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Experimental seismic behavior of a two-story CLT platform building

John W. van de Lindt[a,*], Jace Furley[b], M. Omar Amini[c], Shiling Pei[d], Gabriele Tamagnone[e], Andre R. Barbosa[f], Doug Rammer[g], Philip Line[h], Massimo Fragiacomo[i], Marjan Popovski[j]

[a] Department of Civil and Environmental Engineering, Colorado State University, Fort Collins, CO 80523-1372, USA
[b] Colorado State University, USA
[c] Department of Civil and Environmental Engineering, Colorado State University, Fort Collins, CO 80523, USA
[d] Department of Civil and Environmental Engineering, Colorado School of Mines, Golden, CO 80401, USA
[e] University of Trieste, Italy
[f] School of Civil and Construction Engineering, 101 Kearney Hall, Oregon State Univ., Corvallis, OR 97331, USA
[g] Forest Products Laboratory, One Gifford Pinchot Drive, Madison, WI 53726, USA
[h] Structural Engineering, American Wood Council, 222 Catoctin Circle SE, Suite 201, Leesburg, VA 20175, USA
[i] University of L'Aquila, Italy
[j] Building Systems, FPInnovations, 2665 East Mall, Vancouver, BC V6T 1Z4, Canada

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ABSTRACT

Cross-laminated timber (CLT) manufacturing and construction has been steadily growing since its inception in Europe in the 1990s. In the US, the growth of the CLT adoption is inhibited by the lack of codified design provisions for CLT in high seismic regions. This led to a multi-year study conducted by Colorado State University to investigate suitable seismic design parameters of CLT shear wall systems. This paper presents the results from a series of shake-table tests featuring a full-scale two-story mass-timber building utilizing CLT Seismic Force Resisting Systems (SFRS). The building was designed using an R- factor equal to 4.0 under the equivalent lateral force procedure specifications of the ASCE 7-16 Standard. The test program included three phases with different wall configurations, reflecting different wall panel aspect ratios and the existence of transverse CLT walls. Test results indicate that the code-level life safety objective was achieved in all test configurations. The addition of transverse walls did not affect the ability of the panels to rock, and improved the performance of the building structural system.

1. Introduction

In the United States of America (US), Cross-laminated Timber (CLT) has only been introduced into the construction market recently, but it has been used in Europe since the 1990s. The use of CLT in the US has largely been limited to regions of low seismicity primarily due to the lack of code and standards allowing the use of CLT as a seismic force resisting system (SFRS). Existing design using CLT in these regions has to either adopted a different lateral system (steel or concrete) or used the alternate design means and methods available within American Society of Civil of Engineers Standard 7 (ASCE 7). The additional cost associated with the alternate design pathway makes the CLT lateral solutions less competitive economically. As the interest in directly using CLT as a SFRS has been growing in North America, there is an immediate need for further research into CLT as a code- recognized SFRS in the US.

Research into CLT as a SFRS has been a globally notable trend for over a decade [13]. One of the first studies investigating CLT as a SFRS was performed at the University of Ljubljana, Slovenia [7]. That study involved subjecting fifteen CLT panels with various anchorages and vertical loads to reversed cyclic lateral loading. From these experiments, it was determined that the type of anchorage and the vertical load both have a significant effect on performance, and that failure was most likely to occur from connector/anchorage failure, or from localized failures in the wood material. Although these conclusions may seem obvious at present, they were quite novel at the time setting a stage for numerical modeling approaches. The SOFIE Project, an international study funded by the Trento Province in Italy, was an influential comprehensive study on the feasibility of CLT as a SFRS in mid-rise buildings. The objectives of the project were extensive and

* Corresponding author.
E-mail addresses: jace.furley@colostate.edu (J. Furley), omar.amini@rams.colostate.edu (M.O. Amini), spei@mines.edu (S. Pei), gabriele.tamagnone@phd.units.it (G. Tamagnone), Andre.Barbosa@oregonstate.edu (A.R. Barbosa), drammer@fs.fed.us (D. Rammer), pline@awc.org (P. Line), massimo.fragiacomo@univaq.it (M. Fragiacomo), marjan.popovski@fpinnovations.ca (M. Popovski).

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included investigations into material behavior, fire durability, earthquake behavior, and culminated in three-story and seven-story full-building tests in Japan. Gavric et al. [8,9] summarizes the essential information for the design of seismic connections for CLT, specifically connections that remain undamaged after seismic excitation. The experimental components of that project included CLT shear wall tests, connection testing, and several other full-scale tests cumulating in a shake table test of a full-scale seven-story CLT building at Japan's E-Defense Shake Table. A full summary of the results from this test can be found in Ceccotti et al. [6].

The SOFIE project demonstrated the capability and suitability of CLT structures for use in high seismic regions and interest soon spread to other areas of the globe. Okabe et al. [10] investigated the structural performance of CLT manufactured with Sugi softwood (Japanese cedar) under seismic and wind loading. The testing was separated into two methods, both utilizing cyclic lateral loading, but with differing boundary conditions introducing shear and rocking responses. The main results demonstrated that rocking was the main deformation mode for the CLT panels with the corners of the panel determining the strength and deformation of the shear wall and the inter-panel LVL connectors were shown to be vulnerable to splitting failures during the testing. The study also reinforced earlier findings that increased vertical loading improved the lateral performance of the shear walls. In North America, Popovski et al. [14] conducted a quasi-static test program of a two-story CLT house with varying wall configurations throughout the structure subjected to monotonic and cyclic loading. The results of the testing demonstrated that rocking and sliding motions are caused by failures in the brackets and are responsible for most of the deformation and displacement of CLT structures, reinforcing earlier work by other researchers. It was also observed that the CLT wall panels exhibit rigid body motion (such as rocking) and that the floors acted as rigid diaphragms. The study also stressed the need for further research into the effects of different panel aspect ratios on the performance of CLT walls. In the U.S., the FEMA P-695 (FEMA, 2008) methodology is being applied by a research team led by Colorado State University (CSU) to enable the use of CLT in high seismic areas without the need of alternative means and methods in ASCE 7 [2].

In this paper, the results of a two-story shake-table test program is presented, which sought to (1) demonstrate the effectiveness of CLT shear-walls as a SFRS to provide life-safety; (2) provide insight into the aspect ratios where CLT panels might transition from rocking to sliding behavior for platform style construction; and (3) suggest design provisions of shear walls using the equivalent lateral force procedure.

2. Materials and methods

2.1. Test building layout and configuration

The two-story test building constructed in this study utilized CLT panel shear walls in a platform construction configuration. The CLT shear walls were connected by generic shear connectors at the base and ceiling to a CLT floor/roof diaphragm. The overturning was resisted by continuous hold down rods typically used in stacked shear walls in light-frame wood construction. The gravity frame of the test building was designed to facilitate the needs of two previous stages of testing [5,12] that investigated CLT rocking wall systems. This paper focuses exclusively on the third stage of testing and its three sub-phases: Phase 3.1, Phase 3.2, and Phase 3.3. Phase 3.1 and Phase 3.2 investigate the effects of different CLT panel aspect ratios on the performance of the stacked shear walls, while Phase 3.3 investigates the effects of adding transverse walls to either end of each shear wall stack. The test structure was a two-story mass-timber building with glulam and CLT panel components, as can be seen in Fig. 1, which was constructed over several days by professional contractors experienced in CLT construction.. The dimension of the building is shown in the figure with a height, width, and length of 6.7 m (22 ft.), 6.1 m (20 ft.), and 17.7 m (58 ft.), respectively. Structurally, the building can be divided in to three sub- systems, namely the gravity frame, the floor/roof diaphragms, and the CLT shear wall seismic force resisting system (SRFS).

Fig. 1 (a) shows the gravity frame, which consists of the glulam beams and columns that support the dead load (gravity) loads of the structure, which can be split into the first and second story. The column system on the first story consists of 12 columns, all grade L2, with four (4) columns being continuous to the second story and roof level. The first floor level consists of four grade 24F-V8 longitudinal beams spanning the N-S direction, nine (9) grade 24F-V4 transverse beams spanning the E-W direction, and with beams and columns connected using commercial connectors [15]. The second story of the structure has a similar column layout to the first story with 12 total columns, also grade L2, with the four (4) continuous columns terminating at the roof level, and eight (8) discontinuous columns located above their respective first story counterparts. The roof level beam system was significantly different than the first floor level since it had only beams in the longitudinal directions. There were a total of six (6) beams, four (4) grade 24F-V8, and two (2) grade 24F-V4 which are also connected using similar hardware to the first floor level.

Similar to the gravity frame, the diaphragms for the first floor level and the roof floor level were different with the objective of investigating two design configurations in one test [4]. The floor diaphragm (first floor) level panels were all 3-ply grade V1 CLT panels, although the size of the panels varied. The roof diaphragm was a composite system consisting of CLT panels with a concrete topping slab. The CLT panels were 1.5 m × 6 m (5 ft × 20 ft), with the exception of two 1.2 m × 6 m (4 ft × 20 ft) panels, 5-ply grade V1 panels spanning the E-W direction, as seen in Fig. 1 (a). The concrete layer was a 57.2 mm (2.25 in.) concrete topping on the CLT panels with composite action being achieved with 45-degree anchors installed into the CLT panels.

The SFRS system used for the testing varied for each phase, but were all consisted of CLT panels connected with metal hardware to form a wall at each story. There are two parallel shear wall lines as it is shown in Fig. 2. The two-story stacked wall system was designed against overturning by two continuous steel tie-down rods while the shear demand was designed to be resisted by inter-panel connectors and generic angle brackets (both fastened using 16D sinker nails) [1]. The assumption of decoupled shear and overturning force transfer using different components is a common design assumption that leads to conservative designs, and is the main assumption consistent with the design approach for platform CLT construction standards developed in the US. The design method being proposed for CLT platform construction in the U.S. assumes the brackets only take shear forces and the continuous steel rod hold downs take only uplift, both of which are not exactly correct since the wall does try to rock. However, the phi factor for capacity reduction was adjusted within the design method to account for this during calibration of the proposed design procedure. The loading used in the design was derived using the Equivalent Lateral Force (ELF) method as described in ASCE 7 [3], with design spectral values of 1.5 g for SDS and 1.0 g for SD1 for a site near San Francisco, California. An R- factor of 4.0 was used for the design of the lateral resisting system and the diaphragm was assumed to be rigid. The number and spacing of the shear connectors for each phase were such to maintain a constant design shear story capacity of 174 kN (39 kips) and 120 kN (27 kips) per wall line for the first and second stories respectively. The linear per foot design capacity varied between the phases however, and for Phase 3.1 and Phase 3.3, the shear connectors had a capacity of 4.7 kN/m (3.4 kips/ft) and 3.4 kN/m (2.5 kips/ft) for the first and second stories respectively, while in Phase 3.2 the connectors had a capacity of 5.4 kN/m (4 kips/ft) and 4.2 kN/m (3 kips/ft) for the first and second stories. The steel tie-down rods resisting the overturning moment in Phase 3.1 and Phase 3.3 had a design capacity of 124.5 kN (28 kips) and 320 kN (72 kips) per wall for the first and second stories respectively, while the tie-down rods in Phase 3.2 had a

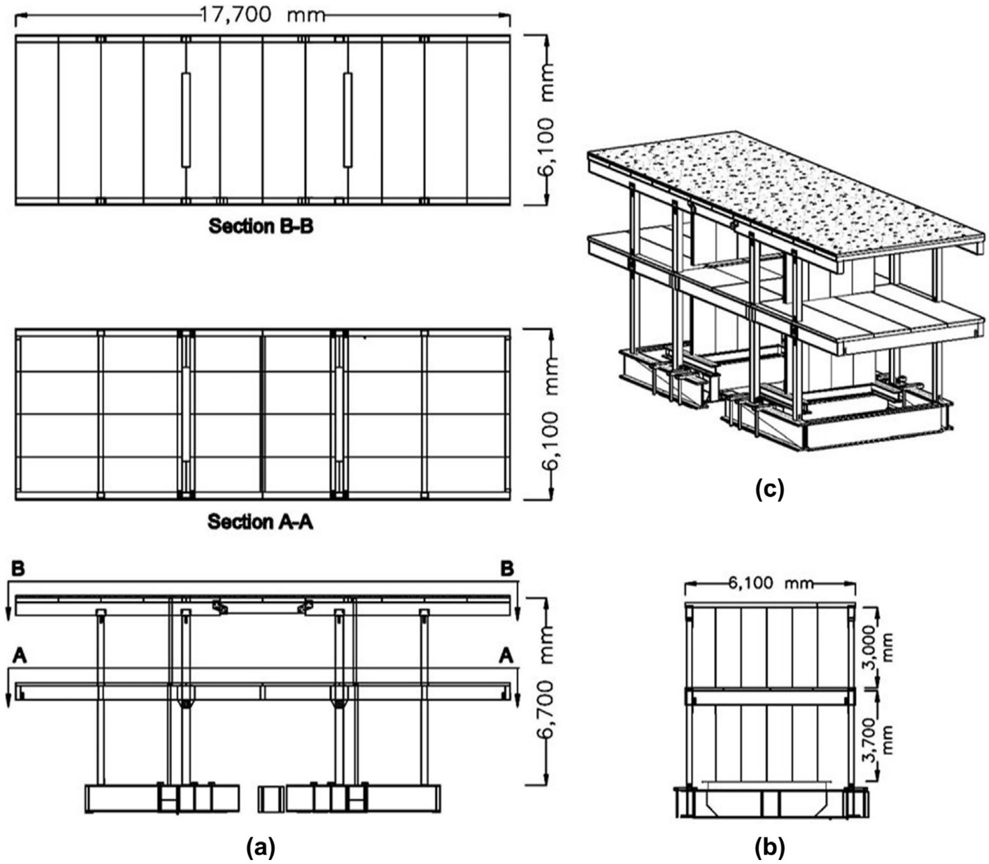

Fig. 1. Test Building: (a) Front elevation and plan view; (b) Side elevation view; (c) Isometric view.

design capacity of 138 kN (31 kips) and 351 kN (79 kips) for the first and second stories.

The NHERI@UCSD Shake-Table is a 7.6 m × 12.2 m (25 ft × 40 ft) uniaxial table equipped with two actuators with a total maximum payload of 20,000 kN (4496 kips). This location has been used for many previous tests, and the full details of the shake-table can be found in Ozcelik et al. [11]. The footprint of the test structure was larger than the table in the N-S, which is the direction perpendicular to the actuator movements that apply shaking in the E-W direction, so large steel outrigging beams were used as a way to extend the table to the appropriate dimensions. To ensure safety, two steel towers were installed in front of the control building to the south of the building specimen. A third tower was installed on the east side of the structure with safety straps wrapping around the center of the structure, with the straps remaining slack unless the structure became unstable and began to collapse. Fig. 2 shows the orientation of the structure on the shake-table and the location of the safety towers.

2.2. Phase 3.1 test description

Two shear walls were designated as the north and south wall system throughout the test program. The first story wall height was 3200 mm (126 in) and second story wall height was 2.8 m (110 in). There were shear connectors installed at both the top and the bottom of the wall. The overturning restraint was provided by the ATS rods at the end of each wall similar to that used in stacked light-frame wood shear walls. The Phase 3.1 walls consisted of four (4) 900 mm × 3200 mm (36 in × 126 in) CLT panels with a thickness of 105 mm (4–1/8 in), resulting in a total wall length of 3700 mm (144 in), which is shown in the schematic of Fig. 3. The shear capacity of the walls is provided by the inter-panel flat shear connectors along the vertical splices between panels and angle shear connectors at the top and bottom of the each panel. The base/top shear connectors were generic 76 mm × 57 mm × 3 mm (3 in × 2–1/4 in × 3/25 in) L angle brackets with a length of 121 mm (4–3/4 in) with the number of connectors varying between the first and second story based on the calculated shear demands. Brackets were installed on the first and second story based on the required shear capacity. On the first story, the brackets were placed on both sides of the wall, resulting in a total of eight (8) brackets per panel and 32 per wall on the first story. The brackets on the second story walls were only located on the outside face of each wall and were spaced to ensure three brackets per panel on both the base and top, resulting in a total of six (6) brackets per panel and 24 per wall. The brackets were secured to each panel using 16 16D box nails (4.2 mm diameter × 89 mm). The brackets were connected to the diaphragm and the foundation by two 19 mm (3/4 in) diameter fully threaded, A36 bolts. The inter-panel shear resistance consists of inter-panel connectors vertically spaced at 406 mm (16 in). These inter-panel connectors are placed on both sides of each of the three inter-panel joints in the walls, resulting in a total of 14 connectors per vertical joint on the first story (total of 42) and 10 per vertical joint on the second story (total of 30). The overturning moment resisting system was comprised of vertical steel rods spanning the full height of the building. The tie down rods were located at the end of each wall on both sides, for a total of four (4) per wall and eight total and were allowed to reach at each floor (second flood diaphragm and roof level in this case) using a bearing plate to distribute the load. The diameter of the rods was not constant throughout the height of the building, with a coupler located above the first floor level bearing plate reducing the diameter from 31.8 mm (1–1/4 in) at the base to 19 mm (3/4 in) at the roof. The rods were not designed for compression loading, only tension, so to prevent buckling the tie-down rods were allowed to slip through the oversized holes at the diaphragm when the corresponding side of the shear wall was in compression. The SFRS configuration for Phase 3.1 is presented

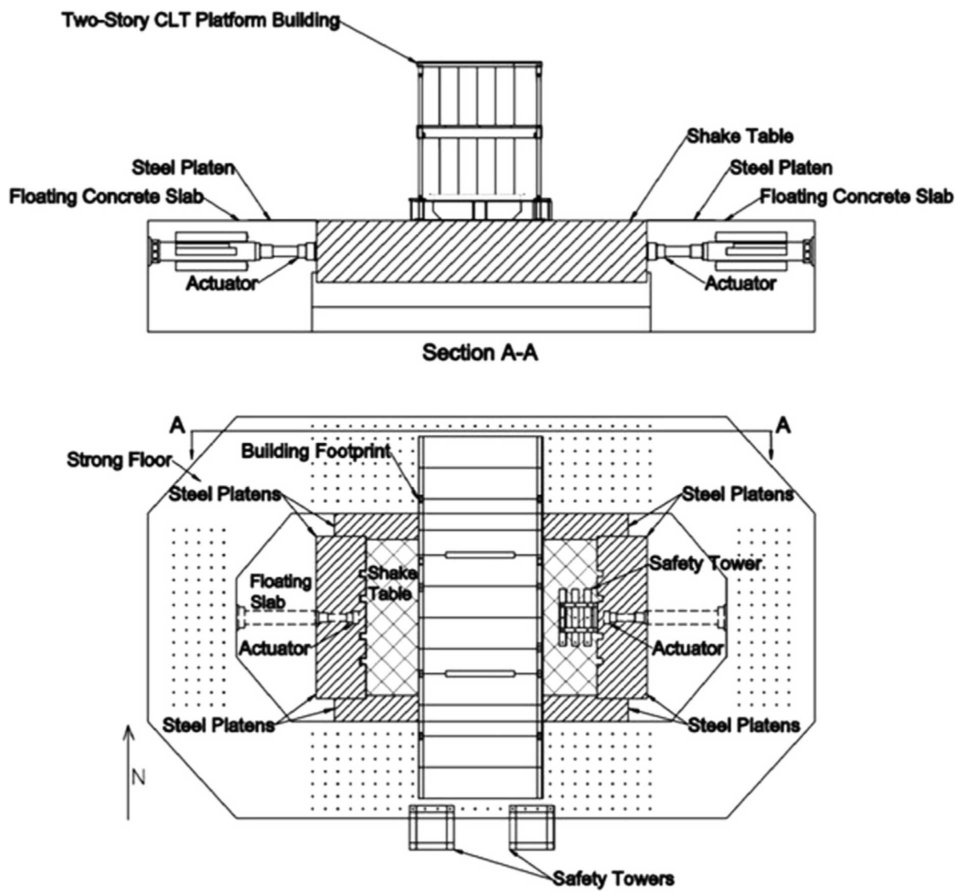

Fig. 2. Position of the two-story CLT Platform Building on the shake table.

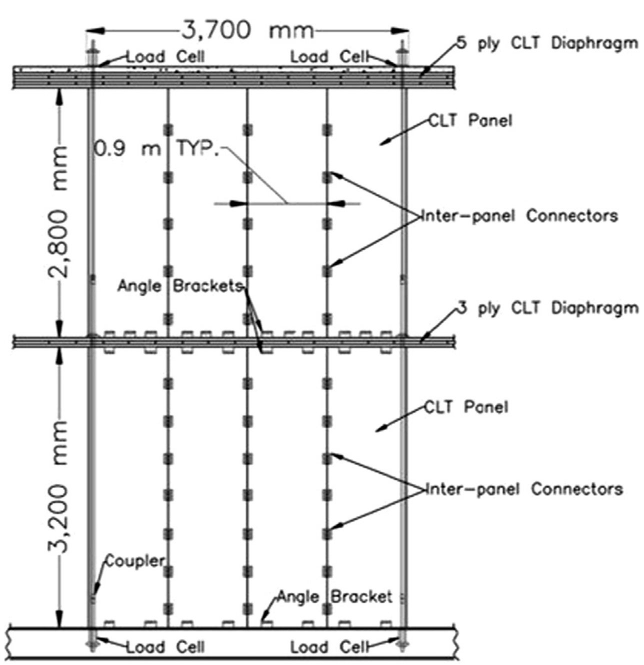

Fig. 3. Phase 3.1: 3.5:1 aspect ratio panels shear wall stack layout.

in the schematic of Fig. 3.

2.3. Phase 3.2 test description

The Phase 3.2 wall system was similar in composition to the Phase 3.1 wall system but with CLT panels having a different aspect ratio (height/width). The wall configuration for Phase 3.2 consisted of two 1520 mm × 3000 mm (60 in × 126 in) CLT panels with a thickness of 105 mm (4–1/8 in), resulting in a total wall width of 3 m (120 in) and an aspect ratio of 2.1:1, which can be seen in Fig. 4. The first story had 16 angle brackets per panel with four located on the base and top of the panel on both sides. The second consisted of 12 brackets per panel with three brackets on the base and top of the panel as well as on both sides. The first story had eight (8) inter-panel connectors on each side of the joint between panels (total of 16) and six (6) per joint on the second story (total of 12). The ATS rods for overturning were increased to a 22.2 mm (7/8 in) diameter tie-down rod instead of a 19 mm (3/4 in) tie-down rod due to larger expected overturning moment, i.e. a function of aspect ratio of the panels making up the wall system for Phase 3.2 as shown in Fig. 4.

2.4. Phase 3.3 test description

The Phase 3 wall system was identical to the Phase 3.1 wall system except for the addition of transverse CLT wall panels at the ends as shown in Fig. 5 (a). The transverse panels were added to examine the effect of these transverse walls on seismic performance. The first story transverse walls consisted of one CLT panel at each end of the shear wall. In order to avoid existing gravity frame beams, a 533 mm × 483 mm (21 in × 19 in) section was notched out from the top corner of each panel to fit it around the beams. The walls were secured to the diaphragm on the top and the steel beam at the base by six angle brackets with three for the top and base respectively. The second story transverse walls were similar to the first floor with CLT panels on both ends of the shear walls. These panels were secured to the top and base diaphragms by three angle brackets on both the top and

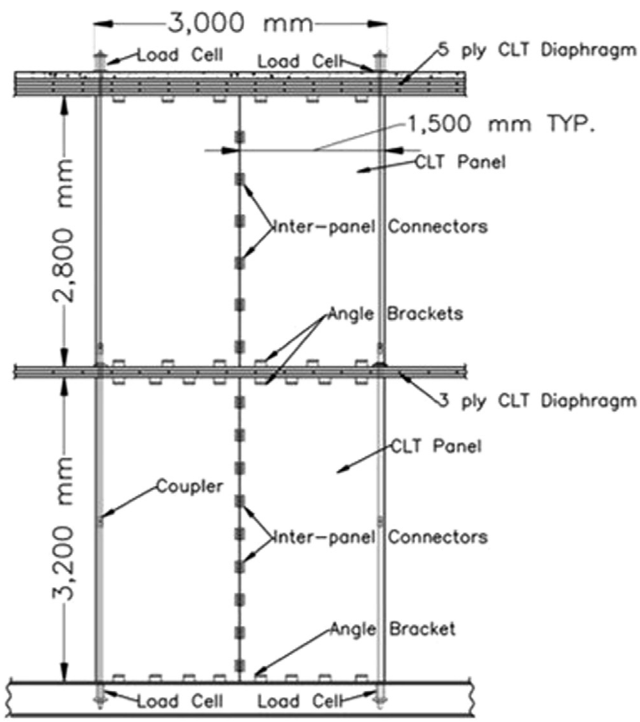

Fig. 4. Phase 3.2: 2.1:1 aspect ratio panels shear wall stack layout.

bottom. The Phase 3 wall system can be seen in Fig. 5.

2.5. Instrumentation

The response of the building during shaking was recorded by over 300 sensors placed in strategic locations throughout the building. As mentioned previously, the structure was part of a collaborative test program with various lateral systems and throughout the testing, the sensors installed to measure the response of the gravity frame and diaphragms remained unchanged. However, the sensors installed on the SFRS varied from each stage and in some cases within the stage. The

Table 1
Instrumentation on gravity frame and diaphragm.

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

diaphragms and gravity frame had 274 sensors installed throughout both floors of the building, and the quantity of each type of sensor can be seen in Table 1. Strain gauges were used to measure the deformation in the chord splices on the diaphragms as well as the rebar installed in the concrete of the composite roof. Linear potentiometers were used to measure the relative displacement between various components of the structure such as: CLT panels and the diaphragm, the diaphragm and gravity frame, and between the concrete and CLT panels in the composite roof diaphragm. String potentiometers were installed in the center of the diaphragm as well as on each corner of both floors to

measure the global displacement of the building. They were also used to measure the relative vertical displacement between floors and the relative vertical and horizontal displacement of the two farthest corners of the structure. Three directional accelerometers units were installed to measure the acceleration in the X, Y, and Z directions, and were installed in a similar fashion on each floor with one located at each corner of the diaphragm, as well as at quarter points along the centerline of the diaphragm.

Fig. 6 shows the location and details of the typical Phase 3.1 wall instrumentation. A total of 72 sensors were installed for Phase 3.1 (see Table 2) on both the north and south walls systems to capture the wall responses. Linear potentiometers were installed horizontally on the base and top of the first and second stories on both the north and south wall systems in order to capture any sliding motion. They were also installed vertically at each corner on the first and second stories on the south wall system to capture the uplift behavior of the walls. However, due to constraints on the quantity of sensors, the north wall system only had vertical linear potentiometers on two corners of the walls on both the first and second story. String potentiometers were installed in the horizontal and vertical directions on two of the three joints between the

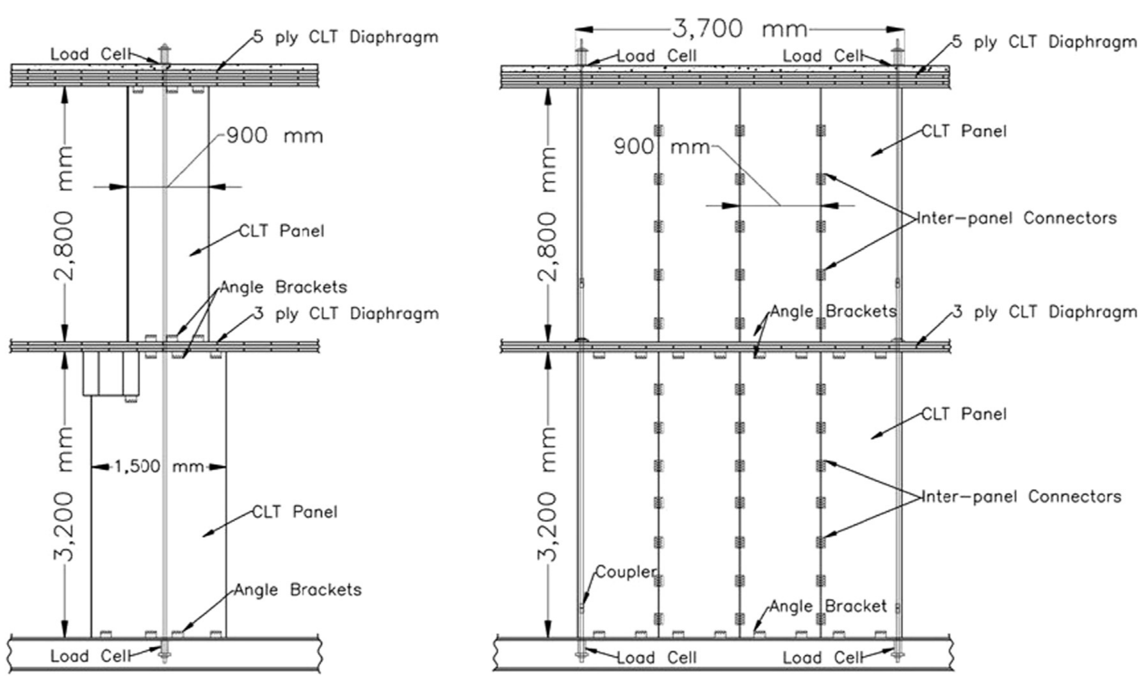

Fig. 5. Phase 3: (a) Transverse CLT walls; (b) 3.5:1 aspect ratio panels shear wall stack layout.

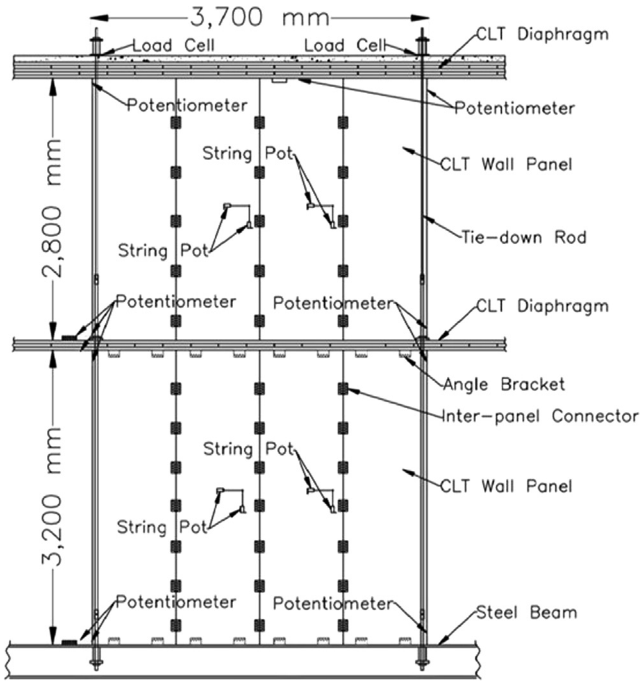

Fig. 6. Phase 3.1 instrumentation on south face of south shear wall (typ. across phases).

Table 2
Instrumentation on shear walls.

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

CLT panels on each of the four walls that comprised the north and south wall systems, to measure any relative panel displacement in either the vertical or horizontal directions. Two string potentiometers were also installed diagonally in opposing directions on both a first and second story wall on the north wall system to measure any panel deformation. A total of 16 load cells were installed in line with the anchor bolts on the base and top of each ATS anchor rod on both the north and south wall system to measure the tension in the rods caused by the overturning moment. Strain gauges were also installed strategically along the lengths of the rods as a redundant measurement to determine the tension in the tie down rods in case of load cell failure.

The Phase 3.2 SFRS sensors were installed in a similar fashion to Phase 3.1; however, there were some minor differences due to the smaller wall dimensions and fewer CLT panels making up each wall. With only one inter-panel joint in each wall, there were several unused string potentiometers previously utilized in Phase 3.1 for relative panel deformation. These sensors were instead used to measure panel deformation on additional panels on both the north and south wall systems. All four panels comprising the two-story north wall system were measured for panel deformation as well as one panel from each floor on the south wall system. The total quantity of sensors for Phase 3.2 can be seen in Table 2.

The Phase 3.3 string potentiometers, load cells, and strain gauges were installed in the same locations as Phase 3.1; however, the locations of some of the linear potentiometers changed due to the addition of transverse walls on either end of all four walls comprising the north

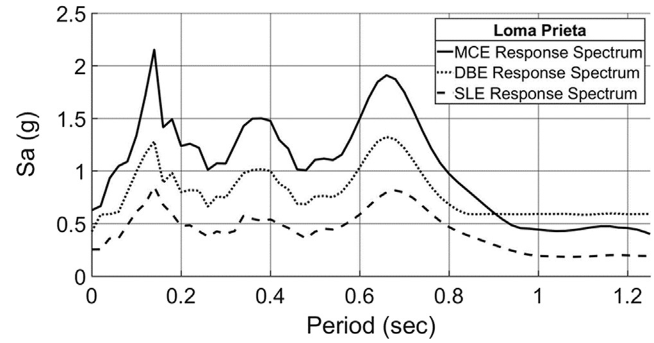

Fig. 7. Spectral accelerations for Loma Prieta scaled to SLE, DBE, and MCE levels.

and south wall systems. Some of the sensors measuring sliding were moved from the end of the walls towards the center due to the previous locations being blocked by the installed transverse walls. One potentiometer from each floor on the south wall system was moved to the end of the transverse wall on their respective floors to capture any potential uplift. The total quantity of sensors for Phase 3.3 can also be seen in Table 2.

2.6. Ground motion and testing program

Component 2 of 1989 Loma Prieta earthquake record from the Capitola recording station was used and scaled to various intensities for all testing according to ASCE 7-16. The scaling of the motion was done according to FEMA P695 for a location in San Francisco, California. Intensities were selected based on levels corresponding to a service level earthquake (SLE), design basis earthquake (DBE), and maximum considered earthquake (MCE), with mean return periods of 72 years, 475 years, and 2475 years respectively. For the MCE motion, this led to a S_{DS} and S_{DI} of 1.5 g and 1.0 g, respectively. The MCE scaled response spectra is presented in Fig. 7, while the spectral accelerations for each phase is shown in Table 3. Due to time constraints for repairs, Phase 3.2 and 3.3 testing used only the SLE and MCE scaled ground motions, which optimized the amount of data collected with a minimal amount of repair.

The natural period of the test structure varied between tests due to structural damage and changes in wall configuration. White noise tests were conducted before and after every test in order to determine the natural period. Fig. 8 presents a plot of the change in the natural period of the structure over the platform CLT testing program. Initially with Phase 3.1 (3.5:1 aspect ratio shear walls), the test structure had a natural period of approximately 0.38 s, which increased following testing with the peak natural period following Phase 3.1 being approximately 0.89 s. The installation of the Phase 3.2 shear walls, which had an aspect ratio of 2.1:1, returned the natural period of the structure to approximately 0.41 s. The peak natural period following Phase 3.2 testing was 0.73 s after the MCE level test. Phase 3, with the return to the 3.5:1 aspect ratio shear walls, and the addition of transverse CLT walls, reduced the natural period of the structure to 0.17 s. This decrease is most likely due to the additional stiffness provided by the transverse CLT walls, including some potential “flanging” action during smaller deformations in and near the elastic range. The peak natural period for the structure following Phase 3 testing after the MCE level test was the shortest of the phases at 0.35 s. This indicates that the least amount of softening occurred in Phase 3 with the transverse walls in place.

Table 3
Test sequencing and global story response for each phase.

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

[a] Phase 1 (3.5:1 aspect ratio CLT shear walls), Phase 2 (2.1:1 aspect ratio), Phase 3 (4:1 aspect ratio with additional transverse walls).
[b] Loma Prieta earthquake ground motion was used for all tests, and scaled appropriately to SLE, DBE, and MCE levels respectively.
[c] Average of the displacements recorded at the center, and both ends of each story.
[d] 1 in = 25.4 mm.
[e] 3663 mm (144 in) and 3,043 mm (120 in) were used for the height of the first and second story respectively.
[f] 342.5 kN (77 kips) and 422.5 kN (95 kips) were used for the weight of the first and second story respectively.
[g] 1 kip = 4.448 kN.

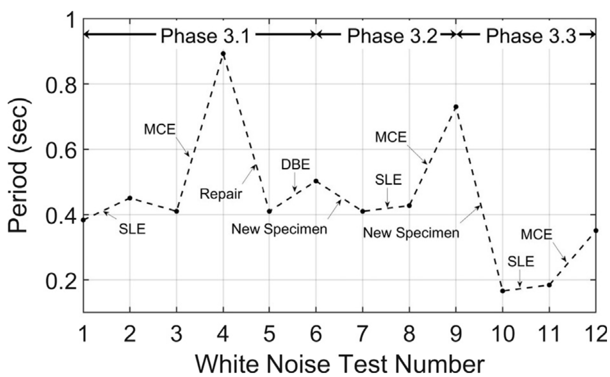

Fig. 8. Fundamental building period and the effect of repairs and different wall configurations.

3. Results and discussion

3.1. Displacement profile

Fig. 9 presents the displacement profile of the test structure for each test, with the profile constructed using the average maximum displacement of the story relative to the shake table. The horizontal displacement was measured at the north, south, and middle of the diaphragm on each story. Across all of the phases, primarily a first mode response was observed, with the peak interstory drift for each story occurring simultaneously to the peak displacement. Phase 3.1 and Phase 3.2 had similar peak MCE level average displacements of 158 mm (6.22 in.) and 159 mm (6.26 in), respectively, and Phase 3.3 had the smallest peak average displacement of 151 mm (5.94 in.). As mentioned previously, the objective of the testing was to demonstrate the performance of CLT shear walls using typical design techniques, in the case when ELF was used for the design. In the design, 2/3 of the MCE level spectral acceleration is reduced by a seismic reduction factor (R), developed over the course of the project, to account for the nonlinear response of the structure, specifically through its deformation capacity and ability to dissipate energy. The nonlinear response is confirmed by the results in Fig. 9, as the MCE level motion is 1.5 times greater than the DBE level motion, but the maximum displacement for the Phase 3.1 MCE test (158 mm) is greater than 1.5 times the maximum displacement for the Phase 3.1 DBE test (94 mm). It was theorized in Popovski et al. [14] that the addition of walls transverse to the shear walls would improve the performance of the structure, and as can be seen in Table 3 and Fig. 9, even though Phase 3.3 MCE had the largest peak ground acceleration (PGA) recorded by the table, its peak floor displacements were less than Phase 3.1 and Phase 3.2 on both floors.

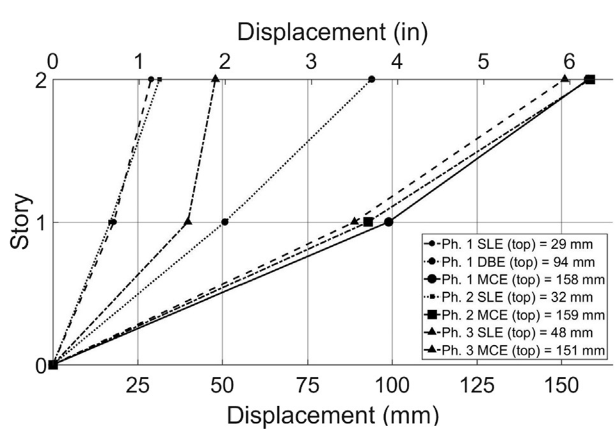

Fig. 9. Average displacement of first and second stories for each test.

3.2. Interstory drift

The interstory drifts (ISDs) of the test structure were obtained by finding the average relative displacement of each story. For the first story, this was done by taking the difference of the string potentiometers on the first floor level diaphragm and the table displacement feedback data, then averaging the results. The second story relative displacement was calculated in a similar way, except that instead the difference between the first and roof floor levels string potentiometers was used. The peak interstory drift was then divided by the relevant story height, 3663 mm (144 in) and 3043 mm (120 in) for the first and second stories, respectively, to obtain drift as a percentage of the story height. Table 4 shows the ISDs for all of the tests, and it can be seen that the first story ISDs were larger than the second story, which may imply slight soft story behavior of the structure, but could also have been influenced by the different story heights. This is particularly evident in the Phase 3.1 MCE level ISD results, where it can be seen in Fig. 10 that the first story has an ISD 0.75% greater than the second story. The addition of transverse walls in Phase 3.3 improved the performance significantly of the first story even though the PGA was greater than in Phase 3.1.

3.3. Global hysteresis

Newton's second law was used to calculate the inertial force of each story by using the average of acceleration time histories recorded on

Table 4
Test sequencing and local story response for each phase.

		Peak uplift[a] (mm)	Peak vertical relative panel displacement[b] (mm)	Peak sliding[c] (mm)		Peak ATS rod force[d] (kN)		Peak ATS rod force[d] (kN)	
Phase	Test	Story 1	Story 2	Story 1	Story 2	Story 1	Story 2	Story 1	Story 2
3.1	SLE	4.7	3.3	2.6	1.2	1.7	2.4	44.5	9.8
	DBE	13.2	12.5	7.8	4.7	8.1	9.0	105.1	31.3
	MCE	21.7	16.7	19.6	10.0	12.0	9.4	170.0	66.3
3.2	SLE	4.2	2.7	1.9	1.0	1.1	1.7	45.8	12.7
	MCE	26.1	12.2	13.6	9.9	56.1	32.7	237.6	74.4
3.3	SLE	3.7	2.7	2.1	1.4	2.6	1.1	31.6	10.2
	MCE	14.5	9.4	15.7	13.6	7.2	6.5	–	–

[a] Uplift was measured at the upper and lower corners of each wall on both the first and second story.
[b] Vertical Relative Panel Displacement is the vertical displacement measured at the inter-panel joint between CLT panels.
[c] Sliding was measured and the top and base of each wall on both the first and second story.
[d] Load cells were placed on the top and base of each ATS rod.

both floors at each corner and along the center of the diaphragm at the north edge, center, and south edge. The shear force for each story was then calculated accordingly, and Table 3 presents the shear force for all seismic tests and both stories. It can be seen that the largest shear force of 873 kN (196 kips) occurred during Phase 3.3 and Phase 3.2 MCE. The maximum shear force in the first story of both Phase 3.2 and Phase 3.3 exceeded the design capacity of the shear connectors, however, it should be noted that the shear walls were designed to less than 2/3 of the design capacity that will be proposed. The walls were not in danger of failing and the overstrength seen in the testing could have had several possible origins. In Amini et al. [1] the R factor was determined using the average capacity of the connectors throughout testing, and it is possible the shear walls in the full scale test exceeded the average. The testing in the previously mentioned study was also done with minimal gravity loading, and the additional gravity loading here could have increased the capacity to the wall. Finally, the manufacturer of the CLT panels was not the same although the wood species was the same and both met current standards for manufacturing CLT.

Fig. 11 compares plots of the floor displacements versus the story shear for Phase 3.3 MCE and Phase 3.1 MCE. Recall that Phase 3.1 and Phase 3.3 both had 3.5:1 aspect ratio panels, but Phase 3.3 also included the addition of transverse walls, which was done to simulate a real structure where there would be shear walls in both directions and to investigate their effect on performance. As can be seen, while the story shear is higher in Phase 3.3, the displacements are either similar or less than Phase 3.1. Since the PGA was larger in Phase 3.3, this implies that the transverse walls had no negative effect on performance, and in fact improved performance of the structure, which was expected but is quantified here.

3.4. Torsion

Fig. 12 shows the deformed shape of the diaphragm indicating some torsion, with the south end of the second floor level displacing 35 mm (1.38 in) more than the center of the structure. It can also be seen that the torsion on the first story was not as large (25 mm), it was still however present and it should be noted that the north end of the structure did not experience much torsion across either story. The torsion experienced by the structure increased further into the testing program, but despite this, the CLT shear wall stacks still performed well.

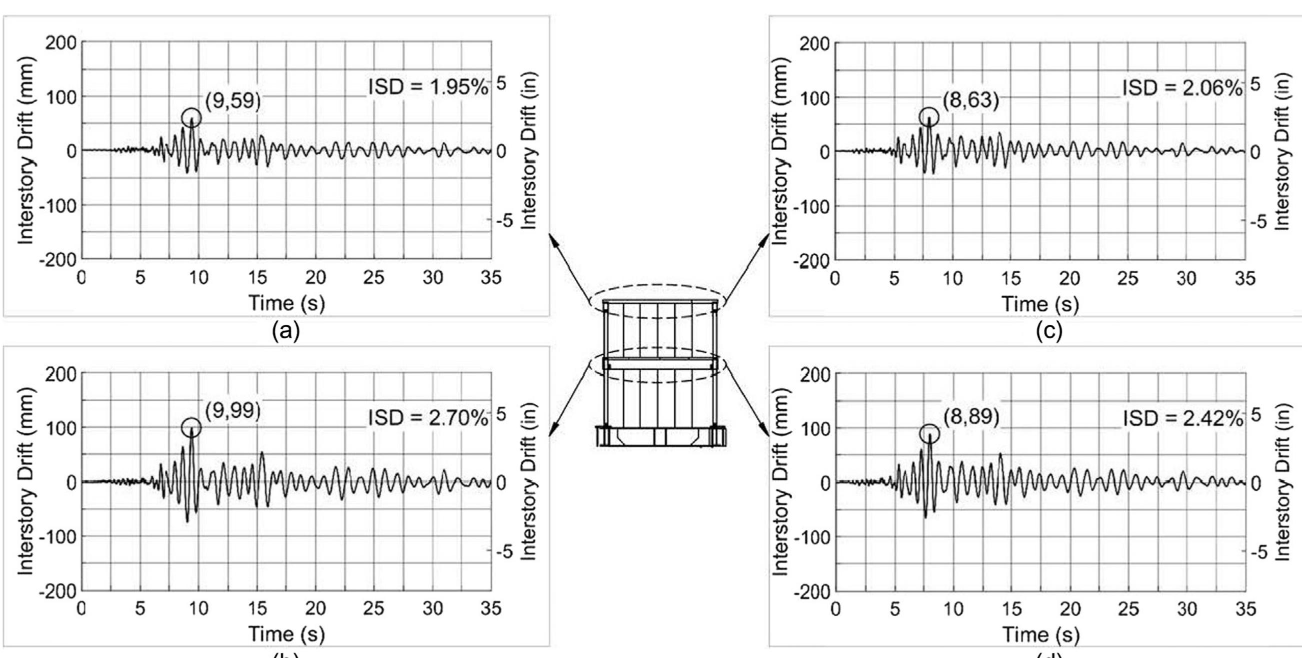

Fig. 10. Interstory drift: (a) Phase 3.1 MCE second story; (b) Phase 3.1 MCE first story; (c) Phase 3.3 MCE second story; (d) Phase 3.3 MCE first story.

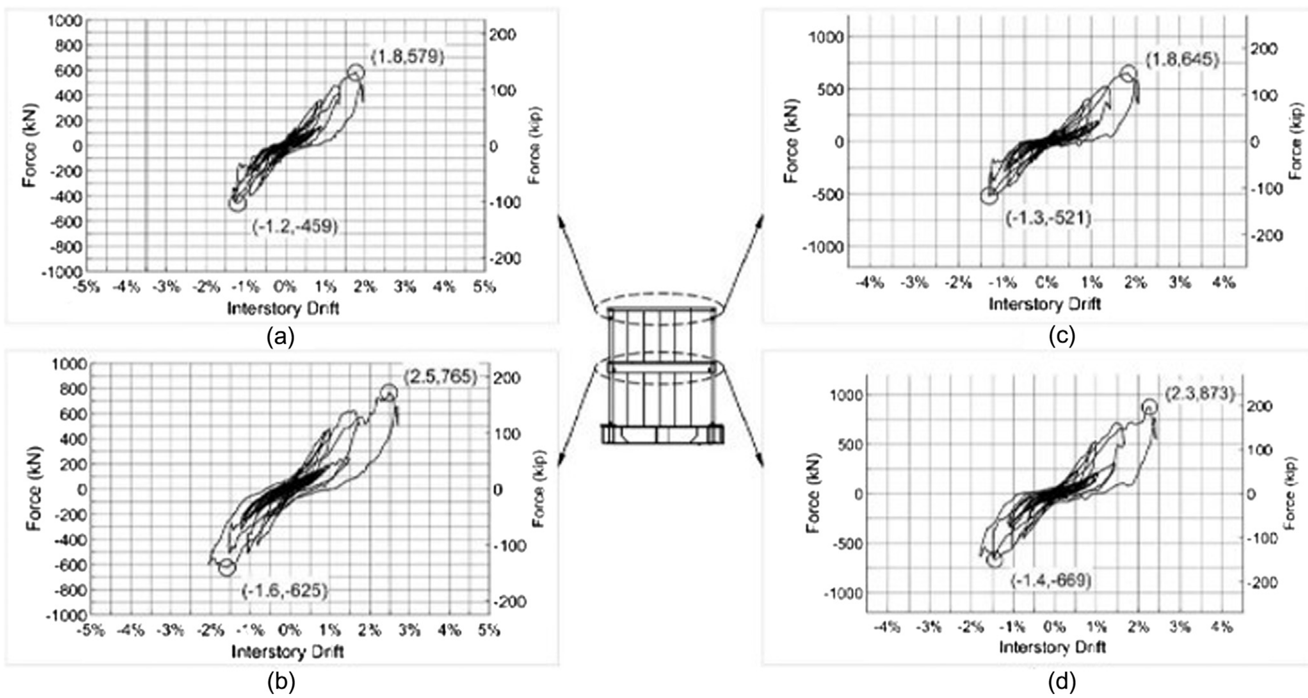

Fig. 11. Global hysteresis curves: (a) Phase 3.1 MCE second story; (b) Phase 3.1 MCE first story; (c) Phase 3.3 MCE second story; (d) Phase 3.3 MCE first story.

3.5. CLT panel uplift

One of the principal objectives in the testing was to observe individual CLT wall panel behavior and investigate the effect of aspect ratio and in the case of Phase 3.3, the addition of transverse walls on that behavior. To capture any potential uplift of the panels, linear potentiometers were used at the four corners of the shear wall on both the first and second stories, and Table 4 summarizes the results. In addition, for Phase 3.3, linear potentiometers were installed on the transverse walls to capture any potential uplift. Fig. 13 shows the uplift recorded

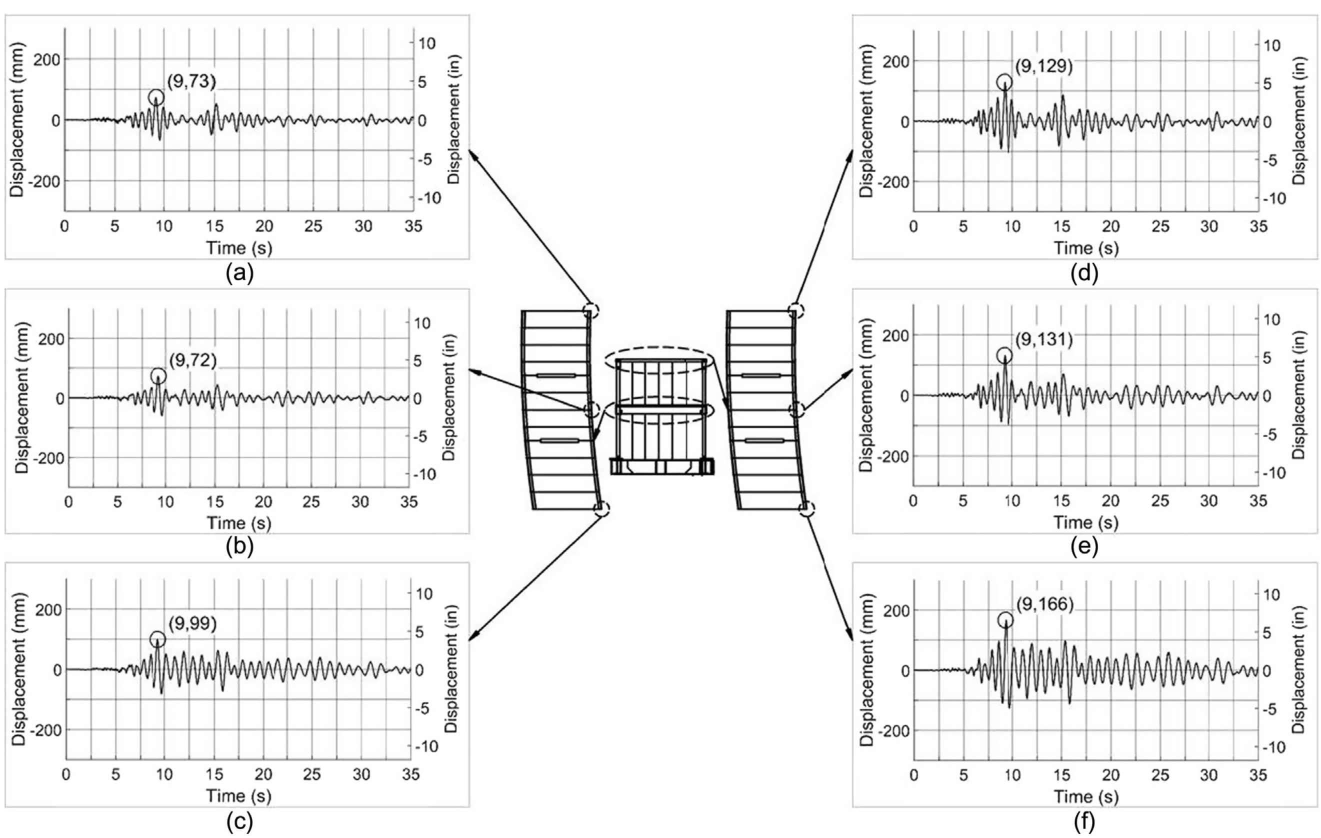

Fig. 12. 3.1 MCE Displacement: (a) first floor level northeast corner; (b) first floor level center east side; (c) first floor level southeast corner; (d) second floor level northeast corner; (e) second floor level center east side; (f) second floor level southeast corner.

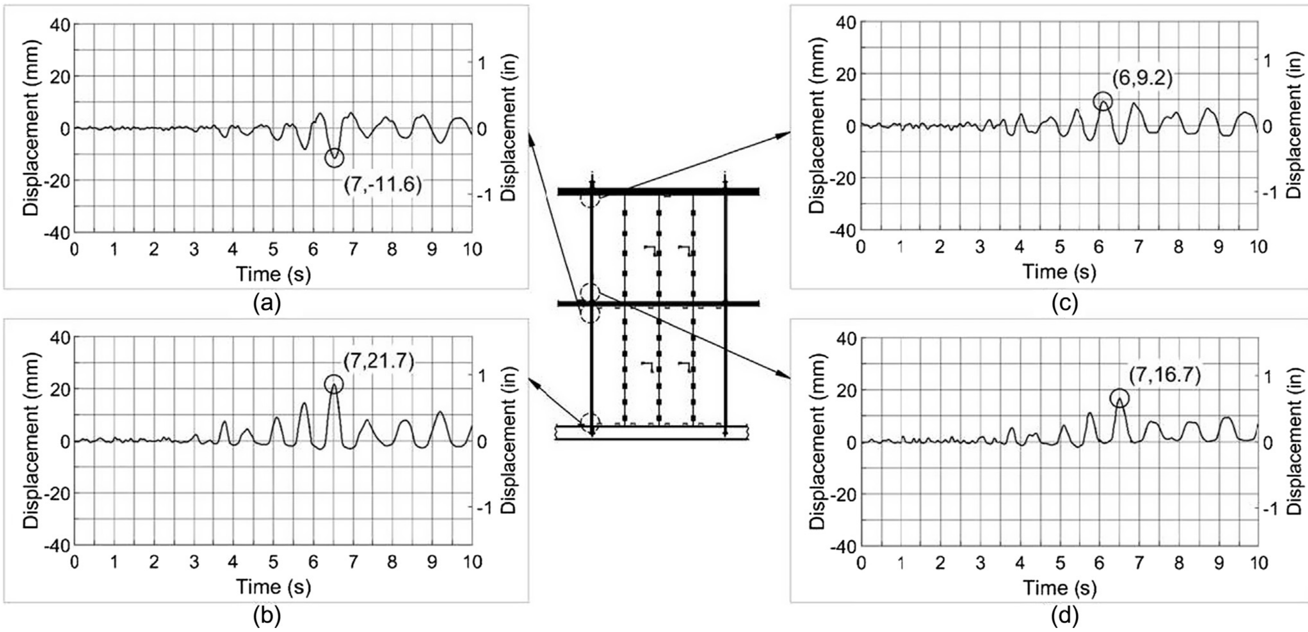

Fig. 13. Phase 3.1 MCE Wall Uplift: (a) First story shear wall top west corner; (b) First story shear wall bottom west corner; (c) Second story shear wall top west corner; (d) Second story shear wall bottom west corner.

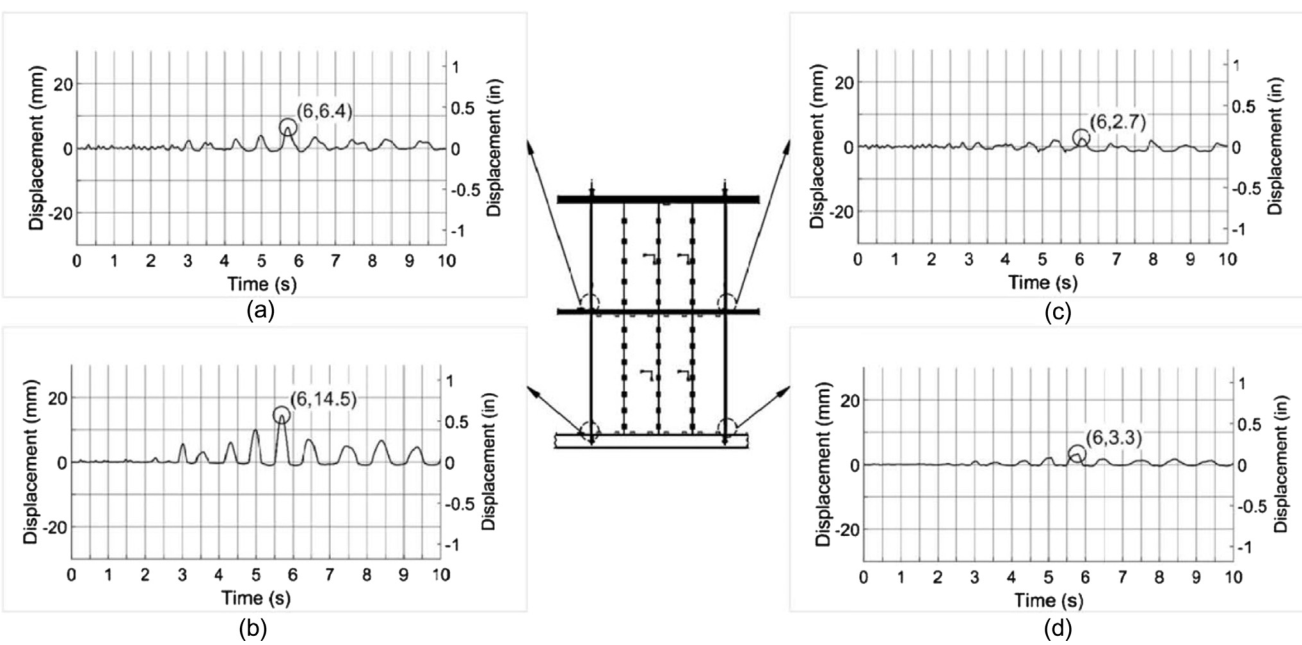

Fig. 14. Phase 3.3 MCE Uplift: (a) Second story shear wall bottom west corner; (b) First story shear wall bottom west corner; (c) Second story transverse wall top south corner; (d) First story shear wall bottom south corner.

on the west side of the south shear wall stack during Phase 3.1 MCE, and it can be seen that the bottom of the panels experienced the most rocking, with the first story having the largest uplift values. The top of the panels experienced smaller uplift and hence less rocking, which was expected. Fig. 14 shows the uplift recorded during Phase 3.3 MCE on both floors of the south shear wall stack as well as the uplift recorded at the base of the transverse walls. There was a concern that the addition of transverse walls on either end of the shear wall stack could inhibit the ability of the CLT panels to rock. Fig. 14 (a and b) demonstrate that the CLT panels were still able to exhibit rocking behavior even with transverse walls present, and the behavior was similar to Phase 3.1. It was expected that the transverse walls would experience minimal uplift during testing, and Fig. 15(c and d) show that is indeed the case. The largest uplifts recorded during testing occurred during the Phase 3.2 MCE test, with a peak uplift of 24.4 mm (0.96 in) recorded at the bottom of the first story, but it should be noted that this measurement could have been affected by the nail shearing observed on the first story. Even though the uplift was the largest in Phase 3.2, the test structure still exhibited no risk of collapse. The uplift in Phase 3.2 followed a similar pattern to the other phases with the bottom of the CLT panel experiencing the most rocking behavior on both the first and second stories.

3.6. Sliding

The sliding of the shear wall panels was recorded using linear

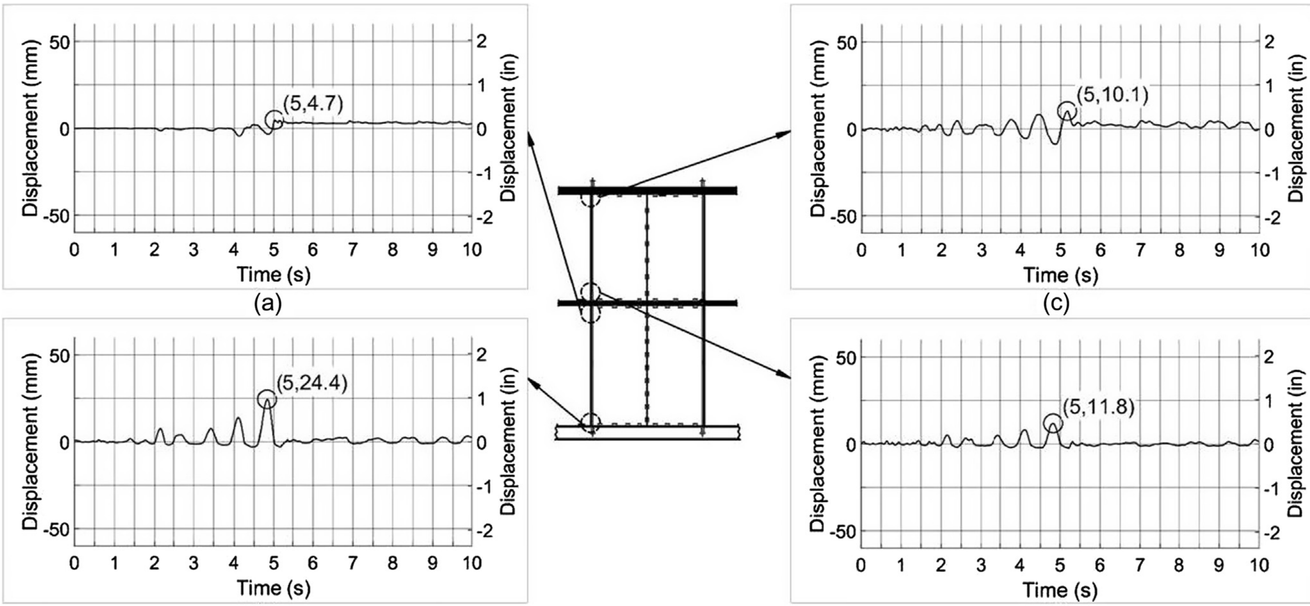

Fig. 15. Phase 3.2 MCE Uplift: (a) First story shear wall top west corner; (b) First story shear wall bottom west corner; (c) Second story shear wall top west corner; (d) Second story shear wall bottom west corner.

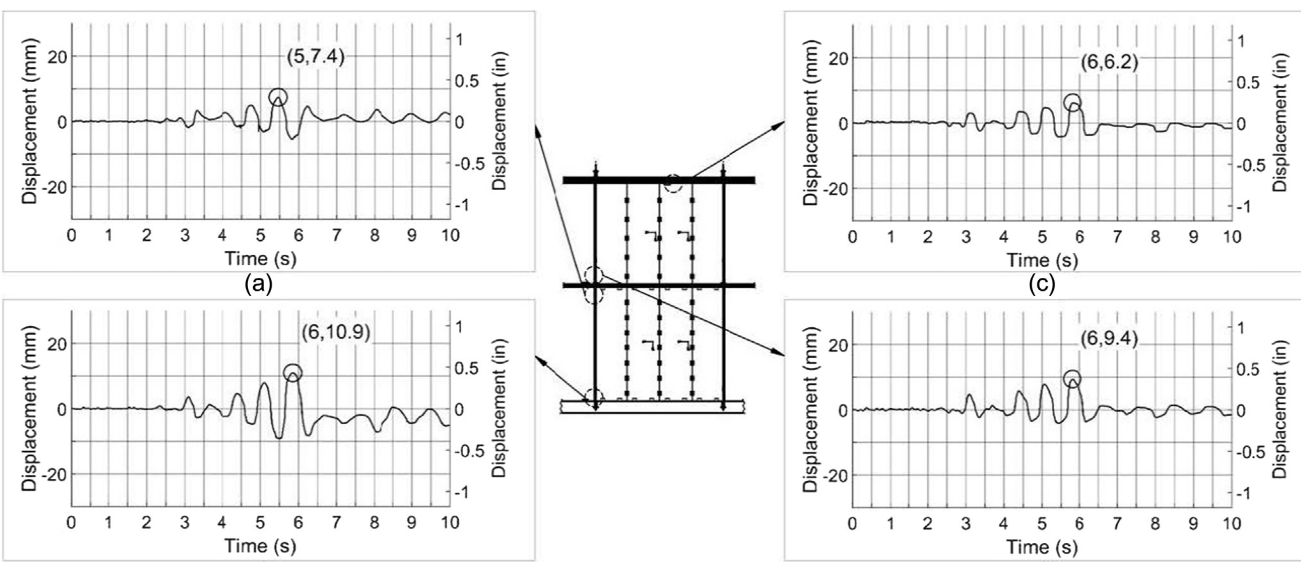

Fig. 16. Phase 3.1 MCE Sliding: (a) First story shear wall top west corner; (b) First story shear wall bottom west corner; (c) Second story shear wall top west corner; (d) Second story shear wall bottom west corner.

potentiometers positioned horizontally at the base and top of each shear wall, and Table 4 summarizes the results. The panel sliding followed a similar pattern to the uplift, with the largest sliding occurring at the base of the panel at each story, and this can be seen in Fig. 16 for Phase 3.1 MCE. The sliding behavior can be partially explained by nail withdrawal as the deformation of the angle brackets work the nails from the CLT. This was especially present for the Phase 3.2 MCE test, where nail shear was observed on the first story, resulting in a sliding of 56.1 mm (2.2 in) as seen in Fig. 17, but although the sliding was large, the test structure was in no danger of collapsing with the tie down rods providing uplift restraint and likely some level of shear through bearing on the hole they passed through in the CLT diaphragm and, in the case of the lower shear walls, the steel support beam acting as a foundation into the shake table.

3.7. Relative panel displacement

Relative panel displacement was measured in the horizontal and vertical direction using string potentiometers at the inter-panel joints of each CLT panel for all tests and phases. No significant relative displacement was recorded or observed in the horizontal direction, so the vertical direction will be the primary focus, and Table 4 summarizes the results across the phases. Fig. 18 presents the Phase 3.1 MCE results, and as can be seen, the first story experienced more relative panel displacement than the second story, which was expected given it deformed more overall. The displacement was also relatively consistent across the story with only a 3 mm (0.11 in) difference between the two recorded joints for Phase 3.1 MCE. The relative panel displacements for the 2.1:1 shear wall aspect ratio followed a similar trend to the 3.5:1 aspect ratio walls in Phase 3.1, the largest displacement occurred on the

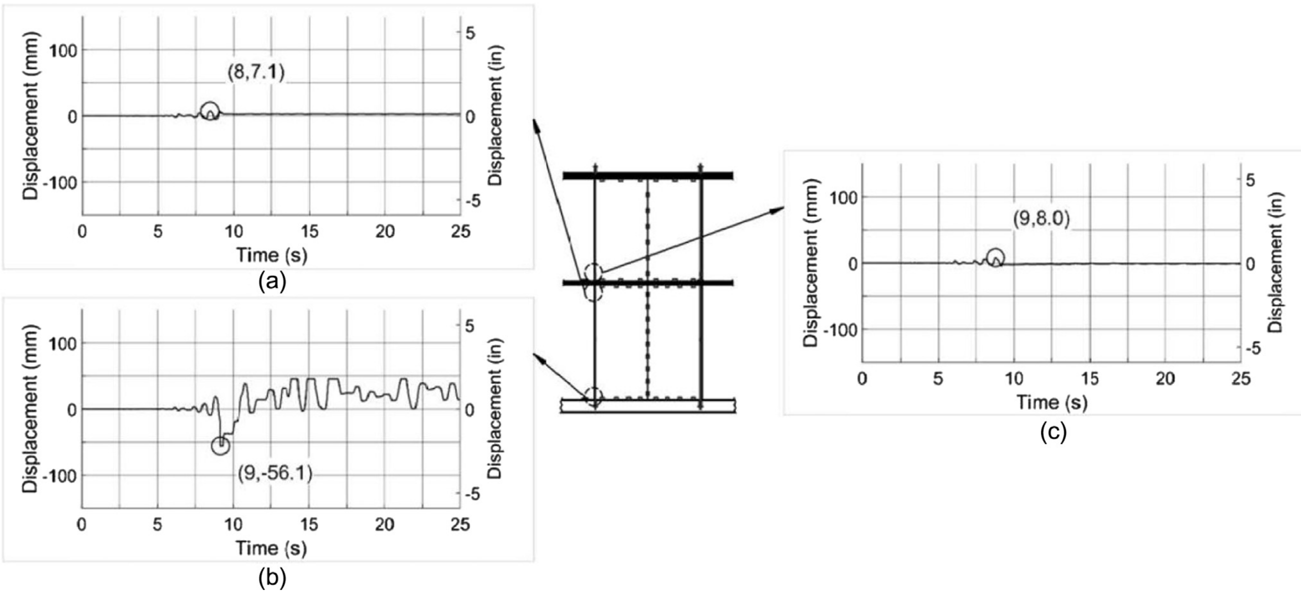

Fig. 17. Phase 3.2 MCE Sliding: (a) First story shear wall top west corner; (b) First story shear wall bottom west corner (nail shear failure occurred in brackets); (c) Second story shear wall bottom west corner.

first story, this can be seen in Fig. 19.

3.8. Forces in tie-down rods

ATS tie-down rods were installed in the SFRS, with a rod on each end of the wall and both faces, with a bearing plate providing a reaction point on each story, similar to standard construction for light-frame wood buildings. This was consistent throughout all the phases, and in the design of the SFRS, it was assumed that the CLT wall panels worked as a continuous segment with the inter-panel connectors transferring shear, and because of this, tie-down rods were only required at the ends of the walls. The tie-down rods are designed to absorb tension from the overturning moment present in the structure, while the CLT panels and brackets transfer shear. The rods therefore are not designed to take any compression force, and are thus allowed to slide through at the bottom of the structure, allowing the CLT panels to absorb the compression load. The summary of the recorded forces in the rods can be seen in Table 4. No load cell data was available for the Phase 3.3 MCE level test, so data from strain gauges placed on the tie-down rods was used instead. The largest tension force in the rods was 237.6 kN (53 kips) recorded at the base on the first story during the Phase 3.2 MCE test, which was approximately 94% of the ultimate capacity of the rod. This test included the 2.1:1 aspect ratio walls, and it most likely due to the nail failure in the shear brackets, essentially forcing the hold downs to do 100% of the resistance in uplift. Although the shear brackets are assumed to only resist shear in the design approach for the platform

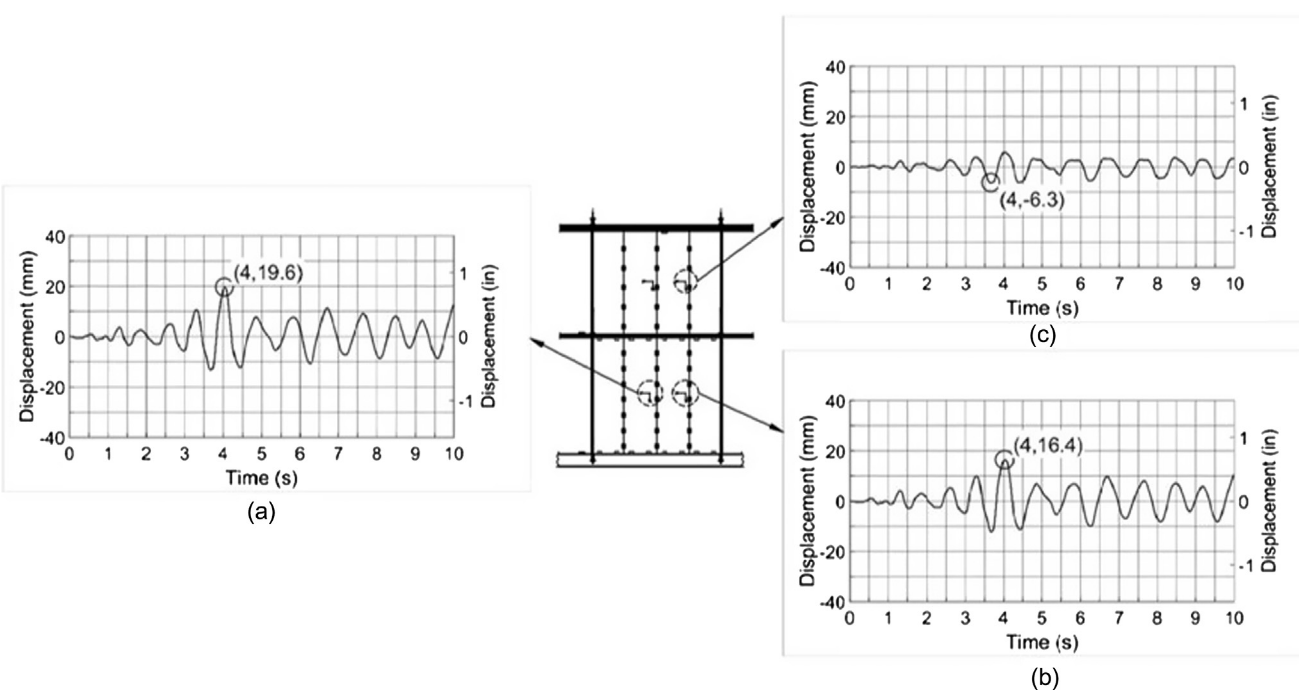

Fig. 18. Phase 3.1 MCE Vertical Relative Panel Displacement: (a) First story left middle CLT panel; (b) Second story right middle CLT panel; (c) First story right middle CLT panel.

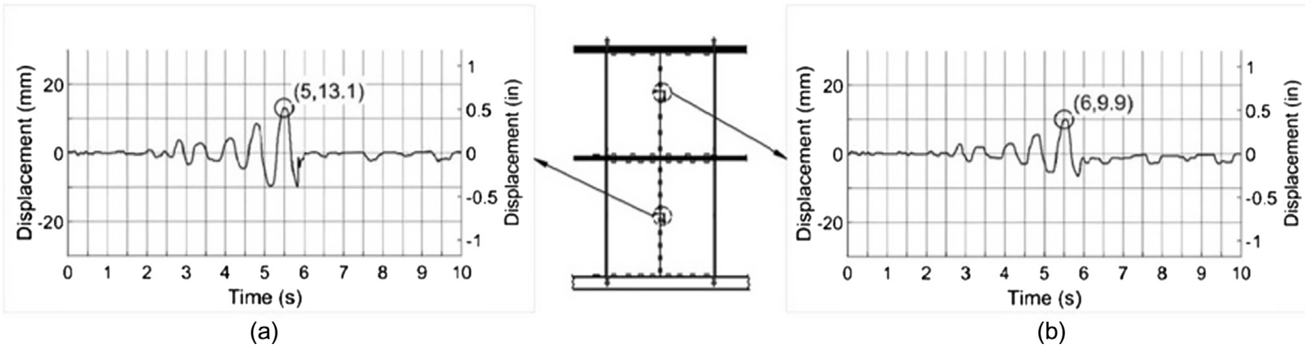

Fig. 19. Phase 3.2 MCE Vertical Relative Panel Displacement: (a) First story CLT panel; (b) Second story CLT panel.

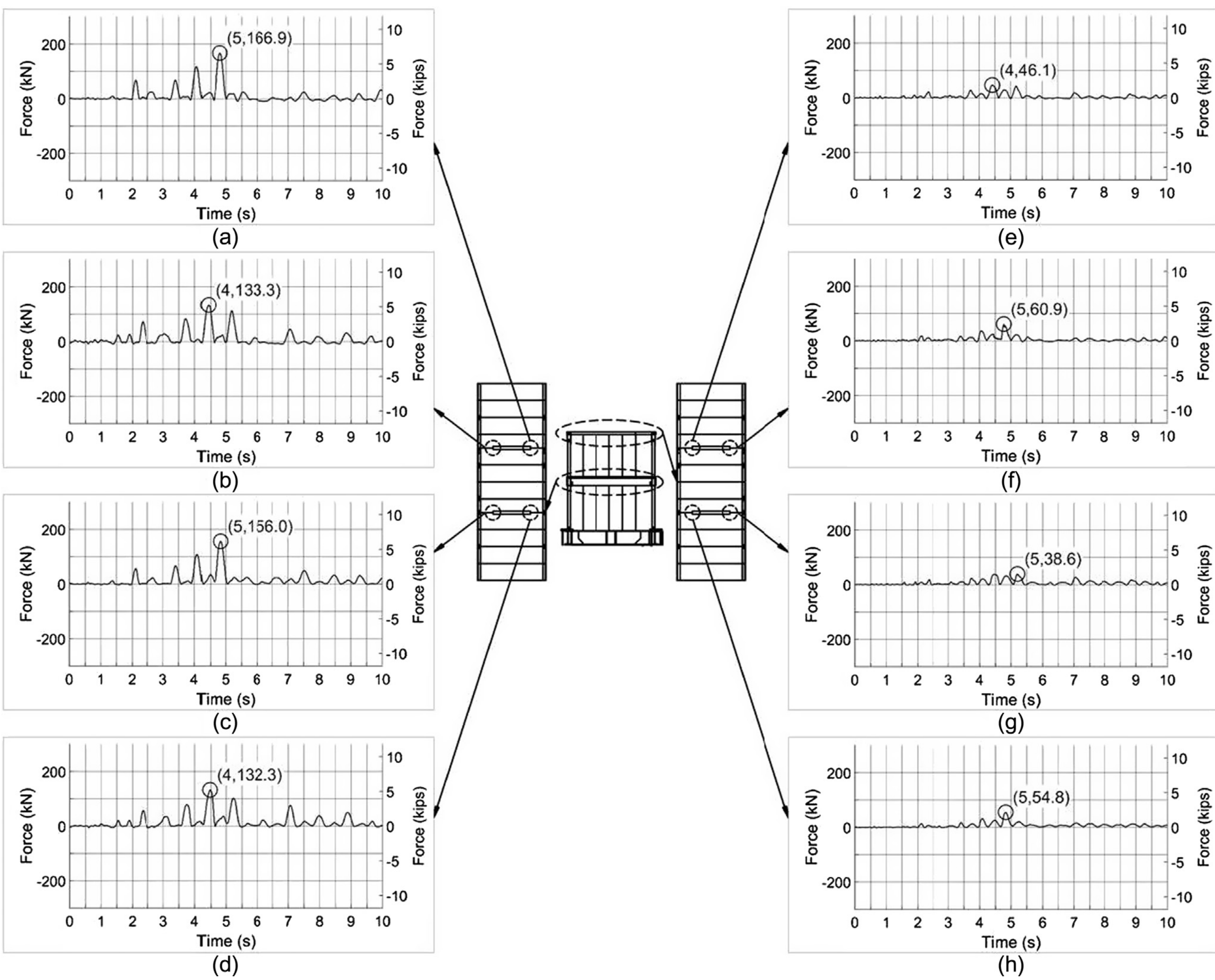

Fig. 20. Phase 3.1 ATS Rod Load Cells: (a) First story north wall east side; (b) First story north wall west side; (c) First story south wall west side; (d) First story south wall east side; (e) Second story north wall west side; (f) Second story north wall east side; (g) Second story south wall east side; (h) Second story south wall west side.

CLT stacked walls herein, they do resist some overturning as the CLT panels rack. A large tension force of 170 kN (38 kips), or 68% of the rod's ultimate capacity, was also recorded during the Phase 3.1 MCE level test, and the tension both in this test and the Phase 3.2 MCE test were enough to cause some yielding in the A36 tie-down rods. Fig. 20 presents the average tension force across both wall faces on the CLT panels from Phase 3.1 MCE level test at both the base and roof of the structure. It can be seen that the loading was not homogenous across the structure, and the east side received more load than the west side. This is a result of the torsion discussed earlier, and both Phase 3.2 and Phase 3.3 showed a similar trend. An objective of the test program was to determine if the addition of transverse walls in Phase 3.3 would decrease the loading in the tie-down rods. Unfortunately, load cell data for Phase 3.3 MCE was unavailable due to technical difficulties. However, strain gauges were placed in strategic locations along the tie-down rods to act as a backup in such an event. Phase 3.3 MCE had a maximum recorded strain of 6.92e-04, while Phase 3.1 MCE had a maximum recorded strain of 7.15e-03. The difference in strains between the tests are rather significant on both stories, with the Phase 3.3 MCE level test experiencing strains an order of magnitude smaller, thus making the conversions to force unnecessary to analyze the performance of the structure. As anticipated, the addition of transverse walls reduced the strain in the tie-down rods, implying reduced force, and ultimately, reduced overturning moment and improved performance.

3.9. Overall performance

In all three phases presented herein, each of which consisted of several different ground motion intensities, the structure provided life safety had there been occupants in the building. Phase 3.1 and Phase 3.3 CLT panels both clearly were governed by rocking behavior as had been demonstrated to occur in high aspect ratio panels. Partial nail pull-out and some steel angle bracket deformation was observed during Phase 3.1 and Phase 3.3 MCE level tests, but this was expected and results indicated that the connectors behaved as intended. It was also observed that the transverse walls installed in Phase 3.3 did not significantly affect the ability of the CLT panels to rock or the connections to perform as designed. Phase 3.2 and the lower aspect ratio panels were governed by sliding, and during the MCE level test, significant sliding and nail shearing was observed, resulting failure of the shear brackets at the base. Although this is far from an ideal behavior, it should be noted that the design used in this test was less than 2/3 the capacity eventually to be proposed for design of platform CLT systems in the U.S.; and also it is important to note that the stability of the structure was never in jeopardy. The tie-down rods experienced some yielding effects in both the Phase 3.1 and Phase 3.2 MCE level tests, but still performed as designed and resisted the overturning moment. In Phase 3.3, there was no yielding observed due to the improvement in performance provided by the addition of the transverse walls. At SLE and DBE level tests, there was no observable damage in the connections in the shear wall stacks, and no yielding recorded in the tie-down rods. Towards the end of the testing, torsion in structure began to become more pronounced, and introduced some asymmetric loading into the structure, but the SFRS still performed such that there was never risk of collapse at MCE level shaking.

4. Summary and conclusions

A full-scale two-story CLT platform building with two CLT panel shear wall stacks with tie-downs rods with three different shear wall configurations each tested, namely, 3.5:1 aspect ratio panels, 2.1:1 aspect ratio panels, and 3.5:1 aspect ratio panels with transverse walls installed on each end, being tested. All of the configurations were subjected to the 1989 Loma Prieta ground motion scaled to intensities corresponding to SLE, DBE, and MCE levels respectively, with spectral accelerations ranging from 0.52 g to 1.5 g. The design of the shear wall stacks used the equivalent lateral force procedure with the intent of providing life safety to would-be occupants. Each of the designs met the life safety criteria, and the structure was never in danger of collapse over the course of the testing program. This met the primary objective of the test to demonstrate the effectiveness of CLT shear-walls to provide life safety. The configurations performed as expected with the 3.5:1 aspect ratio being governed by rocking of the panels, and the 2.1:1 aspect ratio governed by sliding of the panels. However, the sliding in the 2.1:1 panels in Phase 3.2 was the result of nail shear failure in the base shear brackets, and thus no true conclusion that 2.1:1 aspect ratio panels has a sliding mechanism as its failure model can be drawn. This was more of a capacity issue given the shear wall capacities were designed at less than 2/3 of what is to eventually be proposed. This was mainly the result of the timing of the shake table test, which was an opportunity to perform the test with a gravity frame in place already, prior to completion of the design method for CLT platform construction being fully validated. However, as described in this study, a number of valuable conclusions were able to be drawn related to the behavior of this type of construction and connections for CLT, and accomplished the goal of providing insight into CLT panel aspect ratios. For example, it was also confirmed that the addition of transverse walls does not affect the ability of the panels to rock, and improves the performance of the tie-down rods. The results of this testing will be used to further refine the methodology for designing CLT SFRS using the equivalent lateral force procedure in the U.S., which will be used to suggest design provisions in the future.

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Appendix A. Supplementary material

Supplementary data to this article can be found online at https://doi.org/10.1016/j.engstruct.2018.12.079.

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