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TESTS ON SHEAR CONNECTIONS IN PREFABRICATED COMPOSITE CROSS-LAMINATED-TIMBER AND CONCRETE ELEMENTS

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ABSTRACT: The authors report on an experimental test program on different shear connectors to be used in floor elements where prefabricated concrete beams are to be connected on top of cross-laminated timber (CLT) panels. The objective is to evaluate the level of composite action that can be achieved and the increase of strength and stiffness when combining effectively these structural components in a floor element. Shear tests were performed on 12 specimens in total. The shear connector consists of a steel plate as the common part to connect to both the concrete beam and the CLT panel. To the steel plate, studs or a metal mesh were welded and casted in the concrete beam to form a prefabricated unit. The CLT panel was then attached to the steel plate by screws. The main slip occurred between the CLT panel and the steel plate. The test results show a high ultimate load capacity for all types of shear connectors and a ductile type of behaviour for most of them. The test results for the shear connectors were compared with the capacity obtained from analytical models. Also, the strength and stiffness of partially composite floor elements with these shear connectors were analysed. It was found that from a structural point of view these shear connections are suitable to be used to form floor elements with a high level of composite action.

KEYWORDS: CLT, prefabricated concrete, shear test, shear connectors, slip modulus, coach screws

1 INTRODUCTION

The company Stora Enso is developing a new building system based on composite structures with prefabricated elements, including the use of cross-laminated timber (CLT) panels. In that process, the company wanted to evaluate the contribution of a CLT panel added underneath to the main load-carrying concrete beams with respect to strength and stiffness of the total composite floor structure. This floor element is not a conventional composite concrete-timber structure, but rather a combined structure for the purpose of making use of and benefitting from the materials and components present in the total floor structure by connecting them together. In addition, by connecting several concrete beams on a CLT panel, a prefabricated floor element ready for assembly on site is obtained. One focus then is the development of an efficient shear connection between the prefabricated concrete beams and the CLT panel. Luleå University of Technology (LTU) was given the assignment to evaluate the above mentioned composite system and to study different shear connection systems designed for connecting together components prefabricated separately. The aim of the paper is to present the outcomes of the test program that has been performed at LTU, consisting in shear tests on two different types of connectors for prefabricated concrete-CLT elements.

2 TIMBER-CONCRETE COMPOSITE STRUCUTRES

2.1 BACKGROUND

Extensive research has been carried for several decades for assessing the behavior of timber concrete composite structures, with a strong focus on shear connectors since it is a key parameter governing the overall behavior and efficiency of composite structures. However, most of the systems used today work with “wet connections” where concrete is poured on timber elements preinstalled with shear connectors. The drawbacks of the traditional timber-concrete structures are well known, such as the time needed for concrete to cure, the low stiffness of the structure and the shrinkage that takes place during the curing time, as well as the problems related to the introduction of wet components in the timber construction process [1]. The interests for developing prefabricated timber concrete structures are growing since an off-site production of elements could reduce the influence of the above mentioned shortcomings [1], [2]. Some work has been done on prefabricated timber-concrete composite floors elements in the last years [1], [3]. However, the solutions developed are not applicable for the present floor configuration where concrete beams are to be connected on top of a CLT panel. New types of shear connectors were thus developed for this floor structure.

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2.2 REQUIREMENTS FOR THE FLOOR ELEMENTS PRESENTED

The floor system for which the connectors are developed is not conventional since concrete is not used in form of a plate but in form of beams elements. Concrete beams, attached on top of the CLT panel allow for some technical installations to be placed between the beams. In this floor configuration, the CLT panel forms a platform for mounting and connecting the installations from above during the prefabrication process at the factory. An objective for the floor elements is to allow for a high level of prefabrication in order to ensure a quick assembly on site. For manufacturing these floor elements the concrete beams should be prefabricated due to the following aspects:

- Avoid a wet assembly process and avoid deflections induced by concrete shrinkage.
- Reduce the space required for storage of the elements during curing time compared with concrete beams casted on CLT.
- Be able to produce the CLT and the concrete at different locations and assemble them at the last possible moment in the prefabrication process.
- Be able to reduce the production time both on site and at the factory.
- Bring about a demountable system.

The requirements for the design of the shear connectors were to enable a dry assembly process, and to be able to mount the connectors from the top side of the CLT panel in order to have them protected in case of fire.

3 EXPERIMENTS - SHEAR TESTS

The test program consisted of asymmetric shear tests on CLT-steel-concrete blocks (see Figure 1) with five different connection configurations (with different number of screws and screw lengths) for two series differing by the steel-concrete shear connector used, series S (studs) and series M (expanded metal mesh). For each series, the connection was tested with 4 and 8 screws. Totally, twelve specimens were tested.

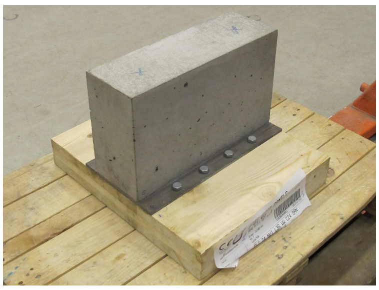


Figure 1: *Picture of a test specimen*

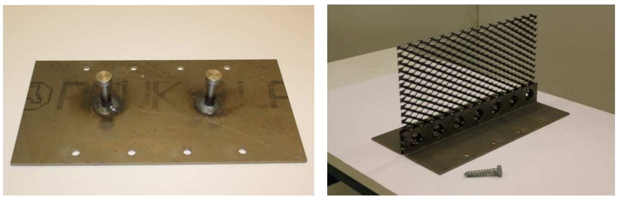
3.1 MATERIALS

3.1.1 CLT

The CLT panels, provided by Stora Enso, were made with 3 layers of timber of strength class C24 according to EN 338 [4]. The grain direction of the outer layers is oriented in the direction of the concrete beams. All the tests were performed with panel of nominal thickness 74 mm (lamella structure in mm: 27.5/19/27.5), except one in which the CLT had a nominal thickness of 103 mm (lamella structure in mm: 42/19/42). Density and moisture content were measured on all CLT specimens from timber located near the failure region, giving $\boldsymbol{\rho_m = 448\ kg/m^3}$ and $\boldsymbol{\rho_k = 396\ kg/m^3}$, and average moisture content of **11.6 %** at testing.

3.1.2 Steel shear connectors

Two different types of connectors were chosen for the tests, see Figure 2. The first type of connector, named *Studs* in this paper, consisted of a 6 mm thick steel plate with two welded headed studs. The second type, noted *Mesh* in this paper, was made using an expanded metal mesh welded in between two folded steel plates of 4 mm thickness with large holes, similar to the so-called "perfobond" shear connector. Both steel shear connectors were made out of mild steel S355. Pre-drilling of the steel plates for the screws can be made vertically only, unless some specific drilling tools are used. Thus, vertical holes of 12.0 mm diameter were made in the steel plates for the screwed connection to the CLT with 80 mm spacing. Geometrical details for the shear connectors are presented in Figure 4.



(a) (b)

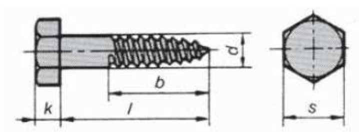
Figure 2: *Pictures of the shear connectors (a) Studs, (b) Mesh*

3.1.3 Concrete

Concrete was casted onto the steel shear connectors in a factory to form the concrete-steel prefabricated units. The same batch of concrete was used for all the test specimens and the nominal strength class was C25/30. 15 cubic samples were made for strength testing, giving a compressive strength $f_{cm,cube}$ = 40,81 N/mm^2, at 28 days, in accordance with the strength class C25/30. Two rebars of 8 mm of diameter were placed in each prefabricated unit as the minimum bottom reinforcements in all concrete blocks.

3.1.4 Mechanical fasteners

Due to the limited depth of the CLT panel, coach screws with rather large diameter were chosen in order to provide the connection with sufficient strength and stiffness. The coach screws used to connect the concrete beam blocks to the CLT panels for tests specimens 1 to 11 were timber screws DIN 571 - 4.6/FZV - 12x65 with dimensions according to Figure 3.



k = 8 mm, l = 65 mm, d = 12 mm, s = 19 mm, b = 50 mm

Figure 3: *Geometrical description of the screws*

For the test specimen 12, longer screws were used, with geometrical parameters k = 8 mm, l = 100 mm, d = 12 mm, s = 19 mm, b = 60 mm, according to Figure 3.

3.2 TEST SPECIMENS

CLT elements, 500 mm wide and 420 mm long, were all predrilled with 8 holes according to EC5 recommendations. The lead hole for the threaded portion of the screws had a diameter of 8.5mm and the lead hole for the shank of the screw had a diameter of 11.5mm since it was the diameter actually measured on the shank of the screws used. The steel-concrete blocks were mounted with 40 kN·m torque applied on each screw. Geometrical details for the S-series, with studs, and M-series, with mesh shear connectors are presented in Figure 4. Complementary specifications for the test specimens are given in Table 1.

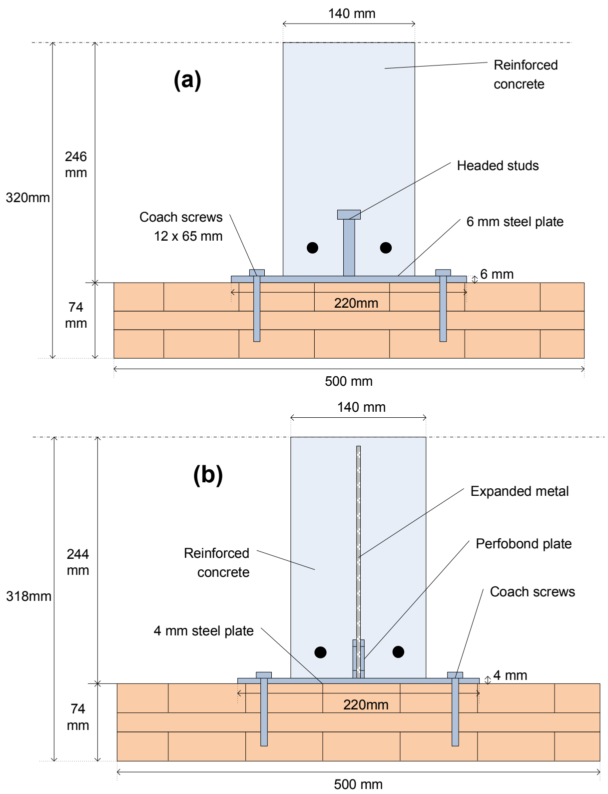


Figure 4: *Test specimens, (a) S-series, (b) M-series*

Table 1: *Description of the test series*

Test series	Tests	Shear connector / steel plate thickness	Screw spacing in loading direction / edge distance (mm)	CLT panel thickness (mm)
S-8S-65	T1, T2, T3	Studs /6mm	80 / 90	74
S-4S-65	T4, T5, T6	Studs /6mm	240 / 90	74
M-8S-65	T7, T8, T9	Mesh /4mm	80 / 90	74
M-4S-65	T10, T11	Mesh /4mm	240 / 90	74
M-8S-100	T12	Mesh /4mm	80 / 90	103

3.3 DESIGN OF THE SHEAR CONNECTIONS

The load carrying capacity and the slip modulus of the steel-timber connection were calculated according to Eurocode 5 [5] and the European technical approval for CLT from the manufacturer [6] and are given in Table 2.

3.4 TEST SETUP

The shear tests were performed on asymmetric elements in a hydraulic press, Dartek Server hydraulic load frame of capacity ± 600 kN, with two data acquisition systems HPM 8 Channels Spider 8. Height LVDTs (Linear Variable Differential Transformer) were used for each test. The specimens were supported on the end face of the CLT panel, with the concrete block hanging by the side of the CLT panel by the shear connection (Figure 5). Due to size limitations of the test setup, it was not possible to install a roller support at the top of the concrete to prevent the rotation of the test specimen. Instead, Teflon material was taped onto the vertical steel frame support and an oiled steel plate was placed between the concrete block and the steel frame to minimise friction. The top surface of the concrete block was grinded, in order to provide an even surface for the contact with this steel plate.

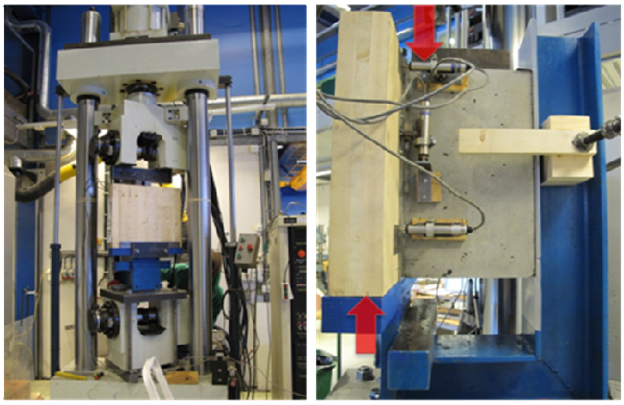


Figure 5: *Pictures of the test setup for the shear test*

3.5 SLIP MEASUREMENTS

Slip measurements were made symmetrically on both sides of the specimen. The results presented are average values from both sides. The longitudinal slip was measured between the CLT and the concrete block and between the steel plate and the concrete in order to verify that the steel-concrete slip is negligible compared to the slip at the steel-timber interface as it is assumed in the calculation model. The separation between the concrete block and the CLT panel (Figure 6) was measured as a control of the test setup and are not presented explicitly in this paper.

Displacement-controlled tests were performed with loading conditions according to ISO Standard 6891 [7], except for specimen T1, which was a load-controlled test. Only the ultimate load for this latter test is used as the test results. The rate of the different tests was adjusted for each test according to the estimated load and the results from previous tests. The tests were stopped if 15 mm slip was reached at the steel-timber interface. The

ultimate load capacity is evaluated as the absolute maximum value occurring during the test within 15 mm of slip.

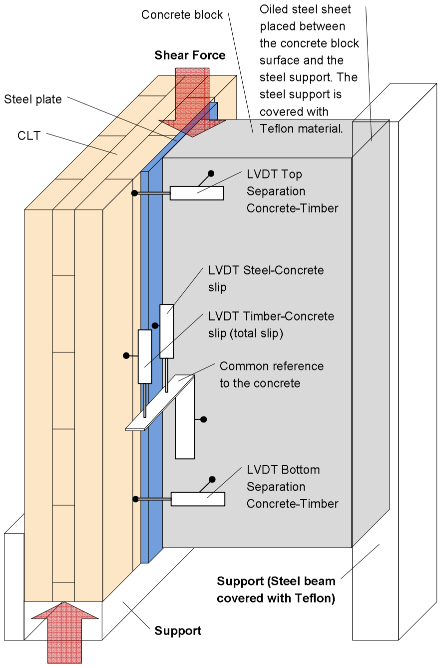


Figure 6: *Location of the LVDTs on the test specimens*

The adjustment of the estimated maximum capacity F_{est} between the different tests in accordance with ISO 6891, affects the estimation of the slip modulus as pointed out in [8]. In this paper, the slip modulus is evaluated based on the observed maximum value for each test and not on the estimated value. The slip modulus for the serviceability limit state is evaluated as the slope of the curve up to the level of 40 % of the maximum load. Due to the unusual character of the curve in the beginning of the load-slip relationship, the suggested lower level of 10 % of the maximum load usually used for the evaluation is disregarded in this paper, see below.

4 RESULTS

Three different values for the slip between the different components of the specimen are used:

- The **total slip** between the concrete and the CLT panel (2 LVDTs).
- The **steel-concrete slip** between the steel plate and the concrete beam (2 LVDTs).
- The **steel-timber slip** between the steel plate and the CLT panel (obtained as the difference between the two former values).

4.1 TEST RESULTS FOR ALL SPECIMENS

The load-displacement curves for the total slip are presented in Figure 7 for all five test series according to Table 1.

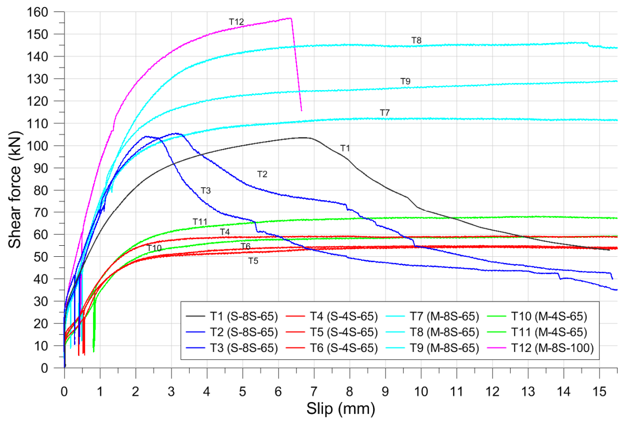


Figure 7: *Load slip curves for all shear tests - Total slip (timber-concrete)*

The first group of tests (S-8S-65: T1-T3) with studs in the concrete and 8 screws in the panel, showed semi-brittle behaviour; the failures occurred in the concrete after yielding of the studs in the concrete at an applied load of about 105 kN (Figure 8). After the peak load, the curves were gradually softening due to increased cracking of the concrete.

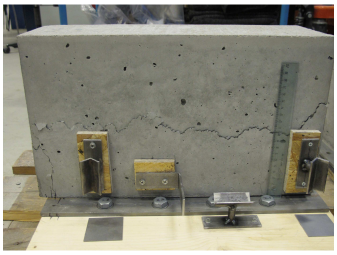


Figure 8: *Picture of the failure in the concrete (Test series S-8S-65)*

Test specimens T4-T6 (S-4S-65), with 4 screws in the panel, exhibited a very ductile type of behaviour, an almost perfect plastic behaviour with an extended yield plateau. The failure occurred by combined embedment failure in the CLT panel and yielding of the screws (Figure 9). The average ultimate load of 56.3 kN was about half of that of the first series with twice as many screws.

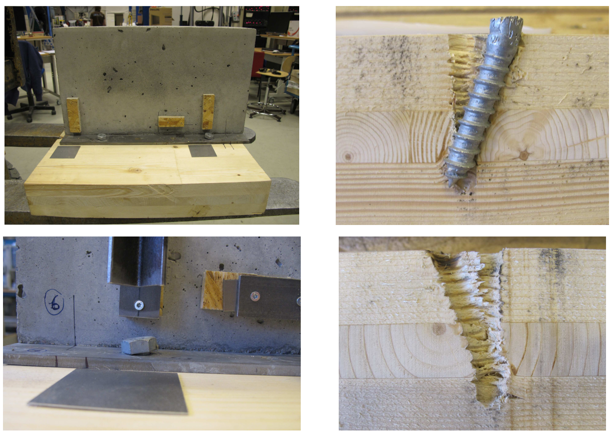


Figure 9: *Pictures of the failure in the timber (Test series S-4S-65)*

The third series (M-8S-65: T7–T9) with a mesh in the concrete and 8 screws in the panel, behaved almost the same as the second series, however, with an average ultimate load of 129.2 kN, about twice as high (Figure 10).

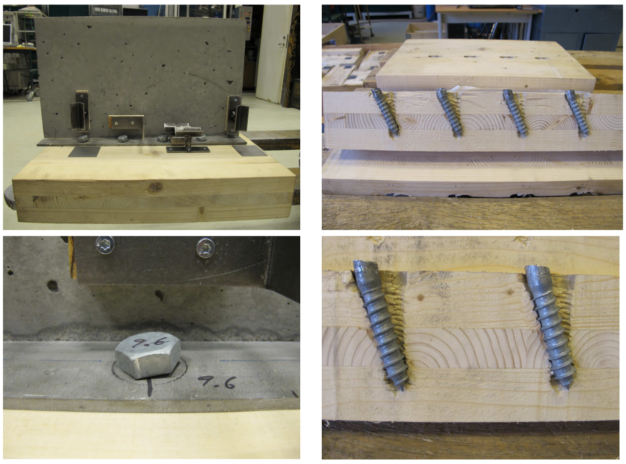


Figure 10: *Pictures of the failure in the timber (Test series M-8S-65)*

Tests T10 and T11 (M-4S-65) with a mesh in the concrete and 4 screws in the panel, had both the same characteristics and level of yielding as the second series with an average ultimate load of 63.7 kN (Figure 11).

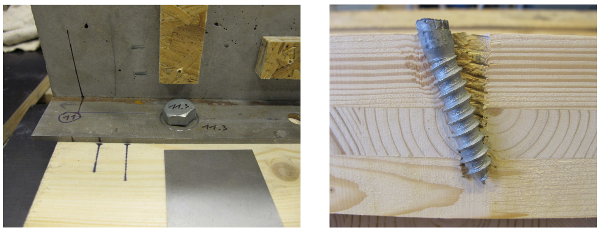


Figure 11: *Pictures of the failure in the timber (Test series M-4S-65)*

The final series (M-8S-100: T12) with a mesh in the concrete and 8 screws in the panel, but with thicker panel and longer screws compared to other series, had a brittle failure after some yielding; the concrete block was split apart in line with the expanded metal mesh at 157 kN applied shear force (Figure 12).

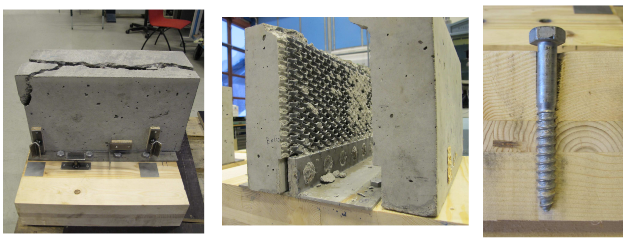


Figure 12: *Pictures of the failure in the concrete (Test series M-8S-100)*

As is evident from Figure 7, the initial stiffness is very high, probably due to friction between the steel plate and the CLT-panel (the screws were tightened rather hard).

A close look at the measured data shows that even some negative slip occurred. The LVDTs measuring the separation at the top and bottom of the test specimen (Figure 6) actually showed that the concrete block was slightly rotating during the test, due to some imperfections in the test setup. This affected the measurements of the total and the steel-concrete slips. The negative slip measured at the steel-concrete interface never exceeded -0.05 mm for all tests and reached a maximum for applied shear forces between 20 kN and 50 kN. It was therefore not possible to evaluate quantitatively the steel-concrete slip modulus.

However, for the steel-timber slip values, which are obtained as the difference between the total and the steel-concrete slips, this negative absolute rotation is eliminated in the subtraction. Detailed load-slip curves for this steel-timber interface are shown in Figure 13. The high initial stiffness is observable up to 10 kN and 20 kN of applied shear force for connections with 4 and 8 screws, respectively. These load values are near or below the serviceability load levels. It means that it is reasonable to exclude this high initial stiffness branch when evaluating the slip modulus. In this paper, the slip modulus will be evaluated using the 40 % level of the maximum shear force as the upper limit and the origin as the lower limit.

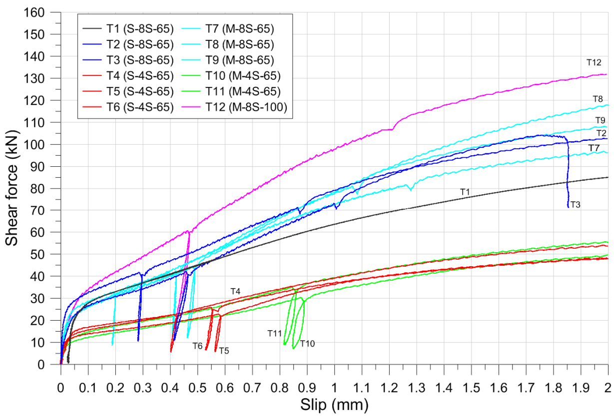


Figure 13: *Detailed load-slip curves for all shear tests – slip at the steel-timber interface up to 2 mm.*

After the initial stiffness area, there is a drastic change in the stiffness according to Figure 13. At a later stage and, especially, after un- and reloading, a slight increase in the stiffness can be observed. When the deformations become larger the screw head will be more and more clamped and, at the same time, press the steel plate against the CLT panel. Also, the deformation rate was changed during the course of the test. The small drops in the shear force observed for shear forces above 70 kN in Figure 13 is due to this change of rate.

The measured mean values of the ultimate load capacity, $F_{v,R,mean}$, and the mean slip modulus for serviceability limit state, $K_{0.4,mean}$, are presented in Table 2 for each type of shear connection tested. Just for the comparison, the corresponding characteristic values obtained from the Eurocode 5 are also given in the table (the CLT panel

measured mean density and characteristic density were used in the calculation for the slip modulus and the ultimate strength, respectively)

Table 2: *Load-carrying capacity and slip modulus of the shear connections. Ultimate load and slip modulus per screw is given in parentheses) – EC5 estimated values and test results (mean values)*

	Estimated values according to EC5		Tests results (mean values)		
Name	$F_{v.Rk}$ (kN)	Slip modulus (steel/timber) K_{ser} (kN/mm)	Failure mode	$F_{v.Rmean}$ (kN)	Slip modulus (steel/timber) $K_{0.4,mean}$ (kN/mm)
S-8S-65	38.0 (4.7)	79.2 (9.9)	Semi-brittle	104.4 (13.1)	112.9 (14.1)
S-4S-65	25.8 (6.4)	39.6 (9.9)	Ductile	56.3 (14.1)	44.1 (11.0)
M-8S-65	39.3 (4.9)	79.2 (9.9)	Ductile	129.2 (16.2)	93.0 (11.6)
M-4S-65	26.6 (6.7)	39.6 (9.9)	Ductile	63.7 (15.9)	40.5 (10.1)
M-8s-100	61.8 (7.7)	79.2 (9.9)	Brittle	157.1 (19.6)	126.5 (15.8)

(Estimations according to EC5 are made by using d_{eff} to estimate the embedment strength and d for K_{ser})

The limited number of tests does not allow us to estimate the characteristic load-carrying capacity based on the test results and to compare it with the predicted ones. However, it can be observed that the mean values for the maximum load exceeded 2.2 to 3.3 times the characteristic values provided by EC5 for the steel-timber connection. An evaluation of the capacity according to an analytical model is proposed in section 5.2.

It is important to note that when a brittle failure occurred in the concrete (related to the concrete-steel connection), the failure load should not be compared with the predicted failure load given by EC5, which is based on a failure mode for the steel-timber connection. In addition, the slip moduli for the steel-timber connection of connections S-8S-65 and M-8S-100, where brittle failure occurred in the concrete, are not fully relevant because the 40 % level is based on an ultimate load for a different failure mode (the slip moduli for these connectors are about 50 % higher than that predicted by the EC 5). For test series S-4S-65, M-4S-65 and M-8S-65, where ductile failure due the steel-timber connection occurred, the test results for the slip modulus were between 2 – 18 % higher than those predicted by EC 5.

There was no effect observed with respect to multiple fastener in a row, in the tests for the comparable series M-4S-65 and M-8S-65 (2+2 screws with a spacing of 240 mm and 4+4 screws with a spacing of 80 mm, respectively). The EC 5 predicts a reduction of ultimate strength of 26 %.

The test results showed that the slip between concrete and steel can be neglected compared to that between steel and timber. The contribution from the concrete-steel slip to the total slip was about 2 % for connections with 4 screws and at maximum 12 % for connections with 8 screws.

It is obvious from Figure 7 that the design of the shear connections according to test series S-4S-65, M-8S-65 and M-4S-65 are appropriate and have the desired load-slip characteristics. They all behave as predicted by the analytical models discussed below. Therfore, these designs are evaluated in more detail.

4.2 DETAILED RESULTS FOR SHEAR CONNECTORS WITH DUCTILE CHARACTERISTICS

In this section, detailed tests results are given for the connection types S-4S-65, M-4S-65 and M-8S-65. These test series all have ductile characteristics and, therefore, are suitable for practical applications and adaptable for evaluation with respect to the analytical models for the shear connectors according to section 5.2 and for the partially composite beam with these shear connectors according to section 6.1.
The load-slip curves for the steel-timber interface for the 3 test specimens T4–T6 (S-4S-65) are presented in Figure 14.

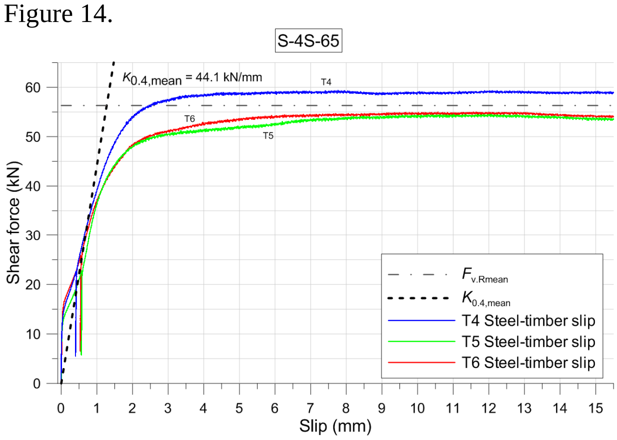


Figure 14: *Load-slip curves for the steel-timber interface – Test series S-4S-65 (T4-T6)*

As given by Figure 14 and Table 2, the average maximum load-carrying capacity for this type of connection is 56.3 kN or 14.1 kN per screw. The average value for the slip modulus is 44.1 kN/mm or 11.0 kN/mm per screw. The test results show that the steel-timber slip governs the overall behaviour of the connection. The contribution from the steel-concrete slip did no exceed 2.5 % of the total slip.

The load-slip curves for the steel-timber interface for the test specimens T10–T11 (M-4S-65) are presented in Figure 15. According to Table 2 and Figure 15, the average load carrying capacity for this type of connection is 63.7 kN (15.9 kN per screw) and the slip modulus is 40.5 kN/mm (10.1 kN/mm per screw). In this test series, the contribution of steel concrete slip did not exceed 1.5 % of the total slip. These results are similar to those of the S-4S-65 series.

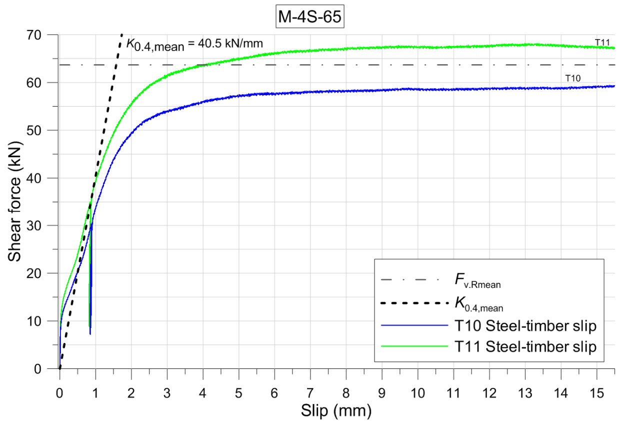


Figure 15: *Load-slip curves for the steel-timber interface – Test series M-4S-65 (T10-T11)*

The load-slip curves for the steel-timber interface for the test specimens T7–T9 (M-8S-65) are presented in Figure 16. According to Table 2 and Figure 16, the average load carrying capacity for this type of connection is 129.2 kN (16.2 kN per screw) and the slip modulus is 93.0 kN/mm (11.6 kN/mm per screw). In this test series, the contribution of steel-concrete slip was larger than in the test series with 4 screws. For tests T7 – T9, the contribution was 6.5 %, 11.5 % and 4 % of the total slip, respectively, at 75 % of the ultimate load. In this paper, only the connections with 4 screws will therefore be considered in a numerical application.

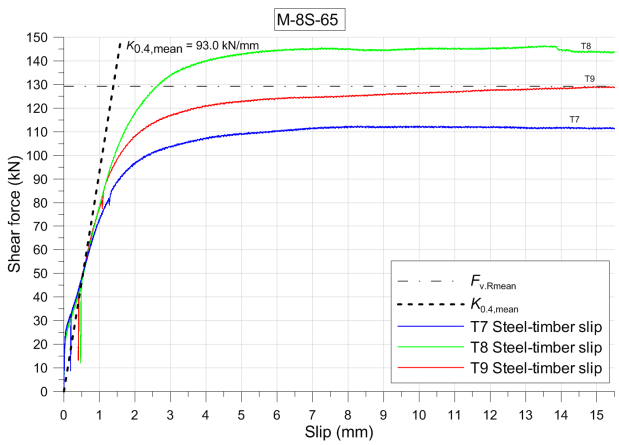


Figure 16: *Load-slip curves for the steel-timber interface – Test series M-8S-65 (T7-T9)*

The slip modulus per screw as well as the steel-timber ultimate load capacity for the connection types S-4S-65, M-4S-65 and M-8S-65 are similar and the difference between the results from the test series with mesh and studs connectors is marginal.

5 ANALYSIS AND EVALUATION

5.1 EVALUATION OF THE SLIP MODULUS

As mentioned above, the slip modulus will be evaluated as the slope of the load-slip curve between the origin of the curve and the point on the curve corresponding to 40 % of the measured ultimate load. It means that the initial high stiffness observed in the load-slip curves will not be taken into account. This will give a conservative secant modulus.

5.2 LOAD-CARRYING CAPACITY OF SHEAR CONNECTION

For the shear connections with a failure in the CLT panel, the load-carrying capacity can be derived using the K. W. Johansen´s plastic model (European Yield Model) [9]. It is based on the assumption that both fasteners and the timber are ideal rigid-plastic materials. The embedment strength of the different layers of the CLT-panel is here assumed to be the same parallel and perpendicular to grain. A distinction is made between the yielding moment in the threaded part of the screw and in the smooth shank of the screw due to their different diameters. Four failure modes are derived according to Figure 17.

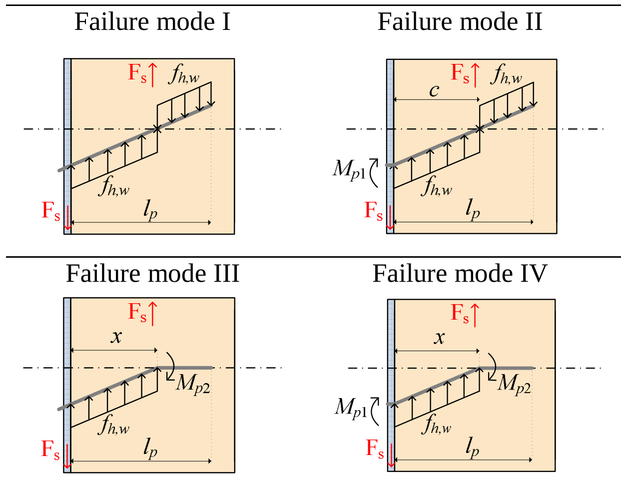


Figure 17: *Failure modes I, II, III and IV*

5.2.1 Failure mode I

According to [9], we have

$$F_s = \left(\sqrt{2}-1\right) f_{h,w} d \cdot l_p \tag{1}$$

where $f_{h,w}$ is the embedment strength of the wood, d the nominal diameter of the screw and l_p the penetration length of the screw.

5.2.2 Failure mode II

Fore equilibrium and moment equilibrium at the steel-timber interface, respectively, give

$$F_s = f_{h,w} d \cdot c - f_{h,w} d\left(l_p - c\right) = 0 \tag{2}$$

$$M_{p1} - f_{h,w} d \frac{c^2}{2} + f_{h,w} d\left(l_p - c\right)\frac{l_p + c}{2} = 0 \tag{3}$$

Solving for c gives

$$c = \frac{l_p}{\sqrt{2}}\sqrt{1+\frac{2M_{p1}}{f_{h,w} d \cdot l_p^{\,2}}} \tag{4}$$

Substituting (5) in (3) gives

$$F_s = f_{h,w} d \cdot l_p\left[\sqrt{2}\sqrt{1+\frac{2M_{p1}}{f_{h,w} d \cdot l_p^{\,2}}}-1\right] \tag{5}$$

5.2.3 Failure mode III

According to [9], we have

$$F_s = \sqrt{2M_{p2} f_{h,w} d} \quad (6)$$

$$x = \frac{F_s}{f_{h,w} d} \quad (7)$$

5.2.4 Failure mode IV

Moment equilibrium at the distance x from the steel-timber interface gives

$$M_{p1} - F_s x + f_{h,w} d \frac{x^2}{2} + M_{p2} = 0 \quad (8)$$

M_{p2} is maximum where the shear force is zero, i.e.

$$x = \frac{F_s}{f_{h,w} d} \quad (9)$$

Which then gives

$$F_s = \sqrt{2(M_{p1} + M_{p2}) f_{h,w} d} \quad (10)$$

5.2.5 Evaluation of the capacity of the shear connectors

The evaluation of the ultimate capacity of the ductile shear connectors according to the 4 failure modes presented above is made according to the geometrical data of the tests specimens. The steel plate thickness differs in the connectors with studs and with mesh. Therefore the penetration length l_p , is 59 mm and 61 mm, for connectors with studs and mesh, respectively. f_u = 400 MPa, d = 12 mm, and d_t = 8.7 mm. According to Eurocode 5, due to the small length of the smooth shank of the 65 mm long screws, an effective diameter d_{ef} is assumed for estimating the screw load carrying capacity with $d_{ef} = 1.1d_t$. The values estimated for the embedment strength and the yield moment are as follows:

$f_{h,w}$ = 41.73 N/mm^2

M_{p1} = 76750 N·mm

M_{p2} = 33260 N·mm

The estimated capacity according to failure modes I, II, III, and IV is given by $F_{s,I}$, $F_{s,II}$, $F_{s,III}$ and $F_{s,IV}$, respectively in Table 3.

Table 3: *Evaluation of the ultimate capacity per screw according to failure modes I, II, III and IV*

	Capacity in kN per screw	
	Connectors with studs	Connectors with mesh
$F_{s,I}$	9.8	10.1
$F_{s,II}$	11.2	11.5
$F_{s,III}$	5.0	
$F_{s,IV}$	9.2	

The observations of the failure from the test series S-4S-65, M-4S-65 and M-8S-65 show that the failure mode was between mode I and mode II with a yielding of the screws in the region of the screw head followed in a later stage by a yielding of the threaded part of the screw just below the smooth shank. This observation was made by comparing the screws of specimens loaded up to 15 mm slip and of some specimens where larger displacements at the steel-timber interface occurred. Considering that the effective number of screw is equal to the actual number of screws, the estimated load carrying capacity according to mode I and mode II obtained for studs connectors with 4 screws and mesh connectors with 4 and 8 screws is given in Table 4.

Table 4: *Comparison between test results and estimated load carrying capacity for the shear connectors according to failure modes I and II*

	Ultimate load capacity (kN)		
	S-4S-65	M-4S-65	M-8S-65
Failure mode I	39.0	40.4	80.7
Failure mode II	44.8	45.9	91.8
Test results (mean values)	56.3	63.7	129.2

The difference between the test results and the estimated capacity according to failure mode I and II might be explained by the fact that the friction between the steel and the CLT as well as the withdrawal contribution was neglected in the equations.

6 APPLICATION – COMPOSITE CLT-CONCRETE FLOOR

6.1 ANALYSIS OF COMPOSITE FLOORS WITH STUDS SHEAR CONNECTORS

Composite CLT-concrete floors with shear connectors as discussed above are analysed in this section. The partial interaction introduced by the mechanical shear connectors is taken into account. The purpose is to evaluate the increased performance (strength and stiffness) that is achieved by connecting the prefabricated concrete beam to the CLT-panel.

Here the analysis will be based on the simplified analysis method for composite beams with interlayer slip [10]. In this somewhat unusual composite structure, the concrete beam will be cracked and the stiffness reduced. The concrete will be assumed to be in stage II, i.e. a linear elastic stress distribution is assumed in the compressed zone. Long-term behaviour of the concrete is not included. The concrete is assumed to have no tensile strength.

Two elements are considered in the analysis. Element 1 is the cracked reinforced concrete beam and Element 2 is the CLT panel. The CLT panel has a symmetrical layup and is considered as fully composite section where the contribution of the core layer is neglected. The area of uncracked concrete in the analysis is determined based on the analysis of the non composite section (NCS), see Figure 18. The position of the centroid of element 1 (equal to the depth of uncracked concrete) and of element 2, respectively, is given by

$$z_{cg,1} = \frac{E_s A_s}{E_c b_1}\left(\sqrt{1+2d\,\frac{E_c b_1}{E_s A_s}}-1\right) \quad (11)$$

$$z_{cg,2} = h_1 + \frac{h_2}{2} \quad (12)$$

The axial stiffness for each element is obtained as

$$EA_1 = E_c b_1 z_{cg,1} + E_s A_s \quad (13)$$

$$EA_2 = E_t b_2 \left(h_{21} + h_{23}\right) \quad (14)$$

The sum and product of the axial stiffness of sub-elements, respectively, are given by

$$EA_0 = EA_1 + EA_2 \quad (15)$$

$$EA_p = EA_1 \cdot EA_2 \quad (16)$$

The bending stiffness of elements 1 and 2 is given by

$$EI_1 = E_c\,\frac{b_1 z_{cg,1}{}^3}{3} + E_s A_s (d - z_{cg,1})^2 \quad (17)$$

$$EI_2 = \frac{E_t b_2}{12}\left(h_2{}^3 - h_{22}{}^3\right) \quad (18)$$

The bending stiffness for the non-composite and fully composite section, respectively, can be expressed as [10]

$$EI_0 = EI_1 + EI_2 \quad (19)$$

$$EI_\infty = EI_0 + \frac{EA_p r^2}{EA_0} \quad (20)$$

where

$$r = r_1 + r_2 = z_{cg,2} - z_{cg,1} \quad (21)$$

Considering the cross section shown in Figure 18, equilibrium requires

$$V = V_1 + V_2 \quad (22)$$

$$M = M_1 + M_2 + N_1 r \quad (23)$$

According to the simplified method for composite beams with interlayer slip [10], the internal actions are given by

$$N_{1,eff} = \left(1 - \frac{EI_0}{EI_{eff}}\right)\frac{M}{r} = N_{2,eff} \quad (24)$$

$$M_{1,eff} = \frac{EI_1}{EI_{eff}} M \quad (25)$$

$$M_{2,eff} = \frac{EI_2}{EI_{eff}} M \quad (26)$$

$$V_{s,eff} = \left(1 - \frac{EI_0}{EI_{eff}}\right)\frac{V}{r} \quad (27)$$

where EI_{eff} denotes the effective bending stiffness for the partially composite section and $V_{s,eff}$ is the interlayer shear force per unit length.
The effective bending stiffness EI_{eff} of the cross section is according to [10] given by

$$EI_{eff} \approx \left[1 + \frac{EI_\infty / EI_0 - 1}{1 + (\mu/\pi)^2(\alpha L)^2}\right]^{-1} EI_\infty \quad (28)$$

where

$$\alpha L = \sqrt{\frac{Kr^2}{EI_0\left(1 - EI_0 / EI_\infty\right)}}\,L \quad (29)$$

K [N/m/m] is the smeared slip modulus of the partial shear connection per unit length, and L the span of the beam element.

Knowing the interlayer slip force, the load on each connector is given by $F_{s,eff} = V_{s,eff} \cdot s$, where s is the spacing between the shear connectors along the composite beam.

Here we use the simplifying assumption that the neutral axis coincides with the centroid of the cross section, i.e. we are neglecting the shift downwards of the neutral axis and, hence, the centroid that is caused by the internal axial force N_1 (Figure 18). This assumption is conservative with respect to the maximum concrete compression stresses, but not with respect to the tensile stresses in the reinforcement. It is a conservative assumption with respect to the stiffness of the concrete beam and, therefore, also for the composite beam.

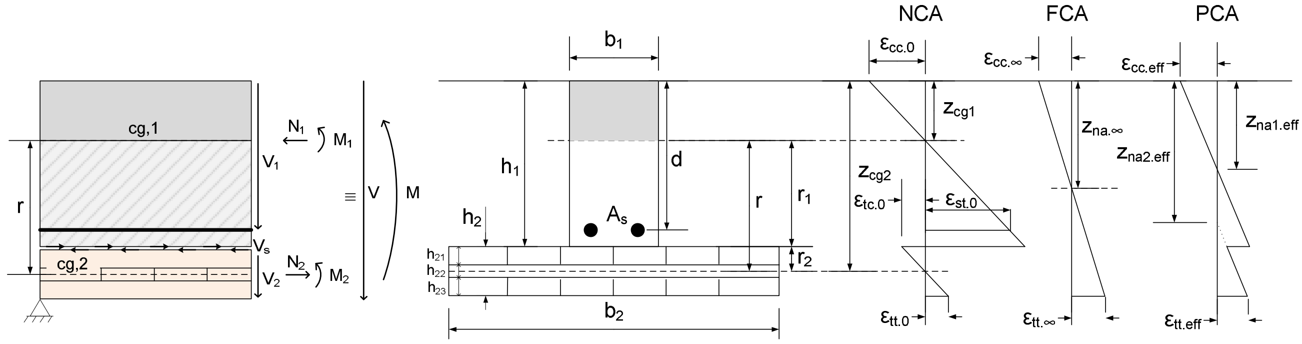


***Figure 18**: Cross section geometry and strain diagrams for non composite action (NCA), full composite action (FCA) and partial composite action (PCA)*

The maximum normal stresses at the top of the concrete beam, in the steel reinforcement and at the bottom of the CLT-panel, respectively, are given by

$$\sigma_{cc,eff} = -\frac{N_{1,eff}E_c}{EA_1} - \frac{M_{1,eff}E_c}{EI_1} z_{cg,1} \tag{30}$$

$$\sigma_{st,eff} = -\frac{N_{1,eff}E_s}{EA_1} + \frac{M_{1,eff}E_s}{EI_1}\left(d - z_{cg,1}\right) \tag{31}$$

$$\sigma_{tt,eff} = \frac{N_{2,eff}E_t}{EA_1} + \frac{M_{2,eff}E_t}{EI_2} r_2 \tag{32}$$

6.2 COMPUTATION

Consider the following geometric and material properties according to the cross section presented in Figure 18.

<u>Element 1:</u>

h_1	250	mm
b_1	140	mm
E_c	31	GPa
A_s	402	mm^2 (2Φ16)
E_s	210	GPa

<u>Element 2:</u>

E_t	11	GPa
h_2	74	mm (layup 27.5/19/27.5)
b_2	500	mm

<u>CLT-concrete shear connection:</u> S-4S-65

$K_{0.4}$	44.1	kN/mm
s	600	mm

$K = K_{0.4}/s = 73.5$ MN/m^2

For a simply supported composite beam element (μ = 1), of span L = 7 m, and uniformly distributed loads at serviceability and ultimate limit state $q_{0.SLS}$ = 2.8 kN/m, and $q_{0.ULS}$ = 3.9 kN/m, respectively, corresponding to a 2 kN/m^2 live load and to the self-weight of the finished floor structure, the following results are obtained.

EI_0	2.69	$MN.m^2$	
EI_∞	10.47	$MN.m^2$	
EI_{eff}	7.95	$MN.m^2$	
EI_{eff} / EI_0	2.95		
$F_{s,eff,SLS}$	18.2	kN	
$F_{s,eff,ULS}$	25.6	kN	
$\sigma_{cc,eff,ULS}$	12.6	MPa	(< f_{cd} = 16.67 MPa)
$\sigma_{st,eff,ULS}$	55.8	MPa	(< f_y = 355 MPa)
$\sigma_{tt,eff,ULS}$	3.9	MPa	(< $f_{t,0,d}$ = 8.96 MPa)

The normal compression stresses estimated in the concrete are brought to an acceptable level in the partially composite beam, while in the non composite section (i.e. when $EI_{eff} = EI_0$), the design strength of the concrete would be exceeded.

$\sigma_{cc,0,ULS}$	20.84	MPa	(> f_{cd} = 16.67 MPa)
$\sigma_{st,0,ULS}$	276.38	MPa	(< f_y = 355 MPa)
$\sigma_{tt,0,ULS}$	3.60	MPa	(< $f_{t,0,d}$ = 8.96 MPa)

The partially composite section has a bending stiffness about 3 times greater than the non-composite section. The mid-span deflection at the serviceability limit state is given by $\delta_{eff,max,SLS} = (5q_{0,SLS} \cdot L^4)/(384EI_{eff})$ = 10.9 mm for the instantaneous deflection. This is well below the criteria demanded in the Eurocode.

7 CONCLUSIONS

Five types of shear connections have been tested and three of them (S-4S-65, M-4S-65 and M-8S-65 series) showed desired ductile characteristics and can be recommended for use in composite floor elements. The main slip occurred at the steel-timber interface. The slip modulus and ultimate capacity were satisfactorily high, providing a high degree of composite action.

The test results compared reasonably well with the shear capacity obtained analytically. For a typical design of a partially composite floor structure with these types of shear connectors the bending stiffness was increased almost three times.

The behaviour of the shear connections could probably be improved by using inclined self tapping screws.

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REFERENCES

[1] E. Lukaszewska: "Development of Prefabricated Timber-Concrete Composite Floors," Doctoral Thesis, Luleå University of Technology, 2009.

[2] D. Yeoh, M. Fragiacomo, M. De Franceschi, K. Heng Boon: "State of the art on timber-concrete composite structures: Literature review," Journal of Structural Engineering, vol. 137, pp. 1085-1095, 2011.

[3] L. A. Bathon, O. Bletz, J Schmidt: "Hurricane proof buildings – An innovative solution using prefabricated modular wood-concrete-composite elements," presented at the WCTE, 2006.

[4] CEN, "EN 338: Structural Timber - Strenght classes," ed. Brussels: European Committee for Standardization, 2009.

[5] CEN, "EN 1995-1-1:2004 (E)," Eurocode 5 Design of timber structures, ed: CEN, 2004.

[6] DIBT, "European Technical Approval ETA-08/0271," ed: Deutsches Institut für Bautechnik, 2009.

[7] I. O. f. Standardization, "Timber structures - Joints made with mechanical fasteners - General principles for the determination of stength and deformation characteristics," ed, 1983.

[8] A. M. P. G. Dias, H. M. P. Cruz, S. M. R. Lopes, J. W. van de Kuilen: "Stiffness of dowel-type fasteners in timber-concrete joints," Proceedings of the Institution of Civil Engineers: Structures and Buildings, vol. 163, pp. 257-266, 2010.

[9] K. W. Johansen: "Theory of Timber Connections," International Association of Bidge and Structural Engineering, vol. 9, pp. 249-262, 1949.

[10] U. A. Girhammar: "A simplified analysis method for composite beams with interlayer slip," International Journal of Mechanical Sciences, vol. 51, pp. 515-530, 2009.