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Seismic performance evaluation of innovative balloon type CLT rocking shear walls

T.Y. Yang [a,*], S. Lepine-Lacroix [a], J.A. Ramos Guerrero [a], J.B.W. McFadden [b], M.A.Q. Al-Janabi [c]

[a] *Department of Civil Engineering, The University of British Columbia, Vancouver, Canada*
[b] *Canadian Construction Materials Center, National Research Council Canada, Ottawa, Canada*
[c] *Department of Civil Engineering, Al-Nahrain University, Baghdad, Iraq*

A R T I C L E I N F O

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A B S T R A C T

Balloon type cross laminated timber (CLT) rocking shear walls are a novel seismic force resisting system. In this paper, the seismic performance of four 12-story balloon type CLT rocking shear walls, designed by a structural engineering firm located in Vancouver (Canada) using the performance-based design procedure outlined in the technical guideline published by the Canadian Construction Materials center (CCMC)/National Research Council Canada (NRC), is assessed. The seismic performance of the prototype CLT rocking shear walls was investigated using nonlinear time history analyses. Robust nonlinear finite element models were developed using OpenSees and the nonlinear behavior of the displacement-controlled components was calibrated using available experimental data. A detailed site-specific hazard analysis was conducted and sets of ground motions suitable for the prototype buildings were selected. The ground motions were used in a series of incremental dynamic analyses (IDAs) to quantify the adjustable collapse margin ratio (ACMR) of the prototype balloon type CLT rocking shear walls. The results show that the prototype balloon type CLT rocking shear walls designed using the performance-based design procedure outlined in the CCMC/NRC technical guideline have sufficient ACMR when compared to the acceptable limits recommended by FEMA P695.

1. Introduction

The growing environmental concerns regarding the carbon emissions produced around the globe have created a need to develop safe, carbon-neutral, and sustainable ecofriendly structural systems [1]. Mass timber products, cross-laminated timber (CLT) in particular, have become a popular option of global interest in timber construction [2]. CLT is a mass timber product composed of orthogonal layers of sawn lumber that are laminated and glued together to form panels that have high in-plane strength and stiffness. Most CLT shear wall systems studied by previous researchers rely on the nonlinear behavior and damage of connections [3–6] to achieve energy dissipation during an earthquake. However, there is a need to develop seismically resilient mass timber lateral force-resisting systems suitable for performance-based seismic design [7]. Encapsulated mass timber has been incorporated into both the 2020 version of National Building Code of Canada (NBCC) [8] for buildings up to 12 stories and the 2021 version of the International Building Code (IBC) for buildings up to 18 stories (ICC 2021) [9] as gravity load-resisting structures. In addition, the NBCC 2020 adopted the platform type CLT shear wall for lateral load resisting structures and referred to the Canadian Standard for Engineering Design in Wood (CSA O86) [10] for detailing provisions. A substantial number of tests conducted on CLT assemblies and other systems have shown that rocking is the preferred response for those systems, mainly because it limits the internal forces that are developed and provides some amount of self-centering capacity [11–13,37–39]. A significant amount of research has been conducted on platform type CLT shear walls [14–20]. However, limited research has been done on balloon type CLT rocking shear walls and designers have limited guidance on how to design such systems [18]. To overcome this deficiency, the Canadian Construction Materials center (CCMC)/National Research Council Canada (NRC) [21] has published a performance-based design procedure to allow engineers to design different seismic force resisting systems (including balloon type CLT rocking shear walls).

This paper presents a systematic evaluation of the seismic performance of balloon type CLT rocking shear walls designed according to the CCMC/ NRC [21] procedure. Two 12-story prototype balloon type CLT rocking shear wall buildings located in Vancouver and Montreal were designed by the professional engineering firm Read Jones Christoffersen Consulting Engineers (RJC Engineers), located in Vancouver, Canada, [22].

* Corresponding author.
E-mail address: yang@civil.ubc.ca (T.Y. Yang).

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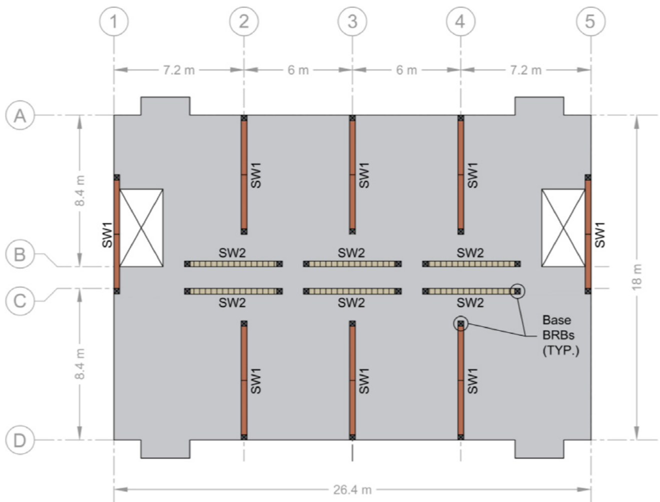

Fig. 1. Typical floor plan view.

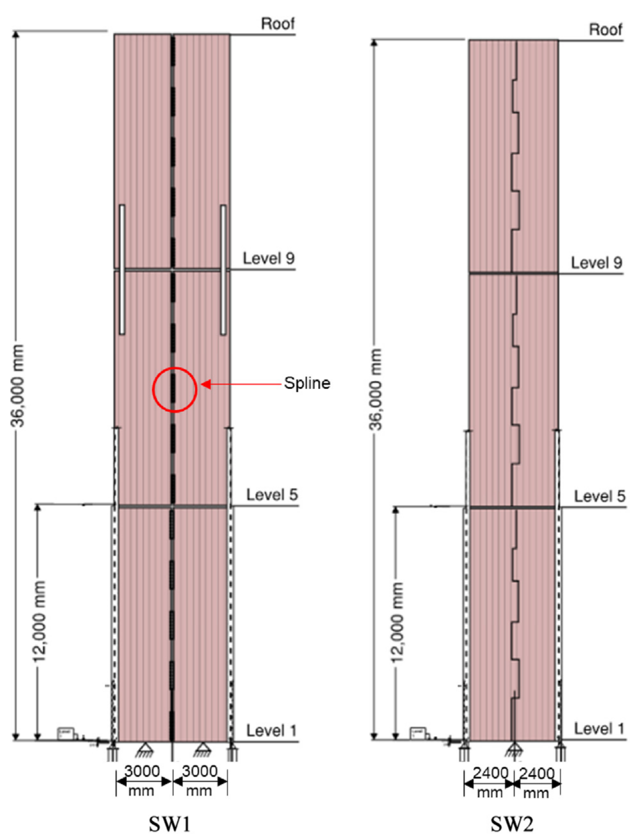

Fig. 2. Overview – shear walls SW1 and SW2 (RJC engineers, 2021) [22].

The paper first presents a brief overview of the CCMC/NRC performance-based design procedure, followed by a detailed summary of the designed prototype buildings. The paper then offers a detailed presentation of the finite element models developed in OpenSees [23] which were used to assess the seismic performance of the balloon type CLT rocking shear walls. In addition, the paper presents the detailed site-specific seismic hazard analyses and the ground motions selection and scaling performed for the prototype sites. Lastly, the paper presents the incremental dynamic analyses (IDAs) conducted to quantify and validate the seismic performance of balloon type CLT rocking shear walls.

Table 1
Modal periods.

Shear wall	T1 [s]	T2 [s]
Vancouver – SW1	1.36	0.37
Vancouver – SW2	1.69	0.31
Montreal – SW1	2.22	0.45
Montreal – SW2	3.28	0.35

Table 2
Seismic hazard deaggregation – Vancouver (T = 1.5 s, 2/50 shaking intensity).

Hazard source	Hazard contribution [%]	Mean Hypocentral Distance [km]	Mean Magnitude
Crustal	12.6	25	6.9
Inslab	38.6	72	7.1
Subduction interface	48.8	136	8.9

Table 3
Period-based ductility (μ_T) values.

Shear wall	Vancouver	Montreal
SW1	3.43	3.59
SW2	4.64	7.16

2. Overview of the CCMC performance-based design procedure

The objective of the Canadian Construction Materials center (CCMC) [21] guideline is to provide a simple, systematic, and sufficient procedure to evaluate the performance of new or existing Seismic Force Resisting Systems (SFRS) and to determine the appropriate ductility-related (Rd) and over-strength related (Ro) force modification factors to be implemented in the National Building Code of Canada (NBCC) [24]. The procedure relies on the application of nonlinear dynamic analysis to quantify the seismic performance of an SFRS.

In simple terms, the steps involved in the CCMC performance-based design approach are: (i) Design an archetype structure using the equivalent static procedure or a linear dynamic procedure with preliminary Rd and Ro values; (ii) Develop a nonlinear finite element model with explicit modeling of the yielding mechanism, yielding elements and the mass distribution of the system. The design team shall specify the failure criteria for the displacement-controlled actions (such as deformation capacity of the yielding elements and the inter-story drift ratio) which shall be verified using available experimental data. The rest of the elements are designed as non-yielding elements which shall be capacity designed using the forces obtained from nonlinear time history analyses performed using ground motions selected and scaled to match the uniform hazard spectrum (UHS) over the entire period range of interests as outlined in NBCC Commentary J [24]. To ensure the non-yielding elements have a sufficient safety margin against failure, the design forces shall be taken as 1.6 times the mean of the peak forces obtained from the nonlinear time history analyses at 100% of the UHS intensity. If more than 10% of the ground motions result in unacceptable structural response (such as excessive system drift, excessive component displacement or failure of a non-yielding element) at 100% UHS intensity, the design is deemed as unacceptable. (iii) If the design passes the 100% UHS intensity check, as outlined in step (ii), the design team shall perform a second set of nonlinear time history analyses with ground motions scaled to 200% of the UHS intensity. Recognizing that the design check at 200% UHS intensity is beyond the code design requirement (100% UHS), the allowable inter-story drift limit may be increased. Typically, the allowable inter-story drift limit is set as 2.5% for the 100% UHS intensity and it may be increased to 4.5% for the 200% UHS intensity. Similarly, the design demands for the non-yielding elements are set as 1.3 times the mean of the peak forces obtained from the nonlinear time history analyses at 200% UHS intensity. If less than

Table 4
Failure criteria – SW1.

Parameter	*Vancouver*	*Montreal*
Inter-story drift	100% of UHS > 2.5%	100% of UHS > 2.5%
	200% of UHS > 4.5%	200% of UHS > 4.5%
BRB strain	> 2.5%	> 2.5%
Force in BRB connection	> 3960 kN	> 975 kN
Spline displacement	> 20% splines exceed 78 mm shear displacement	> 20% splines exceed 58 mm shear displacement
CLT shear	> 800 kN	> 340 kN
Force in I section @ base	> 3560 kN	> 870 kN
Force in each screw between CLT panels and steel shapes @ base	> 4 kN	> 7 kN
Force in tension strap @ level 5	> 3250 kN	> 1267 kN
Force in tension strap @ level 9	> 1675 kN	> 775 kN
Rocker support reactions	Shear > 855 kN	Shear > 360 kN
	Compressive > 3410 kN	Compressive > 965 kN
	Tensile > 3245 kN	Tensile > 600 kN

Table 5
Failure Criteria – SW2.

Parameter	*Vancouver*	*Montreal*
Inter-story drift	100% of UHS > 2.5%	100% of UHS > 2.5%
	200% of UHS > 4.5%	200% of UHS > 4.5%
BRB strain	> 2.5%	> 2.5%
Force in BRB connection	> 2870 kN	> 450 kN
CLT shear	> 1850 KN	> 620 KN
Force in I section @ base	> 2185 kN	> 400 kN
Force in each screw between CLT panels and steel shapes @ base	> 3 kN	> 3 kN
Force in tension strap @ level 5	> 3375 kN	> 1275 kN
Force in tension strap @ level 9	> 1980 kN	> 810 kN
Rocker support reactions	Shear > 1850 kN	Shear > 650 kN
	Compressive > 360 kN	Compressive > 330 kN
	Tensile > 175 kN	No tension observed

50% of the ground motions result in unacceptable response, the design is considered satisfactory. This procedure is significantly simpler than the FEMA P695 procedure [25] because it only requires checking the structural performance at the 100% and 200% UHS shaking intensities rather than conducting an extensive incremental dynamic analysis.

3. Prototype buildings

In this study, two 12-story balloon type CLT prototype buildings located in (i) Vancouver and (ii) Montreal on class D soil according to the 2015 edition of the National Building Code of Canada (NBCC 2015) [24] were designed using the procedure described in Section 2. The prototype buildings have a typical story height of 3 m and typical floor dimensions of 26.4 x 18 m. The prototype buildings consist of 8 coupled rocking shear walls (labeled SW1) in the north-south direction and 6 single rocking shear walls (labeled SW2) in the east-west direction. Fig. 1 shows the typical floor plan of the prototype buildings. SW1 was designed using Rd and Ro values of 2.0 and 1.5, respectively. SW2 was designed using Rd and Ro values of 4.0 and 1.2, respectively. This results in a lateral force resisting system (LFRS) comprised of 315E 9-ply CLT shear walls equipped with energy dissipation devices. SW1 shear walls consist of buckling-restrained brace (BRBs) (properties of each BRB: Area A = 6300 mm^2 (Vancouver), Area A = 1750 mm^2 (Montreal), Length L = 2.85 m, Yield stress Fy = 350 MPa) at each side of their base. In addition, 30 ductile splines (15 on each face) are provided between the panels to dissipate additional seismic energy for the Vancouver prototype, while 9 regular splines are provided for the Montreal prototype. Similarly, SW2 shear walls consist of BRBs (properties of each BRB: Area A = 4250 mm^2 (Vancouver), Area A = 670 mm^2 (Montreal), Length L = 4.5 m (Vancouver), Length L = 2.85 m (Montreal), Yield stress Fy = 350 MPa) at each side of their base. Fig. 2 shows the detailed dimensions of SW1 and SW2 shear walls. Fig. 3 shows the typical details used to connect the BRBs to the shear walls. As shown in Fig. 3, steel sections are used on both sides of the SW1 and SW2 shear walls. These steel sections are not expected to behave nonlinearly. Lastly, tension straps are provided for both SW1 and SW2 walls at level 5 and level 9.

4. Numerical modeling of balloon type CLT rocking shear walls

Two dimensional (2D) numerical models for SW1 and SW2 were built using OpenSees [23]. Fig. 4 shows an overview of the OpenSees models that were developed.

The balloon type CLT rocking shear walls were modeled using elastic Timoshenko beam-column elements with a modulus of elasticity E = 12.4 GPa and a shear modulus of G = 775 MPa. It should be noted that the area A, the moment of inertia I and the shear area Av were multiplied by 0.55, since the simplified method proposed by Chen and Popovski [26] was used to model the shear walls. In short, this method assumes that only the longitudinal layers of the panels resist axial, bending and shear loads. In addition, it should be noted that the physical width of the CLT panels was accounted for by using rigid elastic beam-column elements. A calibrated SAWS material model was used to model the nonlinear shear behavior of the splines, while the axial behavior was assumed to be rigid. Fig. 5a shows a comparison between the SAWS model and the test results for the regular splines. The ductile spline, on the other hand, was developed to enhance the strength and ductility of the regular spline. The force-deformation relationship of the ductile spline was extrapolated from the regular spline as it has not yet been tested. Fig. 5b compares the force-deformation response of the SAWS model for the regular splines (used in Montreal model) and the ductile splines (used in the Vancouver model). The steel sections near the base of the structure were modeled using elastic beam-column elements. These elements were connected to the CLT panels using zero-length elements with an elastic shear stiffness of 2953 kN/m per screw [22] to simulate the screws used to connect the steel sections to the CLT panels. The BRBs were modeled using nonlinear truss elements with a modi-

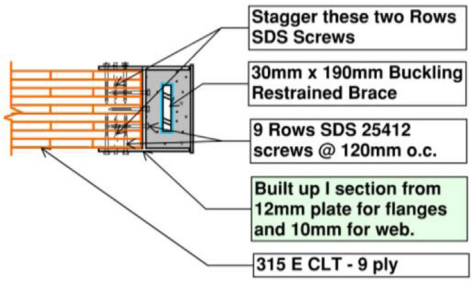

Fig. 3. Typical BRB connection detail (RJC Engineers, 2021) [22].

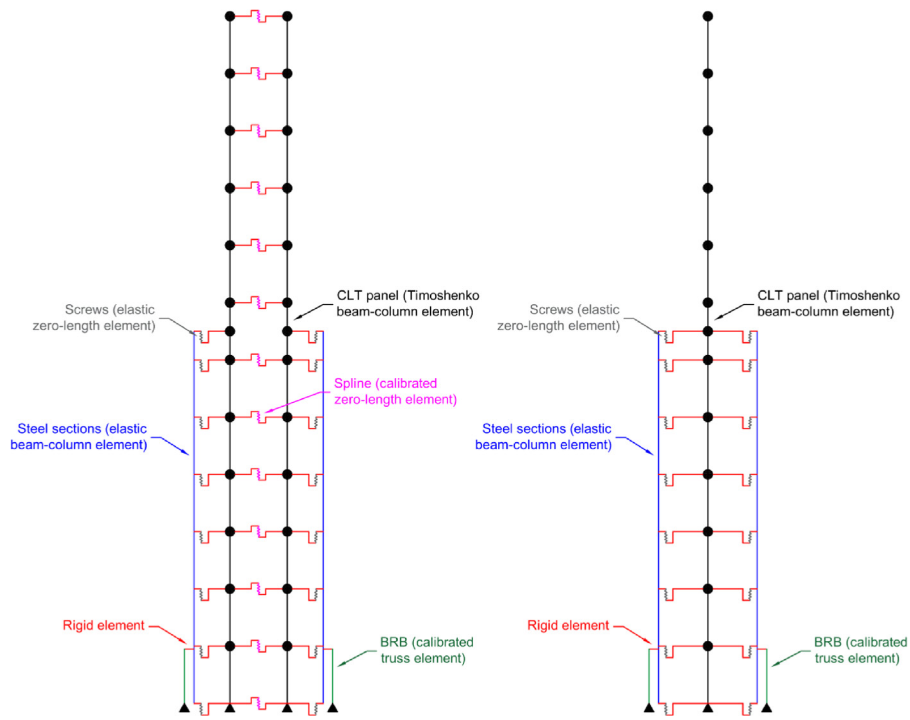

Fig. 4. Overview of the OpenSees models.

fied Steel02 material that allowed for different tension and compression nonlinear properties. The material was calibrated to match the stress-strain curve provided by Tremblay et al. [27]. A strain limit of 2.5% was added to the material model to simulate BRB fracture. Fig. 6 shows a comparison between the experimental data and the numerical model developed for the BRBs.

The effect of the floor diaphragms was accounted for by using equalDOF multi-point (MP) constraints that constrained all nodes at a given floor to have the same horizontal translation. The total mass of the prototype building was taken as 1.91×10^6 kg. As provided by RJC Engineers [22], the most critical SW1 wall has a tributary mass equal to 16.5% of that of the building, while the most critical SW2 wall has a tributary mass equal to 12.5% of that of the building. These masses were modeled as horizontal masses at every floor. Because vertical excitations are not considered in this study, the vertical mass does not have a significant impact on the seismic behavior of the CLT shear walls. However, the masses representing the self-weight of the walls were added as vertical mass in the models. The shear walls were designed to not resist any gravity loads other than their self-weight of 140.8 kg/m^2, which was provided by the manufacturer [28]. The self-weight of the walls was modeled using vertical point loads. Lastly, 2.5% Rayleigh damping was defined using the first two modes of vibration of the models. Table 1 shows the first two modal periods of the shear wall models.

5. Probabilistic seismic hazard analysis and ground motion selection

This section presents a summary of the probabilistic seismic hazard analysis that was performed for this study. The seismic hazard deaggregation for the Vancouver site was performed using the software EZ-FRISK [29] for a 2% probability of exceedance in 50 years (2/50) shaking intensity. This is consistent with the UHS shaking intensity outlined in the 2015 edition of the National Building Code of Canada (NBCC) [24]. The deaggregation revealed that the Vancouver site has three dominant hazard sources (crustal, inslab and subduction interface earthquakes), and that these sources have the properties and contributions shown in Table 2 for a period of T = 1.5 s, which corresponds approximately to the mean fundamental period of both shear walls (SW1 and SW2). Based on the seismic hazard deaggregation, it was found that the dominant period ranges are of 0–1.0 s, 0–1.5 s and 1.0–10 s for the crustal, inslab and subduction interface hazard sources, respectively.

For the Montreal site, seismic hazard deaggregation data provided by Natural Resources Canada [30] indicated that the site has only one dominant hazard source (crustal earthquakes) for all periods and probabilities of exceedance. For a period of T = 3.0 s (which is approximately equal to the mean period of both shear walls (SW1 and SW2) and for a

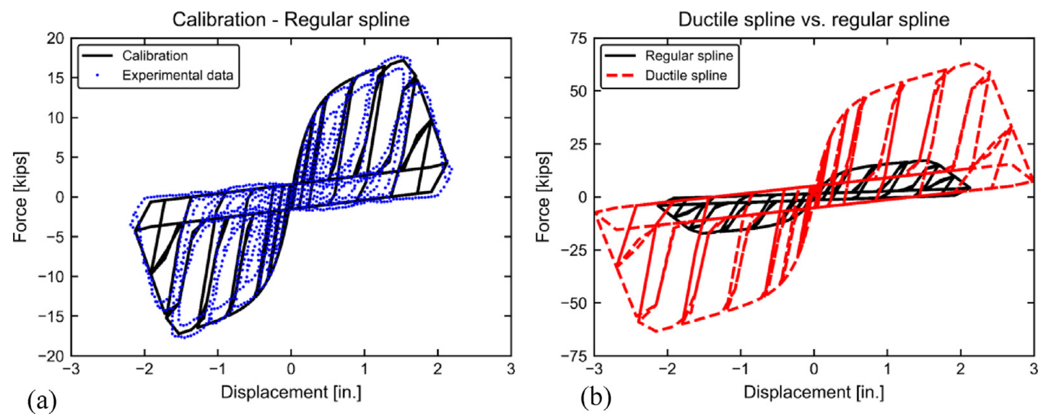

Fig. 5. Calibration of the splines.

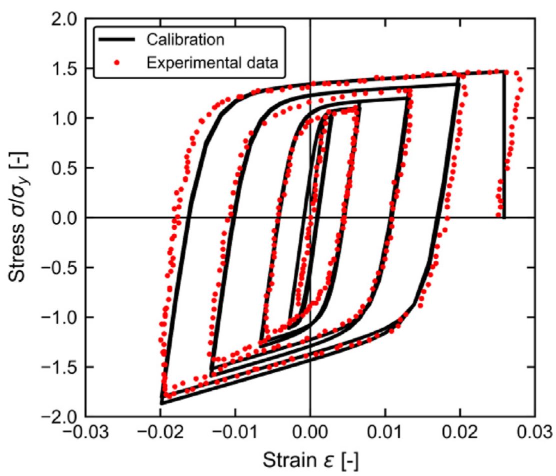

Fig. 6. Calibration of the BRBs.

probability of exceedance of 2% in 50 years, the mean magnitude of the earthquakes is of 7.1 and their mean hypocentral distance is of 74 km.

Based on the seismic hazard deaggregation, suites of ground motions suitable for the sites were selected and scaled using method A of Commentary J of NBCC 2015 [24]. For the Vancouver site, a suite of 11 ground motions was first selected for each of the three hazard sources (for a total of 33 ground motions). The crustal ground motions records were obtained from PEER NGA-West2 database [31], while the inslab and subduction interface records were obtained from the S2GM [32] database and from the preliminary release of the PEER NGA-Sub database [31]. The crustal, inslab and subduction interface suites were then scaled over a period range of 0.1–1.0 s, 0.1–1.5 s and 1.0–6.0 s, respectively. To minimize overscaling of the ground motions, the scaling factors were limited between 0.5 and 4.0 as prescribed by Commentary J of NBCC 2015. The results of the ground motion selection and scaling for the Vancouver site are shown in Fig. 7.

For the Montreal site, a similar procedure was used. Consequently, 1 suite of 11 crustal ground motion records selected from the PEER NGA-West2 database was selected and scaled over a period range of 0.4–6.6 s following the provisions of Commentary J of NBCC 2015. The results of the ground motion selection and scaling for the Montreal site are shown in Fig. 8.

6. Pushover analysis

Fig. 9 shows the pushover curves obtained for shear walls SW1 and SW2 for both the Vancouver and Montreal prototype buildings. FEMA P695 [25] defines the period-based ductility, μ_T, as the roof drift ratio

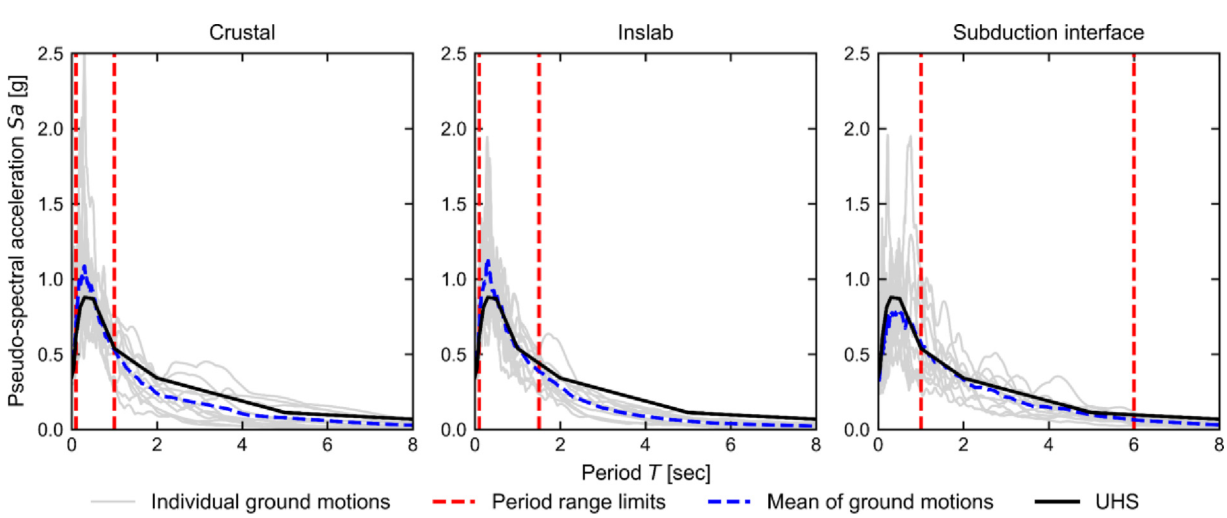

Fig. 7. Scaled spectra – Vancouver prototype.

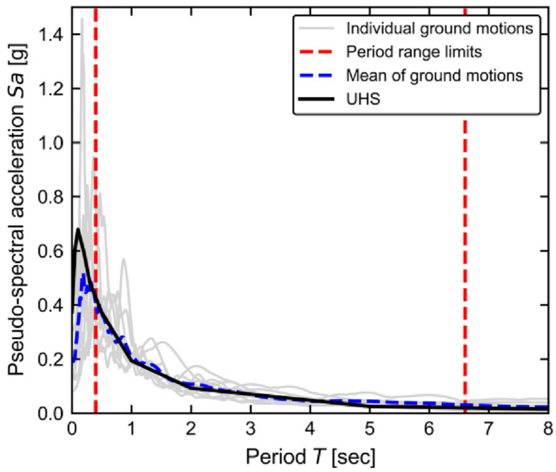

Fig. 8. Scaled spectra - Montreal prototype.

when the shear load is equal to 80% of the maximum shear load V_{max} (after the peak shear load has been reached) divided by effective roof yield drift ratio (Δ_{yeff}). In this study, Δ_{yeff} was taken as the roof drift ratio at which the initial slope of the pushover curve intersects the peak shear load. These variables are illustrated in the pushover curves shown in Fig. 9, and Table 3 presents the period-based ductility values obtained for the various shear walls. It should be noted that the pushover curves consider all failure criteria (both simulated and non-simulated) that are presented in the next section of this paper.

Table 6
Collapse margin ratios (CMR).

Shear wall	Vancouver	Montreal
SW1	2.4	2.6
SW2	2.3	2.7

Table 7
Spectral shape factors (SSFs).

Shear wall	SSF (Crustal)	SSF (Inslab)	SSF (Subduction interface)
SW1 - Vancouver	1.30	1.26	1.17
SW2 - Vancouver	1.33	1.29	1.22
SW1 - Montreal	1.33	-	-
SW2 - Montreal	1.50	-	-

7. Incremental dynamic analyses (IDAs)

Incremental dynamic analyses (IDAs) [33] were conducted for the prototype models using the ground motions that were selected and scaled to 100% of the UHS. Tables 4 and 5 show the failure criteria that were used in the IDAs.

Fig. 10 shows the result of the IDAs. The vertical axis shows the scaling factor with respect to the ground motions scaled to 100% of the UHS. The circles represent the first instance where a failure criterion is met for a given ground motion. Fig. 11 shows the fragility curves obtained for the 4 prototype buildings

The collapse margin ratio (CMR) is defined as the ratio of the shaking intensity over the UHS when the prototype building has 50% probability of collapse. Table 6, as well as Fig. 11, show the CMR values obtained for the 4 prototype buildings.

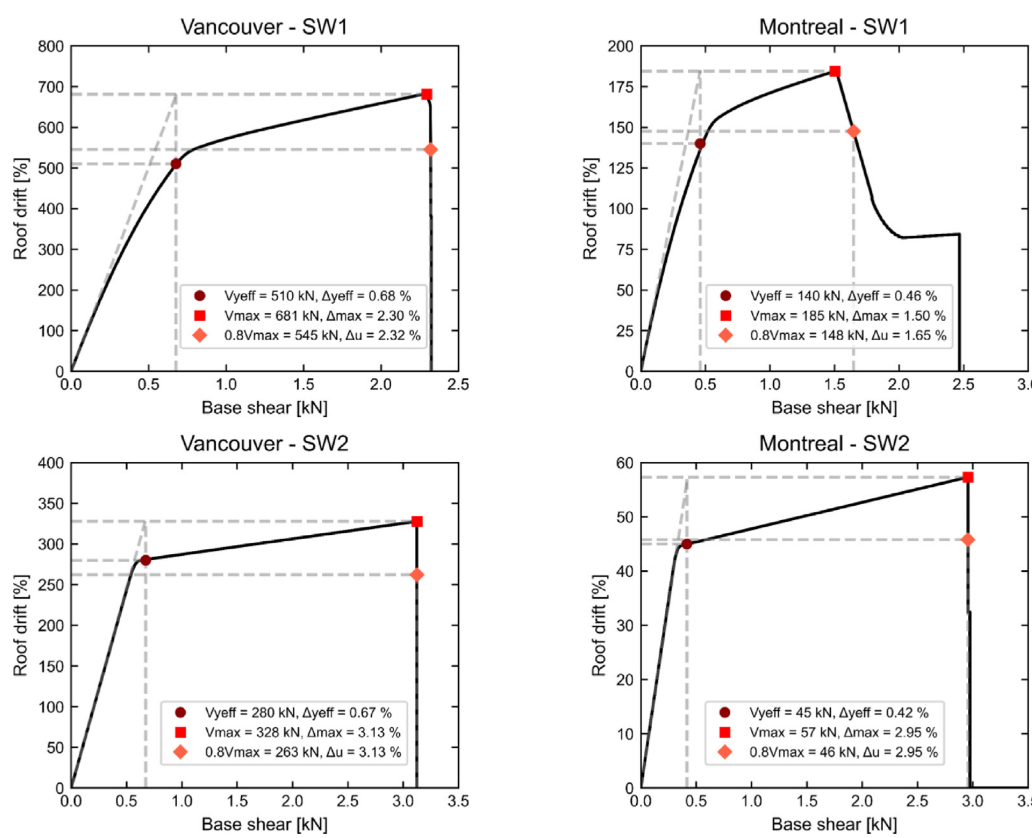

Fig. 9. Pushover curves for shear walls SW1 and SW2.

Table 8
FEMA P695 assessment.

Shear wall	ACMR (Crustal)	ACMR (Inslab)	ACMR (Subduction interface)	ACMR(Total)	Acceptance ACMR	Pass/Fail
SW1 – Vancouver	3.11	3.33	2.53	2.91	1.90	Pass
SW2 – Vancouver	3.07	3.06	2.82	2.94	1.90	Pass
SW1 – Montreal	3.48	-	-	3.48	1.90	Pass
SW2 – Montreal	4.10	-	-	4.10	1.90	Pass

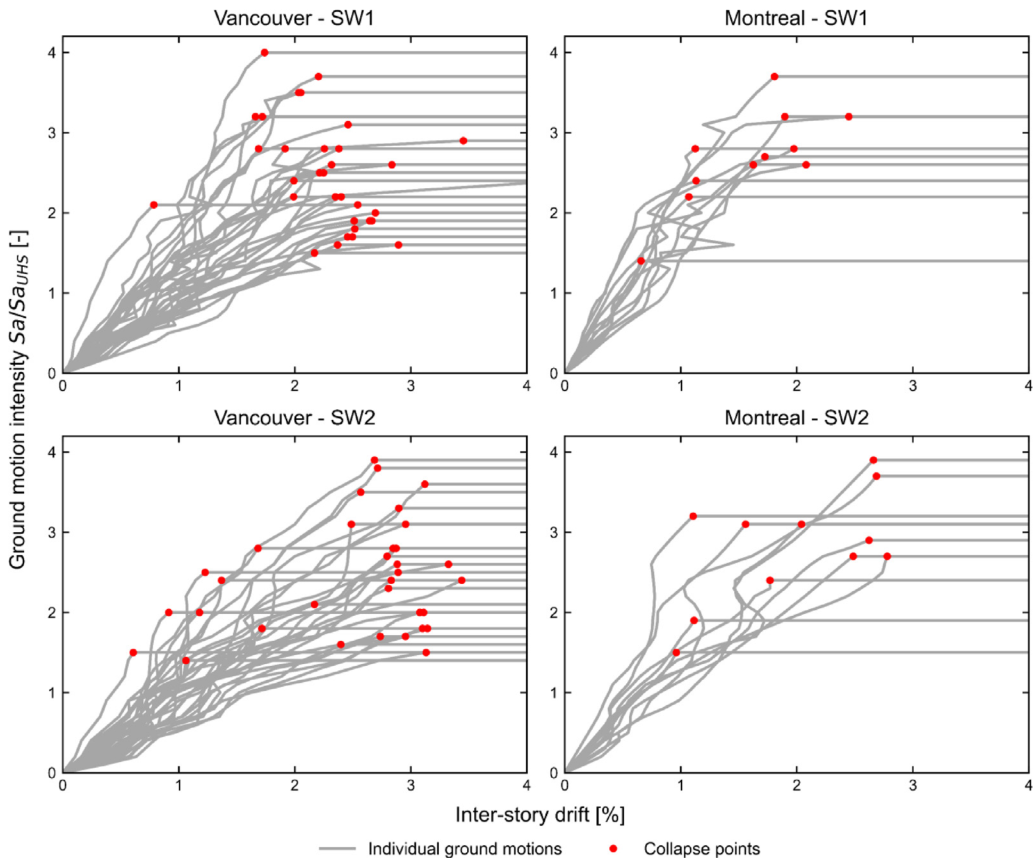

Fig. 10. IDA curves for shear walls SW1 and SW2.

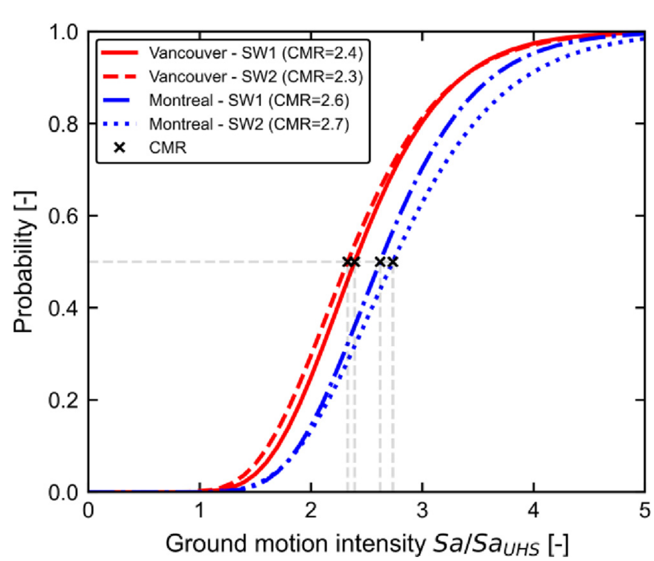

Fig. 11. Fragility curves for shear walls SW1 and SW2.

Because the ground motions used in this study are different than the set of ground motion presented in FEMA P695 [25], new sets of spectral shape factors (SSFs) needed to be calculated to adjust the CMRs to the adjusted collapse margin ratios (ACMRs). In this study, the SSFs were computed using Eqs. (1) and (2), as presented in Bagatini-Cachuco and Yang [34]. Table 7 shows the SSF values for each of the shear walls.

$$SSF = e^{\left(\frac{\beta_1 \ln(CMR)}{\sigma_{GMPE}}\right)} \tag{1}$$

where β_1 is computed from Eq. (2) and σ_{GMPE} represents the logarithmic standard deviation of the ground motion prediction model. In this study, σ_{GMPE} for crustal ground motions was obtained from the BA08 attenuation model [35]. For the inslab and subduction interface ground motions, it was obtained from the BC16 attenuation model [36].

$$\beta_1 = 0.14\ \left(\mu_T - 1\right)^{0.42} \tag{2}$$

where the period-based ductility, μ_T, was taken from Table 3.

The spectral shape factors (SSFs) obtained using these two equations are shown in Table 7.

In order to calculate the total ACMR, the ACMR from each hazard source were weighted and combined using the procedure outlined in Bagatini-Cachuco and Yang [34]. Finally, the total ACMR was compared to the acceptable ACMR (= 1.90) listed in FEMA P695 [25], which was calculated using a probability of collapse of 10% and a total system collapse uncertainty of 50%.

Table 8 shows the design check against the FEMA P695 procedure. The results show that all 4 designs satisfy the requirements of FEMA P695 (i.e. they provided a sufficient margin of safety).

8. Summary and conclusion

This paper presents the seismic performance assessment of four 12-story balloon type CLT rocking shear walls located in Vancouver and Montreal. The prototypes were designed using the performance-based design guideline published by the Canadian Construction Materials center (CCMC)/National Research Council Canada (NRC). Detailed finite element models were developed using OpenSees and calibrated to match the available experimental data. The developed OpenSees models were subjected to a series of incremental dynamic analyses to assess the seismic performance of the balloon type CLT rocking shear walls. The results show that the prototype balloon type CLT rocking shear walls designed using the performance-based design procedure outlined in the CCMC/NRC technical guideline have sufficient adjusted collapse margin ratio (ACMR) when compared to the acceptable limits recommended by FEMA P695. This study shows that balloon type CLT rocking shear walls are a valid seismic force resisting system and that the simplified performance-based design procedure outlined in the CCMC/NRC technical guideline can be used as an efficient tool to design robust seismic force resisting systems.

Declaration of Competing Interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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