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TIMBER-CONCRETE-COMPOSITE BEAMS WITH DISCRETE PERFORATED STEEL PLATE SHEAR CONNECTORS

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ABSTRACT: This paper describes an experimental research on timber-concrete-composite (TCC) beams made with discrete perforated steel plates shear connectors. The research included shear tests on different connectors made with perforated steel plates glued with epoxy in timber ribs and short and long-term test on beams with 8.88 meters of span. An unbonded tendon along with a self-tensioned system was used to limit the deflections of the beams on some of the short-term tests. A locked tendon was also used in long-term tests.

The different proposals of shear connectors made with discrete perforated glued steel plates showed a great strength and stiffness, with almost negligible slip between timber and concrete in service. This shear connectors, were used to design 8.88-meter span beams that showed a behaviour close to the full composite action in short-term tests, indicating a great potential to design medium and long-span beams and floors.

Some of the beams have been subjected to long-term loads for more than two years in an interior and uncontrolled environment. After that, the stiffness of the beams was evaluated under different test configurations and they were finally tested until failure.

KEYWORDS: long-span floors, timber-concrete composite, discrete shear connector, glued steel plates.

1 INTRODUCTION

Extensive research has been carried out in recent years around TCC systems, due to the enormous advantages they offer compared to only wood or only concrete solutions. The efficiency of these systems lies in the design of the shear connections between both materials, which must limit the relative slip in order to achieve a composite action as full as possible.

Traditionally, these connection systems have been classified into three large groups: notched connections, screwed connections and glued connections [1]. Within glued solutions, it can also be distinguished between direct glued solutions, in which the adhesive is the only connecting element between the wood and the concrete; and solutions with bars or plates glued into the wood, these being the ones that act as connectors with the concrete layer.

A considerable number of experimental studies have been carried out on connectors made with glued bars. Some studies have been carried out with continuous glued steel meshes used as shear connector, most of them linked to the system HBV® [2]. The studies carried out with short steel sheets placed discreetly along the length of beams or floor elements to constitute TTC systems are much smaller.

This paper summarizes the studies carried out by our team on the behaviour of a system based on the use of discrete perforated steel plates as shear connectors. The study carried out includes the experimental study of various arrangements of the perforated plates and the bending test of pieces with spans of 8.88 m in both short-term and long-term configurations. The effect of a unbonded but locked tendon was studied in short- and long-term tests. In some of the short-term bending tests, a tensioning system has been used to improve the behaviour of the floor in service.

2 SHORT-TERM TESTS ON SHEAR CONNECTORS

The specimens consisted of a fibre-reinforced concrete slab with 80 mm thickness and a rib of glued laminated timber with a cross-section of 120x160 mm and GL28h strength class. The Fibre-reinforced concrete, made from CEM II/B-M (V-L) 32.5N cement with a content of 350 kg/m3 and polyolefin macro-fibres Sika-Fiber48, was used to minimize the effect of shrinkage. The shear connectors were made from hot-galvanised 5-mm

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thickness steel plates, with 10-mm staggered holes every 15 mm. Two 8-mm thickness slots were made on the upper face of the wooden ribs to place the perforated galvanized steel plates glued using a two-component epoxy adhesive with a thickness of 1.5 mm.
Two batches of tests were carried out to evaluate different shapes of the connectors. In the first batch tests were carried out on 3 types of connectors: only with the plate, including transverse reinforced bars passing through the holes in the plates, including longitudinally welded reinforced bars on the plates (Figure 1).

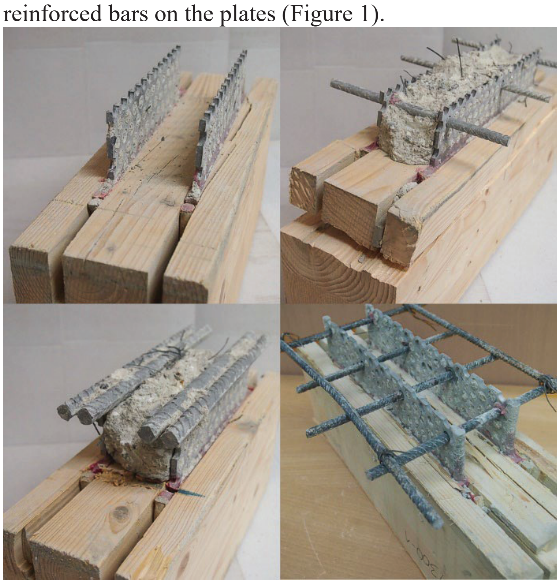

Figure 1: *Shear test specimens open after failure to check gluelines.*

Two connector lengths, 180 and 300 mm, were studied for each type of specimen. 18 specimens of 6 different types were subjected to failure in a shear test, obtaining the failure values, failure modes, and relative slip between the wood and the concrete [3]. All the specimens failed mainly by shear in timber (Figure 1).
Failure load ranged from 157.1 to 195.1 kN (average 179.1 kN) for 180-mm specimens and from 232.9 to 326.4 kN (average 260.4 kN) for 300-mm specimens. Specimens with longitudinally welded bars obtained, in average, the better results, although the difference between the three types was small. The load-slip behaviour of the connections lead to slip moduli between 707.5 and 1164.3 kN/mm for 180-mm specimens and between 2301.9 and 4298.1 kN/mm for 300-mm specimens.
In the second batch 6 new types of specimens were tested including vertically welded reinforced bars at the ends of the plates that were inserted a longer length in pre-drilled holes in the timber member (Figure 2). The types tested included three different configurations of bars and plates, varying the height of the plates and the anchorage length of the bars. For each of these configurations, two connector lengths, 180 and 300 mm, were tested again [4].

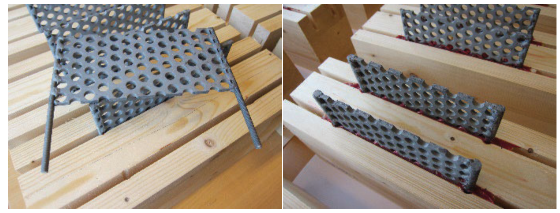

Figure 2: *Gluing of shear test specimens with plate+rods arrangement.*

All the specimens failed mainly by shear in timber again. Those made with plates+rods showed a greater ductility because of the anchorage of the rods although there was not significant difference for ultimate failure load when comparing specimens with the same length of the connector. In this case, failure load ranged from 175.1 to 213.6 kN (average 194.7 kN) for 180-mm specimens and from 213.6 to 318.6 kN (average 265.6 kN) for 300-mm long specimens. Slip moduli ranged between 959.8 and 1259.4 for 180-mm specimens and between 2103.0 and 3407.3 kN/mm for 300-mm specimens.
From these results, a simple predictive analytical model was proposed to determine the ultimate capacity of the shear connection [5].
Given that all the connectors tested showed high resistance and k_{ser} values, the simplest system was used for the elaboration of specimens for the full-scale beam tests.

3 SHORT-TERM TESTS ON TCC BEAMS

Eight beams with a total length of 8.88 meters and a T-section were made. The upper concrete slab had a dimension of 720x60 mm and the lower wooden rib had a section of 180x210 mm. An additional 30-mm thickness formwork board makes up a total height for the beam of 300 mm and a slenderness L/30. The characteristics of the materials were similar to those used in the previous shear tests. A 30x60 mm longitudinal groove was made on the underside of the wooden rib to house a tendon in some of the test configurations (Figure 3).
Four of the eight beams housed a tendon of 20 mm in diameter and the other four of 26.5 mm in diameter. Tendons were Y1100H Steel Dywidag threaded bars (with ultimate tensile strength 1100 MPa, and yield strength 900 MPa). The tendons were not adhered to the timber joists. Figure 3 shows the temporary blocks used solely for the purpose of positioning the tendons within the grooves.
Perforated S235 grade galvanized steel plates with dimensions 300x105x5 mm were used as shear connection between timber and concrete The pattern of the holes in the plates was similar to that of the previous shear push-out tests. A two-component epoxy adhesive was used to glue the plates to the timber joist. The perforated plates were arranged with variable spacing along the beam, as can be seen in figure 4. The minimum distance between them was 450 mm and the maximum 650 mm. In the area of the centre of the span, section

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without shear, a distance between connectors of 1390 mm was disposed.

Figure 3: *Geometrical configuration of 8.88-m long beams.*

Four different test configurations were carried out, with the aim of evaluating the difference in deflections between them: test of pieces without tendons (test 1); test with non-adherent tendons and no pretensioning force, but with the tendons affixed at the ends of the beams (test 2); test with non-adherent tendons and a pretensioning force of 30 kN (test 3) and test with non-adherent tendons fixed to a self-tensioning system patented by the authors (test 4) [6].

Figure 4: *Arrangement of the beams previous to the pouring of the concrete.*

For tests 2 to 4, four results were available for each of the tendon diameters studied. Tendon washer-type load cells were place at both end of tendons in tests 2 to 4 to record the loads reached in the tendons.

Figure 5: *Short-term test on 8.88-meter span beams.*

Beams were tested under a four-point bending test. Two load hypotheses were studied in each type of test: a quasi-permanent load hypothesis (with a value of 4.0 kN/m^2) and another full load hypothesis (with a value of 7.5 kN/m^2). For all tests the relationship between load and displacement at the midpoint was recorded. Additionally, a measurement was made in the central area of the beam corresponding to the relative deformation of two points separated 2 meters from each other, with the aim of determining the local stiffness (EI_{local}) in the area where there is only flexion since the shear is null. For this, a rigid aluminum bar was arranged as shown in Figure 5. Finally, six of the eight beams were tested until failure, reserving the remaining two for long-term tests. All the beams failed due to tension in the lower fibre of the wooden rib (Figure 6). No significant damage or cracks were observed in the concrete slab.

Figure 6: *Tensile failure in the lower fibre of the timber joist.*

Discreetly placed glued plate connections (at variable distances along the beams) resulted in a composite action close to 99% in all cases. Additionally, the use of non-adherent tendons allows reducing the deflection of the beams. The reduction in deflection was around 10% in the case of non-prestressed tendons and reached values close to 30% when the self-tensioning system was used [7].

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In any case, the timber-concrete composite section proved to be very efficient in bending even in non-tensioned solutions, with deflections lower than 1/400 of the span for a ratio span/height of 30.

Finally, thebeams were tested until failure keeping the tendons locked.

Failure loads ranged from 69.3 to 122.2 kN, with no significant differences between the mean failure load of the beams with 20 or 26 mm diameter tendons.

The analysis of the two proposed service load hypotheses showed practically linear load-displacement curves in all cases.

4 LONG-TERM BEHAVIOUR OF TCC BEAMS

Two 8.88m-spam beams were subjected to long-term loads for more than two years (823 days), from October 2019 to December 2021, in an interior and uncontrolled environment (in Lugo, Spain). The test protocol during that period included the measurement of the environmental conditions (relative humidity and temperature), the humidity content of the beam components and the deflection at different moments of the process. The environmental conditions were continuously recorded by a thermo-hygrometer model LASCAR EL-USB-2 that collects data at half-hourly intervals. Periodically, the moisture content of the wood was recorded with a moisture meter model GANN Hydromette RTU 600.

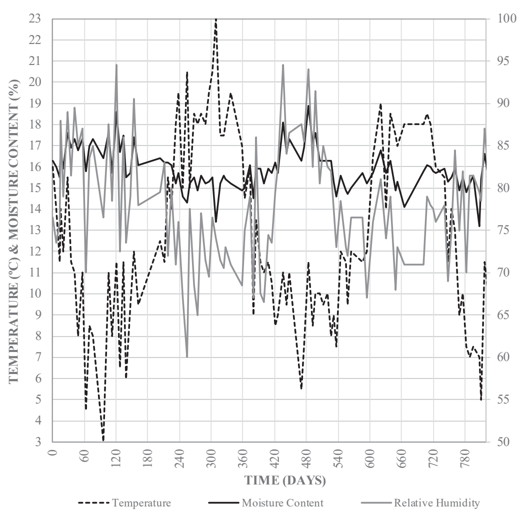

Figure 7: Variation of the environmental conditions and the moisture content of the wood during the test period.

The ambient temperature varied during this period between a minimum of 3.0 and a maximum of 23.0º Celsius (average value 13.5ºC), and the relative humidity between 60.0 and 94.5% (average value 76.5%). The average moisture content of the wood (determined from the measurement made at various points on the beams) ranged from a minimum of 13.2% to a maximum of 18.9%, with an average value of 16.5% (Figure 7).

Figure 8: Arrangement of the long-term test.

The beams were arranged as simply-supported. One of them had the Dywidag tendon affixed at the ends, without any pre-tensioning force. Washer-type load cells were arranged to record the variation in tendon load over time. The other beam was completely free, the cross-section only with timber and concrete, without any tendon blockage.

The self-weight of the beams was evaluated in 1.44 kN/m. The imposed load was applied by boxes filled with water which represents a uniform imposed load of 1.60 kN/m and an equivalent load of 5.0 kN/m^2, including self-weight (Figure 8).

Figure 8 shows the deflection values recorded throughout the entire period. The instantaneous deflection values, 13 and 11.3 mm for the beam without and with tendon, respectively, correspond only to the imposed load. The deflection values throughout the period respond to the total load (including self-weight), and respond both the deflection due to creep and the deflection derived from environmental variations. Considering the ratio between the self-weight and the imposed load, the instantaneous deflection corresponding to the total load would be about 24.7 and 21.5 mm, respectively. Figure 8 also represents the load value recorded in the blocked tendon and the moisture content of the wood. The relationship between deflection variations, tendon load, and moisture variation are evident.

Figure 9 shows the difference in deformation of the beam with a blocked tendon and the beam with an unblocked tendon. The instantaneous deflection of the beam with a locked tendon is 86% of that of the one without the locked tendon. This proportion remains substantially constant throughout the process, varying between 79 and 95%. The difference between both deformations decreases as time progresses, until reaching that 95% towards the end of the period.

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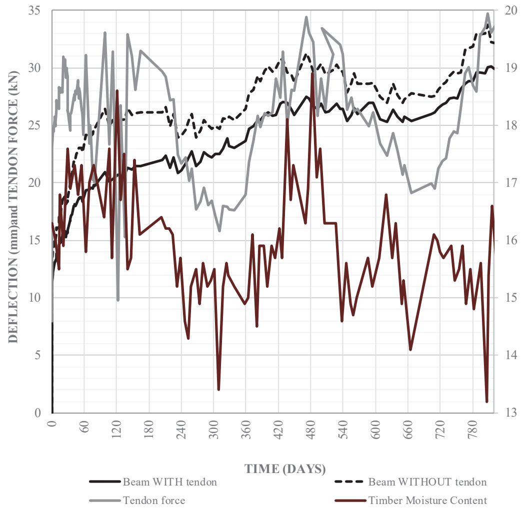

Figure 9: Evolution of the total deflection during the long-term test along with the force in the tendon and the timber moisture content.

In Figure 10 only the creep deformation is represented, also related to the moisture content and the force in the tendon. In this figure, it is more evident how the creep deformation of the two beams tends to equalize over time. Figures 9 and 10 also shows the relationship between the increase in deformation and the increase in the moisture content of the wood. The efficiency of the tendon is greater when the moisture content of the wood increases, since in the Figures 8 and 9 it can be seen that in these cases the difference between the creep deflections of the two beams also increases.

The creep deformation after two years is somewhat lighter than the instantaneous deformation (the total deformation is slightly below than twice the instantaneous).

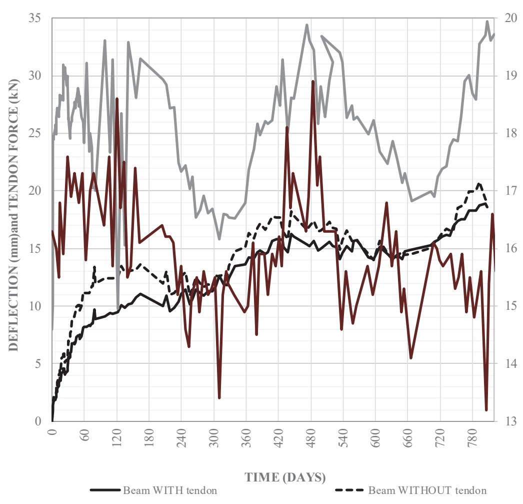

Figure 10: Evolution of the creep deflection during the long-term test along with the force in the tendon and the timber moisture content.

5 SHORT-TERM TESTS OVER PRELOADED BEAMS

After the long-term test, the beams were unloaded and remained supported along their entire length for about 60 days. Then, the beams were tested again in short-term. Two types of test were carried out: type A and type B.

In test type A the beams were arranged as simply-supported with two cantilevers, with a span between supports of 6.90 meters. In test type B the beams were arranged as simply-supported with a span equal to 8.7 m. A schematic representation of the load arrangement can be seen in Figure 11.

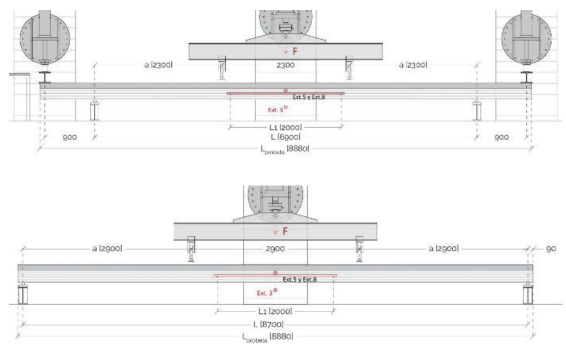

Figure 11: Type A and type B arrangements for the short-term tests made after long-term tests.

In test type A, the beams were loaded three times. The first load is carried out with the tendon unlocked and without any restriction at the ends of the cantilevers. The beams are loaded to a maximum of 7.5 kN and the deflection at the midpoint is recorded. Subsequently, the ends of the beams were blocked at a distance of 0.9 meters from the supports, generating a reaction to the upward vertical displacement of the ends. To achieve this blocking, loads were applied by means of two load cells located at the ends of the cantilevers that remained in a fixed position after applying a load of 1 kN. In this situation, the beam was again loaded to a maximum of 7.5 kN in two different situations, the first without locking the tendons and the second locking the tendons by means of end locking nuts. In both cases, the descent of the midpoint and the load at the ends were recorded (Figure 12).

Given that a very linear load-deflection behaviour had been observed in the previous tests, low-magnitude loads were applied in this case to measure the bending stiffness in order not to damage the pieces in the different configurations tested. The load was applied by constant speed according to EN 408:2010+A1. A hydraulic testing machine equipped with a 600 kN load cell was used. In the same way as in the previous short-duration tests, a deflection measurement was carried out in the central area of the beam of the relative deformation of two points separated 2 meters from each other, with the aim of determining the local stiffness (EI_{local}), using a rigid aluminium bar as indicated in Figure 11.

Figure 12: Type A test arrangement, with cantilevers at the end of the beam.

In the type B test, the beams are arranged as simply supported with a span of 8.7 meters. In this case, a first load phase is carried out with a maximum value of 9.8 kN and the deformation of the pieces with the tendons not locked and with the tendons locked is recorded. Finally, the pieces are tested until failure with the tendons not locked.

The type A test allows verifying the ability of the fibre-reinforced concrete slab to withstand small-magnitude tensions without cracking, which would allow the design of continuous elements over several spans.

Table 1 shows the mean values of breaking load and flexural stiffness (EI) for the tests carried out before and after long-term tests.

Table 1: *Failure loads of simply-supported beams.*

Tests	Locked tendon	Average failure load [kN]	Average bending stiffness EI [kN·m]
First short-term tests	No	-	11660
	Yes	93.7	12680
Second short-term tests (after long-term load)	No	68.9	10660
	Yes	-	12580

The increase in the global bending stiffness of the beams with the tendon locked with respect to the tendon without locking is between 9% and 32%. The difference between the beams that have been subjected to long-term tests and those that have not undergone long-term tests is bigger for beams without tendon. The result is practically the same in the case of tests performed with the tendon locked. To this regard, it is necessary to remember that in the case of beams from long-term tests, only two specimens are available, so conclusions must be taken with caution.

The failure load is not comparable between the two tests, since in one case the beams were tested until failure with the tendon locked and in the other without it. But, considering the little difference in behaviour observed in terms of stiffness, it could be concluded that the arrangement of a locked tendon (without pre-stressing) could increase the bending strength of the beam by up to 27%.

6 CONCLUSIONS

Discrete perforated glued steel plates constitute a very effective solution for the development of timber-concrete-composite systems.

Shear connection with perforated steel plates between timber and concrete can be improved by using additional steel bars. However, the improvement in performance is slight, so the use of additional bars must be justified by other design criteria.

Full scale short-term tests were made on TCC beams with 8.88-meter span. The use of perforated plates as shear connector showing a composite action close to 100%.

The use of non-adherent tendons allows to reduce de deflections in beams between 10 and 30%, depending on the pre-tensioning force of the self-tensioning device used.

The use of non-prestressed tendons can also reduce long-term deflections, especially when the moisture content of the wood is high.

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