Rocking Behavior of High-Aspect-Ratio Cross-Laminated Timber Shear Walls: Experimental and Numerical Investigation

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Abstract: Cross-laminated timber (CLT) is a mass timber product that has recently garnered considerable attention for lateral-force resisting system (LFRS) applications. The main objectives of this study were to investigate the rocking behavior of a high-aspect-ratio (height/width) CLT shear wall without post-tensioning, and to validate a finite-element (FE) model based on the cyclic and dynamic response of the wall. To this point, high-aspect-ratio walls in the literature have primarily been post-tensioned. The testing component of this study included connector tests, quasistatic cyclic shear wall tests, and shake-table tests under four different ground motions scaled to design earthquake (DE)- and maximum considered earthquake (MCE)-level intensities. A generic shear connector was used for this study to allow for proprietary and other systems to demonstrate equivalence. The connectors were tested under shear and uplift, and shear-wall tests were performed using the Consortium of Universities for Research in Earthquake Engineering (CUREE) displacement protocol, which has been widely used for light-frame wood structures. Interstory drift (ISD) ratios in the shake-table tests ranged from 0.97% to 2.02%, and the tests demonstrated the system's ability to resist seismic loading. An FE model of the CLT wall was developed that showed good agreement with the cyclic and shake-table tests. The difference between the ISD ratios in the numerical model and the shake-table tests ranged from 5.4% to 31.3%, with an average of 17.9%, which was in good accordance with the accuracy of the existing CLT models. This system can be utilized as a retrofit option, in conjunction with light-frame wood shear walls, where lack of space may be a challenge. **DOI: 10.1061/(ASCE)AE.1943-5568.0000473.** *© 2021 American Society of Civil Engineers.*

Author keywords: Cross-laminated timber; Shake-table testing; Nonlinear modeling.

Introduction

Although variations in properties are common in all materials, wood properties vary considerably, because it is a natural material. As a result, engineered wood products (EWPs) or wood-based composites were introduced in order to ensure uniformity and control over the range of wood properties, to efficiently utilize available resources in a cost effective way, and to open up new opportunities for the creative use of wood (FPL 2010; McKeever 1997; Youngs 2009). Cross-laminated timber (CLT) is one of the more-recently developed engineering materials for wood-building structural use.

Of its many potential applications, which include residential, industrial, and commercial buildings, it is the multistory construction of CLT that has attracted the interest of many architects, structural engineers, and researchers. In North America, the U.S. CLT Handbook (Karacabeyli and Douglas 2013) was developed, the ANSI/APA PRG 320 CLT (APA 2019) product standard was created and updated, and a new chapter focused on CLT was incorporated into the 2015 National Design Specifications (NDS) for Wood Construction (ANSI/AWC 2015). More broadly, numerous CLT buildings have been constructed around the world. These structures include the nine-story Stadthaus apartment building in London, the 10-story CLT building in Melbourne (KLH) and four nine-story tower buildings in Milan (Bernasconi 2016). With the exception of Italy, most CLT structures have been constructed in low-seismic regions. No CLT-based lateral-force resisting system (LFRS) is currently recognized in U.S. design codes. While recent research and development (e.g., Pei et al. 2016; Ceccotti et al. 2013) have indicated the capability of such lateral systems, the lack of a systematic and codified design and analysis approach is still an issue that hinders the application of CLT lateral systems in mid-rise building construction.

Recently, there has been a significant increase in CLT-related seismic research. Consequently, this study aimed to provide a novel contribution to the developing body of knowledge in this new area of engineering and construction through experimental results and the numerical model development of a non-post-tensioned system with steel tie-down rods at the ends of the wall. This configuration is different from previously examined CLT rocking-wall systems, which had post-tensioned rods positioned near the center or incorporated energy-absorbing hardware/devices at the panel sides or base. A comprehensive overview of CLT seismic research and implementation in North America through 2014 is provided in Pei et al. (2016). The post-tensioned rocking-wall concept has been investigated by Pei et al. (2017), Ganey et al. (2017), and Akbas et al. (2017), those studies focusing on the testing and analysis of a two-story-tall wall. Shake-table tests have been conducted on a

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full-scale two-story mass-timber building with post-tensioned CLT rocking walls, using a LFRS (Pei et al. 2019). A recent study investigated the use of slip-friction connections with CLT rocking walls (Fitzgerald et al. 2020). Some of the studies were performed using proprietary connectors and overturning restraining systems, with researchers in North America (Ho et al. 2017; Ganey et al. 2017; Kovacs and Wiebe 2016) looking to adapt the timber-rocking system developed in New Zealand using laminated-veneer lumber (LVL) (Palermo et al. 2006; Buchanan et al. 2008; Iqbal et al. 2015) to CLT. Recent studies in New Zealand have focused on CLT rocking walls that incorporate novel energy-dissipating devices in the panel (Hashemi et al. 2018; Hashemi and Quenneville 2020). A recent comprehensive review of the seismic behavior of CLT can be found in Izzi et al. (2018a).

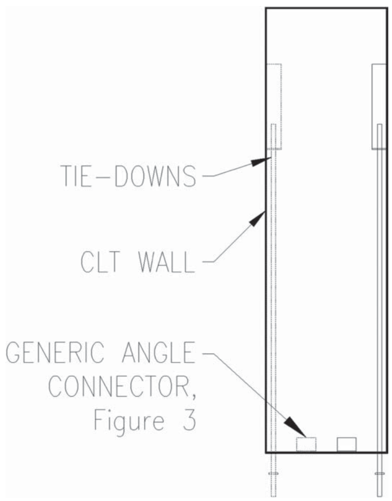

Fig. 1. Typical high-aspect-ratio CLT shear wall.

Much of the existing CLT lateral-system research has focused on post-tensioned CLT walls. Although post-tensioned systems are effective at recentering, the majority of post-tensioned systems require equipment and installation processes that are not commonly used in traditional wood-building processes. However, Bahmani et al. (2016) developed a conventional non-post-tensioned CLT-panel rocking wall as a retrofit option for soft-story light-frame wood buildings. During that study, however, the system was not tested to collapse. Building on this existing work (Bahmani et al. 2016), the intent of this study was twofold: (1) to conduct systematic testing of a high-aspect-ratio CLT LFRS to study its performance under large deformations up to the point of near collapse; and (2) to validate a nonlinear dynamic model for the proposed system through systematic testing at the connection, assembly, and system levels. The dynamic (shake-table) testing of CLT walls to close to the point of collapse has not previously been performed. A typical high-aspect-ratio CLT shear wall, with generic connectors at the base and appropriate overturning moment restraint, is shown in Fig. 1. The shear and uplift forces were transferred by these components. There was no explicit shear key installed for this system. The evaluation process is summarized in Fig. 2, with the details explained

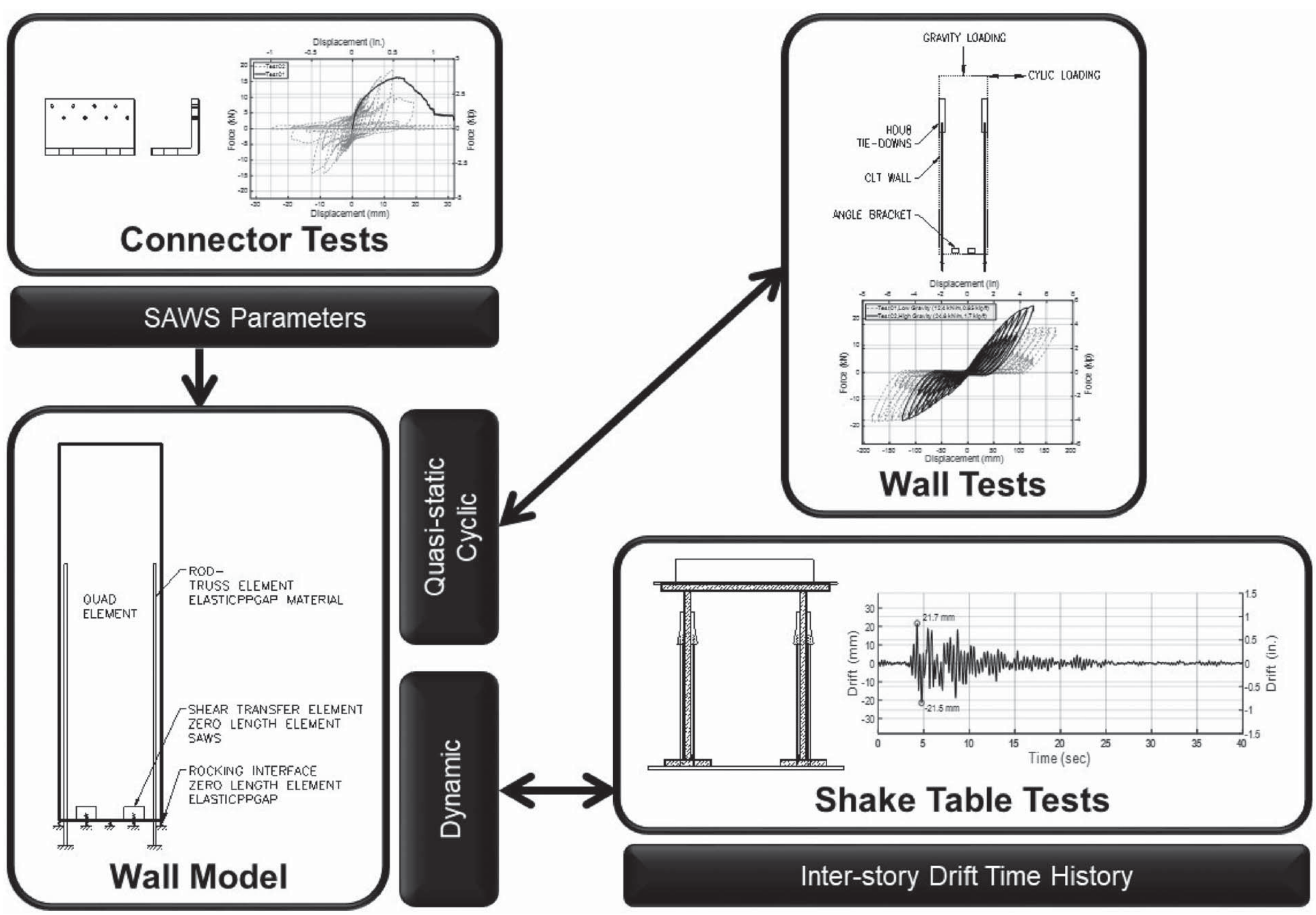

Fig. 2. CLT panel rocking-wall evaluation process.

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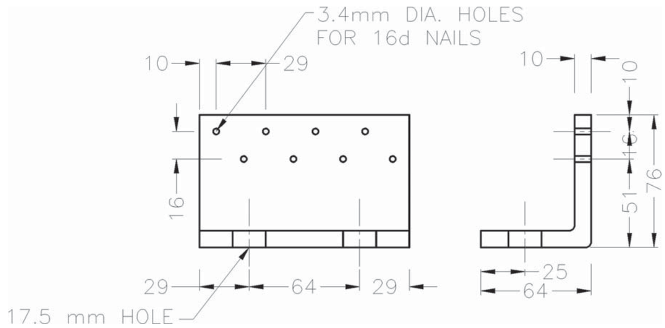

Fig. 3. A1-type connector with eight 16d box nails.

throughout this paper. Connector tests were performed to determine connector behavior and establish a test-based performance baseline. As these connectors were generic, not proprietary, it was important to understand their behavior for modeling purposes. Hysteretic parameters obtained from the connector tests were used for wall modeling, with the results from these models then being compared with the quasistatic wall and shake-table tests.

Testing

Connector Tests

Previous studies (Lauriola and Sandhaas 2006; Dujic et al. 2006) have shown that CLT panels themselves exhibit linear-elastic behavior, and any energy dissipation or ductility in CLT systems are achieved using connectors that behave nonlinearly. As the connector layout and properties have an influence on CLT shear-wall performance, investigating their behavior is fundamental to model development. For the purpose of this study, generic angle connectors were manufactured to establish a test-based performance baseline for manufacturers to demonstrate equivalency with. These angle connectors, shown in Fig. 3, were used to attach the CLT panel to the supporting element (i.e., the CLT base) using eight 16d box nails (3-1/2 in. × 0.375 in., 88.9 mm × 9.5 mm) and two 5/8 in. (15.88 mm) diameter bolts, designed per NDS (ANSI/AWC 2018) and AISC for the steel bracket. The angle connectors were overly thick so as to enforce mode III and mode IV nonlinear behavior in the nails. These modes correspond to the formation of one and two plastic hinges in the fastener, respectively, with the intention of achieving a ductile failure mechanism.

Shear and uplift tests were conducted on the connectors under monotonic and cyclic loading. The cyclic shear tests were performed using a test protocol developed by Krawinkler et al. (2000) specifically for light-frame wood fasteners, walls, and other assembly tests. The general form of what has become known as the Consortium of Universities for Research in Earthquake Engineering (CUREE) test protocol is shown in Fig. 4. The CUREE displacement protocol consists of initiation, primary and trailing cycles. The reference displacement, Δ, is defined as the deformation at which the load drops, for the first time, below 80% of the maximum load applied to the specimen, with the reference displacement being obtained from monotonic tests. In the uplift tests, the angle connectors were subjected to a non-reversed CUREE loading, with a pause at the zero point before each loading system was initiated. The connector test matrix is provided in Table 1.

The connector shear and uplift test configurations are shown in Figs. 5 and 6, respectively. The primary mode of failure in the shear and uplift testing was nail shear failure (Fig. 7) and nail withdrawal (Fig. 8), respectively.

Example connector hystereses for the shear and uplift tests are shown in Figs. 9 and 10, respectively. The test results were

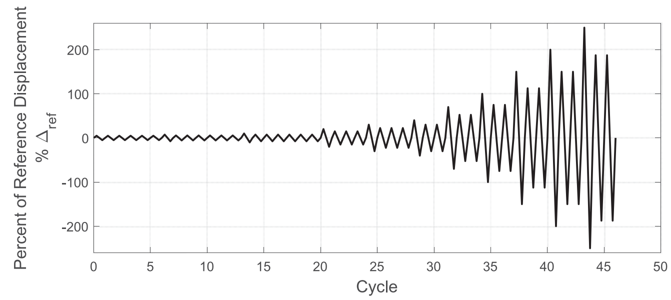

Fig. 4. CUREE loading protocol.

Table 1. CLT connector tests

Connector details	Test type	Test	Test#
(8)16d box (3-1/2 in. × 0.375 in., 88.9 mm × 9.5 mm) nails in vertical leg and two 5/8 in.	Shear	One monotonic, three cyclic	Test 01 Monotonic Test 02–04 Cyclic
(15.9 mm) diameter rods (ASTM 1554 Grade A36 steel) in horizontal leg	Uplift	One monotonic, three non-reversed cyclic	Test 05 Monotonic Test 06–08 Cyclic

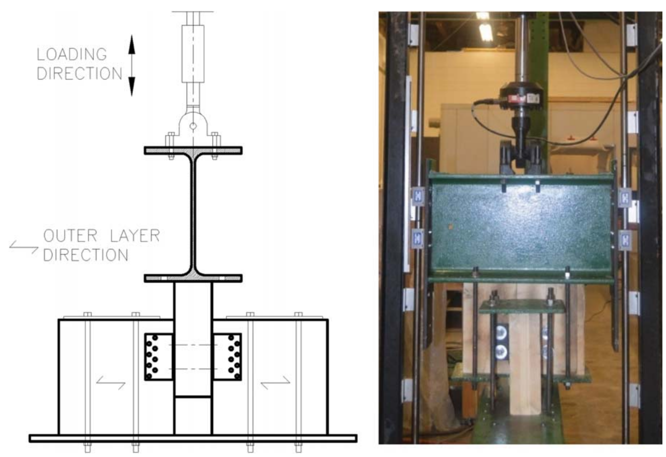

Fig. 5. Connector shear-test configuration.

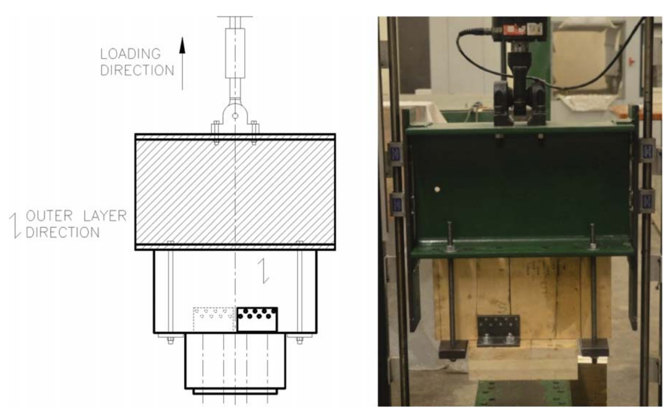

Fig. 6. Connector uplift-test configuration.

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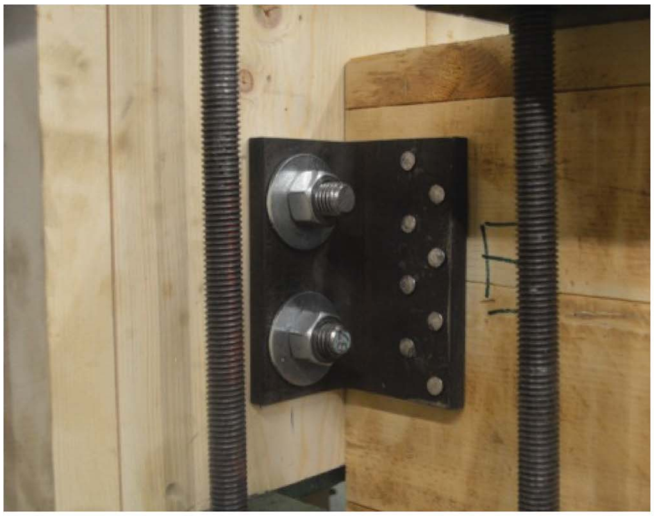
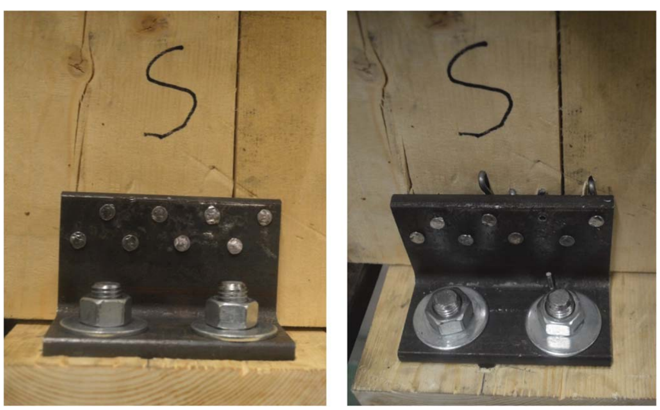


(a)

(b)

Fig. 7. Connector shear test: (a) before test; and (b) after test.



(a) (b)

Fig. 8. Connector uplift test: (a) before test; and (b) after test.

analyzed based on the procedure outlined in FEMA P795 (FEMA 2011), which is similar to ASTM E 2126 (ASTM 2009), except for the definition of Δ_{yield}. An example of Test 02 is illustrated in Fig. 11, with the parameters for the positive and negative excursions being reported in Table 2.

CLT Rocking-Wall Tests

The setup configurations for the wall tests are shown in Figs. 12 and 13. Reversed cyclic tests were performed on 0.61 m × 2.24 m × 90 mm (2 ft × 7 ft 4 in. × 3.5 in.) CLT panels. A CLT base was used to replicate a typical floor condition to provide insights into crushing perpendicular to the grain of the CLT floor owing to the rocking behavior of the panel. The horizontal actuator was under displacement control, using the CUREE displacement

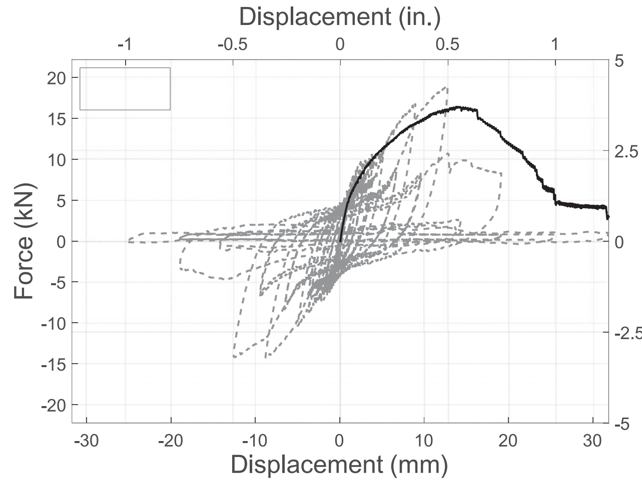


Fig. 9. Connector hysteresis for the shear tests.

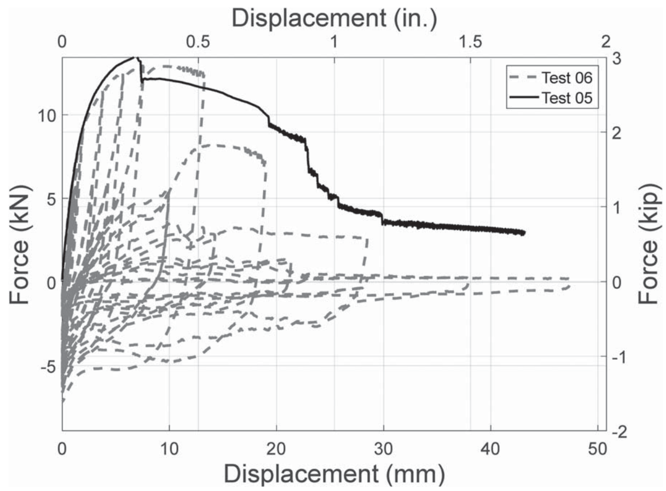


Fig. 10. Connector hysteresis for the uplift tests.

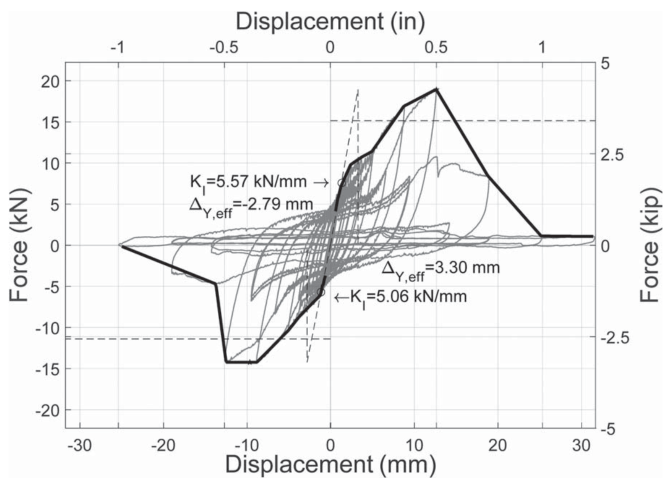


Fig. 11. Test 02 connector hysteresis with cyclic envelope properties.

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Table 2. Cyclic envelope parameters for connector and CLT shear-wall tests

Test type	Test #[a]	Initial stiffness K_I (kN/mm)	Effective yield $\Delta_{Y,\text{eff}}$ (mm)	Ultimate load $F_{\max}$ (kN)	Displacement corresponding to ultimate load $\Delta_{F\max}$ (mm)	Ultimate deformation (0.8 $F_{\max}$) Δ_u (mm)[b]	Effective ductility capacity μ_{eff} ($\Delta_u/\Delta_{Y,\text{eff}}$)
Connector	02 +ve	5.57	3.30	18.90	13.21	13.72	4.15
Shear test	02 −ve	5.06	2.79	14.23	9.65	12.70	4.55
	Avg.	5.32	3.05	16.57	11.43	13.21	4.33
	03 +ve	3.47	5.08	17.75	12.45	17.78	3.50
	03 −ve	3.12	4.57	13.97	12.45	13.34	2.92
	Avg.	3.30	4.83	15.86	12.45	15.56	3.22
	04 +ve	4.39	4.32	19.04	12.45	13.72	3.18
	04 −ve	4.36	3.56	15.17	12.45	13.97	3.93
	Avg.	4.37	3.94	17.10	12.45	13.84	3.52
Connector	06	6.29	2.03	12.81	7.37	14.99	7.38
Uplift test	07	5.77	2.29	12.83	5.59	15.75	6.89
	08	5.59	2.29	12.43	5.84	14.61	6.39
Wall test	01 +ve	0.22	85.09	18.64	149.35	177.80	2.09
	01 −ve	0.16	103.38	16.64	177.80	177.80	1.72
	Avg.	0.19	94.23	17.64	163.58	177.80	1.89
	02 +ve	0.30	82.55	24.86	126.24	127.00	1.54
	02 −ve	0.23	79.25	18.01	125.73	127.00	1.60
	Avg.	0.26	80.90	21.44	125.98	127.00	1.57

[a]+ve and −ve refer to positive and negative excursions of the cyclic envelope, respectively.
[b]In cases where no post-peak degradation was observed, the ultimate deformation was taken as the maximum deformation observed in the test.

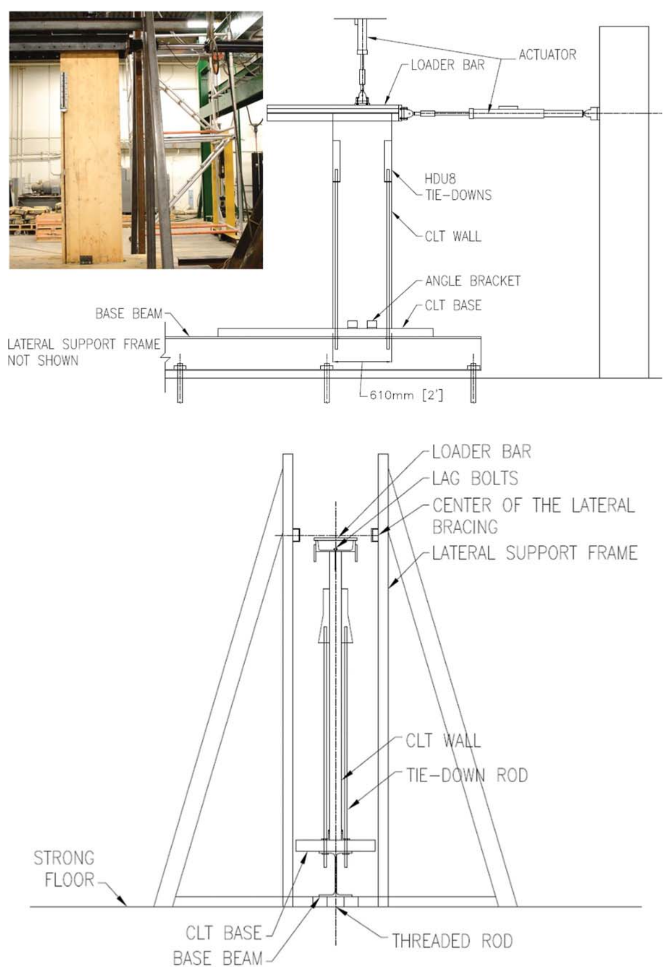

Fig. 12. Wall test setup configuration with a 4:1-aspect-ratio wall.

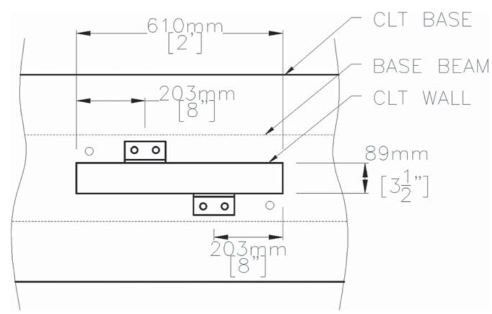

Fig. 13. Connector placement layout.

protocol shown in Fig. 4, whereas the vertical actuator was under load control for application of the gravity load. The horizontal displacement was applied at a rate of 0.1 Hz. Two reversed cyclic tests were performed on identical panels with low and high gravity loads of 12.4 kN/m (0.85 kip/ft) and 24.8 kN/m (1.7 kip/ft), respectively.

CLT rocking walls were designed based on the assumption that the angle connectors would carry the shear loads whereas the hold-downs would resist the uplift loads. This assumption was adopted in the initial stages of the Italian SOFIE project (Ceccotti 2008): a multifaceted effort aimed at enabling the use of CLT in residential and commercial buildings that included an extensive testing and analysis program. The assumptions for the shear and uplift design accord well with the already established design assumptions for light-frame wood shear walls. A comparison of different analytical models with experimental data (Gavric et al. 2015) has shown this assumption to be conservative. The tests were performed using the same type of angle connectors used for the connector tests. For uplift, two Simpson Strong-Tie HDU8 tie-downs were used with 15.9 mm (5/8 in.) (ASTM 1554 Grade A36 steel) threaded rods. The CLT rocking-wall hystereses are shown in Figs. 14 and 15, with a comparison provided in Fig. 16. Test results at different aspect ratios are discussed in Amini et al. (2018); as shown in Fig. 16, an increase in gravity results in an increase in both stiffness and strength. However, for the 4:1 aspect-ratio wall, the deformation capacity was also affected. The CLT rocking-wall tests were analyzed in a similar way to the connector tests, with the results provided in Table 2, and an example cyclic envelope along with the properties given in Fig. 17.

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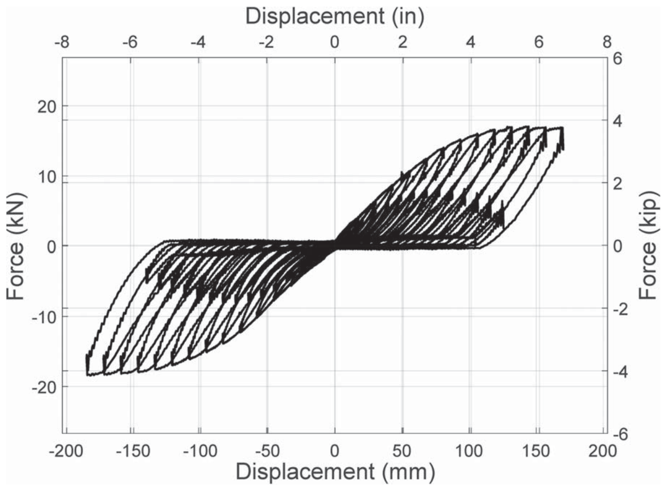

Fig. 14. Test 01 hysteresis, low gravity (12.4 kN/m, 0.85 kip/ft).

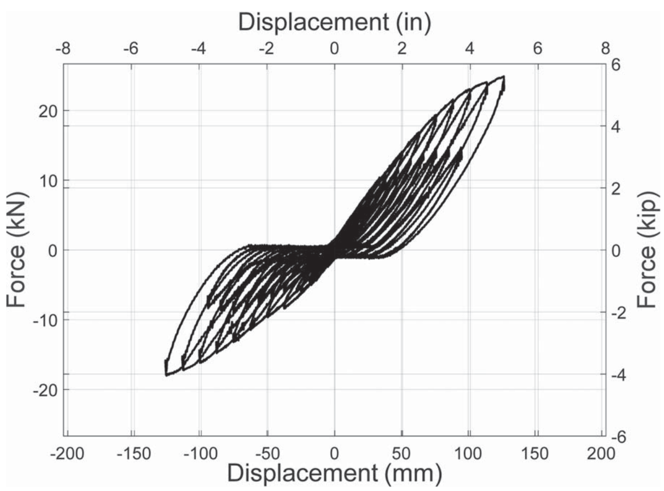

Fig. 15. Test 02 hysteresis, high gravity (24.8 kN/m, 1.7 kip/ft).

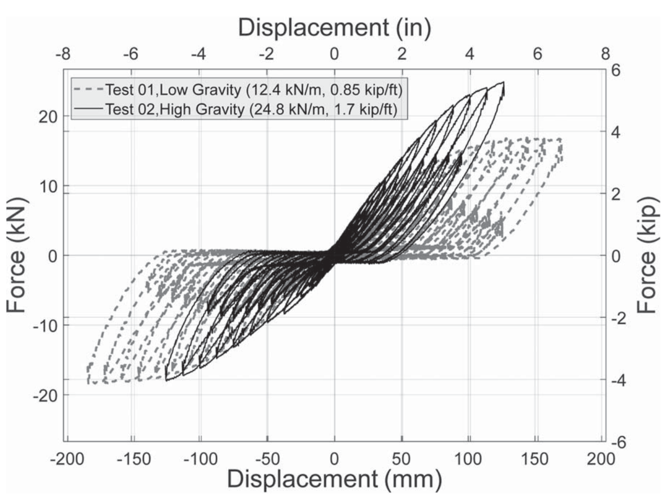

Fig. 16. Comparison of wall hystereses with low and high gravity.

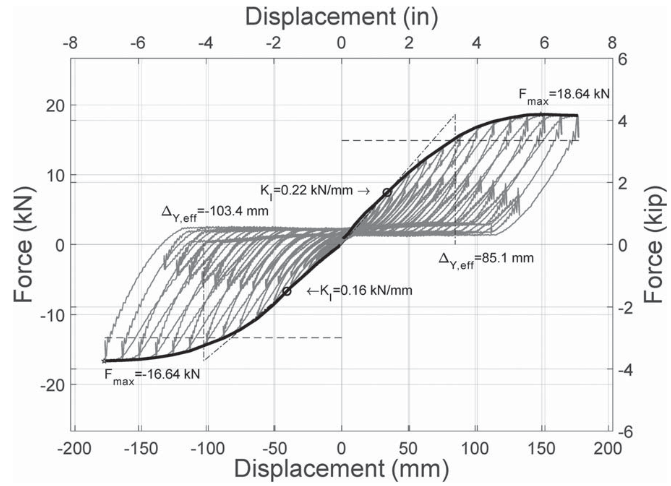

Fig. 17. Test 01 hysteresis with cyclic envelope properties.

Shake-Table Tests

The CLT specimen placed on the shake table prior to testing is shown in Figs. 18 and 19. A large steel plate was bolted to the diaphragm to facilitate the installation of seismic mass, and several steel beams were welded to the plate to provide a total weight of 15.12 kN (3.4 kip). With a 0.61 m × 2.24 m × 90 mm (2 ft × 7 ft 4 in. × 3.5 in.) CLT shear wall provided on each side, the seismic weight was equivalent to 12.4 kN/m (0.85 kip/ft). The wall configuration, including the connector type and layout, hold-down size and placement, and seismic weight, were the same as in Test 01 in the isolated CLT rocking-wall test. Connectors similar to the A1 type illustrated in Fig. 3 were used to attach the wall to the CLT roof. However, unlike the tested connector that had bolts in one leg, in this case, both connector legs had eight 16d nails in them. These details are shown in Figs. 20–22.

The tests were conducted using the 1987 Superstition Hills (El Centro Imp. Co.), 1989 Loma Prieta (Capitola) and 1994 Northridge (Rinaldi) ground motions. These grounds motions were selected because they represent recent major seismic events in the U.S. Furthermore, both the Loma Prieta and Northridge events resulted in significant damage to light-frame wood buildings, particularly those of the soft-story type. Thus, significant testing using these motions has been conducted in the U.S., making comparisons possible. The site location for the structure was chosen as California, with a stiff (D) soil class. The mapped maximum considered earthquake (MCE) spectral acceleration parameters for a short period (0.2 s), S_s, and at 1 s, S_1, determined the use of the ASCE 7-16 seismic maps (ASCE 2016), being 1.5g and 1.0g, respectively, with the corresponding site coefficients being a F_a of 1.0 and F_v of 1.5. Two levels of seismic hazard intensity were considered, including the design earthquake (DE) and MCE, corresponding to mean return periods of 474 years (10% probability of exceedance in 50 years) and 2,475 years (2% probability of exceedance in 50 years), respectively. The scaling was performed with the ground motions scaled to these two levels of intensity at the fundamental period of the structure, which was calculated to be 0.095 s based on Section 12.8.2.1 of ASCE 7-16. The design spectrum and response spectra of the scaled ground motions are given in Fig. 23. The specimen was subjected to the ground motions in the order shown in Table 3. No repairs were made between the ground motion applications, except the hold-downs were retightened by hand, if needed.

The instrumentation consisted of two string potentiometers attached to the floor diaphragm, intended to obtain the global response in terms of displacement. The experimental time history of the interstory drift (ISD) under different ground motions, and the corresponding maximum and minimum displacements, are discussed in the following. The aspect ratio of the panel resulted in all

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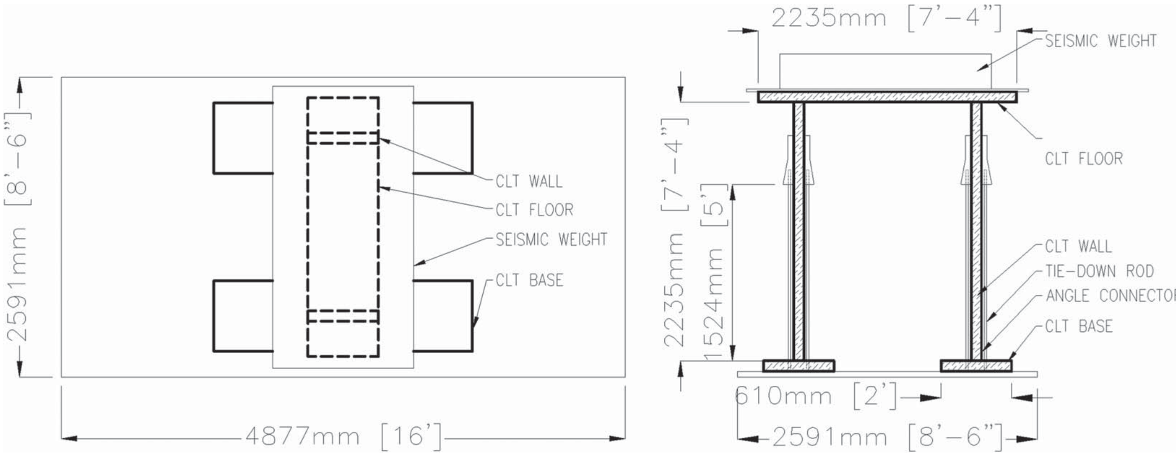

Fig. 18. Shake-table testing configuration.

Fig. 19. Shake-table specimen.

Fig. 20. Simpson Strong-Tie HDU8 hold-down with 15.9 mm (5/8 in.) A36 rod.

Fig. 21. A1 connector attaching the wall to the base CLT.

the deformation being caused by rocking, with no sliding being observed during the testing.

Numerical Modeling

A schematic of the numerical model developed using OpenSees (version 3.2.1) (Mazzoni et al. 2006) software is shown in Fig. 24. The model consists of an orthotropic membrane for the CLT and springs that simulate connectors, hold-downs and the supporting element. It is similar to the models presented in Dujic et al. (2008) and Aranha et al. (2016) on a rigid foundation, except this approach incorporated the characteristics of a base CLT. The CLT panels provided for this project were imported from Europe, with the layers consisting of boards of strength class C24 (EN 338). Owing to its cross-lamination, the characteristics of the CLT panel were calculated based on the method proposed by Blass and Fellmoser (2004). This method provides effective strength and stiffness values for a homogenized cross section using a set of composition factors, E1 and E2, which were calculated as 7,450 MPa (1,080.5 ksi) and 3,900 MPa (565.6 ksi), respectively. The modulus of elasticity in the out-of-plane direction was 400 MPa (58 ksi)

(Bogensperger et al. 2011; Brandner and Schickhofer 2014), while the shear modulus was calculated, based on the equations provided in Bogensperger et al. (2010), as 400 MPa (58 ksi).

As the base CLT was used for both the quasistatic and dynamic tests, the contact element between the CLT wall panel and the base CLT was modeled as a no-tension element. The stiffness was set equivalent to perpendicular to the grain stiffness of the CLT in the direction of the foundation, and to zero stiffness in the direction away from the foundation. It is important to point out that, in the case of compression perpendicular to the grain, in situ conditions are often very different from those of the standard test specimens (Leijten and Jorissen 2010) used to evaluate this property. In general, for timber, the stiffness is dependent on the geometry (Madsen et al. 1982) and, particularly in the case of CLT (Serrano and Enquist 2010), on the loading area and orientation.

Based on the test results, similarly to light-frame wood shear walls, the behavior of the CLT connectors was governed by nail and force–displacement responses, which are highly nonlinear. Various numerical models, varying in terms of detail and complexity, have been proposed to predict the hysteretic behavior of fasteners in wood. The 10-parameter hysteretic model, referred to as the CUREE-SAWS (Seismic Analysis of Woodframe Structures) model, developed by Folz and Filiatrault (2001) as part of the CUREE-Caltech project, was used to model the connector behavior in this study. This has been perhaps the most widely used connector model since its development. A number of studies (Shen et al. 2013; Schneider et al. 2015) have utilized the SAWS model to characterize CLT connector behavior, with European studies (e.g., Pozza et al. 2017; Izzi et al. 2018b) also having used similar models. The hysteretic fit for the shear and uplift tests using the model are shown in Fig. 25, with the parameters being provided in Table 4.

The CLT rocking-wall model was initially analyzed under displacement-controlled cyclic loading, similarly to the reversed cyclic test. A comparison of the cyclic test data with the numerical model is shown in Fig. 26. Visually, the model has more slip with

Fig. 22. Connector attaching the rocking wall to the roof.

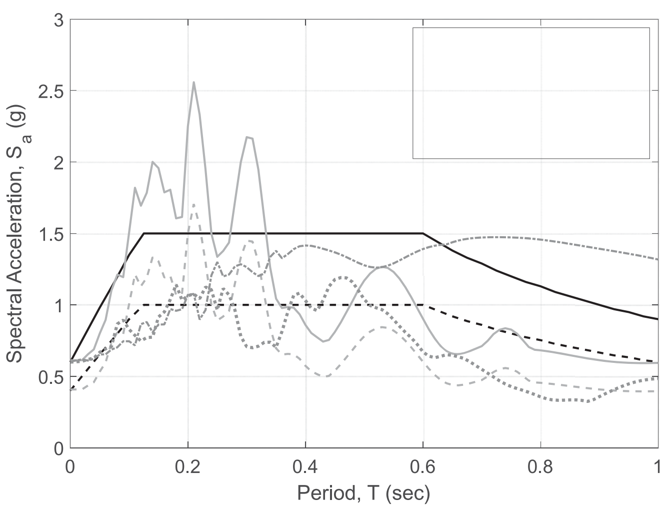

Fig. 23. Design and scaled ground motion acceleration response spectra.

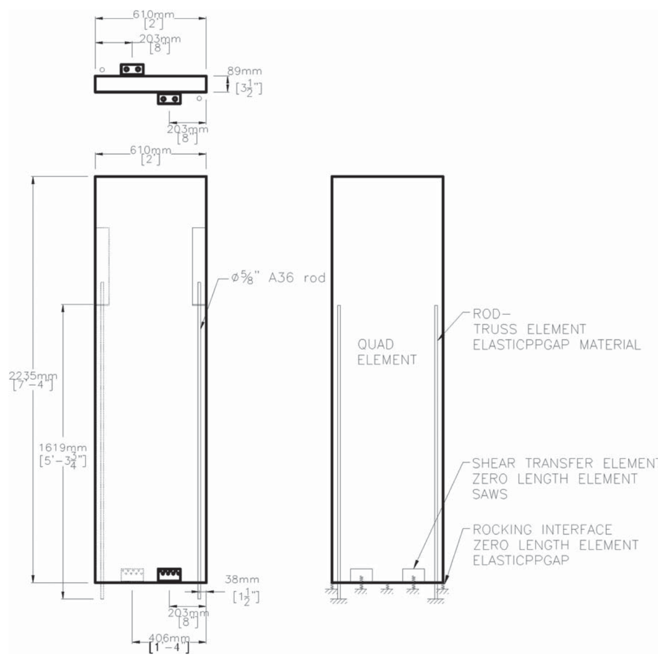

Fig. 24. Description of numerical model in OpenSees.

Table 3. Ground-motion test sequence and ISD

				Maximum ISD				
				Test		Numerical		
Test #	Ground motion	PGA	Component	mm (in.)	ISD[a] %	mm (in.)	ISD[a] %	% Difference
1	Loma Prieta	0.56	G03090 DE	21.70 (0.85)	0.97	28.50 (1.12)	1.28	31.3
2	Loma Prieta	0.56	G03090 MCE	38.60 (1.52)	1.73	36.50 (1.44)	1.63	5.4
3	Superstition Hills	0.36	B-ICC090 DE[b]	40.80 (1.61)	1.83	33.20 (1.31)	1.49	18.6
4	Northridge	0.87	85%NORTHR/RRS_032 DE[b]	45.20 (1.78)	2.02	52.50 (2.07)	2.35	16.2

Note: PGA = peak ground acceleration.
[a]Drift % calculated based on a story height of 2.24 m (7 ft 4 in.).
[b]Reduction in seismic intensity due to the total shake-table stroke length of 508 mm (20 in.).

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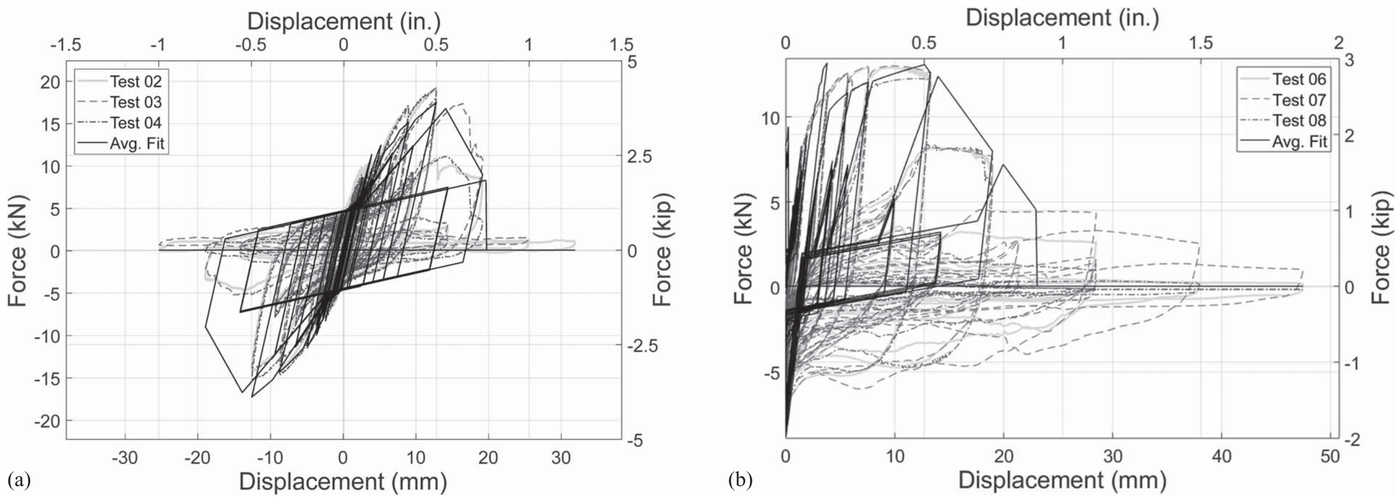

Fig. 25. (a) Average fit for shear tests; and (b) average fit for uplift tests.

Table 4. Connector SAWS parameters for numerical modeling

Test type	Test #	K_o kN/mm (kip/in.)	F_o kN (kip)	F_1 kN (kip)	r_1	r_2	r_3	r_4	Δ_u mm (in.)	α	β
Connector shear test	02	3.94 (22.5)	8.90 (2.00)	4.45 (1.00)	0.18	−0.35	1.00	0.05	12.70 (0.50)	0.75	1.10
	03	3.50 (20.00)	8.90 (2.00)	4.45 (1.00)	0.18	−0.35	1.00	0.05	14.0 (0.55)	0.75	1.10
	04	4.03 (23.00)	8.90 (2.00)	4.45 (1.00)	0.18	−0.50	1.00	0.05	12.7 (0.50)	0.75	1.10
	Avg.	3.82 (21.80)	8.90 (2.00)	4.45 (1.00)	0.18	−0.40	1.00	0.05	13.1 (0.52)	0.75	1.10
Connector uplift test	06	5.69 (32.50)	11.12 (2.50)	1.11 (0.25)	0.025	−0.15	1.00	0.02	12.7 (0.50)	0.60	1.05
	07	5.78 (33.00)	11.12 (2.50)	1.11 (0.25)	0.025	−0.15	1.00	0.02	12.7 (0.50)	0.60	1.05
	08	6.13 (35.00)	10.23 (2.30)	1.78 (0.40)	0.03	−0.15	1.00	0.02	12.7 (0.50)	0.60	1.05
	Avg.	5.87 (33.50)	10.82 (2.43)	1.33 (0.30)	0.03	−0.15	1.00	0.02	12.7 (0.50)	0.60	1.05

zero force, and has more initial stiffness, although, on average, the model is believed to capture the behavior of the cyclic data.

One aim of this study was to use connector- and wall-level data to develop a nonlinear dynamic model for this type of CLT wall in order to provide a time history response to earthquake motions. To achieve this, the OpenSees model described previously was subsequently analyzed in terms of the same earthquakes used in the shake-table test program, also described previously. A damping of 5% was applied, based on classic Rayleigh damping, using the mass matrix and the initial stiffness matrix. The results of the numerical model compared with the shake-table tests are shown in Fig. 27 and Table 3.

As no white-noise tests were performed after each test, the approximate period of the structure was calculated from the ISD-time history results (Fig. 27) by counting the number of cycles in a time period. This gave 0.34 s for the Loma Prieta DE tests and 0.43 s for the remaining tests, although 0.34 s was likely closer to the elastic period. For the numerical model, the corresponding periods were calculated as 0.55, 0.55, 0.58, and 0.63 s, respectively. Comparing the figures, the difference between the excitation in the numerical model and the experimental results after the ground motion had subsided (i.e., the last 10 s of the plots) is notable. This difference can be attributed to the energy loss from contact damping, which was not explicitly modeled, with equivalent viscous damping being used instead to capture the overall effect.

An inspection of Fig. 27 reveals that the numerical model was able to accurately reproduce the experimental seismic behavior of the system. The time history of the ISD and the maximum and minimum drifts for the numerical model and shake-table data are similar. The difference between the numerical model and the test data

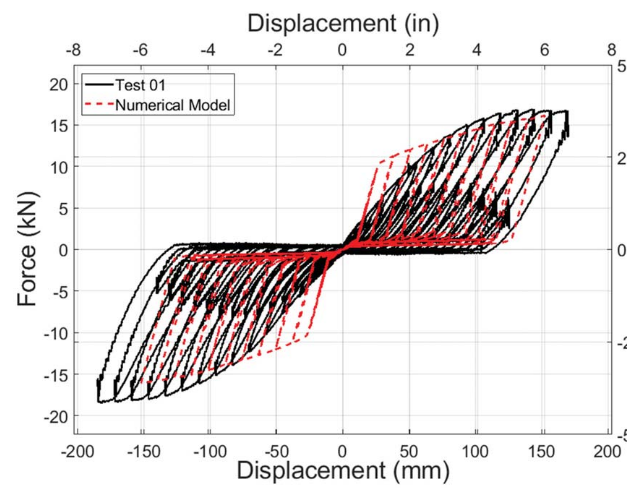

Fig. 26. Test 01 hysteresis versus numerical model.

for maximum ISD ranged from 5.4% to 31.3%, with an average of 17.9%. These differences can be attributed to the idealized modeling of the hold-downs and the perpendicular-to-the-grain behavior of the CLT base. Friction may also have played a role, albeit a very small one, as the CLT wall exhibited rocking behavior. The results accord with similar studies that investigated the performance of one-story (Hristovski et al. 2012) and three- and seven-story (Rinaldin and Fragiacomo 2016) CLT structures. The difference between the experimental results and the numerical model for the

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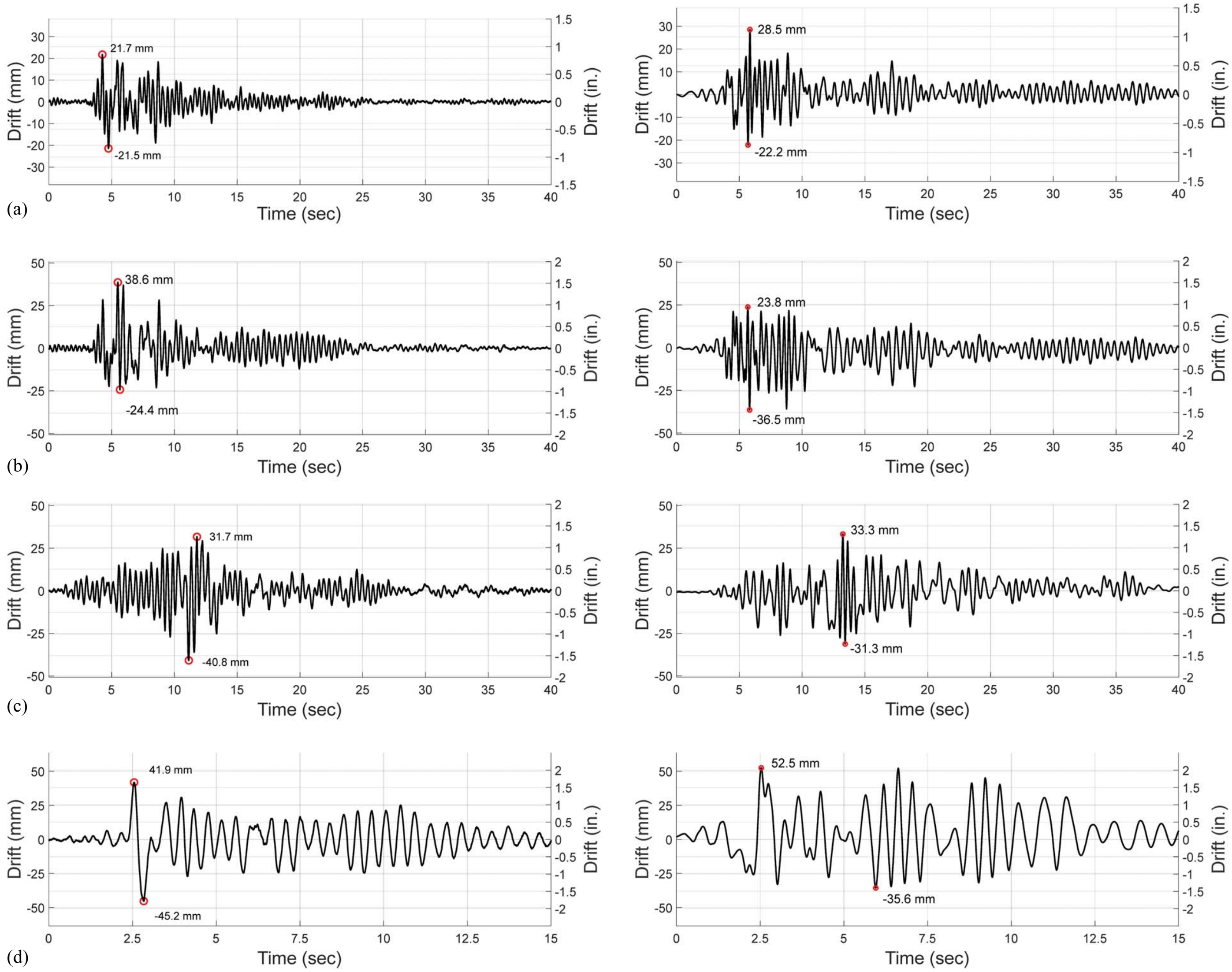

Fig. 27. (left) Experimental versus (right) numerical results: (a) Loma Prieta, DE; (b) Loma Prieta, MCE; (c) Supersititon Hills, DE; and (d) 85% Northridge, DE.

maximum ISD ranged from 1% to 30.6% and from 1.7% to 47.9% in these two studies, respectively.

As there was reasonable agreement between the numerical model and the experiments, and because the results accord with the accuracy of existing CLT wall and building models, the modeling approach presented here may be used for analyzing the viability of this retrofit option.

Conclusion

There is a dearth of data available for understanding the behavior of CLT rocking walls, which this study has helped to address. A systematic testing program, consisting of connector, CLT shear-wall and wall-level shake-table tests, was conducted, and a numerical model was developed and validated based on these tests. The CLT walls were not post-tensioned. The quasistatic reversed cyclic CLT rocking-wall tests exhibited predominantly rocking behavior, which is attributed to the high aspect ratio (height/length) of the panel. The shake-table tests demonstrated that a high-aspect-ratio rocking wall, with generic connectors and an appropriate overturning moment restraint, can withstand DE- and MCE-level grounds motions. The finite-element (FE) model used in this study was in reasonable agreement with the test results, where the difference ranged from 5.4% to 31.3%, with an average of 17.9%. This was determined to be in accord with the findings of similar studies. Although the stand-alone applicability of this system has yet to be determined, previous shake-table tests have suggested that this system can be effectively utilized in retrofitting soft-story structures. In can also be used in conjunction with light-frame shear walls to increase the strength and stiffness of the shear-wall line. Perhaps the most salient point is that this system can be installed by a typical wood-construction crew without any special equipment on site because the recentering system is not post-tensioned, relying instead on components readily available for housing and light commercial wood-building. The potential applications of this system can be further studied at the building level through the proposed modeling approach. In addition, the test-based performance baseline, established through the connector testing, may be used to determine the equivalency of other types of connectors, and their potential use in and impact on such a system.

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Data Availability Statement

Some or all the data, models and code that support the findings of this study are available from the corresponding author upon reasonable request. These include the test data and additional photos and videos.

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