Influence of concrete cracking on wood concrete composite bridges

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Summary

In certain cases the concrete slab of wood concrete composite beams can crack; deformations will increase and the internal forces will be redistributed. At present, the design recommendations available vary from ignoring the effect to reducing stiffness by 60%.

In order to characterize more correctly the influence of the concrete cracking on the force distribution in wood concrete composite beams a research project was conducted involving theoretical analysis and laboratory tests. First, a parameterized nonlinear FEM model was elaborated and calibrated by real test data available from the literature. Second, shear tests were conducted with test specimens that simulate the concrete cracks by a small separation between the concrete and the timber. The results indicate that the concrete cracking does not only reduce the stiffness of the concrete slab itself but also leads to a significant stiffness reduction of the shear connectors embedded in it.

Keywords: wood concrete composite, concrete cracking, timber bridge, Chile.

1. Introduction

Timber beams for composite bridges are an economical and ecologically sustainable alternative to the steel or pre-stressed concrete beams usually used in Chile. Over the last two decades, wood concrete composite bridges have been built in many countries and design recommendations on the assessment of the connection system have been incorporated into the design standards. However, when designing the first Chilean wood concrete composite bridge, it was found that the design recommendations for considering the effect of concrete cracking on the composite beam vary from ignoring the effect [1] to reducing stiffness by 60% [2]. In order to characterize more correctly the influence of the concrete cracking a research project was conducted, whose methodology and results are presented in this paper.

2. Bridge construction in Chile

2.1 General Overview

Historically, bridge construction in Chile was not very different from bridge construction in other parts of the world. Railway bridges were made of steel trusses and concrete arches, suspensions systems and concrete girders were used for road bridges. However, during the last three decades, the variety of structures for new bridges has been limited principally to simply supported beams made of:

- timber beams and a timber deck for provisional bridges with spans of up to 10 m
- steel beams and a timber deck for provisional bridges with spans of up to 30 m

- prefabricated RC beams and a concrete deck for bridges with spans of up to 20 m
- prefabricated PC beams and a concrete deck for bridges with spans of up to 30 m
- steel beams and a composite concrete deck for bridges with spans of up to 40 m

The large part of the new bridges being built in Chile are made of prefabricated beams (RC, PC or steel) with a composite concrete deck. Therefore, the introduction of wood concrete composite bridges, where the prefabricated beams are made of glued laminated timber, does not suppose a radical change in the construction technology and is interesting for the Chilean bridge authorities.

2.2 Timber bridges

The construction of timber bridges in Chile is standardized by the Ministry of Public Works (Ministerio de Obras Públicas), which publishes a Manual for road and bridge construction [3]. According to this manual, timber bridges are provisional bridges with a wooden deck and concrete abutments and one single traffic lane. The typical construction system for span lengths from 2.0 to 10.0 m is given in figures 1 and 2. The cross section of the longitudinal beams varies from 11"x11" to 21"x21", depending on the span length. The wood species used are Chilean oak (Nothofagus obliqua) or Chilean beech (Nothofagus dombeyi), which are especially durable species. Furthermore, all wooden parts are protected by three layers of hot creosote. In the humid south of Chile (average annual precipitation of about 2500 mm), this type of bridge deck has a service time of only about 6 years.

In practice, about 40% of all Chilean bridges have an all timber deck and another 25% have a wooden deck on steel girders. That means that nearly two thirds of all Chilean bridges have wooden decks. Consequently, timber plays a decisive role in bridge construction in Chile and it is necessary to devise for innovative construction systems that increase the service life of these timber bridges.

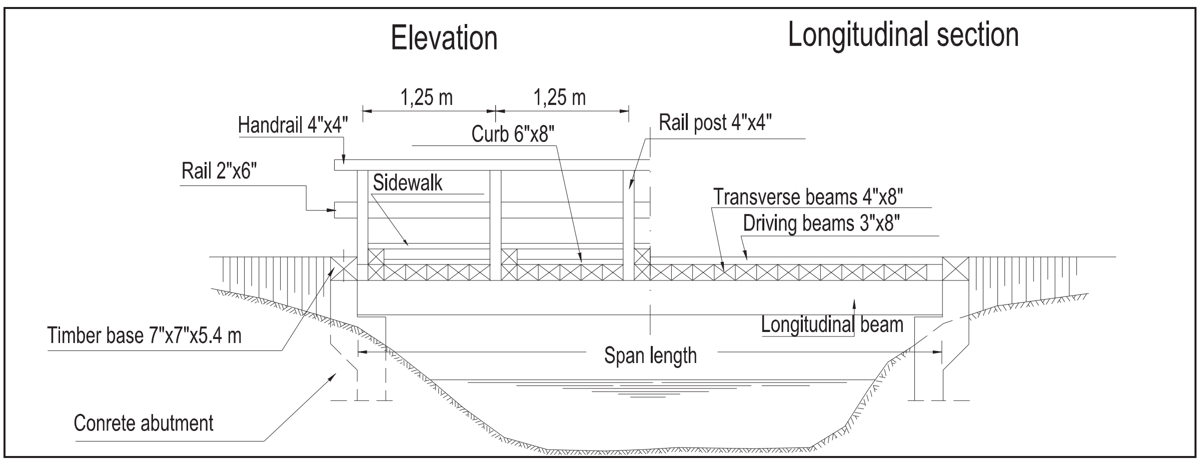

Fig. 1 Elevation and longitudinal section of a standard Chilean timber bridge [3]

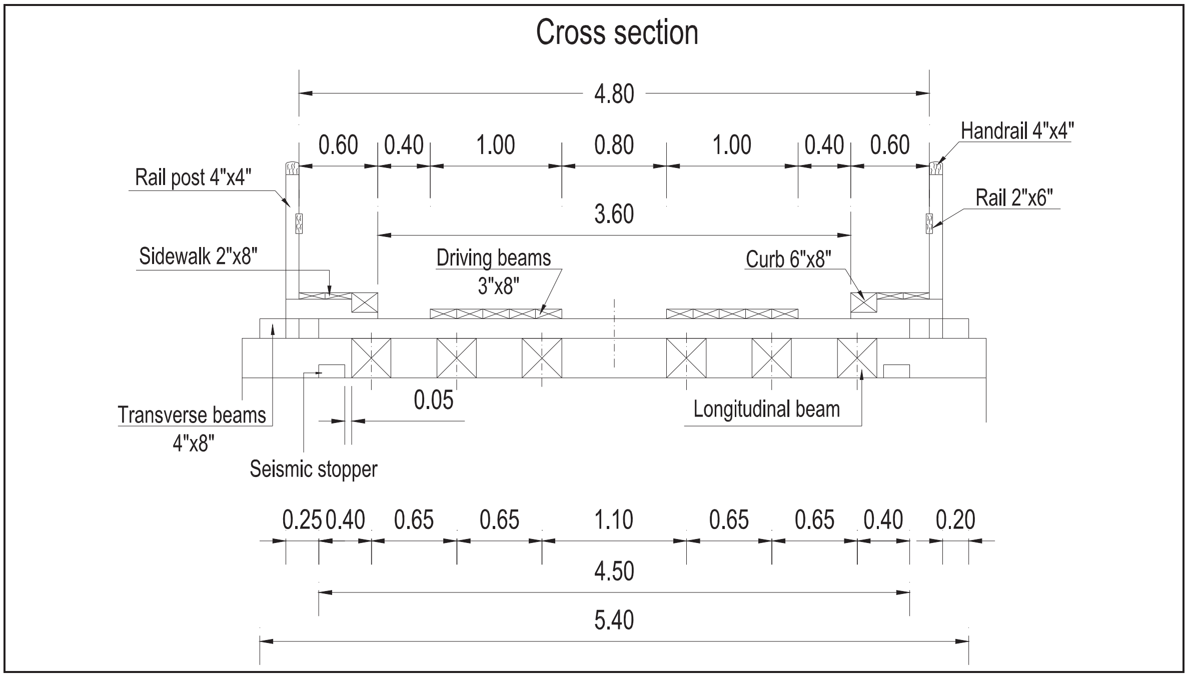

Fig. 2 Cross section of a standard Chilean timber bridge [3]

2.3 Ancahual Bridge

The first Chilean wood concrete composite bridge is going to be the Ancahual Bridge (Fig. 3), located in the state 'Región de la Araucania', close to the town of Loncoche. It is a simply supported girder bridge for two lanes of traffic with a span length of 14.38 m. It was designed with a 18 cm deep concrete slab of and 9 longitudinal glue laminated pine timber (Pinus radiata D. Don) beams. The timber beams are 14.0 m long, with a width of 0.25 m and 1.40 m in depth (Fig. 4).

The shear connector used between the concrete slab and the timber beams are ½"x6" wood screws. The main reason for the decision to use this type of connector was the requirement of the bridge authority to use existing design standards. The head of the ½"x6" wood screws is similar to stud connectors and can be designed as such, while the connection of wood screws to the timber beam is a standard design detail. There are two rows of wood screws on each beam; the longitudinal separation between the connectors is 10 cm. In order to increase the slip stiffness of the connection, the total number of wood screws is considerably higher than necessary to achieve the adequate ultimate load capacity.

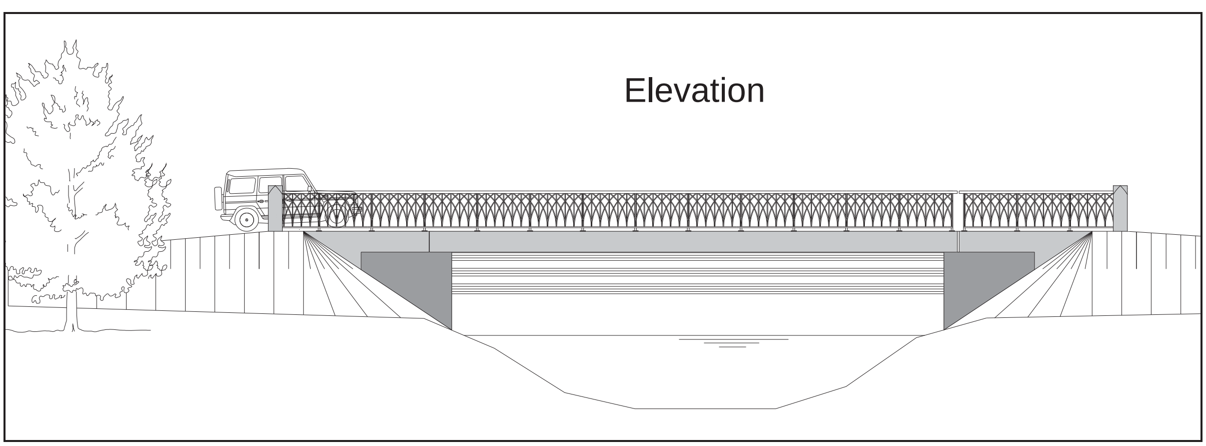

Fig. 3 Elevation of Ancahual Bridge

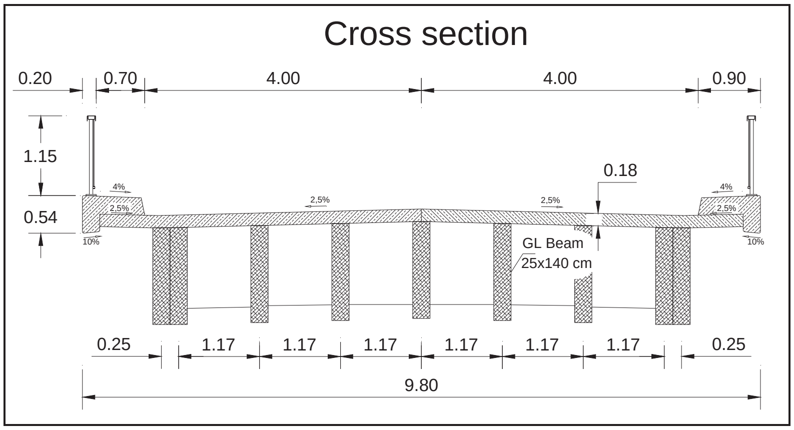

Fig. 4 Cross section of Ancahual Bridge

The Ancahual Bridge is planned to substitute an existing timber bridge at the same location, though currently only 5 m length. During the design process, the bridge length was gradually increased due to different requirements of the bridge authority. Also, the bridge width was increased to accommodate two lanes of traffic. This process provided the opportunity to study different geometries and depth ratios of decks and beams.

For certain geometry or stiffness combinations it was found that traction is produced at the bottom of the concrete slab. In such cases the concrete slab can crack; deformations increase and the internal forces are redistributed. For example, the deformation was found to increase by up to 30% and the bending moment in the timber beams was found to increase by up to 20%. As mentioned above, current design recommendations available from literature vary from ignoring the effect to reducing stiffness by 60%. In the Ancahual Bridge, the influence of concrete cracking was determined for each case by nonlinear finite element calculations, which may not always be cost-effective in the design of simply supported beam bridges. Consequently, a research project conducted at Universidad Austral de Chile aimed at obtaining more generally valid design recommendations.

3. Methodology

3.1 General approach

The research project is dived into three parts: first, existing laboratory tests of wood concrete composite beams, available in literature, are simulated in FEM calculations. The calculation model is preliminarily calibrated by means of the existing results. Second, laboratory tests of shear specimens are conducted in order to check the assumptions made in the preliminary calibrations. In a third and future step, the calculation model will be calibrated by conducting bending tests and a parametric study of a great variety of different geometries and stiffness ratios of wood concrete composite beams.

3.2 Simulation of existing laboratory tests

After a review of the existing publications of laboratory tests of wood concrete composite beams the following three studies were selected for the FEM simulation: Van der Linden 1999 [4], Yeoh 2010 [5], Klingenberg 2012 [6] and Ávila 2012 [7].

Van der Linden conducted 153 bending and shear tests with three types of connection. The first consist of pairs of wood screws, disposed at opposite angles of 45° to the vertical. The second type are steel plates nailed to the timber beam, and the third has notches in the timber beam, filled with concrete and anchored with vertical reinforcement bars. Figure 5 shows the original drawings of Van der Linden.

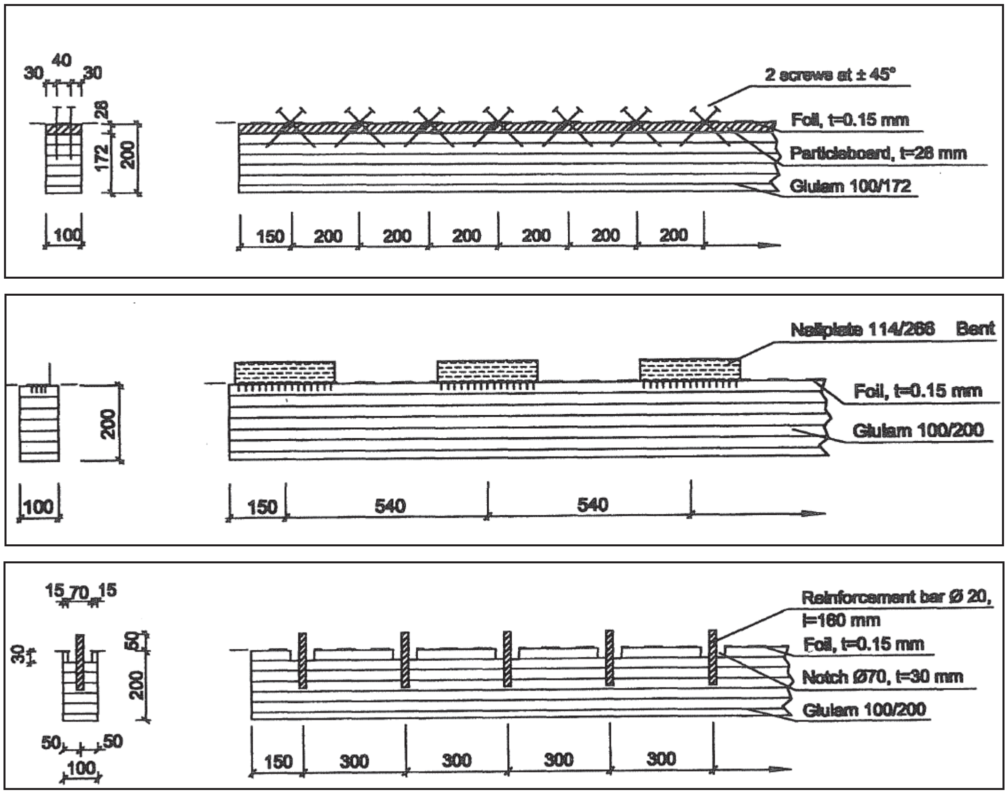

Fig. 5 Connection types used in the laboratory tests of Van der Linden [4]

In the work of Yeoh, the laboratory tests of 30 specimens are described. Different types of connectors were used, that include different notch forms with or without anchor bars and also vertical nail plates. Finally, the laboratory test conducted by Klingenberg and Ávila use ½”x6”wood screws for all tests.

In order to calibrate the finite element model and obtain simulated results as close to the laboratory test results as possible, a parameterized FEM model was elaborated. The parameters incorporated are listed in table 1. The numerical model is based on finite beam elements (Fig. 6). The timber beam and the concrete slab are modelled by 2 node Timoshenko beam elements with the corresponding mechanical properties. The actual reinforcement was assigned to the cross section of the concrete slab. The computer program used for the analysis was SOFiSTiK FEA under academic license.

The vertical contact between the concrete slab and the timber beam was modelled by a kinematic coupling of the vertical displacements of their nodes. The shear connection was modelled by discrete non-linear horizontal springs at the horizontal joint. The springs were connected rigidly to the centres of gravity of the concrete slab and timber beam. The non-linear stiffness behaviour of the springs was one of the parameters varied in order to obtain correlation with the documented results.

Table 1 Variable parameters of the FEM calculation model

Concrete slab	Timber beam	General
Depth	Depth	Time dependent load-displacement behaviour of the connection
Modulus of elasticity	Modulus of elasticity	Distribution of reinforcement
Creep	Creep	Type of loading and load history
Shrinkage	Shrinkage	Length and width
	Swelling	

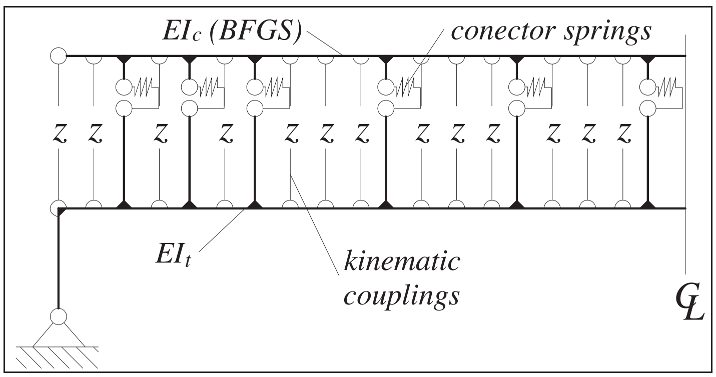

Fig. 6 Schematic view of the numerical analysis model

The non-linear iteration in order to find the force equilibrium between bending moment and curvature of the concrete slab was performed by BFGS (Broyden Fletcher Goldfarb−Shanno) procedure with line search, a generally accepted method for deformation determination in reinforced concrete structures. The stress-strain function of the concrete is adopted from the ACI318-08 [8], which is the valid standard in Chile.

3.3 Laboratory tests

The laboratory tests were designed as shear tests. In the shear test specimens, the cracked concrete was simulated by a reduction of the embedded length of the connectors in the concrete. This was achieved by a separation between the concrete and the timber of 0 mm, 2 mm, 4 mm and 6 mm. The shear test specimens, 3 of each type, have overall dimensions of length / width / depth of 0.6 m / 0.15 m / 0.3 m (Fig. 7).

Consequently, a total of 12 tests were conducted. In all test specimens ½”x6” wood screws at 10 cm spacing were used as shear connectors.

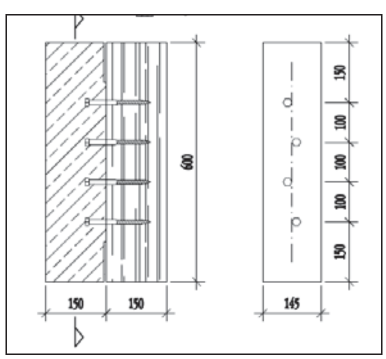

Fig. 7 Test configuration of shear tests

4. Results

4.1 Calibration of the FEM model

In a first step, the results documented in the literature were compared with those of the non-calibrate numerical model, which means, a numerical model was established with nonlinear material and connector properties in compliance with usual design recommendations. As an example, figure 8 shows this comparison for the data of Klingenberg and Van der Linden. As can be seen, in some cases a relatively high correlation was achieved, as in the case of Klingenberg. There, both curves are close together, but the FEM shows a greater stiffness in the elastic range and a less stiffness beyond this range. However, in the case of Van der Linden, there is low correlation between the initial MEF and the test results. There, the FEM model still shows elastic behavior, when the test results had already entered into the plastic range.

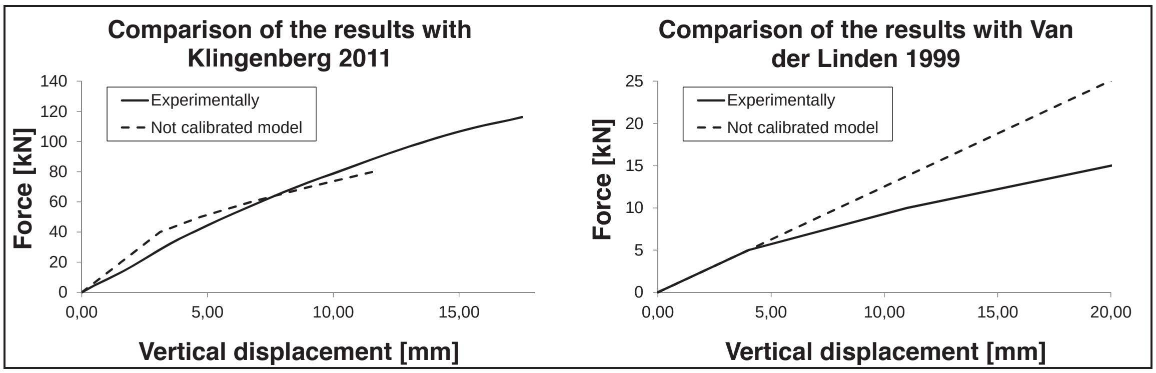

Fig. 8 Comparison of the results from the not calibrated model

The calibration process involved all of the parameters presented in table 1, but the most influential factors in the improving the simulation test correlation were the tensile strength of the concrete, the tension stiffening magnitude, as well as the stiffness and strength of the shear connectors. The decrease of any of these parameters led to an improvement in terms of model calibration.

The stiffness and the strength of the shear connector had the greatest influence. The corresponding theoretic values used for the calculation model were obtained from shear tests, while the examined cases were bending tests. There seems to be a significant difference in the connector behaviour between bending and shear tests. The difference might be due to the fact, that in bending specimens the connectors are embedded in the tensile zone of the concrete. The clamping of the connector to the concrete is less effective, especially if the concrete is cracked.

When applying these calibration concepts to the FEM model, the correlation between simulation and test results improved greatly. Figures 9 to 11 show the results of the calibration process.

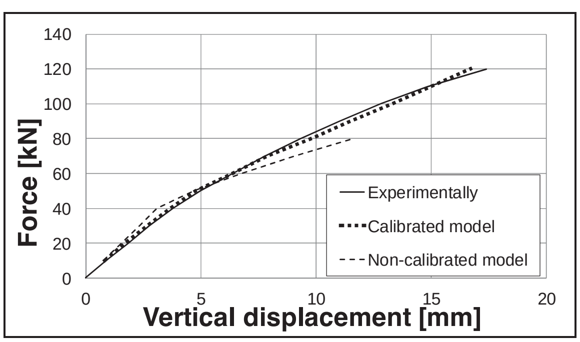

Fig. 9 Calibrated with Klingenberg 2011

The main change in order to obtain a better correlation with the results of Klingenberg was the reduction of the connector stiffness and strength by 50%. Furthermore, for the wood screw connector, the bilinear law proposed [2] is used.

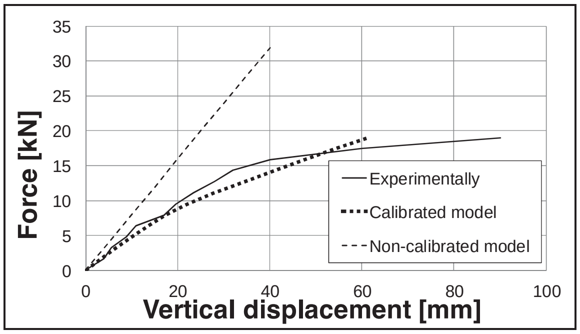

Fig. 10 Calibrated with Van der Linden 1999

Figure 10 shows the comparison of the non-calibrated and calibrated FEM model with the test results of oppositely inclined wood screws. The correlation between the simulation and the laboratory test is improved, when the connector stiffness is reduced by 50%. Also, the numerical model allows for concrete cracking. For loads bigger than about 20 kN the model diverges from the test result. In fact, the test beams seem to reach their ultimate capacity at this load level.

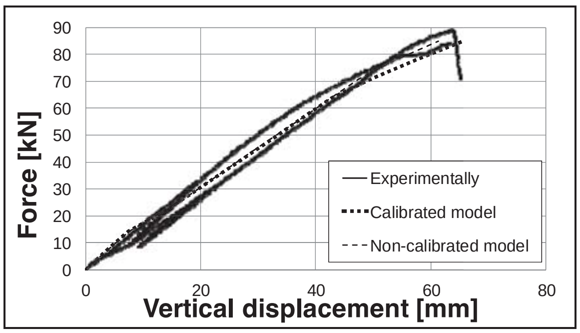

Fig. 11 Calibrated with Yeoh 2001

When the numerical model allows for concrete cracking, the results are very close to the test results of Yeoh (Fig. 11). Due to the notched shear connectors, there is no significant plastic behaviour before rupture. In the elastic range, the theoretic model has a high correlation with the laboratory tests. The use of the calibrated model, with 50% reduced connector stiffness, does not lead to an important improvement, nor does it lead to a poorer correlation.

4.2 Laboratory test results

As mentioned above, the shear test specimens have different separations between the concrete and the timber in order to simulate the effect of loss of embedding length of the shear connectors in case of concrete cracking. There is a supposed relation between the mean crack depth and the separation used in the tests, ruled by a factor of 2 to 5.

The relatively small separations used in the test, 2mm, 4 mm and 6 mm, cause a very significant stiffness loss in the shear connectors. On average, the elastic stiffness is reduced by 63%, 67% and 77% respectively (Fig 12). Considering these results, by conservative assumption cracks 10 mm deep could cause a stiffness reduction of 50% in connectors.

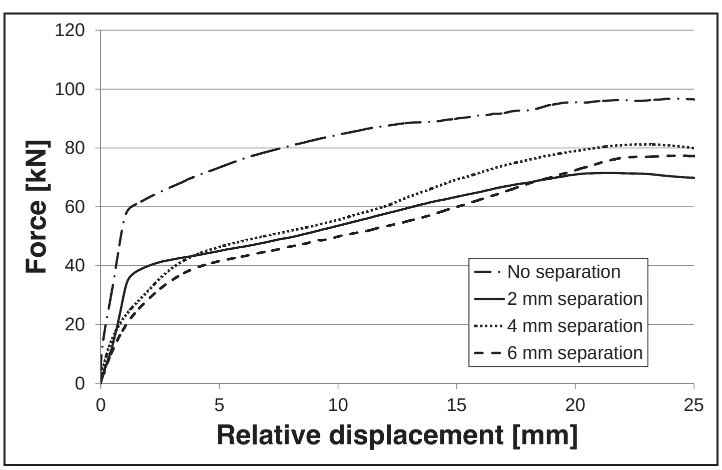

Fig. 12 Results from shear test with different separations between concrete and timber

5. Discussion

During the design of the first Chilean wood concrete composite bridge, it was found that certain timber concrete depth ratios and geometric proportions can cause traction on the bottom side of the concrete slab. However, the existing design recommendations do not give consistent advice. Therefore, this research project on the influence of concrete cracking on wood composite beams was conducted.

The theoretical part of the research aimed to numerically simulate real laboratory tests. The numerical model was calibrated by the results of laboratory tests found in the literature. Surprisingly, the consideration of the mere stiffness reduction of the concrete slab due to cracking has very little effect on the improvement of the correlation between simulated and real test results. For an effective calibration it was necessary to reduce additionally the stiffness of the shear connection by about 50%. This means that a concrete crack might weaken the embedding of the shear connectors in the concrete over the depth of the crack. This behaviour is likely similar to what occurs when there is a small separation between the concrete and the timber.

Indeed, in laboratory tests at the UACH of shear specimens with 2 mm, 4 mm and 6 mm separations it was found that the connection stiffness was reduced by up to 77%. These separations are very small and should have a similar effect as small cracks with depths between 10 to 15 mm.

6. Conclusions

The results from the FEM simulations and of the shear tests indicate that the influence of the concrete cracking is not limited to a stiffness reduction of the concrete slab. In order to calibrate the numerical model it was also necessary to reduce the stiffness of shear connectors. No other parameter was found to significantly improve the correlation between simulations and real tests. Laboratory tests validate that small concrete cracks can cause significant connector stiffness reductions.

In a further research step, laboratory tests of bending specimens should be conducted, with geometric proportions that are especially susceptible to concrete cracking in order to observe if the behaviour of the shear connector changes, and if there are bending cracks at the bottom surface of the concrete slab.

7. Acknowledgements

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