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EXPERIMENTAL CYCLIC TESTS ON CLT HOLD-DOWN CONNECTIONS SUBJECTED TO A COMBINATION OF SHEAR AND TENSION

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Abstract

Cross-Laminated Timber (CLT) structures dissipate energy during earthquake only in mechanical connections, which are located in few specific zones. The full definition of their structural behaviour and their correct design is then of crucial importance, especially in seismic conditions. Many studies were carried out on this topic during the last decade in Europe, North America and Japan in order to define monotonic and cyclic behaviour of mostly used connections when only one action (i.e. tension or shear) is prescribed. Nevertheless, some questions are still unanswered. In particular, during earthquakes, connections are subjected simultaneously to both shear and tension. The interaction between shear and tension forces may affect connector's capacity in terms of strength, stiffness, ductility and dissipation capacity. Moreover, the possibility of brittle failure or excessive strength degradation of connections subjected to combined tension and shear action must be taken into account.

This work presents the results of an extended experimental programme on CLT hold-down connectors conducted at CIRI Buildings & Construction Laboratory, University of Bologna. Cyclic tests were performed using a specifically developed test setup suitable to apply both tension and shear actions on the connections, simultaneously. In particular, the experimental tests on hold-downs were conducted prescribing a shear deformation and then loading the connection in tension according to the cyclic loading protocol prescribed by standards. The results of these tests, in terms of strength, stiffness, energy dissipation, strength degradation and ductility, are presented and critically discussed. A comparison between the experimental values of load-carrying capacity and stiffness and those obtained with calculations using existing design code provisions are given.

Results obtained in this work allow to define some design guidelines and calculation rules for metal connectors in CLT structure. In addition, they provide the basic information for advanced and reliable investigation on the behaviour of CLT structure when subject to earthquake loadings.

Keywords: Cyclic test, CLT, Connections, Hold-down, Hysteretic behaviour

1. Introduction

Cross-Laminated Timber (CLT) is a wood construction technology utilized in Europe for several decades and became more and more popular recently also in US and many other countries. Although the technology is lasting about 20 years, the seismic performance of these structures was explored only recently by a handful of researchers. CLT structures dissipate energy during earthquake motion only in mechanical connections located in specific zones. For this reason, connections are a vital component of the building system. They play a critical role in achieving the necessary strength and ductility of the structure as a whole. The full definition of their mechanical behaviour and their correct design is then of crucial importance, especially in seismic conditions.

Many studies were carried out recently on this topic in Europe, North America and Japan, in order to define monotonic and cyclic behaviour of mostly used connections, wall assemblies and finally entire prototype buildings. The most comprehensive experimental research on seismic behaviour of CLT systems and connections was carried out by CNR–IVALSA (Italy) under the SOFIE Project [1, 2]. Additional studies on the behaviour of special CLT hold-downs and angle brackets connections, characterized by high values of load bearing capacity, were conducted at University of Trento – Italy [3]. FPInnovations in Canada conducted tests to determine the structural properties and seismic resistance of simple CLT shear walls and 3-D structures [4]. Failure mechanisms in large shear-wall systems were studied in several researches conducted in Japan [5].

In almost all the previous studies conducted in this context, only the overall behaviour of entire CLT wall assemblies or multi-storey buildings was investigated [6]. An experimental campaign on single connection elements was conducted by Gavric et al. [7] in order to fully define the behaviour of typical hold-downs and angle brackets under cyclic loadings, but prescribing one individual action only (traction, or alternatively, shear). Nevertheless, during earthquakes, connections are subjected simultaneously to both shear and tension. The interaction between shear and tension forces may affect connector’s capacity in terms of strength, stiffness, ductility and dissipation capacity. Moreover, connections subjected to combined tension and shear forces can be subject to brittle failures.

Some preliminary studies about the effects of tension-shear interaction on connections on the structural behaviour of CLT systems were performed by Pozza et al. [8] analysing a series of experimental tests on CLT wall panel assemblies fixed with different arrangement of connections and subjected to cyclic load. Results from this study show that, when the fastening systems is not appropriate, the single connection elements are subjected to combined tensile – shear forces. Consequently, the resistance parameters and the seismic performances (i.e., behaviour factor, ductility etc..) can be significantly deteriorated with respect to the model considering no coupling effects in the connection elements.

Despite the simultaneous shear-tension actions on connections in real systems, in the design practice it is actually commonly assumed that angle brackets carry shear forces, while hold-downs carry tractions, without any rules or guidelines to avoid the tensile – shear coupling. Current design codes as well prescribe this design approach, disregarding the effects of the tension-shear interaction in the connection elements.

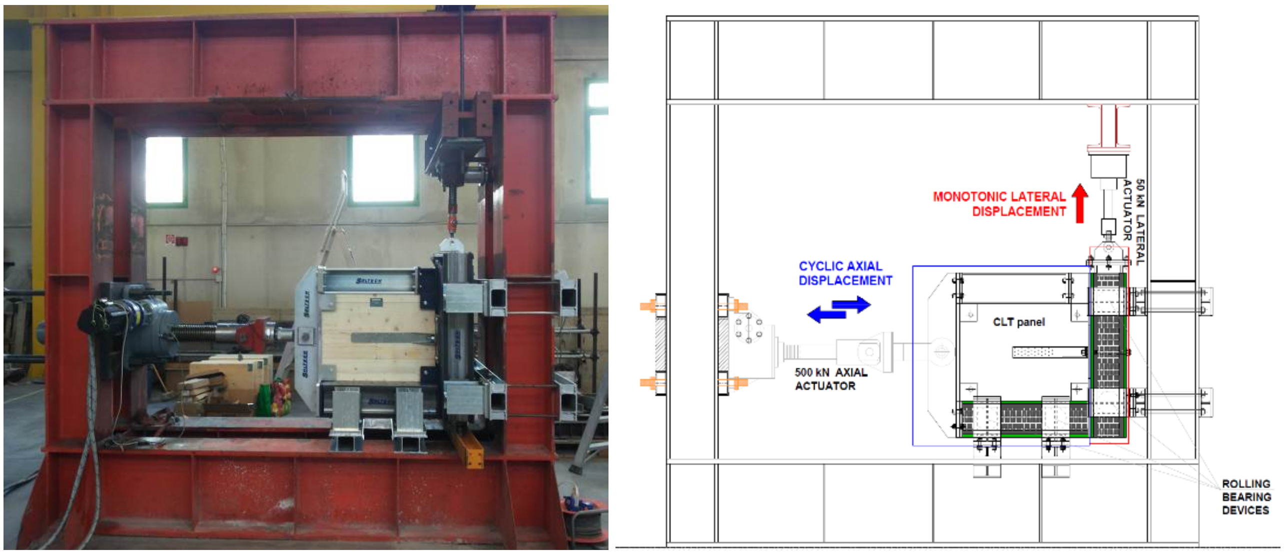
In the present study, in order to have a better understanding of the actual behaviour of hold-down connections, a research program was defined in order to investigate the connection behaviour under both shear and tension actions applied simultaneously, evaluating the influence of hold-down’s axial strength while they are deformed laterally. Obtained results show that the engineering design practice is not coherent with the experimental response and may represent an unsafe calculation hypothesis when hold-downs are subjected to coupled tension-shear actions.

2. Experimental tests

The present research is aimed at investigating the interaction between shear and tension forces on typical hold-down steel devices adopted as connections in CLT structures. In this section, setup, specimen geometry and properties of connections and test procedures are illustrated.

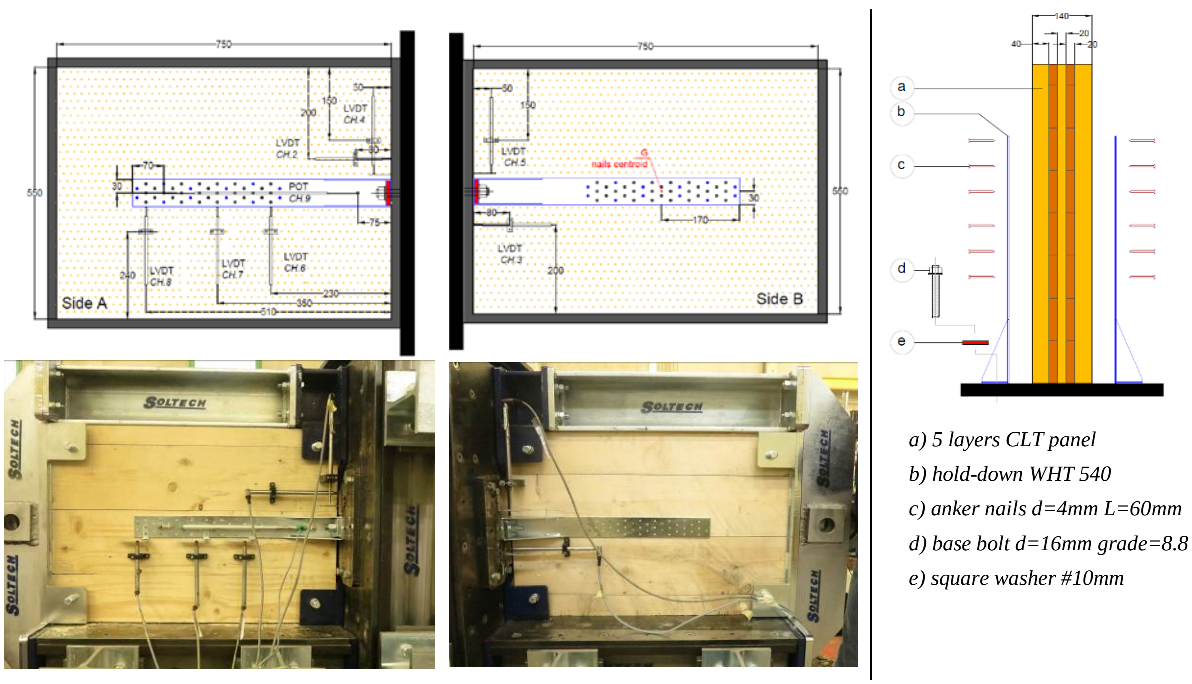
2.1 Experimental setup

Experimental tests on hold-down were performed using a specifically designed test setup suitable to prescribe both lateral and axial deformation to the assembly of CLT panel with connection. The experimental setup and boundary conditions are depicted in Fig.1, together with the loading scheme for cyclic loading.



(a) (b)

Fig. 1 – Test setup (a) and geometric details (b)



(a) (b)

Fig. 2 – (a) Side views of the test setup and (b) geometry of specimen

The test specimen, rotated of 90° with respect to the configuration in an actual in-situ application, was fixed to the supporting rigid steel frame in the direction of application of axial loading. Hence, the axial displacement is due to the panel – hold-down connection deformation only. In order to ensure the required boundary conditions to the specimen during the test, rolling bearing devices were used in both directions. Rolling bearing devices allow specimen displacements without rotations, reducing friction between moving parts. Finally, two hold-downs were used for each specimen, on two sides of CLT panel, so realizing a symmetrical system under testing. In this test setup, the two degrees of freedom (axial and lateral) for the connection were uncoupled.

Different views of one specimen placed into the test device are shown in Fig.2. Axial and lateral displacements of the CLT panel respect to the fixed supports were measured with two electronic transducers (LVDTs) per side. On one side (Side A), three additional vertical LVDTs and one potentiometer were installed along the hold-down length to measure the local lateral deformation of the steel plate of the connection. Two actuators were used to impose the load conditions. Since the main purpose of hold-down connections in actual applications was to withstand axial forces due to overturning moments, an actuator with loading capacity of 500 kN was used to prescribe the cyclic axial displacement (i.e. axial-actuator). Moreover, a 50 kN actuator was used to prescribe the monotonic lateral displacement (i.e. lateral-actuator).

2.2 Specimens characteristics

Cross-Laminated Timber (CLT) panels were used, with 5 orthogonally crossed spruce layers (Fig.2(b)). Thickness of internal and external layers were 20 mm and 40 mm, respectively, for a 140 mm total thickness of the panel. The panels were conditioned at 20°C temperature and 65% relative humidity before performing the tests and are certified according to European Technical Approval (ETA-08/0271-2011) [9]. Dimensions of each panel specimen were 750 × 550 mm. The hold-downs were in the tests are WHT540 type, with 12 annular ringed nails 4x60 mm, and anchored to the base support with 16mm diameter bolts (8.8 grade). The standard dimensions for WHT540 hold-downs followed the European Technical Approval (ETA-11/0086-2011 for hold-downs) [10] prescriptions. Additional details are reported in Fig.2 (b).

2.3 Test procedure

According to the test procedure, a monotonic lateral displacement was prescribed to the connection up to the target value. Subsequently, the cyclic axial displacement was prescribed following the protocol prescribed by EN 12512 (CEN 2006) [11] and maintaining constant the shear displacement up to the end of the test. In order to evaluate the effect of a lateral deformation on the axial behaviour of the hold-down connections, five different configurations have been examined. The tests were then labelled as follows: LD – XY, where the lateral displacement was maintained equal to XY mm during the whole axial cyclic test, with XY = 0, 7.5, 15, 30, 45 mm in the five tests.

MONOTONIC SHEAR LOADING PHASE	
A	displ. rate =50 mm/ms
B	displ. rate =0 mm/ms

CYCLIC AXIAL LOADING PHASE			
	N. cycle	peak displ. [mm]	displ. rate [mm/ms]
1	1x	3	25
2	3x	4.5	50
3	3x	6	100
4	3x	12	150
	3x	24	150
	-	until failure	150

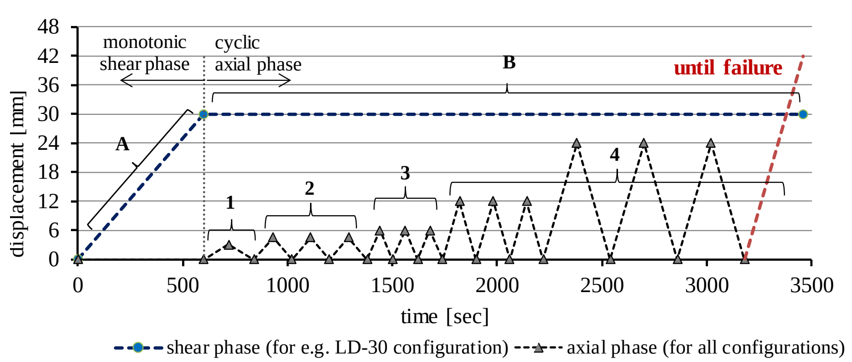


Fig. 3 – Tests procedure for shear and axial loading phases

The standard procedure for axial cyclic testing of joints made with mechanical fasteners prescribed by EN12512 (CEN 2006) was followed in all tests, with input displacement rate varying in the prescribed range (i.e. from 0.02 mm/s to 0.2 mm/s). The suggested procedure for tension tests was modified as reported in Fig. 3 due to the restrained movement in compression direction. The tests were replicated twice for each of the five configuration with different lateral prescribed displacements, for a total of 10 specimens tested.

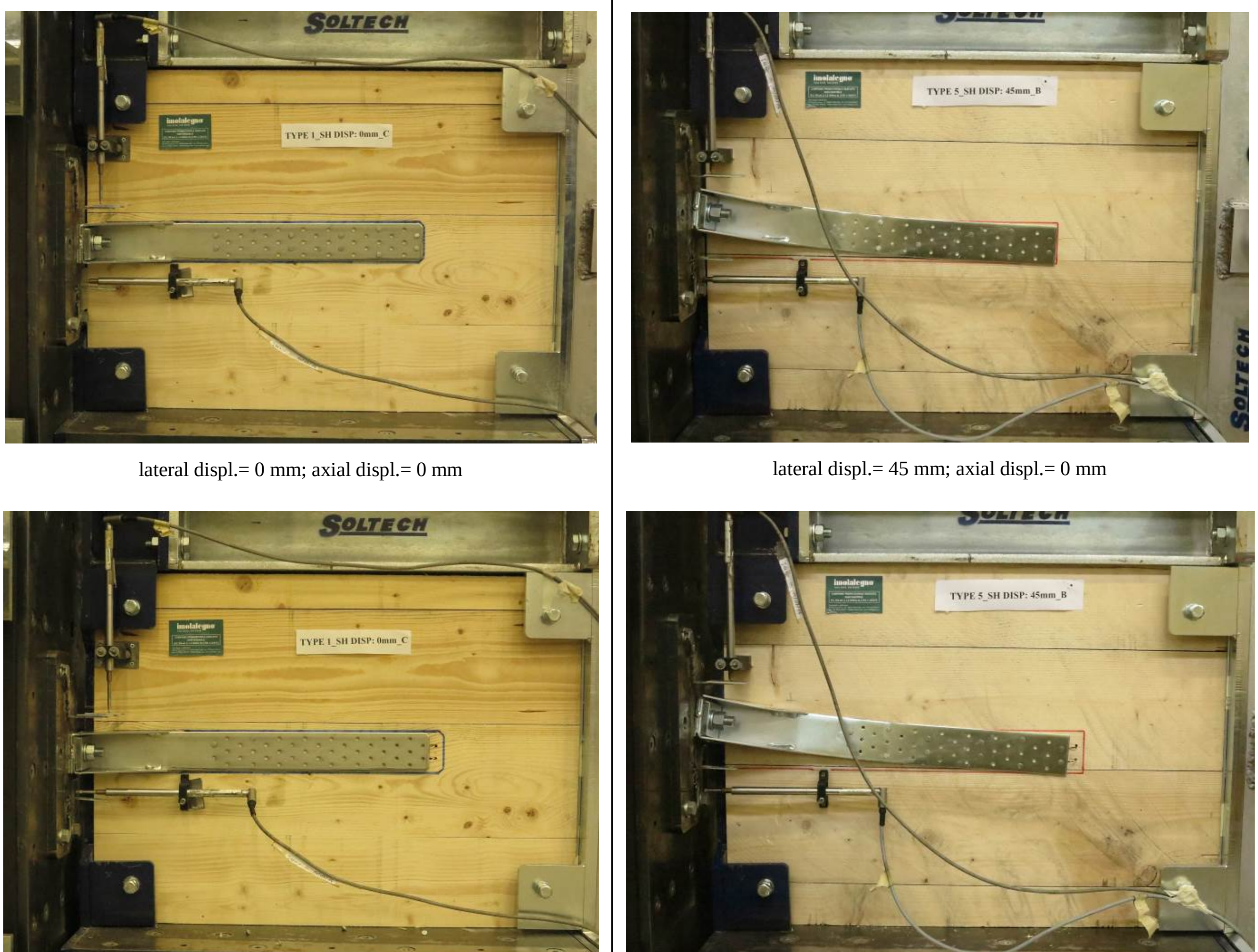
3. Test results

In this section, the test results for the 10 specimens subjected to the 5 different test configurations are shown. Failure modes are investigated, and the most significant load - displacement curves are reported.

3.1 Failure mode

Due to the specific test procedure, the deformed configuration of the connection must be verified, both at the end of the first shear loading phase, when the lateral displacement is monotonically imposed, and at the end of the second phase, when the cyclic axial displacement is prescribed. As an example, Fig. 4 shows the deformed shape of two specimens at the end of the shear loading phase and at the end of the 3rd cycle with 24 mm axial displacement. A solid line on the panels indicates the original position of connections, before the test.

Specimens LD-0 | Specimens LD-45



lateral displ.= 0 mm; axial displ.= 0 mm

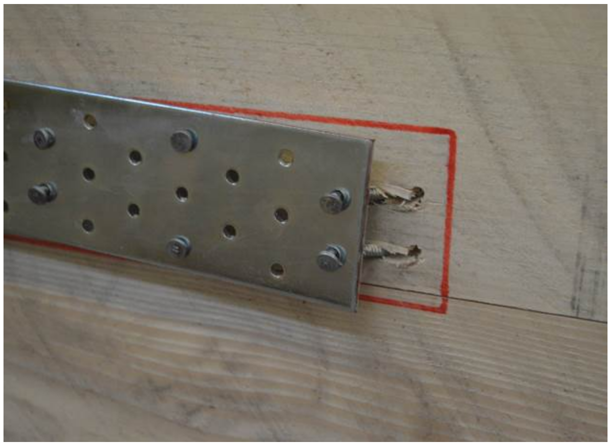
lateral displ.= 45 mm; axial displ.= 0 mm

lateral displ.= 0 mm; axial displ.= 24 mm

lateral displ.= 45 mm; axial displ.= 24 mm

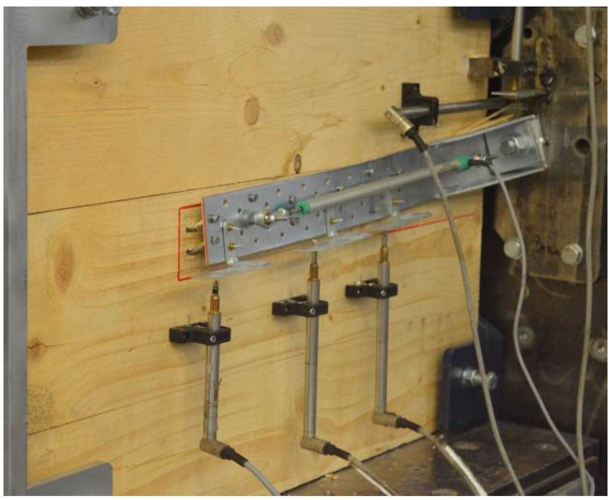
Fig. 4 – Deformed shapes of specimens for different levels of lateral and axial displacements

The failure mode registered at the end of all experimental tests mainly involves the fasteners used to connect the steel plate of the hold-down to the wood panel, with plastic deformation of nails and localized crushing of wood. Two plastic hinges can be recognized in the nails, one under the cap and another one in the shank (10-15 mm below). In addition, localized crushing of wood around the nail shank oriented in the load direction, can be observed (Fig. 5-a,b). Brittle failure involving the fracture of the steel plate never occurred during tests.



(a)

(b)



(c)

(d)

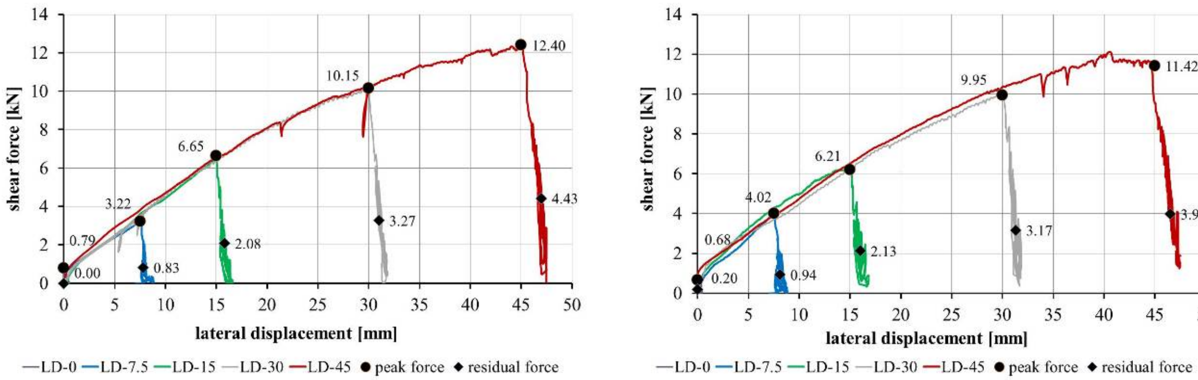
Fig. 5 – Localized failure of specimens: (a) localized crushing of wood, (b) deformed shape of nails with double plastic hinge, (c) failure of connections at the end of cyclic axial loading phase, (d) rigid rotation of the based portion of hold-down

For specimens subject to large imposed lateral displacement (i.e. LD-45), the deformation of hold-down at the end of the shear phase involved a rigid rotation of the based ribbed portion of the steel connection. This rigid rotation is caused by the eccentricity between the bolts axis and the steel plate of the hold-down, inducing a localized out-of-plane deformation of the steel plate (in the portion without nails) and a localized crushing of the wood due to the contact between the panel and ribbed plate (Fig. 5-c,d).

3.2 Load-displacement curves

During the tests, lateral and axial relative displacements between the CLT panel and the external support were recorded. Such data allow to define, for each test, both the monotonic shear force vs lateral displacement curve (Fig.6-a,b) and the cyclic tensile force vs axial displacement curve (Fig. 8).

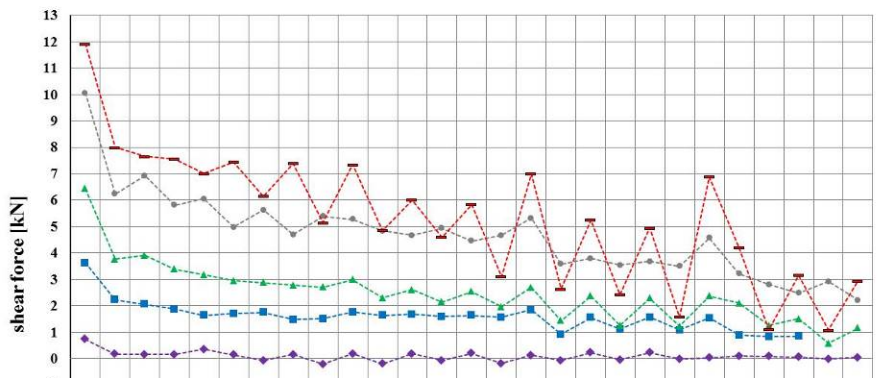
Curves in Fig.6 show the same trend up to the prescribed value of lateral displacement, different for each test. Then, the lateral displacement is maintained (almost) constant during the cyclic axial loading phase and a gradual loss of shear force is registered for all the specimens at the end of the cyclic axial loading phase. At the end of the axial loading phase, the shear force is approximately equal to zero for all the tests.



(a) (b)

Fig. 6 – Shear force vs lateral displacement curves for series a (a) and series b (b)

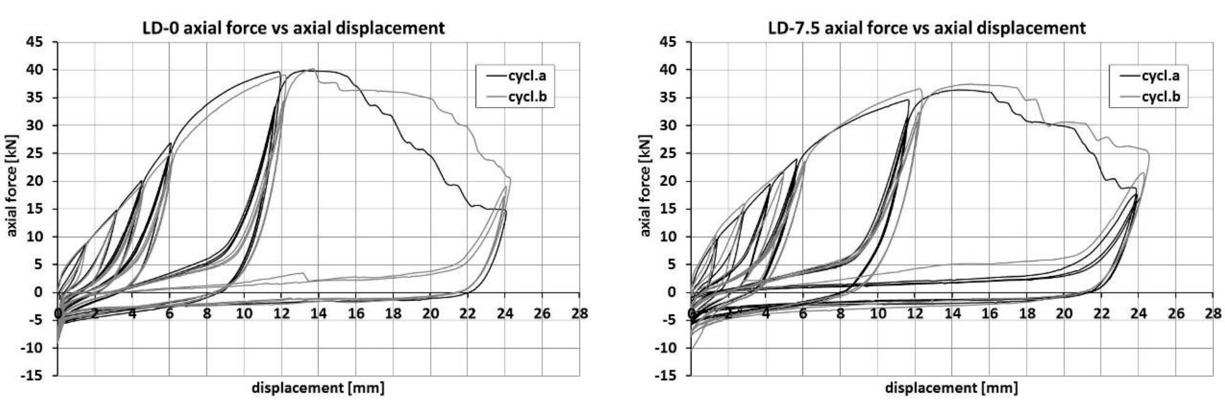
The evolution of the shear force values during the axial phase of the cyclic tests is plotted in the graph of Fig.7 in terms of mean values obtained from the two specimens investigated using the cyclic loading protocol.



	Fsh	3.0	0	4.5	0	4.5	0	4.5	0	6.0	0	6.0	0	6.0	0	12.0	0	12.0	0	12.0	0	Fax	24.0	0	24.0	0	24.0
			1st		2nd		3rd		1st		2nd		3rd		1st		2nd		3rd		1st			2nd		3rd	
			cycle ampl. 4.5mm						cycle ampl. 6.0mm						cycle ampl. 12.0mm						cycle ampl. 24.0mm						
LD-0	0.7	0.2	0.2	0.2	0.4	0.1	-0.1	0.2	-0.2	0.2	-0.2	0.2	-0.1	0.2	-0.2	0.1	-0.1	0.2	0.0	0.2	0.0	0.0	0.1	0.1	0.1	0.0	0.0
LD-7.5	3.6	2.2	2.1	1.9	1.6	1.7	1.7	1.5	1.5	1.8	1.6	1.7	1.6	1.6	1.6	1.8	0.9	1.6	1.1	1.6	1.1	1.5	0.9	0.8	0.9		
LD-15	6.4	3.8	3.9	3.4	3.2	2.9	2.9	2.8	2.7	3.0	2.3	2.6	2.1	2.5	2.0	2.7	1.4	2.4	1.3	2.3	1.2	2.4	2.1	1.3	1.5	0.6	1.2
LD-30	10.1	6.2	6.9	5.8	6.1	5.0	5.6	4.7	5.4	5.3	4.8	4.7	4.9	4.5	4.7	5.3	3.6	3.8	3.5	3.7	3.5	4.6	3.2	2.8	2.5	2.9	2.2
LD-45	11.9	8.0	7.7	7.6	7.0	7.4	6.2	7.4	5.2	7.3	4.9	6.0	4.6	5.8	3.1	7.0	2.6	5.3	2.4	5.0	1.6	6.9	4.2	1.1	3.2	1.1	2.9

Fig. 7 – Shear force evolution during the cyclic axial phase of the test

In Fig. 8, the curves obtained from the tests with different prescribed lateral displacement values (0, 7.5, 15, 30, 45 mm) for two set (“cycl. a” in black line and “cycl. b” in grey line) of investigated specimens are reported. Moreover the curves of series “cycl. b” are superposed for the 5 different configuration LD-XY.



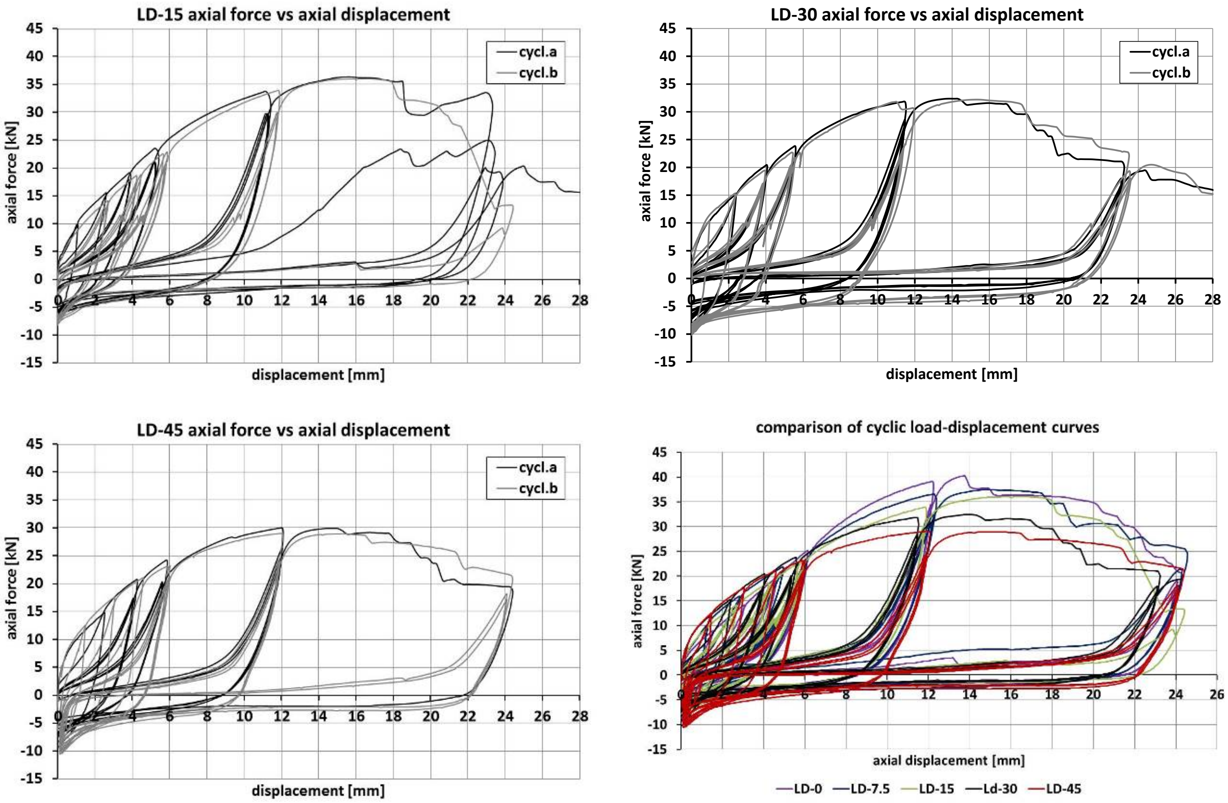


Fig. 8 – Tensile force vs axial displacement curve, for specimens with different prescribed lateral displacement values (0, 7.5, 15, 30, 45 mm) and comparison for the "cycl. b" series

4. Analyses of results

In this section, the test results are analysed according to the procedure prescribed by EN 12512 (CEN 2006). The average values of force and stiffness for each specimens are reported together with the strength degradation and equivalent viscous damping registered at every cycles.

4.1 Force and stiffness evaluation

Method "b" by EN 12512 (CEN 2006) was used to define elastic and post elastic stiffness, yielding point and ultimate conditions of the tested specimens. Table 1 reports the average values of the two series of the parameters necessary to characterize the behaviour of specimens LD-0, 7.5, 15, 30, 45 obtained referring to the envelopes of the cyclic curves.

Table 1 – Analysis of experimental results according to EN 12512 (CEN 2006)

		LD-0	LD-7.5	LD-15	LD-30	LD-45
Elastic stiffness	α [kN/mm]	3.98	4.92	4.93	6.17	6.99
Post elastic stiffness	β [kN/mm]	0.66	0.82	0.82	1.03	1.17
Yielding force	F_y [kN]	34.62	27.11	24.81	21.92	22.96
Yielding displacement	v_y [mm]	8.13	5.04	4.75	3.17	3.03
Ultimate force*	F_u [kN]*	32.03	29.53	28.93	25.83	24.28
Displacement at F_u	v_{Fu} [mm]	19.40	19.66	20.43	20.10	19.78
Maximum force	F_{max} [kN]	40.04	36.91	36.16	32.28	30.35
Displacement at F_{max}	v_{Fmax} [mm]	13.15	14.40	15.25	14.90	11.87
Ductility ratio	μ	2.40	3.90	4.35	6.34	6.84

F_u equal to 0.8 F_{max} according to EN 12512 (CEN 2006)

Fig. 9 shows the tri-linear curve of the specimens obtained following the procedure suggested by EN 12512 – b (CEN 2006). Such tri-linear curves represent the analytical schematization of the elastic branch up to yielding point, of post-elastic hardening branch up to the maximum force and, finally, of the softening branch up to the conventional failure of the connection (here imposed equal to $0.8F_{max}$, as suggested by EN 12512 provisions).

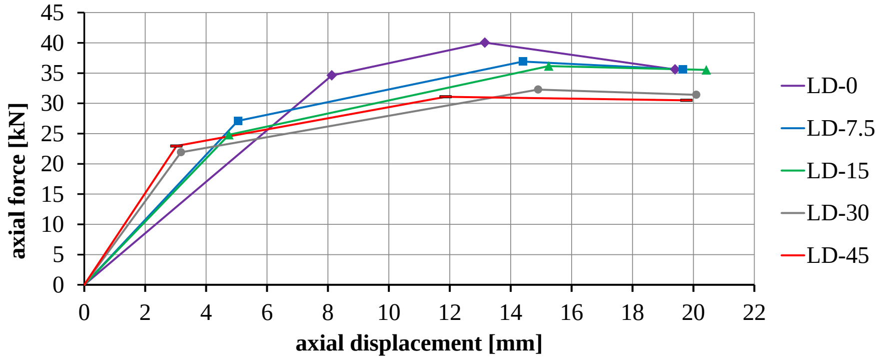
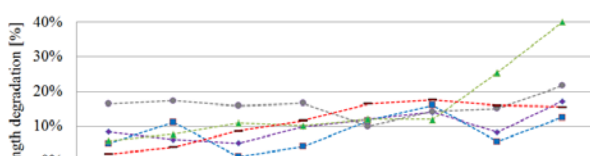


Fig. 9 – Tensile force vs axial displacement of hold-downs: analytical tri-linear curves for the specimens with different prescribed lateral displacement values (0, 7.5, 15, 30, 45 mm)

Results show that the effects of the lateral deformation initially prescribed on the axial response of the specimens can be relevant in terms of reduction of the maximum force achieved during the tests, up to 20% from LD-0 to LD-45. Similarly, a correlation is found between the imposed lateral displacement and the calculated values of yielding points (for both force and displacement values). Consequently, the elastic and post elastic stiffness are affected by the imposed initial lateral deformation. The trends of the yielding force, peak force and of the elastic stiffness for the different level of imposed lateral displacement are reported in Fig. 11. Differently, no correlation is registered between the imposed lateral displacement and the conventional ultimate displacement in traction. Even if these trends are clearly seen in results, it is important to underline that the yielding condition and, therefore, the elastic and post elastic stiffness values are conventionally defined referring to the EN 12512 – b method. Consequently, depending on the method adopted, the yielding point may vary significantly especially in timber structures and connections [12].

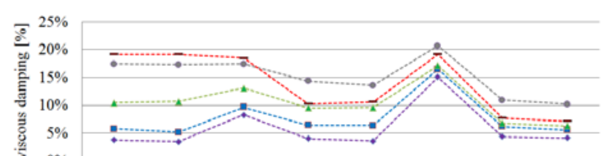
4.2 Equivalent viscous damping and strength degradation

According to EN 12512 (CEN 2006), the cyclic behaviour of the specimens can be fully defined by the equivalent viscous damping values (calculated at each cycle) and by the force degradation values (registered at the end of 2nd and 3rd cycles of each axial displacement level). Figs. 10-a,b report the average values of these parameters for the specimens LD-0, 7.5, 15, 30 and 45 subject to different levels of axial displacement.



	2nd	3rd	2nd	3rd	2nd	3rd	2nd	3rd
	cycle ampl. 4.5mm		cycle ampl. 6 mm		cycle ampl. 12mm		cycle ampl. 24mm	
LD-0	8.3%	6.2%	5.1%	9.9%	11.9%	14.0%	8.2%	17.1%
LD-7.5	5.0%	11.1%	1.2%	4.1%	11.5%	16.0%	5.6%	12.5%
LD-15	5.9%	7.7%	11.0%	10.2%	12.0%	11.9%	25.4%	39.9%
LD-30	16.4%	17.3%	15.8%	16.6%	10.0%	14.2%	15.1%	21.6%
LD-45	2.0%	3.9%	8.6%	11.6%	16.4%	17.6%	16.0%	15.6%

a)



	2nd	3rd	2nd	3rd	2nd	3rd	2nd	3rd
	cycle ampl. 4.5mm		cycle ampl. 6 mm		cycle ampl. 12mm		cycle ampl. 24mm	
LD-0	3.8%	3.5%	8.3%	4.0%	3.6%	15.1%	4.4%	4.1%
LD-7.5	5.8%	5.2%	9.7%	6.4%	6.4%	16.5%	6.2%	5.6%
LD-15	10.5%	10.7%	13.1%	9.5%	9.6%	17.1%	6.8%	6.3%
LD-30	17.4%	17.3%	17.4%	14.3%	13.6%	20.7%	10.9%	10.2%
LD-45	19.2%	19.2%	18.5%	10.3%	10.6%	19.2%	7.7%	7.1%

b)

Fig. 10 – (a) Equivalent viscous damping and (b) strength degradation for the various cycles of axial displacement, whose amplitude is also indicated (in mm)

As far as the equivalent viscous damping ratio is concerned, the results show a growing trend with the level of prescribed lateral deformation, with the exception of LD-45 (Fig.10-a). The same trend is registered for strength degradation, where the reduction in the cycle peak forces increases with the initial lateral imposed deformation, except for LD-30 specimens.

4.3 Analytical evaluation of strength and stiffness

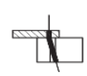
The connection strength of hold-downs can be calculated according to Johansen's yield theory [13]. This theory is universally accepted, sometimes with minor modifications, as the basis for estimating the lateral capacity of slender fasteners like nails. Here, the Eurocode 5 (CEN, 2009) [14] formulas are used to define the connection's characteristic capacity ($F_{v,Rk}$) and stiffness (K_{ser}) for steel to timber connections fastened with nails without pre-hole. The parameters provided by the manufacturers' technical certifications of hold-downs and CLT panels are used, as listed in Table 2.

Table 2 – Adopted parameters for computing connection's capacity and stiffness

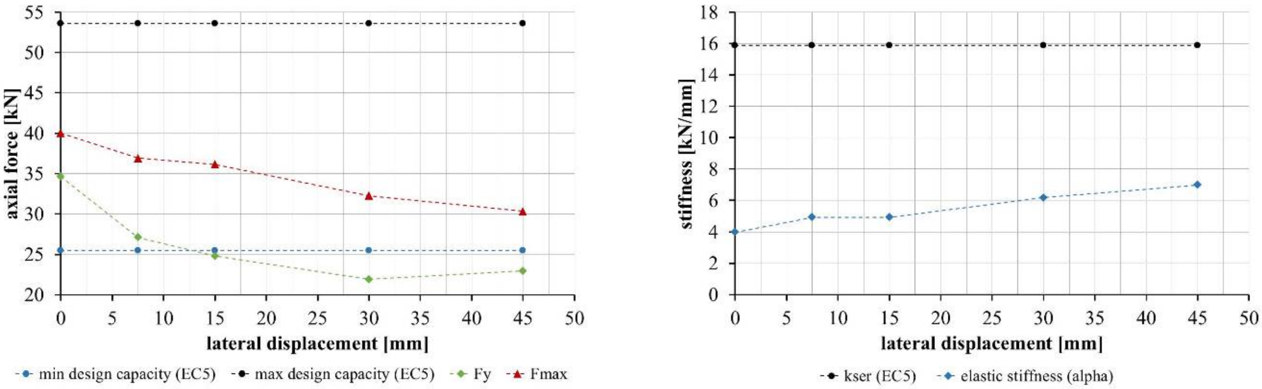
PARAMETER		VALUE
nail diameter	d	4 mm
penetration depth of the fastener into timber	t_1	55.6 mm
characteristic value of fastener yield moment	$M_{y,Rk}$	6.55 Nm
fastener withdrawal capacity	$F_{ax,Rk}$	1.32 kN
characteristic value of panel density	ρ_k	350 kg/m^3
characteristic embedment strength	$f_{h,k}$	18.9 N/mm2
mean value of panel density	ρ_m	420 kg/m^3

The following design coefficients have been also used to define the design capacity of connection ($F_{v,Rd}$): k_{mod} = 1.10, γ_M = 1.00, matching values in Eurocode 5 (CEN, 2009). The predicted capacity design values ($F_{v,Rd}$) are calculated for the three failure modes provided by Johansen's yield theory and obtained multiplying the design capacity per nail by the number of adopted fasteners, as provided by Eurocode 5 (CEN, 2009) and listed in Table 3. In this case, due to the large spacing between nails, no reduction effects are considered. Similarly, the hold-down stiffness is obtained multiplying the nail slip modulus by the number of fasteners and reported in Table 3. The axial resistance and stiffness of the entire hold-down is calculated under the hypothesis that the load carrying capacity of the nails was weaker than the steel plate one, according to the capacity design principles in timber structures. Moreover, the calculations follow the engineering design practice disregarding the effect of the presence of a lateral force on the hold-down resistance and stiffness [15].

Table 3 – Axial capacity (for the different failure modes - c, d and e - provided by Johansen theory (1949)) and stiffness of examined hold-down connection

WHT 540 holdown connection fastened with 12 4x60 nails		 Failure mode c	 Failure mode d	 Failure mode e	min.	Max.
Charact. capacity	$F_{v,Rk}$ [kN]	47.72	25.92	23.16	23.16	48.72
Design capacity	$F_{v,Rd}$ [kN]	53.59	28.51	25.48	25.48	53.59
Stiffness	k_{ser} [kN/mm]				15.88 (mean value)	

Graph in Fig.11-a reports the minimum and maximum values of the hold-down design load bearing capacity ($F_{v,Rd\_min/max}$) overlapped to the experimental yielding (F_y) and maximum (F_{max}) forces for the different level of lateral displacement LD-XY while Fig. 11-b compares the design elastic stiffness (k_{ser}) with the experimental (α) results.



(a) (b)

Fig. 11 – Comparison between experimental mean values of tensile capacity and design values from EC5

Comparing the experimental values with those calculated according to Eurocode 5 (CEN, 2009), the following conclusions can be drawn: (1) the minimum value of the design force ($F_{v,Rd-min}$) underestimates the yielding condition (F_y, in Table 1) of the specimens subjected to small values of lateral displacement before testing (i.e. LD-0 and LD-7.5) but overestimates the yielding condition (F_y, in Table 1) of the specimens subjected to large values of lateral displacement (Fig.11-a); (2) the minimum value of the design force ($F_{v,Rd-min}$) underestimates the peak condition (F_{max}, in Table 1) for all the examined cases; (3) the maximum analytical design force ($F_{v,Rd-Max}$) overestimates the maximum force (F_{max} in Table 1) reached during the tests for all the examined cases; (4) the analytical elastic stiffness (k_{ser}) is much greater than the experimental one (α in Table 1) for all the examined cases.

5. Conclusions and future developments

The axial behaviour of typical hold-down connection when subjected to different levels of lateral displacement before cyclic tests under traction force was the subject of the present study. From the experimental point of view, the results show good repeatability, confirming the quality and adequacy of the designed test setup. Processing of results in accordance with EN 12512 (CEN 2006) shows significant correlation between axial behaviour of hold-down and the imposed lateral deformations, especially in terms of: maximum resistance, yielding point, ductility and equivalent viscous damping. Consequently, the design practice should be updated, taking into account this interaction between the axial and lateral behaviour of hold-down connections. Comparison between experimental results and design load bearing capacity shows that the code approach is adequate to estimate the connection strength only for negligible connection lateral displacements, otherwise code provisions result unsafe. This means that the engineering design practice is not coherent with the experimental response and may represent an unsafe calculation hypothesis when hold-downs are subjected to coupled tension-shear actions.

Results obtained in this work can be used to define some guidelines and calculation rules for a safety design of metal connectors in CLT structure. In addition, they provide the basic information for advanced and reliable investigation of CLT structure by means of specifically calibrated analytical models suitable for define the actual holdown axial resistance at every level of lateral displacement. Finally, more studies and tests are on-going by authors to fully define the interaction between tension and shear in typical hold-down connections, with the objective of defining an interaction strength domain.

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