ENGINEERING TRANSACTIONS • *Engng. Trans.* • **66**, 2, 161–173, 2018
Polish Academy of Sciences • Institute of Fundamental Technological Research (IPPT PAN)
National Engineering School of Metz (ENIM) • Poznan University of Technology

Research Paper

Laboratory Tests of New Connectors for Timber-Concrete Composite Structures

Maciej SZUMIGAŁA, Ewa SZUMIGAŁA, Łukasz POLUS

Faculty of Civil and Environmental Engineering
Poznan University of Technology
Piotrowo 5, 60-965 Poznan, Poland
e-mail: lukasz.polus@put.poznan.pl

This paper presents the results of the laboratory tests of new connectors for timber-concrete composite (TCC) structures. These connectors can be used to join a timber beam with a concrete slab. They consist of two parts – a headed stud and a steel screw. The objective of our analysis was to examine the stiffness and strength of the connection, which can be used in designing TCC beams. The authors introduced formulas for calculating the above mentioned parameters. The results obtained in the experimental tests are compared with those calculated using the proposed formulas.

Key words: composite structures; timber-concrete composite beams; shear connectors; laboratory tests.

1. Introduction

Wood is often used for structural elements and is very competitive to other materials used in the construction market, because of its high strength-to-weight ratio [1] and the prevalence of light-weight structures [2], in which timber is widely used. Wood is one of the oldest structural materials, and it is continuously being improved, e.g., Dudziak *et al.* studied the process of wood plasticization by hot rolling [3, 4], whereas Rapp and Fiszer studied adhesive scarf joints in wooden beams [5, 6].

Glued-laminated timber is a relatively new structural engineered wood product, which enables the creation of structural members much larger than trees [7]. In addition, timber exposed to fire can carry loads for a long time, because of the big cross-section area of the timber beam and the slow rate of charring of the beam's surfaces. In addition, the fire resistance of glued-laminated timber can be significantly improved by the application of fire-resistant surface coatings or pressure impregnation with fire retardants. Therefore, glued-laminated timber can be used for building bridges [8] and roof systems for single-storey warehouses, shopping centres and factories.

The load-bearing capacity and stiffness of a timber beam can be improved by joining the timber beam with a concrete slab [9] (see Fig. 1).

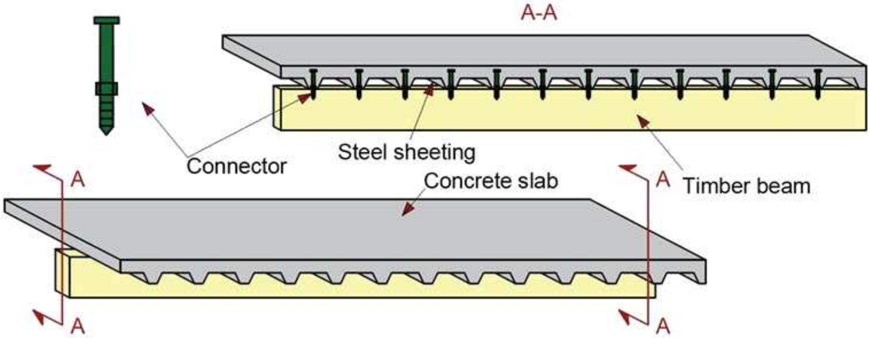

Fig. 1. Timber-concrete composite beam.

The floors of timber-concrete structures have better acoustic and thermal properties as well as higher fire resistance than wooden floors. Timber-concrete composite structures compete well with steel-concrete composite structures presented in [10–12] and less-known aluminium-concrete composite structures presented in [13, 14]. Concrete may be poured into steel sheeting, which may serve both as formwork and tensile reinforcement [15]. The timber beam and the concrete slab should be joined together using shear connectors. The shear connectors used in timber-concrete structures are presented in [16–19]. Gutkowski *et al.* presented laboratory tests of TCC beams with notched shear/key anchor connection [20, 21]. A composite connector for TCC structures was proposed in [22]. It consisted of a composite cylinder made of ultra-high performance fibre-reinforced concrete shell with a cylindrical steel core. The failure modes of this connector are associated with the shearing of the connector, the pulling-out of the steel core from the connector head and the pulling-out of the steel core from the connector shank. The interaction between the timber and the concrete can also be ensured by a glued-in connection, e.g., a perforated mesh made of plywood [23], corrugated re-bars [24] or a perforated steel plate [25]. The analysis of the load-carrying capacity and stiffness of timber-concrete joints made of dowel-type fasteners was presented in [26, 27]. Łukaszewska *et al.* presented a composite system in which the concrete slab was prefabricated off-site, with the connectors embedded and then connected on-site to the timber joists [28]. The prefabrication of the concrete slab reduces construction costs and the impact of the concrete shrinkage on the deflection of the composite beam. The value of the shrinkage strain of concrete has an effect on the bearing strength of a connection with flexible connectors. The problem of concrete shrinkage was presented in [29–31].

To model the behaviour of the TCC beam, the linear elastic model referred as EC 5 in Annex B of [32] may be used. The stiffness and the ultimate strength of the connection are the necessary information for designing TCC beams. In this study, the authors developed a new connector which consisted of two parts – a headed stud and a steel screw (see Fig. 1). This solution is similar to the commonly used screws and dowels, and therefore it combines the advantages of both. Ceccotti classified screws as the least rigid [33], yet Lukaszewska showed that they are the most ductile as well [17]. The proposed connector has a larger diameter (20 mm) than the normally used screws (10, 12, 16 mm). The headed stud of this connector is embedded in the concrete slab and it can be of any height. A steel hexagon flange which tightens the steel sheeting to the timber beam and enables screwing makes this novel connector usable in TCC beams with profiled steel sheeting. This solution was never presented before – it is original and practical. The authors prepared two push-out tests to evaluate the behaviour of the new connectors. There is no standard for the assessment of TCC connections [19]. The ultimate capacity of the connector and the stiffness of the connection are often obtained from a push-out test using the loading procedure presented in EN ISO 26891 [34]. However, the traditional EN ISO 26891 procedure was not used due to the possibility of premature cracks appearing in the concrete slabs. If the premature cracks had appeared, the load would have decreased for a moment. This could stop the test. The tests were carried out in accordance with the principles set out in EN 1994-1-1 [35] and using constant displacement control. When a connector for steel-concrete composite structures is tested, its resistance may be the result of the shear of the connector or the destruction of the concrete part [36]. These failure mechanisms may also occur when testing connectors for timber-concrete structures. Additionally, the timber part surrounding the connector may also become damaged in the analysed structures. Due to the fact that push-out tests are burdened with the risk of premature cracks, the concrete slabs should be thick enough to prevent them [37]. In the analysed models, the concrete slabs were 150 mm thick, and they were made of high-performance concrete to prevent premature cracks. Nowadays, the use of high-performance concrete is prevalent. Denisiewicz and Kuczma presented reactive powder concrete with large compressive strength – 140 MPa [38]. Schafers and Werner used ultra-high performance concrete for timber-concrete composite beams [39]. The use of high-strength concrete makes it possible to reduce the thickness of the concrete slab.

The test configuration can have an impact on results. Researchers use three test configurations: pure shear, double shear push-out and single shear push-out [19]. In the double shear test, two timber elements and one concrete element are used more often than two concrete elements and one timber element. Carvalho and Carraso investigated the impact of the specimen arrangement on the strength and deformations of TCC joints [40]. They observed that the concrete-wood-concrete specimen had the best strength of the TCC connection. The same configuration was used in this study.

2. Theoretical background

The resistance of the shear connector in the laboratory test was calculated using Eq. (2.1) and following the principles set out in [35]:

$$P_{Rd,\,\mathrm{test}} = \frac{0.9P}{\gamma_v}, \tag{2.1}$$

where P is the minimum failure load divided by the number of shear connectors and γ_v is the partial factor.

In Eurocode 4, the design strength of a headed stud shear connector in composite beams with ribs transverse to the supporting beams should be calculated as the lesser of Eqs. (2.2) and (2.3):

$$P_{Rd,a} = k_t \frac{0.8 f_u A_{sc}}{\gamma_v}, \tag{2.2}$$

$$P_{Rd,b} = k_t \frac{0.29 \alpha d^2 \sqrt{f_{ck} E_{cm}}}{\gamma_v}, \tag{2.3}$$

where A_{sc} is the cross-section area of a stud shear connector, f_u is the ultimate strength of steel, f_{ck} and E_{cm} are the cylindrical compressive strength and mean secant (elastic) modulus of concrete respectively, and k_t and α are the reduction factors specified in Eurocode 4.

Equations (2.2) and (2.3) take into account only the strength of the connector and concrete. This is not enough. It is also necessary to take into consideration the strength of timber. The theory of timber connections was presented by Johansen in [41]. The formulas for predicting the load-carrying capacity of dowels used in TCC elements were presented in [26]. These formulas are similar to the equations developed for steel-timber joints and presented in Eurocode 5 [32]. The Eurocode 5 formulas are suggested for calculating the resistance of connectors used in timber-concrete structures:

$$P_{Rd,c} = \frac{k_{\mathrm{mod}} f_{h,k} t d}{\gamma_M}, \tag{2.4}$$

$$P_{Rd,d} = k_{\mathrm{mod}} \frac{f_{h,k} t d \left[\sqrt{2 + \frac{4M_{y,Rk}}{f_{h,k} t^2 d}} - 1\right] + \frac{F_{ax,Rk}}{4}}{\gamma_M}, \tag{2.5}$$

$$P_{Rd,e} = k_{\mathrm{mod}} \frac{2.3\sqrt{M_{y,Rk} f_{h,k} d} + \frac{F_{ax,Rk}}{4}}{\gamma_M}, \tag{2.6}$$

$$P_{Rd,f} = \frac{k_{\mathrm{mod}} 0.4 f_{h,k} t d}{\gamma_M}, \tag{2.7}$$

$$P_{Rd,e} = k_{\mathrm{mod}} \frac{1.15\sqrt{2M_{y,Rk} f_{h,k} d} + \frac{F_{ax,Rk}}{4}}{\gamma_M}, \tag{2.8}$$

where $f_{h,k}$ is the characteristic embedment strength of the timber, t is the penetration depth, d is the diameter of the connector, $M_{y,Rk}$ is the characteristic fastener yield moment, $F_{ax,Rk}$ is the characteristic withdrawal capacity of the fastener, k_{mod} is the modification factor for duration of load and moisture content, and γ_M is the partial factor for material properties.

Thanks to the tests, not only the resistance but also the ductility of the shear connector can be evaluated. Deam *et al.* suggested that a connection can be defined as ductile if it can withstand a relative slip of 10 mm without a reduction in strength exceeding 20% of the peak value [42]. The slip modulus $k_{0.4}$ per connector may be calculated from [32] as

$$k_{0.4} = \rho_m^{1,5} d/23. \tag{2.9}$$

Dias *et al.* analysed the stiffness of dowel-type fasteners in [27], and they arrived at the best correlations for the Eurocode 5 model presented in [32]. The stiffness of the connection is approximately linearly proportional to the dowel diameter [43].

3. Experimental investigation

The tests were carried out on two models shown in Fig. 2. The tested specimens differed in the shape of concrete slab. The authors analysed two types of concrete slabs: one poured into open through a profiled steel sheeting placed in

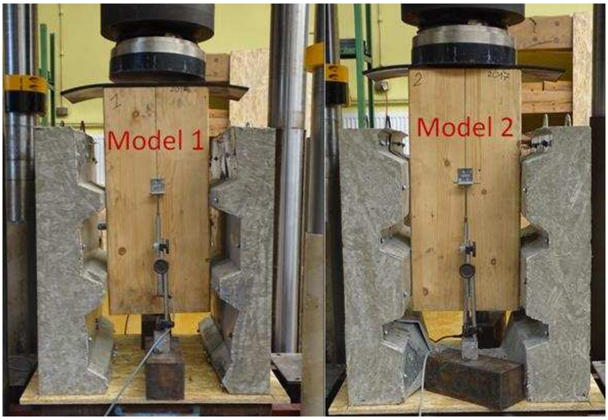

Fig. 2. Models placed on the machine base.

an upward position (model 1) and another poured into open through a profiled steel sheeting placed in a downward position (model 2).

An experimental model consisted of two concrete slabs made of C60/75 concrete, two steel sheets made of 0.7 mm-thick S320GD steel, four steel shear connectors (20 mm), four reinforcing meshes made of 6 mm S235JRG2 steel round bars and a timber beam made of GL28h timber. The concrete slabs were simultaneously cast in the vertical position. The fasteners were embedded 100 mm deep both in concrete and in timber. The connector consisted of two parts – a headed stud made of grade S235J2 steel (19×90 mm) and a steel screw (hexagon-head wood screw DIN 571 4.6, hot dip galvanised, 20×100 mm). The tests were performed 35 days after the casting and using the Instron 8500 Plus test machine. The models were put on a 12 mm-thick OSB board placed on the machine base. The load ranging between 10 kN and 50 kN was applied cyclically 25 times, and subsequently failure load was applied. The longitudinal slip between the concrete slabs and the timber beam was measured continuously during loading using LVDTs. Measurements were also performed when the load decreased. Constant displacement control was used and the piston velocity was 1.0 mm/min. The location of sensors is presented in Fig. 3.

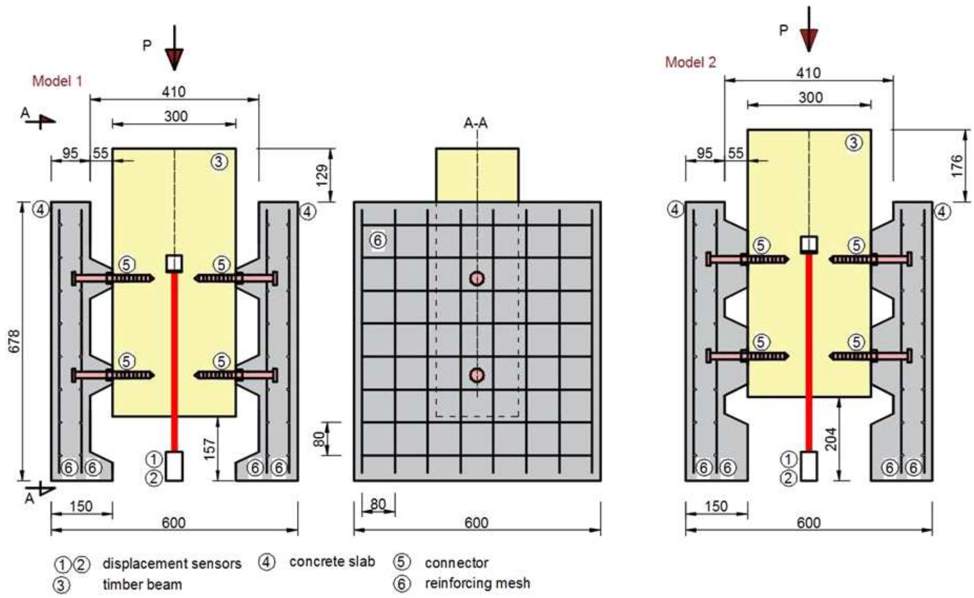

FIG. 3. Constructional details of experimental models.

4. RESULTS

The vertical displacement of the timber beam for every model is presented in Fig. 4. It shows the arithmetic mean of the displacement measured by two

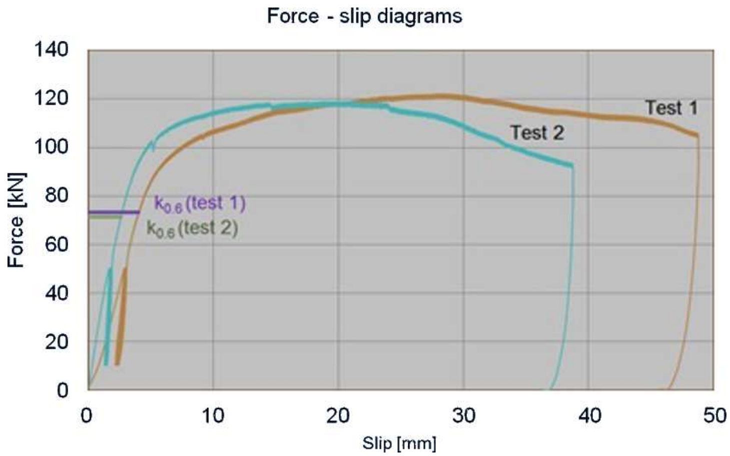

FIG. 4. Force-slip diagrams from push-out tests.

sensors. These line plots also present the stage when the cyclical load was applied. The maximum shear force for the first model was 122.1 kN, with the corresponding slip of 27.9 mm, while the maximum load for the second model was 119.1 kN, with the corresponding slip of 18.7 mm. The mean value of the maximum load was 120.6 kN. The deviation of any individual test result from the mean value obtained from both tests did not exceed 2%. In the first test, when the slip was 37.9 mm, the force was 113.7 kN. This connection was ductile, because a reduction in strength (6.9%) did not exceed 20% of the peak value of the force. In the second test, when the slip was 28.7 mm, the force was 112.0 kN. This connection was also ductile, with a reduction in strength of 6.0%.

The push-out tests showed one distinctive mode of failure (see Fig. 5). This mode was associated with the crushing of the timber (see Fig. 6) and the formation of one plastic hinge within the connector at the timber-concrete interface (see Fig. 7). No cracks appeared in the concrete slab throughout the tests because high strength concrete was used (the mean compressive cube strength of the concrete was 84.4 MPa). When the load reached 88% of the ultimate value, the profiled sheeting in the ribs separated from the concrete only in the first test. The design resistance of a single connector from formula (2.1) was 22.0 kN in test 1 and 21.4 kN in test 2. The slip modulus $k_{0.4}$ was read before the cyclic load was applied. The $k_{0.4}$ stiffness of the connector was 4.4 kN/mm in the first test and 7.1 kN/mm in the second test, while the $k_{0.6}$ stiffness of the connector was 4.6 kN/mm in the first test and 6.9 kN/mm in the second test. In most cases, the $k_{0.6}$ stiffness is lower than the $k_{0.4}$ stiffness. In this article, the $k_{0.4}$ stiffness was higher than the $k_{0.6}$ stiffness in test 2, but it was lower than the $k_{0.6}$ stiffness in test 1. A similar situation was presented in [17]. There, ŁUKASZEWSKA showed that for an SST+S connection (coach screw with steel tube) the $k_{0.6}$

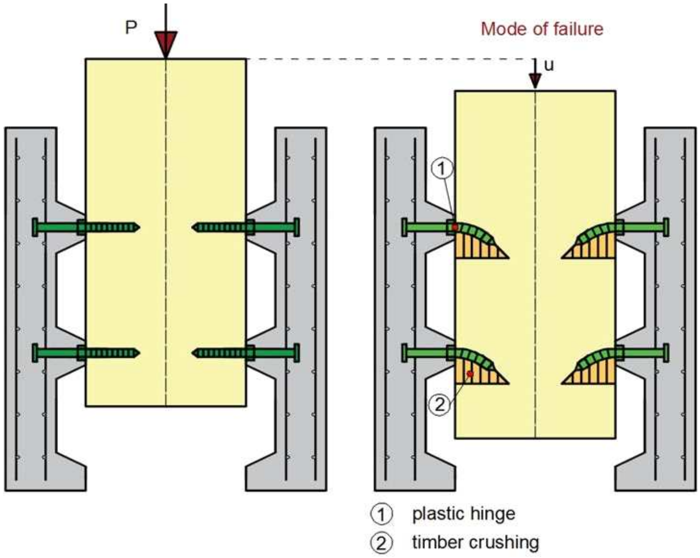

FIG. 5. The failure mode in timber-concrete joints.

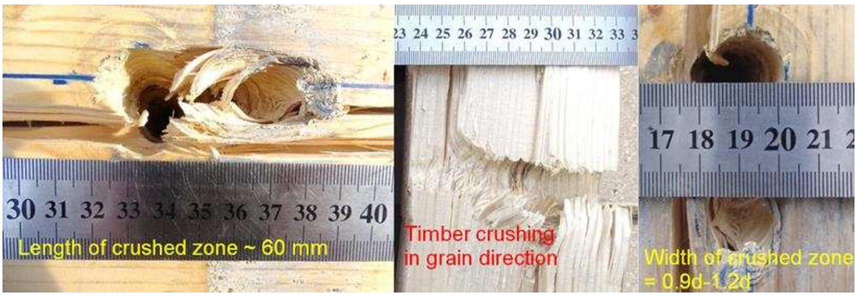

FIG. 6. Timber failure.

FIG. 7. The failure mode in the connector.

stiffness was higher than the $k_{0.4}$ stiffness. The slip modulus of a connection for the ultimate limit state, k_u, should be assumed as 2/3 of the $k_{0.4}$ stiffness [32]. Ceccotti suggested using the $k_{0.6}$ stiffness as the slip modulus k_u [33]. The authors in this article, as a result of obtaining a high value of the $k_{0.6}$ stiffness, proposed to assume k_u as 2/3 of the $k_{0.4}$ stiffness for the analysed connectors. The slip modulus $k_{0.4}$ calculated from Eq. (2.9) was 7.6 kN/mm. The ratio of the slip modulus $k_{0.4}$ calculated from Eq. (2.9) to the slip modulus $k_{0.4}$ obtained in test 2 was 1.07, which is a good correlation. The ratio of the slip modulus $k_{0.4}$ calculated from Eq. (2.9) to the slip modulus $k_{0.4}$ obtained in test 1 was 1.7. This poor correlation may be connected with the nonlinear behaviour of the connection during the initial part of the test (see Fig. 4).

Consequently, the authors determined the design strength of the connector. Table 1 presents the data used for calculations. The resistance calculated from Eqs. (2.1) through (2.8) is presented in Table 2.

Table 1. Data used for calculations.

Parameter	Value
Reduction factor k_t	0.75
Ultimate strength of steel f_u	400.0 MPa
Reduction factor α	1.0
Cylindrical compressive strength of concrete f_{ck}	60.0 MPa
Cross-sectional area of the wood screw A_{sc}	3.14 cm^2
Partial factor γ_v	1.25
Diameter of the wood screw d	20 mm
Mean secant (elastic) modulus of concrete E_{cm}	39.0 GPa
Density of timber ρ	425 kg/m^3
Modification factor for duration of load and moisture content k_{mod}	0.9
Partial factor for material properties γ_M	1.25
Penetration depth t	10.0 cm
Characteristic embedment strength of the timber $f_{h,k} = f_{h,0,k}$	27.9 MPa
Characteristic withdrawal capacity of the fastener $F_{ax,Rk}$	0.0 kN
Characteristic fastener yield moment $M_{y,rk}$	29.0 kN · cm

Table 2. The design strength of the connector from Eqs. (2.1) through (2.8).

Model	The design strength of the connector [kN] calculated from Eq.							
	(2.1)	(2.2)	(2.3)	(2.4)	(2.5)	(2.6)	(2.7)	(2.8)
1	22.0	60.3	106.5	40.1	19.5	21.0	16.1	14.9
2	21.4	60.3	106.5	40.1	19.5	21.0	16.1	14.9

The design strengths obtained from Eqs. (2.2) and (2.3) were much higher than the ones obtained from the tests using Eq. (2.1). Equation (2.2) is connected with the shearing of the connector, whereas Eq. (2.3) is connected with the crushing of the concrete. These failures were not observed during the tests. The lowest design strength, usually used for the steel-timber connection with a thin steel plate, was obtained from Eqs. (2.7) and (2.8). The concrete slab was thick enough to be analysed as a thick element. The ratio of the design resistance of the shear connector obtained from Eq. (2.4) to the resistance of the shear connectors obtained in test 2 was 1.87. Equation (2.4) is associated with timber crushing only. The design strength obtained from Eqs. (2.5) and (2.6) was similar to the one obtained from the tests. Equation (2.6) is associated with the timber crushing and the formation of two plastic hinges, whereas the failure mode was associated with the timber crushing and the formation of one plastic hinge within the connector. This latter mode is associated with Eq. (2.5) (see Fig. 8.3 in EC 5 [32]). During calculations, the authors did not take into account the rope effect ($F_{ax,Rk}/4 = 0.0$). In the design process, the design strength of the connector as the smallest result of Eq. (2.2) through (2.8) – 16.1 kN, was assumed. The ratio of the design resistance of the connector obtained from the proposed equations to the design resistance of the connector obtained in the tests was 0.70 ($14.9/21.4 = 0.70$).

5. Conclusions

This article discussed the results of the tests of new connectors for timber-concrete composite structures. The original contributions of this work include:

- determining the stiffness and strength of the new connector for TCC beams,
- proposing to calculate the design resistance and stiffness of the connector using formulas from EC 4 and EC 5.

The connection was ductile, which was a significant advantage. However, the design strength of the connector was not high, because the resistance of the timber limited it. The failure mode was connected with the crushing of the timber and the formation of one plastic hinge within the connector at the timber-concrete interface. The authors emphasize that the two studied specimens are a very small sample, and future tests should focus on connections having different diameters and on using such connections in composite beams. With the current state of knowledge, the proposed method of calculating design strength of the connector (as the minimum from Eqs. (2.2) to (2.8)) is worth considering when designing this type of connections.

Acknowledgment

Financial support by the grant 01/11/DSMK/0906 and 01/11/DSPB/0805 is kindly acknowledged. The authors would like to thank the employers of the laboratory of the Institute of Structural Engineering at the Poznan University of Technology for their help during the tests.

References

1. Ramage M.H., Burridge H., Busse-Wicher M., Fereday G., Reynolds T., Shah D.U., Wu G., Yu L., Fleming P., Densley-Tingley D., Allwood J., Dupree P., Linden P.F., Scherman O., *The wood from the trees: The use of timber in construction*, Renewable and Sustainable Energy Reviews, **68**: 333–359, 2017.
2. Chybiński M., Garstecki A., *Diagonal versus orthogonal ribs in stability of steel I beams*, Procedia Engineering, **172**: 172–177, 2017.
3. Dudziak M., Malujda I., Talaśka K., Łodygowski T., Sumelka W., *Analysis of the process of wood plasticization by hot rolling*, Journal of Theoretical and Applied Mechanics, **54**(2): 503–516, 2016.
4. Malujda I., *Refinement of thin layer of wood in hot rolling process*, Journal of Research and Applications in Agricultural Engineering, **50**(3): 32–35, 2005.
5. Rapp P., Fiszer S., *Approximate numerical model for adhesive scarf joints in wooden beams*, Drewno, **58**(194): 5–21, 2015.
6. Rapp P., *Application of adhesive joints in reinforcement and reconstruction of weakened wooden elements loaded axially*, Drewno, **59**(196): 59–73, 2016.
7. Moody R., Hernandez R., *Glued-Laminated Timber*, [in:] *Engineered wood products – A guide for specifiers, designers and users*, S. Smulski [Ed.], PFS Research Foundation, pp. 1–39, 1997.
8. Rodrigues J.N., Dias A.M.P.G., Providencia P., *Timber-concrete composite bridge: State-of-the-art review*, BioResources, **8**(4): 6630–6649, 2013.
9. Szumigała E., Szumigała M., Polus Ł., *A numerical analysis of the resistance and stiffness of the timber and concrete composite beam*, Civil And Environmental Engineering Reports, **15**(4): 139–150, 2014.
10. Pełka-Sawenko A., Wróblewski T., Szumigała M., *Validation of computational models of steel-concrete composite beams*, Engineering Transactions, **64**(1): 53–67, 2016.
11. Szewczyk P., Szumigała M., *The trial of optimal strengthening of composite beams*, [in:] Recent Advances in Computational Mechanics – Proceedings of the 20th International Conference on Computer Methods in Mechanics, CMM 2013, Łodygowski T. *et al.* [Eds.], CRC Press., pp. 277–283, 2014.
12. Jankowiak I., Madaj A., *Composite steel-concrete beam with concrete slab in tension* [in Polish], Inżynieria i Budownictwo, **67**(12): 667–671, 2011.
13. Szumigała M., Polus Ł., *Applications of aluminium and concrete composite structures*, Procedia Engineering, **108**: 544–549, 2015.

14. Szumigała M., Polus Ł., *A numerical simulation of an aluminium-concrete beam*, Procedia Engineering, **172**: 1086–1092, 2017.

15. Holomek J., Bajera M., Vilda M., *Test arrangement of small-scale shear tests of composite slabs*, Procedia Engineering, **161**: 716–721, 2016.

16. Łukaszewska E., Johnsson H., Fragiacomo M., *Performance of connections for prefabricated timber-concrete composite floors*, Materials and Structures, **41**(9): 1533–1550, 2008.

17. Łukaszewska E., *Development of prefabricated timber-concrete composite floors*, Doctoral Thesis, Lulea University of Technology, Lulea, 2009.

18. Benitez M.F., *Development and testing of timber/concrete shear connectors*, Proceedings of the 51st International Convention of Society of Wood Science and Technology, November 10–12, Concepción, Chile, 2008.

19. Dias A.M.P.G., Monteiro S.R.S., Martins A.G.D., *Summary of shear connector methods for timber-concrete composites*, Innovative timber composites: Improving wood with other materials, 17 October 2013, Nicosia, Cyprus.

20. Gutkowski R., Brown K., Shigidi A., Natterer J., *Laboratory tests of composite wood-concrete beams*, Construction and Building Materials, **22**: 1059–1066, 2008.

21. Leborne M.R., Gutkowski R.M., *Effect of various admixtures and shear keys in wood-concrete composite beams*, Construction and Building Materials, **24**: 1730–1738, 2010.

22. Auclair S.C., Sorelli L., Salenikovich A., *A new composite connector for timber-concrete composite structures*, Construction and Building Materials, **112**: 84–92, 2016.

23. Daňková J., Mec P., Majstríková T., *Stiffness analysis of glued connection of timber-concrete structure*, Open Engineering, **6**(1): 241–249, 2016.

24. Ceccotti A., Fellow R., Fragiacomo M., Giordano S., *Long-term and collapse tests on a timber-concrete composite beam with glued-in connection*, Materials and Structures, **40**(1): 15–25, 2006.

25. Miotto J.L., Dias A.A., *Glulam-concrete composites: experimental investigation into the connection system*, Materials Research, **14**(1): 53–59, 2011.

26. Dias A.M.P.G., Lopes S.M.R., Van De Kuilen J.W.G., Cruz H.M.P., *Load-carrying capacity of timber-concrete joints with dowel-type fasteners*, Journal of Structural Engineering, **133**(5): 720–727.

27. Dias A.M.P.G., Cruz H.M.P., Lopes S.M.R., Van De Kuilen J.W.G., *Stiffness of dowel-type fasters in timber-concrete joints*, Structure and Buildings, **163** (SB4): 257–266.

28. Łukaszewska E., Fragiacomo M., Johnsson H., *Laboratory tests and numerical analyses of prefabricated timber-concrete composite floors*, ASCE Journal of Structural Engineering, **136**(1): 46–55.

29. Flaga K., *The influence of concrete shrinkage on durability of reinforced structural members*, Bulletin of the Polish Academy of Sciences Technical Sciences, **63**(1): 15–22, 2015.

30. Furtak K., *Evaluation of the influence of shrinkage strain on the fatigue strength of the connection in steel–concrete composite beams*, Archives of Civil and Mechanical Engineering, **15**(3): 767–774, 2015.

31. Jasiczak J., Szymański P., Nowotarski P., *Computerised evaluation of the early age of shrinkage in concrete*, Automation in Construction, **49**(A): 40–50, 2015.

32. European Committee for Standardization, EN 1995-1-1. *Eurocode 5. Design of timber structures – Part 1-1: General – Common rules and rules for buildings*, Brussels 2004.

33. Ceccotti A., *Timber-concrete composite structures*, [in:] *Timber Engineering. Step 2*, Blass H.J. *et al.* [Eds.], Centrum Hout, Chapter E13, 1995.

34. European Committee for Standardization, EN-26891. *Timber structures, joints made with mechanical fasteners, general principles for the determination of strength and deformation characteristics*, Brussels, 1991.

35. European Committee for Standardization, EN 1994-1-1. *Eurocode 4. Design of composite steel and concrete structures – Part 1-1: General rules and rules for buildings*, Brussels, 2004.

36. Lorenc W., Kubica E. Kożuch M., *Testing procedures in evaluation of resistance of innovative shear connection with composite dowels*, J. Archives of Civil and Mechanical Engineering, **10**(3): 51–63, 2010.

37. Polus Ł., Szumigała M., *Tests of shear connectors used in aluminium-concrete composite structures*, [in:] *Recent Progress in Steel and Composite Structures*, Giżejowski M., Kozłowski A., Marcinowski J., Ziółko J. [Eds.], CRC Press-Taylor & Francis Group: Boca Raton, pp. 133–136, 2016.

38. Denisiewicz A., Kuczma M., *Two-scale modelling of reactive powder concrete. Part III: Experimental tests and validation*, Engineering Transactions, **63**(1): 55–76, 2015.

39. Schafers M., Werner S., *Investigation on bonding between timber and ultra-high performance concrete (UHPC)*, Construction and Building Materials, **25**(7): 3078–3088, 2011.

40. Carvalho E.P., Carrasco E.V.M., *Influence of test specimen on experimental characterization of timber-concrete composite joints*, Construction and Building Materials, **24**: 1313–1322, 2010.

41. Johansen K.W., *Theory of timber connections*, Publication 9, International Association of Bridge and Structural Engineering, Bern, Switzerland, 249–262.

42. Deam B.L., Fragiacomo M., Buchanan A.H., *Connections for composite concrete slab and LVL flooring systems*, Materials and Structures, **41**: 495–507, 2008.

43. Gelfi P., Giuriani E., Marini A., *Stud shear connection design for composite concrete slab and wood beams*, Journal Structural Engineering, **128**(12): 1544–1550, 2002.

Received September 13, 2017; accepted version February 12, 2018.