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# MECHANICAL BEHAVIOUR OF NOTCH CONNECTION FOR MARITIME PINE CLT-CONCRETE COMPOSITE

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**ABSTRACT:** Cross Laminated Timber (CLT) is one of the most representative Engineered Wood Products, especially for low- and mid-rise timber buildings. Although in the 2000s, its use in Europe increased significantly, Portugal is still at a starting point and Spain needs a boost for its dissemination. Aiming promotion and dissemination of this type of buildings in SUDOE region, Eguralt project promotes research and innovation capacity for its sustainable growth. As a partner, one of SerQ's objectives is to promote the production of CLT solutions with wood from local species. The advantages of CLT are well known. However, the use of CLT in composite solutions requires the study of the overall performance, namely: stiffness; strength; dynamics and acoustic performance. Continuing recent studies in these areas, this paper presents an experimental campaign on the assessment of CLT-concrete connections and its comparison with the design models available.

CLT was produced at SerQ's facilities with local maritime pine (*Pinus pinaster* Ait.) wood. In situ concrete casting and precast concrete processes were a study variable. Notch shear connectors with different geometries and screws or U-shape rebar cages to prevent uplift, gives the system its composite behaviour. The load capacity of the connectors was theoretically predicted and validated with shear tests, and the experimental results were then compared with those. The results of the specimens cast in situ were satisfactory, while the results of the precast specimens were below expectations.

**KEYWORDS:** CLT-concrete composite, composite floor, connection, experimental testing, maritime pine, shear test.

## 1 INTRODUCTION

Both Glued Laminated Timber (glulam) and CLT are the most common used Engineered Wood Products (EWPs) for timber construction (rehabilitation and new buildings) all over the world [1,2]. They are mainly produced by softwoods, namely Spruce (*Picea abies* L.) and Scots pine (*Pinus sylvestris* L.) [3,4]. The current challenge of using timber structures in construction, is due to their positive environmental impact since they have significantly less embodied energy, and associated carbon emissions than equivalent steel and concrete structural systems [5].

The increasing demand for EWPs requires the study of alternative species, also facing some issues related to the environmental and economic impacts due to long-distance transport. Iberian Peninsula has a small number of CLT producers whose focus is the use of local species, namely Radiata pine (*Pinus radiata* D. Don) and Maritime pine (*Pinus pinaster* Ait.). Bonding and mechanical performance of both species, both for glulam and CLT, have already been validated for different adhesives [6,7]. The performance of CLT buildings by itself is recognized. However, adding a layer of concrete on the floor elements significantly increases their ultimate and serviceability performance throughout the increase of stiffness, load carrying capacity, vibration frequencies, and acoustics [8,9]. The key factor for an effective behaviour of both materials is the connection. Within the scope of Eguralt project, an experimental campaign intends to characterize the mechanical performance of CLT-concrete notched connections aiming to obtain an effective solution for medium/long spans (second phase of the study). In addition, focused on sustainability, the main goal is to develop prefabricated solutions that allow easy decoupling. The results are then compared with design methods namely those specified in the Eurocode 5 [10] and in the Technical Specification CEN/TS 19103 [11].

## 2 MATERIALS AND METHODS

The specimens were prepared in two different stages: S1 (May 2022) and S2 (January 2023). CLT specimens using three layers of maritime pine were produced at SerQ's facilities. In stage S1 the timber was selected according to density, ensuring the class was higher than C18. After planning, the boards had a final thickness of 30 mm and 100 mm wide. In stage S2 the timber was selected according to the dynamic modulus of elasticity, ensuring

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the class was equal to or higher than C24, and the boards had a final thickness of 30 mm and a width of 200 mm. A layer of concrete (maximum aggregate dimension of 14 mm) with 60 mm thickness was considered and reinforced with an electro-welded steel mesh (AQ50; 100 mm by 100 mm) placed at mid depth of concrete to prevent cracking. Table 1 shows the material properties in each stage, where $MC$ is the moisture content, $\rho$ is the density and $E_{dyn}$ is the dynamic modulus of elasticity of the timber; $f_{cm}$ is the concrete mean value of the compressive strength. The timber properties shown in this table are related to the lamellae at the interface with the concrete layer.

***Table 1:*** *Material properties.*

| Stage | Timber | | | Concrete | |
|---|---|---|---|---|---|
| | $MC$ [%] | $\rho$ [kg/m$^3$] | $E_{dyn}$ [MPa] | $f_{cm}$ [MPa] | Class |
| S1 | 14 ± 2 | 580 ± 80 | 12000 ± 4800 | 46.8 | C35/45 |
| S2 | 16 ± 2 | 620 ± 60 | 15500 ± 1500 | | |

Nine connection configurations are considered (see Table 2) based on the concept of a perpendicular/inclined notch and with/without screw to resist uplift stresses. Research on inclined notches is not recent [12]. However, the used shape follows the pattern referred to in Ouch *et al*. (2021) [13]. For those connections with a screw, SPAX 8x120 mm full threaded and washer head screw is installed in the centre of the notch. The connector design using these screws is covered by the European Technical Assessment ETA-12/0114 [14].

***Table 2:*** *Notch configurations.*

| Specimen ID | Notch [mm] | | | Stage | |
|---|---|---|---|---|---|
| | Top Length | Bottom Length | Depth | Timber | Concrete |
| MrP20 | 150 | 150 | 20 | S1 | S1 |
| MiP20 | 90 | 90 | 20 | S1 | S1 |
| MiI20 | 90 | 114 | 20 | S1 | S1 |
| MiP45 | 90 | 90 | 45 | S1 | S1 |
| MiI45 | 90 | 144 | 45 | S1 | S1 |
| MrpP20 | 150 | 150 | 20 | S2 | S1 |
| MrpI20 | 150 | 174 | 20 | S2 | S1 |
| MpP20 | 90 | 90 | 20 | S2 | S1 |
| MpI20 | 90 | 114 | 20 | S2 | S1 |

Meaning of the specimen ID:
M – wood species: maritime pine.
r – reference (150 mm notch top length); i – in situ concreting; p – prefabricated concrete.
P – perpendicular notch; I – inclined notch.
20 and 45 – notch depths in mm.

Three groups of two specimens 500 mm long, 200 mm wide and 90 mm thick were fabricated for each configuration. To comply with the symmetrical system indicated by Annex C of the Technical Specification CEN/TS 19103 [11], each specimen is composed of two CLT panels glued to each other and placed between two reinforced concrete (RC) slabs (see Figure 1). The assembly of each individual layer of RC with the respective specimen of CLT is previously performed. On reference specimens (MrP_) and in situ specimens with perpendicular notch (MiP_), a screw is installed in the centre of the notch – see Figure 2 (a). A U-shape rebar cage is installed on specimens with an inclined notch (MiI_) – see Figure 2 (b).

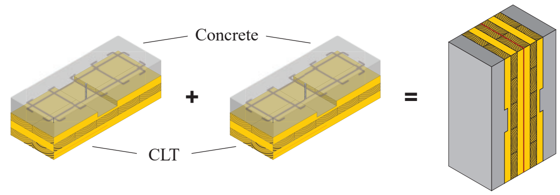


***Figure 1:*** *Specimens bonding.*

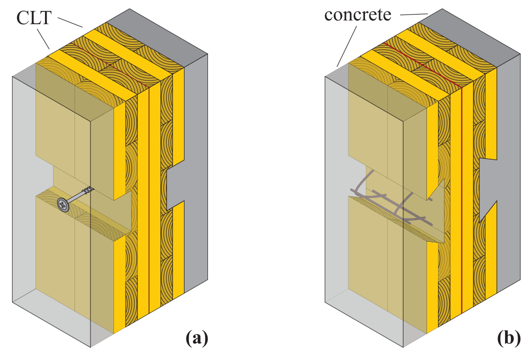


***Figure 2:*** *Notch configurations: (a) perpendicular and (b) inclined notch.*

Precast reinforced concrete panels (Mrp_ and Mp_) are manufactured with a 90 mm square hole in the centre. The assembly is carried out by filling the notch with grout SikaGrout®-218 [15]. In specimens with perpendicular notch (MrpP20 and MpP20), the connection is reinforced with a screw to prevent uplift, while in inclined notch specimens (MrpI20 and MpI20), no steel connector is installed (see Figure 3).

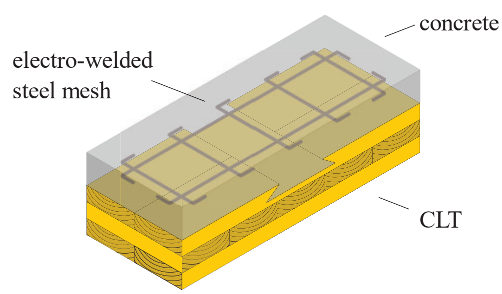


***Figure 3:*** *Inclined notch without steel connector.*

## 3 EXPERIMENTAL APPROACH

For the CLT-concrete composite connections characterization, the push-out test is used. The test setup is made conform to the push-out test given in EN 26891 [16] – see Figure 4– where the dashed line represents a simulation as the graph behaviour depends on the specimen being tested. To design the loading procedure, the maximum loading must be known or estimated. Table 3 shows the estimated load-carrying capacity for some of

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the specimens by Clause 10.3.4.3 of the Technical Specification CEN/TS 19103 [11]. The estimate was made considering timber strength class C24 and concrete strength class C35/40.

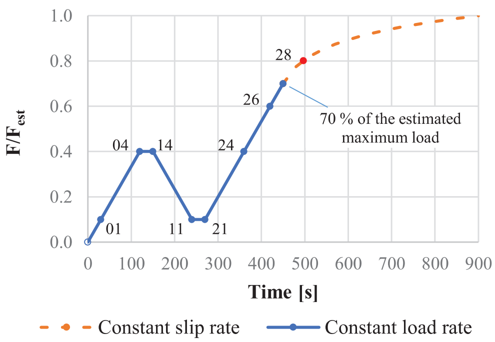


***Figure 4:*** *Loading procedure given in EN 26891* [16].

***Table 3:*** *Estimated maximum load for each stage S1 notch configuration based on characteristic values.*

| Specimen ID | Load-carrying capacity $F_{Rd,shear\ plan}$ [kN] | Slip modulus $k_{ser}$ [kN/mm] | Failure mode |
|---|---|---|---|
| MrP20 | 52 | 200 | Crushing of timber |
| MiP20 | 52 | 200 | Crushing of timber |
| MiI20 | 52 | 200 | Crushing of timber |
| MiP45 | 107 | 300 | Shear of concrete |
| MiI45 | 116 | 300 | Crushing of timber |

Test setup consisted of a force jack with a capacity of 500 kN that applied a uniform vertical load on the top surface of the CLT block through a steel plate that distribute the load across the timber, a symmetrical specimen of CLT-concrete connection, and two reaction steel plates (high enough to allow the timber section to move freely vertically within the limits of the standard: at least 15 mm) to support the reinforced concrete plates. To avoid a dangerous collapse of the specimen, it is restrained at the top and bottom without prestressing, as shown in Figure 5 (a).

To measure relative displacement (slip) between the CLT and concrete panels four LVDTs are placed, at half height, two for each shear plan – see Figure 5 (b).

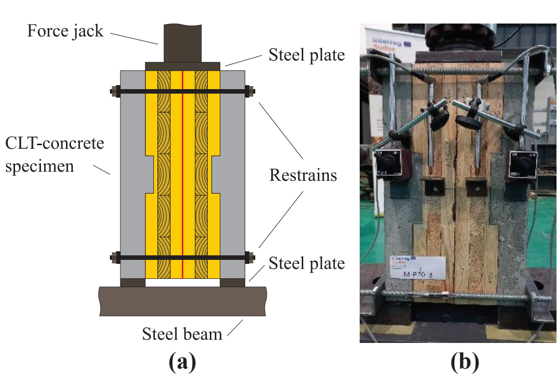


***Figure 5:*** *(a) test setup and (b) LVDT positions.*

Load and slips were recorded at 0.1 s and all tests were filmed by two cameras, one positioned at the front and other at the back and photographed from all sides for later evaluation and clarification of doubts related to failure modes.

## 4 RESULTS AND DISCUSSION

### 4.1 RESULTS

Figure 6 shows the load ($F_{appl}$) *versus* joint slip of the representative shear planes of each perpendicular notch configuration, and Figure 7, of the inclined notches. Generally, the specimen reaches its maximum load-carrying capacity (collapse of the first shear plane) before a differential displacement between the CLT and the reinforced concrete of 2 mm in the notch area. After that, most specimens reach about half their maximum load-carrying capacity (collapse of the second shear plane) for slips greater than 5 mm. The specimens with notches 150 mm long, cast in situ, (reference specimens) have the highest load-carrying capacity and a ductile behaviour was observed. Specimens with a 90 mm long notch have an intermediate load-carrying capacity and prefabricated specimens have a lower load-carrying capacity.

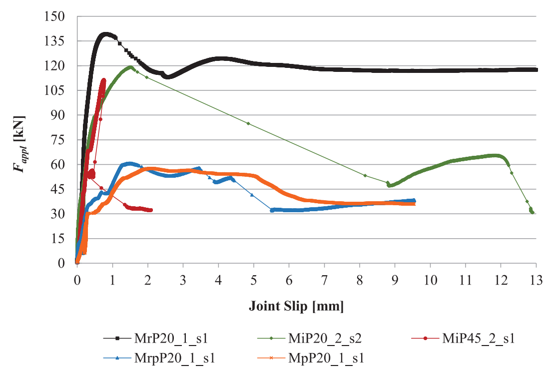


***Figure 6:*** *Force per shear plane versus joint slip (perpendicular notch).*

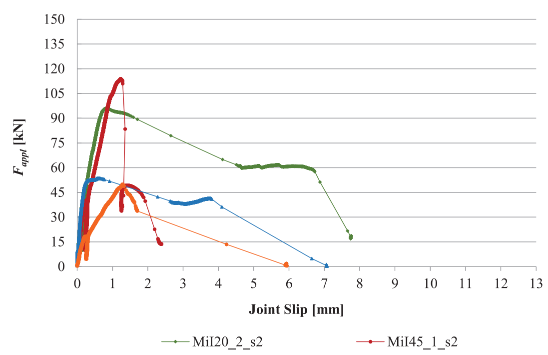


***Figure 7:*** *Force per shear plane versus joint slip (inclined notch).*

To complement the above, see Table 4 that shows the load-carrying capacity ($F_{max}$), and the stiffness of the connections in the service limit state quantified by the slip


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modulus ($K_s$). These values are provided by the average ($\mu$), standard deviation ($\sigma$) and coefficient of variation (relative standard deviation, *RSD*).

***Table 4:*** *Mean values of shear strength and slip modulus per shear plane.*

| Specimen | $F_{max}$ [kN] | | | $K_s$ [kN/mm] | | |
|---|---|---|---|---|---|---|
| | $\mu$ | $\sigma$ [-] | *RSD* [%] | $\mu$ | $\sigma$ [-] | *RSD* [%] |
| MrP20 | 140.6 | 6.9 | 4.9% | 376.0 | 111.0 | 29.5% |
| MiP20 | 117.3 | 4.7 | 4.0% | 302.4 | 78.0 | 25.8% |
| MiI20 | 100.7 | 5.3 | 5.3% | 206.8 | 66.7 | 32.2% |
| MiP45 | 109.0 | 5.2 | 4.8% | 251.1 | 76.0 | 30.3% |
| MiI45 | 109.6 | 10.1 | 9.2% | 177.7 | 37.4 | 21.1% |
| MrpP20 | 57.3 | 5.4 | 9.4% | 162.3 | 37.7 | 23.2% |
| MrpI20 | 52.9 | 6.4 | 12.2% | 194.4 | 62.6 | 32.2% |
| MpP20 | 61.0 | 6.1 | 10.1% | 228.4 | 147.9 | 64.7% |
| MpI20 | 49.8 | 4.8 | 9.7% | 252.4 | 188.5 | 74.7% |

Table 5 shows the maximum mean slips at maximum load ($\delta_{Fmax}$) and just before collapse ($\delta_{Collapse}$). Generally, the reference specimens (MrP20) show a greater slip at maximum load. Higher values were only observed at similar configuration, for pre-cast connection (3.7 mm at maximum load). This happened very close to the collapse which occurred on average at 51 kN per shear plane.

***Table 5:*** *Mean slip values on connections.*

| Specimen | $\delta_{Fmax}$ [mm] | | | $\delta_{Collapse}$ [mm] | | |
|---|---|---|---|---|---|---|
| | $\mu$ | $\sigma$ [-] | *RSD* [%] | $\mu$ | $\sigma$ [-] | *RSD* [%] |
| MrP20 | 1.50 | 0.38 | 25.41% | 2.59 | 1.08 | 41.66% |
| MiP20 | 1.09 | 0.32 | 29.46% | 1.25 | 0.39 | 30.85% |
| MiI20 | 1.46 | 0.66 | 44.88% | 1.87 | 0.76 | 40.76% |
| MiP45 | 0.63 | 0.22 | 34.98% | 0.71 | 0.32 | 45.58% |
| MiI45 | 1.12 | 0.31 | 27.33% | 1.24 | 0.39 | 31.86% |
| MrpP20 | 3.69 | 4.14 | 112.39% | 5.11 | 4.14 | 80.98% |
| MrpI20 | 0.61 | 0.28 | 46.67% | 2.24 | 1.91 | 84.93% |
| MpP20 | 1.44 | 0.44 | 30.27% | 6.10 | 1.56 | 25.61% |
| MpI20 | 1.14 | 0.26 | 23.08% | 1.63 | 0.96 | 59.04% |

Prefabricated specimens all collapsed due to shear of grout at the interface between the concrete and CLT plates. Precast reinforced concrete panels were manufactured with a 90 mm square hole in the centre. Despite the specimens' preparation having been carried out to ensure good adhesion between grout and concrete, in some areas it was found the grout had detached from the concrete. Figure 8 shows a shear plane after the failure. It can be seen areas where the grout was properly connected to the concrete and areas where grout has detached the concrete.

## 4.2 DISCUSSION

The specimens with a reference notch equally presented two collapse failure modes: shear and crushing of timber. In these specimens, the collapse has a ductile behaviour. It is possible to see shear of concrete, which may have been limited by the electro-welded steel mesh. Two of the reference specimens presented both failure modes in the same shear plane (see Figure 9).

Figure 10 shows the top of MrP20_1 CLT specimen where the growth rings of the two lamellae are shown and identified. It can be seen the lamellae with wider growth rings collapsed by crushing parallel to grain and lamellae with more compact growth rings collapsed by shear parallel to grain.

In the remaining configurations, the collapse was due to shear of concrete. Collapse was governed by a brittle behaviour instantly decreasing the load-carrying capacity of the connection.


***Figure 8:*** *Shear of grout in MrpI20_2 specimen.*


***Figure 9:*** *MrP20_1 specimen with shear and crushing of timber in the same shear plane.*

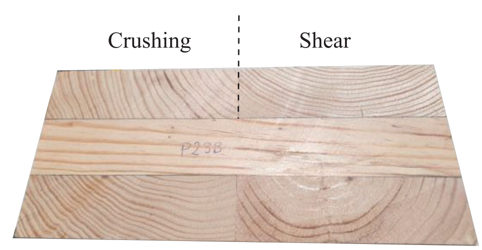


***Figure 10:*** *MrP20_1 specimen with shear and crushing of timber in the same shear plane.*

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Regarding the prediction of the connections load-carrying capacity, results given by the Technical Specification are lower than the experimental results. Nevertheless, connections with a 45 mm deep notch showed similar results. Technical Specification assumes slip modulus increase with the depth of the notch. On the opposite, tests revealed a decrease, comparing the 20 mm and 45 mm deep notches. Technical specification predictions related to failure modes only matched in specimens MrP20 (even with maximum load carrying capacity underestimated) and MiP45.

To evaluate the results of the prefabricated specimens, half of their width (100 mm) was considered to disregard some areas where the grout detached from the concrete. Table 6 presents the average load-carrying capacity and connection stiffness values, per millimetre, of the specimens cast in situ, considering a total width of 200 mm. The values for the prefabricated specimens are also presented, per millimetre, considering the width of 100 mm.

***Table 6:*** *Load-carrying capacity and connection stiffness values per millimetre.*

| Specimen | $F_{max}$ [kN/mm] | $K_s$ [kN/mm/mm] |
|---|---|---|
| MrP20 | 1.41 | 1.88 |
| MiP20 | 1.17 | 1.51 |
| MiI20 | 1.01 | 1.03 |
| MrpP20 | 1.15 | 1.62 |
| MrpI20 | 1.06 | 1.94 |
| MpP20 | 1.22 | 2.28 |
| MpI20 | 1.00 | 2.52 |

Figure 11 shows the notch length influence on results. MrP20 and MiP20 specimens were compared. Additionally, results of configurations A1 and A2 by Yeoh *et al.* (2008) [17] were also compared. In this analysis, MrP20 and A1, by Yeoh *et al.* (2008), specimens were taken as reference. The longer the notch, the higher the load-carrying capacity, slip modulus, and slip at the maximum load capacity value.

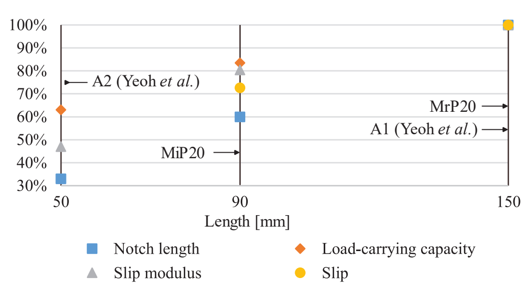


***Figure 11:*** *Influence of notch length on results.*

Figure 12 shows the perpendicular notch depth influence on results. MiP20 and MiP45 specimens were compared. Additionally, results of configurations A1 and A3 by Yeoh *et al.* (2008) [16] were also compared. In this analysis, MiP20 and A1, by Yeoh *et al.* (2008), specimens were taken as reference. In the MiP_ specimens, it was found that increasing the notch depth by 125 %, decreased both the load-carrying capacity and slip modulus by 20 %. The slip at the maximum load capacity value decreased about 40 %. In Yeoh *et al.* (2008) study, with a 50% decrease in the notch depth, slip modulus increased by 40 %, and load-carrying capacity was kept similar.

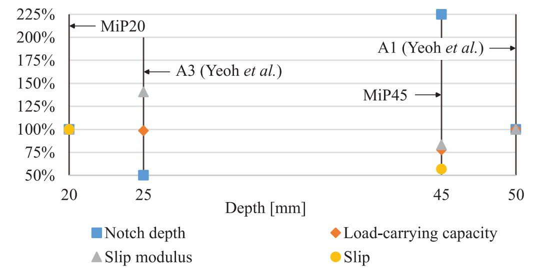


***Figure 12:*** *Influence of perpendicular notch depth on results.*

Figure 13 shows the inclined notch depth influence on results. MiI20 and MiI45 specimens were compared. Comparison with results from literature references was not possible. MiI20 specimen was taken as reference. It was found that increasing the notch depth (125 %), decrease both the slip (20 %) and slip modulus (15 %). The load-carrying capacity increased around 10 %.

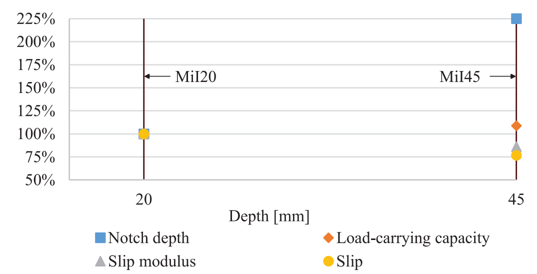


***Figure 13:*** *Influence of inclined notch depth on results.*

Figure 14 shows the inclined notch influence on results. MiI20/MiP20 (20 mm deep) and MiI45/MiP45 (45 mm deep) specimens were compared. MiP20 and MiP45 specimens were taken as reference. Inclination in the 20 mm deep notches resulted in 15 % loss of load-carrying capacity and 30 % of stiffness (slip modulus) of the connection. In the 45 mm deep notches, there was a loss of connection stiffness similar to that of the 20 mm notches, nevertheless, the inclination had no effect on the load-carrying capacity.

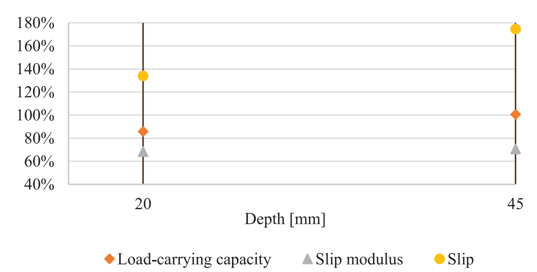


***Figure 14:*** *Influence of inclined notch on results.*


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The 150 mm long prefabricated specimens lose load-carrying capacity (about 20 %) compared to those cast in situ. When the notch is inclined the loss is higher (20 %). In the 90 mm long notches, the prefabricated specimens with perpendicular notches have a higher load-carrying capacity compared to inclined notches. The specimens with inclined notch maintain similar load-carrying capacity despite the length of the notch. The reference notch has less connection stiffness on the prefabricated specimens.

## 5 CONCLUSIONS

Connections between CLT produced with local maritime pine wood and reinforced concrete (cast in-situ and precast) were tested to study the composite system behaviour. The experimental results and the design results provided by the standards were compared.
The load-carrying capacity of the cast-in-situ concrete connection is greater than that of precast concrete, nevertheless, generally, the slip modulus is lower. The notch depth has no significant influence on results. The notch length is the most important factor affecting the connection performance both at mechanical properties and ductile behaviour. More tests must be carried out to increase the reliability of results.

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## REFERENCES

[1] M. Mohammad, S. Gagnon, B.K. Douglas, L. Podesto, Introduction to Cross Laminated Timber, Wood Des Focus. 22 (2012) 3–12.

[2] J. Hildebrandt, N. Hagemann, D. Thrän, The Contribution of Wood-Based Construction Materials for Leveraging a Low Carbon Building Sector in Europe, Sustain Cities Soc. 34 (2017) 405–418. https://doi.org/https://doi.org/10.1016/j.scs.2017.06.013.

[3] R. Brandner, Production and Technology of Cross Laminated Timber (CLT): A state-of-the-art Report, in: Graz, 2013.

[4] G. Fink, J. Kohler, R. Brandner, Application of European Design Principles to Cross Laminated Timber, Eng Struct. 171 (2018) 934–943. https://doi.org/https://doi.org/10.1016/j.engstruct.2018.02.081.

[5] C.A.S. Hill, The Environmental Consequences Concerning the Use of Timber in the Built Environment, Front Built Environ. 5 (2019). https://doi.org/10.3389/fbuil.2019.00129.

[6] C. Martins, A.M.P.G. Dias, H. Cruz, Bonding performance of Portuguese Maritime pine glued laminated timber, Constr Build Mater. 223 (2019) 520–529. https://doi.org/https://doi.org/10.1016/j.conbuildmat.2019.06.143.

[7] S. P., C. J.R., G. L., D. A.M.P.G., D. A., Bonding quality assessment of cross-layered Maritime pine elements glued with one-component polyurethane adhesive, Constr Build Mater. 211 (2019) 571 – 582. https://doi.org/10.1016/j.conbuildmat.2019.03.064.

[8] C. Martins, P.G. Santos, P. Almeida, L.M.C. Godinho, A.M.P.G. Dias, Acoustic performance of timber and timber-concrete floors, Constr Build Mater. 101 (2015) 684–691. https://doi.org/https://doi.org/10.1016/j.conbuildmat.2015.10.142.

[9] A.M.P.G. Dias, J. Skinner, K.I. Crews, T. Tannert, Timber-concrete-composites increasing the use of timber in construction, European Journal of Wood and Wood Products. 74 (2016). https://doi.org/10.1007/s00107-015-0975-0.

[10] EN 1995-1-1:2004+A1:2008, Eurocode 5: Design of Timber Structures, Part 1-1: General - Common Rules and Rules for Buildings, CEN - European Committee for Standardization, Brussels, 2008.

[11] CEN/TS 19103:2021, Eurocode 5: Design of Timber Structures – Structural design of timber-concrete composite structures – Common rules and rules for buildings, CEN - European Committee for Standardization, Brussels, 2021.

[12] M. Brunner, J.H.J.O. Negrão, K. Rautenstrauch, K.-U. Schober, Timber Concrete Composites, in: 2008: pp. 9–15.

[13] V. Ouch, P. Heng, Q.H. Nguyen, H. Somja, T. Soquet, A Notched Connection for CLT-Concrete Composite Slabs Resisting to Uplift Without Metallic Connectors: Experimental Investigation, in: Fib Symposium 2021: Concrete Structures: New Trends for Eco-Efficiency and Performance, Lisbon, Portugal, 2021. https://hal-univ-rennes1.archives-ouvertes.fr/hal-03254979.

[14] SPAX International GmbH & Co. KG, European Technical Approval ETA-12/0114, (2012). www.spax.com.

[15] Sika Portugal SA, SikaGrout®-218, (2020). https://prt.sika.com/pt/solucoes-para-construcao/ref-colagem-estrutural/grout-cimenticio/sikagrout-218.html (accessed February 24, 2023).

[16] EN 26891:1991, Timber Structures - Joints Made With Mechanical Fasteners -General Principles For the Determination of Strength and Deformation Characteristics, Technical, CEN - European Committee for Standardization, Brussels, 1991.

[17] D. Yeoh, M. Fragiacomo, P. Aldi, M. Mazzilli, U. Kuhlmann, Performance of notched coach screw connection for timber-concrete composite floor system, 1 (2008) 294–301.