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EXPERIMENTAL CHARACTERIZATION OF STIFF ALUMINIUM CONNECTORS FOR MULTI-PANEL CLT SHEAR-WALLS

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ABSTRACT: Cross-Laminated Timber (CLT) shear-walls composed of multiple panels are nowadays a viable alternative to single-panel CLT walls in mid-rise construction sector. Prefabrication process, transport and erection limits are some of the reasons why short panels, vertically-jointed by means of mechanical fasteners, are preferred to CLT single-panel walls. When a significant amount of energy dissipation in the vertical joint is not required (i.e. low seismic prone areas), stiff vertical joints are preferred: the wall behaves mostly like a single-panel wall and vertical joints are designed to minimize the relative displacement between the panels. This paper presents an experimental campaign on aluminium stiff shear-key connector, as an alternative to traditional screwed connections for stiff vertical joints in multi-panels CLT walls. Monotonic tests were performed on shear-key connectors and full-scale tests on multi-panel CLT shear-walls were carried out. A numerical model was developed in order to extend the experimental results on full-scale tests by means of a sensitivity analysis.

KEYWORDS: Cross-laminated timber, Multi-panel, shear walls, Shear-key connectors, Hold-down, Vertical joints.

1 INTRODUCTION

Cross-Laminated Timber (CLT) shear-walls composed of multiple panels are nowadays a viable option for mid-rise buildings. Prefabrication process, transport and erection limits are some of the reasons for which short panels, vertically-jointed by means of mechanical fasteners, are preferred to CLT single-panel walls. The lateral load resistance of multi-panel walls is mainly provided by mechanical connections while CLT panels behave mostly like rigid elements. Either nailed or screwed connections are commonly adopted for connecting the panels.

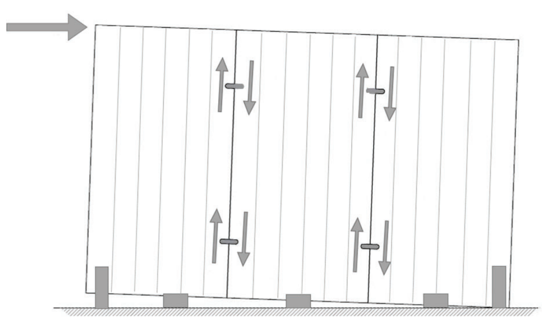

Figure 1: Multi-panel shear wall with stiff connectors

As derived by Casagrande et al. and Masroor et al. [1,20], the relative stiffness and strength capacity of vertical joints and hold-downs have a significant influence on the behaviour of multi-panel CLT shear-walls. When a significant amount of energy dissipation is not required (i.e. low seismic prone areas), relatively stiff vertical joints are preferred: the wall behaves mostly as single-panel wall, see Figure 1, and vertical joints are designed to minimize the relative displacement between the panels so that the stiffness and strength capacity of the wall is approximately equal to that of a single panel wall.

1.1 LITERATURE REVIEW

Several studies have been conducted in order to characterize the mechanical behaviour of CLT panel-to-panel joints. Monotonic and cyclic tests were carried out by Gavric et al. [2] on screwed half-lap and spline joints, showing high ductility and energy dissipation. Hossain et al. [3] performed monotonic tests on spline, half-lap and butt joints where fully- and partially-threated screws with different inclinations were adopted.

Loss et al. [4] presented an experimental campaign on butt joints with crossed self-tapping screws inserted in CLT panels with different inclinations. Half-lap and spline joints connected with partially- and fully-threated self-tapping screws were tested by Sullivan et al. [5], comparing the experimental results with the analytical preditcion in terms of strength and stiffness.

More recently, innovative steel devices have been presented as an alternative to traditional screwed connec-

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tions. Schmidt and Blaas [6] developed a highly dissipative connection made with thin steel plate inserted perpendicularly to the vertical joint. Marchi et al. [7] performed cyclic tests on an innovative X steel bracket, showing high ductility, negligible strength degradation and a limited pinching behaviour.

A significant number of full-scale testes on multi-panel CLT shear-walls with vertical joints assembled with dowel-type fasteners has been conducted. Two-panel CLT shear-walls with half-lap and spline screwed vertical joints were tested by Gravic et al [8], showing an almost rigid behaviour of the CLT panels and a significant energy dissipation in the vertical joints. The cyclic behaviour of single- and multi-panel CLT shear-wall was also investigated by Popovski and Karacabeyli [9].

Conversely, a limited number of tests has been performed with the aim to characterise the behaviour of multi-panel shear walls with stiff vertical joints. This type of shear-walls, regardless the smaller energy dissipation capacity than shear-walls with flexible vertical joints, may be a viable alternative in low seismic prone areas.

1.2 OBJECTIVE

This paper presents an experimental campaign on a innovative stiff aluminium shear-key connector developed with the aim of providing an alternative to nailed or screwed connections for stiff vertical joints in multi-panel CLT walls. The connector is named "SLOT" and is produced by Rotho Blaas company [10]. SLOT connector is made with aluminium EN AW-6005A T6 (yield tensile strength equal to R_y = 225 N/mm^2 and ultimate tensile strength equal to R_u=270 N/mm^2). The length, the width and the thickness of the SLOT connector are equal to l_{SL} = 120 mm, b_{SL} = 89 mm and t_{SL} = 40 mm, respectively, see Figure 2.

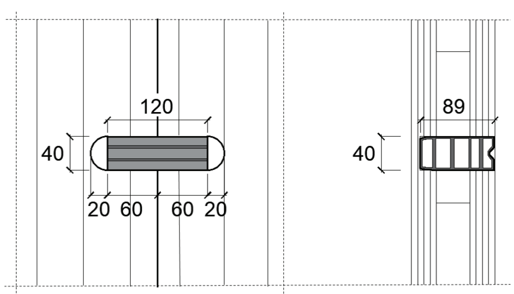

Figure 2: *Aluminium SLOT connector*

Monotonic shear tests were performed both at connector and shear wall level. Numerical finite element models were implemented to predict the mechanical behaviour of tested shear-walls.

2 MONOTONIC TESTS ON STIFF SHEAR-KEY CONNECTORS

2.1 MATERIALS AND METHODS

Monotonic shear tests were performed on CLT panel-to panel joints assembled with SLOT connectors.

Two grooves of dimensions equal to 90x60x40 mm were cut in the panels in order to insert each SLOT perpendicular to the vertical joint line and aligned to the outer face of CLT panels.

Different specimen configurations using different types of CLT panels were assembled for a total of 10 tests. The total thickness and the layout the of CLT panels are reported in Table 1. Tests on spline joints were also performed and the results were compared with those obtained in tests with SLOT connectors. The spline joints were made by connecting the CLT panels to a 27x150 mm LVL beech board by means of 6x70 mm screws, see Figure 3. For spline joints two different values of spacing (50, 100 mm) were tested, yielding a total of 4 tests (Table 1).

The mean value of the moisture content of the CLT panels was equal to 10.1% while the 5th percentile and the mean value of density of the CLT panels were equal to 402 kg/m^3 and 454 kg/m^3, respectively.

Table 1: *Test layout on SLOT connectors and screwed vertical joint*

Test on joints with SLOT connectors			
ID	Test	CLT thickness	CLT layout
SL01	I II III	90 mm	3s (24-42-24)
SL02	I II	100 mm	3s (33-34-33)
SL03	I	100 mm	3s (29-42-29)
SL04	I II III	100 mm	5s (20-20-20-20-20)
SL05	I	145 mm	5s (20+20-20-20-20-45)*
*Slot connector aligned to the outer layer with thickness equal to 20 mm			

Test on joints with screwed connections				
ID	Test	Screw spacing	CLT thickness	CLT layout
SP01	I II	50 mm	100 mm	3s (30-40-40)
SP02	I II	100 mm	100 mm	3s (30-40-40)

The inner panel of each specimen was vertically loaded, while the two outer panels were laterally restrained. The load was applied with a constant load rate equal to 0.1 mm/s and according to EN 26891 [11] the failure occurred within 300 seconds. The relative slip between the inner and the outer panels was measured by means of four LVDTs and the load applied was measured by a 250 kN load cell.

The results in terms of maximum shear load F_{max} and stiffness K_s are reported in Table 2 and 3.

For each specimen the slip value was determined as the average from four LVDTs. The shear load F acting on each single connector was obtained from the ratio between the total shear load measured by the load cell and the number of connectors. The value of stiffness and shear load for spline joints is reported per couple of screws.

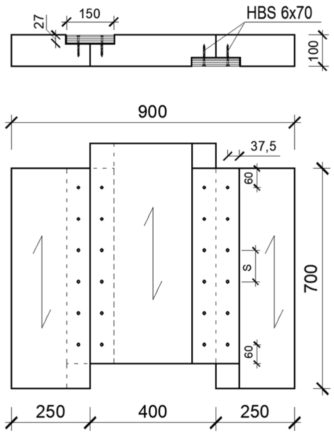

Figure 3: *Monotonic shear tests on spline joint*

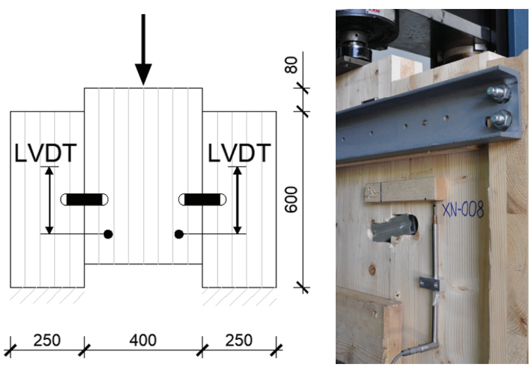

Figure 4: *Monotonic shear tests on aluminium CLT-to-CLT shear-key connector*

2.2 RESULTS AND DISCUSSION

Tests conducted on specimens with SLOTs showed a rigid body rotation of the connectors with no residual plastic deformation. A failure mode characterised by the localized crushing of the wood was observed. In spline joint tests, a plastic deformation of the screws and a local embedment crushing of the wood around the fasteners were detected.

The load-slip curves for test SL01-II and SP01-II are reported in Figure 5. The gap between the SLOT connector and the CLT panels caused an initial slip (0.3 and 0.5 mm) after which an elastic-almost perfectly plastic behaviour is observed. The initial slip was not observed in the tests on screwed joints where, conversely, a significant rope effect after the yielding of the fasters was detected.

The effective number of screws equal to a single SLOT connector in terms of strength and stiffness $n_{eff,Fmax}$ and $n_{eff,Ks}$ have been defined with the aim to compare the performance of joints with SLOT connectors to those with screwed joints. Using the average values of strength and stiffness determined from Table 3 and assuming an effective width $b_{eff,SL}$ of the SLOT connectors equal to 53 mm (see Figure 2, section), the effective number of screws $n_{eff,Fmax}$ and $n_{eff,Ks}$ are equal to 12.6 and 13.0 respectively.

Table 2: *Test results on SLOT connectors*

ID	Test	F_{max} [kN]	K_s [kN/mm]
SL01	I	60.35	13.99
	II	62.30	12.69
	III	58.74	14.75
SL02	I	53.83	14.91
	II	63.24	17.89
SL03	I	60.92	15.43
SL04	I	62.56	13.09
	II	74.88	15.56
	II	71.11	15.34
SL05	I	68.12	13.30

Table 3: *Test results on spline joints*

ID	Test	F_{max} [kN]	K_s [kN/mm]
SP01	I	5.37	0.88
	II	4.75	1.57
SP02	I	5.10	0.92
	II	4.90	1.17

The load and stiffness values are reported per couple of screws

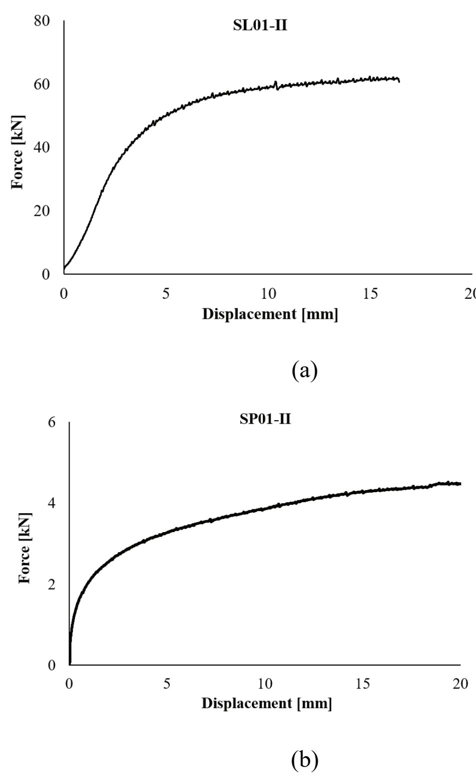

Figure 5: *Load-vs-displacement curves for test on (a) SLOT connectors (b) spline joints*

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3 FULL-SCALE TESTS ON MULTI-PANEL CLT SHEAR-WALLS

3.1 TEST-LAYOUT AND SET-UP

Two monotonic full-scale tests were carried out to investigate the mechanical behaviour of 3-panel 100 mm thick CLT [13] shear-walls with SLOT connectors, see Figure 6a and 6b. Either two or four connectors along each vertical joint were used. An additional multi-panel shear-wall with spline vertical joints was tested, see Figure 6c. As in shear tests on panel-to-panel joints, 6x70mm screws were used with a spacing of 50 mm.

Two typologies of hold-down (WHT 340 and WHT620) were used to limit the rocking of shear-walls, whereas the sliding of each panel was prevented by the contact of the panels bottom edges with properly designed steel devices. The same displacement was applied on the top of each panel according to the analytical model developed by Casagrande et al. (2018) [1]; no vertical load was applied. In accordance with EN 594 [14], displacement-controlled monotonic tests with load rate equal to 0.2 mm/s were adopted.

3.2 ANALYSIS METHOD

Two transducers (LVDTs) were used in order to measure the top lateral displacement of shear-walls, while another LVDT was used to measure the panel sliding.

The difference between the total lateral displacement and the sliding represents the lateral displacement Δ due to the rocking behaviour and the panel deformation.

Trilinear curves were drawn in order to approximate the experimental load-displacement curves. These curves were obtained by connecting the yield point Y, the maximum point M and the ultimate point U determined according to the procedure reported in EN 12512 [15]. The ratio between the yielding load F_Y and the yielding displacement Δ_Y represent the elastic stiffness K, while the ratio between the yielding displacement Δ_Y and the ultimate displacement Δ_U represent the ductility of the shear-wall μ.

Table 4: *Shear-wall test layout*

Test	N. of panels	Type of connector in vertical joint	N. of connector	Hold-down
Wall-I	3	SLOT connector	2	WHT 340
Wall-II	3	SLOT connector	4	WHT 620
Wall-III	3	Spline joint – 6x70 mm screws	44 / spacing 50 mm	WHT 340

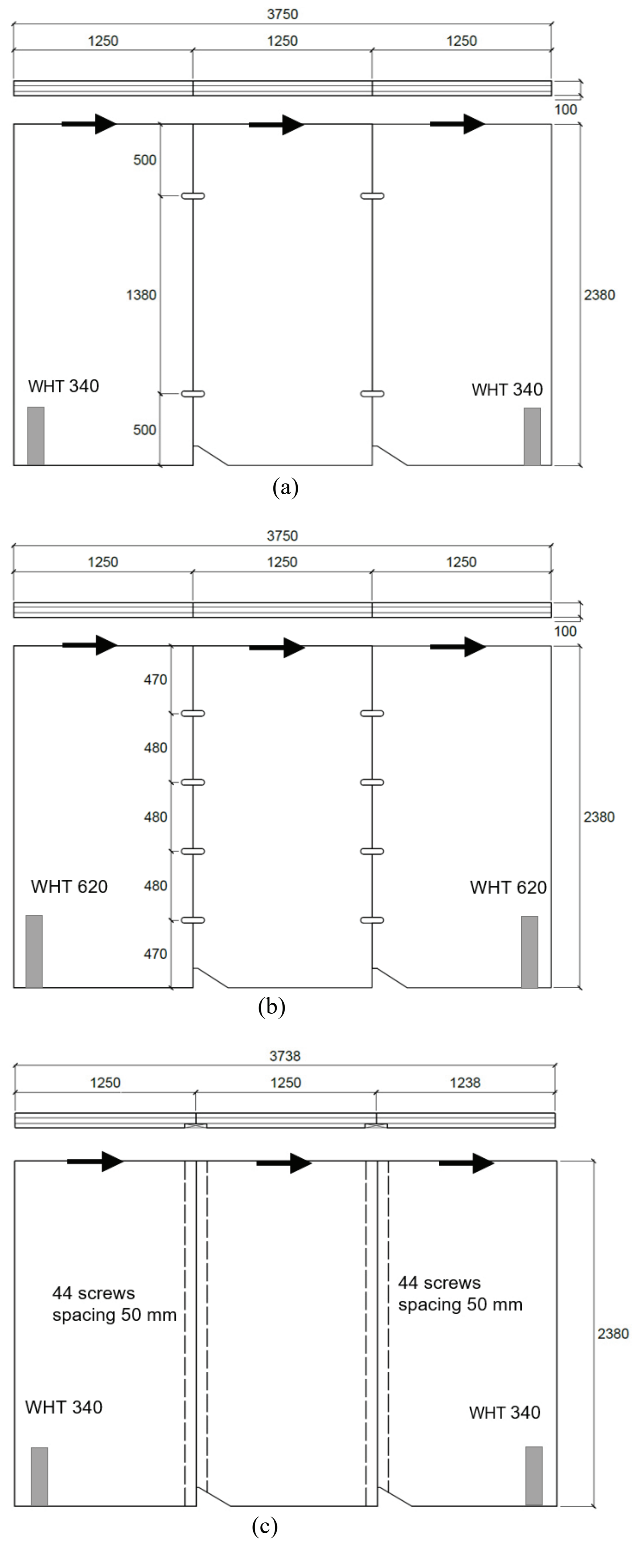

Figure 6: *Shear-wall test specimens (a) Wall-I, (b) Wall-II, (c) Wall-III*

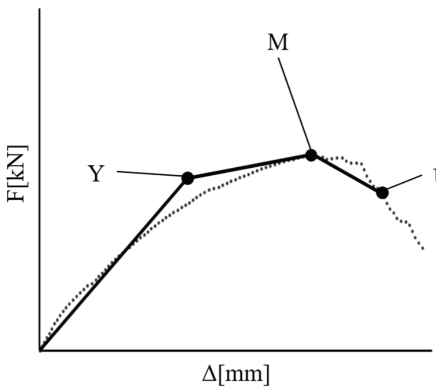

Figure 7: *Load F vs displacement Δ curves, trilinear approximation.*

3.3 RESULTS AND DISCUSSION

All the tested walls showed a kinematic mode characterised by a single global centre of rotation (see Figure 8), as also confirmed by a value of uplift of the hold-down greater than the panel-to-panel slip, see Table 5.

Table 5: *Uplift of hold-down and relative panel-to-panel slip corresponding to the maximum lateral load*

Test	Maximum load	Uplift hold-down	Slip panel 1-panel 2	Slip panel 2-panel 3
	F_M [kN]	v_{H-D} [mm]	$\delta_{1.2}$ [mm]	$\delta_{2.3}$ [mm]
Wall-I	115	19.5	2.0	2.0
Wall-II	167	16.0	1.1	1.2
Wall-III	120	20.8	2.4	3.5

The failure of the hold-down was achieved in all three tests: ductile failure with plastic deformation of nails was observed for WHT340 in Wall-I, while in Wall-III a brittle failure of the steel plate was observed. A brittle failure of the steel plate occurred also in the test on Wall-II, where WHT620 was used. Either SLOT connectors and screwed joints exhibited an almost elastic behaviour with no residual deformations. The force and displacement values related to the yield point Y, the maximum point M and the ultimate point U are reported in Table 6, whereas the experimental load-vs-displacement curve and the corresponding trilinear curve are reported in Figure 7.

Table 6: *Mechanical parameters obtained from tri-linear curves of shear-wall tests*

Test	F_Y	Δ_Y	F_M	Δ_M	F_U	Δ_U	K	μ
	[kN]	[mm]	[kN]	[mm]	[kN]	[mm]	[kN/mm]	[-]
Wall-I	101	9.4	115	17.4	92	21.8	10.7	2.3
Wall-II	141	5.9	167	13.5	167	13.5	23.8	2.3
Wall-III	86	5.0	120	19.5	116	20.2	17.3	4.1

As shown in Figure 9, the load-vs-displacement curves of the test on Wall-I and that on Wall-III are comparable. The maximum shear load reached in both tests is very similar, but due to the physical gap between the SLOT connector and the CLT panels, the stiffness of Wall-I is lower than that of Wall-III.

Figure 8: *Centre of rotation in a monotonic full-scale test on 3-panel CLT shear-wall with SLOT connector*

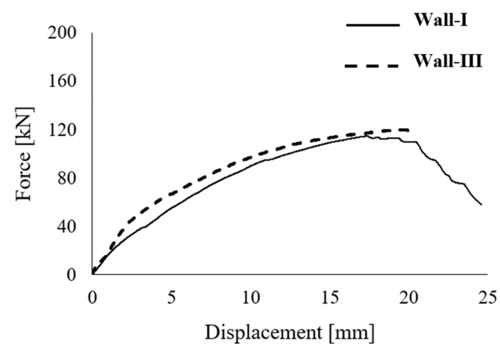

Figure 9: *Load-vs-displacement curves obtained from Wall-I and Wall-II*

4 FINITE ELEMENT NUMERICAL ANALYSES ON SHEAR-WALLS

Finite Element (FE) numerical models were implemented with the commercial software SAP2000 [16] to predict the mechanical behaviour of the three tested shear-walls, see Figure 10. Orthotropic homogenous elastic 100 mm thick shell elements were adopted for the modelling of CLT panels. Effective modulus of elasticity $E_{eff.v}$ and $E_{eff.h}$ were defined along the vertical and horizontal direction. Respectively, according to the composite theory presented by Blass and Fellmoser [17] and adapted to CLT panels as expressed by Equations 2 and 3:

$$\boldsymbol{E_{eff.v}} = \frac{\boldsymbol{E_0 \cdot \sum t_{v.i}}}{\boldsymbol{t}} = \mathbf{7200}\boldsymbol{MPa} \tag{2}$$

$$\boldsymbol{E_{eff.h}} = \frac{\boldsymbol{E_0 \cdot \sum t_{h.j}}}{\boldsymbol{t}} = \mathbf{4800}\boldsymbol{MPa} \tag{3}$$

where E_0 is the modulus of elasticity of wooden boards equal to 12000 MPa, t is the total thickness of the panel equal to 100 mm, $\sum t_{v.i}$ and $\sum t_{h.j}$ are the total thickness of the vertical and horizontal layers equal to 60 mm and 40 mm, respectively.

An effective in-plane shear modulus G_{eff} equal to 479 MPa was assigned to shell elements according to the expressions proposed by Brandner et al. [18] to take into account the effective shear and torsional deformation of wooden boards. Rigid gap elements were adopted to model the contact of CLT panels with the ground. The shear-key connectors or the screwed connections along the vertical joints were modelled by 2-joint uni-axial multi-linear link elements.

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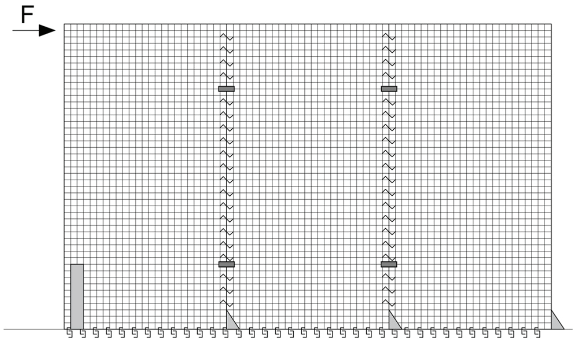

Figure 10: *FE numerical model for Wall-I*

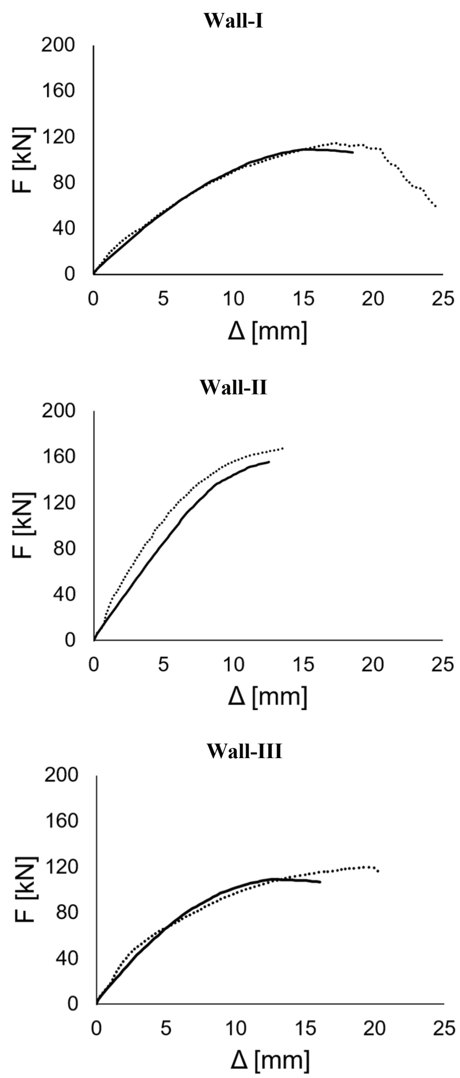

Figure 11: *Load-vs-displacement curves obtained from FE model (continuous line) and shear-wall tests (dot line)*

The multi-linear behaviour assigned to the link along the direction parallel to vertical joint was defined elaborating the experimental curves obtained from the tests on connectors presented in Section 3. In order to simulate the panel-to-panel contact, rigid gap elements acting along the direction perpendicular to the joint were used.

A multi-linear 1-joint uni-axial link element was adopted to model the hold-down when subjected to a vertical tensile load. In order to characterize the non-linear behaviour of hold-down link element, four monotonic tests were carried out on WHT 340 hold-down. The experimental curve for WHT620 is conversely available in Casagrande et. al 2021 [19].

A diaphragm constraint was applied at joints of the FE model on the top of each panel. Non-linear static analyses were performed by increasing the lateral displacement at the top of the wall from zero up to the value corresponding to the global failure condition.

The validation of the FE models with the experimental results was carried out in terms of load F vs displacement Δ curves as shown in Figure 11. It is noteworthy to mention that the values of the lateral displacement Δ were measured in the model at the same location of the LVDTs used during the tests to monitor the lateral displacement of the shear-wall. A reasonable fit is obtained for all analysed tests.

5 CONCLUSIONS

This paper presents an experimental campaign on aluminium rigid shear-key SLOT connectors. The tests were carried out both at connector and wall level. The curves related to connector tests, after an initial slip due to the gap between the connector and the CLT panels, showed an almost elasto-perfectly plastic behaviour.

At wall level, a single-wall kinematic mode with a single global centre of rotation was observed; all tested wall specimens showed a failure mode related to the hold-down.

The shear-wall with two SLOT connectors in each vertical joint (Wall-I) and the shear-wall with screwed vertical joints (Wall-III) showed similar behaviors: the maximum shear load was almost the same, but, due to the physical gap between the SLOT connector and the CLT panels, the stiffness calculated analytically was smaller in Wall-I than that in Wall-III.

The load-vs-displacement curves obtained from the FE numerical model reasonably fit the experimental curves obtained from the shear-wall tests. The numerical models can hence be adopted to predict the mechanical behaviour of multi-panel with rigid shear-key connectors for different geometrical dimensions and mechanical properties.

The results of the experimental and numerical investigations showed how these connectors can be considered to be a viable alternative to traditional connections for stiff vertical joints in multi-panels CLT walls.

6 ACKNOWLEDGEMENTS

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