Mechanical characterization of an innovative wall-to-floor connection for Cross-Laminated Timber structures

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ABSTRACT
Typical connectors for Cross Laminated Timber walls, hold-downs and angle brackets, can exhibit brittle failures if they are not designed according capacity design principles. Moreover, these connections can show reductions of the mechanical performances if they are subject to biaxial loads (shear and tension).
This article presents an experimental study on an innovative wall-to-floor connection for Cross Laminated Timber structures, designed to withstand both shear and tension loads. Experimental tests aimed to investigate the coupling effect of shear and tension loads are presented and discussed. Shear and tension tests, carried out in both monotonic and cyclic regime, are presented giving information on the uncoupled behavior of the connector. With the aim to investigate the coupled behaviour for biaxial loads, monotonic and cyclic tests with 45° load inclination are presented and discussed. Failure modes and mechanical parameters including strength, stiffness, ductility, equivalent viscous damping, energy dissipation and strength degradation are described and discussed in detail. Finally, strength domains of the connection based on yielding, maximum and ultimate forces are proposed.

1 INTRODUCTION

In the last decades timber structures are increasing their impact in the scenery of the constructions, for their qualities as sustainability and development of increasingly performing products. A great contribution to the field was given with the development of Cross Laminated Timber (CLT) panels, which allowed the realization of ever more performing wooden structures. In the context of seismic engineering, these structures showed up an good behaviour due to the lightness and the relatively high strength of the material combined with the capacity of their connections to dissipate energy.

CLT structures are built mechanically jointing the panels, using metal fasteners. These connections, if designed according to "capacity design" principles, exhibit a good behaviour in case of seismic actions. This is mainly due to plasticization of the metal fasteners into the timber. In this context, an important role is performed by the connections at the base of the walls: hold downs and angle brackets. Generally, the firsts are positioned at the ends of the wall to prevent rocking mechanisms, while the seconds are uniformly distributed at the base of the wall to prevent sliding mechanisms. Although in more common design method hold downs are designed for tension forces and angle brackets for shear ones (Wallner-Novak et al. 2013), due to the horizontal load coming from the floor these connectors are subjected to both tension and shear forces (Reynolds et al. 2017).

Mechanical behaviour of hold down and angle bracket subjected to tension and shear loads was studied from different authors, using different approaches. Gavric et al. 2014 experimentally studied the mechanical behaviour of hold-downs subjected to shear forces and angle bracket subjected to tension forces. They found that hold-downs are not able to withstand high loads in shear direction, whereas angle brackets exhibited mechanical characteristics in the tension direction similar to those in the shear one. However, angle bracket loaded in tension showed failures typically not considered ductile, such as withdrawal of the nails. Similar results were found by Casagrande et al. (2016) in an experimental study aimed to investigate mechanical behaviour of several connection types for CLT structures and Light Frame Timber (LFT) structures. Tension tests on a specific angle bracket leaded to the conclusion that such metal connector could be used as an excellent alternative to hold downs. A similar mechanical behaviour in tension and shear direction was found for the angle brackets even by . Flatscher et al (2014). This result was found by means of experimental tests on single connectors and on CLT walls.

Other researchers more deeply studied mechanical behaviour hold down and angle bracket loaded with both tension and shear forces. Liu and Lam (2018 and 2019) studied the mechanical behaviour of both hold-downs and angle brackets simultaneously subjected to shear and tension loads, by means of experimental tests. They found that shear behaviour and tension behaviour for both angle brackets and hold downs are strongly coupled. Their results showed that simultaneous loads decrease the mechanical performance of the connectors, especially when relatively high co-existing loads are applied. Pozza et al. (2018a and 2018b) carried out experimental tests on hold-downs and angle brackets subject to both shear and tension forces. They found that, applying axial displacements, shear capacity of angle bracket decreases. Similar results were found for hold-downs loaded in tension, when a shear displacement was applied. Moreover, all aforementioned researches showed that biaxial loads worsen the mechanical parameters related to cyclic behaviour, as strength degradation and energy dissipation.

In this article an experimental campaign aimed to investigate the coupled behaviour of an innovative angle bracket for tension and shear forces is presented. An experimental and numerical study on the mechanical behaviour of same connector for tension and shear loads can be found in D’Arenzo et al. (2018). Here, a new test methodology to evaluate mechanical behaviour of biaxially loaded metal connectors is presented.

2 BIAXIAL MECHANICAL BEHAVIOUR

2.1 Wall-to-floor connector

Hold downs and angle brackets can exhibit brittle failure mechanisms if a balanced design is not carried out in the different parts of the connector. As discussed by Gavric et al. (2014) the metal connectors for CLT structures actually available on the market can be improved to achieve better mechanical performances. The proposal done by those authors consisted of: use of screws with larger diameters in the bottom part of angle brackets and re-design of the metal connector geometries.

The innovation of the angle bracket presented in this article consists of the capacity to withstand both tension and shear loads, with relatively high mechanical performance. The achievement of this mechanical behaviour is obtained with the insertion of fully threaded screws in the connection with the floor panels. These screws, in fact, are able to withstand high axial loads due to their high withdrawal strength. The angle bracket presents also improvements in the geometry to prevent brittle failures. The main one is a higher thickness compared to typical angle brackets, in order to avert brittle failures as pull-through of the head of the screws or net failure of metal cross section.

2.2 Biaxial tests: background

Two different methods can be used to test metal connectors in biaxial loading conditions. The first where the tension and shear loads are simultaneously variated, following defined laws in shear and tension direction. The second where the connector is tested in the primary direction (tension for the hold downs and shear for the angle brackets) while fixed load values are applied in the perpendicular direction. Pozza et al. (2018a) define the first the “strength domain” protocol and the second the “two phases” protocol.

Certainly, both methods have advantages and disadvantages. The “strength domain” method (Figure 1a), for instance, allows to find the points of the connection’s strength domain choosing a fixed proportion between tension and shear forces. Moreover, if the displacement laws in the horizontal and vertical direction are known, the same load history which the connector at the base of the wall is subjected can be applied on the single connector. However, to know the horizontal and

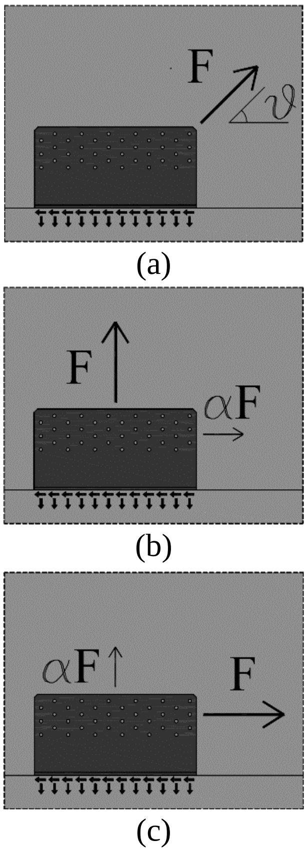


Figure 1. "Strength domain" method (a) and "two phases" method (b, c)

vertical displacements can require much efforts, as for instance several wall tests, resulting relatively complex. The "two phases" method (Figure 1a and Figure 1b) is adequate to find out the mechanical behaviour of the connector in the primary direction when fixed values of load or displacement are applied in the perpendicular direction. This procedure is more suitable for the calibration of numerical or analytical procedure but, at the same time, it is not representative of the connector's load history anchored at the base of the wall.

Several researchers studied the biaxial behaviour of hold downs and angle brackets with the "two phases" method. Liu and Lam (2018) and Pozza et al. (2018b), for instance, used this method to test the coupled behaviour of these connections. On the other hand, none experimental test was found in the literature where the "strength domain" method was used to explore the coupled behaviour of these metal connectors. A numerical study on the coupled behaviour of hold downs and angle brackets where the "strength domain" method was used can be found in Izzi et al. (2018b).

In this paper, the coupled behaviour of the connector is experimentally studied using the "strength domain" method. An inclined load with an angle θ=45° was used to generate equal tension and shear forces and investigate the behaviour for biaxial loads.

2.3 *Analytical considerations*

To ensure a ductile behaviour and prevent brittle failures a balanced design should be performed in the different parts of the connector. A ductile behaviour can be achieved in connections of timber structures through a shear failure mechanism. Secondly, the achievement of one or two plastic hinges in the metal fasteners produces a failure more or less ductile. On the other hand, connections with fastener axially loaded do not exhibit ductile failures if compared to shear loaded connections. Izzi et al. (2018a) suggest to consider as dissipative zones the shear loaded connections whereas axially loaded connections shall be considered as non-dissipative. These principles suggest that the connection of the angle bracket to the bottom panels shall be designed with overstrength respect to the connection with the wall panel. In this way no ductile failures as the withdrawal of the screws or,

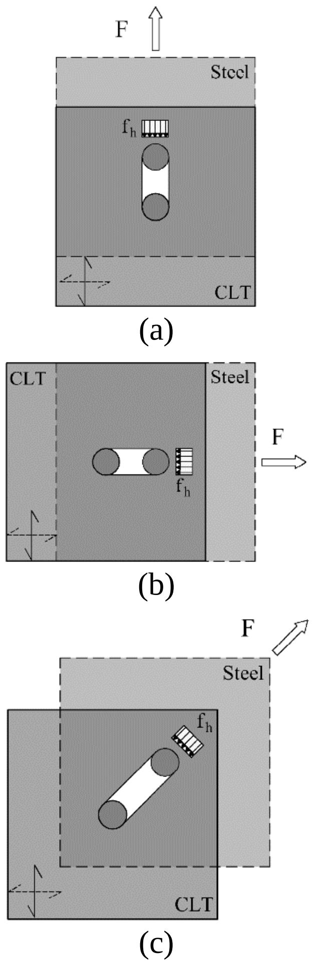


Figure 2. Embedment strength in tension (a), shear (b) and 45° directions (c)

in the worst case, withdrawal of the nails can be avoided. The design of the connection can be carried out using Eq.1 where $F_{B,k}$ denotes the characteristic strength of the bottom connection, $F_{U,k}$ denotes the characteristic strength of the upper connection and γ_{Rd} is the overstrength factor (Follesa et al. 2018).

$$F_{B,k} \geq F_{U,k} \times \gamma_{Rd} \tag{1}$$

Eq.1 can be used to design in both tension and shear load conditions. In case of tension load configuration, $F_{B,k}$ is a withdrawal strength whereas $F_{U,k}$ denote a shear strength. In case of shear load condition, both $F_{B,k}$ and $F_{U,k}$ denote a shear strength.

Designing in overstrength the bottom connection, the failure of the connector can be predicted considering solely the connection to the wall panel. The shear capacity of this part can be evaluated through the European Yield Model, which is based on Johansen theory. It permits to evaluate the capacity of timber-to-timber and steel-to-timber connections loaded in shear. In case of steel-to-timber joints, the capacity of the connection depends on following parameters:

- f_h: embedment strength in the timber member
- t: penetration length of the fastener into the timber
- d: fastener diameter
- M_y: fastener yield moment

The shear capacity of the connection F_v can be expressed in the form reported in Eq.2.

$$F_v = F_v\,(f_h, t, d, M_y) \tag{2}$$

Since the angle bracket is designed to withstand tension, shear and biaxial loads, the evaluation of the shear capacity should be considered in these different configurations. Varying the angle of the applied force to the connector (θ in Figure 1 a), between 0° and 90° it is possible evaluate the capacity of the connection in the different configurations. However, in this case study, the mechanical parameters reported in Eq. 2 are not sensitive to the angle of the force. For fasteners with diameters less than 6mm, which are used in this kind of connectors, Eurocode 5 (2004) considers the embedment strength depending on fastener diameter and wood density, neglecting the influence of the angle between the load and the grain direction (Figure 2).

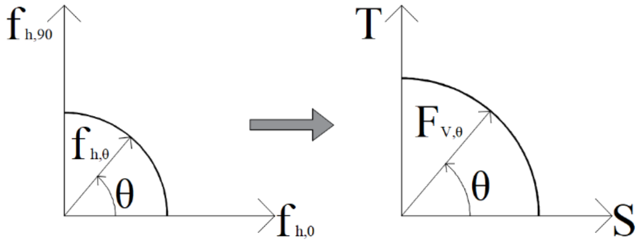


Figure 3. Domain of embedment strength and angle bracket capacity

This entails that a circular domain can be expected for the embedment strength and consequently for the angle bracket's strength (Figure 3).

3 EXPERIMENTAL TESTS

3.1 Experimental setups

In this section, the experimental setups used to characterize the coupled behaviour of the innovative angle bracket are presented.

The coupled behaviour was studied with the "strength domain" method, loading simultaneously the connection with same shear and tension forces. In this case the proportion between the tension and shear forces was fixed applying, through the use of only one actuator, a 45° inclined load. Tension and shear tests were also performed to define the connector's behaviour in uncoupled conditions. The tests were performed in monotonic and cyclic regime.

All tests were carried out using 5 layered CLT panels 150 mm thick. The metal connectors were fastened with 24 annular ringed nails 4×60mm to both wall and floor panels. Five fully threaded screws 11×150mm were used for tension and 45° tests whereas two screws of same dimensions were used for the shear tests. In this way an over-strengthening with factor 1.2 satisfies Eq. 1. Overall more than 12 tests were performed whose at least one monotonic and three cyclic for each configuration.

Tension tests were performed using a symmetric setup, as showed in Figure 4. Two angle brackets were used to connect the CLT panels representing wall and floor. Shear tests were performed using symmetric setup as showed in Figure 5. Two lateral CLT panels, representing the walls, were connected to a central panel, using

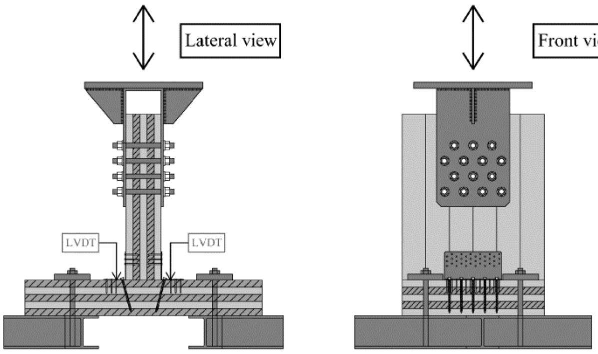


Figure 4. Tension tests setup

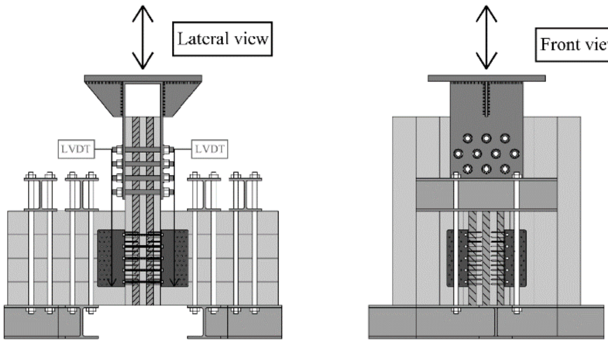


Figure 5. Shear tests setup

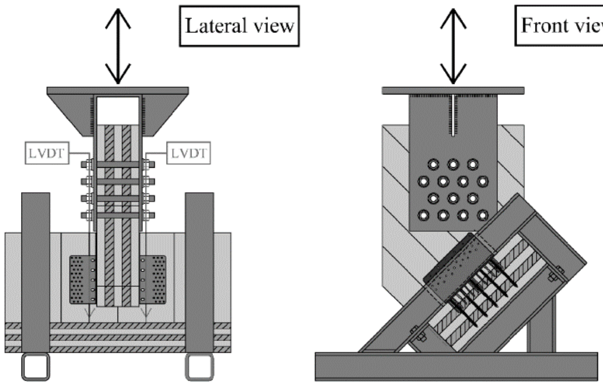


Figure 6. 45° tests setup

four angle brackets. 45° tests were performed using a symmetric setup. A central CLT panel, representing the wall, was connected to a 45° inclined panel, representing the floor, as illustrated in Figure 6. The inclined CLT floor panel was fully restrained to the ground, through a properly designed steel structure. From the figure can be seen that load was applied to the specimen through a vertical actuator. The geometry of the vertical panel and the 45° inclination of the floor panel permitted to load the connector with a 45° force.

3.2 Experimental load protocol

Monotonic test were performed following prescriptions indicated in EN26891 (1991). This norm recommends to perform monotonic test in force control, since an inversion of the load has to be performed between 40% and 10% of the maximum force. In this case, the monotonic tests were conducted in displacement control and the inversion of the load was performed knowing the displacement values in correspondence of 40% and 10% of the maximum force, from previous tests. The load protocol used for the monotonic tests is showed in Figure 7. For these tests the displacement was applied with a rate of 0.05 mm/sec.

Cyclic tests were performed according to EN12512 (2001). Cycle amplitudes were chosen depending on the yielding displacement values obtained from the monotonic tests. They were applied to the specimens with a rate of 0.5 mm/sec. Two different load protocol were used for the cyclic tests. The shear tests were performed with a

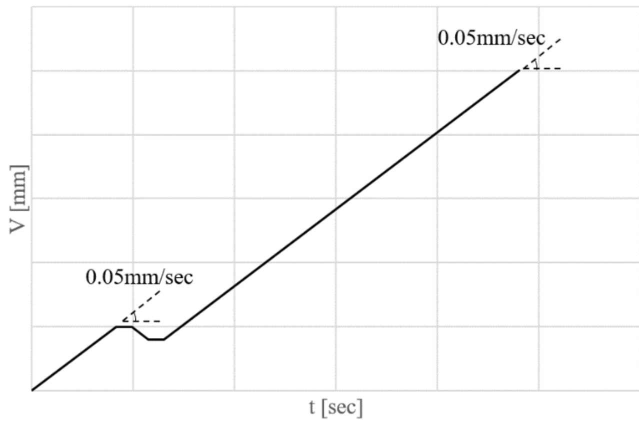


Figure 7. Protocol followed for tension, shear and 45° monotonic tests

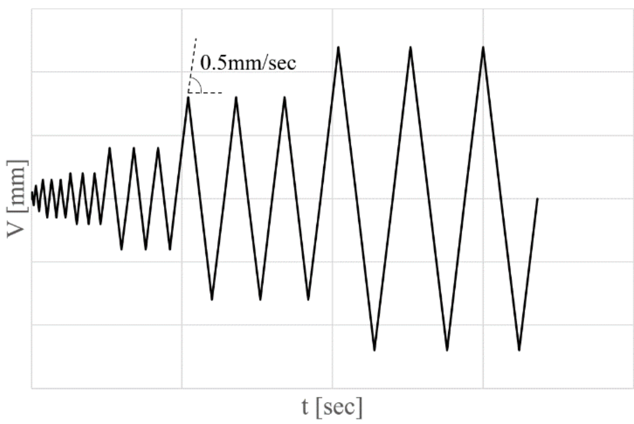


Figure 8. Load protocol followed for shear cyclic tests

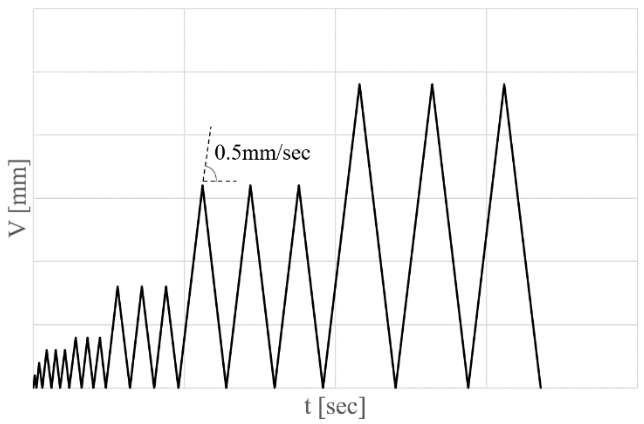


Figure 9. Load protocol followed for tension and 45° cyclic tests

total reversal load protocol, as showed in Figure 8. On the other hand, the tension and 45° tests were performed imposing cycles from zero to positive (tensile) values (see Figure 9). As can be observed from Figures 4 and 6 these two test setups did not allow the excursion in negative displacement values.

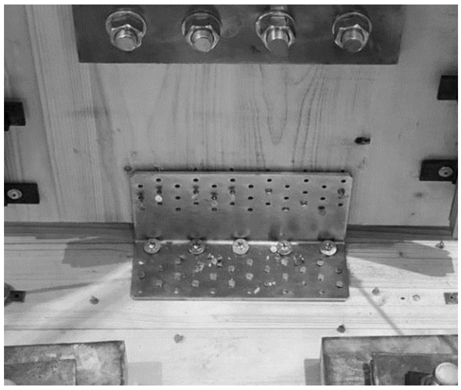
3.3 Experimental results

Similar failure mechanisms occurred at the end of tension, shear and 45° tests. As predicted in the section 2.3, the over-strengthening of the bottom connection ensured a shear failure of the nails connected to the vertical panels. Figure 10 shows the specimens at the end of tension, shear 45° cyclic tests. It can be seen that in all three type of tests, for effect of cyclic load, some nails were extracted from the timber while in others the break of heads occurred.

The experimental results were evaluated following the procedure prescript in EN12512 (2001). The parameters evaluated were: the initial stiffness K_{el}, the yielding force F_y, the yielding displacement V_y, the maximum force F_{max}, the displacement corresponding to the maximum force V_{max}, the ultimate force F_u, the ultimate displacement V_u and the ductility D.

The mean values of the mechanical parameters of monotonic and cyclic tests are reported in Table 1. It can be observed that similar values of yielding, maximum and ultimate forces were found between the 45° and tension tests. In this case the differences were less than 4%. On the other hand greater differences in the forces were found between the 45° and shear tests, with a maximum difference equal to 16,8%. The ultimate displacement of 45° tests differed of 20,8% and 10,6% with the tension and shear tests respectively. However, in this case, a scatter between the monotonic and the cyclic curves in the tension tests affected the result. Considering solely the averages on the cyclic curves, the differences between the 45° and tension and shear tests decreases to 11,4% and 5%.

Monotonic and cyclic load displacement curves of tension, shear and 45° tests are showed in Figure 11. From the cyclic curves, it can be noticed that the sets of three cycles (3-cycles) at the same displacement were five for the tension and 45° tests and four for the shear tests.



(a)

(b)

(c)

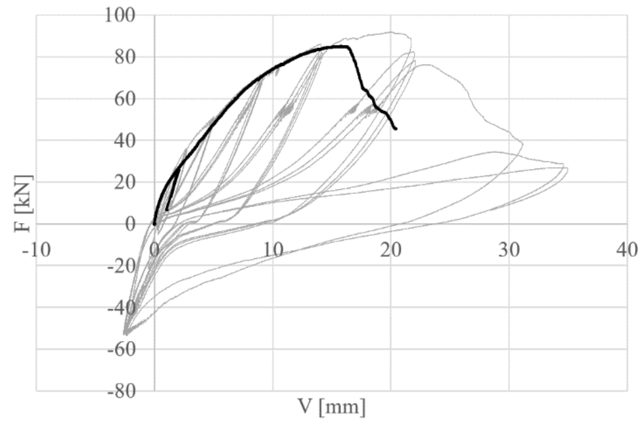
Figure 10. Photos of the specimens at the end of tension (a), shear (b) and 45° (c) tests

Table1: Mechanical properties from tension, shear and 45° tests

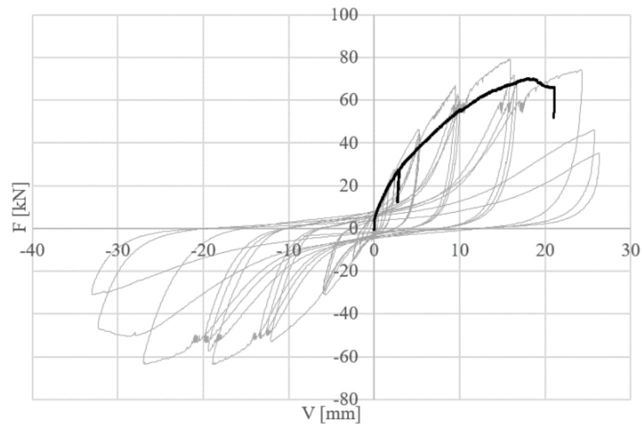
	Tension	Shear	45°
K_{el} [kN]	11.08	7.67	10.43
F_y [kN]	64.86	66.72	63.43
F_{max} [kN]	84.21	77.86	87.11
F_u [kN]	67.37	62.28	69.69
V_y [mm]	5.82	7.79	5.70
V_{max} [mm]	18.27	17.03	2308
V_u [mm]	21.67	23.57	27.28
D [-]	3.88	3.03	4.96

This was due to the yielding displacement value found in the shear monotonic tests, which was higher than those of the tension and 45° monotonic tests. A good matching can be observed between the monotonic and cyclic curves of the shear and 45° tests whereas in the case of tension test, the failure in the monotonic test occurred earlier than that of cyclic ones.

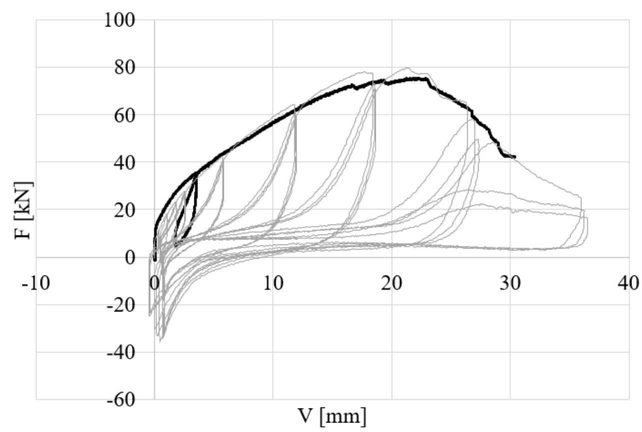
Mechanical parameters related to the cyclic behaviour were also evaluated, according to EN12512 (2001). Equivalent viscous damping υ_{eq} was evaluated at each cycles *i* as the ratio between the available potential energy $E_{p,i}$ and the dissipated energy $E_{d,i}$ multiplied by 2π. The available potential energy $E_{p,i}$ was evaluated as $E_{p,i}=1/2\ F_i\ V_i$ where F_i is the maximum force in the cycle *i* and V_i is the corresponding displacement. Both dissipated and available potential energies were evaluated considering the positive range of displacements. Thus for the tension and 45° tests the whole load displacement curve was considered for the evaluation of equivalent viscous damping. On the contrary, only half of the load displacement curve was considered for the shear tests. Figure 12 shows the equivalent viscous damping in the 1^{st}, 2^{nd} and 3^{rd} cycles for all set of 3-cycles until the failure. In all tests the equivalent viscous damping decreases between the 1^{st} the 2^{nd} and the 3^{rd} cycle. The only exception occur in high displacement levels (around 30mm), in the fifth set of 3-cycles, where the equivalent viscous damping in the 2^{nd} cycle is higher than that in the 1^{st} cycle. For low displacement levels, up to the third sets of 3-cycles, a quasi-constant equivalent viscous damping can be observed in all tests. For instance, the equivalent viscous damping of the 1^{st} cycles of the first three sets of 3-cycles was 0.11, 0.16 and 0.15 for tension, shear and 45° tests respectively. For higher displacement, a higher equivalent viscous damping was found in all tests, especially



(a)



(b)



(c)

Figure 11. Typical monotonic and cyclic load displacement curves of tension (a), shear (b) and 45° (c) tests

in the fifth set of 3-cycles of the tension and 45° tests, where values equal to 0.27 and 0.20 were found for the 2^{nd} cycles.

The dissipated energy E_d was evaluated as the area of the whole hysteresis loops for the tension and 45° tests and the area of half hysteresis loops for the shear tests. Figure 13 shows the dissipated energy in cumulated form for tension, shear and 45° tests. From the figure can be observed that a very similar trend occurs between the curves of shear and 45° tests, up to the fourth set of 3-cycles. In the same region, a slightly lower dissipated energy values were found for the tension tests. At the end of the tests, the specimen dissipated 5900kNmm, 5400kNmm and 3800kNmm in the tension, 45° and shear tests. The lower value of the shear test occurred since in this case was not reached the fifth set of 3-cycles, as in tension and 45° tests.

The impairment of strength ΔF_{1-3} was evaluated as the difference between the strengths of 1st and 3rd envelope curve, at the at the same displacement level. Figure 14 shows the impairment of strength dimensionless with respect to the strength of the 1st envelope curve, for the different set of 3-cycles. From this graph can be noticed that the impairment of strength increase in all tests, up to the fifth set of 3-cycles. It can be noticed that when an impairment of strength near to 0.50 was reached, all the tests reached the failure.

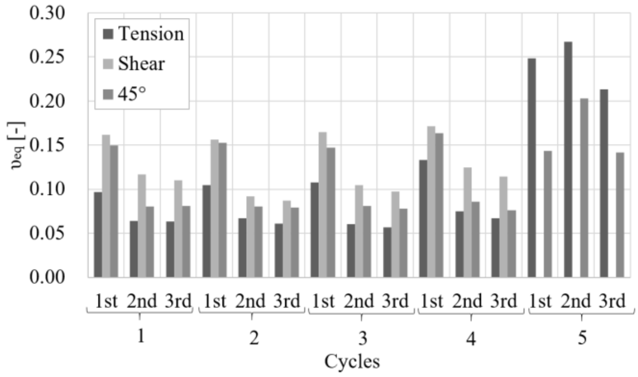


Figure 12. Equivalent viscous damping vs cycles for tension, shear and 45° tests

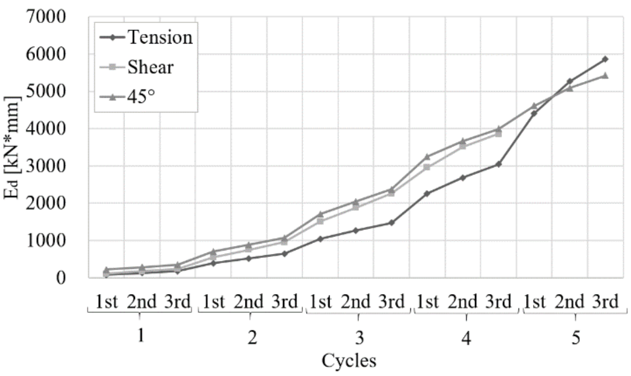


Figure 13. Cumulative dissipated energy vs cycles for tension, shear and 45° tests

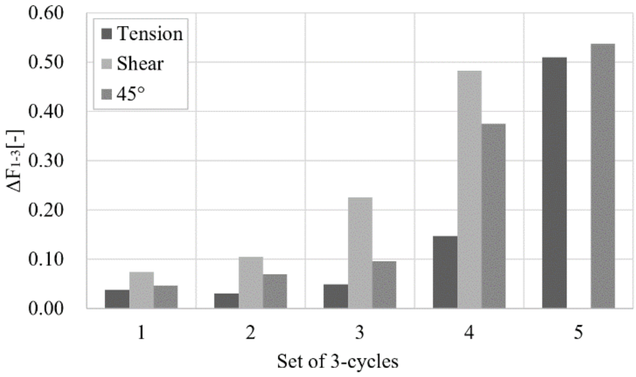


Figure 14. Impairment of strength vs number of set of 3-cycles for tension, shear and 45° tests

3.4 *Experimental force domains*

As discussed in section 2.3 a force domain can be defined for the innovative angle bracket. On the theoretical basis discussed in section 2.3, a circular strength domain should be expected. With the aim to define a force domain based on the experimental results, in table 2 are reported the mean values and the coefficient of variation of the yielding, maximum and ultimate forces found in all tension, shear and 45° tests.

Table 2: Mean values and coefficient of variations of yielding, maximum and ultimate forces from tension, shear and 45° tests

	Means	CoV [%]
F_y [kN]	64.85	10.18
F_{max} [kN]	83.46	10.40
F_u [kN]	66.77	10.40

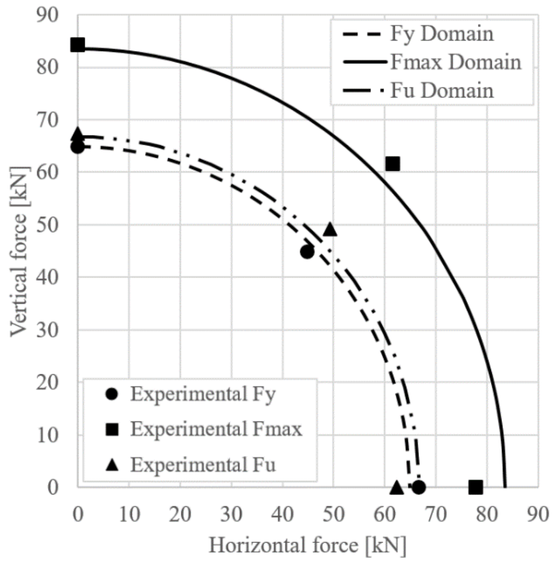


Figure 15. Force domains and experimental force values

These values can be used to define three different force domains, for yielding, maximum and ultimate condition. Figure 15 reports the three force domains and the values of experimental forces in tension, shear and 45°. It can be noticed that not a great scatter between the force domains and experimental force values is noticeable.

These force domains can be compared with the strength domains found in D'Arenzo (2019), through numerical analyses. In that case a strength domain with circular shape was found having the same range of force values found in this paper.

4 CONCLUSIONS

This article presents results of an experimental campaign on an innovative metal connector for CLT structures. The innovation of the angle bracket consists of the capacity to withstand tension and shear forces, differently from more typical metal connectors for CLT structures. This mechanical performance was achieved using fully threaded screws to connect the angle bracket to the floor panel. The insertion of fully threaded screws permitted to avoid typical brittle failure as withdrawal of the nails of the bottom flange.

To characterize the coupled behaviour of the angle bracket, three type of tests were performed: tension, shear and 45° tests. 45° tests were performed with the "strength domain" method, applying simultaneously the same horizontal and vertical forces. All tests were carried out monotonically and cyclically.

The analyses of the experimental results showed small scatter of mechanical parameters. In all three, tension, shear and 45° tests similar collapses were achieved, through failure of nails in the vertical flanges. Mechanical parameters related to the cyclic behaviour showed also a good correspondence.

Finally the "strength domain" test method permitted to propose an estimation of different force domains for the connector. According to EN12512 (2001), domains of yielding, maximum and ultimate forces were presented. The comparison between force domains and experimental forces confirmed the analytical consideration according to which a circular force domain can be expected for this angle bracket.

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