MECHANICAL BEHAVIOUR OF MULTI-PANEL CROSS LAMINATED TIMBER SHEAR-WALLS WITH STIFF CONNECTORS

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Abstract

Multi-panel Cross Laminated Timber shear-walls with stiff vertical joints may represent a valuable structural solution for multi-storey timber buildings in low seismic prone areas. In this study, the opportunity of using shear-key connectors in the assemblage of stiff vertical joints as an alternative to short-spacing traditional dowel-type mechanical fasteners is evaluated. The mechanical behaviour of two types of stiff shear-key connectors is investigated through an experimental campaign conducted at joint and wall level. Shear tests were firstly carried out to determine the mechanical properties of panel-to-panel joints assembled with the shear-key connectors. The results are compared with those obtained from shear tests on joints with screwed connections (half-lap and spline joints). Full-scale shear-wall tests were then performed to determine the mechanical performances of the connectors at wall level. Joints with a single shear-key connector showed values of stiffness and strength comparable to those that would be obtained for vertical spline joints with approximately thirteen couples of 6 mm screws. The failure modes on the joints with shear-key connector were characterized by either a localized crushing in the wood or the shear failure in outer lamination of the CLT panels with no plastic deformation of the connectors.

Keywords: Cross Laminated Timber; multi-panel; vertical joints; screws; lateral load; disassembling.

1. Introduction and background

Cross Laminated Timber (CLT) shear-walls composed of multiple panels vertically joined by means of mechanical dowel-type fasteners (e.g. nails or screws) represent nowadays a widespread structural type for low-to-midrise timber buildings. The faster prefabrication process and the simpler transport of narrow panels in addition to the geometrical limitations in the production of large panels are some of the reasons for which segmented walls may be preferred to single-panel walls in the construction of multi-storey CLT buildings.

The lateral load resistance of multi-panel CLT shear-walls is mainly related to the strength capacity of the mechanical connections while wooden panels behave mostly like rigid elements [1]. Mechanical anchors, such as hold-downs and angle brackets, are typically adopted to prevent the rigid body rotation and the sliding of shear-walls, respectively, whereas vertical joints are designed to transfer the internal shear actions between adjacent panels.

The relative stiffness and the relative strength capacity between the vertical joints and the mechanical anchors are the key parameters which determine the rocking kinematic modes of multi-panel CLT shear-walls [2]. Relatively stiff vertical joints are commonly adopted in order that shear-walls behave mostly like single-panel walls - i.e. single-wall (SW) behaviour, see Figure 1a [3,4]. Conversely, relatively flexible vertical joints are typically designed to yield and dissipate energy – i.e. coupled panel (CP) behaviour, see Figure 1b [5].

According to the Canadian Standards for timber structures [6], CLT shear-walls made with panels with aspect ratios (height-to-length) between 2:1 and 4:1 may exhibit a moderate-to-high ductile behaviour dissipating the seismic energy through the yielding of dowel-type fasteners in the vertical joints. Conversely, shear-walls that are constructed with panels with aspect ratios smaller than 2:1 exhibit a limited energy dissipation capacity and shall be designed for forces calculated assuming an almost elastic behaviour of the structure. Similarly, in the proposal for the revision process of timber chapter of Eurocode 8 [31], multi-panel CLT shearwalls designed to ensure the yielding of mechanical fasteners in vertical joints are able to achieve high values of ductility (i.e. high ductility class) whereas either single-panel or multi-panel shearwalls that behave primarily as single-panel walls exhibit a medium ductility capacity (i.e. medium ductility class).

In recent years, several studies have been conducted with the aim to characterize the mechanical behaviour of CLT panel-to-panel joints. Gavric et al. [7] carried out both monotonic and cyclic tests on screwed half-lap and spline joints under two different loading directions, namely parallel and perpendicular to the joint. High ductility capacity and energy dissipation of such connections were observed. Hossain et al. [8] performed monotonic tests on spline, half-lap and butt joints where either 90° oriented partially-threaded or 45° oriented fully-thread screws were adopted to connect the CLT panels. The results showed a moderate ductility capacity of 90° oriented screws (primarily subjected to a shear load) with large relative displacements between the CLT panels. Limited ductility and high strength capacity was conversely observed for 45° oriented fully-thread screws (primarily subjected to an axial load).

An extended experimental campaign on butt joints with crossed self-tapping screws inserted in CLT panels with different inclinations was conducted by Loss et al. [9]. The influence of the insertion angle of screws, the friction and the loading direction on the joint performance in terms of strength capacity, stiffness and ductility was investigated. Sullivan et al. [10] tested half-lap and spline joints connected with partially- and fully-threaded self-tapping screws with the aim to compare the experimental values of strength and stiffness with those calculated from the formulas reported in the National Design Specification for Wood Construction [11] and in Eurocode 5 [12].

As an alternative to traditional screwed connections, Stecher et al. [13] studied the use of dovetail connectors, fabricated from Beech plywood, to transfer shear along the edges of CLT members while Schmidt and Blass [14] investigated the mechanical behaviour of ten different types of wooden (i.e. Beech or Radiata Pine) contact joints for panel-to-panel CLT joints. Schmidt and Blass [15] also conducted an experimental campaign on Beech LVL stiff shear-key connectors for CLT panel-to-panel joints both in floors and shear-walls, showing high values of stiffness and strength capacity. The mechanical performance of double dovetail shape X-fix connectors made from birch or beech veneer plywood and used as shear connectors to connect CLT panels each other was investigated by Ayansola and Tannert [32].

More recently, innovative steel devices have been developed to achieve high energy dissipation in panel-to-panel vertical joints in CLT shear-walls. A highly dissipative connection with ductile thin steel plate, inserted perpendicular to the joints, was developed by Schmidt and Blass [16]. An X steel bracket was presented by Marchi et al. [17]: the cyclic tests showed high values of ductility, negligible strength degradation and limited pinching behaviour. Hashemi et al. [18] proposed Resilient Slip Friction (RSF) joints developed to couple CLT panels each other or to connect CLT panels to steel columns in timber-steel hybrid structures.

Several experimental tests have been conducted on full-scale multi-panel CLT shear-walls with vertical joints assembled with dowel-type fasteners and designed to yield. Gavric et al. [1] performed cyclic tests on 2-panel CLT shear walls, with half-lap and spline screwed vertical joints, showing an almost rigid behaviour of the wooden panels and a significant energy dissipation in the vertical joints. Popovski and Karacabeyli [19] performed quasi-static monotonic and cyclic tests on single- and multi-panel CLT shear-walls while Yasumura et al. [20] compared the mechanical behaviour of single- and multi-panel CLT shear-walls through full-scale tests and numerical analysis on low-rise CLT structures.

A limited number of tests has been carried out on multi-panel shear-walls with stiff vertical joints designed to behave almost elastically. Despite the more limited ductility and the smaller energy dissipation capacity than shear-walls where the yielding of vertical joints is engaged, multi-panel shear-walls with stiff vertical joints may represent a valuable structural solution in low seismic prone areas where the lateral load resisting systems are designed to behave elastically and resist primarily wind loads.

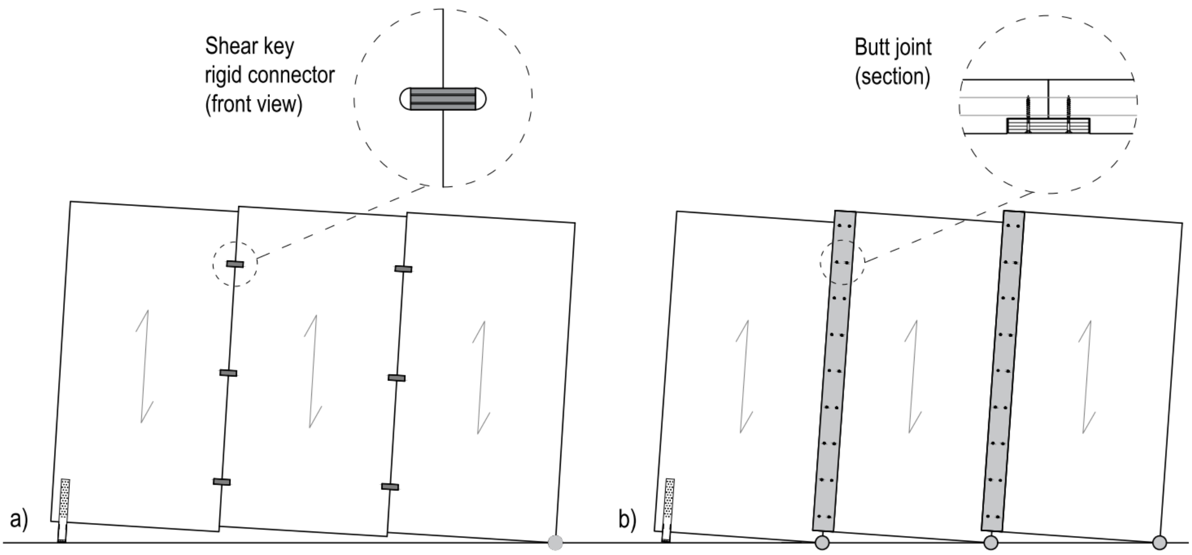


Figure 1: *Multi-panel CLT panels assembled with stiff shear-key connectors (a) and traditional dowel-type mechanical fasteners (b); wall kinematic modes CLT panels single-wall-SW (a) and coupled-panel - CP (b)*

This study aims to investigate the opportunity of using stiff shear-key connectors in the assemblage of stiff vertical joints (Figure 1a) as an alternative to short-spacing traditional dowel-type mechanical fasteners (Figure 1b). In particular, the mechanical behaviour of two types of stiff shear-key connectors is investigated The former connector, known as SLOT connector, is made with extruded aluminium and produced by Rothoblaas company (Figure 2a); the latter, made with LVL beech, was studied and developed by the Karlsruhe Institute of Technology in Germany [14,15] (Figure 2b).

An experimental campaign at joint and wall level was conducted. Shear tests were firstly carried out at joint level to determine the mechanical properties of panel-to-panel joints assembled with the shear-key connectors and compare them with those obtained from shear tests on joints with screwed connections. Full-scale shear-wall tests were then performed to determine the mechanical performances of the connectors at wall level. Additional shear-wall tests were also conducted on single-panel CLT shear-walls and multi-panel CLT shear-walls with screwed vertical joints to compare their mechanical performances with those of shearwalls with shear-key connectors.

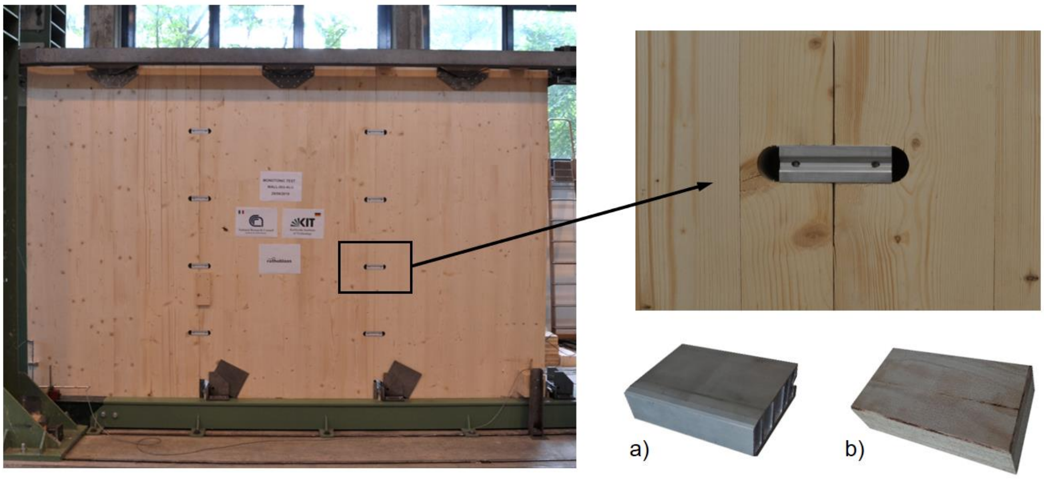


Figure 2: *Multi-panel CLT shear-wall with SLOT shear-key connectors and details of SLOT a) and LVL connectors b)*

2. Shear tests on panel-to-panel joints

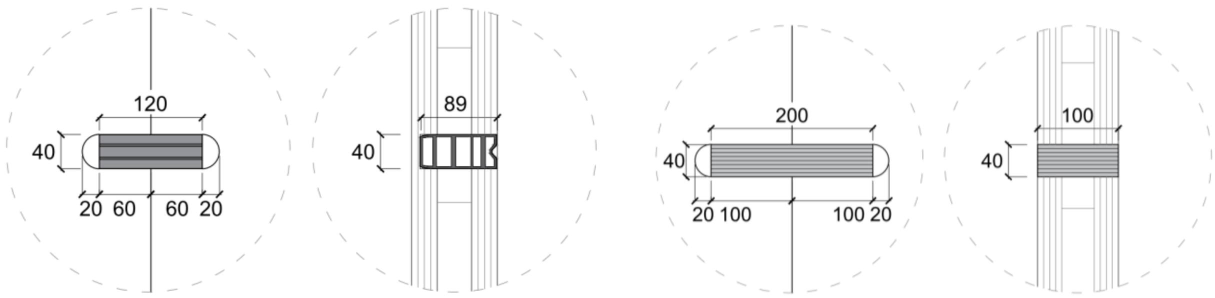
2.1 Materials and methods

In this section the results obtained from monotonic shear tests carried out on CLT panel-to-panel joints assembled with either SLOT connectors, LVL Beech connectors and screwed connections are discussed.

SLOT connector is made with aluminium EN AW-6005A T6 (the yield and ultimate tensile strength are equal to $R_{p0,2}$ = 225 N/mm² and R_m = 270 N/mm², respectively) and characterized by a length l_{SL} equal to 120 mm, a width b_{SL} equal to 89 mm and a thickness t_{SL} equal to 40 mm according to ETA-19/0167 [21]. Each SLOT connector was inserted in two grooves of dimensions equal to 90x60x40 mm and was aligned to the outer face of CLT panels, as shown in Figure 3a.

Two types of joints assembled with screwed connections were tested, namely spline and half-lap joints. The spline joints were made by connecting the CLT panels to a 27x150 mm LVL Beech board by means of 6x70 mm screws [30] whereas 8x100 mm screws [30] were adopted in the half-lap joints.

No shear test was carried out in this study on joints with LVL Beech connectors. Adequate information was in fact obtained from the experimental campaign conducted by Schmidt and Blass [15] at joint level on LVL Beech connectors characterized by a width b_{LVL} equal to 100 mm, a thickness t_{LVL} equal to 40 mm and a length between 80 and 280 mm (Figure 3b).



a) b)

Figure 3: *Geometrical dimensions of SLOT (a) and LVL connector (b)*

The wooden specimens for all shear tests were made with CLT panels produced according to ETA 12/0362 [22]. A mean value of the moisture content equal to 10.8% was measured while the 5th percentile and the mean value of density were equal to 402 kg/m^3 and 454 kg/m^3, respectively. A symmetric test setup characterized by three CLT panels with the outer layers parallel to the load direction was equipped.

Either one or two SLOT connectors were used along each joint, see Figure 4a and 4b, while three different values of spacing of the screws, equal to either 50 mm, 100 mm or 150 mm, were adopted for tests on spline (Figure 4c) and half-lap joints (Figure 4d). The total thickness and the layout of the CLT panels as well as the number of SLOT connectors or the spacing of the screws are reported in Table 1, yielding a total of 16 tests on joints with SLOT connectors and 4+4 tests on spline and half-lap screwed joints, respectively. The dimensions of CLT specimens and the joint layout for the tests are shown in Figure 4.

With the only exception of 90 mm 3-layer (tests SL 01-02-03) and 100 mm 5-layers panel (test SL 07-08-09) for which three replicates were performed, either one or two replicates were carried out on the other configurations. This choice was made primarily in order to increase the number of parameters to be investigated, such as the panel layouts, the number of connectors and the edge distance, without significantly increasing the total number of tests at connection level. It is noteworthy to mention that additional tests should be carried out in order to provide a more reliable statistical distribution of strength and stiffness values of such connectors.

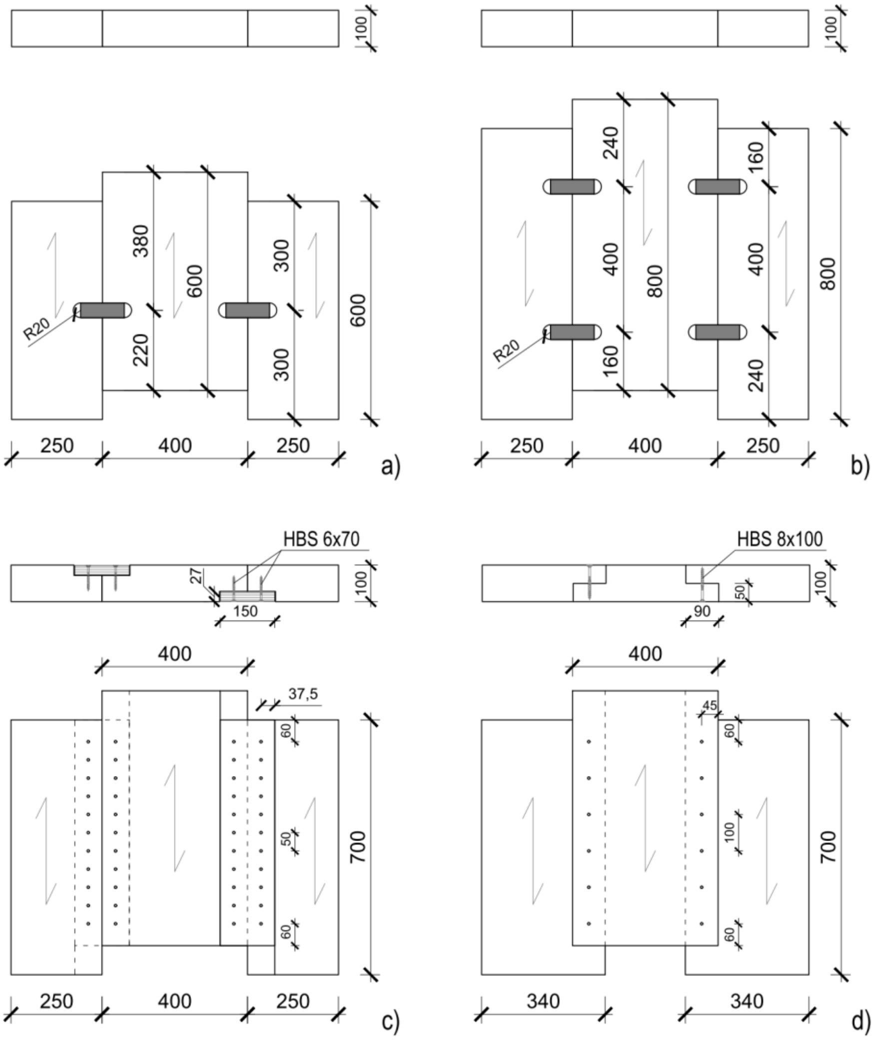


Figure 4: *Specimens for tests on SLOT connectors (a and b), spline joint (c) and half-lap joint tests (d)*

A 3-layer 100 (40-20-40) CLT panel characterized by a mean value of density equal to 455 kg/m^3 was adopted by Schmidt and Blass [15] for the shear tests on the LVL connectors.

A vertical compressive load was applied to the inner panel with a constant load rate equal to 0.1 mm/s up to the failure of the specimen reached typically within 300 seconds according to EN26891 [23]. The two outer panels were laterally restrained by means of two steel braces in order to avoid any rigid rotation of the specimens. Four LVDTs measured the relative slip between the inner and outer panels as shown in Figures 5. A 250 kN load cell was used to measure the load applied on the inner element.

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

Test on joints with SLOT connectors

#	Connector	Total thickness of CLT panels [mm]	CLT panel layout: number and thickness [mm] of layers	Number of connectors per vertical joint
SL 01	SLOT	90	3s (24-42-24)	1
SL 02	SLOT	90	3s (24-42-24)	1
SL 03	SLOT	90	3s (24-42-24)	1
SL 04	SLOT	100	3s (33-34-33)	1
SL 05	SLOT	100	3s (33-34-33)	1
SL 06	SLOT	100	3s (29-42-29)	1
SL 07	SLOT	100	5s (20-20-20-20-20)	1
SL 08	SLOT	100	5s (20-20-20-20-20)	1
SL 09	SLOT	100	5s (20-20-20-20-20)	1
SL 10	SLOT	145	5s (20-20-20-20-45)*	1
SL 11	SLOT	90	3s (24-42-24)	2
SL 12	SLOT	100	3s (29-42-29)	2
SL 13	SLOT	100	5s (20-20-20-20-20)	2
SL 14	SLOT	145	5s (20-20-20-20-45)*	2

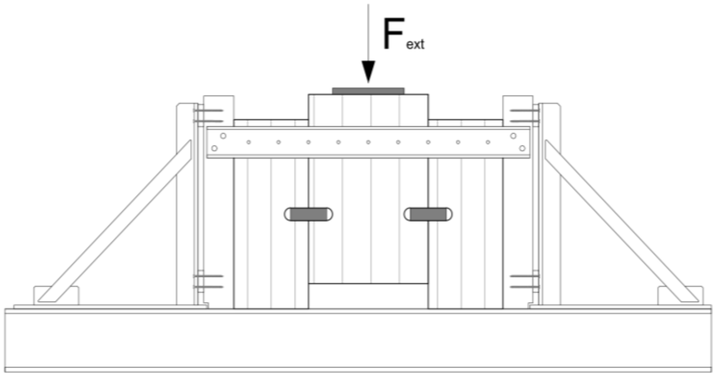
**The Slot connector was aligned to the outer layer with thickness equal to 20 mm.*

Test on joints with screwed connections

Test	Type of screwed joint	Total thickness of CLT panels [mm]	CLT panel layout: number and thickness [mm] of layers	Screw spacing
SP 1	Spline	100	3s (30-40-30)	50 mm
SP 2	Spline	100	3s (30-40-30)	50 mm
SP 3	Spline	100	3s (30-40-30)	100 mm
SP 4	Spline	100	3s (30-40-30)	100 mm
HP 1	Half-lap	100	3s (30-40-30)	100 mm
HP 2	Half-lap	100	3s (30-40-30)	100 mm
HP 3	Half-lap	100	3s (30-40-30)	150 mm
HP 4	Half-lap	100	3s (30-40-30)	150 mm

The results obtained from the tests on the joints with SLOT connectors and screwed connections are calculated in terms of: *i)* maximum shear load acting on each connector or screw, F_{max}; *ii)* slip corresponding to the maximum shear load $v_{F\mathrm{max}}$; *iii)* shear load acting on each connector or screw corresponding to a slip equal to 15 mm, $F_{15\mathrm{mm}}$; *iv)* slip corresponding to the 10% and 40% of the maximum shear load, $v_{0.1F\mathrm{max}}$ and $v_{0.4F\mathrm{max}}$; *v)* elastic stiffness per each connector or screw, K_s, determined as the slope of the line connecting the points on the load-slip curve corresponding to the 10% and 40% of the maximum shear load.

The slip values were determined as average from the four LVDTs while the values of the shear load acting on each connector was calculated as the ratio between the load applied and total number of connectors or screws used in each specimen. For spline joints the values of shear load and stiffness were related to each couple of screws (1+1).



a)


b)

Figure 5: *Test and measurement set-up (a), detail of tested specimen (b)*

2.2. Results and discussion

The results obtained from the tests on the joints with SLOT connectors and screwed connections are reported in Table 2 and 3, respectively.

The failure modes on the joints with SLOT connector were characterized by either the localized crushing in the wood (Figure 6a) or the shear failure in outer lamination of the CLT panels (Figure 6b). In both failure modes, a rigid body rotation with no residual plastic deformations of the connector was observed. The wood crushing failure mode was primarily observed in single-connector tests (SL01 to SL10) where the edge distance of the connector was equal to 380 mm; the outer lamination failure mode was achieved in tests where two connectors for each side were adopted (SL11 to SL14) with an edge distance of connectors equal to 240 mm. Wood crushing was also observed for LVL connectors in tests performed by Schmidt and Blass [15] and in the tests at shear-wall tests conducted in this study (Figure

6c). In the spline and half-lap joint tests, the formation of plastic hinges in the screws as well as the local embedment crushing of the wood around the fasteners were detected (Figure 6).

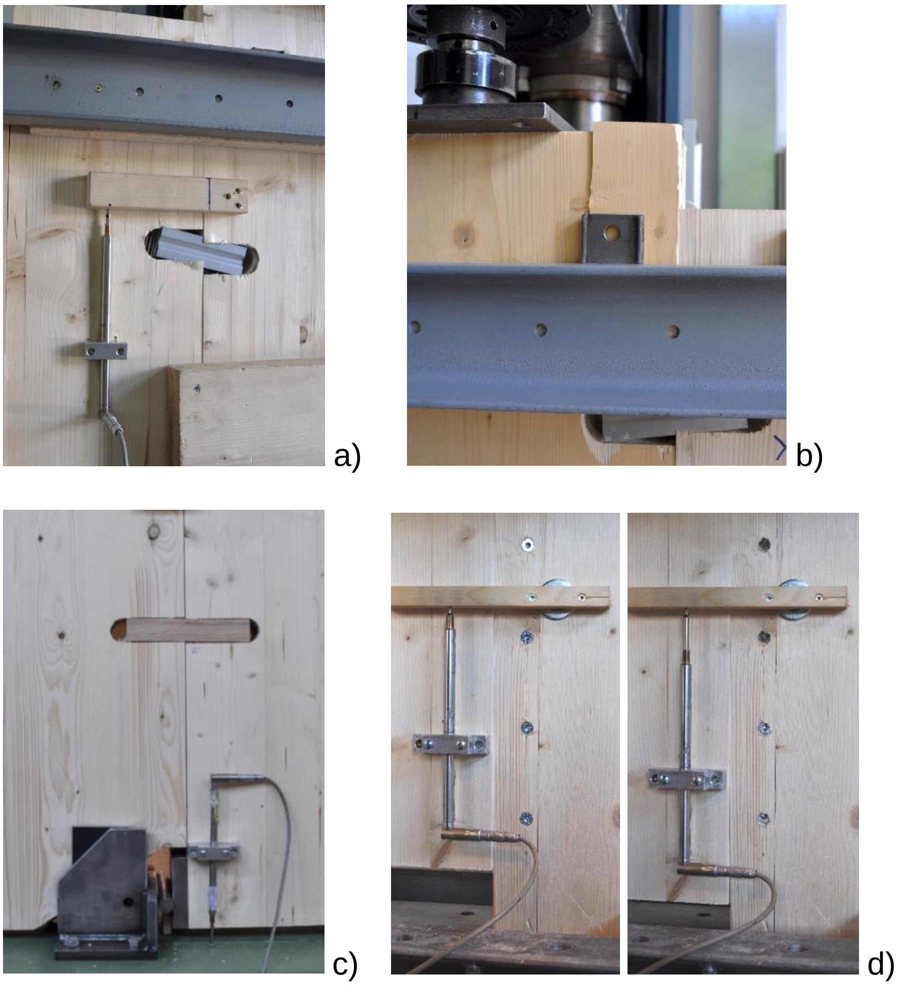


Figure 6: *Failure mode: localized crushing of the panel in test SL 01 (a); shear failure of outer lamination in test SL 05 (b), LVL shear-key connector at the end of test Wall04; screwed joint HP 02 before and at the end of the test (c)*

In Figure 7, the load-vs-slip curves for tests SL 01, SL 05, SL 07, SP 01 and HP 01 are shown (the shapes of the curves of the other tests are almost identical to those of the curves shown in Figure 7 for the same typology of connector). The curves related to SLOT connector tests show an initial slip (0.3 and 0.5 mm) due to the physical gap between the connector and the CLT panels and are characterized by an elastic-almost perfectly plastic behaviour, due to the crushing of the wooden laminations parallel to the load direction. In the screwed joints, no initial slip occurred and a marked rope effect is observed after the yielding of the fasteners.

In order to investigate the influence of the different panel layouts on the mechanical properties of the joints with SLOT connectors, the effective strength capacity f_{max} and stiffness k_s per unit width were calculated as expressed by Equation 1 and reported in Table 4 for the tests SL 01 to SL 010. Tests SL 011 to SL 014 were excluded since interrupted at a value of load corresponding to the maximum load of the testing machine.

$$f_{max} = \frac{F_{max}}{b_{eff}} \qquad k_s = \frac{K_s}{b_{eff}} \tag{1}$$

where $b_{eff,SL}$ is the effective width of the connector calculated according to [21] for CLT panels as:

$$b_{eff} = \sum d_0 + 0.1(b_{SL} - \sum d_0) \tag{2}$$

where $\sum d_0$ is the total layer thickness parallel to the load direction of CLT members within the width b_{SL}.

The mean values of effective strength f_{max} and stiffness k_s per unit width for SLOT connector are equal to 1.20 kN/mm (CoV 11%) and 0.28 kN/mm^2 (CoV 8%). The uniformity of results and the relatively small values of the CoVs for the tested CLT specimens show a negligible influence of the panel layouts on the effective strength and stiffness per unit width.

A mean value of strength and stiffness on the LVL connectors equal to 69.3 KN and 18 kN/mm was reported by Schmidt and Blass [15] for the shear tests on LVL connectors with length equal to 120 mm (i.e. the same length of SLOT connectors). Considering a total layer thickness of the CLT members parallel to load direction equal to 80 mm and a width of the connector of 100 mm, the effective width b_{eff} is equal to 82 mm, getting a mean value of the effective strength and the effective stiffness per unit width equal to 0.84 kN/mm and 0.22 kN/mm^2, respectively.

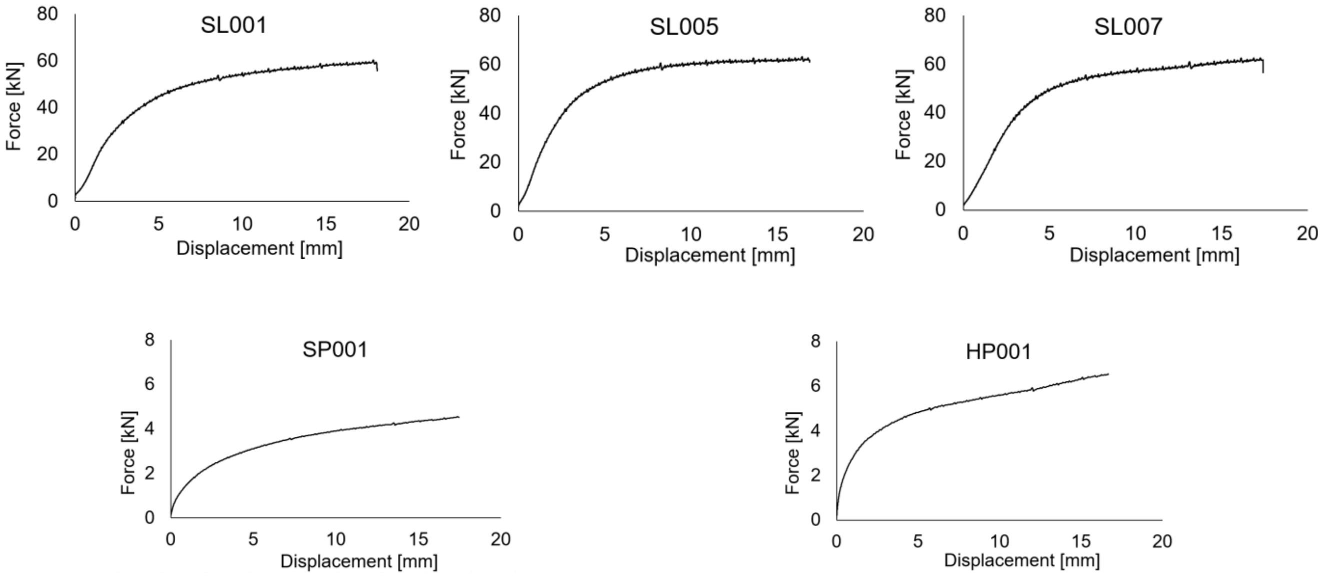


Figure 7: *Load-vs-displacement curves for test on shear-key connectors and screwed vertical joints*

The mechanical performance on the joint with SLOT connectors are compared with those obtained from spline and half-lap joints through the definition of the effective number of screws, $n_{eff,Fmax}$ and $n_{eff,Ks}$, that should be adopted to achieve the same strength and stiffness for a joint with a single SLOT connector. Assuming an effective width $b_{eff,SL}$ of the SLOT connector equal to 53 mm, the average values of strength and stiffness related to a SLOT connector can be calculated as: $F_{max} = f_{max,mean} \cdot b_{eff,SL} = 1.20 \cdot 53 = 63.6\,kN$ and $K_s = k_{s,mean} \cdot b_{eff,SL} = 0.28 \cdot 53 = 14.8\,kN/mm$. The average values of strength and stiffness per each fastener are determine from Table 3 and are equal to 5.03 kN and 1.14 kN/mm for spline joints and 8.13 kN and 1.52 kN/mm for half-lap joints, respectively. The effective numbers of screws $n_{eff,Fmax}$ and $n_{eff,Ks}$ are hence equal to 12.6 and 13.0 for spline joints and equal to 7.8 and 9.7 for half lap joints, respectively.

Table 2: *Test results on SLOT connectors*

#	F_{max} kN	F_{15mm} kN	$0.1F_{max}$ kN	$0.4F_{max}$ mm	V_{Fmax} mm	$v_{0.1Fmax}$ mm	$v_{0.4Fmax}$ mm	K_s kN/mm/connector
SL 01	60.4	58.0	6.04	24.1	17.8	0.41	1.70	14.0
SL 02	62.3	61.8	6.22	24.9	15.4	0.57	2.05	12.7
SL 03	58.7	56.7	5.87	23.5	17.4	0.43	1.62	14.8
SL 04	53.8	52.6	5.37	21.6	16.8	0.50	1.59	14.9
SL 05	63.2	62.5	6.37	25.3	16.4	0.32	1.38	17.9
SL 06	60.9	59.4	6.11	24.3	15.6	0.33	1.51	15.4
SL 07	62.6	60.4	6.27	25.0	17.3	0.44	1.87	13.1
SL 08	74.9	73.4	7.49	29.9	15.6	0.55	1.99	15.6
SL 09	71.1	67.9	7.12	28.4	18.9	0.47	1.85	15.3
SL 10	68.1	66.4	6.82	27.2	19.0	0.43	1.96	13.3
SL 11*	48.1	-	4.80	19.3	13.7	0.45	1.42	14.8
SL 02*	56.6	-	5.66	22.6	9.04	0.48	1.57	15.6
SL 13*	57.2	-	5.72	22.9	5.76	0.46	1.66	14.3
SL 14*	58.1	-	5.84	23.3	6.01	0.33	1.11	22.2

The load and stiffness values are reported per single connector
* Tests on 4-connectors specimens: interrupted at a total applied load lower than 250 kN (maximum load of the testing machine)

Table 3: *Test results on screwed vertical joints*

#	F_{max} [kN	F_{15mm} kN	$0.1F_{max}$ kN	$0.4F_{max}$ mm	V_{Fmax} mm	$v_{0.1Fmax}$ mm	$v_{0.4Fmax}$ mm	K_s kN/mm/screw*
SP 1	5.37	4.37	0.54	2.15	29.9	0.14	1.98	0.88
SP 2	4.75	4.00	0.47	1.90	29.8	0.00	0.90	1.57
SP 3	5.10	4.10	0.51	2.04	30.0	0.08	1.76	0.92
SP 4	4.90	4.87	0.49	1.96	29.9	0.05	1.31	1.17

*The load and stiffness values are reported per couple of screws (1+1)

#	F_{max} kN	F_{15mm} kN	$0.1F_{max}$ kN	$0.4F_{max}$ mm	V_{Fmax} mm	$v_{0.1Fmax}$ mm	$v_{0.4Fmax}$ mm	K_s kN/mm/screw
HP 1	7.93	6.32	0.78	3.13	30.0	0.08	1.30	1.92
HP 2	7.92	5.93	0.79	3.17	29.7	0.06	2.94	0.82
HP 3	8.58	6.61	0.86	3.43	29.8	0.04	1.64	1.62
HP 4	8.07	6.70	0.81	3.23	30.0	0.03	1.45	1.71

The load and stiffness values are reported per single screw (1)

Table 4: *Effective strength and stiffness for tests on joints with SLOT connectors*

#	Total thickness of CLT panels [mm]	$\sum d_0$ [mm]	$b_{eff,SL}$ [mm]	f_{max} [kN/mm]	k_s [kN/mm^2]
SL 01	90 (24-42-24)	47	51	1.18	0.27
SL 02	90 (24-42-24)	47	51	1.22	0.25
SL 03	90 (24-42-24)	47	51	1.15	0.29
SL 04	100 (33-33-33)	55	58	0.92	0.26
SL 05	100 (33-33-33)	55	58	1.08	0.31
SL 06	100 (29-42-29)	47	51	1.19	0.30
SL 07	100 (20-20-20-20-20)	49	53	1.18	0.25
SL 08	100 (20-20-20-20-20)	49	53	1.41	0.29
SL 09	100 (20-20-20-20-20)	49	53	1.34	0.29
SL 10	145 (20-20-20-20-45)	49	53	1.29	0.25
	Mean			**1.20**	**0.28**
	CoV			**11%**	**8%**

3. Shear-wall tests

3.1 Test-layout and set-up

In this section the experimental campaign carried out on eight full-scale shear-walls is presented. Four tests (W01 to W04) were carried out on multi-panel shear-walls with vertical joints assembled with either SLOT or LVL shear-key connectors. Two tests (W05 and W06) were conducted on single-panel shear-walls with the same total length of multi-panel walls with shear-key connector and, lastly, two tests (W07 and W08) were performed on multi-panel shear-walls with traditional spline and half-lap screwed vertical joints, respectively.

The joints in the shear-walls assembled with either SLOT connectors or screwed connections were installed with the same geometrical pattern and spacing of those used for the tests at joint level. The LVL connector was characterized by a length equal to 200 mm. a width equal to 100 mm and a thickness of 40 mm and was inserted in grooves with dimensions equal to 100x100x40 mm. The test-layout for the experimental campaign on shear-walls is reported in Table 5.

Table 5: *Shear-wall test layout*

Test	Number of panels	Type of connector/Vertical joint	Number of connector/screws in vertical joint	Hold-down
Wall 01	3-panel	SLOT connector	2	WHT 340
Wall 02	3-panel	LVL connector	2	WHT 340
Wall 03	3-panel	SLOT connector	4	WHT 620
Wall 04	3-panel	LVL connector	4	WHT 620
Wall 05	Single-panel	-	-	WHT 340
Wall 06	Single-panel	-	-	WHT 620
Wall 07	3-panel	Spline joint - 6x70 mm screws	44 / spacing 50 mm	WHT 340
Wall 08	3-panel	Half lap joint – 8 x100 mm screws	22 / spacing 100 mm	WHT 340

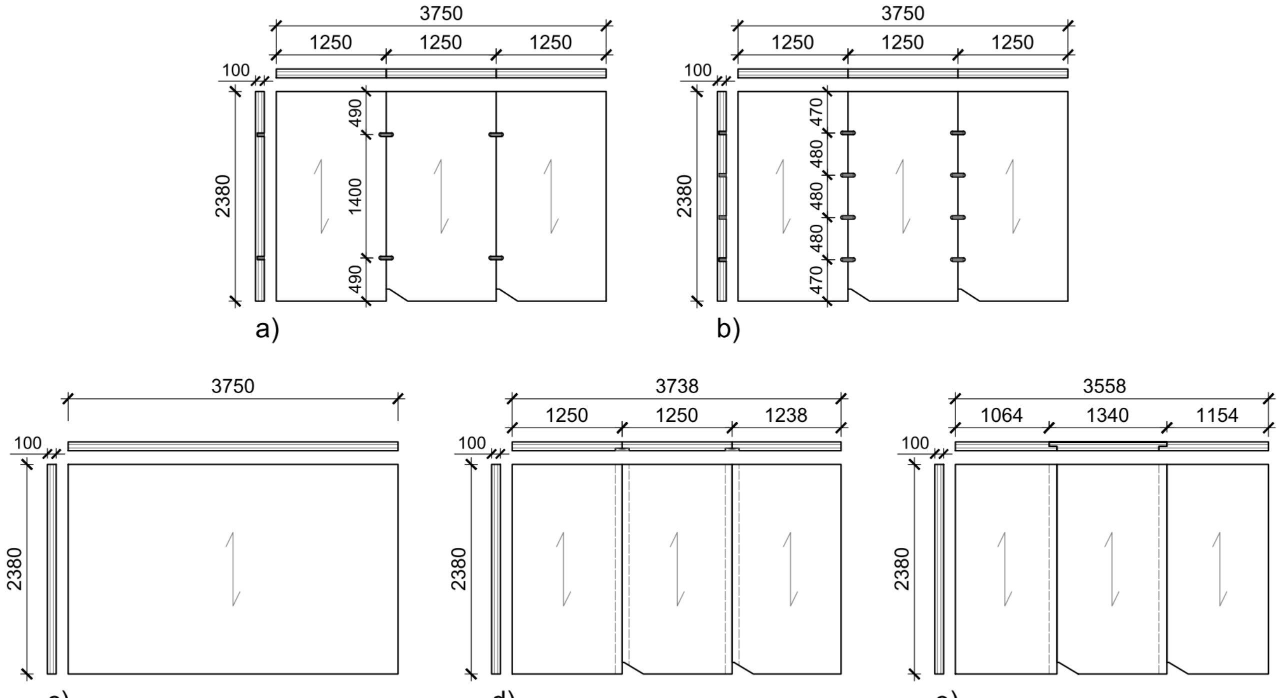
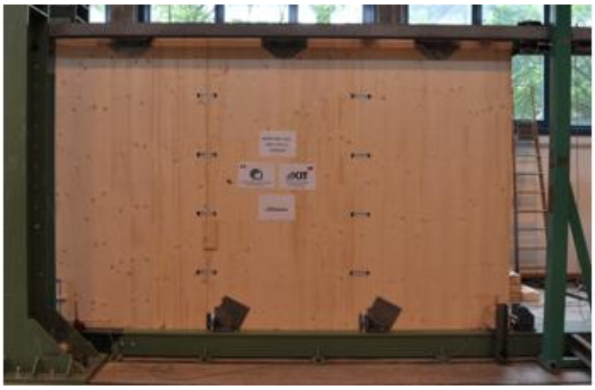


Figure 8: *Shear-wall test specimens: a) Wall 01-02, b) Wall 03-04, c) Wall 05-06, d) Wall 07 and e) Wall 08*

All the shear-walls were characterized by a height h equal to 2380 mm and a total length l equal to 3750 mm and were made with 3-layer 100 mm thick (30 mm - 40 mm - 30 mm) CLT panels [22]. The multi-panel walls were composed of three panels of length b equal to 1250 mm, see Figure 8 and Figure 9. The shear-walls were anchored to the steel foundation beam by means of either one WHT 340 or one WHT 620 hold-down connected to the CLT panel with twenty and fifty-five 4x60 mm annular-ringed shanked nails, respectively. The sliding of the shear-walls was prevented by a shear blocking systems designed not to give any additional vertical and rotation restrain to the CLT panels. For this purpose, a steel cylinder was located between the shear blocking and the steel plate in contact with the CLT narrow face, see Figure 10a. The dimensions of the steel plate were chosen to prevent any local crushing at the bottom edge of the CLT panels. It is noteworthy to mention that the shear blocking system works only in case of a monotonic load tests since not capable to provide any sliding restraint in case of an opposite direction of the applied load. In other words, this system could not be used for cyclic tests. All the shear-walls specimens were designed in order to exhibit a kinematic behaviour characterized by a single centre of rotation, failure in the hold down and almost elastic behaviour of vertical joints.

Each panel was connected to an upper rigid horizontal steel beam by means of steel hinges as shown in Figure 10b. A horizontal steel 24 mm diameter cylinder was inserted in two outer steel plates screwed to each CLT panel. A 15 mm gap between the upper horizontal steel element and the CLT panels was left in order not to limit the uplift and the rotation of each panel. The load set-up was specifically designed to transfer the same lateral horizontal displacement to the top of all three panels according to the assumption of diaphragm behaviour of CLT floors.

Displacement-controlled monotonic tests with a load rate equal to 0.2mm/s were carried out according to EN 594 [24]. No vertical load was applied on the shear-walls. A 500 kN load cell was used to measure the applied lateral load. Nine displacement transducers (LVDTs) were installed to measure the absolute and relative displacements of shear-walls components. see Table 6 and Figure 11. Further tests will be performed in the near future to investigate the cyclic behaviour of such shear-walls as well as study the influence of vertical load.



a) b)

Figure 9: *Tests on multi-panel shear-wall Wall 03 (a) and Wall 05 (b)*


a)

b)

Figure 10: *Shear blocks and steel rod (a); steel plate and steel bar for load transfer to each panel (b).*

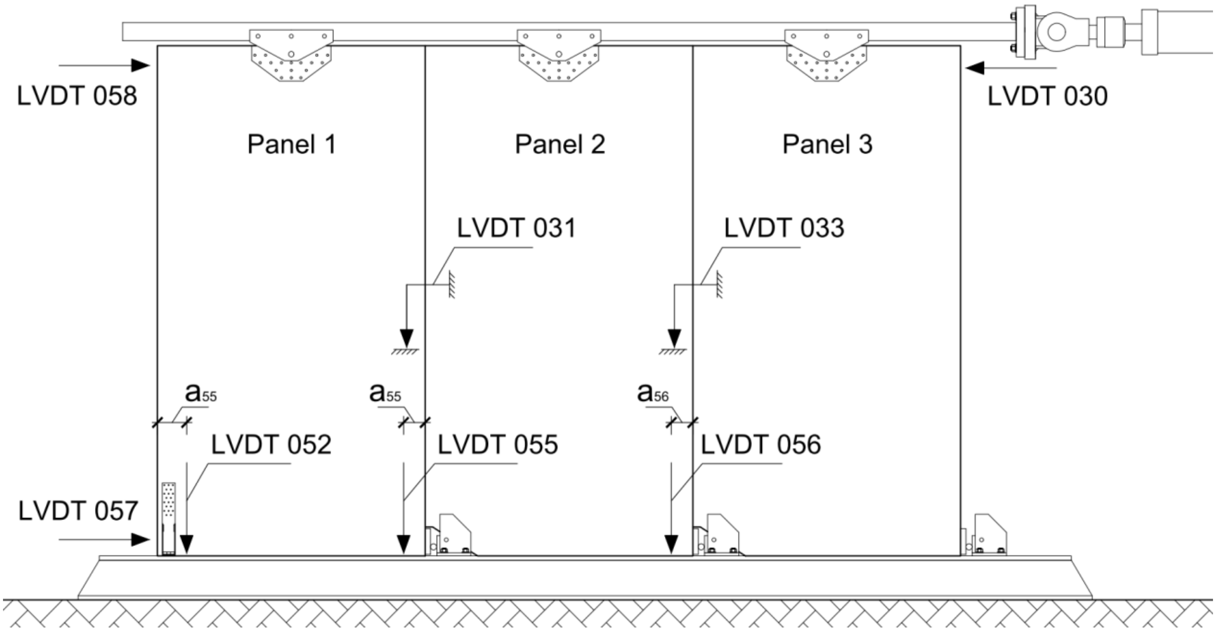


Figure 11: *Test and instrumentation set-up*

Table 6: *Description of LVDTs location*

LVDT	#	Measure
L057	u_{057}	Horizontal displacement at the base
L058	u_{058}	Horizontal displacement at the top
L030	u_{030}	Horizontal displacement at the top
L052	v_{052}	Vertical uplift of panel 1 – slip of hold-down
L055	v_{055}	Vertical uplift of panel 2
L056	v_{056}	Vertical uplift of panel 3
L031	$\delta_{1.2}$	Relative displacement along the vertical joint between panel 1 and 2
L033	$\delta_{2.3}$	Relative displacement along the vertical joint between panel 2 and 3

3.2 Analysis method

The lateral displacement *Δ'* at the top of shear-walls was calculated as the mean value of the lateral displacements measured by the two displacement transducers LVDT 058 and 030 located at two top corners of each wall as expressed by Equation 3.

$$\Delta' = \frac{u_{058}+u_{030}}{2} \quad (3)$$

The sliding of the shear-walls Δ_a was directly measured by the LVDT 057. namely $\Delta_a = u_{057}$. The lateral displacement due to the rocking behaviour and panel deformation was determined as difference between the total lateral displacement and the sliding of shear-walls (Equation 4).

$$\Delta = \Delta' - \Delta_a \quad (4)$$

The rigid body rotation of each panel θ was determined as expressed by Equations 5 according to kinematic mode shown in Figure 12. For the innermost panel (i.e. Panel 3) a contact length c equal to 150 mm was assumed.

$$\theta_1 = \frac{v_{052}-v_{055}}{b'}; \; \theta_2 = \frac{v_{055}-\theta_1 \cdot a_{055}+\delta_{1.2}-v_{056}}{b''}; \; \theta_3 = \frac{v_{056}-\theta_2 \cdot a_{056}+\delta_{2.3}}{b'''} \quad (5)$$

where the panel lever arms b', b'', b''' are calculated as expressed by Equation 6 to 8 and a_{052}, a_{055}, a_{056} are shown in Figure 11.

$$b' = b - a_{052} - a_{055}; \tag{6}$$

$$b'' = b - a_{056}; \tag{7}$$

$$b''' = b - c \tag{8}$$

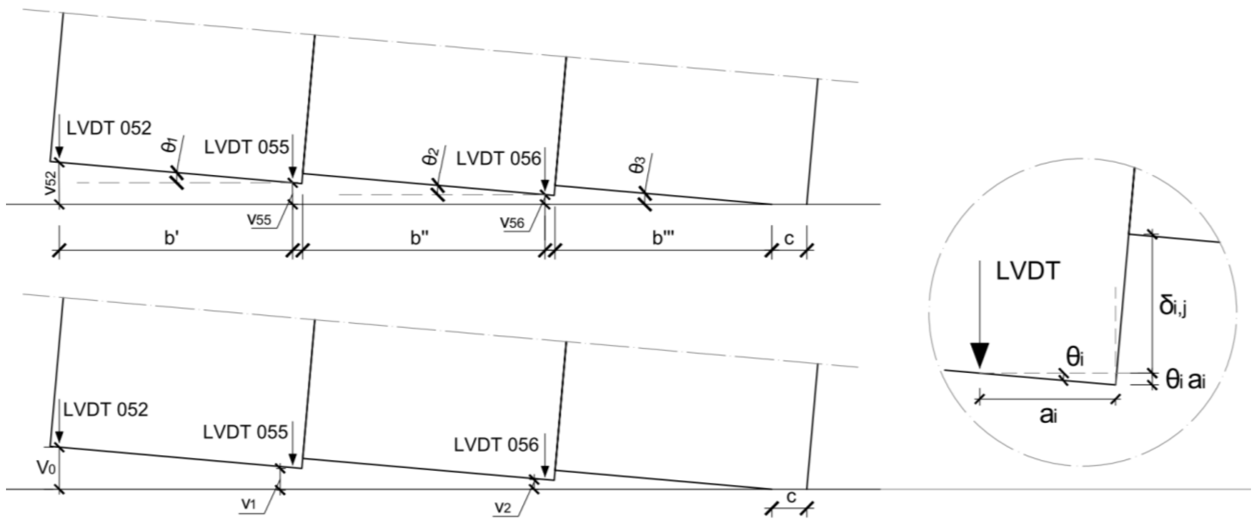


Figure 12: *Geometrical dimensions for the data processing*

The experimental load F-vs-displacement Δ curves of shearwalls were approximated with tri-linear curves that were drawn by connecting the yield *Y*, the maximum *M* and the ultimate *U* points determined as follows. The point *Y* was determined according to the procedure reported in EN12512 [25] as the intersection of two lines. The former line is obtained by connecting the points on the experimental curve corresponding to 10% and 40% of the maximum load. The second line is the tangent to the experimental curve and characterized by a slope equal to 1/6 of the former line. The point *M* corresponds to the point on the experimental curve at the maximum load while the point *U* is located on the decreasing branch of the experimental curve corresponding to a reduction of load equal to 20%. The elastic stiffness K and the ductility μ of the shear-walls were calculated as expressed by Equations 9 and 10:

$$K = \frac{F_Y}{\Delta_Y} \quad (9)$$

$$\mu = \frac{\Delta_U}{\Delta_Y} \quad (10)$$

3.3 Results

A kinematic mode characterized by a single global centre of rotation at the bottom corner of innermost panel (Panel 3) was observed for all the tested shear-walls, see Figures 13a and 13b, as confirmed by the not-null values of the uplifts for all the three panels measured when the maximum lateral load was applied on the shear-walls, see Table 7.

The values of yield load F_Y, maximum load F_M and ultimate load F_U, and the corresponding values of displacement Δ_Y, Δ_M and Δ_U are reported in Table 8.

Table 7: *Uplift of panels corresponding to the maximum lateral load*

Test	Maximum load	Panel 1	Panel 2	Panel 3
LVDT	F_M [kN]	v_{052} [mm]	v_{055} [mm]	v_{056}[mm]
Wall 01	115	19.5	11.1	4.2
Wall 02	101	23.8	14.2	7.0
Wall 03	167	16.0	9.4	4.4
Wall 04	180	16.7	10.6	5.1
Wall 05	111	21.7	14.2	6.8
Wall 06	174	15.1	10.1	4.7
Wall 07	120	20.8	11.4	3.3
Wall 08	94	22.2	14.4	6.4

For all tests the failure mode was related to the hold-down. A ductile failure mode with plastic deformations of nails was observed for WHT340 hold-downs (Figure 13c), with the only exception of the test on Wall 07 where a brittle failure of the WHT340 hold-down steel plate was detected. A brittle failure mode of the steel plate conversely occurred for all WHT620 hold-downs (Figure 13d). An almost elastic behaviour of the vertical joints with no residual plastic deformations in aluminium connectors, LVL beech connectors (see Figure 6c) as well as screwed joint were observed.

The experimental load F-vs-displacement Δ curves and the corresponding approximated tri-linear curves are shown in Figure 14.

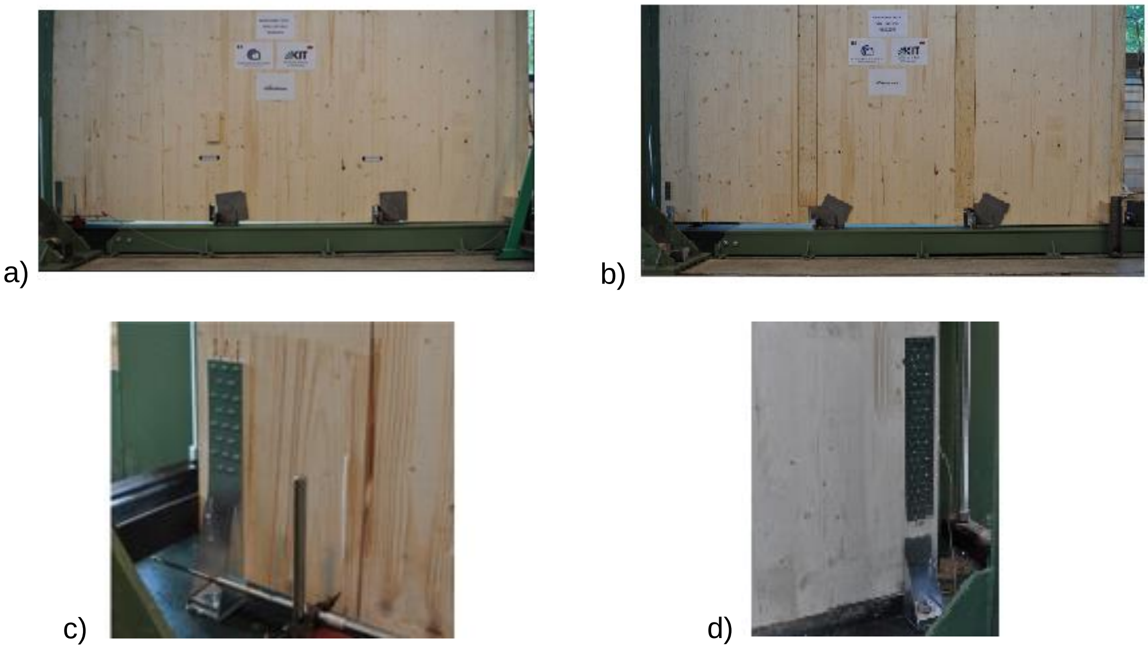


Figure 13: *View of kinematic mode for Wall 01 (a) and for Wall 07 (b); hold-down failure mode in Wall 01 (c) and in Wall 04 (d)*

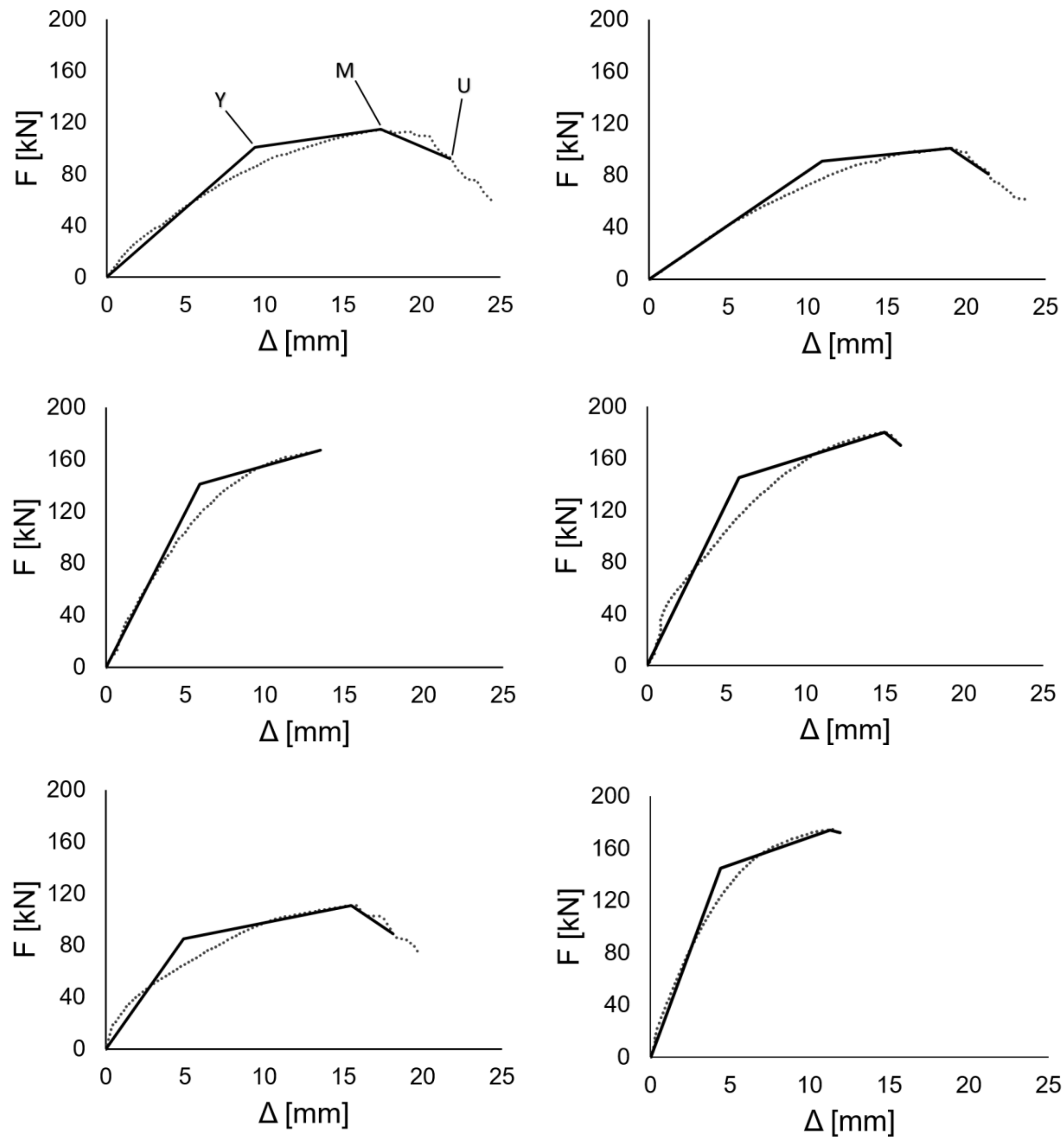
The values of the sliding of shear-walls Δ_a and the panel-to-panel slip in the vertical joints $\delta_{1.2}$ and $\delta_{2.3}$ are reported in Table 9 at the maximum load on the shear-walls. In Figures 15 the wall lateral load is plotted as function of the panel-to-panel slip in vertical joints for Wall 01 and Wall 02.

Table 8: *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 01	101	9.4	115	17.4	92	21.8	10.7	2.3
Wall 02	91	10.9	101	19.0	81	21.4	8.3	2.0
Wall 03	141	5.9	167	13.5	167	13.5	23.8	2.3
Wall 04	145	5.8	180	15.0	170	16.0	25.5	2.8
Wall 05	85	4.9	111	15.5	89	18.1	17.4	3.7
Wall 06	145	4.4	174	11.3	172	11.9	32.6	2.7
Wall 07	86	5.0	120	19.5	116	20.2	17.3	4.1
Wall 08	71	4.7	94	16.1	75	23.8	15.2	5.0

Table 9: *Sliding and panel-to-panel slip in shear-wall tests corresponding to the maximum lateral load*

Test	F_M [kN]	Δ_a (mm)	$\delta_{1.2}$ (mm)	$\delta_{2.3}$ (mm)
Wall 01	115	2.9	2.0	2.0
Wall 02	101	2.4	2.3	1.8
Wall 03	167	4.0	1.1	1.2
Wall 04	180	7.2	1.9	0.7
Wall 05	111	2.8	-	-
Wall 06	174	8.2	-	-
Wall 07	120	2.2	2.4	3.5
Wall 08	94	3.4	0.9	0.1



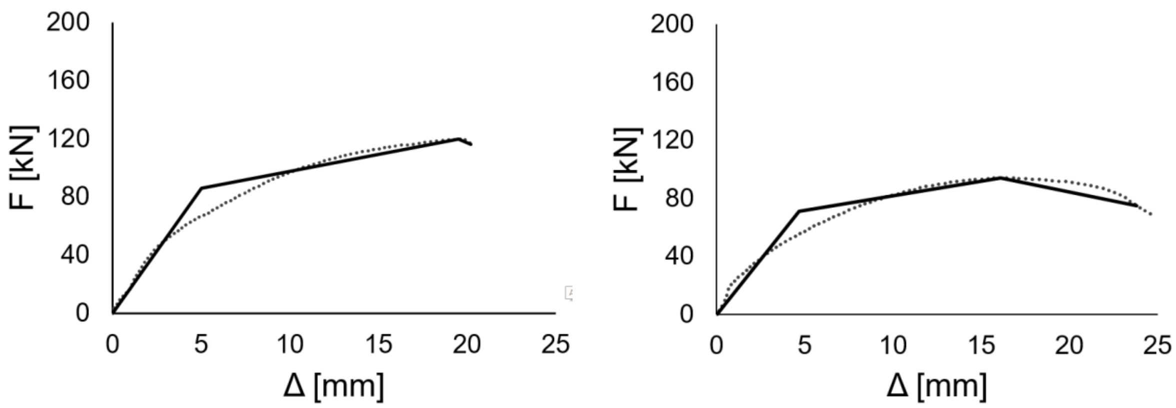


Figure 14: *Load F vs displacement Δ curves from the experimental tests on shear-walls (dot line) and trilinear approximation (continues line)*

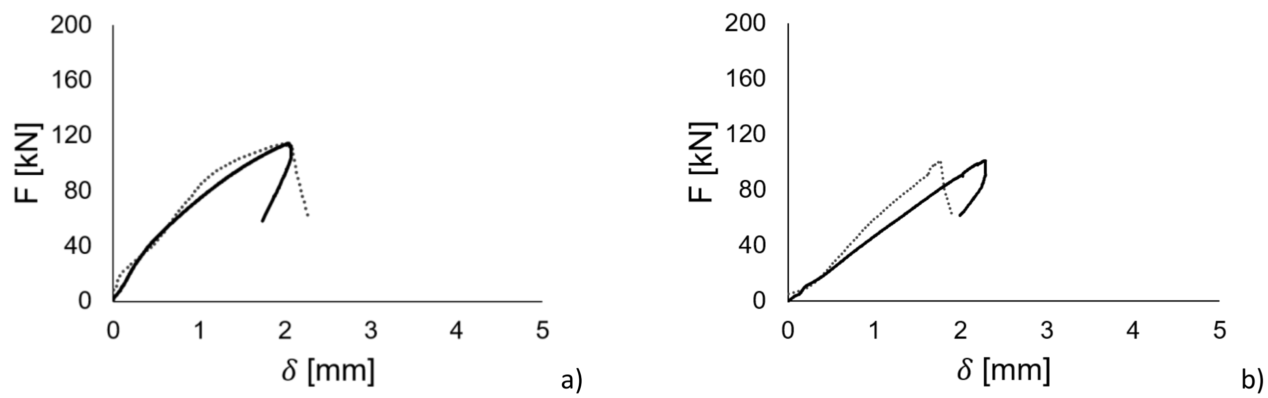


Figure 15: *Lateral load vs panel-to-panel slip curve $\boldsymbol{\delta}_{1.2}$ and $\boldsymbol{\delta}_{2.3}$ for Wall01 (a) and Wall02 (b)*

3.3 Discussion

The main outcomes that can be drawn from the experimental tests on shear-walls are the following.

- The values of maximum load of shear-walls with stiff shear-key connectors (Wall 01 to Wall 04) were approximately equal to those obtained from the single-panel shear-walls (Wall 05 and Wall 06). The failure mode for both shear-wall types was related to the hold-down.

- Multi-panel shear-walls with stiff shear-key connectors (Wall 01 to Wall 04) showed a lateral stiffness smaller than that obtained for single-panel shear-walls (Wall 05 and Wall 06). A reduction of stiffness equal to -67% and -58% was calculated for Walls 01 and Wall 02 (characterized by two stiff shear-key connectors per vertical joint) from the value of stiffness related to Wall 05. respectively. A reduction of stiffness equal to -27% and -22%. was obtained for Walls 03 and Wall 04 (characterized by four stiff shear-key connectors per vertical joint) from the value of stiffness of Wall 06. respectively. A summary of the comparison between multi-panel shear-walls with stiff shear-key connectors and single-panel walls is reported in Table 10.
- The shear-walls with aluminium and LVL shear-key connectors were characterized by similar values of strength and lateral stiffness, highlighting not much difference between the two connectors.
- A single-panel kinematic behaviour with one centre of rotation was shown in all multi-panel shear-walls. The values of the relative displacement along the vertical joints ($\delta_{1.2}$ and $\delta_{2.3}$) were much smaller than the values of uplift of hold-down v_{52}.
- The multi-panel shear-walls with screwed vertical joints (Wall 07 and Wall 08) showed similar strength and higher stiffness than shear-walls with the shear-key connectors (Wall 01 and 02). It is noteworthy to mention that this comparison can not be generalized since either a different number of shear-key connectors or different values of spacing of the screws could have been adopted. Wall 07 and Wall08 presents a high number of fastener (low spacing) compared with typical CLT panel-to-panel joint. Increasing the number of connectors (e.g. Wall03 and Wall04), similar or higher values of the lateral stiffness between shear-walls with shear-key connectors and screwed joints could be obtained.

Table 10: *Comparison in terms of maximum load and lateral stiffness between shear-wall with stiff shear connectors and single-panel shear-walls*

Hold-down	Test	Connector	F_M	F_M/ $F_{M(Wall05)}$	K	K/ $K_{(Wall05)}$
			(kN)	(mm)	(kN/mm)	(-)
WHT340	Wall 01	2 SLOT	115	+4%	10.7	-58%
	Wall 02	2 LVL	101	-9%	8.3	-67%
	Wall 05	Single panel	111	-	17.4	-
WHT620	Wall 03	4 SLOT	167	-4%	23.8	-27%
	Wall 04	4 LVL	180	+3%	25.5	-22%
	Wall 06	Single panel	174	-	32.6	-

4. Conclusions

In this paper the mechanical behaviour of two typologies of stiff shear-key connectors for CLT structures was investigated both at connector and wall level. The monotonic tests carried out on joints with shear-key connector showed values of stiffness and strength, expressed for a single SLOT connector, comparable with continuous vertical joint composed by a number of screws equal to 12.6 and 13.0 for spline joints and equal to 7.8 and 9.7 for half lap joints, respectively. An elasto-perfectly plastic behaviour of the vertical joint with SLOT connectors was observed with an initial slip between 0.3 and 0.5 mm. The failure mode was related in all cases to CLT panels (either local plastic deformation parallel to the grain of vertical layers or brittle failure in outer edge of panel) with no plastic deformation of connectors.

A single-wall behaviour of multi-panel shear-walls with stiff shear-key connectors was observed in all the tests at wall level. The same values of strength were obtained for multi-panel and single-panel shear-walls. A maximum reduction of stiffness equal to -67% and -27% was calculated comparing the experimental results of multi-panel shear-walls (with 2 and 4 stiff shear-key connectors) and single-panel walls, respectively.

The results obtained from this research show that stiff shear-key connectors can be considered as a valuable alternative to traditional screwed connections in multi-panel shear-walls in low seismic prone areas where stiff vertical joints are designed to behave elastically. Further studies will be carried out in order to investigate the mechanical behaviour of multi-panel shear-walls subjected to cyclic load as well as study the influence of vertical load.

The outcomes from this study can provide additional valuable information for the use of shear-key connectors in floor panel-to-panel longitudinal joints as an alternative to, or in combination with, traditional inclined screws. Further research will be conducted in the near future on the use of shear-key connectors in floor panel-to-panel longitudinal joints and on the combined use of shear-key connectors with traditional screws. The use of screws in combination with shear-key connectors might be in fact essential to ensure an adequate out-of-plane stability of the wall panels, especially during the transport and erection phases of the panels.

In a design for disassembly perspective the use of point connections, that are not screwed/nailed but simply inserted in CLT grooves, could lead to an important simplification of disassembly phase both for walls and floor elements giving the possibility to recycle/reuse the structural panels.

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