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# ANALYTICAL AND EXPERIMENTAL STUDY ON REVERSIBLE STEEL-TIMBER COMPOSITE CONNECTION SYSTEMS

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**ABSTRACT:** This paper presents a study on the structural behavior of steel-timber composite (STC) with a special focus on reversible connection systems. Particularly, it addresses the research of composite structure floors comprising cold-formed steel (CFS) beams and cross-laminated timber (CLT) panels that allow a highly industrialized and reversible construction as well as high mechanical efficient since it overcomes the inherent limitations of each component. The analysis is based on an experimental campaign carried out to assess the mechanical behavior of the proposed connection systems in terms of stiffness and strength as well as relevant failure modes. A series of new bolted connection systems are proposed and push-out tests on five types of CFS-CLT composite connections were performed to obtain the load-slip curves and failure modes. The test results were compared with those provided by EN 1995-1-1 analytical models. The results showed that bolted joints using special types of nut obtained higher load bearing capacity and slip modulus than screwed joints of similar diameter. The failure modes were, mostly, simultaneous occurrence of embedding strength failure on CLT and slight developing of plastic hinges on connectors.

**KEYWORDS:** Steel-Timber Composite, Cross-Laminated Timber, Cold-Formed Steel, Connections

## 1 INTRODUCTION

Composite structures have been used as an effective solution for modern buildings combining the benefits of dissimilar materials to overcome their individual limitations and improve the structural system performance [1-2]. Besides the structural efficiency due to the composite action, the application of steel-timber composite (STC) structures allows to use prefabricated elements and, in some cases, a fully reversible construction system. STC applications for buildings presents many advantages proving their suitability as a structural solution, for instance: lightness, structural in-plan stability, low environmental impact, as well as the possibility of recycling and replace the degraded elements [3].

The engineered wood products with high mechanical performance such as Laminated Veneer Lumber (LVL), Glued Laminated Timber (Glulam) and Cross-Laminated Timber (CLT) are prefabricated structural elements of timber whose use have increased significantly in the construction industry due to lower carbon footprint, relatively high mechanical performance and lower self-weight in comparison with other traditional solutions [4-5]. The association of steel profiles and CLT slabs allows highly industrialized construction, as the elements are connected quickly, precisely, and using full-dry mechanical devices [3]. Comparatively to traditional lightweight timber structures, CLT panels are higher strength-to-weight ratio, and due to its crossed-layers and massive nature, the structural capacity and fire resistance are improved, making it possible to design and build large timber structures that are competitive against steel and reinforced concrete but significantly more sustainable [6-9]. The CFS profiles are attractive options in commercial and residential buildings due to high structural performance, lightweight, ease of prefabrication and quick assembly on site [10-12]. The increasing application of CFS in the construction industry has brought researches to overcome the safety issues and to

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improve the use of CFS elements as primary structures [13].

Therefore, the association of CFS beams and CLT panels can also satisfy requirements in terms of lightweight, prefabrication, modularity, sustainability, on-site installation, and relative manufacturing costs [1]. Navaratnam *et al.* (2021) [14] investigated the structural performance through FEM model of CLT-CFS composite beam for the flooring system. Karki *et al.* (2022) [15] carried out push-out tests of CFS-plywood connections with screws and bolts, as well as the influence of adhesives at the shear interface. Based on the results, the authors recommended using self-drilling screws, M8 nuts and bolts for CFS-timber composite flooring systems. Vella *et al.* (2020) [16] performed experimental tests of CFS and timber panel (*i.e.* plywood and particleboard) connections with inclined screws. This study evaluated the influence of type of timber panel, type and inclination of connector, and the thickness of CFS. The authors showed that the CFS thickness influences significantly the resistance to thread withdrawal failure and degree of rotational restraint of the fastener head.

In this paper, novel connection systems for CFS-CLT composites are presented in order to achieve an efficient mechanical behavior, allowing a simple and quick assembly/disassembly process, requiring fewer connectors, and thus, globally, turning the composite system into a more economical and efficient solution than the current ones found for CFS-CLT composites.

## 2 CONNECTIONS FOR CFS-CLT COMPOSITE STRUCTURES

The connector types used in this study comprise self-drilling screws and bolts, with 60 mm length and 8 mm diameter. More details are given in Table 1 and Figure 2.

***Table 1 - Parameter of connections***

| ID | Connector | Description |
|---|---|---|
| S1 | Screw | Self-drilling screws, 8 mm diameter, $f_{y,k}$ = 1000 N/mm² |
| B1 | Bolt + washer + nut | Bolt, 8 mm diameter, grade 8.8, with washer (24 mm external diameter) and nut |
| B2 | Bolt + T-nut | Bolt, 8 mm diameter, grade 8.8, with T-nut for wood (11 mm height, 9 mm external diameter and 22 mm outer diameter) |
| B3 | Bolt + threaded insert | Bolt, 8 mm diameter, grade 8.8, with threaded insert (19 mm height and 12 mm external diameter) |
| B4 | Bolt + threaded insert + T-nut | Bolt, 8 mm diameter, grade 8.8, with threaded insert (same as in B2 connection) and T-nut (same as in B3 connection) |

***Figure 1 – Connection components (a) Screw; (b) Bolt; (c) Washer and nut; (d) T-nut for wood and, (e) Threaded insert for wood***

Connections S1 and B1 ("S" stands for screw and "B" for bolt) are commonly used in timber structures and those were considered as reference connectors to the new solutions B2 (using a T-nut), B3 (using a threaded insert) and B4 (a combination of concepts of connections B2 and B3. Bolted connection systems are considered due to easiness for quick assemble and disassemble (presumably and assuming normal conditions of the use of the structure, i.e., service limit states) as well the possibility of pre-fabrication on factory. Threaded inserts and T-nut were used to provide stiffness to the bolted connections (i.e. to reduce the stiffness lose effect due to the hole clearance) and at the same time keeping the disassemble advantages of the bolted solution. In the bolted connections, 8 mm diameter holes were predrilled and at one of the surfaces of the CLT (solutions B1, B2 and B4), a 28 mm diameter hole was drilled insertion of washer/nuts and avoided the protruding from the outer surface of the CLT panel.

## 3 EN 1995-1-1 DESIGN RULES FOR CFS-CLT DOWEL-TYPE CONNECTIONS

The design rules for dowel-type that are recommended in EN 1995-1-1 [17] are based on the Johansen's Theory [18]. The method establishes the limit equilibrium equations for timber-to-timber and steel-to-timber connections, assuming the rigid-plastic behavior of the connectors and the timber subjected to crushing by the rigid connector. Two concepts that translate these hypotheses into the calculation are the localized crushing resistance of the timber and the plastic moment of the connector. Depending on the mechanical and geometric characteristics of the elements involved in the connection, the failure can take different forms; however, the friction forces between the connected elements and the withdrawal strength of the fasteners are ignored in Johansen's model [19-20]. The failure modes always imply the localized crushing of the timber, combined or not with plastic hinges in the connector [21].

The characteristic load-carrying capacity of the dowelled connections depends on the yield strength, the embedment strength, and the withdrawal strength of the fastener. EN 1995-1-1 [17] provides equations to determine the load-


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carrying capacity of single- and double-shear plane in different connection types [19].

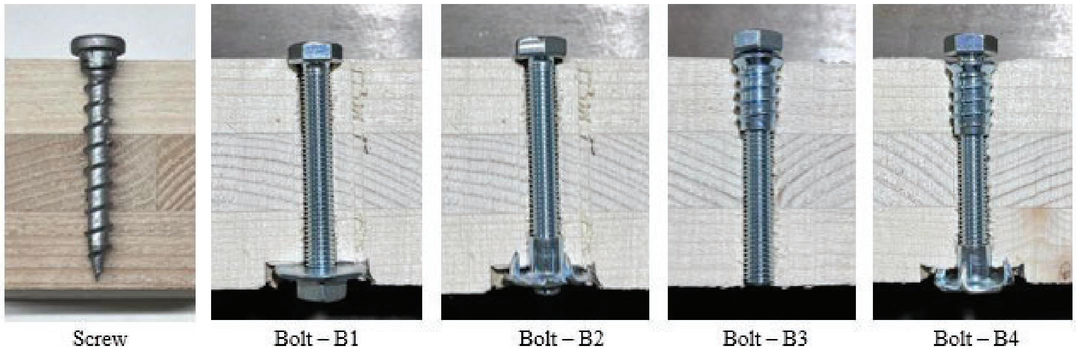


***Figure 2 – Types of tested connection systems.***

For a thin steel plate in single-shear, the characteristic load-carrying for screws per shear plane per fastener $F_{v,Rk}$, is taken as the minimum value found from Equation (1a):

$$F_{v,Rk} = min\begin{cases} 0.4\, f_{h,k}\, t_1\, d & (1a) \\ 1.15\sqrt{2\, M_{y,Rk}\, f_{h,k}\, d} + \frac{F_{ax,Rk}}{4} & (1b) \end{cases}$$

Where, $f_{h,k}$ is the characteristic embedment strength in the timber member; $t_1$ is the thickness of the timber member or the penetration depth; $d$ is the screw diameter; $M_{y,Rk}$ is the characteristic fastener yield moment; $F_{ax,Rk}$ is the characteristic withdrawal capacity of the screw.

For cold-formed steel profiles, thin-walled sheets are usually considered since their thickness is generally less than half the fastener diameter. In Figure 3, two failure modes are presented for thin steel-timber connection that represent, respectively, the embedment of timber member obtained from Equation (1a) and the same failure plus the plastic moment of the connector from Equation (1b).

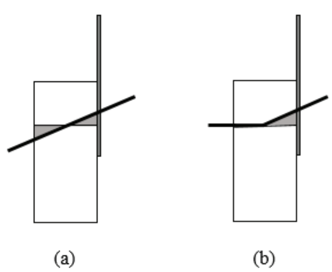


***Figure 3 - Failure modes for thin steel-timber connection; (a) embedment of timber member; (b) embedment of timber member plus plastic moment of the connector***

Models for determination of the embedment strength in CLT connections were developed in several studies [19-22] and an empirical model for the load-slip behavior of steel-CLT composite connections was proposed by Hassanieh *et al.* (2017) [23].

The value of the characteristic embedment strength in the timber member with pre-drilled is given in EN 1995-1-1 [17] by Equation (2):

$$f_{h,k} = 0.082\,(1 - 0.01\, d)\,\rho_k \qquad (2)$$

Where, $d$ is the screw diameter and $\rho_k$ is the characteristic density of wood.

The characteristic fastener yield moment ($M_{y,Rk}$), according to EN 1995-1-1 [17], is calculated from Equation (3):

$$M_{y,Rk} = 0.3\, f_{u,k}\, d^{2.6} \qquad (3)$$

Where, $f_{u,k}$ is the material characteristic tensile strength.

For bolted connections, according to EN 1995-1-1 [17], the characteristic withdrawal capacity is given by the minimum of the following possible failure mechanisms: maximum bolt tensile capacity ($F_{R,t,k}$), punching of the washer in the timber ($F_{ax,Rk,\ bearing}$) or punching of the bolt in steel plates ($B_{p,Rk}$), as shown in Equation (4).

$$F_{ax,Rk} = min\begin{cases} F_{\mathrm{R,t,k}} \\ F_{ax,Rk,bearing} \\ B_{p,Rk} \end{cases} \qquad (4)$$

The bearing capacity of steel plate should not exceed that an equivalent washer, whose external diameter corresponds to a minimum between *12t* and *4d*.

The slip modulus per shear plane per fastener under service load ($K_{ser}$) for screws or bolts are given by the Equations (5):

$$K_{ser} = {\rho_m}^{1.5}\, d/23 \qquad (5)$$

For steel-to-timber connections, $K_{ser}$ should be based on $\rho_m$ for timber member and could be multiplied by 2.0.

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# 4 EXPERIMENTAL PROGRAM

Experimental characterization of the developed CFS-CLT composite connections is carried out through push-out tests according to EN 26891 [27]. In total, five types of CFS-CLT connections systems were tested to characterize the shear-slip behavior of the proposed connections.

As result of tests, load-slip curves of CFS-CLT composite connections were obtained to determine the ultimate load capacity and slip modulus.

## 4.1 MATERIALS

The timber elements used in this study were a CLT panel made from 3-layered of Spruce lamellae of strength class C24 (Table 2). The panel was cut in 350 mm wide x 400 mm long stripes. In the push-out test the CLT panel was loaded in the direction parallel to the external layers' grain. The cold-formed steel profile used was an Omega profile S350GD whose properties are given in Table 3.

*Table 2 - Properties of CLT panel*

| Parameter | |
|---|---|
| Modulus of elasticity (MPa) | 11000 |
| Compressive strength (MPa) | 24 |
| Density (kg/m³) | 420 |
| Thickness (mm) | 3 x 20 |

*Table 3 - Properties of CFS profile*

| Parameter | |
|---|---|
| Modulus of elasticity (MPa) | 210000 |
| Yield strength (MPa) | 350 |
| Ultimate strength (MPa) | 420 |
| Thickness (mm) | 2.5 |
| Height (mm) | 250 |
| Length (mm) | 400 |

## 4.2 FABRICATION OF SPECIMENS

Five types of CFS-CLT connection (Figure 2) in a total of eighteen specimens (six for S1 and three for each bolted solution) were produced (Table 1). Each specimen was composed of a CLT panel connected to the flanges of two CFS profiles (each with two connectors per shear plane) using a symmetric layout, resulting in two independent shear planes, as shown in Figure 4.

## 4.3 LAYOUT AND CONFIGURATION OF PUSH-OUT TESTS

The specimens were placed in the testing rig as shown in Figure 5. Four linear variable differential transformers (LVDTs) were used to measure the relative displacement (or slip) between the CLT panel and CFS profile.

The loading procedure was carried out as specified in EN 26891 [27]. The test stop conditions were failure of the specimen, 15 mm of slip or 900 s of test, whatever occurred first. The loading procedure was updated with respect to the ultimate load obtained in the first specimen tested within each solution (pre-test).

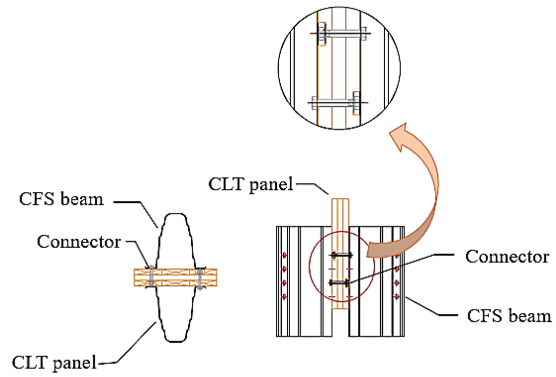


*Figure 4 – Scheme of connection tests*

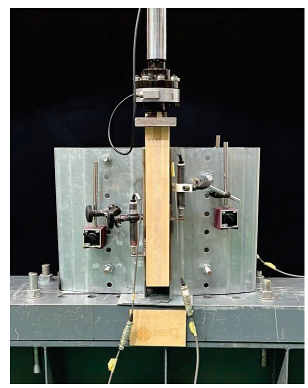

*Figure 5 - Layout of push-out test*

# 5 RESULTS AND ANALYSES

## 5.1 LOAD-SLIP CURVES

The load-slip curves of the CFS-CLT connections with screws and bolted connectors obtained per specimen and per solution are shown in Figures Figure 6 to Figure 10.

With regard to the ultimate load capacity ($F_{ult}$) of the composite connections it can be observed that the bolted solutions present higher values than joints with screws. Although the solutions B3 and B4 were pre-drilled, the bolts were connected with a nut at the top of the hole, which provided higher initial slip modulus than solutions B1 and B2. Although screwed connection had on average higher stiffness than solutions B1 and B2, the screwed joint showed rapid loss of stiffness with increase on load. All load-slip curves demonstrated ductile behavior. Similar to the results shown in Hassanieh *et al.* (2016) [4], the load-slip curves of the joint solutions with screws (Figure 6) showed close to elastic-perfectly plastic behavior, while the bolted solutions presented some hardening characteristics.


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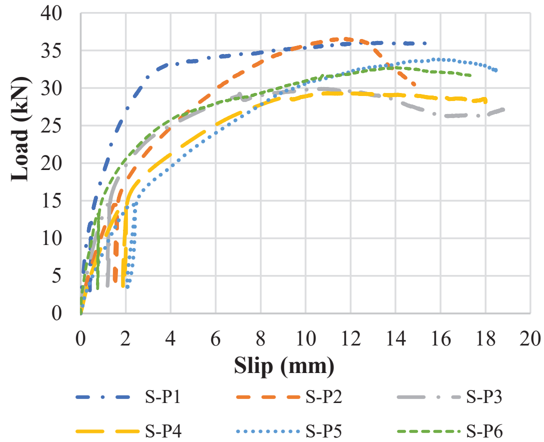


*Figure 6 – Screw load-slip curve*

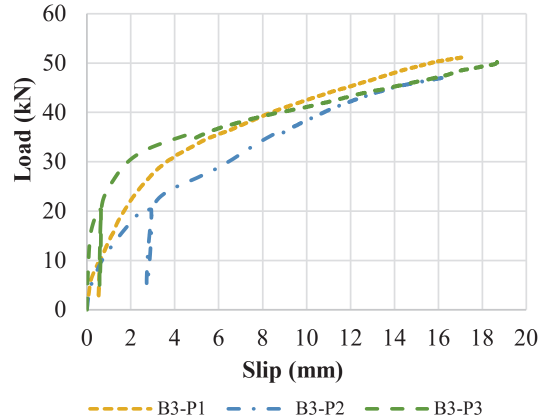


*Figure 9 – Bolt-B3 load-slip curve*

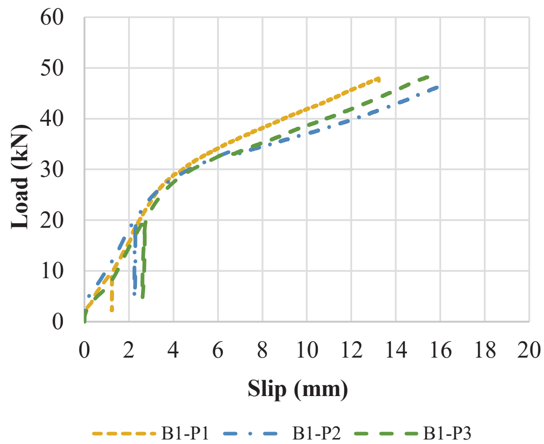


*Figure 7 – Bolt-B1 load-slip curve*

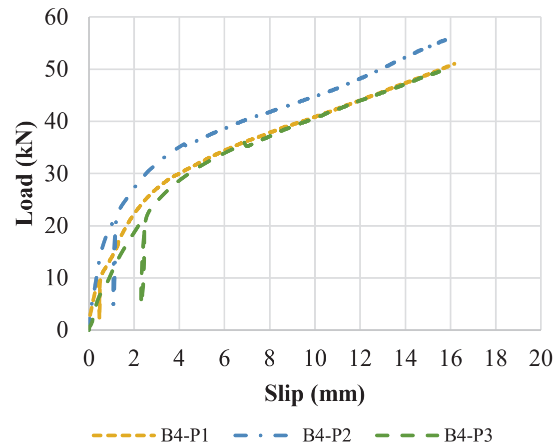


*Figure 10 – Bolt-B4 load-slip curve*

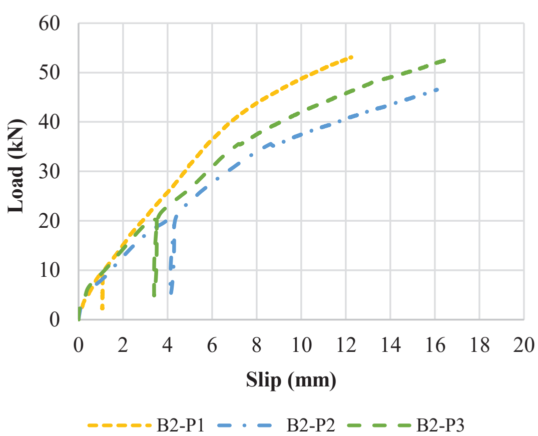


*Figure 8 – Bolt-B2 load-slip curve*

## 5.2 TEST RESULTS

The mean values obtained in the tests for the ultimate load capacity ($F_{ult}$) and slip modulus ($k_s$) - defined according to EN 26891 [27] - are presented in Table 4, as well the characteristic values calculated according to EN 14358 [28]. The mean ultimate load capacity for the screwed connections was 8.3 kN while for the bolted solutions, in descending order, was: 13.1 kN (B4), 12.7 kN (B2), 12.4 kN (B3) and 11.9 kN (B1). Regarding the mean slip modulus, for screws a value of 2773 N/mm was achieved while for the bolted solutions, in descending order, was: 4413 N/mm (B3), 3604 N/mm (B4), 1737 N/mm (B1) and 1543 N/mm (B2). Overall, the ultimate load capacity of the different bolted connections was quite similar (maximum difference around 1.2 kN). Both B2 and B3 connections presented similar load capacity, however B2 presented the lowest stiffness while B3 the highest. On the other hand, B4 connection (which was a combination of concepts of connections B2 and B3) resulted in a solution with the second highest slip modulus (3604 N/mm) and the highest ultimate load capacity of all solutions (13.1 kN).

***Table 4 – Test results per connector, mean and characteristic values, and comparison between EN 1995-1-1 predictions.***

| Type | Specimen nº | $F_{ult}^{*}$ (kN) | $k_s$ (N/mm) |
|---|---|---|---|
| S1 | 1 | 9.0 | 6044 |
| | 2 | 9.1 | 1781 |
| | 3 | 7.5 | 2249 |
| | 4 | 7.3 | 1465 |
| | 5 | 8.5 | 1227 |
| | 6 | 8.2 | 3871 |
| | Mean | 8.3 | 2773 |
| | CoV (%) | 9.1 | 67.0 |
| | Ch. value | 6.6 | - |
| | EN 1995-1-1 $F_{v,k}$ \| $K_{ser}$ | 7.5 | 5988 |
| | $F_{v,k}/F_{ult}$ \| $K_{ser}/k_s$ | 1.1 | 2.2 |
| B1 | 1 | 12.0 | 1604 |
| | 2 | 11.6 | 1833 |
| | 3 | 12.2 | 1774 |
| | Mean | 11.9 | 1737 |
| | CoV (%) | 2.5 | 6.8 |
| | Ch. value | 10.2 | - |
| | EN 1995-1-1 $F_{v,k}$ \| $K_{ser}$ | 5.8 | 5988 |
| | $F_{v,k}/F_{ult}$ \| $K_{ser}/k_s$ | 0.6 | 3.5 |
| B2 | 1 | 13.3 | 2003 |
| | 2 | 11.6 | 1036 |
| | 3 | 13.1 | 1591 |
| | Mean | 12.7 | 1543 |
| | CoV (%) | 7.2 | 31.5 |
| | Ch. value | 10.0 | - |
| | EN 1995-1-1 $F_{v,k}$ \| $K_{ser}$ | 5.8 | 5988 |
| | $F_{v,k}/F_{ult}$ \| $K_{ser}/k_s$ | 0.6 | 3.9 |
| B3 | 1 | 12.8 | 4015 |
| | 2 | 11.8 | 1591 |
| | 3 | 12.6 | 7634 |
| | Mean | 12.4 | 4413 |
| | CoV (%) | 4.1 | 68.9 |
| | Ch. value | 10.6 | - |
| | EN 1995-1-1 $F_{v,k}$ \| $K_{ser}$ | 5.8 | 5988 |
| | $F_{v,k}/F_{ult}$ \| $K_{ser}/k_s$ | 0.6 | 1.4 |
| B4 | 1 | 12.8 | 4428 |
| | 2 | 14.1 | 4390 |
| | 3 | 12.5 | 1994 |
| | Mean | 13.1 | 3604 |
| | CoV (%) | 6.4 | 38.7 |
| | Ch. value | 10.7 | - |
| | EN 1995-1-1 $F_{v,k}$ \| $K_{ser}$ | 5.8 | 5988 |
| | $F_{v,k}/F_{ult}$ | 0.5 | 1.7 |

Note: CoV: Coefficient of variation; Ch. Value – Characteristic value calculated according to EN 14358 [28]; *Ultimate load capacity equally distributed on four connectors.

In general, B1 solution compared poorly against the other bolted ones regarding stiffness and strength. Comparing the bolted solutions with the screwed one, it is found that the firsts performed better in terms of strength while in terms of stiffness only B3 and B4 solutions superseded the S1 connection.

Table 4 shows a comparison of the ultimate load capacity and slip modulus between push-out test results and those estimated according to EN 1995-1-1 [17]. According to the predictions, the characteristic load-carrying capacity per connector and per shear plane was $F_{v,k}$ = 7.5 kN and $F_{v,k}$ = 5.8 kN for the screwed and bolted connections, respectively. The slip modulus under service was $K_{ser}$ = 5988 N/mm for both screws and bolts.

The estimated ultimate load capacity was mostly lower than the value obtained in tests, with a test-to-estimated ratio between 0.5 and 0.6 for bolts. In terms of the estimated slip modulus, the results showed that those were significantly higher than the test values - the mean ratio was 2.2 for the screwed connection and between 1.4 and 3.9 for the bolted connections.

It should be noticed that the coefficient of variation of the results was noticeable (67.0% for screwed joints and between 6.8% and 68.9%), and this may be the reason for the discrepancy found. Likewise, Vella *et al.* (2021) [29] presented the same analysis for CFS and plywood composite connections and reported that the estimated load capacity was significantly lower than that measured from the tests, with a mean test-to-estimated ratio $F_{v,EN}/F_{v,k}$ of 0.55 and even higher ratios of $K_{ser}/k_s$ (around 6.20) on their connections.

Overall, from the novel connections proposed and assessed in this study, solutions B3 and B4 presented the highest and more balanced performance, and thus showing a significant potential to be used in CFS-CLT structures.

## 5.3 FAILURE MODES

According to EN 1995-1-1 [17], two failure modes are possible for steel-to-timber composite connection with thin steel plate in single shear – (a) embedment of timber and connector rotation; (b) connector bending and plastic hinge.

Figure 11 shows slight developed plastic hinges on a bolt and on a screw (less pronounced in the last). Figure 12 and 16 show local deformations around the CFS holes and rotation of the screws and bolts head at the end of the tests, being more evident in the bolt connections because those were subject to higher loads. Figures 13 to 15 shows the crushing of timber and the embedding of the bolts. In the present study, the CFS sheet of the specimens was unable to significantly restrict the rotation of the connectors, which created a localized deformation around the CFS hole, a large rotation of the connector and excessive crushing in the CLT panel. Therefore, the failure modes obtained in the tests were much closer to mode (a) presented in Figure 3.


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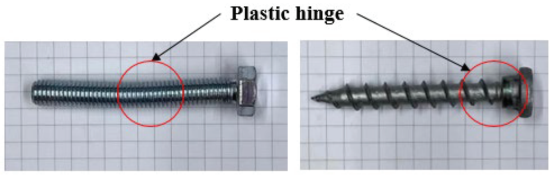


*Figure 11 - Failure details of screw and bolt.*

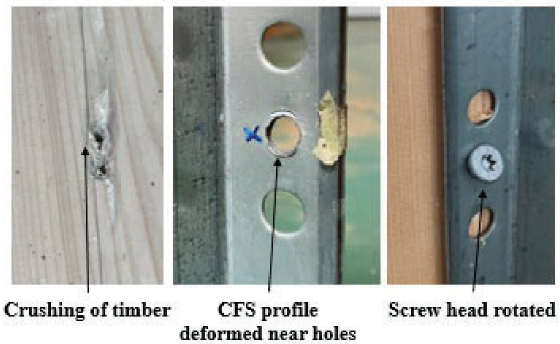


*Figure 12 - Failure details of screwed connection.*


*Figure 13 – Failure details of CLT panels (screws).*

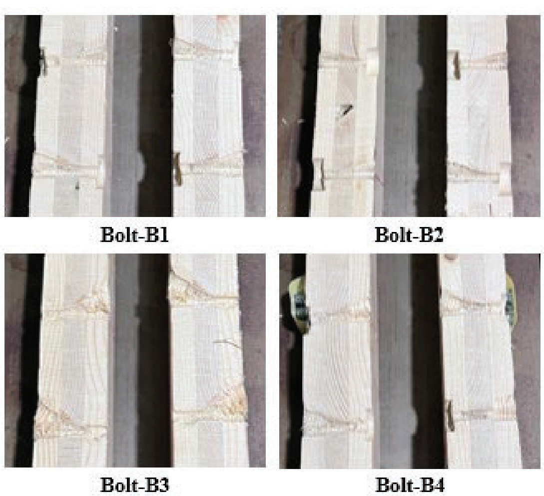


*Figure 14 – Failure details of CLT panels (bolts).*

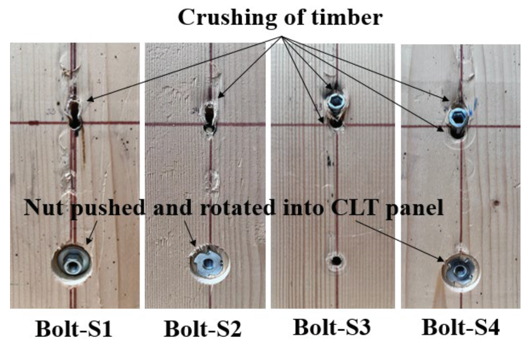


*Figure 15 - Failure details of bolted connection (CLT panel side).*

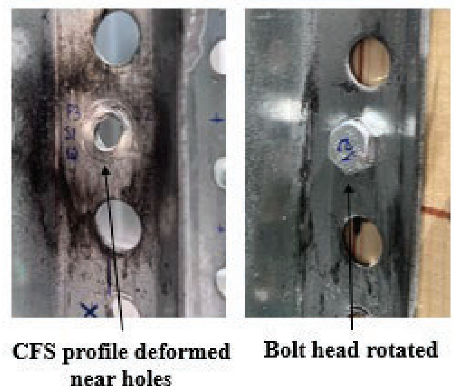


*Figure 16 - Failure details of bolted connection (CFS side).*

## 6 CONCLUSIONS

This study presented novel proposed types of connection for CFS-CLT composite solutions and their mechanical behavior. Those solutions include self-drilling screw S1 and bolt B1, which are commonly used in timber structures. In addition, new bolted solutions were proposed, *i.e.* B2 (using a T-nut), B3 (using a threaded insert) and B4 (a combination of concepts of connections B2 and B3.

Push-out tests on CFS-CLT connections were carried out according to EN 26891 [27]. The load-slip curves and failure modes revealed ductile behavior, with failure occurred after a large post-peak branch in the load-slip curve. It can be seen the failure modes were simultaneous occurrence of the embedding strength failure of CLT and slight developing plastic hinges in both connectors, as well as local deformations around the CFS holes and rotations of the screws and bolts.

Overall, the bolted connection solutions B3 and B4 obtained the highest ultimate load capacity and slip modulus, and thus showing a significant potential to be used in CFS-CLT structures.

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