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ORIGINAL ARTICLE

Evaluation of non-linear behavior of timber–concrete composite structures using FE model

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Abstract This article describes a numerical model that was developed for the analysis of composite timber–concrete beams. This model presents a simplified methodology for determining the effective bending stiffness of the timber–concrete composite structure. It is based on previous work done usually referred to in some non-normative literature by *γ-method*. The implemented methodology assumes some simplifications, as for instance, linear elastic behavior of all components, constant stiffness of the connection and sinusoidal loading. For comparison purposes, the work benefits from an experimental program in which full-scale beams were tested in bending and timber–concrete connections were tested in shear. The FE model has shown the ability to overcome the simplifications of the Eurocode, namely the variation of shear force along the beam axis. The numerical model is capable of detecting and quantifying the influence of the non-linear behavior of the connections on the composite structure. Different parameters are analyzed and, for instance, the ductility behavior of the timber–concrete connection could be more important than the maximum strength, which is an interesting result. By comparing theoretical predictions with test results, it is clear that the numerical model used in this work is a very interesting method when compared with the usual design models, such as that of Annex B of Eurocode 5 (EN 1995-1-1). The influence of the connections behavior on the ultimate load of the composite structure is very important and the described approach proved to give good predictions.

Keywords Timber–concrete floors · Composite structures · Timber structures · Beams · Computational analysis

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1 Introduction

A research work related with timber–concrete slabs has been carried out at the University of Coimbra for some years [15–19, 35]. The next step was to investigate the use of lightweight concrete in these structures [27, 28].

Apart from the above cited works, few other research works on timber concrete composite beams can be found in bibliography. Mascia and Soriano [37] presented some benefits of timber concrete composite structures in rural bridges. Frangi et al. [24] presented some aspects of the fire behavior of timber concrete beams using glued laminated timber. Fragiacomo and Ceccotti [23] Fragiacomo [21], Ceccotti et al. [8] and Fragiacomo, Amadio and Macorini [22] have presented a set of studies on timber concrete structures. They proposed a FE model for long term analyses of this kind of structures and the computing proposal proved to be very accurate, but it is somehow complex for normal design situations. They reported some tests on laminated timber concrete beams. Brunner et al. [4] presented some tests on timber concrete slabs. They used an adhesive connector. Deam et al. [14] reported a study at the University of Canterbury in where different types of connectors were tested in timber concrete solutions. A study on the performance of connections for prefabricated timber–concrete composite floors was presented by Lukaszewsk et al. [36]. Gutkowski et al. [26] have also presented some laboratory tests on composite wood–concrete beams, but the connection is quite different from that presented in this article. Lantos [32] and Cramer [12] shown that the capacity of individual fasteners in a multiple-fastener connection is not always equivalent to the capacity of a fastener in a single-fastener connection. Nowadays, by numerical modeling is it possible to determine the behavior identified by Lantos and Cramer, but it still needs excessive computation time due to its complexity, especially where friction and damage models are considered [38]. As far as FE models are concerned, a basic study by Battini et al. [1] deserve to be cited, because the authors propose a general methodology to predict the mechanical behavior of composite beams with interlayer slips. Some researchers have concentrated on the development of numerical methods to solve two-layers with interlayer slip [25, 31, 40] but the mechanical behavior of concrete–timber structures is somehow different from that of timber–timber structures. Steel–concrete composite structures also behave differently from timber concrete structures [41]. Timber concrete slabs are a solution for ancient buildings, but few works cover this aspect. As an example, the article by Faggiano et al. [20] deserves to be mentioned.

The recent Eurocodes cover most structural materials, but no Eurocode exists dealing with timber–concrete composite structures, such as those represented in Fig. 1. Consequently, designers must use design rules proposed in technical articles, complemented with rules adapted from Eurocodes 2 and 5 [9–11].

To design timber–concrete composite structures, the designer takes advantage of the interaction of their components (timber and concrete). However, such structures are often analysed and designed based on simple linear elastic models. The best known design model is presented in Annex B of Eurocode 5—Part 1.1 [10].

This model presents a simplified methodology for determining the effective bending stiffness of the timber–concrete composite structure. It is based on previous work done by Möhler [39] usually referred to in some non-normative literature by the γ*-method*. The methodology assumes some simplifications, as for example, linear elastic behavior of all components, constant stiffness of the connection and sinusoidal loading. Each of these simplifications has different impacts in the application of this system.

The assumption of constant stiffness of the connection is related with the incapacity of solving the elastic-line equation, a fourth-order differential equation with non-linear coefficients. The variation of stiffness along the beam axis depends on the connectors spacing. If the longitudinal shear is not constant (it depends on the vertical shear), connectors spacing should preferably be proportional to vertical shear. Having this in mind, Eurocode 5 [10] allows a ratio of maximum to minimum spacing on connections up to four and then it considers an equivalent constant spacing.

Tests on timber–concrete composite structures show a great non linear behavior up to the maximum load [27]. This non-linearity is also related with the non-linear behavior of the connection between timber

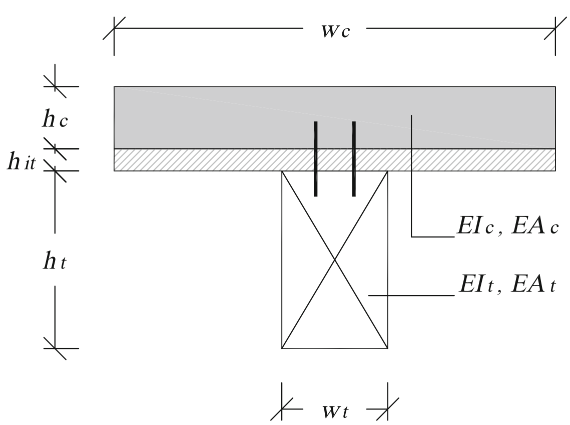

Fig. 1 Timber–concrete composite cross-section

and concrete. EC5 takes this behavior indirectly into account by proposing a reduction of the connection stiffness at Ultimate Limit States by considering a modified stiffness, k_u, equal to 2/3 k_s.

2 Calibration

To compare the experimental results of flexural tests on timber–concrete composite beams, a Finite Element Modelling was developed using the available features of SAP 2000® software package, CSI [13].

The model replicates half of the structure, taking advantage of the symmetry in the static arrangement of the test. The timber and concrete components were modelled as simple beam-type finite elements placed parallel to each other at a distance equal to the distance between their centroids. The beam element is modelled as a straight line; connecting two points (nodes) with 6 degrees of freedom with linear-elastic behavior (see Fig. 2).

The timber–concrete connection is modeled by spring type elements, which allow modeling the non linear behavior obtained in laboratorial shear tests of timber–concrete specimens. The deformations corresponding to the 6 degrees of freedom are all restrained, with one exception, which allows the type of deformations shown in Fig. 2. In this case, the imposed load–slip relationship is that represented in Fig. 3. The load–slip pattern was determined by fixing some key points: $0.4F_{\max}$, $0.8F_{\max}$, $F_{\max}$ and $0.8F_{\max}$ (in the descending branch). For each load, the corresponding deformation (v_i) was found by computing the average of the available test results.

The geometric characteristics of the model are visible in Fig. 2. The contacts of the structure with the exterior are idealized by a simple support and two slides. The exterior point load is applied at the node located 1.80 m away from the simple support. The static analysis considers the non linear behavior of the connection.

The elastic properties of every system component adopted in the FE model came from the experimental campaign. The Young modulus of concrete is the only exception, since it was not possible to measure it in every beam specimen and therefore, in order to guarantee consistency between the several modelled configurations, this property was determined according to Eurocode 2 formulation [9]. The load–deflection of the spring follows a model behavior, according to the experimental results (this will be described later).

The failure criterion was not determined by the connection failure. The beam was considered to fail if the load would drop 20% after the peak load or any of the materials (timber or concrete) would reach their strengths.

The model is suitable for all types of connection, provided that the load–slip relationship of a particular connection is known.

The FE model described above lays on simple formulation and on low computation effort, and does not need complex finite element software. This will enable easy access by designers to the calculation procedure for common situations. This procedure is

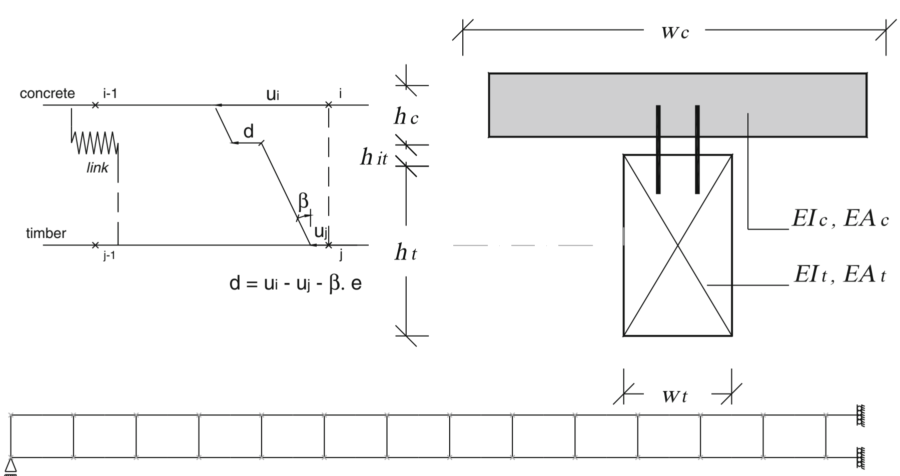

Fig. 2 Finite element model

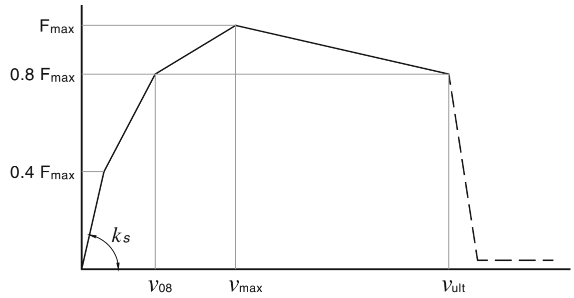

Fig. 3 Model behavior of the load–slip relationship

preferred to simple calculation methods, because it predicts much better the behavior of the structure. In fact, the proposed model can handle the non-linear stage of the composite beam and accepts the non-regular connection spacing, which means variable connection stiffness along the beam.

3 Experimental program description

Calibration of FE model was necessary in order to proceed with further analysis based on numerical data. The lightweight concrete produced for the testing programme had a compression strength between 20 and 30 MPa and a density between 1400 and 1600 kg/m^3.

The experimental programme started with shear tests on connection specimens, according to EN 26891 (CEN, 1994). Load–slip was recorded from 353 test specimens and the corresponding individual elastic properties and load capacity were computed from test data [29]. Scattering of load–slip diagrams was visible (this will be seen in the Sect. 4.1), especially after connection yielding. Nevertheless, the coefficients of variation of elastic stiffness and load carrying capacity were generally below 15 and 10%, respectively.

Several connection configurations have been tested, mainly with the use of special screws for timber–concrete connections, the SFS VB 48-7.5x100®. This is a very common solution for timber–concrete composite (TCC) structures and many research studies have proved their suitability for composite slabs. The connection provides large plastic deformation after the yielding phase. This fact enables a certain level of non-linear behavior of the structure which could be very important if the stresses at the materials (timber and concrete) are below to the strength limits.

Bending tests on simply supported 5.4 m span beams were carried out under static loading made on typical four-point bending test configuration [30]. The beam cross-section was made from a glulam web, connected to a concrete layer flange, thus forming a T-beam configuration, with or without interlayer. The presence of interlayer on the cross-section usually slightly decreases the mechanical properties of the connection. On the other hand, it acts as formwork when the concrete is poured, and, in existing timber floors, it might be already in place before strengthening. The load–deflection curves of these beams are very different from those of reinforced concrete beams [2, 3, 5–7, 33, 34].

The mid-span deflection and slip at the beam ends were recorded and this was very important to compare numerical predictions with experimental values.

4 Parameter study

4.1 On the experimental programme

Based on tests performed in several different configurations of timber–concrete composite T-beams, it was possible to calibrate and confirm the adequacy of the finite element model which enables to replicate the real behavior.

Figure 4 shows the deformation predicted from numerical analysis and that obtained from tests. As seen on the graph, the two curves closely follow each other. The numerical model gives the stress in concrete and the stress ratio on timber, as shown in Fig. 4. The positive values correspond to tensile stresses. These two curves are also plotted on the same graph.

Figure 5 repeats the predicted curve for the mid span deflection and the predicted curves for the forces developed at selected connectors. It shows that, up to the maximum load of the beam, all the connectors are under forces bellow to their strength capacity. Figures 6 and 7 show how the load–slip diagram pattern used in FE model was constructed. Figure 6 shows the cloud of curves obtained in the tests carried out for this research. Figure 7 shows the minimum and maximum boundaries of Force F as well as the average and medium curves. Figure 4 shows that for mid-span deflections of approximately 30 mm, stresses in concrete become positive, which means that tensile

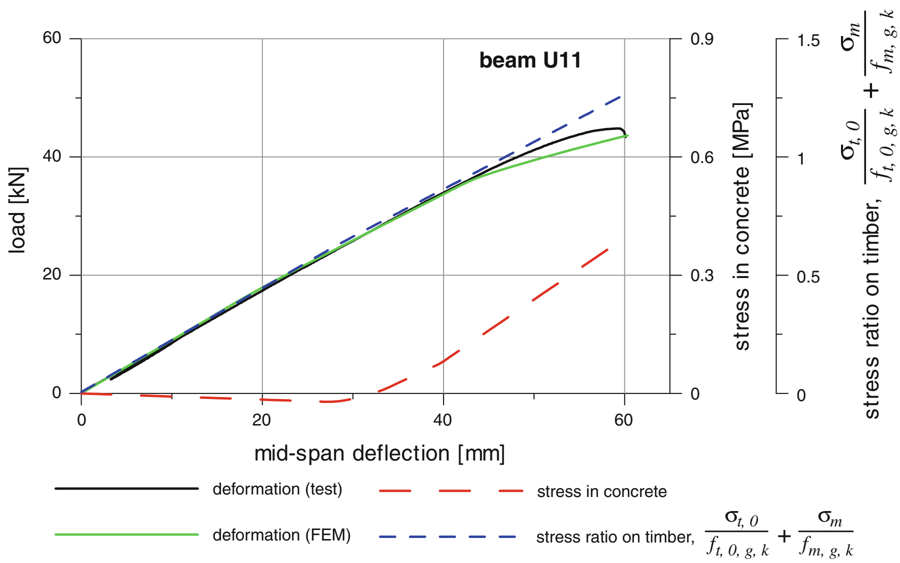

Fig. 4 Numerical simulation of bending test of Beam U11

stresses developed in concrete. The stresses in concrete go from negative to positive when the load at the connectors correspond to a point on the second phase of the model curve adopted for FE model (see Figs. 3, 6 and 7). Therefore, at this stage, the stiffness of the connection is lower than that of the starting of loading and proves the relevance of the connection stiffness for the composite behavior. The numerical analysis indicates that this beam reached failure without reaching the maximum strength of the connection. It can be seen that above 80% of their maximum load the stiffness suffered a further reduction.

Another parameter that was recorded during the tests was the slip at both ends of the beams. Figure 8 shows the example of Beam H9. It can be seen that both slip (dashed lines) and mid-span deflection increase faster for loads above approximately 25 kN. The predicted curve (FE model) is very close to the actual behavior obtained during the test.

Beam B24 presented the experimental behavior plotted in Fig. 9. Although the shape of the actual load–slip curve was very unusual, the numerical prediction was, also in this case, very satisfactory.

4.2 On the influence of connection stiffness variation along the beam axis

The spacing between connectors is an important issue. The practical option of considering this spacing constant along the length of the beam would not lead to the most efficient solution. As mentioned before, there is no analytical solution for the equation of the

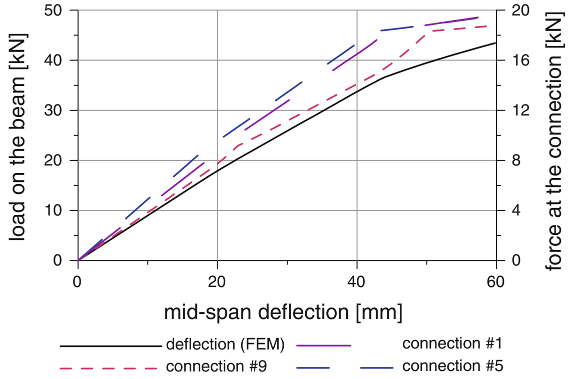

Fig. 5 Numerical simulation of bending test of Beam U11

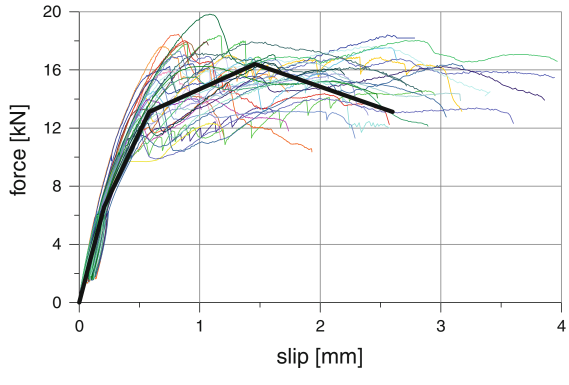

Fig. 6 Load–slip diagram pattern for connection, Series H

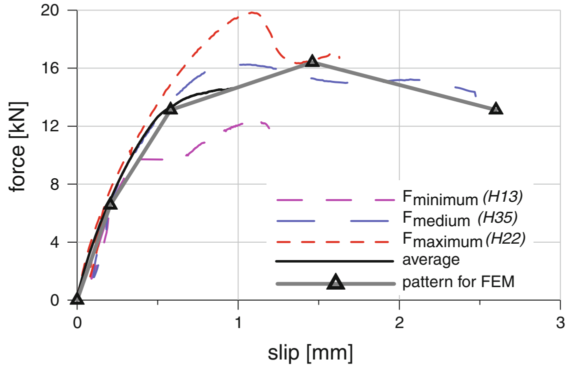

Fig. 7 Load–slip diagram pattern, Series H

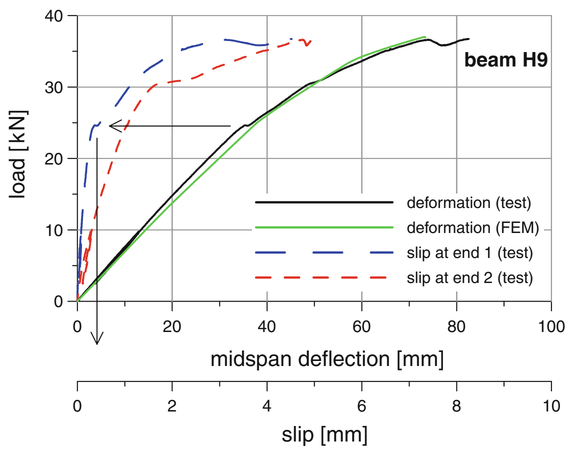

Fig. 8 Results of Beam H9

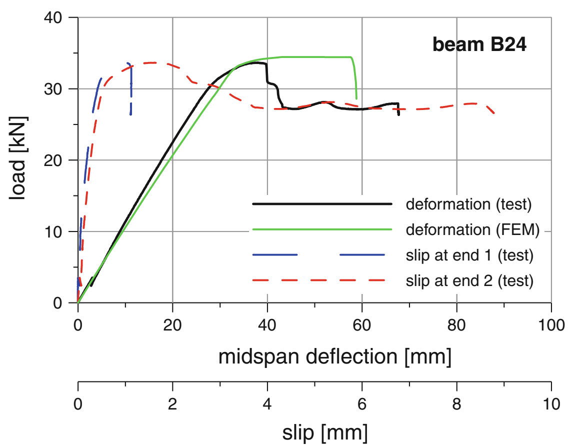

Fig. 9 Results of Beam B24

elastic line when the stiffness of the connection is not constant. Eurocode 5 overcomes this problem by proposing an equivalent spacing (a constant value) providing that: (a) the ratio of maximum to minimum spacing is limited to 4; (b) the equivalent spacing is then equal to 75% of the minimum spacing plus 25% of the maximum spacing.

In the work described here, FE model simulations were performed in order to evaluate the use of non-constant spacing. Possible types of connectors' layout are presented in Table 1 (the span was 5.4 m long and the values of spacing of Table 1 are in cm). Unlike the other types, Types B and C are not within the limitation specified by Eurocode 5. According to Eurocode 5, for analysing the composite beam with non-constant spacing of connections, the equivalent spacing for types D and E would be 14 cm.

The forces at the connectors are presented in Fig. 10. A good choice of the spacing schedule might increase the load carrying capacity of the beam, because this is often governed by the strength of the most stressed connection. As it will be explained in the next Section, in design, the control of the beam behavior by controlling the strength of the connection is a fundamental approach to ensure ductility to the beam.

Figure 10 indicates that Type A layout should be avoided, because the maximum force is notably higher than those of the other layout type. In fact, the forces at the individual connectors of Types B to D are not very different from each other. Nevertheless, in this study, Type B proved to be the best solution because the maximum force reached at the connectors is the lowest of all.

Figure 11 shows the displacements of the beams for each type of connectors layout. This figure shows that a solid choice of connectors might not only imply some advantages on the ultimate behavior of the structures, but also might also imply some advantages on the service behavior of the structures. In fact, the stiffness seems to be related with such a choice on the connectors, and therefore, the displacements are also influenced by it.

From the five studied solutions, Type B is the best solution with respect to deformations. This type was also the best solution with respect to ultimate behavior (Fig. 10), but the difference to the second best solution is clearer for deformations than for ultimate behavior. The restrictions of EC5 are not encouraging for the use of a simplified model and only the numerical model presented here enables to highlight certain advantages with sufficient exactness. Furthermore, the

Table 1 Connections spacing in each type (layout)

Connection layout	Connections spacing: Type A Constant	Type B Geometric progression ($s_{max}/s_{min} > 4$)	Type C Arithmetic progression ($s_{max}/s_{min} > 4$)	Type D 'Stair' type ($s_{max}/s_{min} = 4$)	Type E Arithmetic progression ($s_{max}/s_{min} = 4$)
1st	20.0	5.0	5.0	8.0	8.0
2nd	20.0	6.1	7.5	8.0	10.0
3rd	20.0	7.3	10.0	8.0	12.0
4th	20.0	8.9	12.5	8.0	14.0
5th	20.0	10.7	15.0	8.0	16.0
6th	20.0	13.0	17.5	20.0	18.0
7th	20.0	15.7	20.0	20.0	20.0
8th	20.0	19.0	22.5	20.0	22.0
9th	20.0	23.0	25.0	32.0	24.0
10th	20.0	27.8	27.5	32.0	26.0
11th	20.0	33.6	30.0	32.0	28.0
12th	20.0	40.7	32.5	32.0	30.0
13th	20.0	49.2	35.0	32.0	32.0

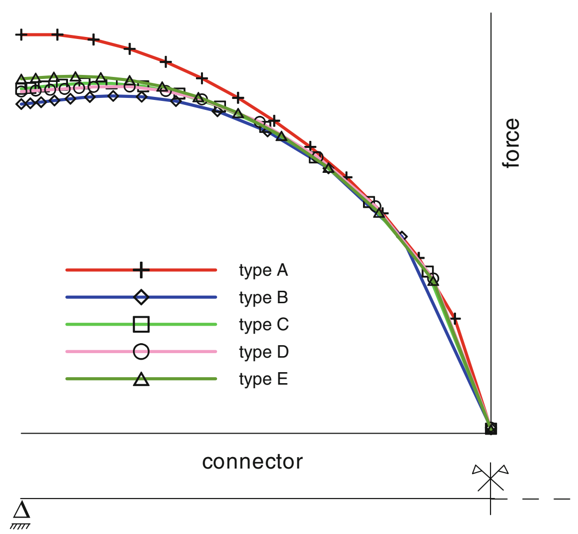

Fig. 10 Force at the connectors

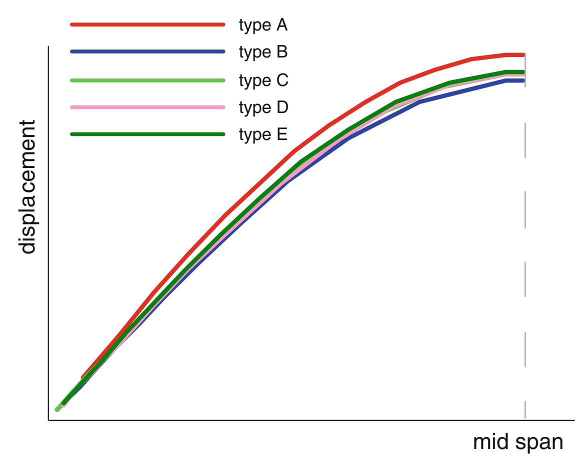

Fig. 11 Deflection by using each type of connection layout

improvement on the structural performance associated with the optimization of the connectors imply a saving in the costs of a structure.

4.3 On the influence of connection load–slip diagram

A sensitivity analysis of the behavior of the timber–concrete composite beam to small differences of the connection characteristics was carried out by using the FE model. Figure 12 shows the force load–slip curves of the connections. Figure 13 shows the predicted mid-span deflection of beams for the situations presented in Fig. 12.

The lines of Fig. 13 correspond to those of Fig. 12. It can be seen that the force–slip behavior of the connection (Fig. 12) influences the load deflection curves of the composite beams (Fig. 13). It is interesting to note that the strength of the connection does have some influence, but the form shape of the

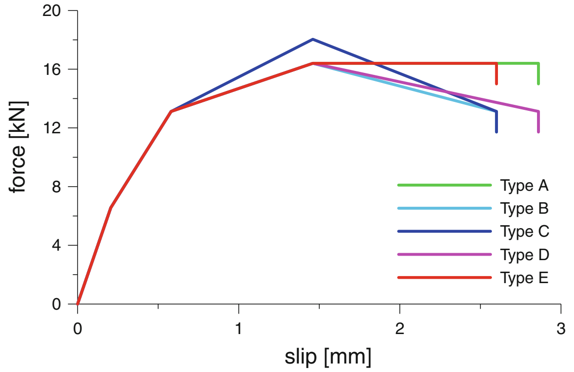

Fig. 12 Load–slip curves of the connections for FE model simulations

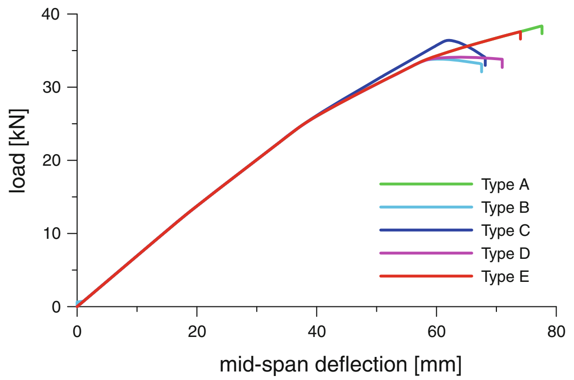

Fig. 13 Load–deflection curves of composite beams from FE model simulations

force–slip curve of the connection beyond the maximum force has a relatively high influence on the load–deflection curve of the composite beam, that could be even higher then the influence of the strength of the connection.

When comparing, for instance, Type A with Type B behaviors, for instance, the maximum forces of both connections are the same, but Type A connection behavior led to a maximum strength of the composite beam of around 13% higher than that of Type B behavior. Comparing two situations with the same ultimate slip and the same maximum strength of the connections (Types A and D), the difference in the strength of the composite beam is approximately 12%.

On the other hand, a 5% increase on the connection strength of Type C behavior, when compared to Type D behavior led to an increase of 6.4% of the strength of the composite beam, which is notably less than the increase that can be obtained by maintaining the maximum strength of the connection and varying the pattern of the post peak force–slip relationship.

These simulations were very useful to evaluate the sensitivity of the behavior of the beam to the characteristics of the connection and helped understanding the experimentally assessed behavior of the composite beams and the respective numerical predictions.

5 Conclusions

The FE model described in this article predicts the experimental behavior of timber–concrete composite structures very satisfactory and its relative simplicity makes it a good method for designing these structures. The simplifications and limitations expressed in the simplified model of Eurocode are advantageously overcome by finite element modeling presented here. The main advantages are the ability to model the non-linear phase of the structure (due to the behavior of connectors) and the ability to more accurately define the non constant spacing of the connectors, as a consequence of the distribution of shear forces.

The influence of the connections behavior on the ultimate load of the composite beam is very important. At a basic level, the use of connections with some plastic deformation capacity is widely accepted by researchers and structural engineers, but only with the FE model the designer is able to take full advantage of that.

Moving to a higher level of complexity, the amount of plastic deformation at the connections influences the overall behavior of the composite beam. The large number of shear tests on timber–concrete connections performed, found a curve which best describes their behavior. This type of knowledge is crucial in order to model the non-linear response of the composite beam. The sensitivity analysis show that small changes of the force–slip curve of the connectors might produce relevant variations in the maximum strength of the composite beams. It was interesting to note that connectors with the same maximum forces and not very different maximum slips might produce a difference of 13% on the maximum load of the composite beam. Therefore, with reliable data for the connection behavior, FE modelling would be quite helpful to optimize strength prediction in this kind of systems. Therefore, it is very important to evaluate the force–slip curve of the connectors with special care.

Even in the linear elastic range, the behavior of the beam at SLS is improved by the FE model, which proved the importance of the model for the evaluation of the best connections layout. In general, the improvement on the structural performance associated with the optimization of the connectors imply a saving in the costs of a structure.

Long-term effects are an interesting issue and is certainly a topic that could be implemented in future works in order to extend the model to take this phenomenon into account. It should be taken into account that the beam has different materials, and materials have different behavior with respect to creep. Furthermore, in the same material, the level of stresses is different along the beam. As a consequence, the inclusion of the creep effects wouldn't be a simple task, but could be interesting topic to be studied.

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