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VARIABILITY OF THE CROSS-LAMINATED TIMBER (CLT) SINGLE-PANEL SHEAR WALLS' RESPONSE UNDER IN-PLANE LATERAL LOADS

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ABSTRACT: In the past, several studies analysed the seismic response of Cross-Laminated Timber (CLT) shear wall systems using experimental tests and numerical analyses. Usually, these analyses were based on the average mechanical properties of connections, which have the largest effect on the non-linear response of CLT shear walls. This paper investigates the non-linear in-plane response variability of single-panel CLT shear walls accounting for the actual scatter of the connection's mechanical properties. Based on previously obtained experimental results of traditional hold-downs and angle brackets, the response of CLT shear walls subjected to in-plane lateral loads is analysed considering the main engineering parameters variability, such as strength, deformation, and ductility through a simulation approach based on random sampling. The study considers three different shear walls width-to-height aspect ratios (1:3, 1:1, 3:1) analysed by changing the vertical loads. For each scenario, one thousand of numerical analyses are carried out by varying the experimental mechanical properties of every single connection. The outcomes of this study show the variability of the non-linear numerical response of single-panel CLT shear walls, which was found to decrease with an increase in the length of the shear wall, while the engineering parameters varied in a range of 1% to 17%.

KEYWORDS: Cross-Laminated Timber, shear wall, variability, non-linear response, mechanical connections.

1 INTRODUCTION

The numerical modelling of Cross-Laminated Timber (CLT) shear wall behaviour under lateral loads is still a subject of research. Among the proposed modelling strategies [1], several authors implemented the so-called component-level (CL) modelling approach since it allows a detailed analysis of the non-linear behaviour. The CL strategy considers each structural system component (i.e. CLT panels, connections) with linear/non-linear behaviour calibrated through experimental tests or analytical equations. Although significant efforts have been made to investigate the CLT shear walls' behaviour [2,3], no research has been conducted to estimate the variability of the non-linear response due to the variability of the connections' actual behaviour.

The average experimentally obtained properties are commonly used to perform non-linear numerical analyses when adopting a CL approach [4,5,6].

This approach is obviously correct when assessing a CLT shear wall's performance. However, for all structural systems, the seismic response of CLT shear walls is ruled by physical aspects of each structural component, including the deformation mechanisms (CLT shear and bending deformations, sliding and rocking) and the variable behaviour of the structural components.

In the case of CLT shear walls without openings, the total lateral deformation of a shear wall [7] is mostly influenced by the connections' flexibility instead of the panel deformability.

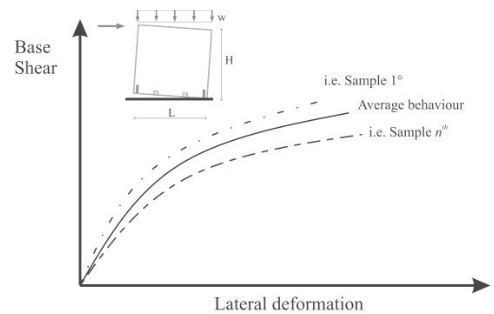

Figure 1: *Different response scenarios due to the connections' variability for the same CLT shear wall configuration*

Due to this phenomenon, the response of a CLT shear wall is predominantly affected by the connection's behaviour [8]. As shown in Figure 1, a CLT shear wall could result in different force-displacement responses due to the variability of the connections' mechanical properties.

In most experimental programmes on timber connections [9-11], a well-defined connection is tested several times to investigate the actual behaviour and the variability of the connection's response. It should be highlighted that mechanical timber connections are prone to a high scatter of results due to the natural variability of the timber as a material [12]. In fact, the fasteners of a connection may be fixed into a quite defective part of the wood (i.e. knot) or layer with less strength which can completely influence the overall behaviour of a connection.

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In this paper, several CLT single-panel shear wall setups are numerically analysed to investigate the effects of the mechanical variability of traditional CLT connections.

A parametric analysis is carried out by examining three different shear wall aspect ratios (1:3, 1:1, 3:1) varying the levels of applied vertical loads and modifying the experimental backbone curves of each connection of the shear wall through a simulation approach based on random sampling. According to this procedure, each configuration is analysed in terms of aspect ratio and vertical load. The specific objectives of this study are to investigate:

- the variability of CLT shear walls response under different conditions (aspect ratios and vertical loads);
- the variability of the main engineering output parameters, such as strength, deformation and ductility.

2 METHODOLOGY

The randomness and the response variability of single-panel CLT shear wall are investigated through a non-parametric stochastic simulation approach.

The study is based on four main phases: *i)* collection of the experimental data of the mechanical connections and calibration of the numerical parameters, *ii)* definition and validation of a numerical model, *iii)* definition of the shear walls configurations in terms of geometry, vertical loads and number of connections, and *iv)* random sampling generation and analyses of a thousand different CLT shear walls for each configuration.

Non-Linear Static Analyses (NLSA) are used to estimate and compare the variability of strength, deformation, and ductility of each shear wall. The pushover curves are linearised according to EN 12512 [14]. Figure 2 shows the adopted nomenclature.

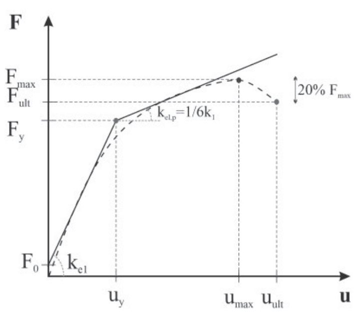
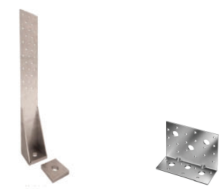


Figure 2: *Linearisation and definition of the parameters according to EN 12512 [14]*

F_0 is the intercept between the elastic branch representing elastic stiffness k_{el} and the y-axis; k_{el} is defined as the secant stiffness between 10% and 40% of the maximum strength F_{max}; $k_{el,pl}$ is the post-elastic branch tangent to the experimental curve and inclined by 1/6 k_{el}. The points (F_y, u_y), (F_{max}, u_{max}) and (F_{ult}, u_{ult}) represent the conventional yielding, maximum strength and ultimate capacity, respectively, where the last one is estimated at the strength drop equal to 20% of F_{max}.

2.1 EXPERIMENTAL DATA AND NUMERICAL CALIBRATION

Traditional timber-to-steel connections such as Hold-Downs (HD) and Angle-Brackets (AB) are considered in this study. The backbone curves of the HD and AB are based on existing experimental data [9]. HD type WHT540 with twelve 4×60 mm annular shanked nails and AB type BMF 90 × 116 × 48 × 3 with 11 annular shanked nails are used (Figure 3).



(a) (b)

Figure 3: *(a) hold-downs (HD) and (b) angle brackets (AB)*

Several specimens were tested for each configuration of HDs and ABs during the experimental programme (tension and shear tests). Results are available for seven samples of HDs in tension, six samples of AB in tension and six samples of AB in shear and were implemented in this work.

A uni-directional vertical-uplift behaviour is assumed for HD since they do not provide a significant contribution in horizontal-shear direction and assuming a bi-directional behaviour would yield too conservative results [7]. A bi-directional behaviour is considered for AB due to the comparable load-bearing capacity in both directions, shear (horizontal-shear) and tension (vertical-uplift).

The "SAWSMaterial" [13] OpenSees numerical model was implemented in the FE model to reproduce the connections' behaviour. All nineteen experimental curves (seven HDs in tension and six ABs in shear and tension) were calibrated based on a fitting procedure. Figure 4 displays the experimental and numerical curves of the analysed connections. The curves are derived from the first backbone curve of the connections' cyclic tests.

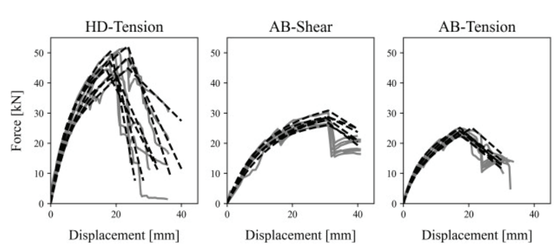


Figure 4: *Experimental data (solid lines) and numerical calibration (dashed lines): HD tension tests (left); AB shear tests (centre); AB tension tests (right)*

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Since ABs are considered with a bi-directional behaviour, thirty-six types of different ABs are defined by combining the six experimental tests in the two main directions.
The experimental data highlights the general scatter between the capacity curves. HDs show a higher variation of the backbone curves compared to ABs, as shown in Figure 4. HDs have a higher scatter in terms of strength compared to the ABs. The main reason is the nature of the failure mechanisms: a sudden significant force drop can be recognised in HD tension tests due to several nails failing simultaneously in shear, including cap tear-off. On the other hands, ABs failure mechanism was a combination of progressive nails yielding, deformation of the brackets and embedment of CLT [9].

2.2 NUMERICAL MODEL OF THE CLT SHEAR WALL AND VALIDATION

A simplified two-dimensional FE model is implemented for the non-linear analyses of the shear wall, as reported in [6]. The model consists of an assembly of rigid and flexible frames to reproduce the CLT wall, whereas zero-length springs schematise the connections (Figure 5). The use of a frame-model instead of area-model for the CLT panels allowed the user to reduce the computational effort. As a shear-type model, the frame model includes the CLT panel deformability through analytical equations due to shear and bending deformation. The horizontal frames placed at the top and the bottom of the wall are assumed as rigid, whereas the two vertical frames on the left and the right edge of the wall are modelled as flexible frames. The flexibility of the vertical frames is calibrated by modifying the second moment of area of the element to account for the overall stiffness of the CLT panel.
All CLT panels are 120 mm thick, made of 5-layers (v30-20-v20-20-v30) with timber strength class C24, where "v" represents vertical orientation of laminations.
Axially rigid friction-contact elements with a Coulomb friction coefficient equal to 0.1 were adopted to simulate the steel-to-timber contact. HDs are modelled with a uni-directional behaviour, while ABs are modelled with bi-directional behaviour. A numerical uncoupled behaviour is assumed for the two orthogonal non-linear laws of the ABs.

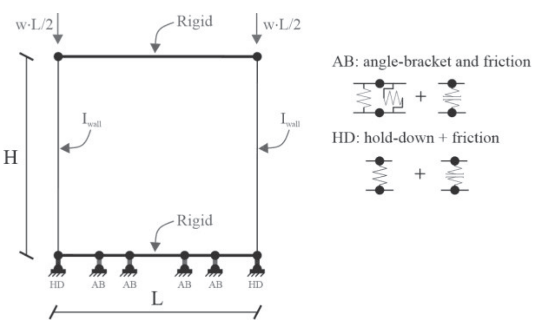

Figure 5: *Finite element model of a CLT shear wall*

The FE model is validated by comparing the experimental response of a CLT shear wall, shown in Figure 6 [7]. The validation was conducted through a NLSA numerical model, where the same connections (HDs and ABs), presented in Section 2.1, were used, in both numerical model and the experimental tests.

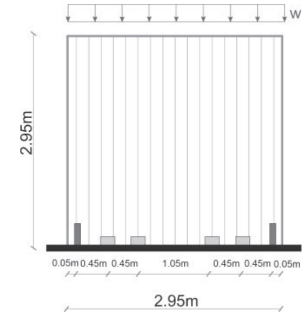

Figure 6: *Validation of the FE model and the parametric procedure of test I.1 [7] with w=18.5kN/m*

The experimental-numerical comparison is made in terms of a mean numerical curve obtained from one thousand different FE shear walls developed with a parametric procedure.
The mean numerical behaviour is reproduced by averaging the parameters of the numerical model respectively for HD in tension and AB in shear and tension. The parametric procedure generated thousand FE shear wall configurations by randomly combining and modifying the properties of every single connection. The parametric shear walls were defined using a random selection among seven different HD responses and thirty-six ABs responses.
As visible in Figure 7, the mean and the parametric curves fit well with the backbone curves of the cyclic experimental test. In addition, for the specific shear wall test, the parametric analysis shows in detail that the maximum strength and the degradation branch have high variability compared to the wall elastic stiffness.

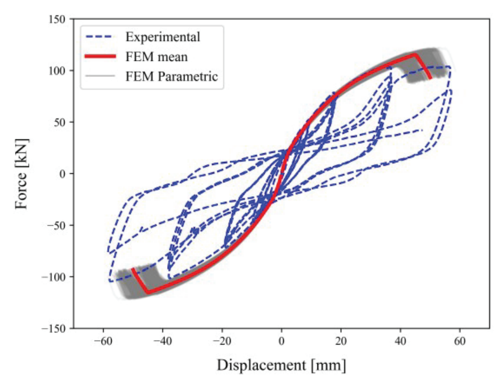

Figure 7: *Numerical validation of test I.1 [7]. Mean numerical curve (red) and a thousand of FE shear walls (gray)*

2.3 CLT SHEAR WALL CONFIGURATIONS

A parametric matrix of CLT shear wall configurations was defined to investigate: *i)* the CLT shear walls response under different conditions and *ii)* the variability of the main engineering parameters (Table 1).

The study considers three different shear wall aspect ratios L:H equal to 1:3 (1 m x 3 m), 1:1 (3 m x 3 m) and 3:1 (9 m x 3 m). The number of ABs is set with 60 cm spacing between the ABs, whereas one HD is placed at each corner of the CLT wall. Three different vertical loads (0 kN/m, 10 kN/m, 20 kN/m) are defined to compare the influence of the gravity loads. A total of nine CLT shear wall configurations are thus defined.

Table 1: *Matrix of the shear walls' configurations for the parametric study*

Vertical load [kN/m]	ID	Aspect ratio L:H		
		1:3 (1m x 3m) 1AB+2HD	1:1 (3m x 3m) 4AB+2HD	3:1 (9m x 3m) 12AB+2HD
0.0	#1	1:3	1:1	3:1
10.0	#2	1:3	1:1	3:1
20.0	#3	1:3	1:1	3:1

2.4 RANDOM SAMPLING AND ANALYSES

Each configuration in Table 1 was analysed with one thousand different shear wall models where each connection has different properties for HDs and ABs randomly generated by combining seven HDs backbone curves and thirty-six types of ABs.

The generation of all shear walls is performed with a Python script using the same random seed to keep the properties and location of each connection constant for the same aspect ratio in all vertical load cases. After applying the vertical loads, each shear wall is analysed through a NLSA with a displacement control approach.

In a post-processing phase, all curves are linearised with EN 12512, and engineering parameters are estimated.

3 RESULTS

Figure 8 shows the pushover curves of all configurations. The outcomes show how the vertical load increases the shear wall's total strength capacity and stiffness. This effect is most evident for 1:3 and 1:1 shear walls where, as expected, the rocking mechanism governed the deformation capacity of the shear wall. The 3:1 aspect ratio is hardly affected by the vertical load variations since the predominant deformation mechanism is sliding. The variability of connection capacity reported in Section 2.1 is evident for 1:3 shear walls, less so for 1:1 shear walls and negligible for 3:1 shear walls. Seven "groups" of pushover curves can be recognised for the 1:3 case. These groups are strictly related to the experimental response of the seven HDs tested since they mostly affect the overall behaviour of the shear wall due to the rocking mechanism. Tables 2-4 list the outcomes in terms of mean value and Coefficient of Variation (CoV) for all configurations, whereas Figures 9-12 represent the CoV of F_y, F_{max}, k_{el} and μ.

As a general trend, the CoV decreases with the length of the shear wall, while they show a limited variation with the vertical loads. The stiffness $k_{el,}$ and $k_{el,p}$ have the same CoV since they depend on each other according to EN 12512.

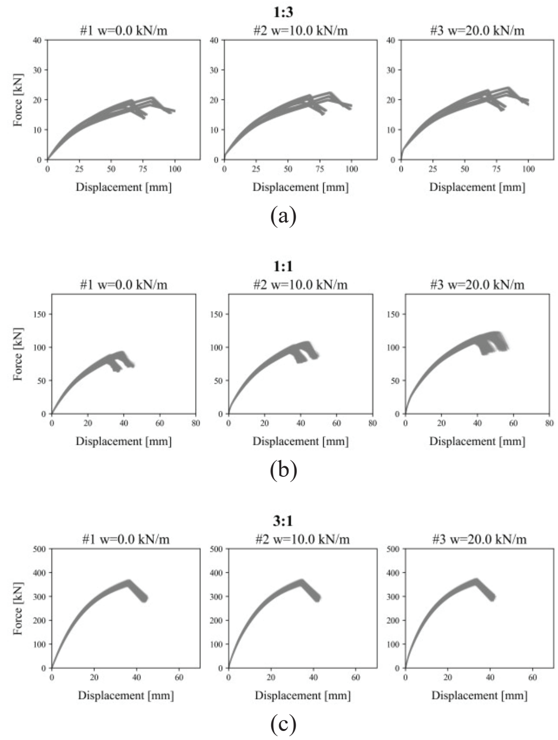

Figure 8: *(a) shear wall 1:3, (b) shear wall 1:1 and (c) shear wall 3:1*

Table 2: *Results of CLT shear wall configuration 1:3*

	1:3			
	w [kN/m]	0.0	10.0	20.0
F_0	mean [kN]	0.39	1.35	1.70
	CoV [%]	16.25	4.01	7.39
F_y	mean [kN]	15.72	17.01	17.64
	CoV [%]	7.96	8.02	8.46
u_y	mean [mm]	33.54	31.90	27.73
	CoV [%]	15.02	15.25	16.78
F_{max}	mean [kN]	18.75	20.41	22.08
	CoV [%]	6.41	5.88	5.44
u_{max}	mean [mm]	71.70	72.68	73.71
	CoV [%]	11.84	11.69	11.53
F_{ult}	mean [kN]	15.20	16.59	18.01
	CoV [%]	6.76	6.58	6.57
u_{ult}	mean [mm]	83.67	85.36	86.94
	CoV [%]	12.84	12.28	11.60
k_{el}	mean [kN/mm]	0.46	0.50	0.58
	CoV [%]	9.67	9.50	9.92
$k_{el,p}$	mean [kN/mm]	0.08	0.08	0.10
	CoV [%]	9.67	9.50	9.92
μ	mean [-]	2.52	2.72	3.20
	CoV [%]	12.67	14.38	15.93

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Table 3: *Results of CLT shear wall configuration 1:1*

	1:1			
	w [kN/m]	0.0	10.0	20.0
F_0	mean [kN]	1.74	5.80	6.39
	CoV [%]	13.52	6.68	6.59
F_y	mean [kN]	71.09	83.93	91.21
	CoV [%]	7.61	4.20	3.97
u_y	mean [mm]	17.56	17.63	16.14
	CoV [%]	11.84	7.92	8.24
F_{max}	mean [kN]	85.37	100.33	115.29
	CoV [%]	5.01	4.27	3.72
u_{max}	mean [mm]	33.90	37.90	43.40
	CoV [%]	10.26	9.95	10.10
F_{ult}	mean [kN]	66.69	78.49	89.49
	CoV [%]	8.53	7.53	8.06
u_{ult}	mean [mm]	39.10	43.46	49.19
	CoV [%]	10.08	9.48	9.44
k_{el}	mean [kN/mm]	3.97	4.45	5.27
	CoV [%]	5.16	5.05	4.87
$k_{el,p}$	mean [kN/mm]	0.66	0.74	0.88
	CoV [%]	5.16	5.05	4.87
μ	mean [-]	2.24	2.48	3.06
	CoV [%]	9.12	10.04	8.48

Table 4: *Results of CLT shear wall configuration 3:1*

	3:1			
	w [kN/m]	0.0	10.0	20.0
F_0	mean [kN]	7.47	13.65	13.65
	CoV [%]	7.27	2.43	3.14
F_y	mean [kN]	293.02	283.65	281.45
	CoV [%]	2.26	1.94	1.96
u_y	mean [mm]	15.20	13.05	11.78
	CoV [%]	3.84	3.71	3.94
F_{max}	mean [kN]	355.49	357.64	361.05
	CoV [%]	1.47	1.47	1.44
u_{max}	mean [mm]	35.88	34.10	33.09
	CoV [%]	1.32	1.53	1.23
F_{ult}	mean [kN]	279.47	281.45	279.46
	CoV [%]	3.84	4.25	5.73
u_{ult}	mean [mm]	44.29	42.52	41.63
	CoV [%]	1.29	1.21	1.13
k_{el}	mean [kN/mm]	18.80	20.70	22.75
	CoV [%]	2.31	2.56	2.76
$k_{el,p}$	mean [kN/mm]	3.13	3.45	3.79
	CoV [%]	2.31	2.56	2.76
μ	mean [-]	2.92	3.26	3.54
	CoV [%]	3.07	2.99	3.28

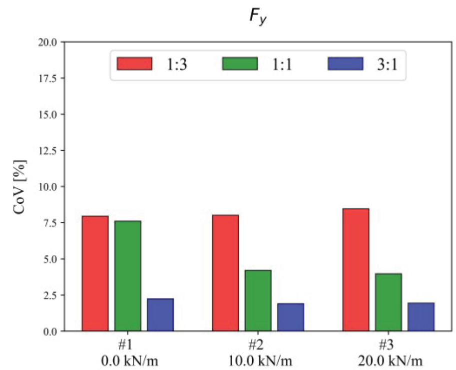

Figure 9: *Coefficient of variation of the yielding strength F_y*

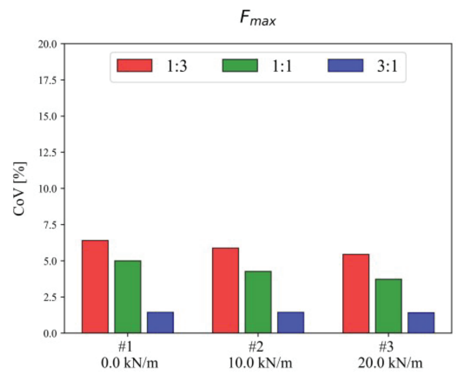

Figure 10: *Coefficient of variation of the maximum strength F_{max}*

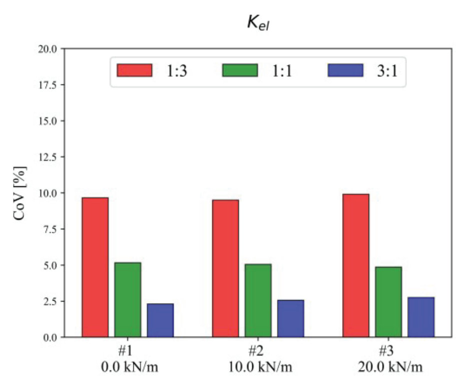

Figure 11: *Coefficient of variation of elastic stiffness k_{el}*

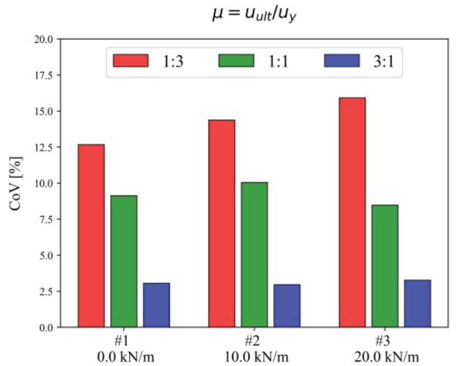

Figure 12: *Covariance of the ductility* μ

Table 5: *Minimum and maximum CoV in percentage*

	1:3 min-max	1:1 min-max	3:1 min-max
F_0	4.0-16.3	6.6-13.5	2.4-7.3
F_y	8.0-8.5	4.0-7.6	1.9-2.3
u_y	15.0-16.8	7.9-11.8	3.7-3.9
F_{max}	5.4-6.4	3.7-5.0	1.4-1.5
u_{max}	11.5-11.8	9.9-10.3	1.2-1.5
F_{ult}	6.6-6.8	7.5-8.5	3.8-5.7
u_{ult}	11.6-12.8	9.4-10.1	1.1-1.3
k_{el}	9.5-9.9	4.9-5.2	2.3-2.8
$k_{el,p}$	9.5-9.9	4.9-5.2	2.3-2.8
μ	12.7-15.9	8.5-10	3.0-3.3

To compare the overall variability, Table 5 summarises the minimum and maximum CoV of all parameters of the three configurations. The following remarks can be done:

- The CoV tends to decrease with the length of the shear wall;
- The strength F_0 shows the most significant variation among all configurations, with the lowest minimum and highest maximum coefficient of variation observed in configuration 1:3 due to the significant role played by the vertical loads;
- The strength F_y is relatively stable across all configurations, with a low coefficient of variation observed in all cases;
- The displacement u_y exhibits significant variations in all configurations, with a higher CoV than the other parameters;
- The parameters F_{max}, u_{max}, F_{ult}, and u_{ult} show variability across the different configurations, but the differences in the CoV among configurations are relatively small;
- The stiffnesses k_{el} and $k_{el,p}$ have the same CoV across all configurations (due to the EN 12512 linearisation);
- The ductility μ significantly varies among the configurations, with the lowest minimum and highest maximum coefficient of variation observed in configuration 1:3.

4 CONCLUSIONS

This paper investigates the variability of Cross-Laminated Timber (CLT) single-panel shear walls' response under in-plane lateral loads. The variability is considered by performing a parametric analysis on three different CLT shear wall configurations consisting of one thousand different finite element shear walls. The mechanical properties of all connections, based on experimental results, are randomly sampled to investigate several possible scenarios of the shear wall.

The results show that all mechanical properties, such as strength (yield, maximum, ultimate), stiffness (elastic, post-elastic), deformations (yield, maximum, ultimate) and ductility are influenced by the geometry of the shear wall and the vertical loads. As a general trend, the Coefficient of Variation (CoV) decreases with the length of the shear wall, whereas it shows limited variation with the vertical loads. All parameters have a different range of minimum and maximum CoV, varying from 1% to 17%.

Future studies are planned to investigate additional shear wall parameters and configurations and extend the analysis to multi-panel CLT shear walls.

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