, while at MCE level, a larger acceleration of 1.36 g was experienced. The response spectrum from the two aforementioned intensities can be seen in Figure 4.

![Response Spectrum for Loma Prieta ground motions.](image)

3.2 Gravity Frame and Diaphragm Instrumentation

As mentioned previously, the gravity frame and diaphragm remained constant throughout testing, this included the instrumentation on both those components of the test building. Four types of instrumentation were placed on the structure, namely strain gauges, linear potentiometers, string potentiometers, and accelerometers. The quantities of each on the combined gravity frame and diaphragm system can be seen in Table 1. Strain gauges were used on metal straps placed across the chord splices on the diaphragm on both the first and second floor to measure the differential movement of the CLT diaphragm panels. Linear potentiometers were used to measure the slip between individual CLT panels in the diaphragm, and to measure the slip between the CLT panels and the glulam beams in the gravity frame. String potentiometers were used to measure the global displacement of the entire structure, with the steel towers around the table as reference points and string potentiometers were also used to measure the relative vertical displacement between each of the floors. Accelerometers were placed at several locations across each floor to measure the acceleration of each floor, and an accelerometer was also placed on the shake table itself to provide a baseline for integration of accelerations to get velocity and displacement from the accelerometers.
<table>
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<tr>
<td>Accelerometer</td>
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### 3.3 Wall Instrumentation

Four types of instruments, string pots, potentiometers, load cells, and strain gauges, were utilized to capture the behaviour of the CLT shear wall stacks, the quantities of which can be found in Table 2. The load cells captured the tension force that was induced in the tie-down rods by the overturning moment. There were a total of 16 locations measured by the load cells, one for each base and roof level of the tie down rods, with strain gauges also placed on the rods themselves to confirm the load cell readings. Potentiometers were placed throughout the structure, a total of 20, to measure the relative uplift and sliding of the wall system. String pots were placed on several CLT wall panels to capture any potential panel deformation caused either by sliding or rocking of the panel. Figure 5 illustrates the placement of the instrumentation on the southern face of the south wall in the structure. All of the wall faces had similar instrument placement, but the wall shown in Figure 5 was slightly more heavily instrumented, and it is due to this extra instrumentation that the results for this wall are presented.

![Figure 5. Instrumentation on the south face of the southern shear wall](image-url)
Table 2. Instrumentation on shear walls.

<table>
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<td>String Potentiometer</td>
<td>20</td>
</tr>
<tr>
<td>Accelerometer</td>
<td>16</td>
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</table>

4. RESULTS

4.1 Phase 1 Roof Displacements

Recall that only one ground motion, the 1989 Loma Prieta earthquake, was used throughout the test program presented in this paper. Figures 5 and 6 present the average displacement for three string pots located on the roof of the north end, south end, and center of the structure for the DBE and MCE level tests, respectively. The objective of this test program was to apply a design method developed for design of CLT platform buildings in the U.S. using typical force based design methods, i.e. equivalent lateral force procedure. This means that the design is performed using 2/3 of the MCE level spectral acceleration and then reduced by a seismic performance factor, or R-factor, which was 4 in this case to account for the deformation capacity and ability of the structure to respond nonlinearly and dissipate energy. The MCE scaling is approximately 1.5 times that of the DBE, but as one can see from Figures 5 and 6, the peak response to the MCE shake is more than 1.5 times that of the DBE indicating a nonlinear response, which has been confirmed numerically. It can also be observed that the peak displacement occurs at roughly the same time during each motion, which would be expected for a typical seismic response.

Figure 6. Average roof displacement of structure during DBE level motion
Figure 7. Average roof displacement of structure during MCE level motion

4.2 Phase 1 Wall Uplift and Sliding

A major point of interest in the testing was the behaviour of the CLT panel components making up the SFRS, specifically whether the panel was governed by uplift, sliding, or a combination thereof. Figure 8 shows the uplift at each of the potentiometer locations, and shows the response at the bottom corner of the first-floor wall. It can be seen from the figure that all of the peak displacements occurred at relatively the same time during the ground motion. The bottom of each CLT panel, with the first floor being the worst, experienced the most severe rocking behaviour, while the top of the CLT panels experienced less rocking, as expected. The sliding of the panels, which can be seen in Figure 8, follows a similar pattern to the uplift with the largest slipping occurring at the bottom of the panel on the first floor. Some of that sliding occurs as the result of partial nail withdrawal from the CLT as the angle brackets at the base work the nails out. Comparing the responses from the uplift and sliding, it can clearly be seen that the rocking behaviour controls the panel at each of the locations measured. Although not the focus of this paper, sliding was observed for lower aspect ratio panels and those results will also be presented at the conference.

Figure 8. Displacements caused by uplift during MCE motion
4.3 Forces in the Rods

There was a total of eight tie-down rods in the SFRS, with one rod on either end of the wall and on both faces. In the design of the SRFS it was assumed that the CLT wall panels worked as a continuous wall with vertical shear connectors between CLT panels making up an individual wall, and tie-down rods were only required on the ends of the walls. The tie-down rods are only able to take tension since they would buckle if put into compression, so they are allowed to slip through at the bottom of the structure and the CLT end-panel within the wall takes the compression loading. The width of the CLT panels are governed by crushing with allowable stress perpendicular to grain computed to ensure that the compression zone is located in less than half of the end-panel making up the wall. Figure 9 presents the average tension force across both faces on the CLT panels induced from the Loma Prieta MCE level ground motion at both the base and roof level. The loading in the tie-down rods was not homogenous across the structure. It is clear that the eastern side of the structure received more load than the western side with the largest tension force of 167 kN occurring at the eastern base of the north wall.
4.4 Torsion in the Structure

The displacements of the roof of the structure were recorded in three different locations, the north end, the south end, and the centre of the structure. A significant concern during the test was the possible torsion that could be induced into the structure once it had undergone more than 30 tests during all major project phases, including some testing that exceeded MCE. Figure 10 presents the response of each of the roof string potentiometers at the aforementioned locations during the Loma Prieta MCE level ground motion. The structure clearly experienced non-uniform displacement with the south end displacing a full 34 mm more than the centre of the structure, introducing torsion into the system. The northern end of the structure experienced significantly less torsion with only a 3 mm difference in displacement. Full details of the diaphragm design and testing are available in Barbosa et al. (2018). The CLT shear walls stacks performed well even with some level of torsion present in the building.

Figure 11. Torsion caused by uplift during MCE motion: (a) North end of structure; (b) Centre of structure; (c) South end of structure.

5. CONCLUSIONS

In all phases and ground motion intensities, the structure performed well and in accordance with code requirements essentially providing life safety. The 4:1 aspect ratio tested in Phase 1 clearly demonstrated rocking behaviour as demonstrated by other researchers, with the tie-down rods performing well with no yielding. The inter-panel connectors and steel angle brackets also performed as expected, with some nail pull-out observed, i.e. approximately 5 to 7 mm. At DBE intensity level there was no observable damage to the connections in the two-storey shear wall stacks. However, following MCE intensity testing, nail withdrawal for the base shear connectors of between 2 and 7 mm was observed for both high aspect ratio tests. This is felt to be repairable, but the shear connectors would need to be shifted/moved to allow penetration of new wood. The nails on inter-panel connectors for those tests had less withdrawal, but would also need repair. For the lowered aspect ratio test (2:1) in the second phase of the program, the nails in the connectors at the base sheared, allowing sliding to take over the response from rocking, but there was never any risk of collapse. Recall the objective of current U.S. building standards is to provide life safety protection to occupants with little or no focus on repairability. The FEMA P695 methodology and resulting design method for platform CLT
construction used to design the walls in these tests aligns with this current philosophy. The testing conducted at the NHERI@UCSD shake table provided valuable experimental data on the behaviour of a SFRS designed using the FEMA P-695 methodology, and will be invaluable for refining the methodology moving forward. Life safety performance at MCE level was achieved during all phases of the P695 testing program.

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7. REFERENCES


