

Systematic experimental investigation to support the development of seismic performance factors for cross laminated timber shear wall systems

M. Omar Amini^{a,*}, John W. van de Lindt^a, Douglas Rammer^b, Shiling Pei^c, Philip Line^d, Marjan Popovski^e

^a Department of Civil and Environmental Engineering, Colorado State University, Fort Collins, CO 80523, USA

^b Forest Products Laboratory, One Gifford Pinchot Drive, Madison, WI 53726, USA

^c Department of Civil and Environmental Engineering, Colorado School of Mines, Golden, CO 80401, USA

^d American Wood Council, 222 Catocin Circle SE, Suite 201, Leesburg, VA 20175, USA

^e Advanced Building Systems Department, FPInnovations, 2665 East Mall, Vancouver, BC V6T 1Z4, Canada

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ABSTRACT

In the US, codified seismic design procedure requires the use of seismic performance factors which are currently not available for CLT shear wall systems. The study presented herein focuses on the determination of seismic design factors for CLT shear walls in platform type construction using the FEMA P-695 process. Results from the study will be proposed for implementation in the seismic design codes in the US. The project approach is outlined and selected results of full-scale shear wall testing are presented and discussed. Archetype development, which is required as part of the FEMA P-695 process, is briefly explained with an example. Quasi-static cyclic tests were conducted on CLT shear walls to systematically investigate the effects of various parameters. The key aspect of these tests is that they systematically investigate each potential modelling attribute that is judged within the FEMA P-695 uncertainty quantification process. Boundary constraints and gravity loading were both found to have a beneficial effect on the wall performance, i.e. higher strength and deformation capacity. Higher aspect ratio panels (4:1) demonstrated lower stiffness and substantially larger deformation capacity compared to moderate aspect ratio panels (2:1). However, based on the test results there is likely a lower bound for aspect ratio (at 2:1) where it ceases to benefit deformation capacity of the wall. This is due to the transition of the wall behaviour from rocking to sliding. Phenomenological models were used in modelling CLT shear walls. Archetype selection and analysis procedure was demonstrated and nonlinear time history analysis was conducted using different wall configurations.

1. Introduction

Since its initial introduction in Europe in the early 1990s and subsequent entry into the building market between 2000 and 2005, Cross Laminated Timber (CLT) has now been commonly accepted as a new-generation engineered wood product that has great potential to expand the wood building market [1].

This innovative mass timber product, sometimes termed X-Lam, offers a number of advantages such as the potential for mass production, prefabrication, speed of construction and sustainability as an environmentally friendly and renewable construction product. Good thermal insulation, acoustic performance, and fire ratings are some additional benefits of the system [2,3].

Despite these advantages, the lack of a current design approach is

one of the challenges inhibiting widespread adoption of CLT in North America. Mohammad et al. [4] identified a multi-level strategy that includes development of a product standard, material design standard, and their subsequent adoption into the building codes. In the US, there has been recent development on all these fronts that included publication of ANSI/APA PRG320, the North American Standard for Performance-Rated Cross-Laminated Timber [5], addition of a chapter dedicated to CLT in the 2015 edition of the National Design Specification for Wood Construction® (NDS®) [6] and recognition of CLT in the 2015 International Building Code [7].

One area that requires attention is the development of seismic performance factors for CLT lateral systems so designers in the US can begin to utilize CLT shear walls in seismic regions. Recent research efforts on seismic performance of CLT buildings can be found in Europe,

* Corresponding author.

E-mail addresses: omar.amini@rams.colostate.edu (M.O. Amini), jwv@engr.colostate.edu (J.W. van de Lindt), drammer@fs.fed.us (D. Rammer), spei@mines.edu (S. Pei), pline@awc.org (P. Line), marjan.popovski@fpinnovations.ca (M. Popovski).

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North America and Japan, with a comprehensive review done by Pei et al. [8]. These include systematic research studies in Slovenia and Macedonia [9–12], the SOFIE project in Italy [3,13–15], studies in North America [16–18], and efforts in Japan [19–21]. For additional studies focused on various aspects of CLT seismic force resisting systems, readers are referred to Gavric et al. [22,23], Tomasi and Smith [24], Izzi et al. [25,26], Pozza and Trutalli [27], and Pozza et al. [28,29].

With the introduction of CLT into the US construction market, many researchers believe that CLT can serve to fill a gap for certain building stock need in the US; specifically, the mid-rise condominium, commercial, and mixed-use building market in seismic regions. CLT based Seismic Force Resisting Systems (SFRS) are not recognized in current US design codes. CLT shear walls cannot be designed via the equivalent lateral force (ELF) design procedures [30]; therefore use of CLT for seismic force resistance can only be accomplished through alternative methods. This approach, however, is usually more costly, making CLT less competitive against other conventional structural systems. A research project at Colorado State University; funded by the USDA Forest Products Laboratory (FPL), is targeted at determining ELF seismic performance factors for CLT shear wall as a SFRS. The study follows the FEMA P-695 [31] methodology which is a systematic approach that integrates design method, experimental results, nonlinear static and dynamic analyses and incorporates uncertainties. Various phases of the project consist of development of the archetypes, design methodology, testing, modelling, and analyses. The testing phase of the project included two main phases, namely (i) connector testing (ii) and CLT shear wall testing. Test data is then used to refine the design methodology and calibrate the proposed numerical models for connectors and CLT shear walls. This paper focuses on the wall-level experimental phase and building archetype development of the project. Specifically, each test comparison presented herein leads to either inclusion or exclusion in the numerical modelling portion of the FEMA P-695 methodology; and is also included in the uncertainty quantification process.

2. Overview of the FEMA P-695 methodology

FEMA P695 is a methodology to evaluate ELF seismic performance factors (SPF), which include the response modification factor (R-factor), the system overstrength factor (Ω), and the deflection amplification factor (C_d) for seismic design in the United States. R is defined as the ratio of the shear developed in the system if the system were to remain entirely linearly elastic under design ground motions, V_E to the design base shear value V . Ω_o is the ratio of maximum shear strength V_{max} of the yielded system to the design base shear. C_d is defined as the ratio of the roof drift of the yielded system under design earthquake ground motions δ to the roof drift under design base shear considering the system to behave linearly elastic δ_E , multiplied by the R factor. SPFs are best described using the following equations and illustrated in Fig. 1.

R = Response Modification Coefficient = V_E/V .

Ω_o = Overstrength Factor = V_{max}/V .

C_d = Deflection Amplification Factor = $(\delta/\delta_E) R$.

The methodology is an iterative process that consists of nonlinear static and dynamic analyses on a number of archetypes that are prototypical representations of the seismic force resisting system. Critical to the P695 process is that the archetypes used within the analyses must comprehensively represent the anticipated design space for the SFRS proposed for inclusion in the US design codes. These analyses result in computing the so-called “margin against collapse” of each archetype and hence the proposed system with specific requirements dictated by FEMA P695. It takes into account uncertainties inherent in the test data and modelling methods as well as inherent variability in the suite of ground motion records. This iterative process is illustrated in Fig. 2.

A key aspect of the methodology is that it is overseen by a technical

peer panel and their involvement is critical throughout. It culminates in a project report along with the peer panel review that is then used to support modification of relevant US design codes.

3. Experimental procedures

The FEMA P-695 methodology requires testing to reliably capture behaviour of the proposed system. Tests include material testing, components and connections, and assembly and system level tests. Material testing is not conducted as part of this project because material design strength is in accordance with the ANSI/APA PRG 320 [5] product performance standard. One critical aspect of this project is the use of non-proprietary components and connectors already addressed by US design codes to facilitate building code recognition but also to provide a test-based performance baseline to allow for proprietary (and other) systems to demonstrate equivalence.

3.1. Connector testing

Investigating connector behaviour is important since the CLT wall and lateral system responses are greatly influenced by the connector layout and properties [9,14]. CLT panels exhibit linear elastic behaviour and the energy dissipation and ductility is primarily achieved through the connectors. A generic connector is used throughout the P-695 process for CLT described herein to ensure applicability of the US design codes and to provide a test-based performance baseline to allow for proprietary (and other) systems to demonstrate equivalence. Most of the connector tests from other studies to date have been performed using proprietary metal connectors and fasteners. The connector testing includes two types: the angle bracket connectors and the inter-panel connectors. Angle brackets were tested under shear and uplift, while inter-panel connectors were tested in shear only. However, in this study, metal connectors (i.e. angle brackets and inter-panel connectors) were manufactured from sheet steel in the machine shop at CSU and commodity nails were used in metal connectors to enable the connector testing to be generic. Steel angle brackets used for attachment of the wall to the supporting element is shown in Fig. 3. These generic connectors are designed per steel design standards and the National Design Specification® (NDS®) for Wood Construction [6] such that the nails yield under loading and pull out of the CLT panel. The CLT shear wall design method requires adequate embedment of the fasteners to ensure Mode III or Mode IV yielding per NDS.

Shear and uplift tests for angle brackets were performed under monotonic and cyclic loading. All shear tests were conducted under displacement control using the CUREE protocol [32], shown in Fig. 4, with the reference displacement obtained from a monotonic test. The reference displacement is defined as the deformation at which the load drops below 80% of the maximum load applied to the specimen. In the case of uplift tests, the specimen was subjected to a single-sided CUREE loading protocol. In order to reliably capture statistical variability in the tests, one monotonic and ten cyclic tests were performed for each connection configuration. Two different grades of CLT, E1 and V2, based on ANSI/APA PRG320 [5] were also considered in the testing. For E1 grade the parallel layers are Spruce-pine-fir and perpendicular layers are No.3 SPF while for V2 grade the laminations are No. 1/No. 2 SPF and No. 3 SPF in parallel and perpendicular directions, respectively. The testing matrix is provided in Table 1 and the testing configuration is shown in the schematic in Fig. 5. Connectors performed as intended and nail yielding and withdrawal was observed during the testing. Fig. 6 shows an A3 type connector before and after the tests.

3.2. CLT shear wall tests

CLT shear wall tests were performed with the same generic connectors used in the connector testing. The purpose of these tests was to systematically investigate the influence of various factors on the

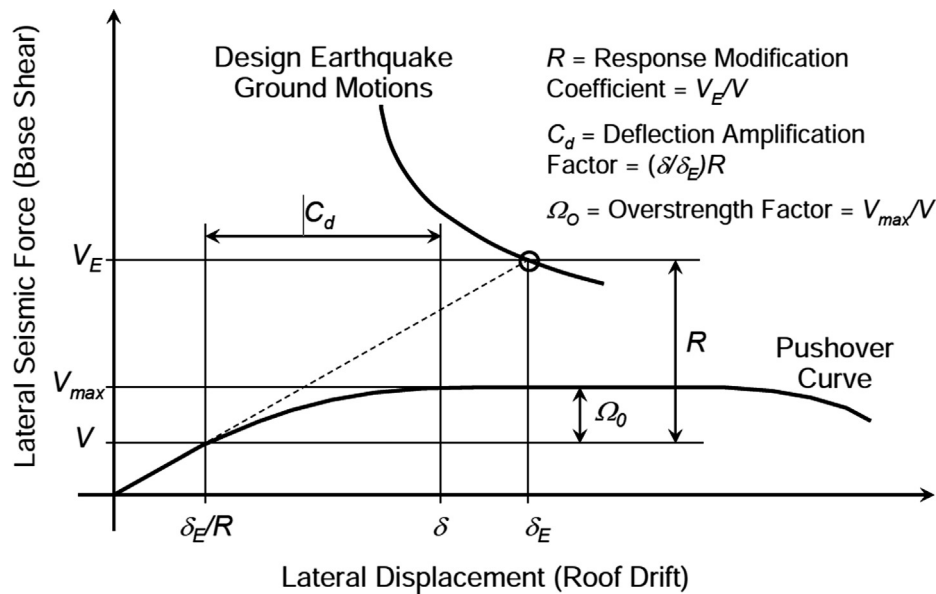


Fig. 1. Illustration of Seismic Performance Factors (after FEMA P695).

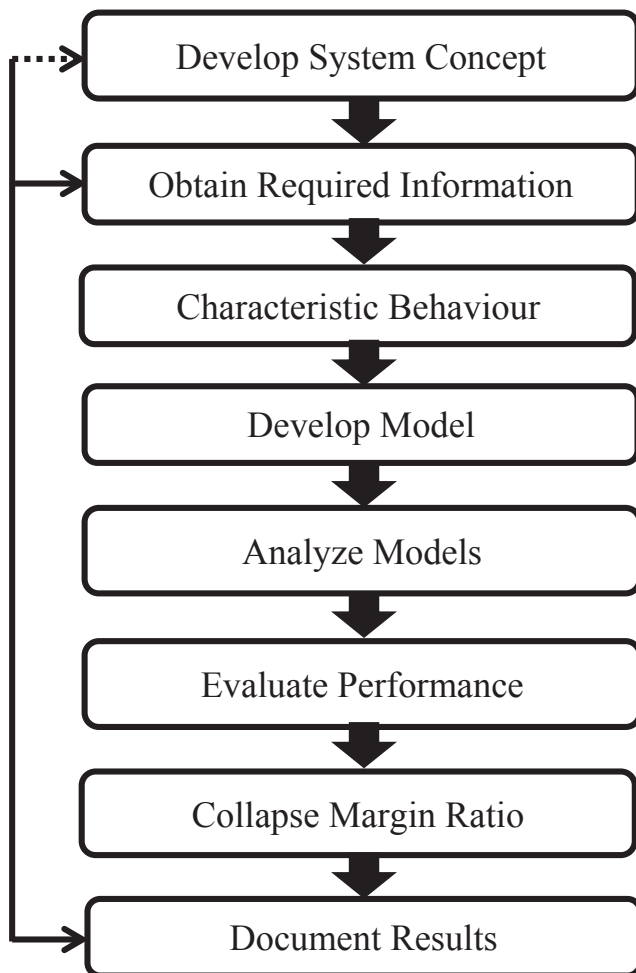


Fig. 2. Overview of the FEMA P-695 methodology.

behaviour of the wall in terms of strength, stiffness, deformation capacity, and energy dissipation. The influential factors considered are: boundary condition of the CLT shear wall imposed by the CLT diaphragm, presence of gravity loading, connector type, connector plate

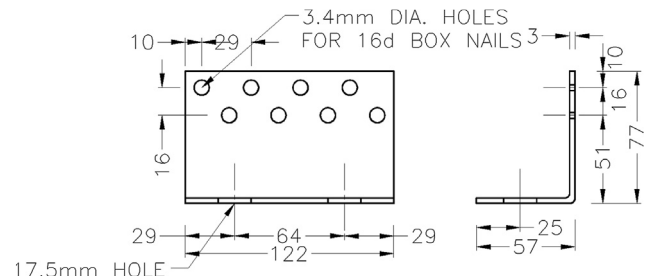


Fig. 3. A3 type connector with (8) 16d box nails (dimensions in mm).

thickness, CLT grade, CLT panel aspect ratio, panel thickness, and presence of inter-panel connector (vertical joint). The main design assumption for these walls as dictated by the CLT shear wall design approach is that all overturning is resisted by overturning anchor (tie rod or hold-downs) at the ends of wall, and that shear is resisted by the angle brackets. As a note, this type of assumption was also utilized in the initial stages of the Italian SOFIE project [3]. However, subsequent studies [22,33] showed the axial contribution of the connectors to be significant. Regardless, the approach adopted in the FEMA P695 study and its design approach aligns well with the already established method for light frame wood shear walls and a comparison of different analytical models with experimental data [23] showed this approach to be conservative. In addition, this assumption, while conservative, will provide designers an easier application in design.

The test matrix is provided in Table 2 and includes information on the type of CLT, panel length, panel thickness, number of connectors and applied gravity load. The wall tests were conducted using the same connectors tested under shear and uplift and the test setup is shown in Fig. 7. Vertical actuators are under force control to apply gravity load (when it was included) while horizontal actuators were under displacement control and applied the CUREE loading protocol [32] based on the reference displacement, Δ_{ref} , of 1 in. A CLT base was utilized under the walls to replicate a typical floor condition and to give insight into crushing perpendicular to the grain on the floor when the panel exhibits rocking behaviour. Similarly, a top CLT panel was used as the loader bar to replicate a floor diaphragm. Lateral roller guides were provided to avoid any out-of-plane movement of the wall. Measurements included displacement and uplift at the bottom of the wall as well as uplift on the other end, uplift at some of the connectors, in

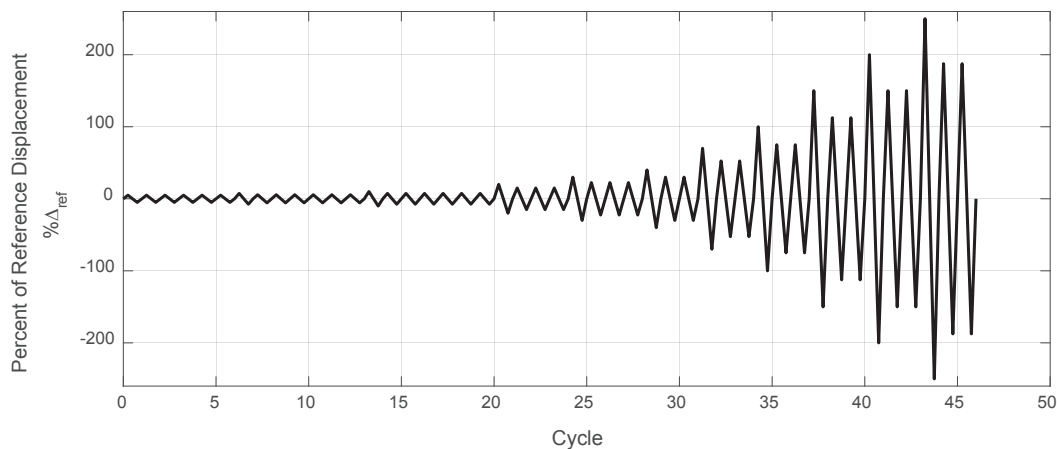


Fig. 4. CUREE loading protocol.

Table 1

Connector test matrix.

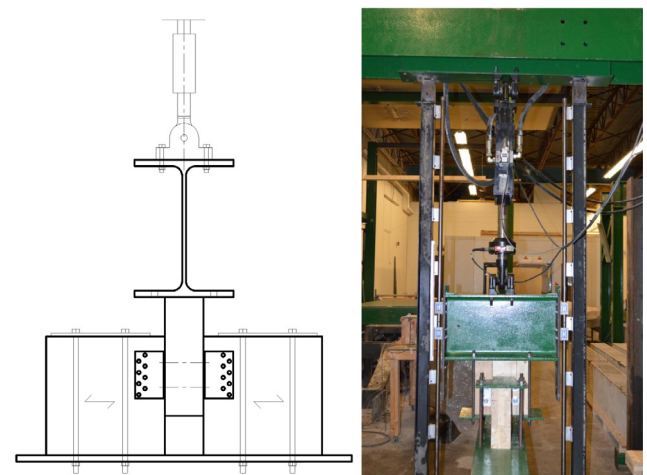
Test type	Connector type	CLT grade	Tests
Shear	A3- (8)16d box nails 88.9 mm × 3.43 mm (3.5 in. × 0.135 in.) and two 15.9 mm (5/8") rods	E1 V2	One monotonic and 10 cyclic
Uplift	A3- (8)16d box nails 88.9 mm × 3.43 mm (3.5 in. × 0.135 in.) and two 15.9 mm (5/8") rods	E1 V2	One monotonic and 10 non-reversed

certain tests, deformation of the wall along the diagonal, and strain gauges on the hold-down rods. Similar to the results obtained from [14,16] CLT panels exhibited rigid behaviour and energy dissipation occurred in the connectors.

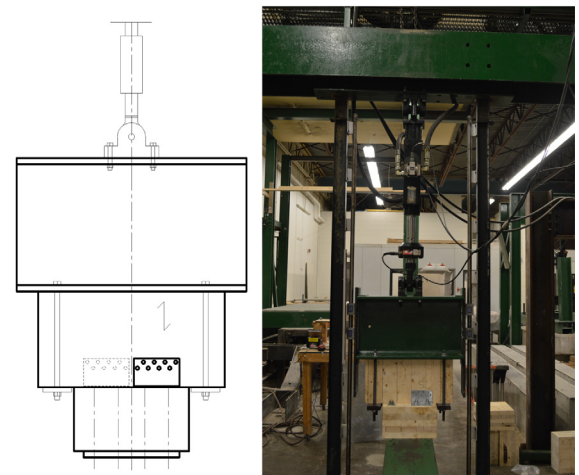
Tests results were analyzed based on the procedure in FEMA P795 [34] which is similar to ASTM E 2126 [35] except for the definition of Δ_{yield} . FEMA P795 is the complementary methodology to FEMA P695 and will likely be used for connectors to show performance equivalence once the FEMA P695 process is completed. Average parameters for the positive and negative excursions are reported in Table 3.

3.2.1. Boundary conditions

Based on the FEMA P695 methodology requirements, the test boundary condition should be representative of typical construction provided it does not result in any beneficial effects. In the case of CLT shear walls an important boundary condition is the interface between the wall and the floor diaphragm. The size of the diaphragm is believed to affect the wall behaviour under cyclic loading since the diaphragm in a structure may be larger compared to the walls and therefore may remain relatively horizontal throughout the lateral motion induced by an earthquake. Rocking of the wall panel may create a gap between the wall panel and the diaphragm. In order to quantify the effect of a top boundary condition, modifications were made to the original test setup, shown in Fig. 7, to include the effect of a top diaphragm into the isolated shear wall test. This was done by adding supports to allow sliding of the top CLT panel while keeping it horizontal during the shear loading. The supports, shown in Fig. 8, consisted of four load cells on each end with acetel polymer plates on top. Load cells were added to determine the effect of friction and allow adjustment of the horizontal actuator force if it is deemed significant. In order to determine the coefficient of friction of the acetel polymer plates, a total of 10 tests, each with three levels of increasing vertical load, were performed. The test results yielded an average friction coefficient of 0.3. Once the results were obtained for the friction tests, two specific tests, Tests 05 and 06, were conducted to investigate the effect of boundary condition on



(a) Shear test configuration



(b) Uplift test configuration

Fig. 5. Configuration for connector tests.

CLT hysteresis. Test 05 was performed without the imposed boundary condition while Test 06 included the boundary condition and thus the force values obtained from Test 06 were adjusted for friction. Connector failure in the wall test is shown in Fig. 9 and the hysteresis for both of these tests is provided in Fig. 10. From inspection of the hysteresis plots, it was found that the test without the boundary condition imposed

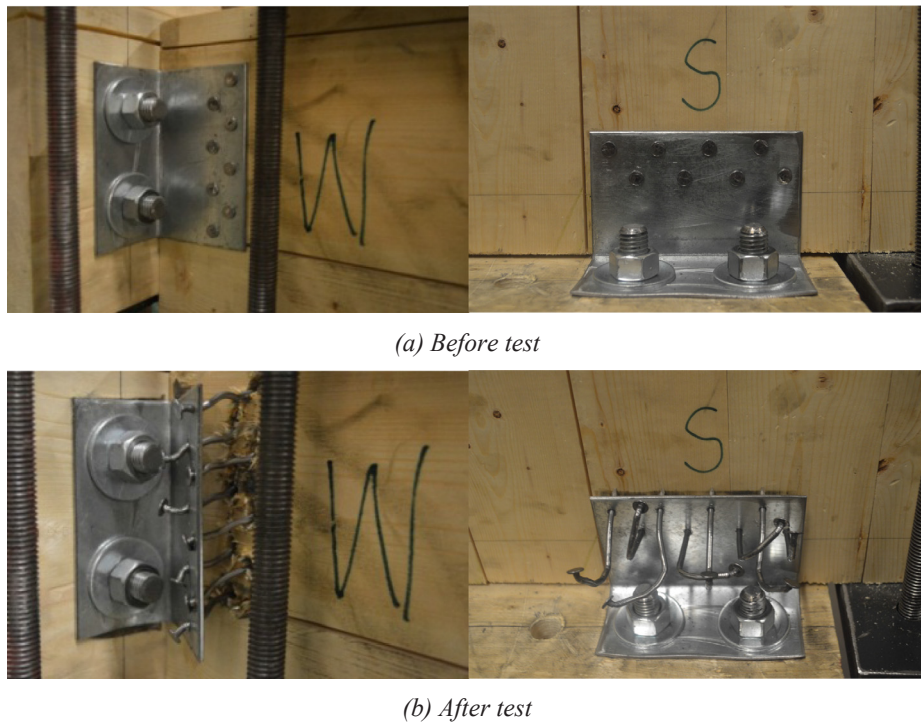


Fig. 6. Wall-to-floor angle bracket shear and uplift tests.

produced similar load deformation response with only slight differences in strength, stiffness, and displacement capacity. As a result, additional testing utilized the less complex test set-up without the boundary condition imposed.

3.2.2. Gravity loads

Gravity load can also affect CLT wall component behaviour and therefore, several tests were performed to determine its effect on the isolated CLT wall tests. A $1.22 \text{ m} \times 2.44 \text{ m} \times 169 \text{ mm}$ CLT wall under three levels of vertical loads that include no gravity, 0.922 kN/m (0.68 kip/ft), and 1.84 kN/m (1.28 kip/ft) were tested and the results of are shown in Fig. 11. These are tests 09, 03, and 04, respectively, in Table 2. From Fig. 11 one can see that an increase in gravity leads to an increase in stiffness of the panel and a slight increase in strength. As a

result, gravity load was removed from the remainder of the tests for conservativeness. The archetype development, which will be explained in more detail later in this paper, is in general a multi-story wall line stack and thus the analysis is two-dimensional.

3.2.3. CLT grade

The effect of CLT grade was investigated by comparing the results of Test 09 with Test 14 and results from Test 10 with Test 17, although the thicknesses are different in the case of the latter comparison. Results are shown in Figs. 12 and 13. Based on the hysteresis, as one would expect, CLT grade has an influence on strength and stiffness of the CLT panels when the exact same connectors and fasteners are used. A similar trend is observed by comparing Tests 11 and 15; however, the hysteresis curves are not shown herein. The strength of wood is significantly

Table 2
Isolated CLT shear wall test matrix.

Test #	Grade & panel #	Height (m)	Length (m)	# Plys	Thickness (mm)	No. connectors*	Gravity load (kN/m)
03	V2	2.44	1.22	5	169	3	0.92
04	V2	2.44	1.22	5	169	3	1.84
05	E1	2.44	1.22	5	175	3	0.92
06**	E1	2.44	1.22	5	175	3	0.92
09	V2	2.44	1.22	5	169	3	–
10	V2	2.44	1.22	3	99	4	–
11	V2	2.44	1.22	5	169	2	–
13	E1	2.44	1.22	5	175	2	–
14	E1	2.44	1.22	5	175	3	–
15	E1	2.44	1.22	5	175	2	–
17	E1	2.44	1.22	5	175	4	–
18	V2	2.44	1.22	3	99	2	–
19	V2	2.44	1.22	3	99	5	–
20	V2	2.44	1.22	7	239	5	–
21	V2	2.44	0.61	3	99	2	–
22	V2	2.44	2.44	5	99	4	–
23	V2	2.44	0.61 (2)	5	169	4	–
26	V2	2.44	0.61 (4)	5	169	8	–

* All the connector types are A3; Connectors in all the tests are evenly spaced along the CLT panel.

** Only Test 06 was performed with the imposed boundary constraint.

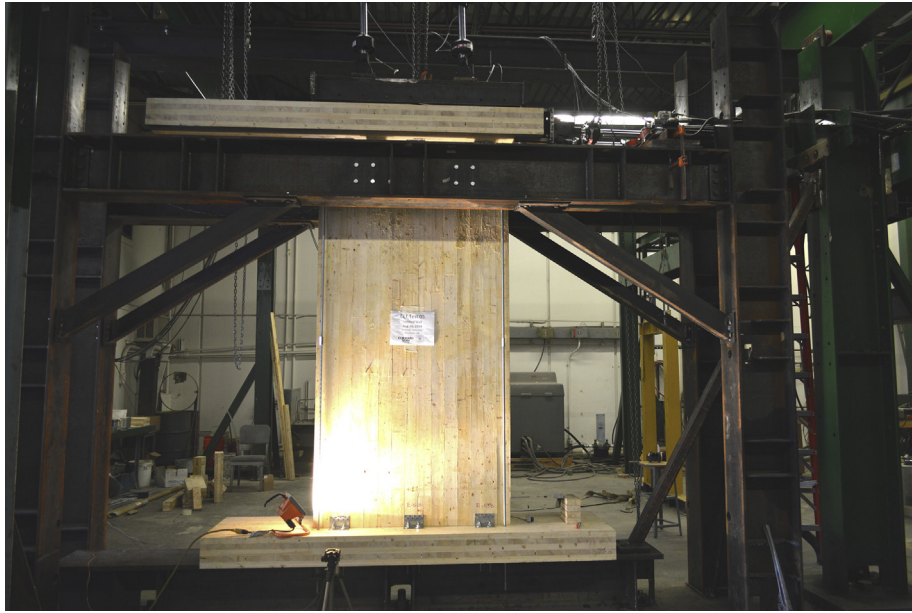


Fig. 7. Isolated shear wall test setup.

influenced by its specific gravity; the higher the specific gravity the denser the wood resulting in higher strength and stiffness values for the fasteners. The effect of CLT grade was partly attributed to the specific gravity of these different grades of CLT which were determined in accordance with ASTM D2395 [36]. E1 grade was found to have on average a higher specific gravity than the V2 grade. Specified SG for each grade is 0.42 in accordance with NDS; however, measured values were 0.47 and 0.45 for E1 grade and 0.45 and 0.37 for V2 grade for outer layer and inner layer, respectively.

3.2.4. Panel thickness

Tests 19 and 20 were performed to examine the effect of panel thickness on overall wall behaviour. Because the CLT shear wall is a rocking system, compression perpendicular to the grain of the CLT floor panel is very likely to have an effect on the rocking behaviour. Fig. 14 indicates that there is only a slight difference in the initial stiffness and maximum strength of different thickness panels with the thicker panel being stronger and stiffer of the two. A similar trend was observed by

comparing the results of Tests 11 and 18, shown in Fig. 15; albeit in this case the difference was less significant. Referring to Table 2, it is important to note that Fig. 14 presents tests with 5 connectors and Fig. 15 presents tests with 2 connectors, thus the vertical axis of the plots is quite different.

3.2.5. Panel aspect ratio

In order to determine the effect of panel aspect ratio, Tests 18 and 21 were performed and the hysteresis compared in Fig. 16. Results indicate that while higher aspect ratio panel (4:1) exhibited less stiffness and somewhat smaller strength, it had more deformation capacity than the lower aspect ratio panel (2:1) and pinched significantly. This added deformation capacity can be attributed to the rocking behaviour of the panel as opposed to rocking and sliding mechanism of other panels tested. However, comparing Tests 22 and 10 in Fig. 17, the difference between a 1:1 and 2:1 panel aspect ratio is minimal. This indicates that there is a lower bound on aspect ratio where it has an insignificant effect on the panel behaviour. This can be attributed to the

Table 3
Cyclic envelope parameters from the CLT shear wall testing.

Test #	Initial stiffness K_i (kN/mm)	Effective yield $\Delta_{y,eff}$ (mm)	Ultimate load $F_{max}(kN)^*$	Displacement corresponding to ultimate load $\Delta_{Fmax}(mm)$	Ultimate deformation (0.8 $F_{max}) \Delta_u(mm)^{**}$	Effective ductility capacity, $\mu_{eff} (\Delta_u/\Delta_{y,eff})$
03	1.17	55.9	65.43	94.5	104.9	1.88
04	1.13	58.9	66.54	94.2	102.4	1.73
05	1.49	54.1	80.24	122.7	152.4	2.82
06	1.15	77.0	85.05	106.9	149.4	1.94
09	0.79	75.7	59.51	99.8	104.9	1.39
10	1.08	58.4	63.03	88.6	94.7	1.62
11	0.66	57.7	37.23	95.0	104.1	1.81
13	0.59	69.1	40.88	118.6	133.4	1.93
14	1.27	63.8	81.26	131.8	139.7	2.20
15	0.69	69.6	47.19	137.7	139.7	2.01
17	1.33	78.2	103.42	114.0	120.7	1.54
18	0.67	47.0	31.22	75.7	95.3	2.03
19	1.32	58.2	76.46	88.9	99.6	1.71
20	1.78	47.2	84.07	75.9	88.9	1.88
21	0.26	112.5	28.91	163.8	177.8	1.58
22	0.96	65.0	62.23	75.7	87.6	1.35
23	0.99	63.8	62.89	120.4	177.8	2.79
26	2.84	37.6	106.89	113.5	165.1	4.39

* Maximum load applied to the component during cyclic loading (FEMA P795).

** In the cases where no descending branch was observed, Δ_u was taken as the maximum deformation executed in the test.

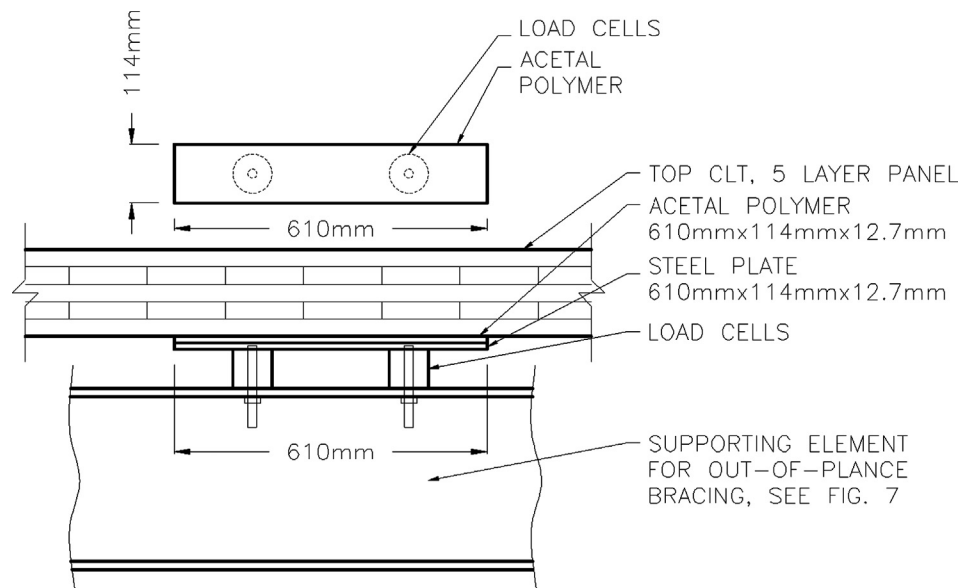


Fig. 8. Floor diaphragm supports.



Fig. 9. Connector failure shear wall Test 05.

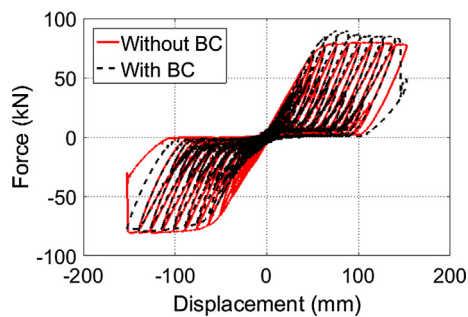


Fig. 10. Test 05 vs. 06- Hysteresis for shear wall tests with and without floor diaphragms.

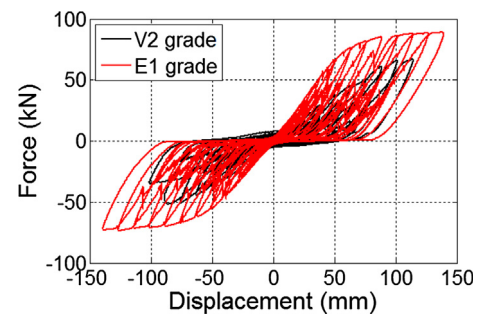


Fig. 12. Test 09 vs. 14-Hysteresis for shear wall tests on two different grades of CLT.

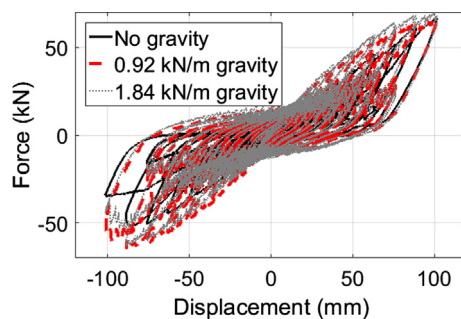


Fig. 11. 1.22 m x 2.44 m x 169 mm shear wall specimen tested under different vertical loading.

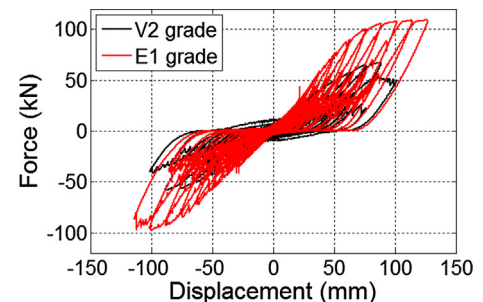


Fig. 13. Test 10 vs. 17-Hysteresis for shear wall tests on two different grades of CLT.

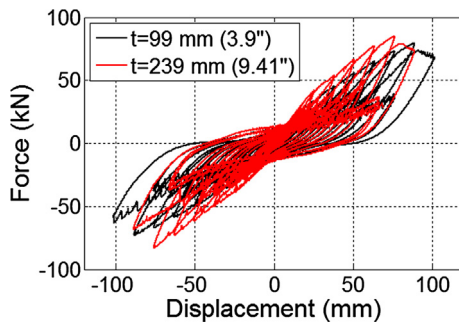


Fig. 14. Test 19 vs. 20-Hysteresis for shear walls with different panel thicknesses.

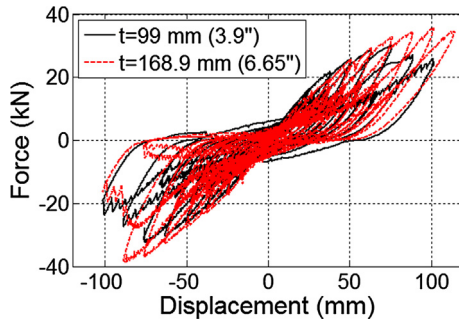


Fig. 15. Test 11 vs. 18-Hysteresis for shear walls with different panel thicknesses.

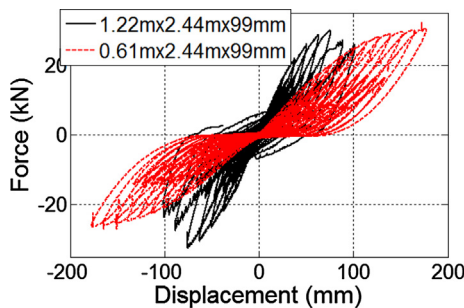


Fig. 16. Tests 18 vs. 21- Hysteresis for shear wall with 2:1 and 4:1 aspect ratio panels.

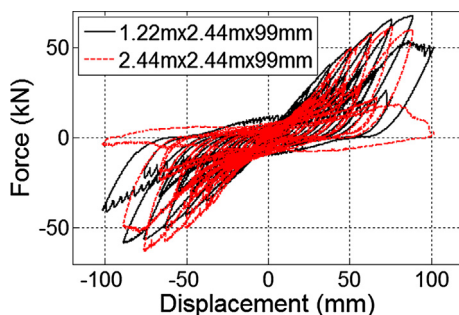


Fig. 17. Tests 22 vs. 10- Hysteresis for shear wall with 1:1 and 2:1 aspect ratio panel.

predominately rocking, a combination of rocking and sliding, and sliding behaviour resulting in deformation which correspond to the aspect ratio of the panels that are 4:1, 2:1, and 1:1, respectively.

3.2.6. Inter-panel connectors

The influence of inter-panel connectors was examined by comparing

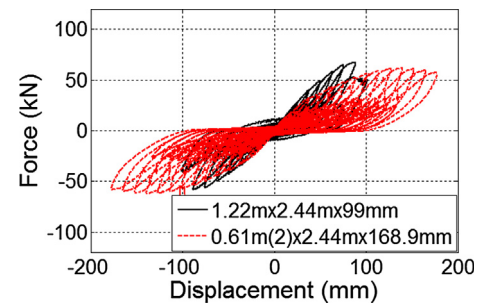


Fig. 18. Test 23 vs. 10- Hysteresis for shear wall with 1.22 m panel and (2) 0.61 m panels.

Tests 23 and 10, although panel thicknesses are different in these two tests it was shown previously that this is not significant. As seen in Fig. 18, inter-panel connectors add to the deformation capacity of the wall comprised of higher aspect ratio panels, but remain very close in peak capacity with only a slight reduction. This is likely due to reduced stiffness of the inter-panel connectors relative to the holddowns which leads to more uplift demand on the base connectors as the panel rocks. Connectors for the vertical joints are sought to provide equivalent shear capacity to that of the angle brackets used in the base and top of the wall, A3 type connectors in this case. Use of alternatives such as LVL or half-lap joints are permissible under the methodology if equivalence is demonstrated through application of the FEMA P795 [34] methodology. The vertical joint is designed to yield before the shear capacity of the base connectors are reached resulting in rocking of the individual panels. This rocking behaviour is intended as part of the CLT shear design method. This behaviour was observed in Tests 23 and 26 and is shown in Figs. 19 and 20, respectively. The hysteresis for Test 26 is shown in Fig. 21 and as seen vertical joints add to the deformation capacity of the wall.

4. Archetype development

Archetype development is an essential part of the FEMA P-695 methodology and the purpose is to study typical applications of the proposed seismic force resisting system and verify its performance. It is critical within the process that the design space for CLT be comprehensively represented to ensure a true suite of archetypes are used in the extensive analysis. Unique and irregular configurations are dealt with on case by case basis, but overall the objective is to ensure that development of new seismic performance factors address a full range of end-use applications for the proposed system and provide a level of life safely and collapse prevention currently available in modern US design codes and standards.

Index buildings that include single family dwellings, multi-family dwellings, and commercial buildings and were developed as an initial part of the process to define the design space. Based on the FEMA P-695 methodology, two-dimensional archetypes are considered acceptable to represent wood walls and for the purpose of this project archetypes are defined as two-dimensional multi-story wall lines. Archetypes extracted from these index buildings are designed and detailed based on the proposed design methodology and modelled using the nonlinear numerical models. Archetype models representing a mathematical idealization of the proposed system are then analyzed under nonlinear static and dynamic loadings.

Table 4 lists a range of design parameters used in the development of archetype configurations. The number of stories ranged between 1 and 12 with story heights of 3.05 m and the proposed system was considered for seismic design category (SDC) D (ASCE 7-16). Non-structural wall finishes are not included in the archetypes because they are not defined as part of the seismic force resisting system. The aspect ratio refers to the aspect ratio of individual CLT panels and longer shear



Fig. 19. Test 23- shear wall with (2) 0.61 m wall panels with vertical joint.



Fig. 20. Test 26- shear wall with (4)0.61 m wall panels with vertical joints.

walls are comprised of multiple high aspect ratio CLT panels that are connected through vertical joints. Archetypes are assembled into various performance groups based on configuration, seismic design category, gravity load, and building height variations. Each performance group includes at least three index archetype configurations and for the purpose of this study there are a total of 96 performance groups. However, the number of performance groups and hence the archetypes analyzed are systematically reduced for analysis. Since the system is intended for seismic design category (SDC) D, this requires evaluation for SDC D_{max} and SDC D_{min} . Once the preliminary analysis shows that the system performs acceptably for SDC D_{min} , this indicates that there is

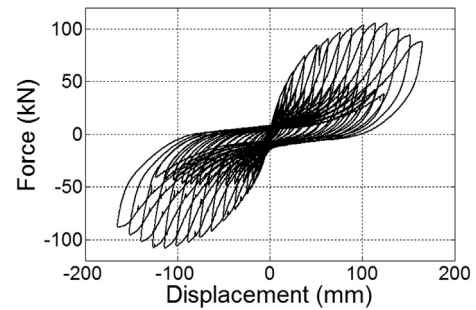


Fig. 21. Test 26 hysteresis- shear wall with (4)0.61 m $\times$ 2.44 m $\times$ 169 mm wall panels.

Table 4

Range of variables considered for the definition of CLT archetypes.

Variable	
Occupancy and use	- Residential, Commercial, and mixed use
Elevation and plan configuration	- Various wall lengths - Low, high, and mixed aspect ratio
Building vertical configuration	- 1–12 Stories - 3.05 m story height
Interior and exterior non-structural wall finishes	- Not considered
Seismic design category	- Cmax/Dmin and Dmax
Gravity Load	- ASCE 7 Table C3-1 [30]

no need to check for SDC C and SDC B and the system will only be analyzed for SDC D_{max} . In addition, based on the testing, the critical case of panel aspect ratio will be determined and engineering judgment will be used to further reduce the number of archetypes.

Fig. 22 shows the elevation and floor plan of an index building developed for this study with the extracted archetype highlighted in red¹. Shear walls utilized as part of the lateral force resisting system are also shown in the figure. Floor plans are identical for each story and each story is 3.05 m (10 ft) clear height. The method of construction is platform construction whereby floor panels bear on and are supported by the wall panels below.

5. Performance evaluation through nonlinear time history analysis

Two key factors identified during the CLT shear wall testing were the aspect ratio of the walls and the inter-panel connectors. This section examines design and evaluation of an as-designed CLT archetype, shown in Fig. 22, with several different configurations.

The FEMA P-695 methodology requires archetypes to be designed for the Design Earthquake (DE) and then evaluated for the Maximum Considered Earthquake (MCE). The seismic load at each story is calculated based on the equivalent lateral force (ELF) procedure explained in Sec 12.8 of the ASCE Standard 7-16 [30] and the archetype is designed using three different configurations shown in Fig. 23. Seismic loads are defined in terms of seismic design category (SDC) and occupancy category of the structure. Based on the methodology, structures considered Occupancy Category I or II receive an importance factor equal to unity, site classification is taken as Site Class D and for the purpose of this example the structure is designed for SDC D_{max} (upper bound of SDC D), $S_s = 1.50$ g, $S_1 = 0.6$ g, $F_a = 1$ and $F_v = 1.5$. The building period, 0.47 s., was calculated in accordance with FEMA P-695 where it was taken as the product of coefficient for the upper limit (Table 12.8-1) and the approximate fundamental period, Section

¹ For interpretation of color in Fig. 22, the reader is referred to the web version of this article.

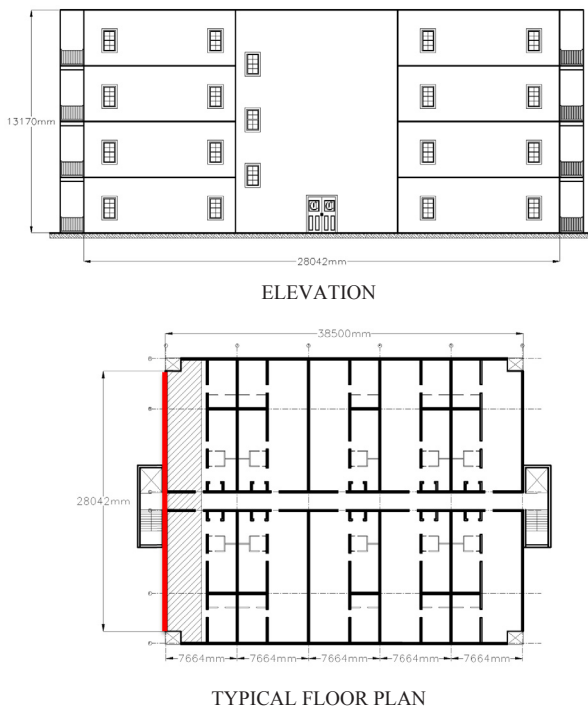


Fig. 22. CLT index building elevation and floor plan.

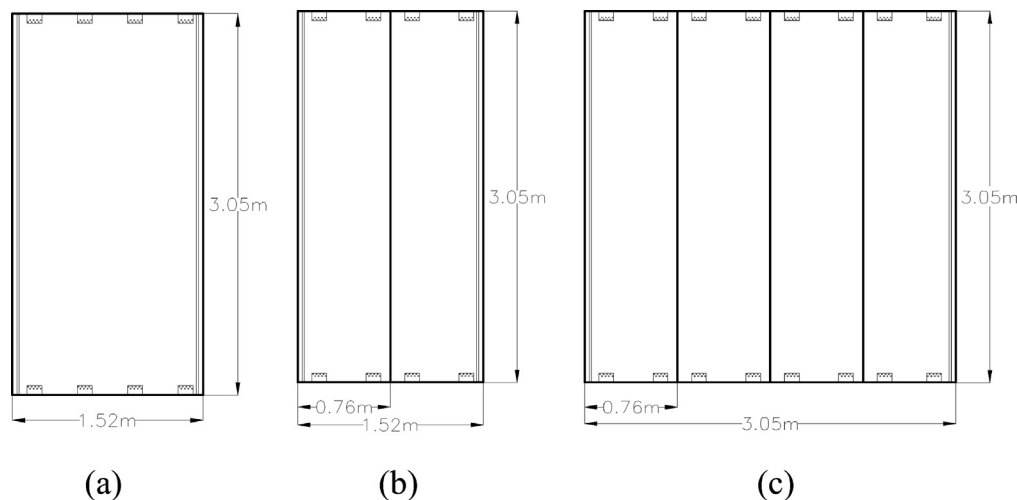


Fig. 23. Shear wall Configurations used in archetype design.

12.8.2.1 of ASCE 7-16.

The ELF procedure requires selection of a suitable R factor and based on a study of a 6-story CLT apartment building, Pei et al. [17] estimated a reasonable R factor would lie between 3.5 and 5.5. However, it is important to note that suggested range was based on a single structure and in a location with a particular seismic hazard. For the purpose of this study, calculations are performed for an illustrative value of $R = 4$. It is important to note that this is an assumed value for illustrative purposes in this paper, but a full P-695 study will likely have between 20 and 50 archetypes required for accurate identification of an R factor. The LRFD strength values for these CLT shear walls are obtained by applying a Φ factor of 0.55. CLT shear walls were also designed to meet the interstory drift limits in Table 12.2-1 of ASCE 7-16. From a design perspective, this resulted in equal lengths of shear walls and an equal number of connectors but in different configurations.

Nonlinear time history analysis (NLHTA) was performed on the designed archetype to evaluate the performance of the different

configurations. CLT modelling can be performed with various levels of complexity and while a finite element (FE) based formulation can be used in certain cases, the computational effort for a study such as FEMA P-695 typically demands utilizing models with simplified kinematic assumptions. Therefore, analysis was performed using the SAPWood software [37] with the assumptions that the structure is composed of rigid diaphragms attached to shear walls that are represented by non-linear springs. SAPWood was developed as part of the NEESWood project for analysis of light-frame wood structures and the accuracy and reliability of the software has been validated through a number of studies using full-scale system-level test data (e.g. [38,39]).

The phenomenological CUREE hysteretic model developed by Folz and Filiatrault [40] as part of CUREE-Caltech (Consortium of Universities for Research in Earthquake Engineering) project was used to characterize the CLT shear wall behaviour. For other modelling methods based on component approaches readers are referred to studies by Pozza et al. [28] and Rinaldin et al. [41]. The CUREE model requires ten parameters to define force, stiffness, and their degradation as part of the hysteretic behaviour. Wall configurations in Fig. 23a–c correspond to Tests 10, 23 and 26, respectively. To account for the difference in panel height between the model (3.05 m) and the tests (2.44 m), the load-displacement test data was adjusted before fitting the hysteretic model. The fitting was performed using curve fitting tool in SAPWood and the parameters were determined considering the least favourable of the positive or the negative envelope curve. The ten-parameter fit are shown in Fig. 24.

A suite of 22 bi-axial ground motions identified in FEMA P-695 as

far-field were used as the input for the NLHTA. In order to better understand behaviour of the different wall configurations, the structure was subjected to three levels of increasing seismic hazard, namely DBE, MCE, and 1.5MCE. The scaling was performed in accordance with the P-695 methodology where a record set is scaled by a single factor such that the median response spectrum of the set only matches the spectral acceleration at the fundamental period of the building. This scaling for MCE demand is shown in Fig. 25.

The analysis was performed for both components of the ground motion resulting in 44 nonlinear time history analyses and a single corresponding maximum interstory drift at each story was recorded from each analysis. These values were then rank-ordered and plotted as a cumulative density function (CDF) which are shown in Fig. 26 for three different configurations and three levels of seismic hazard. Inspection of Fig. 26 shows that the difference in performance between various configurations at the DBE level is insignificant. However, the difference becomes more noticeable with increasing intensity,

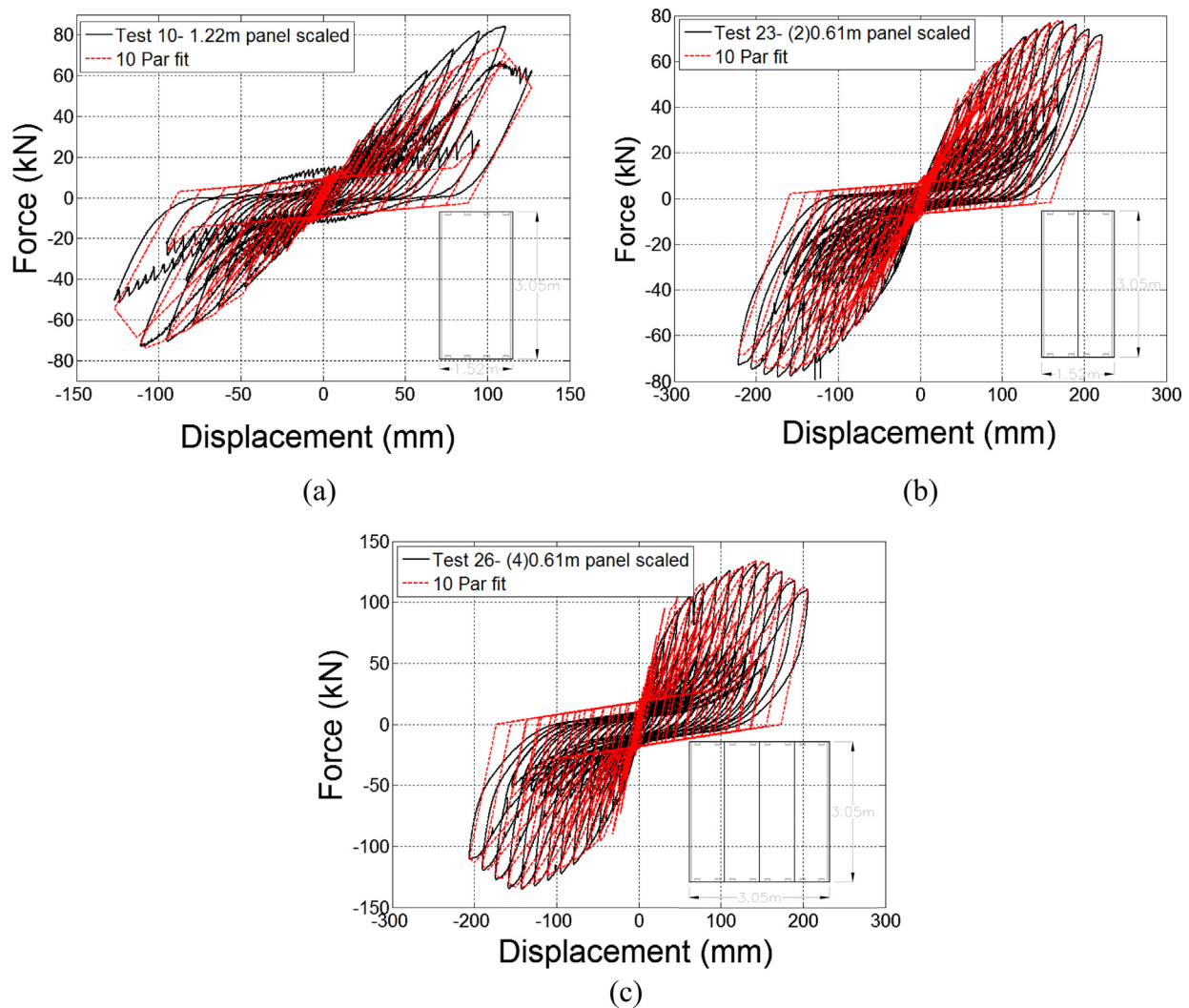


Fig. 24. Calibrated ten-parameter hysteretic model.

especially when one considers the 1.22 m long single panel configuration. Configurations with the vertical joints (inter-panel connectors) exhibit better performance and are somewhat similar to each other. While only three configurations on a single archetype were analyzed in this example, it underscores the importance of using different panel

arrangements in the archetype wall configurations for the collapse analysis study of the FEMA P-695 CLT archetypes.

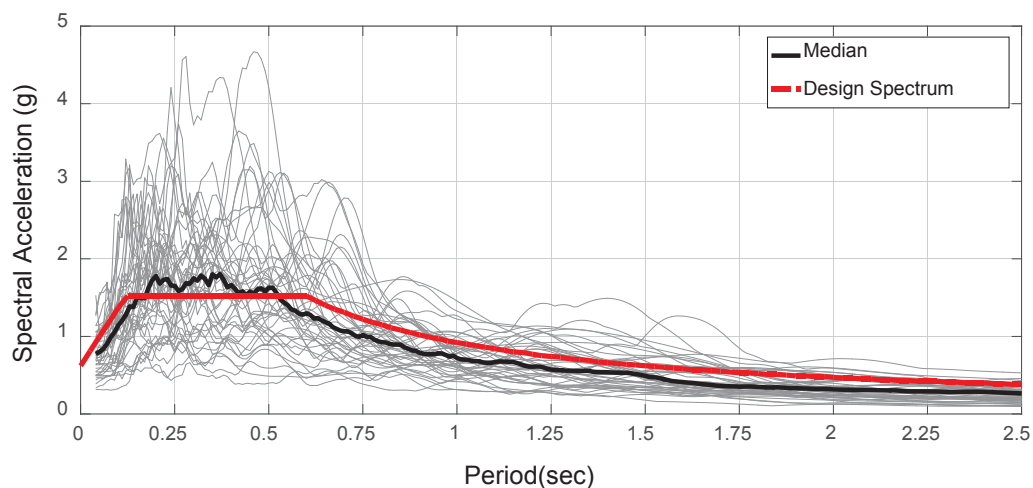


Fig. 25. FEMA P-695 far-field ground motions scaled to MCE demand.

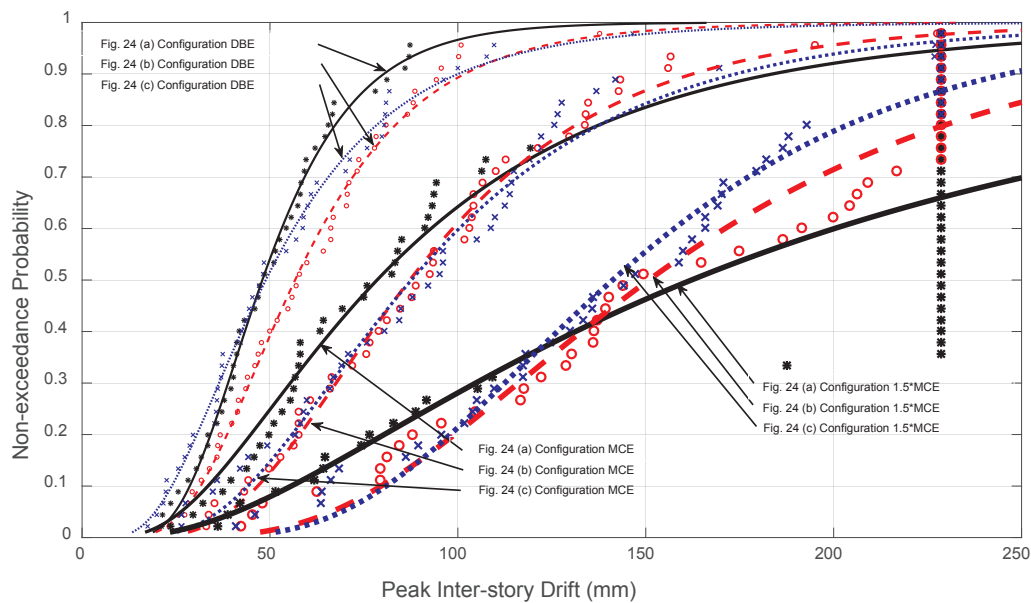


Fig. 26. CDF of peak inter-story drift.

6. Summary and conclusions

CLT is seen as a viable structural system in mid-rise construction and the purpose of the overall project explained herein is to determine seismic performance factors for a CLT shear wall seismic force resisting systems. This is achieved through the application of the FEMA P-695 methodology with the eventual goal of including this new system in the ASCE-7 Standard. Testing is one of the major steps identified in the methodology and this paper presents the results of CLT shear wall tests and then applied this data to systematically illustrate the need to utilize different panel configurations within an archetypical wall line. Specifically, each wall test comparison presented in this paper allowed either the inclusion or exclusion in the upcoming numerical modelling phase.

Connectors performed as intended with the nonlinear behaviour isolated in the connectors and fasteners. Wall test results showed that boundary conditions had a slightly beneficial effect on the CLT wall behaviour and could therefore be neglected in additional testing. Tests on the walls with gravity loads indicated that both stiffness and strength increase as the gravity load increases; however, the change in the latter was less significant. A study of panel thickness showed that thickness has only a slight effect on wall stiffness and strength, as both properties were highly influenced by the connection behaviour. Other comparisons of the panel behaviour based on thickness showed similar trends. Results of a 4:1 aspect ratio panel compared to a 2:1 aspect ratio panel showed the higher aspect ratio panels had significantly less stiffness but had more deformation capacity than the lower aspect ratio panels. The effect of this on the development of the seismic performance factors remains to be seen but it is clear that both have offsetting effects to some degree. The increase in deformation capacity can be attributed to the predominantly rocking behaviour of the panel as opposed to a combination of rocking and sliding mechanism of other panels tested. On the other hand, comparing the results of 2:1 with 1:1 aspect ratio panels the differences in the stiffness and deformation capacity were not as pronounced. Testing has also shown that walls comprised of higher aspect ratio panels that are connected through vertical joints exhibited less stiffness but considerably larger deformation capacity. Considering some preliminary results from NLTHA, wall configurations will be used in different combinations of panel length for the collapse evaluations of the archetypes in the FEMA P695 study. The testing phase of this research underscored the need to only use walls made up

of panels with a 2:1 or greater aspect ratio in the design procedure to ensure deformation capacity.

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