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EXPERIMENTAL SEISMIC BEHAVIOR OF A TWO-STORY CLT PLATFORM BUILDING: SHAKE TABLE TESTING RESULTS

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ABSTRACT: With the increased usage of Cross Laminated Timber (CLT) in the United States, research efforts have been focused on demonstrating CLT as an effective Seismic Force Resisting System (SFRS). Presented in this paper are the findings of full-scale shake table tests of a two-story 223 m^2 (2400 ft^2) building with two sets of CLT shear walls on the first and second story. The testing consisted of three phases, each with a unique wall configuration, but only the first phase is presented herein, which consisted of a shear wall with 4:1 aspect ratio CLT panels. The structure was subjected to ground motions scaled to intensities that correspond to a Service Level Earthquake (SLE), Design Base Earthquake (DBE), and Maximum Considered Earthquake (MCE) respectively. In all phases and motions the structure performed well and was in accordance with FEMA collapse prevention requirements for each motion intensity.

KEYWORDS: Cross Laminated Timber, Seismic Force Resisting System, FEMA P-695 methodology

INTRODUCTION

Cross Laminated Timber (CLT) has been available as an engineered wood product in Europe for approximately two decades and only recently introduced into the US construction market. Its use in the US as a vertical seismic force resisting system (i.e. shear wall) has been limited in high seismic regions because of lack of seismic provisions for CLT, complicating design and increasing design costs. Due to a growing CLT industry in the Pacific Northwest and Canada, there have been recent efforts to demonstrate CLT as an effective Seismic Force Resisting System (SFRS). Although efforts in the US are more recent, research into CLT as a SFRS has been a growing trend for over a decade. One of the early efforts to investigate CLT was performed by Dujic et al. [5]. This experiment tested 15 various CLT panel configurations to cyclic loading, and from these experiments, it was determined that vertical loading on the CLT wall panels as well as the anchorage had a significant effect on the performance of the system. The most common failures occurred from connector failure, anchorage failure, or the local failure of the wood material (crushing). The SOFIE project was a ground-breaking, comprehensive study funded by the Trento Province in Italy that investigated the feasibility of using CLT as a SFRS in mid-rise buildings, concluding that CLT was in fact suitable for mid-rise buildings in high seismicity regions. A detailed summary of the results can be found in [4], [6]. The initial research efforts in central Europe demonstrating the suitability of CLT as a SFRS inspired continued research all over the world. In North America, Popovski et al. [11] conducted a study to further quantify the performance of CLT under seismic loads. The results of that study determined that rocking and sliding are the primary displacement behaviours of CLT panels, usually caused by deformation or failures in the brackets/connectors. That study stressed the need for further research into the effects of the CLT panel aspect ratio on the performance of the SFRS. A full comprehensive review of North American research efforts can be found in [10]. In this paper the methods and results from one major phase of a recent full-scale shake table program testing a two-story 223 m^2 (2400 ft^2) platform type building are summarized. This testing was a collaborative effort between many universities and

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organizations, and included three individual stages, each with their own sub-phases. This paper focuses on the final stage in support of a larger multi-year study whose aim is to develop seismic performance factors for CLT as a SFRS using FEMA P-695 (2009) methodology. The testing in this stage was performed in three separate phases, each with its own shear wall configuration, however this paper will focus on the first phase and its specific objective, which was the most basic design configuration in the test series.

2 SHAKE TABLE TESTING

2.1 GRAVITY FRAME SHEAR WALLS

The two-story building test specimen included two systems, a gravity frame and CLT shear walls acting as the SFRS. A plan and elevation view of the structure is presented in Figure 1, and Figure 2 shows the specimen on the shake table. It should be noted that the typical wall configuration would consist of more conventional platform style wall placement with multiple walls throughout the floor plan, but due to specimen limitations two parallel stacked shear wall systems were utilized. The gravity frame consisted of CLT floor/roof diaphragms, glulam beams, and columns. More information on the design and seismic performance of the diaphragm and gravity frame can be found in [3].

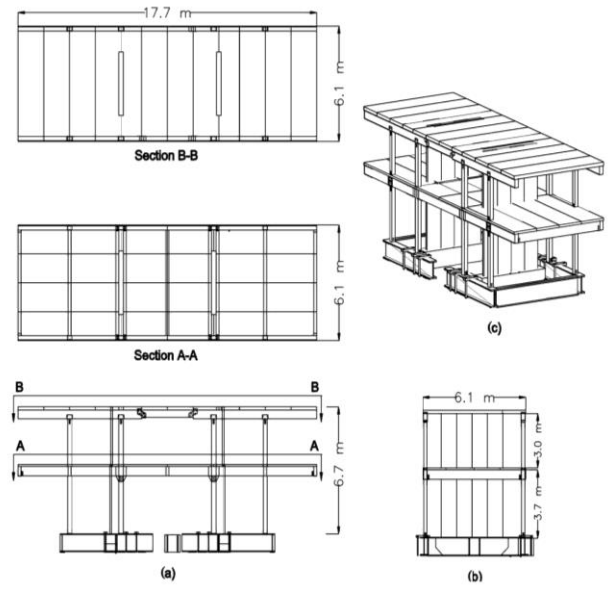

Figure 1: *Elevation, plan, and isometric views of test specimen*

Figure 2: *Two-story test specimen on shake table*

2.2 SHEAR WALLS

The wall configuration for the CLT shear wall was a 3.7 m (144 in) long wall consisting of four 0.9 m (36 in) long panels representing a 3.55:1 aspect ratio. The purpose of this test and the other phases not presented herein was to investigate the performance and behaviour of high and moderate aspect ratio panels in a full-scale shake table test. The CLT wall was designed under the assumption that the tie-down rods resisted overturning moment, and connections at the base and top of the wall, which were made up generic steel angle brackets with nails, solely resisted shear [1]. The tie-down rods provide a continuous load path with bearing provided at each floor with a steel plate reacting against the floor diaphragm, similar to the system used in light-frame wood construction. Inter-panel connectors comprised of nails and steel plates were also added at the vertical connection between panels. The structure was designed using the Equivalent Lateral Force (ELF) procedure in accordance with ASCE Standard 7-16 [2]. The design spectrum values used were obtained for a location in San Francisco with SDS and SD1 of 1.5g and 1.0g respectively (see ASCE 7 for details). Figure 3 shows the layout of the wall configuration including the inter-panel connectors, angle brackets, CLT panels, and tie-down rods.

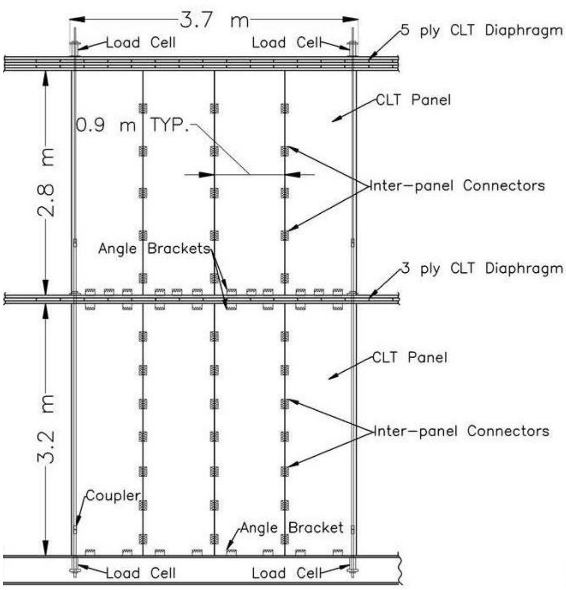

Figure 3: *Shear wall stack layout*

2.3 INSTRUMENTATION

As mentioned previously, the test specimen was part of a larger collaborative effort, with the gravity frame and diaphragm remaining constant throughout the testing. Due to this, the instrumentation on the gravity frame and diaphragm remained constant as well. Table 1 summarizes the types and quantities of the instruments on the gravity frame and diaphragm. In the study presented in this paper, the string potentiometers and accelerometers are of particular interest. There were string potentiometers placed at quarter points on the east side of the diaphragm (for a total of three per story) on the first and second story of the test specimen. These instruments were used to measure the displacement of their respective stories. There were also accelerometers installed on the four corners of the diaphragm, as well as at quarter points along center. These accelerometers were used along with the distributed mass of the structure to determine the story shears. The instrumentation plan for the shear walls included the deployment of string potentiometers, linear potentiometers, strain gauges, and load cells. Linear potentiometers were added to capture wall deformation including sliding and rocking, and shear wall movement relative to the diaphragm. String potentiometers were deployed to capture relative inter-panel connection movement, and any panel shear deformation. Load cells were placed at the bottom of each tie down rod to measure the overturning force. These load cells were duplicated by strain gauges on each steel rod, which were effective if the rods did not yield. The deployment of instruments on the shear wall is summarized in Figure 4.

Table 1. Instrumentation on gravity frame and diaphragm

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

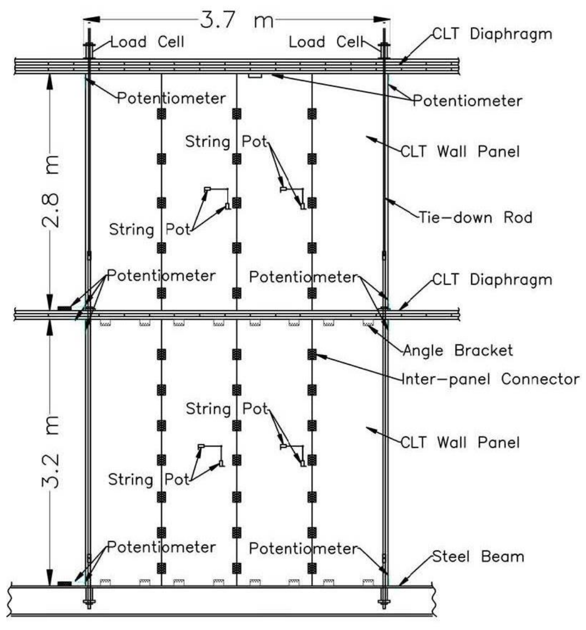

Figure 4: *Shear wall instrumentation deployment*

2.4 GROUND MOTION

The ground motion selected for the testing was the 1989 Loma Prieta earthquake record. This motion was scaled according to a location in San Francisco, CA to intensities corresponding to a Service Level Earthquake (SLE), Design Base Earthquake (DBE), and Maximum Considered Earthquake (MCE), each having a return period of 72 years, 474 years, and 2475 years respectively. Figure 5 shows the response spectra for the various intensities. The natural period of the building changed throughout testing as a result of damage accumulation and subsequent repairs to the structure, so white noise tests were conducted before and after each test in order to record the change in period. The natural period of the structure was 0.38 sec, 0.41 sec, and 0.45 sec for the SLE, DBE, and MCE, shakes resulting in spectral accelerations of 0.525 g, 0.92 g, and 1.36 g respectively.

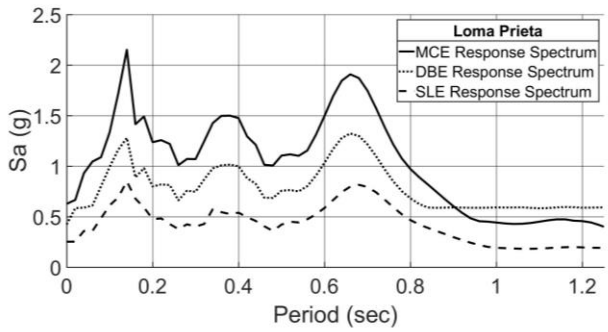

Figure 5: *Acceleration response spectra for SLE, DBE, and MCE motions*

3 RESULTS

3.1 DISPLACEMENT PROFILE AND INTERSTORY DRIFT

Figure 6 presents the displacement profile for the SLE, DBE, and MCE motions. The profile was constructed using the average displacement of the story relative to the ground, and then finding the peak displacement. Figures 7 and 8 present the interstory drift time histories for MCE test for the first and second stories respectively. The structure experienced primarily a first mode response, with the maximum displacement occurring simultaneously to the maximum interstory drift. The maximum displacement experienced by the structure was 158mm (6.22 in) occurring on the second floor during the MCE test. Comparing the displacements of the stories between the DBE and MCE level tests show an increase of more than 1.5 times. This is important to note because the MCE level intensity is scaled to 1.5 times the intensity of the DBE level, and since the max displacement for the MCE was larger than 1.5 times the DBE, it implies a nonlinear response from the structure which was evidenced by slight nail withdrawal in the base shear connectors.

The interstory drift time histories were constructed by comparing the difference in displacement time histories of each story, or in the case of the first story, comparing the displacement time histories to the table feedback data. The results were then averaged across their respective floors, and the peak was divided by the story height to obtain the peak interstory drift as a percentage of the height,3663 mm (144 in) and 3043 mm (120 in) for the first and second stories respectively. The largest interstory drift recorded for the MCE level tests was 2.7% on the first floor (Figure 7 presents the time history). Comparing the peak interstory drifts in Figure 7 and 8, it can be seen that there is a fairly significant difference between the two, with the first story being larger.

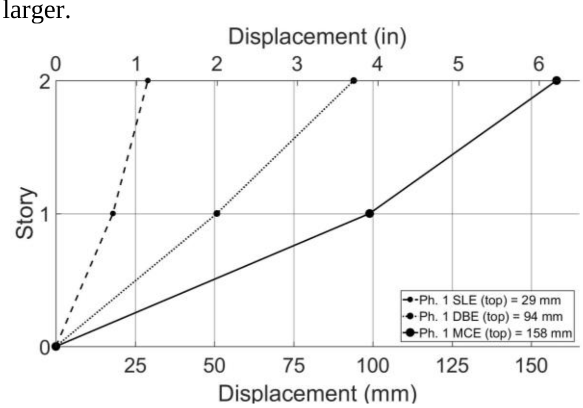

Figure 6: *Displacement profile for SLE, DBE, and MCE*

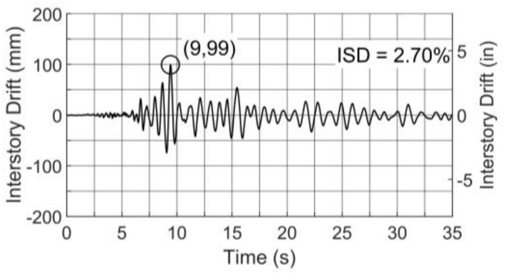

Figure 7: *MCE first story interstory drift*

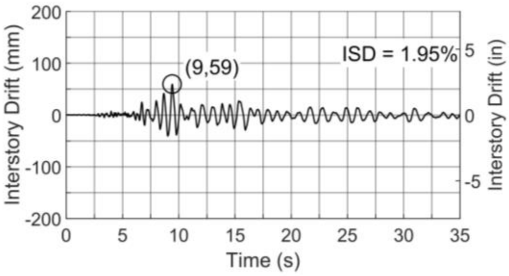

Figure 8: *MCE second story interstory drift*

3.2 GLOBAL HYSTERESIS

To calculate the inertial force of each story, the accelerometers located on each corner and along the center of the diaphragm on each story were utilized in combination with the appropriate building mass. The resulting story shear versus relative story displacement yielded the story hysteresis. Figures 9 and 10 present the story shear for the first and second stories respectively. The maximum base shear of 765 kN (171 kips) occurred during the MCE level test.

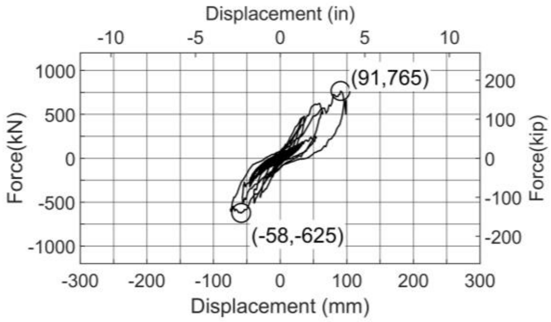

Figure 9: *First story shear versus story displacement*

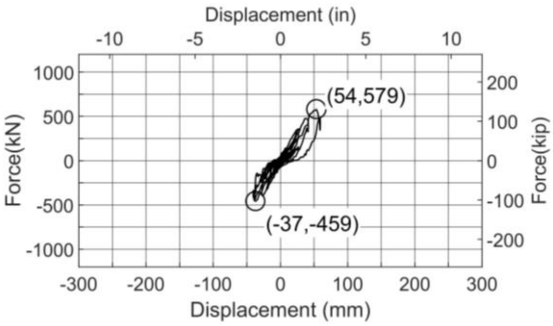

Figure 10: *Second story shear versus story displacement*

3.3 TORSION

As mentioned previously, the test specimen was part of a larger collaborative effort, and due to this, was subjected to over 30 tests, with some being larger than the MCE level test presented in this paper. The SFRS was replaced for this final stage, but because the diaphragms could not be replaced during the full test program, some diaphragm damage was present which may have resulted in torsion. Figure 11 shows the torsional response of the second story of the structure during the MCE test, and the torsion present, with the south end of the story displacing 35 mm (1.38 in) more than the center of the story. The north end of the story however, did not experience significant torsion, implying a variable stiffness across the specimen and like some damage to the splines within the diaphragm. The torsion for the first story and DBE and SLE level tests was not as pronounced, but still present. Despite the introduction of some torsion, which increased during the test program, the CLT shear wall stacks still performed well.

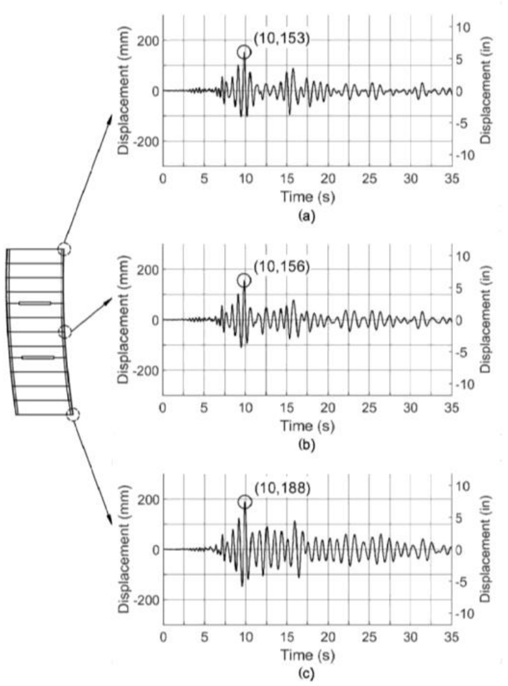

Figure 11: *MCE torsional response of the second story*

3.4 UPLFIT AND RELATIVE PANEL DISPLACEMENT

A rocking behavior was anticipated for the 4:1 aspect ratio walls, and to capture this, two measurements were recorded. The first measurement was the uplift of the CLT panels, which was recorded at the base and top of both end panels on the first and second stories. Figure 12 shows the uplift that occurred at the base of the west CLT panel on both the first and second stories. It is clear that larger uplift occurred on the base of the first story, which was expected. The top of the panels experienced much less uplift than the base in the case of both the first and second stories. Relative panel displacement was the second measurement recorded to capture the potential rocking in the CLT panels. This was recorded on two out of the three inter-panel joints on both the first and second floors, in both the vertical and horizontal direction, but the movement in the horizontal direction proved to be negligible. Figure 13 shows the maximum recorded vertical relative panel displacement for the MCE test on both the first and second stories. The relative panel displacement followed a similar pattern to the uplift with the first story displacement being the largest. The recorded uplift and relative panel displacements show that rocking was indeed present in the testing, and in fact was the controlling behaviour of the 4:1 aspect ratio shear walls. This was expected, and confirmed results from isolated shear wall testing.

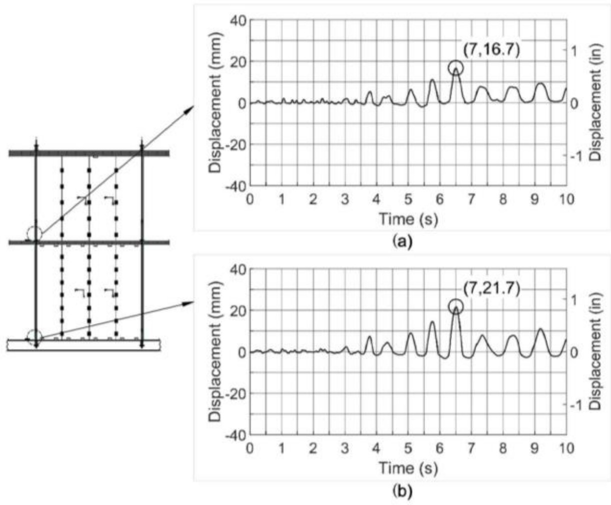

Figure 12: *MCE CLT panel base uplift*

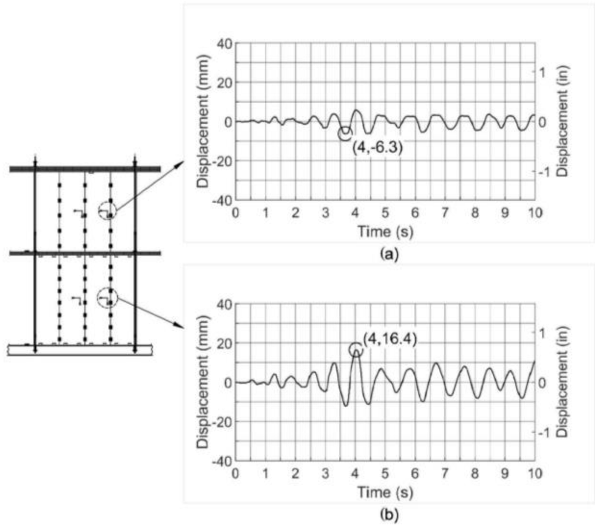

Figure 13: MCE *Vertical relative panel displacement*

3.5 SLIDING

Potential sliding behavior was captured using linear potentiometers located at the base and top of the shear walls on both and second stories, with the primary goal of capturing any sliding and comparing to captured rocking behaviour. Figure 14 presents the sliding recorded at the base of the first and second stories for the MCE level test. The sliding across both stories was relatively uniform, with sliding in the first story being slightly greater. The top of the wall experienced minimal sliding compared to the base. Overall, it is clear comparing Figure 14 to Figure 12 that rocking behavior controlled the deformation behaviour of the 4:1 aspect ratio panels.

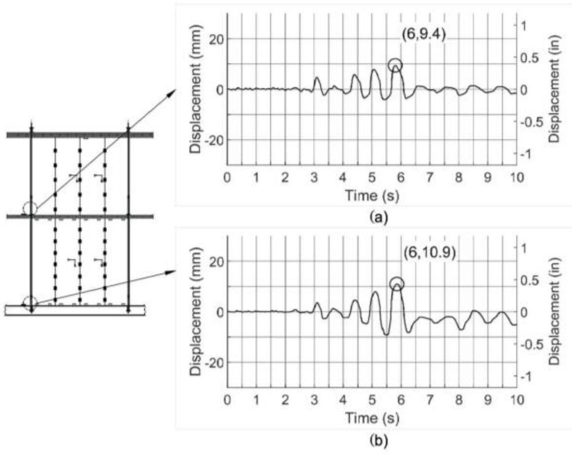

Figure 14: *MCE base sliding for first and second stories*

3.6 HOLD DOWN ROD FORCE

As mentioned previously, ATS tie-down rods were installed on either end of the shear wall stacks, with the design assumption that the tie-down rods would resist the overturning moment, and the CLT shear walls would resist shear and any compression. To facilitate this behaviour, the ATS rods were allowed to slip through at the base of the structure so they did not take any compression. Bearing plates were placed on the diaphragm of each story to provide the reaction for the rods in tension. Figure 15 shows the load cell data recorded at the base of the tie-down rods on the first story during the MCE level test. The largest force recorded was 166.9 kN (37 kips), which was large enough to likely yield the rod, although the structure still performed well. Yielding of the rods is not an issue at MCE level but should not occur at DBE intensity. It is also evident that the loading on the tie-down rods was asymmetric, across the story. This is most likely due to the presence of torsion, as mentioned earlier.

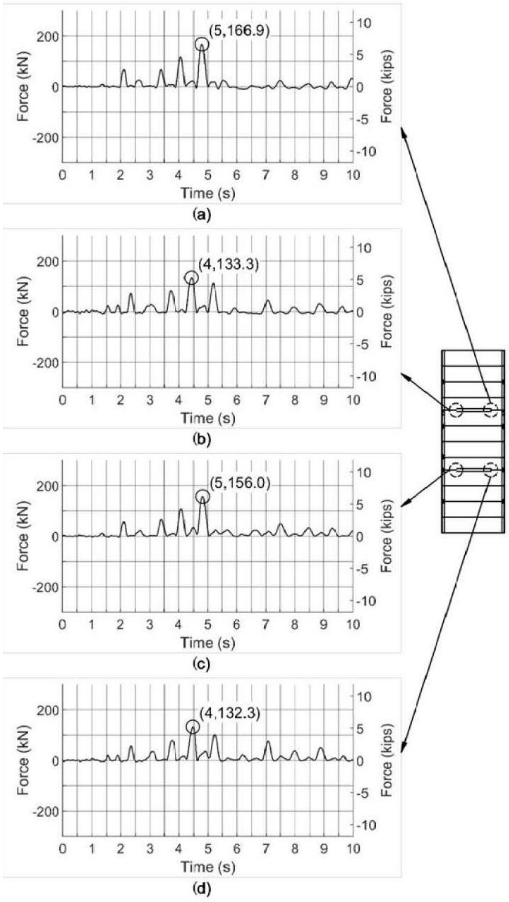

Figure 15: *MCE first story load cells*

4 CONCLUSIONS

In all phases and ground motion intensities, the structure performed well and in accordance with FEMA requirements for life safety. The displacements measured at the floor and roof diaphragms for Loma Prieta at MCE level can be seen in Figure 2 and Figure 3 respectively. Testing Phase 1 and 3 with 4:1 aspect ratio panels exhibited rocking behaviour, while Testing Phase 2 with 2:1 aspect ratio panels mainly showed sliding. The 2:1 aspect walls also demonstrated more nail pull out and deformation in the angle connectors. This testing demonstrated that both the 2:1 and 4:1 panel aspect ratios provide life safety, while also highlighting the differing characteristics of the two aspect ratios.

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