

Experimental Characterization of Cantilever Cross-Laminated Timber Diaphragms under In-Plane Shear Load

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Abstract: Diaphragms transfer the forces exerted by lateral loads to vertical elements through an in-plane shear action. Cross-laminated timber (CLT) has garnered considerable interest as a diaphragm material, particularly for midrise nonresidential construction. This article discusses the findings of tests performed on two full-scale 6.1 m (20 ft) × 6.1 m (20 ft) cantilever CLT diaphragms composed of three-ply CLT panels. The diaphragms were subjected to in-plane shear loads with a cyclic loading protocol. Two different layouts of CLT panels were considered for this study (simple span and two-span continuous panels). The displacement, force, or strain responses of various components of the diaphragms were measured and analyzed. The peak capacity of the diaphragms was governed by the fasteners used and exceeded the LRFD capacity by a factor close to two. Test results showed the primary energy dissipation mechanism for the diaphragms was via the slip of diaphragm components parallel and perpendicular to the direction of load and rotation of CLT panels, nearly all of which could be attributed to fastener deformations. Observation of strain data revealed deep-beam-like behavior of the diaphragms where the resistance to coupled tensile and compressive forces was primarily contributed by the outer portion of the edge panels, with little to no contribution from the inside panels. In the compression zone, a near-uniform compressive strain pattern was observed at the top and bottom ply, aligned along the major axis of the CLT panels. In the tension zone, the tensile strain was observed only in the top ply aligned along the major axis (where tension straps were installed), and compressive strain was observed in the bottom ply. The results of the strain data were used to suggest a design procedure for the tension and compression chords. DOI: 10.1061/JSENDH.STENG-12590. © 2023 American Society of Civil Engineers.

Introduction

Ever since its introduction to the North American market in the early 2000s, cross-laminated timber (CLT) has been gaining interest as a sustainable building material. Implementing CLT in buildings requires understanding a wide range of its characteristics, such as strength, serviceability, and fire performance. In the US, considerable efforts have been made toward standardization of the fabrication, testing, and application of CLT products (Breneman et al. 2016). In 2010, the Engineered Wood Association (APA) and FPInnovations in Canada initiated the development of a product standard for CLT. The American National Standards Institute (ANSI)/APA PRG 320-11 (APA 2011), which provides product performance classes as well as requirements and test methods for quality assurance of performance-rated CLT products, was completed

in December 2011 (Yeh et al. 2012). In 2011, FPInnovations published the Canadian edition of the Gagnon and Pirvu (2011) to provide guidance on the design and construction of CLT systems, especially in cases where technical information on such design was lacking in the building codes. The broad acceptance of this document encouraged a multiteam collaboration to develop the US edition of the *CLT Handbook* in 2013 (Karacabeyli et al. 2013).

CLT was introduced as a structural product in the 2015 edition of both the National Design Specification (NDS) for Wood Construction (American Wood Council 2015) and later in the 2020 edition of the International Building Code (IBC) (ICC 2020). Both codes refer to CLT products manufactured in accordance with ANSI/APA PRG-320 (APA 2019). The 2015 NDS designated Chapter 10 to CLT but only provided guidance for the gravity load design of CLT. The 2015 version of the International Building Code (IBC 2015) allowed for the use of CLT as the structural element in exterior walls, floors, roofs, and interior walls of Type IV construction. In 2021, with the development of mass timber research, both IBC and the American Wood Council (2021) underwent several updates. In the 2021 edition of the IBC (ICC 2020), three new construction types (IV-A, IV-B, and IV-C) have been introduced that allow for taller heights, more stories, and greater allowable areas of mass timber buildings compared with limits set by the 2018 IBC. Currently, Section 504.4 of the 2021 IBC provisions allows for up to 18 stories of type IV-A construction for Business and Residential occupancy classes. The 2021 edition of SDPWS includes basic design provisions for CLT shear walls (Chapter 4.6), and its Appendix B provides requirements for CLT shear wall design. It also provides provisions for the design of CLT diaphragms (Chapter 4.5), including requirements for the design of shear and tension elements, which were developed based on diaphragm tests conducted by Kode et al. (2021).

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Several CLT building projects have been undertaken and completed in North America, and several others are under development by 2021. Many of these projects were carried out while CLT research was still in nascent stage and code guidance was limited in certain areas such as CLT diaphragm design. These projects also required a special permit from the local jurisdiction, and the design was supported using project-specific testing and analysis programs (Pei et al. 2016).

Several researchers have undertaken a numerical and experimental approach to quantify the behavior of CLT diaphragms under in-plane shear load. Ashtari (2012) used calibrated models based on connection tests to develop a two-dimensional (2D) numerical model to quantify the stiffness of CLT diaphragms. Kode et al. (2021) tested several 5 m (16 ft) $\times$ 4.27 m (14 ft) cantilever CLT diaphragms using a displacement-controlled cyclic loading protocol with the goal of understanding load deformation and failure behavior of CLT diaphragms under lateral load. The tests showed that it was essential to have high-strength panel-to-panel tension ties to enable the development of the full design strength of the diaphragm shear connectors and that the CLT panels acted rigid, with the ductility of the fastener providing the major energy dissipation mechanism.

Line et al. (2022) also conducted two 7.3 m (24 ft) $\times$ 7.3 m (24 ft) full-scale CLT diaphragm tests using simply supported boundary conditions with the goal of understanding CLT diaphragm resistance to lateral (cyclic or monotonic) forces. The design was carried out using guidelines provided in the 2021 SDPWS. The test results showed that the diaphragm exceeded the design load capacity, the load-deformation behavior of the diaphragms was dominated by ductile connection slip, and the panels acted as rigid bodies. Additionally, tests on cantilever CLT diaphragms by Beirsto et al. (2022) showed that the average yield strength for cyclic and monotonic diaphragm tests was 0.96 and 1.47 times the design yield capacity, respectively. The actual deflection of the diaphragms was found to be higher than that predicted by equations from the design example by Spickler et al. (2015). Furthermore, the in-plane stiffness

of the diaphragms was determined to be governed by the apparent shear stiffness of the assembly.

Despite recent developments in CLT research, the code guidance for designing CLT diaphragms under lateral loads is limited and needs more supporting information from experiments compared with the current provisions for the design of light-frame wood diaphragm (Sullivan et al. 2018). For this study, an in-plane cantilever diaphragm setup was considered. Such a setup is typical in mass timber buildings with a central shear core. This paper presents the design, experimental results, and observations of two full-scale 6.1 m (20 ft) $\times$ 6.1 m (20 ft) cantilever CLT-glulam composite diaphragms tested up to failure under the Consortium of Universities for Research in Earthquake Engineering (CUREE) cyclic protocol.

Experimental Design

Description of CLT Diaphragm Test Setups

The test diaphragms were two 6.1 m (20 ft) $\times$ 6.1 m (20 ft) CLT-glulam composite diaphragms made up of three-ply V3 grade Southern Pine CLT panels. The first diaphragm consisted of eight 1.5 m (5 ft) $\times$ 3 m (10 ft) panels (Bhardwaj et al. 2021), and the second consisted of four 1.5 m (5 ft) $\times$ 6.1 m (20 ft) panels as shown in Fig. 1. The panels of both diaphragms were supported underneath along the 1.5 m (5 ft) direction by three 6.1 m (20 ft) 20F-V4 Southern Pine glulam beams. Although the locations of the glulam beams were similar in both the test configurations, due to the difference in the length of panels used, the first diaphragm had a single-span CLT layout whereas the second diaphragm had a two-span continuous layout.

In the first diaphragm, the glulam beams were 27.9 cm (11 in.) deep. The outer glulam beams were 7.9 cm (3.125 in.) wide, and the center glulam was 15.2 cm (6 in.) wide. For the second diaphragm, all glulam beams were 17.1 cm (6.75 in.) wide and

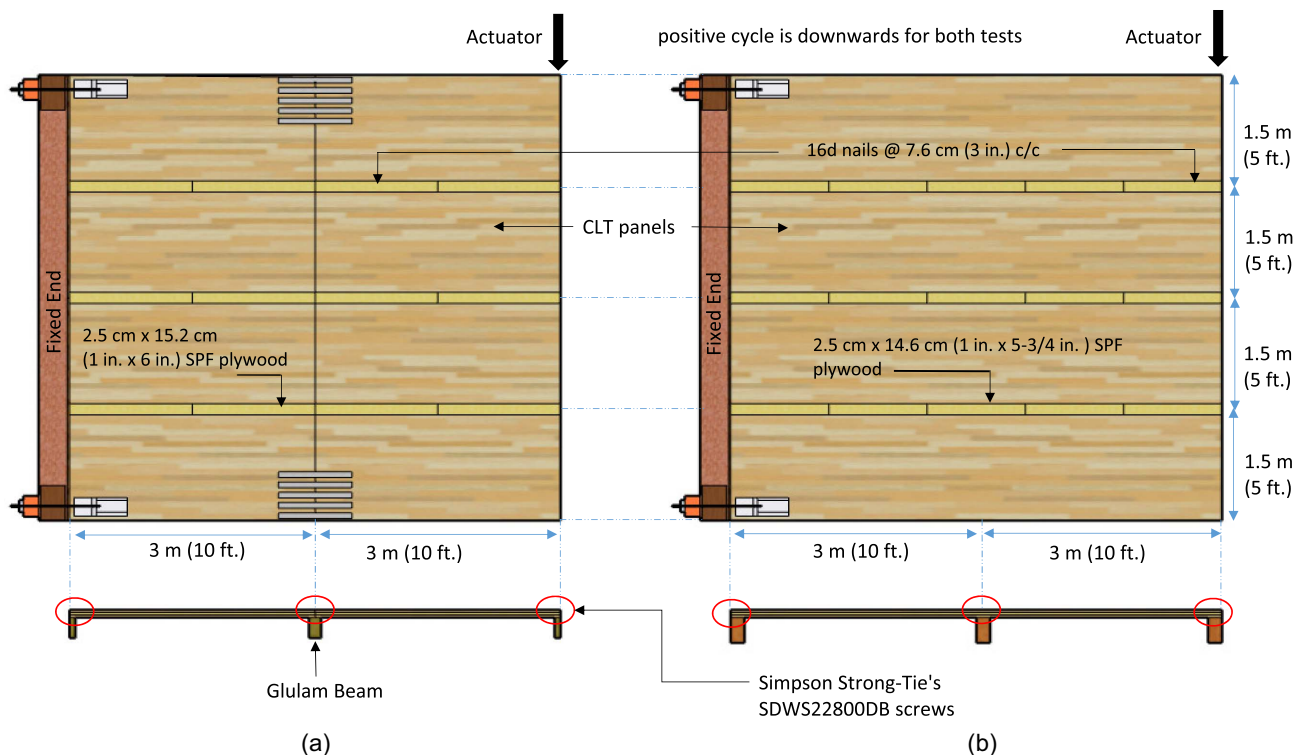


Fig. 1. Overall layout for (a) first; and (b) second diaphragm test setup.

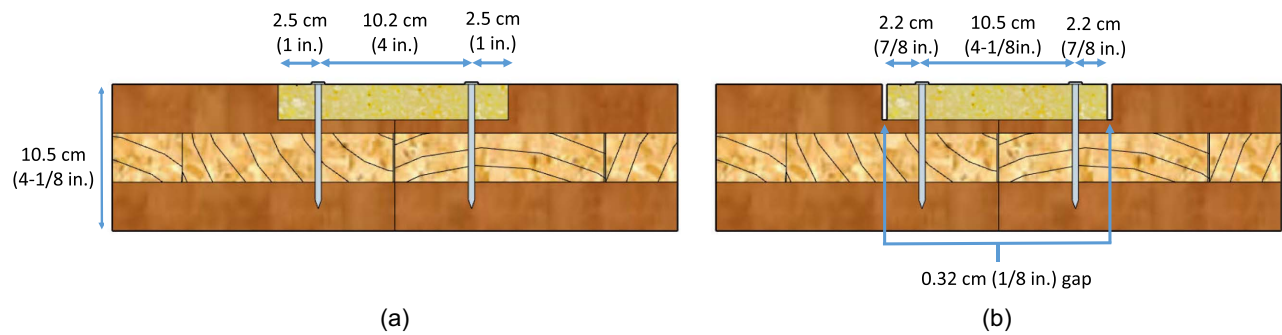


Fig. 2. Surface spline detail for (a) first; and (b) second diaphragm test.

34.9 cm (13.75 in.) deep. Both diaphragms cantilevered (in-plane direction) from a fixed support. Two rows of 12.7 mm (0.5 in.) lag screws spaced 15.2 cm (6 in.) on the center were used to fasten the edge glulam to the fixed support. Self-tapping wood screws [Simpson Strong-Tie (Pleasanton, California) SDWS22800DB] at 15.2 cm (6 in.) on-center were used to fasten CLT panels to the glulam beams, except for the connection at the central glulam for the second test, where 20.3 cm (8 in.) spacing was used. The longitudinal edges of the CLT panels were connected using 2.7 cm (1.0625 in.) thick Spruce Pine-Fir plywood surface splines with 16d nails at 7.62 cm (3 in.) on the center. The plywood for the surface spline was cut to the exact size as the routed section in CLT for the first test. This exact fit led to bearing contact between the spline and adjacent CLT during the test. Thus, the plywood spline was undersized such that a 0.31 cm (0.125 in.) gap existed between the spline and CLT on both sides for the second diaphragm test, as shown in Fig. 2.

Because the first diaphragm had a seam at the center due to the layout of its panels, Simpson Strong-Tie's MSTC40 straps were used to transfer tension forces to aligned CLT panels. Ten tension straps were used in total, five on each side of the diaphragm. The tension straps were evenly spaced within a distance of 69.8 cm (27.5 in.) from the edge of the diaphragm (d_c). For both diaphragms, special custom-made hold-downs designed by Simpson Strong-Tie were used as tension members at the fixed end. The hold-downs were placed such that their centerline was located 25.4 cm (10 in.) from the edge of the diaphragm. The hold-downs were connected to the CLT with 52 lag screws of size 12.7 mm (0.5 in.) $\times$ 88.9 mm (3.5 in.) and to the external frame through concrete-filled hollow steel sections (HSS) with threaded rods. Casters were installed beneath the middle and free-end glulam beams to support the diaphragm vertically. The in-plane shear load was applied using a 667-kN (150-kip) actuator via a channel section connected to the free-end glulam by two rows of lag screws sized 12.7 mm (0.5 in.) spaced at 15.2 cm (6 in.) on center.

Different setups were used on the two diaphragms to prevent excess out-of-plane deformation (i.e., due to buckling). For the first diaphragm, a set of casters hanging from a rigid support were installed near the free end of the diaphragm. For the second diaphragm, 12.7-mm (0.5-in.) holes were drilled at the center of each panel near the free end to hook each panel with inverted I-beam trolleys. The trolleys were allowed to slide laterally with the diaphragm but restrained any upward movement of the panels.

Design of Diaphragm Components

The LRFD strengths of diaphragm components were determined by referencing the procedure outlined in a CLT diaphragm design whitepaper (Spickler et al. 2015), code provisions in the SDPWS

(American Wood Council 2021) and the NDS (American Wood Council 2018). The diaphragm design strength was controlled by the shear capacity of the surface spline nailed connections, 98.3 kN (22.1 kip) or 16.1 kN/m (1,105 plf). The aspect ratio of the diaphragm and the CLT panels governed the forces determined at various components. The CLT-to-glulam shear fasteners (Simpson Strong-Tie's SDWS22800DB screws) that were perpendicular to the surface spline nailed connection were designed to have an additional capacity compared with the design forces at the CLT–glulam interface. The design capacity of the CLT–glulam screw connection came out to be 61% higher than the surface spline nailed connection. All other diaphragm components, such as tension straps, hold-downs, and tension and compression chords, were designed to be at least two times stronger than the design forces. A summary of LRFD strength of individual diaphragm components is given in Table 1.

Instrumentation Setup

Strain gauges, string pots, and load cells were used at key locations to record the response of diaphragm components. Twenty-seven string pots were installed for the first diaphragm test, and 35 were used for the second diaphragm test (Fig. 3). Various displacements, such as interpanel slip, panel–glulam slip, hold-down uplift, deformation of the fixed-end support steel beam, total diaphragm displacement, and so on, were tracked using the string pots. The location of the string pots used in the two tests is shown in Fig. 3. Those string pots installed for the same purposes and in the same locations of both diaphragms were given the same numbering. Additional string pots installed for the second diaphragm were given unique numbers. Two compression load cells were used to record the tension forces in the

Table 1. LRFD strength of diaphragm components

Component	Required capacity ^a [kN (kip)]	Design capacity [kN (kip)]	Strength ratio ^b
Panel–panel nailed surface spline ^c	98.3 (22.1)	98.3 (22.1)	1:1
Panel–glulam screws ^c	98.3 (22.1)	158.4 (35.6)	1.6:1
Tension straps ^d	111.2 (25)	126.9 (28.5)	2.3:1
Tension chord ^c	222.4 (50)	323.8 (72.8)	2.9:1
Compression chord ^c	222.4 (50)	327.4 (73.6)	2.9:1
Hold-downs ^c	222.4 (50)	560.5 (126)	5:1

Note: The design capacity of 98.3 kN (22.1 kip) corresponds to a unit shear capacity of 16.1 kN/m (1,105 plf).

^aRequired capacity for nonshear elements is twice the capacity obtained using design shear forces.

^bRatio of design capacity to capacity obtained using design shear forces.

^cFor both diaphragms.

^dFor first diaphragm only.

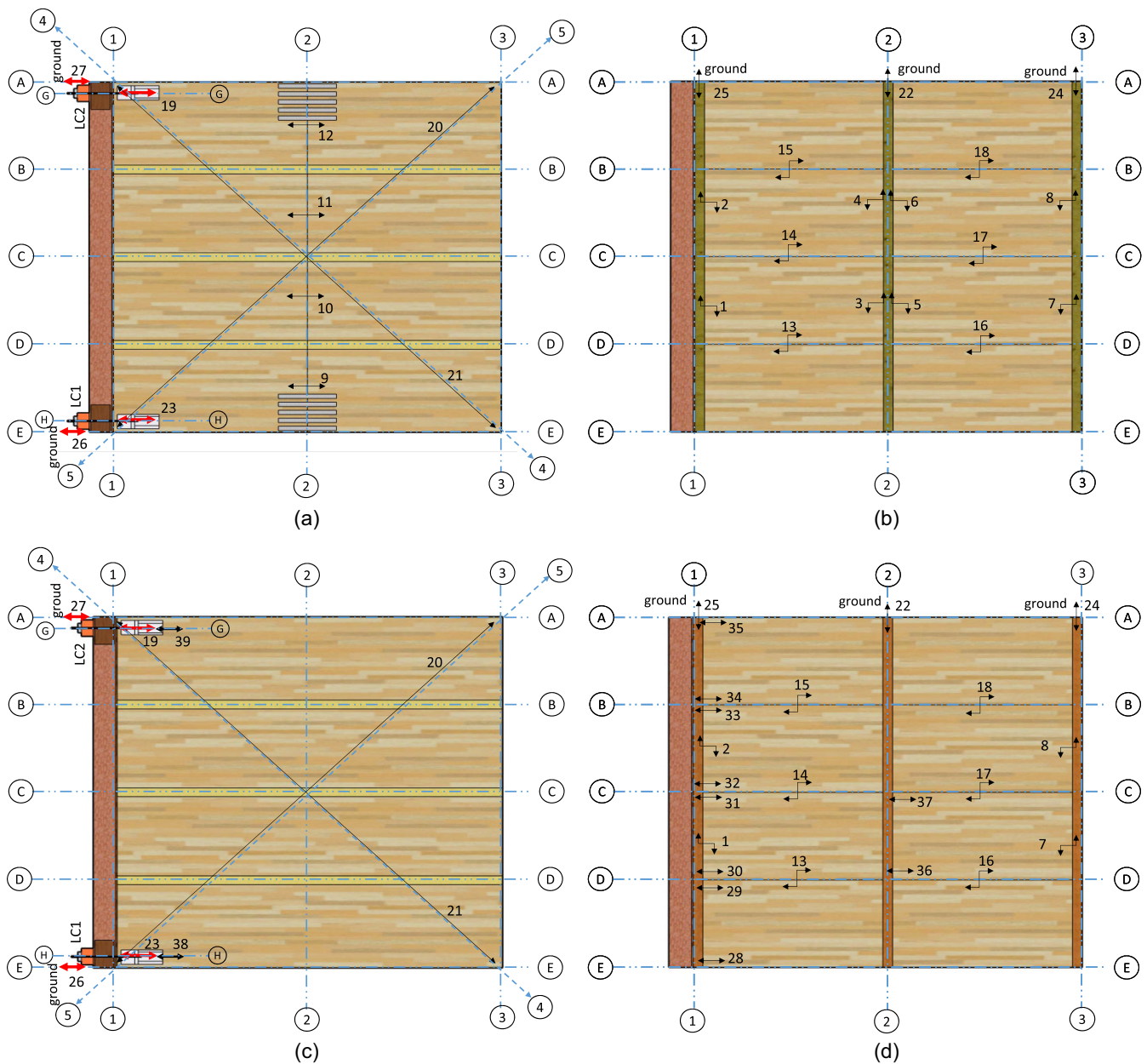


Fig. 3. String pot layouts for (a) top of first diaphragm test; (b) bottom of first diaphragm test; (c) top of second diaphragm test; and (d) bottom of second diaphragm test.

steel rod of the hold-downs, as shown in Figs. 3(a and c) (marked as LC1 and LC2).

Forty strain gauges were used for each diaphragm test setup. For the first diaphragm test, 16 pairs of strain gauges were placed on the top and bottom surfaces of the CLT panels. This was done to observe the variation of strain across the width of the diaphragm (i.e., parallel to the direction of the load) and through the thickness of the CLT panels [Fig. 4(a)]. Eight strain gauges were placed on the tension straps, four on each edge of the diaphragm. For the second diaphragm test, 20 pairs of strain gauges were placed on the top and bottom of CLT panels [Fig. 4(b)]. The strain gauges for the second test had the same layout as the first test for the outer two panels, and two additional strain gauges were installed on each of the inside panels to better capture the strain variation of the inside panels. To avoid the influence of local defects in wood surface (e.g., knots) on strain, the strain gauges were glued on top of

0.13 mm (0.005 in.) metal shims, which were glued to the CLT panels as shown in Fig. 4(b). Such method for strain gauges installation was based on previous research by Phares et al. (2019).

Loading Protocol

The displacement-controlled CUREE cyclic loading protocol (Krawinkler et al. 2001) was used for both tests. To come up with a suitable Δ_{ref} value for the first diaphragm test, tests performed at Colorado State University (Kode et al. 2021) along with results from a numerical model in Timber3D version 1.7, a MATLAB-based program (Pang and Hassanzadeh Shirazi 2013), were analyzed. Based on these studies, a Δ_{ref} value of 10.2 cm (4 in.) was deemed suitable for the diaphragm test. The results from the first diaphragm test confirmed that the Δ_{ref} of 10.2 cm (4 in.) would be suitable for the second test as well.

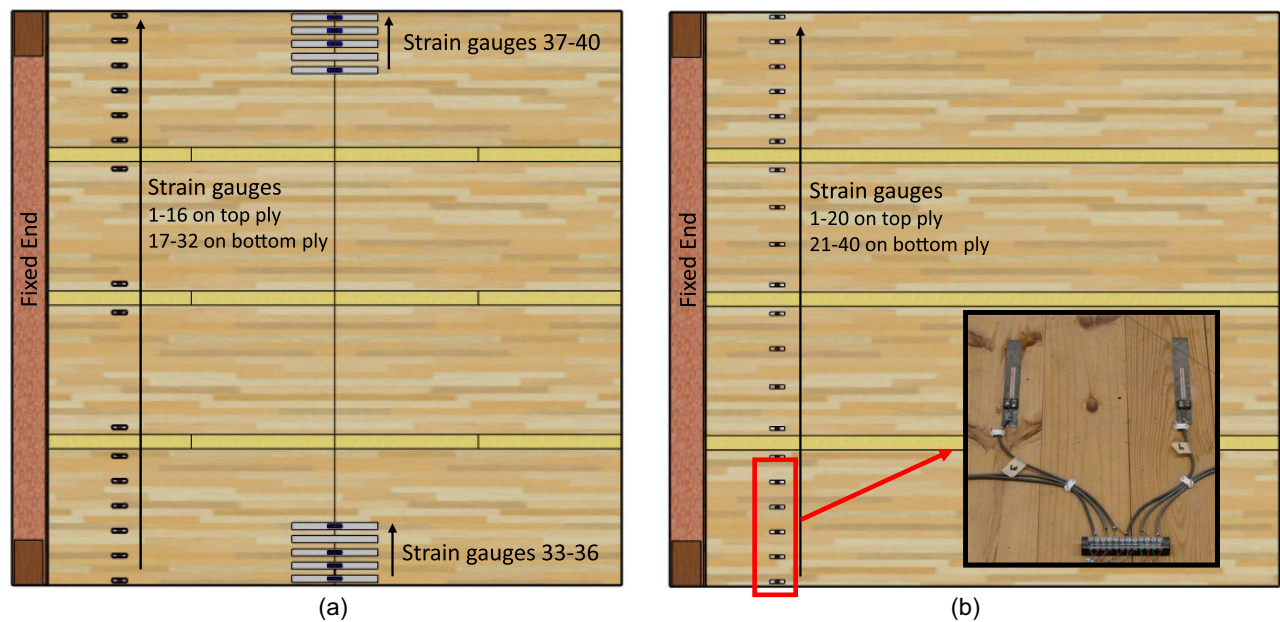


Fig. 4. Strain gauge layouts for (a) first diaphragm test; and (b) second diaphragm test (with callout showing installation of strain gauges on top of metal shims).

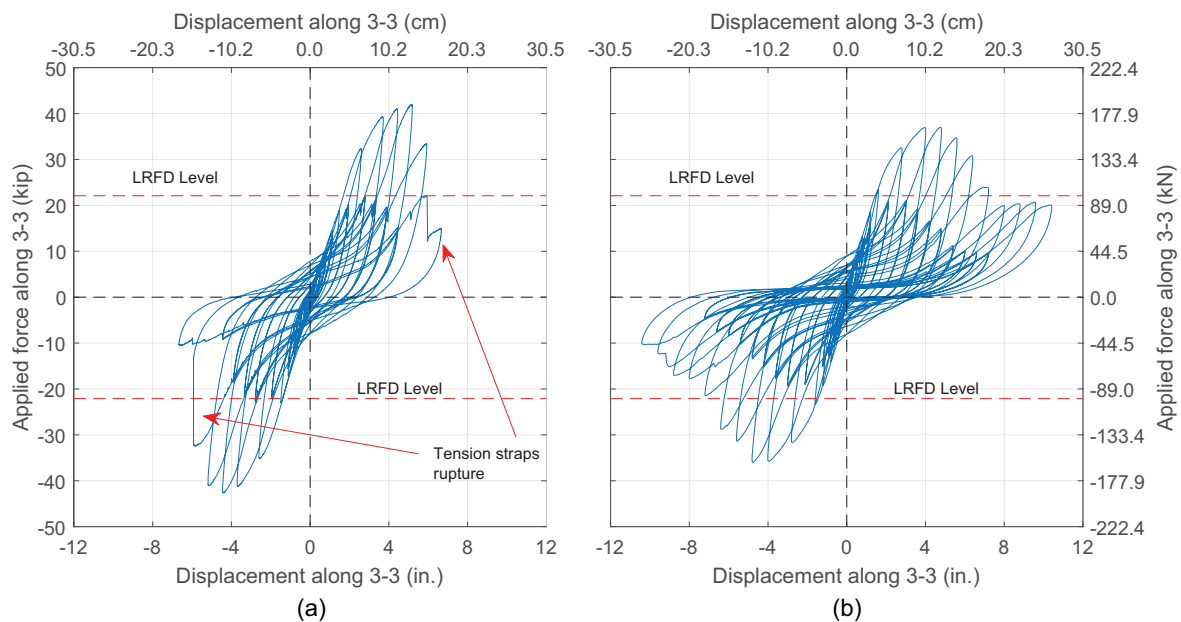


Fig. 5. Load-displacement plot for (a) first; and (b) second diaphragm test.

Experimental Observations and Results

Load versus Displacement Curve

Fig. 5 shows the load versus displacement curve for the two diaphragm tests conducted. Both diaphragms showed nearly symmetrical behavior for positive and negative displacement cycles (Fig. 1). The second diaphragm exhibited slightly lower capacity than the first diaphragm, which was not expected because it was composed of larger CLT panels [four 1.5×6.1 m (5×20 -ft) panels] when compared with the first diaphragm [eight 1.5×3 m (5×10 -ft) panels]. Some possible reasons for this are discussed in the

“Deformation along the CLT–CLT Surface Spline Connections” section. The peak capacity of both diaphragms exceeded the LRFD capacity by a factor close to two. A summary of the diaphragm test results is given in Table 2.

Both diaphragms displayed similar prepeak behavior with similar stiffness. The average secant stiffness (computed as the point from the origin to a point on the backbone curve at LRFD design load and averaged for positive and negative cycles) for the first diaphragm test setup was 25.2 kN/cm (14.4 kip/in.), and that for the second diaphragm test setup was 26.6 kN/cm (15.2 kip/in.). However, the postpeak behavior varied greatly. The first diaphragm underwent rapid loss of strength due to the rupture of all the tension straps

Table 2. Summary of the two diaphragm tests

Parameter	Diaphragm 1	Diaphragm 2
Design LRFD load [kN (kip)]	98.3 (22.1)	98.3 (22.1)
Peak positive load [kN (kip)]	187 (42)	164.5 (37)
Peak positive displacement [cm (in.)]	13.1 (5.16)	11.4 (4.47)
Peak negative load [kN (kip)]	189.5 (42.6)	160 (36)
Peak negative displacement [cm (in.)]	11.2 (4.42)	11.2 (4.41)
Peak/LRFD strength	1.9	1.7

at large deformations (approximately 15.2 cm/6 in. displacement for both positive and negative cycles) [Fig. 5(a)]. A decision was made to stop the test once the straps failed because the diaphragm had less than 25% of residual capacity remaining [at around 15.2 cm (6 in.) displacement]. For the second test, the diaphragm had around 50% of residual capacity left at approximately 20.3 cm

(8 in.) of displacement. Despite significant damage to many fasteners, the second diaphragm still retained, on average, close to 40% of its peak strength at 25.4 cm (10 in.) of displacement.

Strain Variation across Diaphragm and CLT Ply

Fig. 6 shows the prepeak strain variation on the top and bottom surfaces of CLT panels in the direction perpendicular to the load (Fig. 4 shows the orientation of strain gauges) for the two diaphragms at various load levels for both positive and negative cycles. Most of the strain generated was concentrated on the outer panels, with little to no reading on the inner panels. Furthermore, the strain reading rapidly dropped from the extreme fibers in either tension or compression to near zero at the first interpanel boundary (i.e., spline connection). The recorded strain profiles in the diaphragms are similar to the normal strain profiles of deep beams.

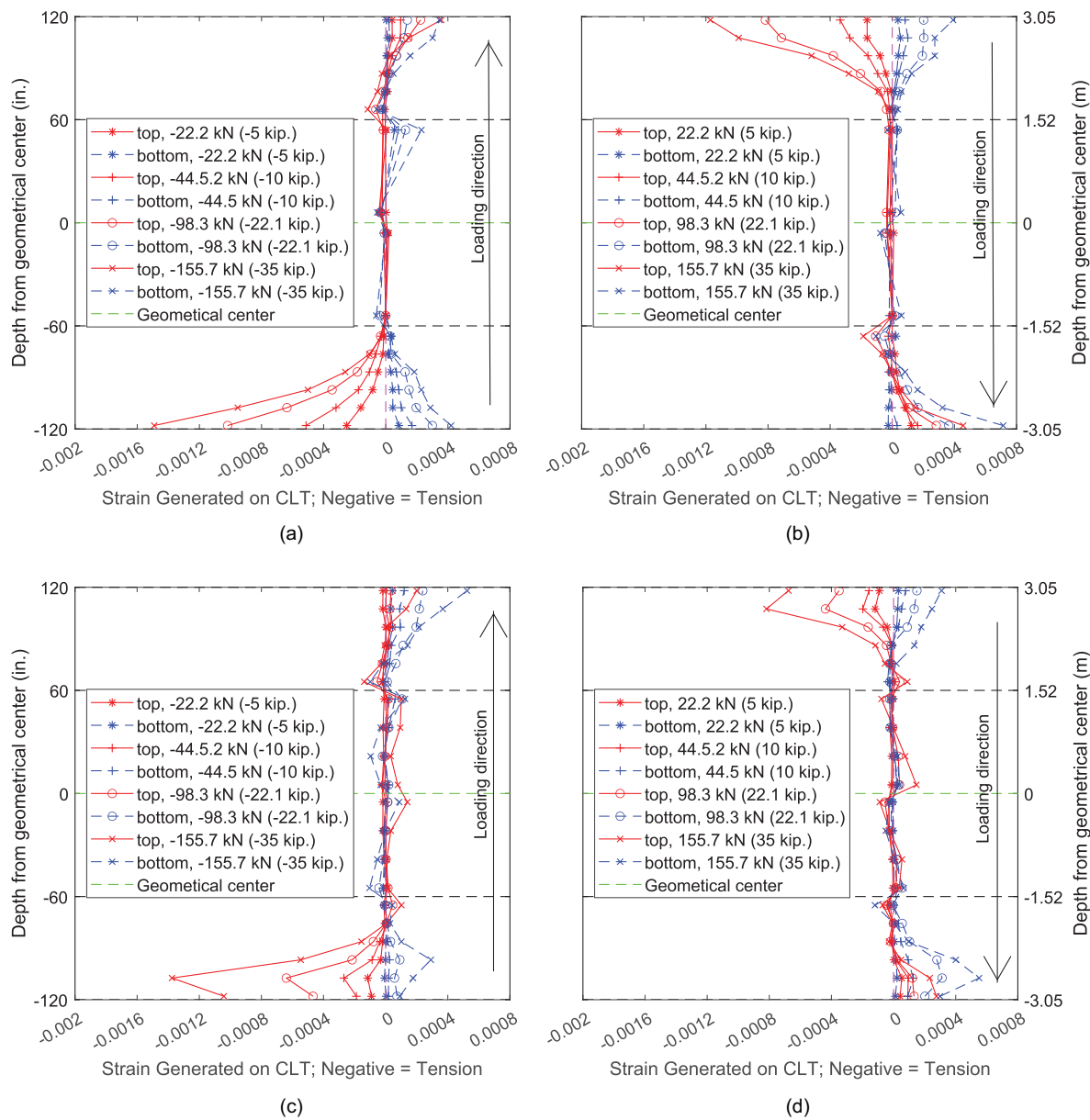


Fig. 6. Prepeak strain variation across CLT diaphragm depth for (a) first diaphragm test, negative cycle; (b) first diaphragm test, positive cycle; (c) second diaphragm test, negative cycle; and (d) second diaphragm test, positive cycle.

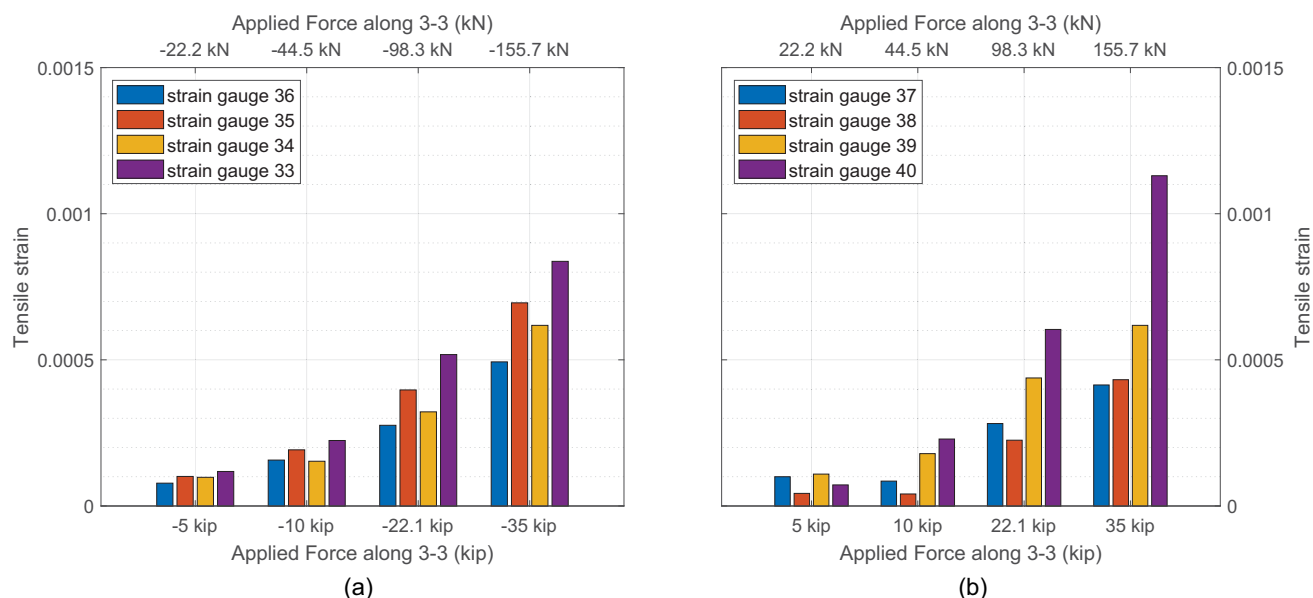


Fig. 7. Variation of strain in tension straps for the first diaphragm test for (a) negative cycle; and (b) positive cycle.

On the compression side, the top and bottom CLT plies displayed near-uniform strain. The compressive strains were generated because of the CLT end panel bearing on the rigid support. It is possible that the top and bottom layers of CLT panels had a non-uniform bearing on the rigid support, which could be one of the reasons behind the minor differences in compressive strain values between the two layers.

On the tension side, however, only the top layer developed tensile strain, whereas the bottom layer developed compressive strain. The hold-down and tension straps were installed on the top layer. The fasteners used in the hold-down were fastened on the top of CLT panels, penetrating through the first two plies and went 6.3 mm (0.25 in.) into the third (bottom) ply, and the fasteners installed on the tension straps went 26.9 mm (1.0625 in.) into the middle ply (i.e., no penetration into the third/bottom ply). Consequently, only the top ply developed tensile strain. The bottom ply provided resistance against the elongation due to the tension of the top ply, which resulted in the upward bending of the CLT panel, causing compression to develop. The magnitude of the compressive strain generated on the tension zone was smaller compared with the tensile strain developed at all force levels. This observation shows that only the ply with full penetration by fasteners (top ply in this case) developed tension strain.

The strain variation on the tension straps (which were present only on the first diaphragm setup) can be observed in Fig. 7. The strain values in both the tension and compression zones were highest on the outside tension straps and gradually decreased toward the interior tension straps in a similar fashion to that in the CLT panels.

Deformation along the CLT–CLT Surface Spline Connections

The variation in nail slip values for different load levels for both diaphragms is shown in Figs. 8 and 9. For the first diaphragm test, the hysteresis loops for nail slip along the longitudinal panel boundaries lying between Gridlines 1 and 2 (Fig. 3 shows the grid lines) are consistent with each other. The tension within this region is resisted by the hold-downs. A similar case exists for those between vertical Gridlines 2 and 3 where the tension is resisted by the tension straps. This suggests that interpanel slips along panels sharing

a common tension-resisting member are near uniform, provided all interpanel connections have same force-displacement response. However, the nail slip values between Gridlines 1 and 2 are several orders of magnitude smaller than those observed between Gridlines 2 and 3, suggesting that the hold-downs were acting stiffer than tension straps.

For the second diaphragm, the slip observed between the inter-panel splines between Gridlines 1 and 2 was found to be identical with the slip observed between Gridlines 2 and 3. This similarity in slip occurred because the CLT panels for the second diaphragm were spanning from Gridline 1 to Gridline 3 in a two-span continuous support condition.

Deformation along the CLT–Glulam Screw Connections

Fig. 10 shows the variation in screw slip with applied load at the CLT–glulam shear interface connected through Simpson Strong Tie SDWS22800DB screws. The deformation in screws increased substantially from Grid 1-1 to Grid 3-3 (i.e., from fixed end to free end). The differences in deformation in the screws could be attributed to the cumulative contribution of other components of the diaphragm to the shear resistance. For a slip between CLT and glulam along Grid 3-3 to occur, the load applied must overcome the friction force between CLT and glulam surface, the friction force between glulam and the floor (glulam was supported vertically by casters rolling on the floor), and the expected resistance of the screw connection. Along Grids 1-1 and 2-2, the shear forces were resisted by CLT–glulam screws and a combination of friction along with a minor contribution to shear resistance by the tension straps and hold-downs. Thus, the screw interfaces closer to the fixed ends (i.e., Grids 1-1 and 2-2) were subjected to slightly smaller forces than those near the free end (Grid 3-3). Over many loading cycles, this minor difference in force subjected could be sufficient to impart failure on the free-end screws while keeping the fixed-end screws relatively intact.

Performance of Hold-Downs

Fig. 11 shows the hold-down force versus the diaphragm free-end displacement along Grid 3-3. For forces below the peak load, the

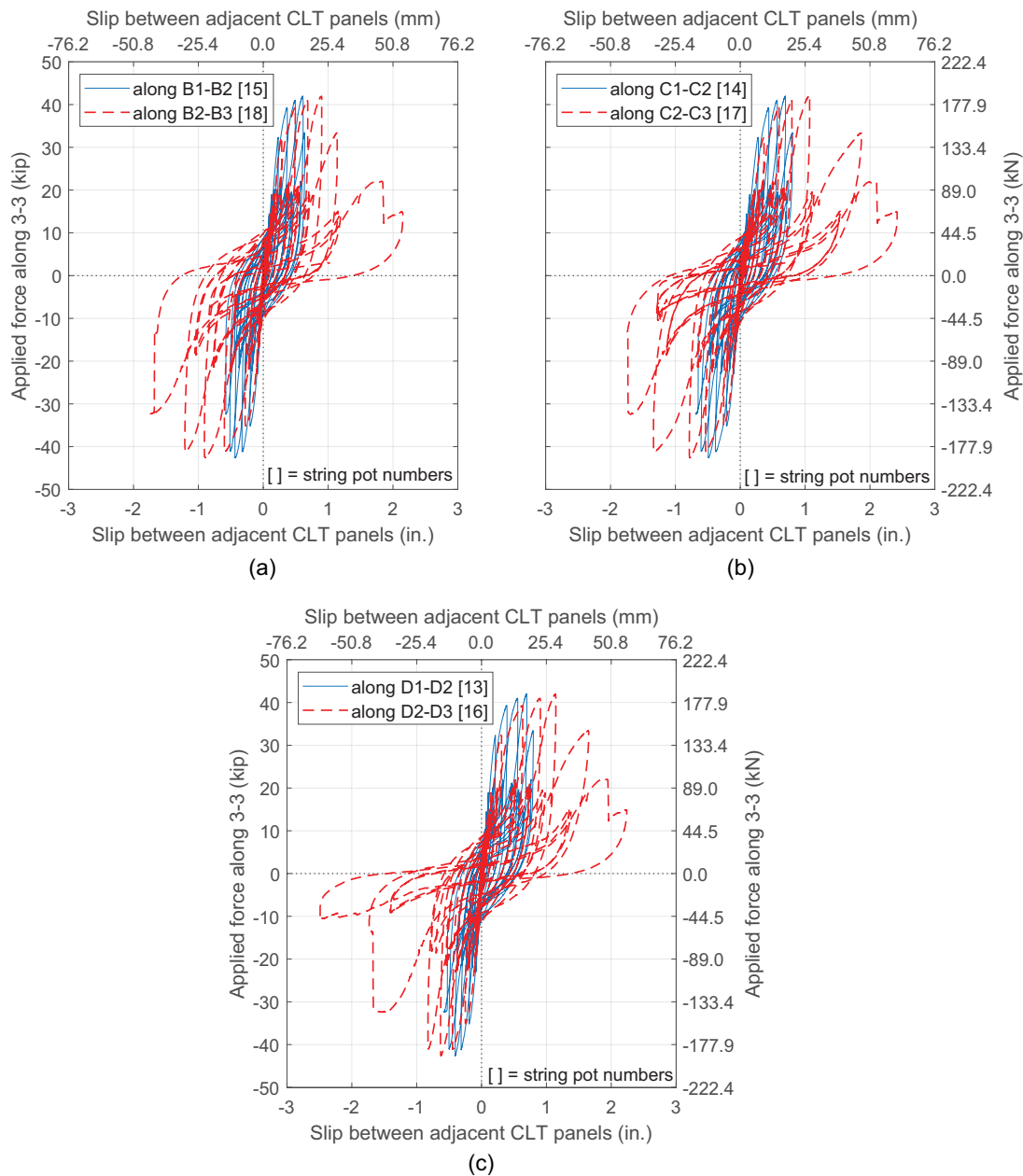


Fig. 8. Slip in nailed surface spline for first diaphragm test.

hold-down force readings measured by the load were slightly smaller in magnitude than the applied force for both tests. This behavior could be attributed primarily to contribution of the CLT-glulam along Grid 3-3 screws to resist the rocking action and the various frictional force existing within the system. For the hold-downs on the first diaphragm test, the force reading dropped as the applied force in the diaphragm dropped below the peak load. Test result suggests the diaphragm peak capacity was governed by the nailed surface splines and tension straps between Grids 2-2 and 3-3. As the tension straps ruptured, the readings in the hold-downs also dropped due to the release of tension forces. On the other hand, the load cell readings in the second diaphragm test continued to increase beyond the peak load of the diaphragm test, primarily due to the absence of a joint in CLT panels along Grid 2-2 [1.5 m (5 ft) $\times$ 6.1 m (20 ft) panels were used in the second diaphragm].

Fig. 12 shows a typical strain profile for the postpeak region of the second diaphragm test. Once the surface splines failed, rocking

of individual CLT panels were observed in contrast to rigid body rotation of the entire diaphragm assemblies as a unit (Fig. 13). The compressive strain values on the inside panels increased drastically suggesting individual bearing of the CLT panels rather than a relatively rigid-body behavior in the cycles leading to peak force with compressive strain concentrated at the extreme edge of the diaphragm, as suggested by the strain diagrams in Fig. 6. This deformation pattern provided new load path for the compression force couple, thereby decreasing the moment arm for the tension-compression couple responsible for resisting generated forces (Fig. 13). Thus, higher hold-down force was required to maintain equilibrium with the applied force.

Deformation along Diagonals

Fig. 14 shows the deformation along the two diagonals of the diaphragms with respect to applied force. All the fastener deformation

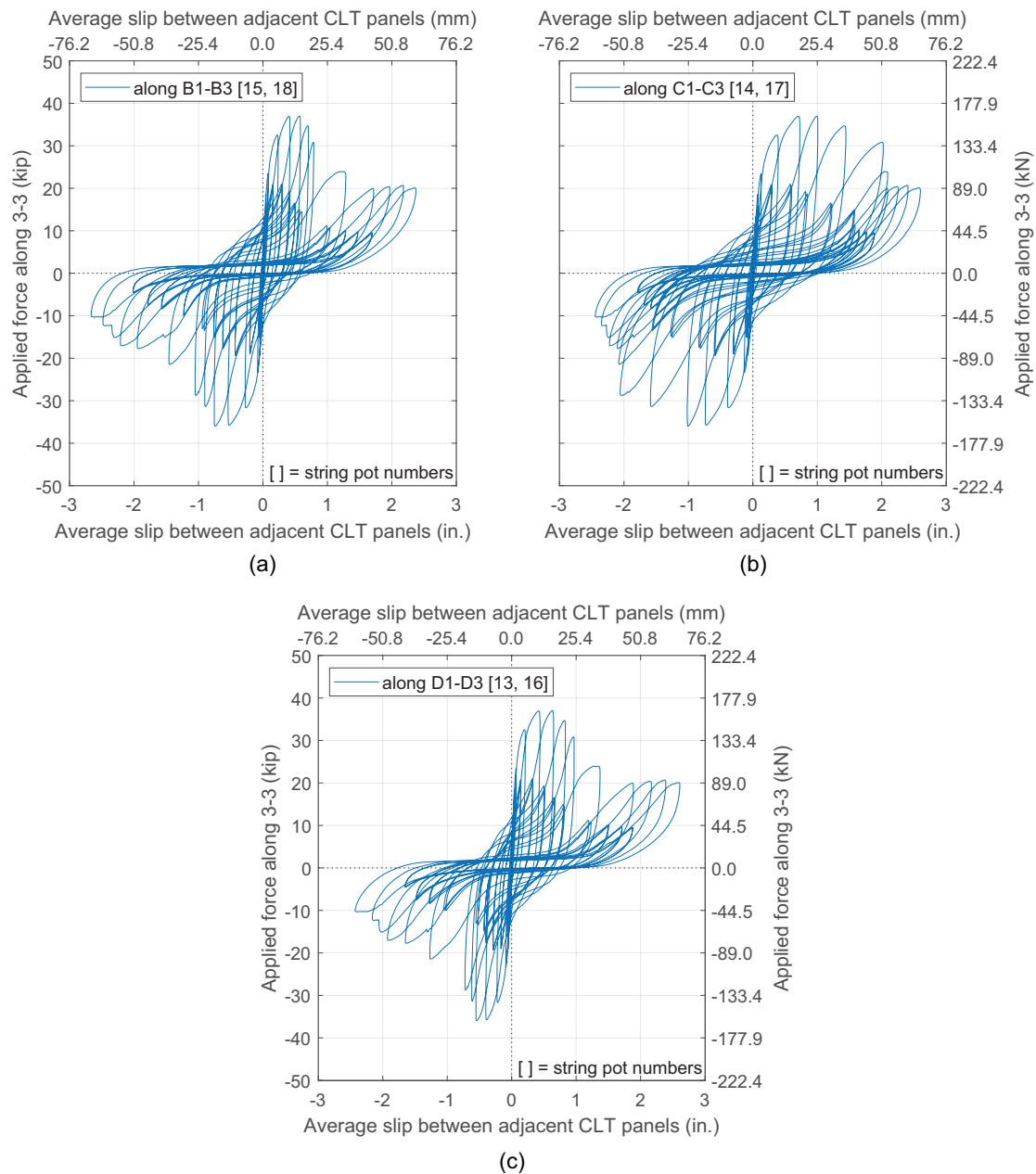


Fig. 9. Slip in nailed surface spline for second diaphragm test.

as well as the bending and shear deformations of the CLT panels contributed to the diagonal distortion (apparent shear deformation). Overall, the second diaphragm displayed slightly higher apparent shear stiffness than the first diaphragm. Because panels 6.1 m (20 ft) in length were used in the second diaphragm (opposed to panels 3 m (10 ft) in the first diaphragm), the middle seam with tension straps was absent from the second diaphragm setup. The deformation on the fasteners at the middle seam in the first diaphragm could have contributed to higher overall shear deformation for the same load; thus, the first diaphragm exhibited a lower shear stiffness when compared with the second diaphragm.

Failure Analysis of Diaphragms

Postmortem analyses were carried out after both tests to document and study the failure mechanisms of the diaphragms. Figs. 15(a and b) reveal that the nailed spline connection experienced failure

during both the tests. As expected, the capacity of the nailed surface spline connections controlled the designs of the two diaphragms (Table 1). During both tests, a considerable uplift of the nailed surface spline connection was observed. Visual analysis after the test showed that the nails had been completely sheared off or substantially deformed. The nails showed a double plastic hinge formation, suggesting that Mode IV yield was the failure mode [Figs. 15(c and d)].

The CLT-glulam screws also displayed yield Mode IV behavior for both the tests [Fig. 15(e)]. With regards to the screws used in the first test, screws along Grid 1-1 (fixed support) did not undergo failure. The screws along Grid 2-2 at diaphragm boundaries for the first test failed in shear once the tension straps failed, whereas those on the inside were relatively intact, as shown in Fig. 15(f). The screws along Grid 3-3 underwent highest deformation among all screws for both the tests but only a small fraction of the overall screws sheared off during the first test. However, during the second

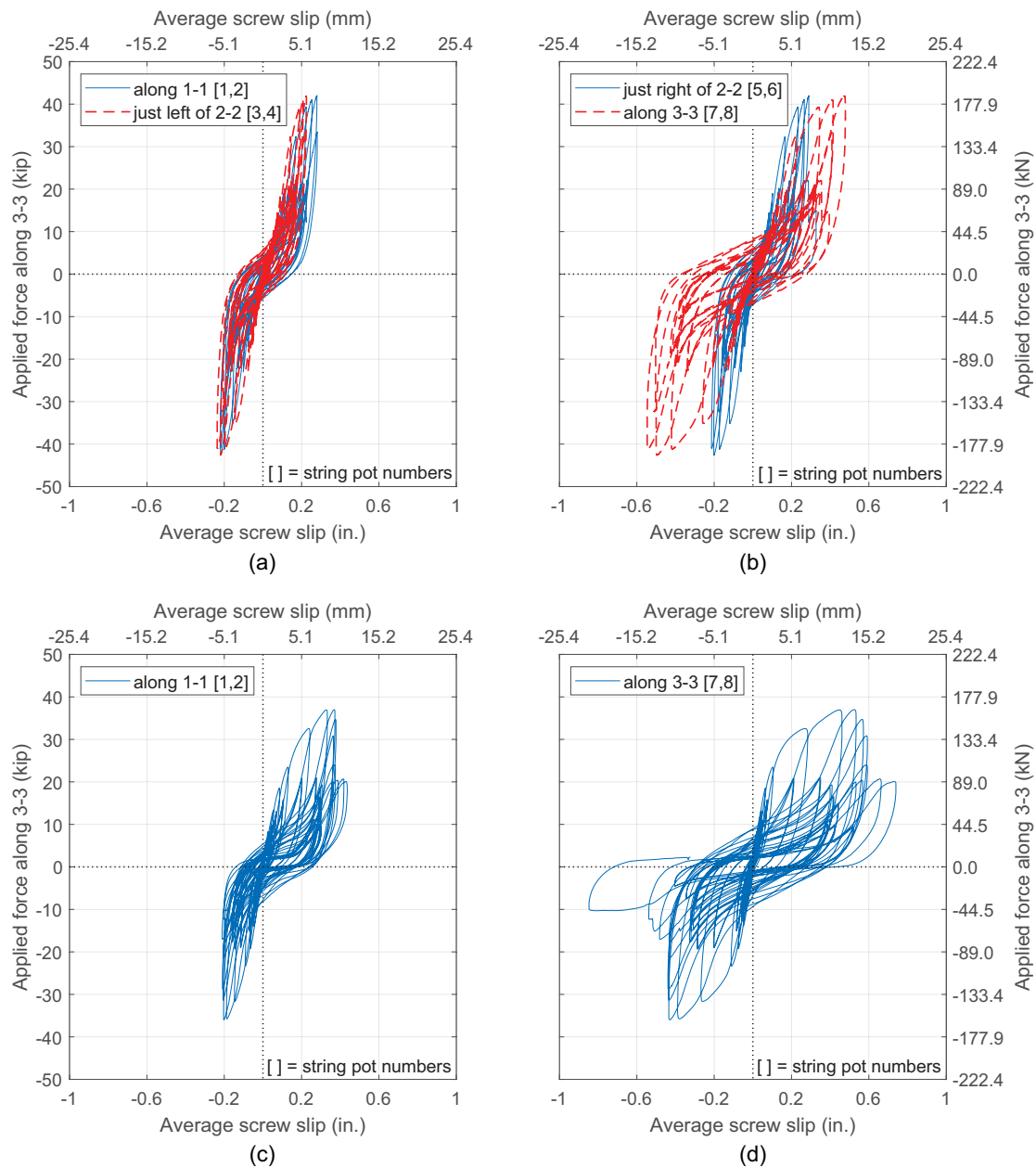


Fig. 10. Screw slip variation for (a and b) first; and (c and d) second diaphragm test.

test, majority of the screws along Grid 3-3 (free end) had undergone failure, as shown in Fig. 15(g).

The second diaphragm setup underwent much larger applied displacement, which could have exceeded the displacement capacity of the fixed-end screws in the second test more than the first test. Some of the screws along Grid 1-1 for the second test had undergone failure, which was different from the observation made during the first diaphragm test. A higher aspect ratio (4:1) of the CLT panels used in the second diaphragm test compared with that of the first diaphragm test (2:1) could have caused more rotation at the extreme ends, which deformed the fixed-end screws more.

The tension straps, which were used only on the first diaphragm test setup, ruptured during the postpeak cycles. The tension straps were designed to be 2.3 times stronger than the nailed surface spline connections to ensure elastic behavior in the tension straps up to the LRFD design strength (prepeak). When the diaphragm was loaded

beyond the design strength and its peak load (i.e., entering postpeak behavior), excessive rocking of individual CLT panels was observed, leading to rupture of the tension straps (the rocking mechanism is shown in Fig. 13). As a consequence, higher loads were imposed on the straps, leading to their failure.

The hold-downs did not show any visible signs of failure for both the tests. Both the hold-down rod and screws were still fully intact after the completion of the tests [Fig. 15(h)]. The capacity of hold-downs were intentionally designed to be at least twice that of the nailed splines (Table 1).

Figs. 16(a and b) show the variation of strain in the end tension straps on each extremity of the first diaphragm test setup with respect to the applied force. The force levels at which the straps began to yield and reached peak strength are synchronous with the actual diaphragm test, suggesting that the tension straps, along with the nailed surface spline connection between Grids 2-2 and 3-3, governed the

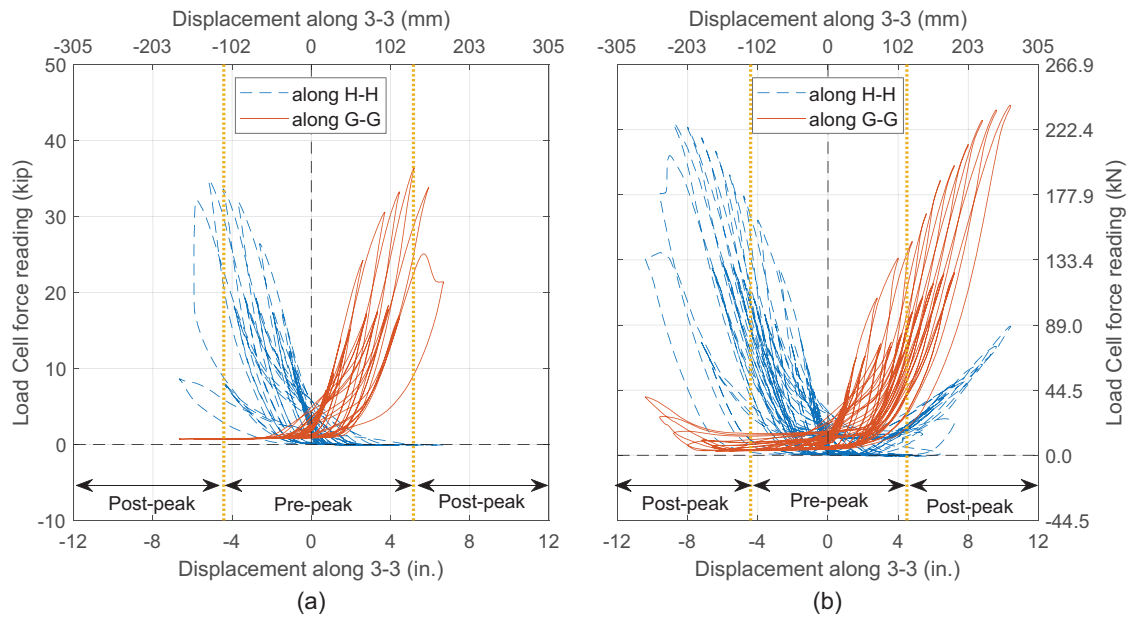


Fig. 11. Load cell force variation with applied displacement for (a) first; and (b) second diaphragm test.

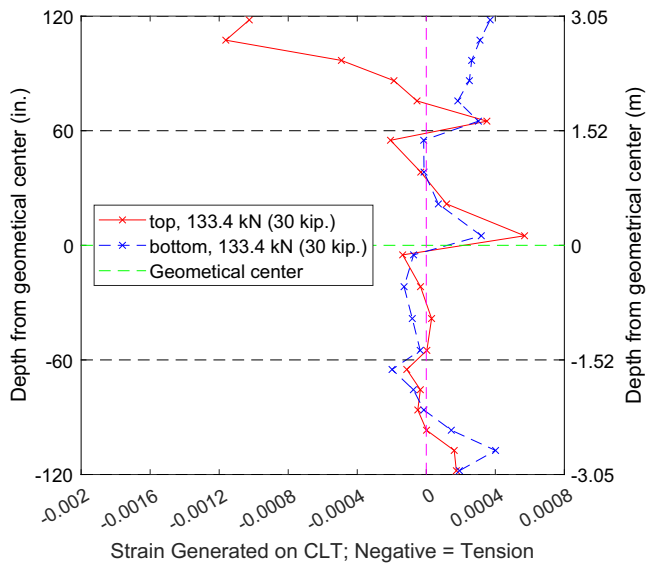


Fig. 12. Postpeak strain variation for second diaphragm test.

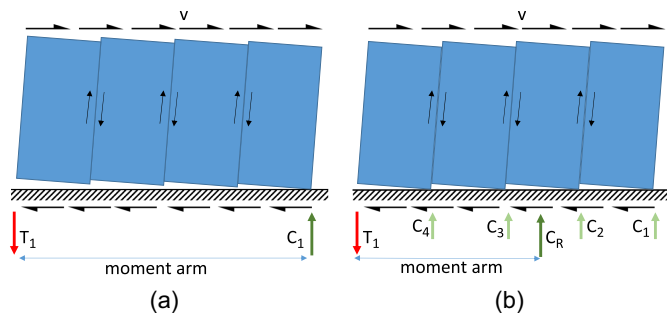


Fig. 13. Conceptual illustration of rocking mechanism for CLT panels: (a) before peak force; and (b) after peak force.

peak capacity of the first diaphragm. The actual rupture of the tension straps [Fig. 15(i)] occurred later during postpeak cycles (Fig. 5).

Development of Design Method

Although some variations in the strain profile exists between the two tests and also between the positive and negative cycles of each test, it can be observed that the strain profiles decayed rapidly from peak strains at the edge of the CLT panels to values close to zero at a distance about two-thirds of the width of the edge panels, as shown in Fig. 6.

For modeling purposes, the net tensile strain developed may be approximated with a triangular profile acting only on the ply with a positive tension load path, i.e., the top ply for this study (because tension straps were installed on the top of the CLT panels). The distance from the edge of the CLT panels to the region where the tensile strain reaches values close to zero will be referred to as the depth of the tension block (d_t). The tension strain profile had the highest values at the boundary of the edge panel and reached zero strain at a distance d_t away from the extreme fiber in tension (Fig. 17). It was observed that d_t was close to two-thirds the width of the edge panels (P_W).

Additionally, it is speculated that d_t is also a function of the design chord width (d_{ch}), which refers to the region on the edge panels where the tension straps were uniformly distributed [which was 69.8 cm (27.5 in.)] for the first diaphragm. The depth of the tension block was observed to be approximately 1.4 times d_{ch} in the first diaphragm test. Based on these observations, the following equation is proposed for approximating the depth of the tension block:

$$d_t = \min\left(\frac{2}{3}P_W, 1.4d_{ch}\right) \quad (1)$$

Eq. (1) and the subsequent equations in this paper only apply to diaphragms with more than one panel in the direction perpendicular to the load.

The distance from the edge of the CLT panels to the region where the compressive strain reaches values close to zero will be

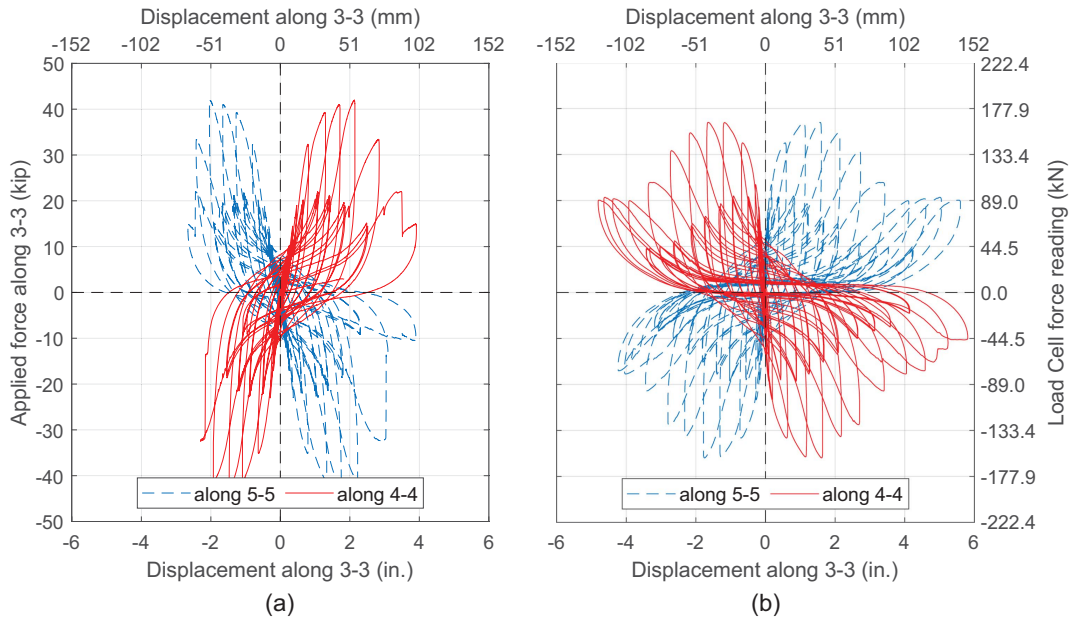


Fig. 14. Deformation along diagonals under applied force for (a) first; and (b) second diaphragm test.

referred to as the depth of the compression block (d_c). The net compressive strain developed may be approximated with two triangular profiles acting on the top and bottom plies with the highest values at the boundary and also reaching zero strain at a distance d_c from the extreme fibers in compression (Fig. 17). Because the wood fibers of the top and bottom plies were parallel to the compression load, both the top and bottom plies were effective in carrying the compression load. It was observed from both the diaphragm tests that d_c was also approximately two-thirds the edge panel width [$2/3P_w = 102$ cm (40 in.)].

The compression forces were introduced through the toe bearing during the rotation of the diaphragm. Due to the lack of edge glue between laminations in a single layer, shear must flow into the near cross ply and back into the next top lamination. As a result of this inefficiency of the mechanisms, it is speculated that the depth of compression block would not exceed 102 cm (40 in.) even if wider panels were employed on the diaphragms [in place of the panels with width 1.5 m (5 ft)]. Based on these assumptions, the following equation is proposed for approximating the depth of the compression block:

$$d_c = \min\left(\frac{2}{3}P_w, 40 \text{ in.}\right) \quad (2)$$

where 40 in. = 102 cm.

For the diaphragm setups used, the centroids of both the approximate triangular tensile and compressive strain profiles are approximately 33.8 cm (13.3 in.) from the panel edges. Table 3 indicates that the centroids of the triangular strain profiles were close to the centroid of the actual observed strain profiles. This further confirms that a triangular strain profile may be used to approximate the actual strain distribution.

From Fig. 17, on a cantilever diaphragm, if P is the applied point load at the free end of the diaphragm, the moment (M) at a distance x from the free end is given by

$$M = Px \quad (3)$$

For equilibrium, internal moment created by the internal tension and compression forces must resist the external applied moment.

The tensile (T) or compressive (C) forces developed to resist the applied moment are equal

$$T = C \quad (4)$$

The values of T and C can be determined as follows:

$$T = C = \frac{M}{d} \quad (5)$$

where d = distance between the centroids of the tension and compression regions, which can be found as follows:

$$d = W - \frac{1}{3}(d_t + d_c) \quad (6)$$

where W = width of the diaphragm [parallel to the applied load direction or length of the fixed support (Fig. 17)]. Substituting Eq. (6) into Eq. (5) yields the following equation for computing the tensile and compressive forces:

$$T = C = \frac{M}{W - \frac{1}{3}(d_t + d_c)} \quad (7)$$

Tensile tests on parallel to grain Southern Pine specimens indicate the stress-strain curve to be linear up to nearly 1% strain (Murray 2007). The highest tensile strain levels observed in diaphragm tests were less than 0.2%. Although the strain values measured by the strain gauges placed on top of metal shims were not the absolute strains, the observed strain profiles clearly showed that the CLT wood fibers were in the linear elastic stress-strain range by the time the fasteners have failed. Thus, the tensile force developed could also be expressed

$$T = E(\varepsilon_{t_{avg}})A_{\text{tension}} \quad (8)$$

where E = modulus of elasticity of the CLT lumber (parallel to the grain); $\varepsilon_{t_{avg}}$ = average strain over the tension block; and A_{tension} = effective area of the tension chord zone wood fibers in tension.

Assuming a triangular tensile strain profile, the average tensile strain, $\varepsilon_{t_{avg}}$, is equal to half the maximum strain ε_{t_i} as follows:



Fig. 15. (a) Spline deformation in first diaphragm test; (b) spline deformation in second diaphragm test; (c) nail deformation in first diaphragm test; (d) nail deformation in second diaphragm test; (e) typical CLT-glulam screw deformation for both diaphragm tests; (f) intact screws along Grid 2-2 for first diaphragm test; (g) screws along Grid 3-3 for second diaphragm test hung outside CLT for reference; (h) typical intact hold-down and screws after first and second diaphragm tests; and (i) tension straps rupture in first diaphragm test.

$$\varepsilon_{t_{avg}} = \frac{1}{2} \varepsilon_t \quad (9)$$

For the tested diaphragms, only the top layer of the three-ply CLT panel was effective in carrying tension force (Fig. 17). Thus, the effective tension chord area $A_{tension}$ can be found as follows:

$$A_{tension} = d_r t \quad (10)$$

where t = thickness of the individual parallel to grain plies of the CLT panels.

Using a similar formulation, the compressive force can be found as follows:

$$C = E(\varepsilon_{c_{avg}}) A_{compression} \quad (11)$$

where $\varepsilon_{c_{avg}}$ = average strain over the compression block; and $A_{compression}$ = effective area of the compression block wood fibers. Assuming a triangular tensile strain profile, the average tensile strain, $\varepsilon_{c_{avg}}$, is equal to half the maximum strain ε_c as follows:

$$\varepsilon_{c_{avg}} = \frac{1}{2} \varepsilon_c \quad (12)$$

For the diaphragm tests, both the top and bottom layers engaged in compression resistance. In case of diaphragms with CLT panels having more than three plies, it is expected that all layers oriented

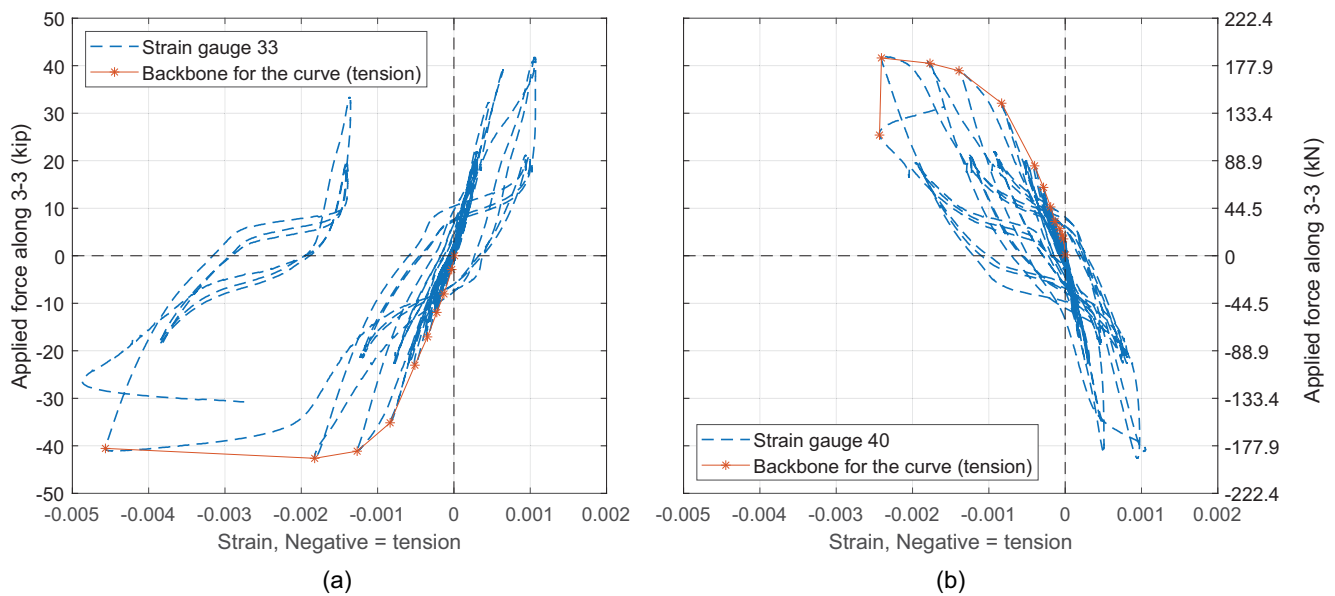


Fig. 16. Strain variation across tension straps for first diaphragm test: (a) strain variation in Strain gauge 33; and (b) strain variation in Strain gauge 40.

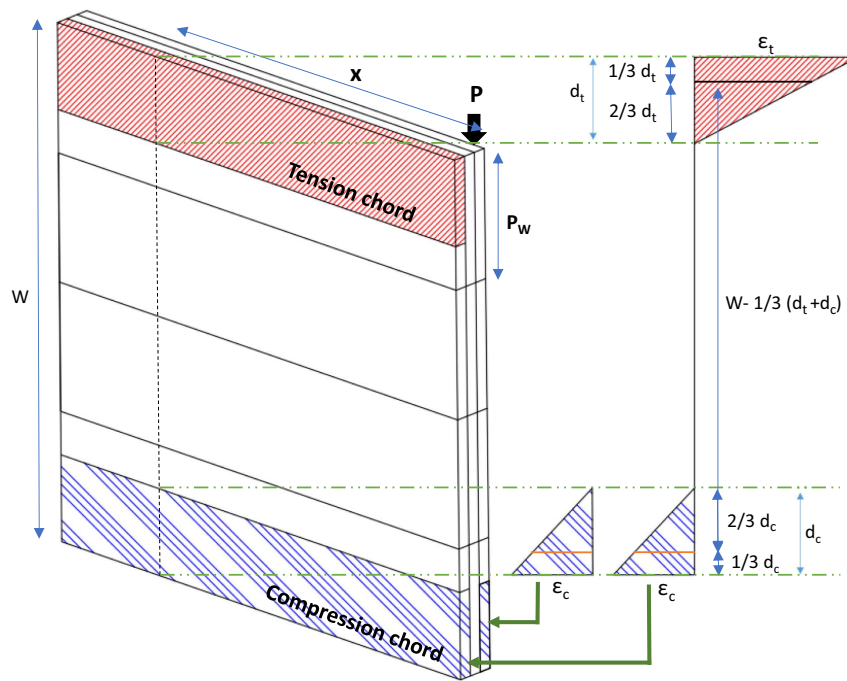


Fig. 17. Equivalent strain diagram.

Table 3. Location of centroids of observed strain diagrams

Test	Cycle	Average location ^a of centroid	
		Tensile strain [cm (in.)]	Compressive strain [cm (in.)]
1	Positive	38 (15)	30 (12)
1	Negative	38 (15)	33 (13)
2	Positive	33 (13)	33 (13)
2	Negative	30 (12)	40 (16)

^aLocation measured from nearest diaphragm edge.

parallel to grain in the direction of compression would offer compression resistance. Thus, a generic equation for $A_{\text{compression}}$ can be written

$$A_{\text{compression}} = n_{\parallel} d_c t \quad (13)$$

where $n_{\parallel}$ = number of layers oriented parallel to grain in the direction of compression. Substituting the values from Eqs. (8)–(13) into Eq. (4) gives the following expression:

$$\varepsilon_t = \frac{n_{\parallel} d_c}{d_t} \varepsilon_c \quad (14)$$

Equating Eqs. (8) and (7) and solving for maximum tensile strain yields the following equation:

$$\varepsilon_t = \frac{2M}{[W - \frac{1}{3}(d_t + d_c)]Ed_t} \quad (15)$$

The following limit state can be used to check for the adequacy of the tension chord design:

$$f_t = \varepsilon_t E < F'_{t0} \quad (16)$$

where f_t = (maximum) stress at the extreme fiber in tension; and F'_{t0} = parallel-to-the-grain adjusted LRFD tensile stress capacity of the CLT wood fiber (i.e., adjusted for temperature, moisture, and other relevant factors of NDS).

Equating Eqs. (11) to (7) and solving for maximum compression strain yields the following equation:

$$\varepsilon_c = \frac{2M}{[W - \frac{1}{3}(d_t + d_c)]n_{\parallel}Ed_t} \quad (17)$$

The following limit state can be used to check for the adequacy of the compression chord design:

$$f_c = \varepsilon_c E < F'_{c0} \quad (18)$$

where f_c = (maximum) stress at the extreme fiber in compression; and F'_{c0} = parallel-to-the-grain adjusted LRFD compressive stress capacity of the CLT wood fiber (i.e., adjusted for temperature, moisture, and other relevant factors of NDS).

Eq. (18) only provides a limit state for pure compression. For a comprehensive design of the compression chord in a CLT diaphragm, other limits such as buckling should also be checked.

Summary and Conclusion

Two full-scale cantilever CLT diaphragms were tested using the CUREE cyclic loading protocol. The test diaphragms were designed such that the nailed surface spline controlled the overall design strength. All other components were designed with a strength factor of at least two. The test results showed that the peak capacity of the diaphragm exceeded the LRFD design capacity by a factor of close to two. The ductile nature of the fasteners, particularly the nail connections at the surface splines and the tension straps, and the CLT–glulam screw connections greatly contributed to the ductility of the entire diaphragm.

The test data also showed that the ply in the CLT panels directly attached to the hold-downs (the top ply in this case) experienced substantial tension. The ply on the bottom that did not have a direct connection to hold-downs was not effective in carrying tension force. Hence, it is recommended that only the ply on which hold-downs have been installed be used for tension chord design. In contrast, the test showed that for the compression region, both the top and bottom CLT plies developed near-uniform compressive strains. Thus, it is recommended that all plies parallel to the compression force generated be used for compression chord design. Additionally, based on the observations of the tension and compression strain profiles, it is suggested triangular stress distributions for both the tension and compression chords be used. According to the two diaphragm tests, the extent or width of tension chord may be approximated as a minimum of two thirds of width of the outermost CLT

panels and 1.4 times the design chord width, and the width of compression chord may be approximated as a minimum of two-thirds of width of the outer most CLT panels and 102 cm (40 in.).

The absence of a gap between the edge of the splines and the routed portion of CLT resulted in bearing contact between the CLT panels and plywood splines, leading to uplift of the splines during the first diaphragm test. However, when a gap sized 0.32 cm (1/8 in.) was introduced between these elements (for the second diaphragm test), there was a notable reduction in the uplifting of the plywood splines. Therefore, the adoption of this gap size proved to be effective in mitigating the issues related to bearing and uplift of the splines.

It is essential to highlight that the findings and observations presented in this paper were derived from two full-scale tests. Diaphragms in practical applications exhibit significant variability in terms of geometry, fastener configurations, relative rigidity of the fasteners, the presence and size of openings, and other factors. Conducting additional tests on various realistic diaphragm configurations is needed to validate the results obtained from the two full-scale tests conducted and have the potential to uncover new findings that can contribute to developing robust code provisions.

Data Availability Statement

Some or all data, models, or code that support the findings of this study are available from the corresponding author upon reasonable request.

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Notation

The following symbols are used in this paper:

- $A_{\text{compression}}$ = cross-sectional area of the compression block;
- A_{tension} = cross-sectional area of the tension block;
- C = compressive force developed on a section at a distance x from applied load;
- C_R = resultant compression force;
- C_1, C_2, C_3 = compression forces generated at rocking point of individual panels;
- d = distance between centroids of tension and compression blocks;
- d_c = depth of the compression block;
- d_{ch} = design chord width;
- d_t = depth of the tension block;
- E = modulus of elasticity of specimen (parallel to grain);
- F'_{c0} = adjusted LRFD compressive stress capacity of specimen (parallel to grain);
- F'_{t0} = adjusted LRFD tensile stress capacity of specimen (parallel to grain);
- f_c = (maximum) stress at the extreme fiber in compression;

f_t = (maximum) stress at the extreme fiber in tension;
 M = moment on a section at a distance x from applied load;
 $n_{||}$ = number of layers oriented parallel to grain in the direction of compression;
 P = applied force at free end (LRFD);
 P_W = individual panel width perpendicular to applied load;
 T = tensile force developed on a section at a distance x from applied load;
 T_1 = tension generated at the heel of the diaphragm;
 t = thickness of the individual parallel to grain plies of the CLT panels;
 W = width of diaphragm (along the direction of force);
 x = distance from point of application of force to the section considered;
 Δ_{ref} = reference displacement for the diaphragm test;
 ϵ_c = maximum compressive strain developed over the compression block;
 $\epsilon_{c_{avg}}$ = average strain developed over the compression block;
 ϵ_t = maximum tensile strain developed over the tension block; and
 $\epsilon_{t_{avg}}$ = average strain developed over the tension block.

References

- American Wood Council. 2015. *National design specification for wood construction: With commentary*. ANSI/AWC NDS-2015. Leesburg, VA: American Wood Council.
- American Wood Council. 2018. *National design specification for wood construction: With commentary*. ANSI/AWC NDS-2018. Leesburg, VA: American Wood Council.
- American Wood Council. 2021. *Special design provisions for wind and seismic*. ANSI/AWC SDPWS-2021. Leesburg, VA: American Wood Council.
- APA (The Engineered Wood Association). 2011. *Standard for performance-rated cross-laminated timber*. ANSI/APA PRG 320-11. Tacoma, WA: APA.
- APA (The Engineered Wood Association). 2019. *Standard for performance-rated cross-laminated timber*. ANSI/APA PRG 320. Tacoma, WA: APA.
- Ashtari, S. 2012. "In-plane stiffness of cross-laminated timber floors." M.S. thesis, Dept. of Civil Engineering, Univ. of British Columbia.
- Beirsto, C., R. Gupta, and T. H. Miller. 2022. "Monotonic and cyclic behavior of CLT diaphragms." *Pract. Period. Struct. Des. Constr.* 27 (2): 04021085. [https://doi.org/10.1061/\(ASCE\)SC.1943-5576.0000658](https://doi.org/10.1061/(ASCE)SC.1943-5576.0000658).
- Bhardwaj, B., W. Pang, D. Rammer, M. O. Amini, and S. E. Pryor. 2021. "Experimental performance testing of cantilever cross-laminated timber (CLT) diaphragm under in-plane shear." In *Proc., World Conf. on Timber Engineering*. Oslo, Norway: World Conference on Timber Engineering.
- Breneman, S., E. McDonnell, and R. Zimmerman. 2016. "An approach to CLT diaphragm modeling for seismic design with application to a us high-rise project." In *Proc., World Conf. on Timber Engineering, WCTE*, 22–25. Oslo, Norway: World Conference on Timber Engineering.
- Gagnon, S., and C. Pirvu, eds. 2011. *2011 Canadian CLT handbook*. 2011 ed. SP-528. Pointe-Claire, QC, Canada: Special Publication.
- IBC (International Building Code). 2015. *2015 International Building Code*. Washington, DC: IBC.
- ICC (International Code Council). 2020. *2021 International Building Code*. Washington, DC: ICC.
- Karacabeyli, E., B. Douglas, and C. Handbook. 2013. *Cross-laminated timber*. US ed. Pointe Claire, QC, Canada: FPinnovations and Binational Softwood Lumber Council.
- Kode, A., M. O. Amini, J. W. van de Lindt, and P. Line. 2021. "Lateral load testing of a full-scale cross-laminated timber diaphragm." *Pract. Period. Struct. Des. Constr.* 26 (2): 04021001. [https://doi.org/10.1061/\(ASCE\)SC.1943-5576.0000566](https://doi.org/10.1061/(ASCE)SC.1943-5576.0000566).
- Krawinkler, H., F. Parisi, L. Ibarra, A. Ayoub, and R. Medina. 2001. Vol. 102 of *Development of a testing protocol for woodframe structures*. Richmond, CA: Consortium of Universities for Research in Earthquake Engineering.
- Line, P., S. Nyseth, and N. Waltz. 2022. "Full-scale cross-laminated timber diaphragm evaluation. I: Design and full-scale diaphragm testing." *J. Struct. Eng.* 148 (5): 04022037. [https://doi.org/10.1061/\(ASCE\)ST.1943-541X.0003308](https://doi.org/10.1061/(ASCE)ST.1943-541X.0003308).
- Murray, Y. D. 2007. *Manual for LS-DYNA wood material model 143*. Washington, DC: Federal Highway Administration.
- Pang, W., and S. M. Hassanzadeh Shirazi. 2013. "Corotational model for cyclic analysis of light-frame wood shear walls and diaphragms." *J. Struct. Eng.* 139 (8): 1303–1317. [https://doi.org/10.1061/\(ASCE\)ST.1943-541X.0000595](https://doi.org/10.1061/(ASCE)ST.1943-541X.0000595).
- Pei, S., D. Rammer, M. Popovski, T. Williamson, P. Line, and J. W. van de Lindt. 2016. "An overview of CLT research and implementation in North America." In *Proc., World Conf. on Timber Engineering (WCTE)*, 25–22. Oslo, Norway: World Conference on Timber Engineering.
- Phares, B. M., T. L. Pence, J. P. Wacker, and T. K. Hosteng. 2019. *Development of a smart timber bridge (phase III): Moisture and strain sensor investigation for historic covered bridges*. Rep. No. FPL-GTR-268. Madison, WI: USDA.
- Spickler, K., M. Closen, P. Line, and M. Pohl. 2015. "Cross laminated timber horizontal diaphragm design example." In *Proc., White Paper Reference*. Surrey, BC, Canada: MTC Solutions.
- Sullivan, K., T. H. Miller, and R. Gupta. 2018. "Behavior of cross-laminated timber diaphragm connections with self-tapping screws." *Eng. Struct.* 168 (Mar): 505–524. <https://doi.org/10.1016/j.engstruct.2018.04.094>.
- Yeh, B., S. Gagnon, T. Williamson, C. Pirvu, C. Lum, and D. Kretschmann. 2012. "The North American product standard for cross-laminated timber." *Wood Des. Focus* 22 (2): 13–21.