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CIRCULAR HOLLOW SECTION CONNECTORS IN TIMBER-CONCRETE COMPOSITE STRUCTURAL ELEMENTS

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Abstract. This paper presents results of the laboratory tests of timber-concrete structural connectors applicable (but not limited to) road/pedestrian bridges. The tested elements are discrete circular hollow section connectors installed in the pre-drilled slots of a glulam. Symmetrical push-out tests are conducted for two groups of connectors: 1) with wood core not removed; no interlayer between wood and concrete; 2) concrete in-fill core instead of the wood core; waterproofing membrane interlayer between concrete and timber elements. Main structural parameters of the connectors are established including ultimate shear capacity and slip modulus. Relationship between the connector's stiffness and ultimate shear capacity is established and failure mechanisms are briefly discussed.

Keywords: bridges, circular hollow sections, composite elements, connectors, glulam, shear tests, slip modulus, timber-concrete.

1. Introduction

Advantages of timber-concrete composite elements in bridge engineering have been widely acknowledged not only through many unconventional bridge projects, but also by the appropriate research activities during the last twenty years, for instance, Jutila and Salokangas (2010) and Miebach (2014). Most of the research efforts deal with connector systems because degree of timber-concrete composite behaviour is essentially governed by the effectiveness of connectors used for the load distribution between these two very different materials. For this particular study, the circular hollow section (CHS) steel connectors with the wood core and concrete in-fill core were examined by symmetric shear tests in the laboratory.

A number of very different shear connectors for timber-concrete composite action systems have been developed and tested. Some of them are more oriented towards residential sector, but others are also successfully implemented in bridge construction, for further studies respective references are given in Fragiacomo, Lukaszewska (2011) and Bathon, Bletz-Muhldorfer (2010).

Considering that there are many geometric variations possible with these connectors, some difficulties arise in order to typify and compare connector elements by their performance for direct applications in structural analysis. For a general guide, reference of (Rodrigues *et al.* 2013) is advised, which reviews most common wood-concrete connector types. Due to the lack of universal methodology of wood-concrete structural design, physical tests of any particular connection system are almost the only way to ensure adequate specification and performance of the design. Statistical variations of different connector parameters researched so far has comprehensively studied in Dias *et al.* (2013).

1.1. Aim of the study

Aim of this study is to acknowledge structural capacity of circular hollow section (CHS) connectors for applications in short and medium span timber-concrete bridges. In timber-concrete bridge engineering designer has to deal with concentrated loads induced by heavy duty traffic resulting in relatively high shear forces at the connector's interface. *Eurocode 1: Actions on Structures – Part 2: Traffic Loads on Bridges*, for instance, typically requires consideration of 150 kN×4 live load configuration at the design stage of the bridge along with the uniformly distributed load (UDL) of 9 kN/m^2 (see details in CEN (2003)). As a result, a demand for structural capacity of such connectors may be considerably higher than for other residential or civil structures.

In the development of the connector for this study, the following structural and functional requirements were raised:

1. High degree of stiffness for bridge deflection control.

2. High ultimate capacity of the connector's system in order to provide structural reserve and redundancy due to the cyclic damage accumulation issues.

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3. Ductility capacity at the ultimate range of loads in order to prevent brittle failure.

4. Wood splitting elimination – challenge of geometrical properties of the connector.

5. Cost effectiveness from manufacturing point of view (preparation, construction etc.).

1.2. Motivation behind the selection of CHS

The choice of CHS connectors within this research is based on the assumption that circular section provides more active and passive pressure surface area in shear than, for instance, simple glued-in rods as in Jutila and Salokangas (2010). In other words, circular hollow sections allow to employ much bigger wood contact area for shear flow distribution. Glued-in rods, therefore, can be viewed as the dowel type connectors characterized by a higher risk of the wood splitting in a normal service load regime and risk of plastic deformation of the wood in a long-term (Khorsandnia *et al.* 2014, 2015).

Other widely used connector type for wood-concrete composite elements is wood shear key or notched joint (Dias 2005). This is usually either a glued-on wood block or a rectangular carving applied directly to the upper surface of the glulam element. Carving is filled either with the concrete or fitted with metal elements of the same size and anchored in a concrete slab as for the *Birkberg* bridge in Rautenstrauch *et al.* (2010). Wood block or concrete key here serves as an obstacle resisting shear flow along its way. This connection type proved to be effective in terms of ultimate capacity and stiffness (Yeoh *et al.* 2011), however, without additional reinforcement, it may lack ductility feature, which ensures early signaling of unforeseen structural problems. Also negative effects of concrete shrinkage has not been quantitatively recognized yet on the loss of stiffness of shear key connections (Fragiacomo, Lukaszewska 2011). Usage of steel CHS connectors in this context can be viewed as a good technical compromise between ultimate strength, stiffness and post-peak behavior of the wood-concrete systems available for practical engineering.

This study is mainly based on the initial hypothesis that by gradually increasing the diameter of the CHS element also stiffness and ultimate capacity of the connector's system is increased. Connector with a higher stiffness *(EI)* in interaction with the wood ensures wider area of active/passive pressure mechanisms near the connector, hence better distribution of the shear forces. Connector's effectiveness towards the ultimate capacity or ductility requirements is controllable by changing the wall thickness of the hollow section, embedment depth in the wood and the in-fill type (in this case wood core or concrete in-fill core).

CHS elements were also researched before (Benitez 2010), but not much information is available regarding the quantitative structural limits of these connectors compared, for instance, to a dowel type or glued-in rod connector systems.

2. Preparation of the specimens

At first, having no information about structural limits of the CHS shear connectors, push-out tests were designed for three different diameters of circular hollow section connectors, i.e., external dimension of 42 mm, 60 mm and 72 mm (Fig. 1). Particular dimensions were chosen based on commercially available circular saw drills with appropriate height of at least 60 mm. Pre-holing for the CHS inserts can be easily done by hand instruments (Fig. 2). Although care has been taken to minimize any possible gap between the wood and connector by matching CHS with a drill size, two component adhesive was applied prior the instalment of the connectors in the glulam (class GL24h, spruce).

Conditioning of the timber was applied prior to the sample preparation and push-out tests, which was aimed to stabilize timber moisture levels in regards to Service class 2 (exterior, under cover conditions), see *Eurocode 5: Design of Timber Structures. General. Common Rules and Rules for Buildings*. Measured moisture levels of timber were in a range of 15–17%.

Wall thickness for all connectors are 2 mm; steel class S235. Tests were also further differentiated in two following groups:

1. Connectors with the wood core not removed (Fig. 3a). No interlayer between wood and concrete; concrete deck is cast directly on the surface of the glulam.

Fig. 1. CHS connectors prepared for a test setup

Fig. 2. Slot preparation for the CHS inserts

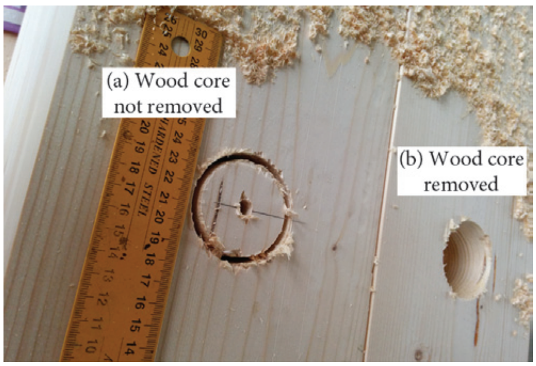

Fig. 3. Sample preparation with a wood core (a) and concrete in-fill core (b)

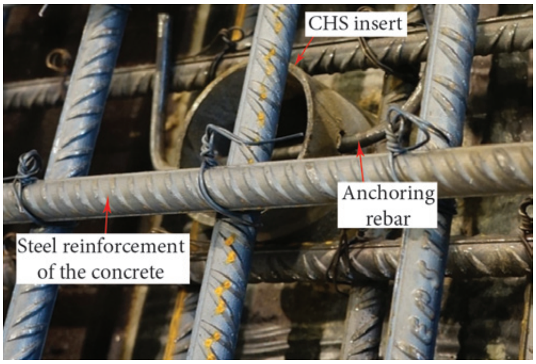

Fig. 4. Sample prepared for concrete casting

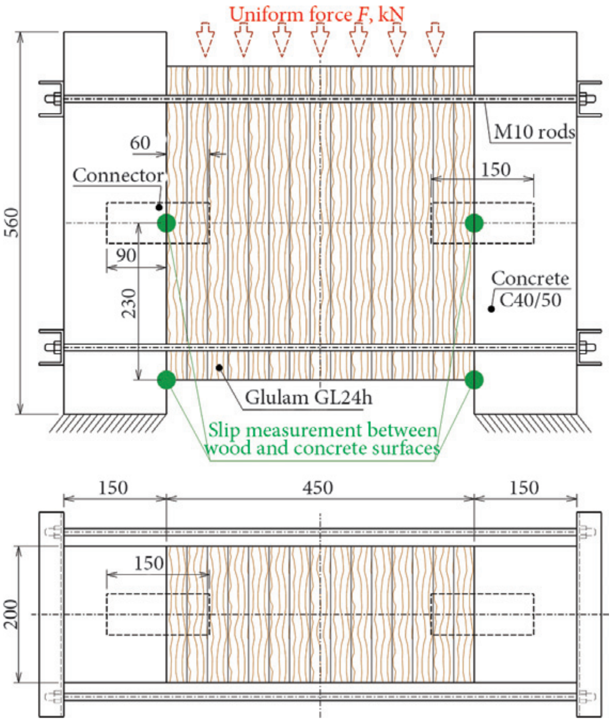

Fig. 5. Elevation and top view of the test setup

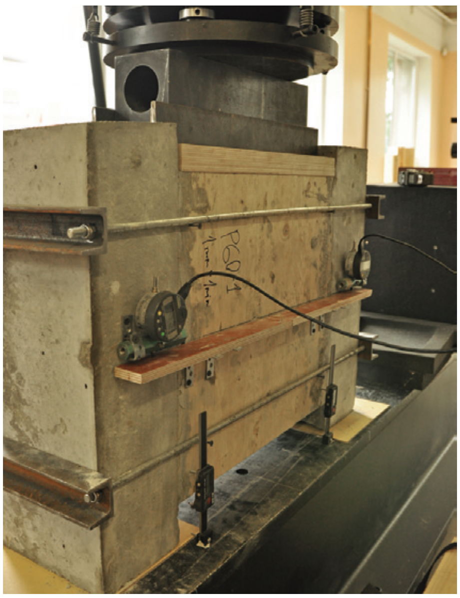

Fig. 6. Laboratory setup of the push-out test

2. Connectors without the wood core (Fig. 3b) and with an interlayer of self-adhesive waterproofing membrane (3 mm). After the borehole is done, wood core is chiseled out, CHS installed and filled with the wet concrete in the deck casting process (Fig. 4).

Membrane application is intended to limit moisture influence within the zone of connectors, which is inevitable during the wet casting of the concrete. Waterproofing membrane on the glulam upper surface can also be used in bridge design situations with a plain concrete deck, i.e., without waterproofing and asphalt layers. In such case waterproofing membrane on top of the glulam ensures minimal ingress of moisture from the deck.

Final layout of the test design and appropriate laboratory setup is depicted in Figs 5–6.

Loosely tightened M10 rods were used to secure positioning of the setup during the loading. Other aspects of sample preparation (including wood and concrete dimensions) were matched to the technology applicable to the real bridge construction situations.

Loading procedure was based on deformation/per time unit scheme. In this case rate of 2 mm/min was used (Instron 600 KN testing machine). Slip measurements were carried out using digital linear scales (Mahr, model 31 ES – accuracy 0.01 mm).

3. Results of the push-out tests

The main goal of the push-out tests is to establish quantitative characteristics of the particular connector for wood-concrete structural systems – it includes ultimate capacity in shear, slip values at different loading stages, linearity level and post-peak character of the system. Graphical results for both test groups are depicted in Figs 7–8. Sample identification is done by the connector's diameter subscripted by the connector's core type and sample number of the particular type, for instance $D76_{wood,1}$.

For timber design purposes, *Eurocode 5* distinguishes two types of variables characterizing shear stiffness of the connector (Porteuos, Kermani 2007). These are dependent on the design limit state used, namely slip modulus

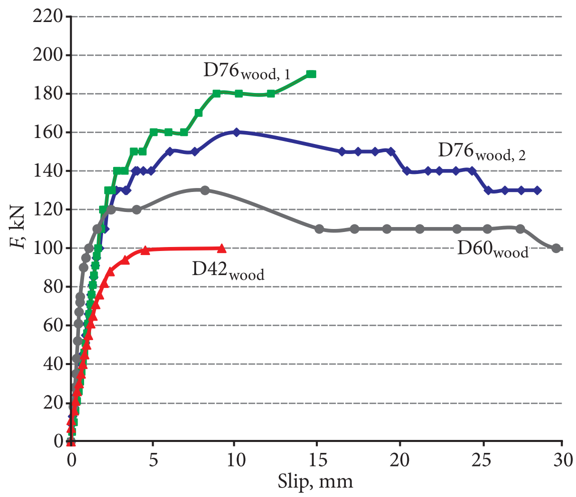

Fig. 7. Force-slip graph of the connectors with a wood core

Table 1. Performance parameters established for a single CHS connector

CHS connection type	F_{max}, kN	K_u, kN/mm	K_{ser}, kN/mm
$D42_{wood}$	100/2 = 50	25.5	27.6
$D42_{concrete}$	92/2 = 46	30.0	33.3
$D60_{wood}$	130/2 = 65	65.0	63.4
$D60_{concrete}$	168/2 = 84	27.5	26.2
$D76_{wood, 1}$	190/2 = 95	30.2	30.2
$D76_{wood, 2}$	160/2 = 80	30.6	30.8
$D76_{concrete}$	208/2 = 104	50.0	48.9

K_u and K_{ser}, which has respective values of slip at $0.6F_{max}$ and $0.4F_{max}$ for ultimate (*ULS*) and serviceability limit state (*SLS*). Values obtained in the tests of this study are summarized in Table 1.

Although laboratory setup is symmetric employing two connectors of the particular type, for practical design situations values in Table 1 are transformed for a single connector.

3.1. Failure modes of the CHS connectors

Amplitude of different failure modes of the tested connectors is best represented by post-failure pictures below (Figs 9–12).

3.2. Analysis of the results

Contrary to the typical nail and dowel type connectors (Jutila, Salokangas 2010), CHS connectors exhibited linear character at load levels even up 70–75% of the ultimate capacity F_{max}. In this case almost no initial slip has been experienced, which can be explained by the fact that connector's nominal size is the same as the circular drill size. Actually connectors were put in their slots by a slight hammering into their position in a glulam, so a local break-in of the wood material might had already been employed.

Due to the linear character of the shear response, values of the slip modulus K_u and K_{ser} are similar for each of the connectors, difference ranging from 0–11%. This aspect is convenient for structural modeling routine of timber-concrete structures as in both of the limit states (*ULS* and *SLS*) designer can utilize the same slip modulus, for instance, by applying appropriate shear spring value, either linear or non-linear depending on design situation (Khorsandnia *et al.* 2014). Tests confirmed initial presumption that specimen's ultimate shear capacity F_{max} is increasing with the diameter increase of the CHS connector. Besides tests indicate that connectors with concrete in-fill are superior compared to connectors with a wood core. Although it was easy to predict, purpose of the tests was to establish by what margin these differences are taking place.

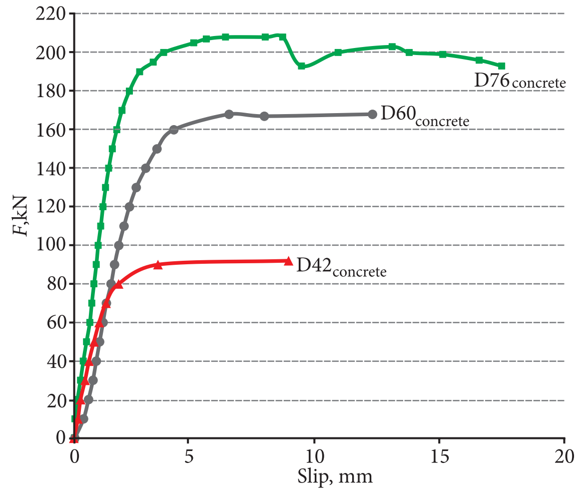

Fig. 8. Force-slip graph of the connectors with a concrete in-fill core

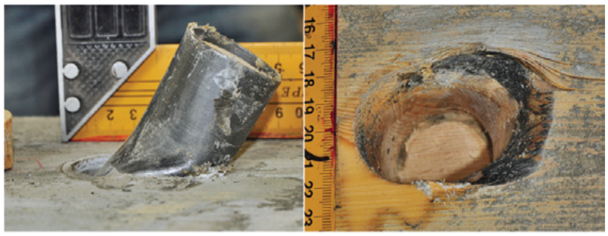

Fig. 9. $D42_{wood}$ sample failure (steel in shear and wood in compression)

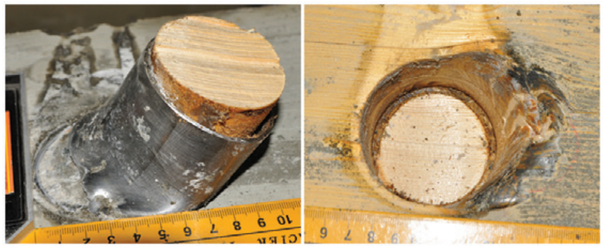

Fig. 10. $D60_{wood}$ sample failure (steel tearing at extreme slip levels and wood in compression)

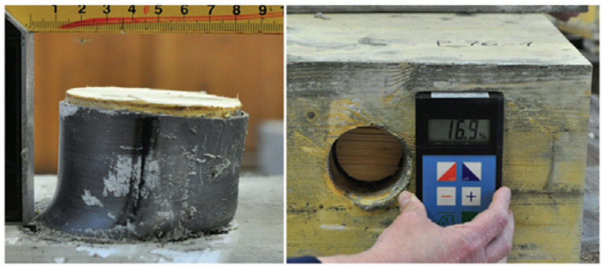

Fig. 11. $D76_{wood}$ sample failure (steel section in shear, wood in compression)

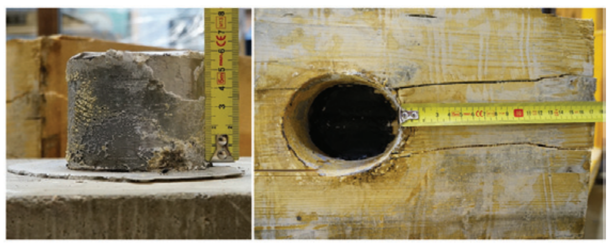

Fig. 12. $D76_{concrete}$ sample failure (steel intact, wood in compression and shear along the grain)

3.3. Stiffness of connectors

In order to compare connectors of both groups, recalculation of steel equivalent stiffness $(EI)_{equiv.}$ was done for wood core and concrete in-fill connectors. Recalculation resulted in an increase of hollow section's wall thickness depending on the material modulus of elasticity E_i and geometrical properties of the core material. This can be considered as a very practical approach for real design situations as plenty of detailing options can be chosen, based on the equivalent stiffness principle. Another advantage of this method is that within the normal range of loads, buckling of the equivalent steel hollow section can be ignored. This is due to the fact that wood core/concrete core encapsulation facilitates distribution of shear forces and prevents the steel section from local buckling. Stiffness properties used for calculation of equivalent stiffness are:

- E_{cm} = 35 GPa for concrete C40/50 (*EN 206:2014 Concrete – Specification, Performance, Production and Conformity*);
- $E_{0,mean}$ = 11.6 GPa for glulam GL24h *(EN 14080:2013 Timber Structures. Glued Laminated Timber and Glued Solid Timber. Requirements*) and
- E_{S235} = 210 GPa for steel connector (*EN 1993-1-1:2005: Eurocode 3: Design of Steel Structures – Part 1-1: General Rules and Rules for Buildings*).

Shear connector's ultimate capacity relationship to its core material and geometry can be well described by the logarithmic model depicted in Fig. 13. Regression coefficient

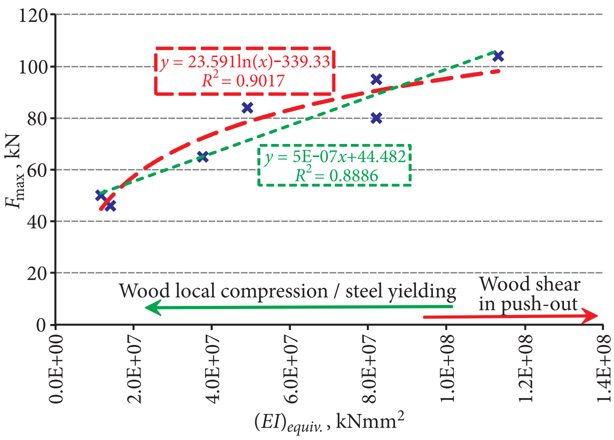

Fig. 13. Stiffness plot versus ultimate capacity of the shear connectors tested

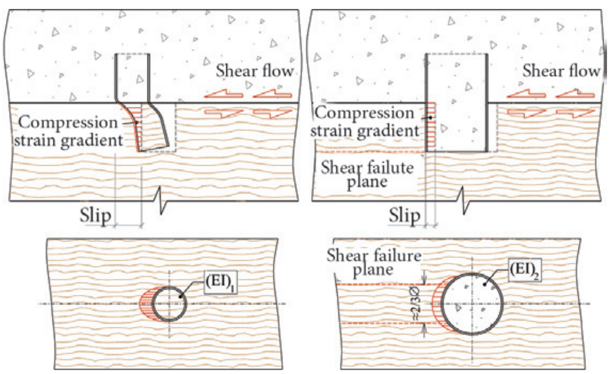

Fig. 14. Failure mechanism in case of $(EI)_1 < (EI)_2$

R^2 for this model is just above 0.9, however, it is applicable for this test series only with particular wood class and connector's embedment height in the wood material.

3.4. Post-peak behaviour

For composite element sections not only strength is what matters. Brittle failure has to be avoided by using elements with a certain degree of ductility. Within these tests all of the samples showed ductile post-peak behavior, ensuring that in real overload situations signals of excessive deformations can be timely noticed. All of the slip content was realized through the steel yielding and wood compression near the connector. In both of connector groups wood core and concrete in-fill kept the steel hollow section from an early wall buckling. However, attention is drawn to the sample $D76_{concrete}$, which requires further studies. This element is considerably stiffer than others and after 8mm slip dominant failure mode seems to be changing from wood compression (Figs 9, 10 and 11) to shear failure of the wood along the embedment height of connector (Fig. 12). This sample implies that additional control of the loaded shear area is necessary for very stiff CHS connectors. Shear failure planes might develop when reaching ultimate load levels; in this case, failure occurred at around 104 kN per connector.

Transition from ductile to brittle failure mechanism observed in this study is depicted in Fig. 14.

In small diameter connector with a small stiffness value, the angle of wood compression in the failed zone is around 45 degrees (see strain gradient in the left top of Fig. 14). Mostly the weak axis of the wood is exploited in this case. As the stiffness of the connector increases, angle of compression decreases. This is depicted in right top of Fig. 14, where strain gradient is uniformly distributed along the embedment height of the connector. Wood compression strength parallel to the grain is typically 8 to 9 times higher than compression perpendicular to the grain (standard *EN 14080:2013 Timber Structures – Glued Laminated Timber and Glued Solid Timber – Requirements*). With a very stiff connector dominant failure mode is no more governed by the wood compression strength, but more by the shear capacity of the wood. That explains development of shear failure planes of the sample $D76_{concrete}$ and brittle character of relatively stiff CHS connectors in general.

4. Discussion

Timber-concrete composite element sections in bridge engineering are rational structural systems with many attractive features such as prefabrication options, usage of local renewable materials, energy consumption savings, aesthetics etc. Structurally, the used connector type can greatly influence effectiveness of such composite element system. For design purposes, engineer needs to have some quantitative basis for connection modelling of timber-concrete structures. For instance, overestimation of stiffness in the design stage may lead to excessive deformations in real life service of the structure. This study demonstrates that wide amplitude of structural demands can be satisfied when using CHS connectors. In this case push-out tests were purposefully done for the lowest structural class of homogeneous glulam (GL24h), meaning that there can be a room for even better performance when using higher class of glulam material.

5. Conclusions

Based on this experimental study following conclusive findings are defined:

1. Arguments based on the stated hypothesis that by gradually increasing the diameter of the connector also stiffness and ultimate shear capacity can be increased proved to be valid.

2. Ultimate shear capacities observed within this study are ranging from 46 kN up to 104 kN per connector, which is appropriate for general bridge designs with small and medium span lengths.

3. Due to the linear-elastic character of the shear deformation, differences between slip modulus K for ultimate and serviceability limit states and are relatively small ranging from 0% to 11%.

4. Connector's wood core and concrete in-fill core both greatly facilitate CHS stability from local buckling and they also increase stiffness of the whole connection system.

5. Logarithmic relationship with a high regression coefficient of $R^2 = 0.9$ has been established between stiffness and ultimate shear capacity of the tested circular hollow section connectors. However, further studies are necessary for glulam class higher than GL24h as a shift in the ultimate shear capacity is most likely to be experienced.

6. Post-peak behavior of the circular hollow section connectors is perfectly plastic, although for the stiffest specimen ($D76_{concrete}$) transition to brittle failure in push-out shear has been documented. This implies that additional attention has to be paid to the longitudinal shear capacity of the glulam itself. In practice, it means that more shear area has to be provided by choosing appropriate distance between adjacent connectors and height of the embedment in the wood base material.

7. Round shape of connector's section allows for gradual distribution of loads locally with no excessive stress concentrations and wood splitting risk.

8. Cost effectiveness and workability of circular hollow section connectors are ensured due to the no expensive or energy consuming equipment involvement (welding, for instance). Typical hand tools such as drills and saw cuts can be used with no compromises to quality.

9. Circular hollow section connector systems have considerable practical potential in timber-concrete composite systems owing to their simple mechanical layout and structural performance characteristics.

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References

Bathon, L.; Bletz-Muhldorfer, O. 2010. Performance of Single Span Wood-Concrete-Composite Bridges Under Dynamic Loading, in *Proc. of International Conference on Timber Bridges (ITCB 2010)*. ICTB 2010 & Tapir Academic Press, Trondheim. Norway. 399–402.

Benitez, M. 2000. Development and Testing of Timber/Concrete Shear Connectors, in *Proc. of the 6th World Conference on Timber Engineering (WCTE 2000)*. Vancouver. British Columbia. Canada.

Dias, A. 2005. *Mechanical Behaviour of Timber-Concrete Joints*: PhD thesis. Delft University of Technology.

Dias, A.; Martins, A.; Simoes, L. 2013. Statistical Analysis of the Load Slip Behaviour in Timber-Concrete Connections, in *Proc. of the 14th International Conference on Civil, Structural and Environmental Engineering Computing*, Civil-Comp Press, Stirlingshire, UK. http://dx.doi.org/10.4203/ccp.102.166

Fragiacomo, M.; Lukaszewska, E. 2011. Development of Prefabricated Timber–Concrete Composite Floor Systems, *Structures and Buildings* 164(SB2): 117–129. http://dx.doi.org/10.1680/stbu.10.00010

Yeoh, D.; Fragiacomo, M.; De Franceschi, M; Buchanan, A. 2011. Experimental Tests of Notched and Plate Connectors for LVL-Concrete Composite Beams, *Journal of Structural Engineering* 137(2): 261–269. http://dx.doi.org/10.1061/(ASCE)ST.1943-541X.0000288

Jutila, A.; Salokangas, L. 2010. Wood-Concrete Composite Bridges – Finnish Speciality in the Nordic Countries, in *Proc. of International Conference on Timber Bridges (ITCB 2010)*. ICTB 2010 & Tapir Academic Press, Trondheim, Norway, 383–392.

Khorsandnia, N.; Schänzlin, J.; Valipour, H.; Crews, K. 2015. Coupled Finite Element-Finite Difference Formulation for Long-Term Analysis of Timber–Concrete Composite Structures, *Engineering Structures* 96: 139–152. http://dx.doi.org/10.1016/j.engstruct.2015.03.047

Khorsandnia, N.; Schänzlin, J.; Valipour, H.; Crews, K. 2014. Time-Dependent Behaviour of Timber–Concrete Composite Members: Numerical Verification, Sensitivity and Influence of Material Properties, *Construction and Building Materials* 66: 192–208. http://dx.doi.org/10.1016/j.conbuildmat.2014.05.079

Khorsandnia, N.; Valipour, H.; Crews, K. 2014. Structural Response of Timber-Concrete Composite Beams Predicted by Finite Element Models and Manual Calculations, *Advances in Structural Engineering: an International Journal* 17(11): 1601–1621. http://dx.doi.org/10.1260/1369-4332.17.11.1601

Miebach, F. 2014. Construction of Two New and Inventive Timber-Concrete-Composite Bridges in Lohmar Near Cologne/Germany. *COST – Timber Bridges Conference 2014*, Switzerland, 1–6. Available from Internet: http://www.ib-miebach.de/cms/upload/pdf/full_paper_CTBC2014_Biel_CH_Miebach_en.pdf

Porteuos, J.; Kermani, A. 2007. *Structural Timber Design to Eurocode 5*. Blackwell Publishing. 68 p. http://dx.doi.org/10.1002/9780470697818

Rautenstrauch, K.; Mueller, J.; Simon, A. 2010. The First Timber-Concrete Composite Road Bridge in Germany, in *Proc. of the 11th World Conference on Timber Engineering (WCTE 2010)*. Italy.

Rodrigues, J.; Dias, A.; Providencia, P. 2013. Timber-Concrete Composite Bridges: State-of-the-Art Review, *Bioresources* 8(4): 6630–6649. http://dx.doi.org/10.15376/biores.8.4.6630-6649

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