World Conference on Timber Engineering Oslo 2023

EVALUATION OF CLT SHEAR WALLS FOR INCREASED STRUCTURAL HEIGHT LIMIT AND FOR EFFECT OF VARYING PANEL ASPECT RATIO

M. Omar Amini[1], Philip Line[2], John W. van de Lindt[3], Doug Rammer[4], Francisco Flores[5]

ABSTRACT: In the US, two CLT shear wall systems are defined in SDPWS 2021 and ASCE 7-22, *CLT shear wall* with CLT panel aspect ratio ranging from 2:1 to 4:1 (*CLT-R3*) and *CLT shear wall with shear resistance provided by high aspect ratio panels only* (*CLT-R4*). Both of these systems are applicable up to a structural height of 19.8 meters [65 feet] in Seismic Design Categories B through F, thus limiting some applications. To increase usability of this system, two analytical study cases are presented in this paper: Study Case 1, increase in structural height limit for *CLT-R3* and *CLT-R4* systems in moderate seismic regions (i.e., SDC C_{max}); and Study Case 2, panel aspect ratio range for *CLT-R4* system. This evaluation was performed using the FEMA P695 methodology on several archetypes that were prototypical representation of the CLT shear wall system. For both study cases the archetypes passed the collapse performance objective of FEMA P695 indicating that *CLT-R3* and *CLT-R4* systems up to and including 8 stories can be used in SDC C_{max} and *CLT-R4* system with CLT panel aspect ratio ranging from 3.5:1 to 4.5:1 can be used in SDC D_{max}.

KEYWORDS: Cross-laminated timber (CLT), seismic performance factors, FEMA P695

1 INTRODUCTION

There has been a recent increase in the number of studies geared towards investigating CLT system behavior and performance under cyclic and dynamic loading. Most of these studies originated in Europe (e.g., Dujic et al. [1]; Ceccotti [2]; Hristovski et al. [3]), in North America (e.g., Popovski et al. [4]; Pei et al., 2013 [5]; Popovski and Gavric [6], van de Lindt et al. [7]) and in Japan (e.g., Okabe et al. [8]; Tsuchimoto et al. [9]). While some studies aimed to provide a novel contribution to the developing body of knowledge in this new area of engineering and construction, some of the studies have adopted a systematic approach to investigating seismic behavior of CLT with the eventual goal of obtaining seismic performance factors or codification of some kind. A review of some of these studies is provided in Pei et al. [10] and a more recent comprehensive review of seismic behavior of CLT can be found in Izzi et al. [11].

In the US, the cross-laminated timber (CLT) shear wall seismic force resisting system was recognized in building code reference standards for the first time with inclusion of seismic design requirements in 2021 Special Design Provisions for Wind and Seismic (SDPWS) [12] and in ASCE 7-22 Minimum Design Loads and Associated Criteria for Buildings and Other Structures [13].

Seismic performance factors, namely, the response modification factor, R, deflection amplification factor, C_d, and overstrength factor, Ω, were developed by applying the FEMA P695 [14] methodology to the CLT shear wall system [7, 15, 16]. The evaluation process is summarized in Figure 1 with the details explained in Amini et al. [15]. The methodology consists of experimental testing, development of a design methodology, archetype development, numerical modelling, nonlinear analyses, and peer panel review. The process was applied to a number of archetypes that were prototypical representations of the seismic force resisting system. These analyses resulted in computing the so-called "margin against collapse" of each archetype and hence the CLT shear wall system with specific requirements dictated by FEMA P695. The methodology takes into account uncertainties inherent in the test data and modelling methods as well as inherent variability in the suite of ground motion records.

Two CLT shear wall systems are defined and recognized by SDPWS and ASCE 7; *CLT shear wall* with CLT panel aspect ratio ranging from 2:1 to 4:1 with R=3, C_d=3, and Ω = 3; and *CLT shear wall with shear resistance provided by high aspect ratio panels only* with CLT panel aspect ratio equal to 4:1 with R=4, C_d=4, and Ω = 3; hereafter referred to as *CLT-R3* and *CLT-R4*, respectively. Both

[1] Manager, Wind and Seismic, American Wood Council, 50 Catoctin Circle NE, Suite 201, Leesburg, VA 20175, USA. E-mail: oamini@awc.org

[2] VP, Codes and Regulations, American Wood Council, 50 Catoctin Circle NE, Suite 201, Leesburg, VA 20175, USA. E-mail: pline@awc.org

[3] Harold Short Endowed Chair Professor, Department of Civil and Environmental Engineering, Colorado State University, Fort Collins, CO 80523, USA. E-mail: jwv@engr.colostate.edu

[4] Research General Engineer, Forest Products Laboratory, One Gifford Pinchot Drive, Madison, WI 53726, USA. E-mail: douglas.r.rammer@usda.gov

[5] Manager, Engineering Technology, American Wood Council, 50 Catoctin Circle NE, Suite 201, Leesburg, VA 20175, USA. E-mail: fflores@awc.org

https://doi.org/10.52202/069179-0400

systems have a structural height limit of 19.8 meters [65 feet] in Seismic Design Category (SDC) B through F. Seismic performance factors and structural height limits appearing in ASCE 7-22 [13] are summarized in Table 1. This paper summarizes results of application of the FEMA P965 method to increase usability of the CLT shear wall systems defined in SDPWS. In one study, an increase in the structural height limit beyond the current limit of 19.8 meters [65 feet] in moderate seismic hazard areas (i.e., SDC C_{max}) is evaluated. In a separate study, an expanded aspect ratio range between 3.5:1 and 4.5:1 is evaluated for *CLT-R4* as a replacement for the existing exact limit of 4:1.

2 DESIGN METHODOLOGY

The design of the CLT shear wall system in accordance with SDPWS [7] includes requirements for use of CLT panels meeting aspect ratio requirements, prescribed connections (i.e., nails and connectors) at the bottom and top of the walls and at vertical edges of multi-panel shear walls, and tie-downs to transfer overturning induced tension forces. These requirements result in combined bending yield and pull-out of the nails from the CLT panels which exhibit rocking and sliding behaviour under in-plane shear loading. Example single-panel and multi-panel CLT shear wall configurations are shown in Figure 2.

Table 1: *Design Coefficients and Factors for CLT Seismic Force-Resisting Systems (appearing in ASCE/SEI 7-22 Table 12.2-1)*

Seismic Force-Resisting System	Detailing Requirements, ASCE/SEI 7-22 Section	R	Ω_0	C_d	Structural Height, h_n, Limit Seismic Design Category B, C, D, E & F
Cross-laminated timber shear walls	14.5	3	3	3	19.8 meters [65 feet]
Cross-laminated timber shear walls with shear resistance provided by high aspect ratio panels only	14.5	4	3	4	19.8 meters [65 feet]

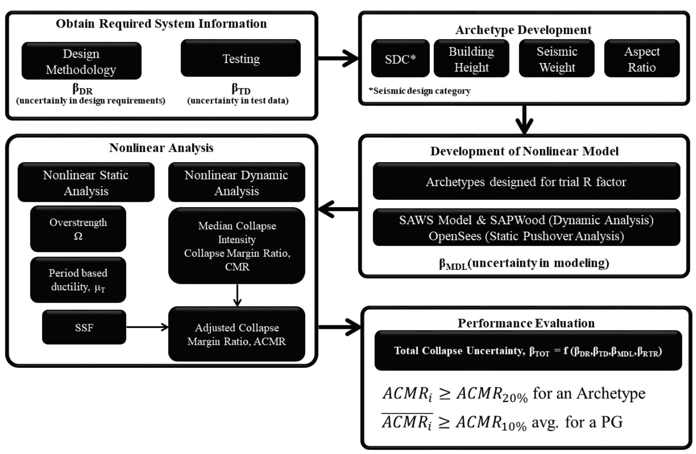

Figure 1: *CLT Shear Wall System Evaluation Process (van de Lindt et al., 2020)*

3 ARCHETYPES

A floor plan for an example index building with the extracted archetypes highlighted in red dashed lines is shown in Figure 3. While prior studies [7, 16] evaluated 72 archetypes (extracted from index buildings) for a full comprehensive study on CLT shear wall system, a reduced number of archetypes is analysed for this study based on prior findings that collapse probabilities are correlated to pushover strength (V_{max}/W) with superior collapse performance associated with increased strength. This study conservatively evaluated archetypes associated with smaller values of pushover strength. Study cases are shown in Table 2. For increased structural height limit for

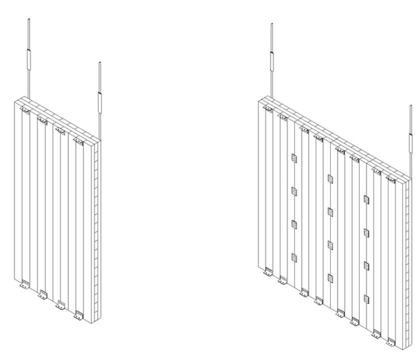

Figure 2: *(a) Single panel configuration (b) Multi-panel configuration*

CLT-R3 system, 8-story archetypes are considered and designed for SDC C_{max}. These were then grouped into two performance groups, PG-1 and PG-2 based on the CLT panel aspect ratio. For the evaluation of aspect ratio range for *CLT-R4* system, archetypes of 2-6 stories were considered and designed for SDC D_{max}. These were also grouped into two performance groups, PG-3 and PG-4 based on the CLT panel aspect ratio. Both study cases were with high gravity load and shear wall line length of 6.1m-18.3m extracted from the index buildings. In total 18 archetypes were considered in this study.

Table 2: *Study cases*

	CLT shear wall system	**Seismic Design Load Level**
Increased structural height limit (25.91 meters [85 feet])	*CLT-R3* *CLT-R4*	SDC C_{max}
Expanded aspect ratio range (3.5:1 – 4.5:1)	*CLT-R4*	SDC D_{max} /SDC D_{min}

The FEMA P-695 methodology requires archetypes to be designed for the Design Earthquake (DE) and then evaluated for the Maximum Considered Earthquake (MCE). Seismic loads are defined in terms of seismic design category (SDC) and occupancy category of the structure. Based on the methodology, structures are considered Occupancy Category I or II and receive an importance factor equal to 1.0. For Study Case 1 (increased structural height limit) the archetypes were designed for the Design Earthquake (DE) in SDC C_{max}, S_s= 0.55g, S_1=0.132g, F_a=1.36 and F_v=2.28. For Study Case 2 (expanded aspect ratio range) the archetypes were designed for SDC D_{max}, S_s= 1.50g, S_1=0.6g, F_a=1 and F_v=1.5, since the highest SDC governs the performance.
The building period 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, ASCE 7-16 [17]) and the approximate fundamental period, Section 12.8.2.1 of ASCE 7-16. Designs were based on the ELF procedure explained in Sec 12.8 of ASCE 7-16 using R of 3 and 4 for Study Cases 1 and 2, respectively.
The selected archetypes were designed to ensure over-strength (i.e., provided shear capacity over shear demand) was minimized. After the design, the as-designed archetypes were then evaluated via numerical modeling for a set of predefined ground motions. Performance groups, archetypes and their corresponding seismic design parameters are provided in Table 3.

4 NUMERICAL MODELLING

In order to match prior study analysis methods, CLT shear wall behavior is characterized using the phenomenological CUREE-SAWS model [18]. This 10-parameter model defines force, stiffness and degradation as part of the hysteretic behaviour. The 10-parameters used for CLT shear walls are based on CLT shear wall test results which incorporated tension rods for overturning, and the prescribed shear connections in accordance with the design method in SDPWS 2021 [12]. This hysteretic fitting is discussed in van de Lindt et al. [7] and an example of the SAWS model fitted to the test data is shown in Figure 4. In Study Case 2, for CLT panels with aspect ratios of 3.5:1 and 4.5:1, parameters were scaled based on test data for CLT shear walls with panel aspect ratio of 2:1 and CLT shear wall with panel aspect ratio of 4:1. The effect of this scaling was seen in terms of peak strength and deformation capacity. An example for the two panel multi-panel configuration is shown in Figure 5. Looking at Figure 5, the lower aspect ratio (3.5:1) configuration has a larger strength but smaller deformation capacity while the higher aspect ratio panel case (4.5:1) has a lower strength but a gentler slope for the post peak behavior.
Nonlinear static pushover and incremental dynamic analysis (IDA) (Vamvatsikos and Cornell [19]) were performed in OpenSees [20] assuming that the structure is composed of rigid diaphragms attached to shear walls that are represented by nonlinear hysteretic springs. P-Delta effects were also included using a leaning column.
Static pushover was performed for each archetype to determine maximum base shear resistance, V_{max}, and ultimate displacement, δ_u. The results of these analyses were then used to determine overstrength factor, Ω, and period-based ductility, μ_T. The former is defined as the ratio of maximum base shear over design base shear and the latter is obtained from the pushover analyses using the following equation.

$$\mu_T = \frac{\delta_u}{\delta_{y,eff}} \tag{1}$$

where δ_u is the roof displacement corresponding to 80% post peak load ($0.8V_{max}$) and $\delta_{y,eff}$ is the effective yield roof displacement.

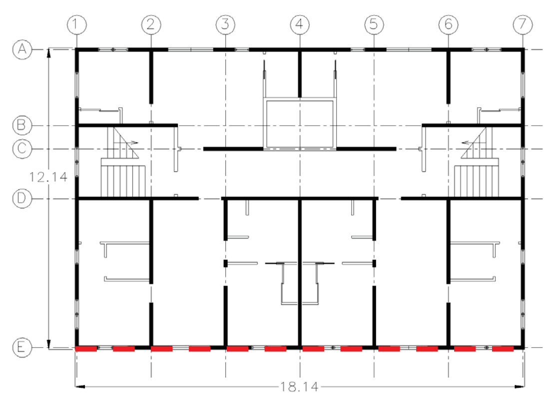

Figure 3: *Example index building floor plan (dimension in meters)*

https://doi.org/10.52202/069179-0400

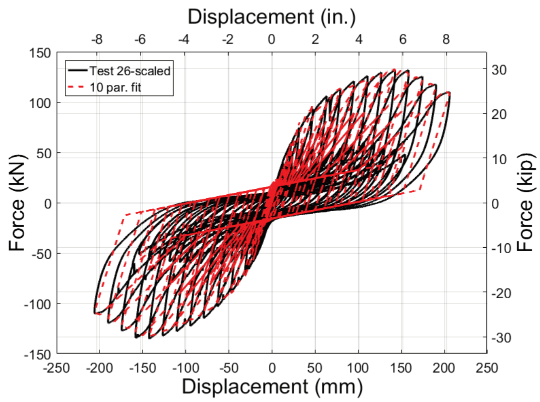

Figure 4: *Four panel multi-panel configuration scaled data and hysteretic fit*

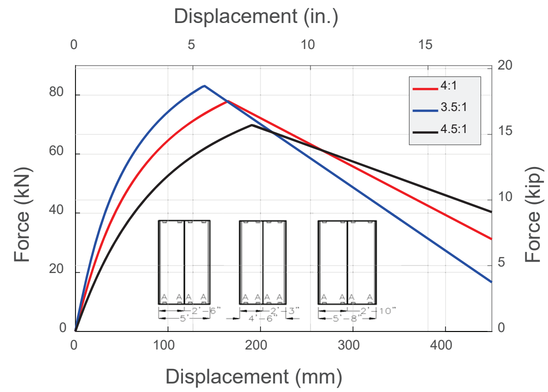

Figure 5: *Two panel multi-panel configuration with different aspect ratios*

Table 3. *Performance groups*

Study Cases	Performance Group	Aspect Ratio	Seismic Design Category	Archetype No.	Number of stories	T (sec)	T_1 (sec)	V_b, kN (kip)	W, kN (kip)	S_{MT} (g)
Study Case 1, Increased structural height limit	PG-1	Low aspect ratio panels, R=3	SDC D_{min}/C_{max}	1	8	0.802	1.72	166.13 (37.35)	1999.82 (449.6)	0.374
				2	8	0.802	1.26	185.3 (41.66)	2229.34 (501.2)	0.374
				3	8	0.802	1.12	299.08 (67.24)	3598.97 (809.12)	0.374
	PG-2	High aspect ratio, R=4		4	8	0.802	1.90	124.54 (28)	1999.82 (449.6)	0.374
				5	8	0.802	1.76	139 (31.25)	2229.34 (501.2)	0.374
				6	8	0.802	1.57	224.31 (50.43)	3598.97 (809.12)	0.374
Study Case 2, Expanded high aspect ratio range	PG-3	3.5:1 aspect ratio, R=4	SDC D_{max}	7	3	0.36	0.56	86.69 (19.49)	330.04 (74.2)	1.50
				8	2	0.26	0.50	77.35 (17.39)	309.58 (69.6)	1.50
				9	3	0.36	0.54	264.34 (59.43)	1003.91 (225.7)	1.50
				10	6	0.60	0.77	182.72 (41.08)	733.92 (165)	1.49
				11	6	0.60	0.62	203.76 (45.81)	819.77 (184.3)	1.49
				12	6	0.60	0.63	328.89 (73.94)	1323.72 (297.6)	1.49
	PG-4	4.5:1 aspect ratio, R=4		13	3	0.36	0.76	86.69 (19.49)	330.04 (74.2)	1.50
				14	2	0.26	0.69	77.35 (17.39)	309.58 (69.6)	1.50
				15	3	0.36	0.74	264.34 (59.43)	1003.91 (225.7)	1.50
				16	6	0.60	1.05	182.72 (41.08)	733.92 (165)	1.49
				17	6	0.60	0.86	203.76 (45.81)	819.77 (184.3)	1.49
				18	6	0.60	0.63	328.89 (73.94)	1323.72 (297.6)	1.49

IDA [19] was performed for a set of 22 bi-axial ground motions (44 records) termed "Far-Field" in FEMA P695 [14]. The ground motion scaling was performed in accordance with the FEMA P695 methodology where a record set is scaled by a single factor such that the median response spectrum of the normalized set matches the spectral acceleration of interest at the fundamental period of the building. According to the FEMA P695 methodology, the damping can be in the range of 2% to 5% of critical damping and for the purpose of this study the conservative lower bound of 2% critical damping was assumed. This damping was applied based on classic Rayleigh damping using the mass matrix and the initial stiffness matrix.

IDA results were used to generate fragility curves which were then used to determine median collapse spectral acceleration ($\hat{S}_{CT}$) defined as the spectral acceleration at which half the ground motions cause collapse (Ibarra et al. [21]). The non-simulated collapse criteria based on an inter-story drift ratio of 4.5% and 5.5% (specified limit state) were used for cases of low aspect ratio panels (2:1) and high aspect ratio panels (4:1), respectively. These are discussed in detail in van de Lindt et al. [7]. A sample fragility obtained based on the CLT shear wall collapse criteria is shown in Figure 6. For Study Case 2, the non-simulated collapse criteria were scaled to reflect the corresponding deformation capacities. Inter-story drift ratio of 5.25% and 5.75% were used for the two high aspect ratio panel cases of 3.5:1 and 4.5:1, respectively.

5 PERFORMANCE EVALUATION

Collapse Margin Ratio (CMR) for each archetype was calculated as the ratio of median collapse intensity, $\hat{S}_{CT}$, to MCE ground motion intensity, S_{MT}. CMR was then adjusted to an Adjusted Collapse Margin Ratio (ACMR) using the Spectral Shape Factor (SSF) to account for the effects of spectral shape.

$$ACMR = CMR * SSF \tag{2}$$

SSF was determined based on the SDC for which the archetype was designed, and the period-based ductility obtained from the pushover analysis. ACMR is then compared with the acceptable ACMR that is calculated considering the uncertainties corresponding to the record-to-record variability (β_{RTR}), design requirements (β_{DR}), test data (β_{TD}), and modelling (β_{MDL}). The methodology accounts for these uncertainties using total system collapse uncertainly, β_{TOT}, given by the following equation:

$$\beta_{TOT} = \sqrt{\beta_{RTR}^2 + \beta_{DR}^2 + \beta_{TD}^2 + \beta_{MDL}^2} \tag{3}$$

These β values are based on qualitative ratings that are then translated to quantitative values. The ratings used for evaluation in this study are provided in Table 4. Considering the completeness of design requirements, β_{DR} of 0.10 was used in this study which is smaller than 0.20 used in the original study [7]. A detailed discussion on consideration of uncertainties on collapse evaluation is provided in Chapter 7 of FEMA P695 [14] and van de Lindt et al. [7]. The effect of total system collapse uncertainty on the fragility curve is demonstrated in Figure 6 shown earlier. While the median remains unchanged, additional uncertainty flattens the curve resulting in higher probability of collapse at MCE spectral intensity.

For performance evaluation all the individual archetypes needed to pass the $ACMR_{20\%}$ criteria (FEMA P695 criteria for an individual archetype to pass) and the average of the performance groups exceed the $ACMR_{10\%}$ criteria (FEMA P695 criteria for a performance group) for the performance groups. The results are provided in Table 5. Looking at the table, all the individual archetypes and performance groups for both study cases pass their respective collapse criteria. Performance groups results are also summarized in Figure 7 showing all the performance groups passing their corresponding criteria.

6 CONCLUSIONS

The evaluation of increased structural height limit for both CLT shear wall systems defined in SDPWS, namely *CLT-R3* and *CLT-R4*, showed that archetypes up to and including 8 stories met the collpase performance objective of FEMA P695 for use in SDC C_{max}. The evaluation of expanded aspect ratio for *CLT-R4* system, showed that archetypes of 1-6 stories with CLT panel aspect ratio ranging from 3.5:1 - 4.5:1 met the collapse perfromance objective of FEMA P695 for use in SDC D_{max}.

Table 4: *Quality ratings used for evaluation*

Uncertainty	Quality rating value	Description
Record-to-record (β_{RTR})	0.40	$\beta_{RTR} = 0.1 + \mu_T \leq 0.40$
Design requirements (β_{DR})	0.10	Superior: High in completeness and robustness and high confidence
Test data (β_{TD})	0.20	Good: Medium in completeness and robustness and high confidence
Modeling (β_{MDL})	0.20	Good: Medium in completeness and robustness and high confidence

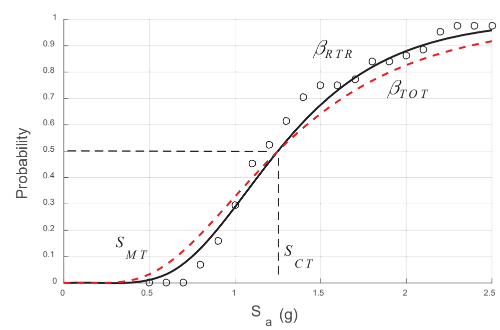

Figure 6: *Archetype 1 collapse fragility curve with lognormal fit*

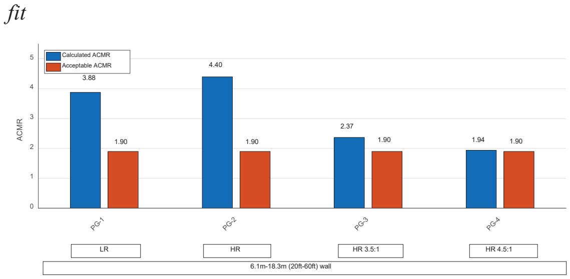

Figure 7: *Summary of ACMR for performance groups, ξ=0.02 (damping ratio), NSC (Non-simulated collapse); Study Case 1=4.5% Inter-story drift for LR (Low aspect Ratio) (2:1) and 5.5% for HR (High aspect Ratio) (4:1), Study Case 2=5.25% Inter-story drift for HR (High aspect Ratio) (3.5:1) and 5.75% for HR (High aspect Ratio) (4.5:1)*

https://doi.org/10.52202/069179-0400

Table 5: *Collapse Evaluation Parameters*

Study Case	No.	Collapse Margin Parameters Ω	μ_T	$\hat{S}_{CT}$	CMR	SSF	Acceptance Check ACMR	Acceptable ACMR	Pass/Fail
Study Case 1, Increased structural height limit	**PG-01**								
	01	3.10	2.08	1.25	3.33	1.17	3.89	1.52	PASS
	02	3.53	2.46	1.28	3.42	1.19	4.09	1.52	PASS
	03	2.58	3.18	1.11	2.96	1.23	3.65	1.52	PASS
	PG	**3.07**			**3.24**		**3.88**	**1.90**	**PASS**
	PG-02								
	04	3.48	2.52	1.30	3.47	1.20	4.16	1.52	PASS
	05	3.04	3.15	1.40	3.73	1.23	4.60	1.52	PASS
	06	2.62	3.80	1.32	3.53	1.26	4.45	1.52	PASS
	PG	**3.05**			**3.58**		**4.40**	**1.90**	**PASS**
Study Case 2, Expanded aspect ratio range	**PG-03**								
	7	3.72	3.31	2.71	1.80	1.20	2.16	1.52	PASS
	8	4.04	3.88	2.67	1.78	1.22	2.16	1.52	PASS
	9	3.40	3.83	3.01	2.01	1.22	2.44	1.52	PASS
	10	3.66	2.71	2.96	1.99	1.18	2.35	1.52	PASS
	11	3.78	3.61	3.27	2.19	1.22	2.68	1.52	PASS
	12	2.76	4.18	2.93	1.96	1.24	2.44	1.52	PASS
	PG	**3.56**			**1.96**		**2.37**	**1.90**	**PASS**
	PG-04								
	13	3.12	3.02	2.31	1.54	1.18	1.82	1.52	PASS
	14	3.40	3.51	1.97	1.31	1.20	1.58	1.52	PASS
	15	2.86	3.43	2.63	1.75	1.20	2.10	1.52	PASS
	16	3.07	2.48	2.24	1.50	1.17	1.76	1.52	PASS
	17	3.18	3.21	2.74	1.84	1.21	2.22	1.52	PASS
	18	2.30	3.67	2.64	1.77	1.23	2.17	1.52	PASS
	PG	**2.99**			**1.62**		**1.94**	**1.90**	**PASS**

REFERENCES

[1] Dujic B., Aicher S., Zarnic R. Racking Behavior of Light Prefabricated Cross-Laminated Massive Timber Wall Diaphragms Subjected to Horizontal Actions. Otto Graf Journal, Materialprüfungsanstalt Universität, Otto-Graf-Institut, Stuttgart. 2006; (17).

[2] Ceccotti A. New Technologies for Construction of Medium-Rise Buildings in Seismic regions: The XLAM Case. Structural Engineering International 2008; 18 (2): 156-165.

[3] Hristovski V, Dujic B, Stojmanovska M, Mircevska V. Full-scale shaking-table tests of XLam panel systems and numerical verification: Specimen 1. Journal of Structural Engineering. 2012 Oct 13;139(11):2010-8.

[4] Popovski M, Schneider J, Schweinsteiger M. Lateral load resistance of cross-laminated wood panels. In World Conference on Timber Engineering 2010 Jun (pp. 20-24).

[5] Pei, S., van de Lindt, J. W., and Popovski, M. (2013). "Approximate R-factor for Cross Laminated Timber Walls in Multi-story Buildings." *J. Archit. Eng.*,19(4), 245–255

[6] Popovski, M., & Gavric, I. (2015). "Performance of a 2-story CLT house subjected to lateral loads." *Journal of Structural Engineering*, *142*(4), E4015006.

[7] van de Lindt, J. W., Amini, M. O., Rammer, D., Line, P., Pei, S., & Popovski, M. (2020). Seismic performance factors for cross-laminated timber shear wall systems in the United States. *Journal of Structural Engineering*, *146*(9), 04020172.

[8] Okabe M, Yasumura M, Kobayashi K, Haramiishi T, Nakashima Y, Fujita K. Effect of vertical load under cyclic lateral load test for evaluating Sugi CLT wall panel. InWorld Conference on Timber Engineering, Auckland, New Zealand 2012 Jul (pp. 15-19).

[9] Tsuchimoto, T., Kawai, N., Yasumura, M., Miyake, T., Isoda, H., Tsuda, C., Miura, S., Murakami, S., Nakagawa, T. Dynamic and static lateral load tests on full-sized 3-story CLT construction for seismic design. In 13th World conference on timber engineering WCTE, Quebec 2014 Aug (Vol. 1014).

[10] Pei S, Van De Lindt JW, Popovski M, Berman JW, Dolan JD, Ricles J, Sause R, Blomgren H, Rammer DR. Cross-laminated timber for seismic regions: Progress and challenges for research and implementation. Journal of Structural Engineering. 2014 Nov 6; 142(4).

[11] Izzi, M., Casagrande, D., Bezzi, S., Pasca, D., Follesa, M., & Tomasi, R. (2018). "Seismic behaviour of Cross-Laminated Timber structures: A state-of-the-art review." *Engineering Structures*, *170*, 42-52.

[12] American Wood Council (ANSI/AWC). (2021). "Special design provisions for wind and seismic." *Leesburg, VA*.

[13] ASCE. Minimum Design Loads for Building and Other Structures. ASCE Standard ASC/SEI 7-22, American Society of Civil Engineers, Reston, Virginia; 2021.

[14] FEMA. Quantification of building seismic performance factors: FEMA P695. Federal Emergency Management Agency, Washington, D.C.; 2009.

[15] Amini, M. O., van de Lindt, J. W., Rammer, D., Pei, S., Line, P., & Popovski, M. (2018). Systematic experimental investigation to support the development of seismic performance factors for cross laminated timber shear wall systems. *Engineering Structures*, *172*, 392-404.

[16] van de Lindt, J., Rammer, D., Amini, M. O., Line, P., Pei, S., and Popovski, M. (2021) "Determination of Seismic Performance Factors for Cross-Laminated Timber Shear Walls Based on the FEMA P695 Methodology." General Technical Report FPL-GTR-281, Madison, WI: U.S. Department of Agriculture, Forest Service, Forest Products Laboratory, (In press).

[17] ASCE. Minimum Design Loads for Building and Other Structures. ASCE Standard ASC/SEI 7-16, American Society of Civil Engineers, Reston, Virginia; 2016.

[18] Folz B, Filiatrault A. Cyclic analysis of wood shear walls. Journal of Structural Engineering. 2001 Apr; 127(4):433-41.

[19] Vamvatsikos, D. and Cornell, C. A. (2002), "Incremental Dynamic Analysis", *Journal of Earthquake Engineering and Structural Dynamics.* 31(3), 491-514.

[20] McKenna, F. et al. (2011). "Open system for earthquake engineering simulation (OpenSees) (2.3.2 ed.)." Berkeley, CA: Pacific Earthquake Engineering Research Center, University of California.

[21] Ibarra L., Medina R., Krawinkler H. (2002). "Collapse assessment of deteriorating SDOF systems" *Proceeding of the 12th European Conference on Earthquake Engineering, London, UK, Paper reference 665, Oxford: Elsevier, September 9-13.*

https://doi.org/10.52202/069179-0400