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SEISMIC CONNECTIONS FOR CLT STRUCTURES WITH TUBULAR ELEMENTS AND UNCOUPLED TENSION-SHEAR INTERACTION

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ABSTRACT: Cross Laminated Timber (CLT) panels possess high in-plane stiffness, and traditional earthquake resistant connections, originally designed to counteract separately rocking and sliding phenomena, are subjected to a combined tension-shear interaction. Detailed models involving tension-shear interaction may be considered unpractical to apply by designers with the dilemma of simpler ones not representative of the real behaviour of the panels. This work presents the mechanical response of a novel connection system which limits tension-shear interaction aiming to contribute towards the reliability of capacity design of CLT shear wall assemblies. Such connection system extends the concept of rocking shear-walls with shear keys used for high-rise CLT buildings to the case of low-to medium rise buildings. Latest results from monotonic and cyclic-loading experimental tests of the connection device are presented and the design implication of this solution is analysed via a comparative analysis adopting non-linear numerical simulations of a CLT structure.

KEYWORDS: Earthquake resistant connections, Cross-laminated Timber, Aluminium, Shear test

1 INTRODUCTION

Cross Laminated Timber (CLT) are large factory-manufactured panels commonly used in mass timber construction as both horizontal and vertical structural elements due to their flexibility in interpreting challenging architectural solutions. The high-level of prefabrication ensures a reduced time for the erection of such systems. Their high performance/weight ratio justifies their acceptance in several high-seismicity areas such as Mediterranean European countries, North American territories, Japan and New Zealand.

The seismic response of these structures is strongly dependent on the cyclic behaviour of connections between panels and to foundation because CLT panels behave almost elastically under in-plane forces. In particular when large-sized panels are used where the density of connections is drastically decreased [1]. Although a great effort of the research is toward the developing of novel dissipative connection/damping systems to be used for the realization of tall buildings, CLT is still mostly used to realize low-to medium rise buildings, e.g., residential dwellings and public buildings (schools, community centers). Here, traditional hold-downs and angle brackets, initially designed for intrinsically ductile and dissipative platform-frame structures, are still the most employed fastening systems.

As CLT panels possess high shear stiffness [2], these traditional connections, originally designed to counteract separately rocking (uplift) and sliding (shear) phenomena, are subjected to a combined tension-shear interaction [3, 4]. Furthermore, application of capacity design principles requires the shear connection to be over designed for strength and stiffness [5]; this may inevitably lead to an even higher tension-shear interaction of the connections. This can result in lower-dissipative cyclic response and often unpredictable connection failure; hence conservative behaviour factors are adopted in various International Seismic Codes [6, 7] and revision proposals [8].

Detailed models involving tension-shear interaction may be considered unpractical to apply by designers [9] with the dilemma of simpler ones not representative of the real behaviour of the panels. Therefore, a high number of fasteners are generally required as shear connectors to fulfill capacity design resulting in economically inefficient solutions. From a practical point of view novel connection systems able to simplify and speed up the realization of wall-floor-wall nodes in CLT structures are of paramount relevance.

This paper presents the mechanical response of a novel connection system which limits tension-shear interaction aiming to contribute towards the reliability of capacity design of CLT shear wall assemblies. The idea is to exploit metallic tubular elements inserted edge-wise into predrilled holes operating as shear-keys. They can also operate as tension-resistant connection (such as hold-downs) if the tubular connectors are connected to CLT with screws or dowels. Such connection system extends the concept of rocking shear-walls with shear keys used for high-rise CLT buildings [10] to the case of low-to medium rise buildings. Latest results from monotonic and cyclic-loading experimental tests of the connection device are presented. The obtained mechanical parameters are

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compared in terms of stiffness, strength, and ductility to that of traditional connection systems and to analytical and numerical models.

2 DESIGN AND CONCEIVING

2.1 OVERVIEW OF THE CONNECTION

The investigated connection system exploits tubular elements realized from extruded steel or aluminium inserted into pre-drilled holes at the edge of the timber panels (Figure 1). The location of the tubes along the wall is equivalent to standard hold-down and angle brackets connections, with the elements on the wall edges contrasting the wall rocking and the inner ones opposing to the horizontal slip. While the outer tubes are connected to the wall with dowel-type fasteners to provide a vertical restraint, the inner tubes are meant to work as traditional shear-keys thus restraining the horizontal displacements. The complex tension-shear interaction is almost completely avoided by simply increasing the diameter of the holes on the floor panel where the outer tubes are placed, i.e., forcing the transmission of the shear force by contact of the inner tubes only (Figure 2).

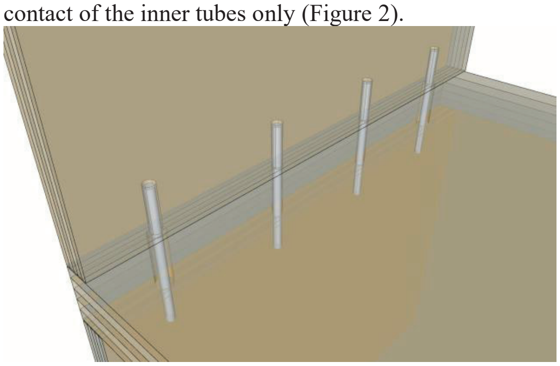

Figure 1: *Conceptualization of the analysed connection system*

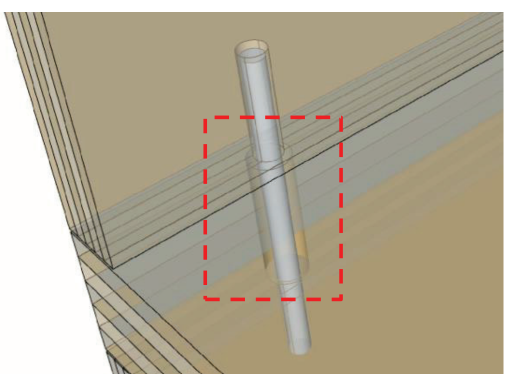

Figure 2: *Gap of the floor panel for the edge connections*

The material properties of the tube are of utmost importance in this application. Aluminium is already being successfully applied as good medium to realize other timber joints thanks to its lightness and good mechanical performance [11, 12]. In this case, its use as tension-resistant connection would benefit for the types of screws or dowels as less drilling force is required to pass through the tube walls. On the contrary, its lower stiffness compared to steel may provide reduced performance as a shear-key connector.

2.2 ANALYTICAL DESIGN

The conceiving phase started with the analytical determination of the load-carrying capacity of a dowel-type fastener drilled into the tube and locking it to the CLT panel. This was mandatory to overdesign the tube cross-section, namely diameter and thickness that guarantee a sufficient overstrength to the element.

The CLT panel thickness and layer arrangement implicitly define the possible failure modes of the dowel-type fasteners used to fasten the tube to the wooden element. Considering that the connection was purposely designed to be used into a low-rise structure (up to two-three storey high), the reference CLT wall panel thickness can be figured in the range of 100-140 mm in a 3-to-5-layer configuration.

Within the 3-layer configuration it is likely possible that the tube will exploit the whole inner layer and therefore the possible failure modes for this condition are already included in the Eurocode 5 [13] for the case of steel plates of any thickness as the central member of a double shear connection:

$$F_{v,Rk} = min\begin{cases} f_{h,k}td & (f) \\ f_{h,k}t\,d\left[\sqrt{2+\dfrac{4M_{y,Rk}}{f_{h,k}dt^2}}-1\right]+\dfrac{F_{ax,Rk}}{4} & (g) \\ 2{,}3\sqrt{M_{y,Rk}f_{h,k}d}+\dfrac{F_{ax,Rk}}{4} & (h) \end{cases}$$

For a 5-layer configuration the failure modes can be reconstructed by following the approach of [14] starting from the Johansen model. In summary, the possibility to obtain one or two plastic hinges is split into two sub-cases as reported in the equations below.

Mode 1 - wood embedment:

$$F_{v,Rk} = f_h \cdot d \cdot t \tag{1}$$

Mode 2.1 - one plastic hinge:

$$F_{v,Rk} = f_h \cdot d \cdot t\left(\sqrt{2}\sqrt{\beta\left[\beta(1-\psi)^2+\psi(2-\psi)+\frac{M_{y,Rk}}{f_{h1}\cdot d\cdot t^2}\right]} + \beta(\psi-1)-\psi\right) \tag{2}$$

Mode 2.2 - one plastic hinge:

$$F_{v,Rk} = f_h \cdot d \cdot t\left(\sqrt{2}\sqrt{2+\psi(\psi-2)-\beta(1-\psi)^2+\frac{M_{y,Rk}}{f_{h1}\cdot d\cdot t^2}} + \beta(1-\psi)+\psi-2\right) \tag{3}$$

Mode 3.1 - two plastic hinges:

$$F_{v,Rk} = \sqrt{2}\sqrt{2M_{y,Rk}\cdot f_{h1}\cdot d\cdot\beta} \tag{4}$$

Mode 3.2 - two plastic hinges:

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$$F_{v,Rk} = f_h \cdot d \cdot t\left(\sqrt{\left[(1-\psi)^2(1-\beta) + 4\frac{M_{y,Rk}}{f_{h1} \cdot d \cdot t^2}\right]} + \beta(1-\psi) + \psi - 1\right) \quad (5)$$

The tensile strength of the tube could be easily calculated following provisions from Eurocode 3 [15] or 9 [16], whether steel or aluminium is used.

Main outcomes from the analytical approach suggested that a 4-mm thick extruded profile of aluminium alloy equal to EN AW 6082 [17] with an external diameter equal to 40mm could be used for the purpose. The fastening of the tube shall be entrusted to self-drilling dowels with nominal diameter equal to 7 mm realized with steel grade S275.

3 TENSION-RESISTING JOINT: EXPERIMENTAL TESTS

Monotonic tests were performed to assess the main mechanical parameters of the joint if exploited as tension-resistant element.

3.1 SPECIMEN CHARACTERISTICS

The joint in the tension-resisting configuration was tested according to a simple compression setup using a universal testing machine (Figure 3). The CLT specimens were cut from a 5 layer 100-mm thick CLT panel (20-20-20-20-20) with dimension sufficient to respect actual edge distance provisions valid for the chosen connections. The resistance of the self-drilling was evaluated in two distinct configurations:

- a single dowel with length equal to 93mm placed orthogonally to the CLT panel face;
- a single dowel with length equal to 133mm placed at an angle of 45° to the CLT panel face.

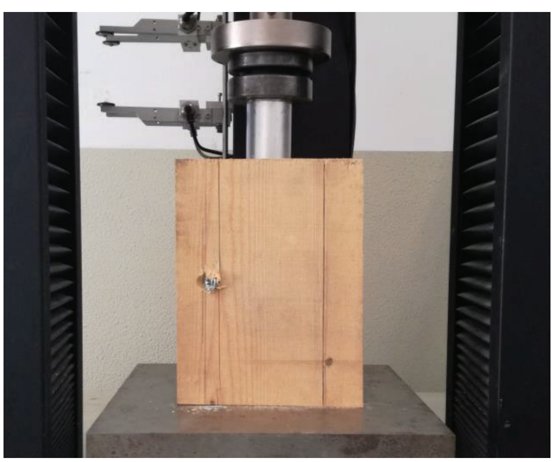


Figure 3: *Tensile test*

3.2 RESULTS

Preliminary tests were conducted to assess the contribution of friction between tube and CLT by pushing the tube into the hole without the insertion of the dowels. Results shows a resistance of about 500N to be reached before the triggering of the slippage phenomena. Force increased up to 1450N at 20mm of insertion.

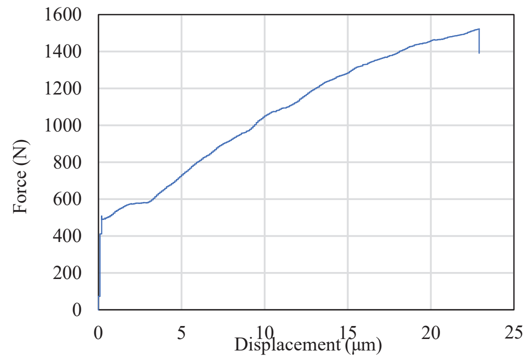
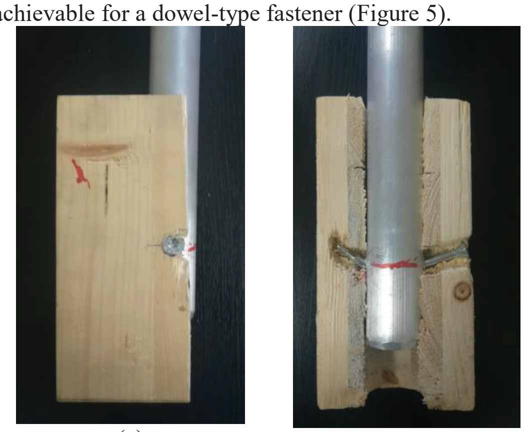


Figure 4: *Frictional contribution between tube and panel*

All monotonic tests returned a combined wood embedment failure coupled with the formation of two plastic hinges, technically the most ductile failure mode achievable for a dowel-type fastener (Figure 5).



(a) (b)

Figure 5: *Specimen inspection at failure*

Table 1 summarize the mechanical parameters calculated from the force displacement curves, for which average trends are reported in Figure 6 and Figure 7.

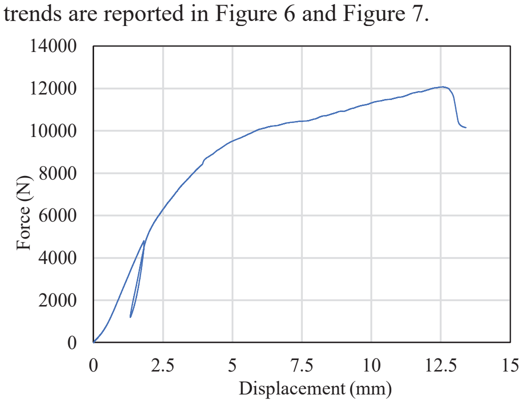


Figure 6: *Average force-displacement curve of a single dowel orthogonal to the panel (90°)*

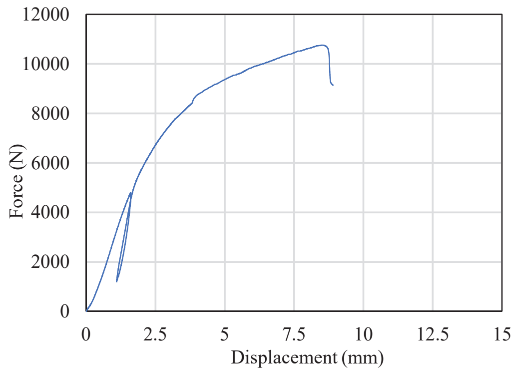


Figure 7: *Average force-displacement curve of a single dowel orthogonal to the panel (45°)*

dowel; iii) elastic-plastic orthotropic response for the timber element.
Large deformations and frictional contact laws were used to simulate the interaction (i.e., force transfer) between the different elements. The CLT panel was discretized according to the different composing layers. A bonded contact was applied between the different layers.
The simulated timber elements were a 200-mm thick CLT panel working as a floor, and a 100-mm thick CLT panel as a wall. A 5-layer configuration was chosen for both elements with 20 mm and 40 mm of thickness for the floor and wall respectively. Symmetry planes were adopted to reduce the computational costs of the models (Figure 8).
Firstly, a parametric analysis was performed to evaluate the changes of the mechanical performance by varying three key parameters defining the aluminium tube: strength class (yielding and ultimate strength in detail), external diameter and thickness. This becomes important not to overdesign the tube but also to obtain a cost-efficient solution.

Table 1: *Summary of tensile tests between the two tested fastening configurations*

	F_{MAX} [kN]	d_{FMAX} [mm]	F_Y [kN]	d_{FY} [mm]	K_{el} [kN/mm]	K_{pl} [kN/mm]	μ [-]	Ductiliy class [-]
Angle 90°								
Mean	12.01	14.80	8.99	2.15	4.60	0.24	7.53	H
Dev.St.	0.62	4.72	1.01	0.13	0.61	0.03		
5% Fractile	10.46		**6.48**					
COV	5.20%	31.88%	11.21%	6.21%	13.33%	10.54%		
Angle 45°								
Mean	11.15	9.95	9.63	2.42	4.93	0.21	5.05	M
Dev.St.	1.09	1.68	1.25	0.31	0.88	0.10		
5% Fractile	8.44		6.53					
COV	9.80%	16.84%	12.98%	12.78%	17.85%	48.84%		

The two configurations show comparable values of the yielding point, expressed by the resistance F_y and displacement d_y (evaluated according to EN 12512 method b [18]). A more pronounced post-elastic response of the dowels placed at 90° allows to reach a greater static ductility value. This returns that only the first configuration can be assigned to the highest ductility class according to [8].
Considering an average load bearing capacity at yielding of about 6.5 kN it results that about 6 to 8 dowels should be fastened to the same tube to obtain an overall capacity comparable to traditional hold-downs fastened with annular ringed-shank nails.

4 SHEAR-RESISTING JOINT: NUMERICAL SIMULATIONS

Given the high complexity and effort of designing a test to investigate in deep the behaviour of the tubular element when used as a shear-resisting connection, a 3D finite element model of the joint was developed with the commercial software Ansys Mechanical Workbench to test the performance of the joint. The analyses included material non-linearities for all the components: i) multilinear Ramberg-Osgood law [19] for the aluminium element; ii) bilinear elastic-plastic constitutive law for the

A parametric analysis was performed to assess the different response to be obtained when the floor panel is oriented parallel or perpendicular to the wall. Additionally, the contribution of the frictional forces that can develop at the wall-to-panel contact interface and at the CLT-tube interface was also included in the parametric analysis.

4.1 RESULTS

Sections 4.1.1 and 4.1.2 summarize the results obtained by considering the CLT panel representing the floor oriented parallel or perpendicular to the wall respectively. By comparing Figure 8 to Figure 11 it emerges the different location of the plastic hinge that is the consequence of the different stiffness of the external layer of the floor panel (light shaded elements on the left of the figure).
Considering a reference displacement of 15 mm and frictionless boundary conditions, it results in an important difference between the two panel orientations with an applied force equal to 93.5 kN for a parallel layout and 76.6 kN (-18%). Introducing friction at both interfaces raise the force level up to 113.9 kN and 91.1 kN respectively (-20%).

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The contribution of the frictional phenomena can also be appreciated from the superposition of the force-displacement curves obtained from the parametric analysis. Such occurrence, to be neglected in the design phase, can produce an increase of the total shear force between 8 to 10% attributed to the panel-to-panel interface and an additional 10% to the tube-to-CLT interface.

By adopting the same bilinearization method of the tensile-resisting configuration the yielding force F_y can be estimated between 50 to 70kN; within this range of shear load bearing capacity, a single tube can easily provide similar or higher performance of the last generation of angle-brackets.

4.1.1 FLOOR PARALLEL TO THE WALL

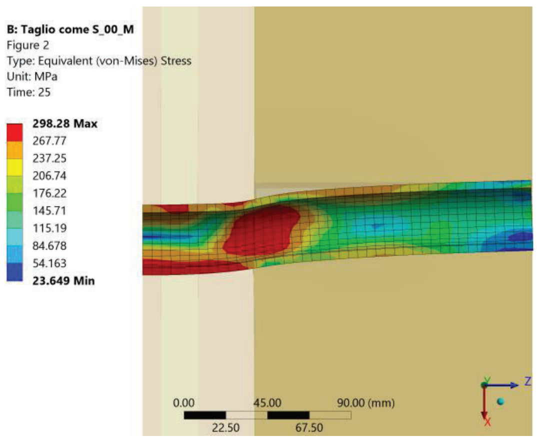

Figure 8: *von-Mises stress in the tube and plastic hinge location*

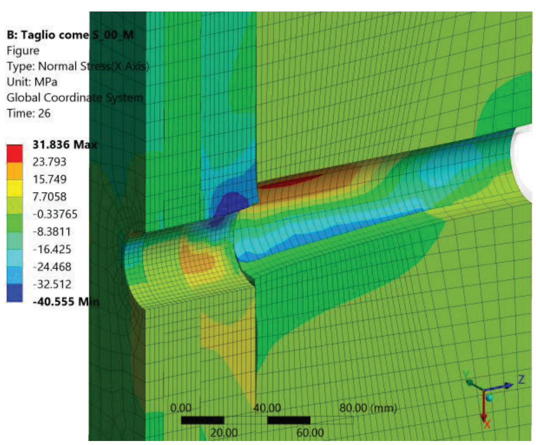

Figure 9: *principal stress in the CLT elements parallel to the loading direction*

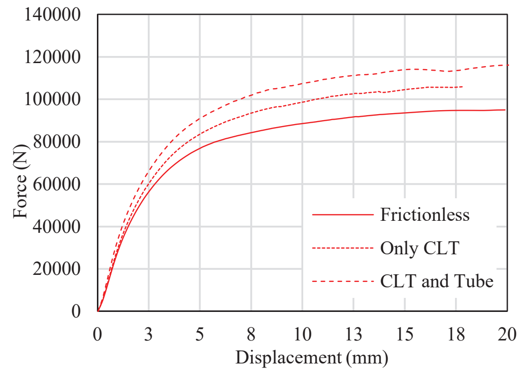

Figure 10: *numerical force-displacement curve with floor direction parallel to the wall*

4.1.2 FLOOR PERPENDICULAR TO THE WALL

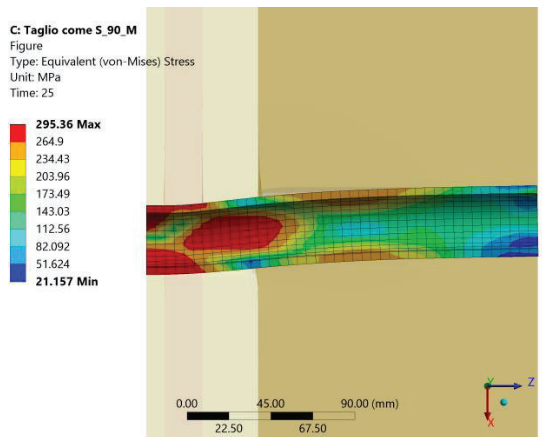

Figure 11: *von-Mises stress in the tube and plastic hinge location*

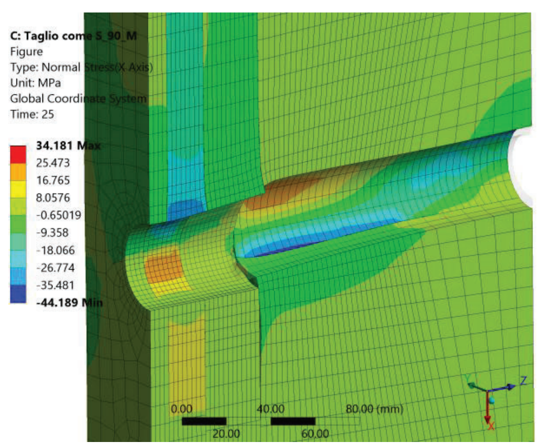

Figure 12: *principal stress in the CLT elements parallel to the loading direction*

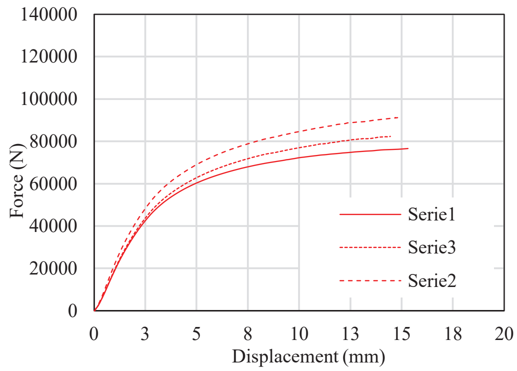

Figure 13: *numerical force-displacement curve with floor direction parallel to the wall*

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