2005

Shear and bending performance of a novel wood-concrete composite system

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Shear and Bending Performance of a Novel Wood–Concrete Composite System

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Abstract: This paper introduces a new, structural wood–concrete composite system. The system is formed by joining a wood component, such as a floor beam or laminated plate, to a concrete slab utilizing a continuous steel mesh of which one half is glued into a slot in the wood while the other half is embedded into the concrete. Two series of tests were performed and are presented: static push-out tests (to establish shear properties of the connector) and a full scale bending test with a span of approximately 10 m. Test results reveal that the steel mesh performs favorably—as a stiff yet ductile shear connector between the wood and the concrete. Design equations, per European standards (in absence of North American standards) are described and used to predict the failure load of the bending test. Calculations indicate that the tested beam performs with near full composite action—specifically, 97% effective stiffness and 99% strength of that of a beam with full composite action. This is a marked improvement in the efficiency of wood–concrete systems developed to date. The system shows itself to be superior to alternative systems in its high structural efficiency as well as being relatively easy to install and economic.

DOI: 10.1061/(ASCE)0733-9445(2005)131:9(1404)

CE Database subject headings: Composite structures; Concrete slabs; Connections; Experimental data; Interface shear; Wood; Bending.

Introduction

Wood–concrete composite systems are engineered to benefit from composite action in much the same manner as steel concrete composite floor systems or reinforced concrete members. The composite action that these systems offer results in a substantial improvement to stiffness and strength of the overall structure in comparison to when the materials act independently. In a wood concrete system, the concrete component (slab) is designed to resist primarily compression stresses, while the wood component (a beam or plate) is used mainly in tension providing exceptional strength relative to added weight of the overall composite.

The degree of composite action between the wood and the concrete depends on the stiffness of their interfacial connection. Soft connectors, which allow substantial horizontal slip at the junction between the two materials, produce only a small degree of structural efficiency. Conversely, the highest degree of efficiency is found by using rigid connectors. A large variety of connector systems currently exist—such as nails, screws or spikes, concrete shear keys or proprietary devices—each with varying degrees of structural efficiency as well as level of ease and cost of installation (Ceccotti 1995; Blaß and Schlager 1996; Avak and Glaser 2002; Clouston et al. 2004). The challenge lies in developing a shear connection system that provides minimal slip between the two components while at the same time allowing a simple and inexpensive assembly.

A primary application for the wood concrete composite is in the use of floor systems—in housing units, schools and public buildings. In this setting, advantages are found through improvement of sound and vibration performance as a result of the added mass of the concrete. The system also performs well in terms of fire resistance. Concrete is naturally resistant to fire while large timbers form a char to naturally insulate both the timber core and the shear connector from direct exposure to fire. A further advantage is that the concrete slab inherently provides diaphragm action to resist lateral loading. Acknowledgement of these advantages has prompted an increased use of wood–concrete composite systems in residential, commercial and industrial applications over recent years (Ahmadi and Saka 1993; Natterer et al. 1996; Bathon and Graf 2000; Jamnitzky 2001; Yttrup 2002).

In the use of road or pedestrian bridge decks, durability is improved. A solely wooden bridge deck can suffer enhanced deterioration from exposure to wear and tear, rainwater, and road salts. In a wood–concrete composite, the concrete slab is able to protect the wood beams beneath, provide water runoff, and a wear-resistant surface. Also, the potential for concrete cracks is reduced as it is designed to experience compressive stresses exclusively. Particularly in this application, the composite system promises a market for durable prefabricated systems.

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Note. Associate Editor: Rob Y. H. Chai. Discussion open until February 1, 2006. Separate discussions must be submitted for individual papers. To extend the closing date by one month, a written request must be filed with the ASCE Managing Editor. The manuscript for this paper was submitted for review and possible publication on November 11, 2003; approved on January 4, 2005. This paper is part of the ***Journal of Structural Engineering***, Vol. 131, No. 9, September 1, 2005. ©ASCE, ISSN 0733-9445/2005/9-1404–1412/$25.00.

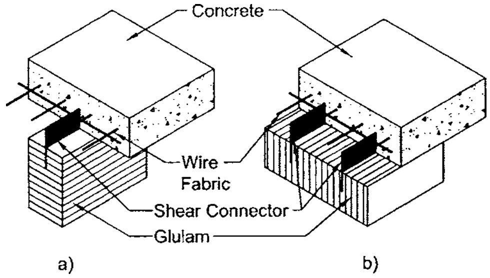

Fig. 1. Components of wood concrete composite: (a) T-beam system; and (b) plate system

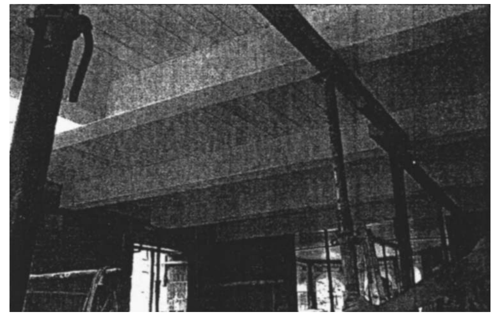

Fig. 2. Wood concrete composite T-beam system under construction, Aschaffenburg, Germany

Shear Connector Design Philosophy

Theoretically, failure of a wood–concrete composite may occur in any one of the three materials—the wood, the concrete, or the steel connector. Considering the relatively nonductile failure modes of the concrete and wood in tension, it is most logical to design for failure to occur in the shear connector. Here, one can expect higher stiffness (steel is naturally stiffer than the other materials) and ductility through an elasto–plastic steel failure (defined by a plateau on the load–deflection curve, over which deflection increases while strength is maintained). The advantage of ductility is the overt warning of impending failure. Consequently, it is most advantageous to use a shear connector that provides: (1) a stiff connection between the wood and the concrete while undergoing stresses in the elastic range and (2) a ductile response while undergoing stresses in the plastic range. These were primary design considerations of the composite developed in this study. Ensuring failure initiates in the steel component also improves system reliability due to the consistent nature of steel strength in comparison to wood or concrete strength.

Any slip at the interface of the two main components produces substantial deflection of the overall system and thereby reduces system performance. Slip can occur at one or more of the following slip plane locations: between the concrete and the connector, within the shear connector itself, and/or between the connector and the wood. Many wood–concrete shear connector systems in use today are inherently soft in terms of the latter slip plane. For example, dowels (i.e., nails, pins, screws, and bolts) are generally considered to be the least rigid of all connectors (Ceccotti 1995; Blaß and Schlager 1996; Avak and Glaser 2002) as they tend to split and crush the wood under load, thus incurring slip. However, structural adhesives have the potential to provide extremely rigid connections, as demonstrated with numerous wood composite products (e.g., glulam, I-joists, and structural composite lumber). As such, adhesive was used to minimize interlayer slip between the connector and the wood in the composite system developed in this study.

Proposed Wood–Concrete Connector System

The composite system in this study is formed through the use of a continuous steel mesh shear connector of which one half is glued into a slot in glued-laminated timber (glulam) while the other half is embedded into concrete. Two versions of this system currently exist: a T-beam system and a plate system [see Figs. 1(a and b)]. In both versions, a reinforcing wire fabric is used above the connectors to mitigate concrete shrinkage and cracking. Additional reinforcement may be added to the concrete in building applications to resist negative moment as required (e.g., over supports in continuous decks). The T-beam system is well suited to applications with weight and stiffness limitations while a plate system has the potential to minimize ceiling depth. Low-grade wood can be used in the latter due to inherent system effects. The plate system is similar to laminated decks used in wood bridges in North America since the early 1930s, whereby wood planks are laminated (nailed or glued) with their larger cross section dimension vertical. This type of framing is not common for residential or commercial applications in North America as yet; however, it may have potential as such given that it is a cost efficient approach being used in Europe.

Both systems are currently employed in buildings in Germany. In fact, the system has recently received code approval in Germany under document *DIBT Z-9.1-557* (DIBT 2004). Figs. 2 and 3 are photos of demonstration projects under construction using the T-beam and plate system, respectively. Transverse planking (maximum thickness of 30 mm) may be used between the timber and the concrete, seen as lines in the ceiling in Fig. 2. The slab is typically cast in place and the planking acts as a permanent formwork and finished ceiling. The system requires temporary shoring during construction, as seen in the photos, to support the weight of the wet concrete prior to curing and achieving composite action. Long span wood–concrete applications are often designed with some camber to offset dead load deflection. Adequate bearing surface should be provided to avoid wood crushing of the supports, particularly for the T-beam application; however, for most applications, standard glulam connection hardware suffices.

Preliminary experimental tests, prior to the tests described in this paper, were performed to optimize general features of the shear connector (i.e., dimensions, steel mechanical, and surface properties). Also, adhesives, which were capable of transmitting the applied loads, were narrowed down to two suitable types: a

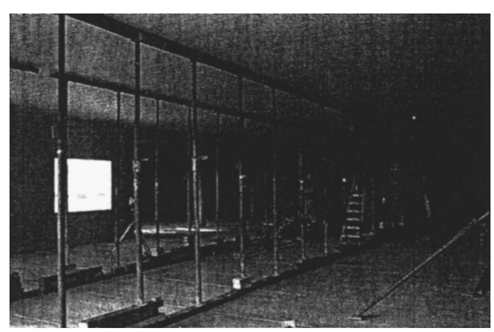

Fig. 3. Wood concrete plate system under construction, Griesheim, Germany

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two component epoxy (used in Germany for many years in timber restorations and thus judged reliable in its use) and a two component polyurethane adhesive.

There are some notable drawbacks to using adhesive in structural assemblies: in particular, uncertainty of how the glue performs under temperature fluctuations (i.e., outdoor climate or fire) or long term loading. For instance, standard light-frame construction adhesive is often neglected in strength or deflection calculations for medium to long duration loads such as dead or floor live load. However, a wide array of high strength adhesives exist which are well suited to structurally demanding applications. For example, in concrete construction, anchoring epoxies are commonly used and in wood construction, phenol resorcinol adhesive has undergone extensive research (and more than 100 years of service in glulam) to prove its reliability for structural use. Comprehensive understanding of the adhesive's behavior is a critical component of the development of this connector system and to that end, testing is ongoing at the Univ. of Applied Sciences Wiesbaden, Germany as well as at the Univ. of Massachusetts, Amherst, Mass.

Another component important to the development of the wood–concrete system is understanding long-term system performance. Wood and concrete exhibit creep under prolonged loading. For both materials, the amount of creep-related deflection is a function of load duration as well as applied stress level. For wood, however, its creep behavior is also influenced by moisture and temperature changes due to its hygroscopic nature. While temperature has a negligible influence under normal service conditions (Andriamitantsoa 1995), elevated moisture levels and particularly changes in moisture content can create deflections in the range of 3–4 times the instantaneous creep (Schänzlin 2003). Since wood and concrete are tightly joined together, different creep rates (concrete creeps faster than wood) may also have a detrimental effect, particularly in a range between 3 and 7 years of service (Schänzlin 2003). In addition to magnified deflections, the differential creep also tends to migrate internal forces from the concrete to the wood leading to higher stresses in the wood (Ceccotti 1995).

A common approach to estimating the long-term behavior (deflection and stresses) of a wood–concrete system is through the reduced-modulus method where both materials' moduli of elasticity are reduced to account for creep (Ceccotti 1995). Although this simplified approach is adequate for constant environments (such as indoors), it has been shown that seasonal climate changes, as they would occur for a bridge structure, may lead to a more unfavorable creep behavior, which cannot be accurately estimated by the reduced-modulus method (Schänzlin 2003). Further investigation is necessary in this area and work is currently being carried out at the Univ. of Applied Sciences, Wiesbaden, Germany as well as at the Univ. of Massachusetts, Amherst, Mass.

The tests outlined in this paper were performed at the Timber Research Laboratory at the Univ. of Applied Sciences, Wiesbaden, Germany. Two series of laboratory tests were conducted—static push-out tests and full scale bending tests. The push-out tests are necessary to evaluate failure mode of the shear connector, but also to establish connector stiffness and capacity for use in design equations. The bending tests provide a check on capacity as well as overall beam failure mode.

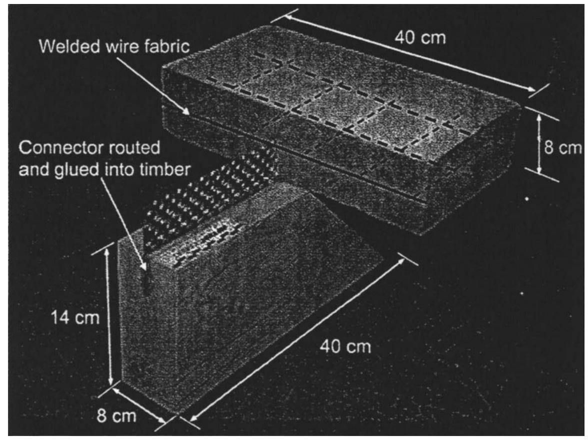

Fig. 4. Cut open view of push-out specimen

Testing Program

Materials

The timber used for both series of tests (push-out and bending) was a German Spruce glulam, grade BS11, with an allowable bending strength of 11 MPa. This is comparable to a North American glulam stress class 16F-1.3E. Specimens were fabricated and tested under ambient laboratory conditions. The moisture content of the timber both during construction as well as at testing was determined to be an average of 10% with minimal variation. Moisture contents of less than 15% are generally adequate for adhesive bonding, as specified by the adhesive manufacturer. The adhesive used in this study was a two component epoxy produced by WEVO-CHEMIE GmbH, Stuttgart, Germany. The concrete was a B25 class, average standardized concrete with a minimum compression capacity of 30 MPa. The shear connector consisted of an St 37 (comparable to an A36) mild steel, perforated and expanded mesh.

Push-Out Test Specimens

A total of six push-out specimens were fabricated. Referencing Fig. 4, specimens consisted of an $80\times140\times400$ mm timber section conjoined to an $80\times400\times400$ mm concrete slab through a 2 mm thick $\times400$ mm long steel mesh. The steel mesh was 100 mm high and reached 50 mm into the timber and 50 mm into the concrete. The connector rested in a 50 mm deep rout in the timber that was cut approximately 3 mm wide with a standard circular saw. Glue was filled into the rout using a manufacturer approved dispensing system (i.e., an applicator gun with a duo-pak cartridge and a mix nozzle). A continuous bead of glue was dispensed into the rout with several passes until full, ensuring uniform distribution of the glue. Shear connectors were pressed slowly but firmly into the channel. It is common and desirable to observe glue overflow, which can be easily removed, if desired. The premise to this approach is to provide adequate glue coverage such that the glue can flow into the voids in the steel lattice to provide mechanical interlocking. The adhesive was cured for a minimum of 16 h.

The timber sections were then placed upside-down on formwork in preparation for pouring the concrete. It is noted that actual building slabs would naturally not be cast inverted; however, this was done for convenience for the push-out test specimens only. In doing so, it was assumed that cast orientation has no

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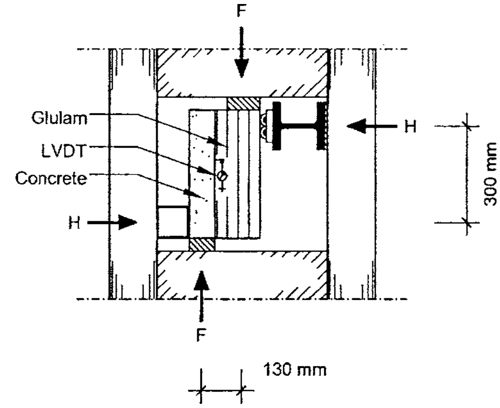

Fig. 5. Loading and support conditions of push-out test

influence on shear strength or stiffness since shear properties are evaluated in the longitudinal axis of the specimen. A moisture protection layer was used between the timber and the concrete to prevent excessive wetting of the timber during casting. This is also necessary for actual building projects. Minimal welded wire fabric (with a steel cross section of 131 mm^2/m), placed directly on the shear connector, was used to minimize concrete shrinkage. The concrete was poured and vibrated to remove air pockets and was then cured while maintaining a wet surface for a total of 3 days. After 28 days, samples showed that the concrete had an average compression strength of 55 MPa.

Push-Out Test Setup

The specimens were subjected to a one-sided push-out test under ambient laboratory conditions. Load and support conditions are illustrated in Fig. 5. An inherent eccentricity of 130 mm exists in the setup resulting in a horizontal force couple that is linearly proportional to the applied shear force. To minimize friction at the horizontal contact points, a roller bearing was used. The load was applied by a 500 kN capacity universal material testing system with computerized data acquisition. One linear variable displacement transducer monitored the relative displacement (slip) between the timber and concrete at the centerline of the specimen. The test was performed according to the European standard "Deutsches Institut für Normung-English" *DIN EN 26891* (DIN 1991) which requires a semicycling loading whereby loading is applied to 40% of the estimated failure load, paused, reduced to about 10%, and then increased until failure to remove inherent slack in the low-load region.

Table 1. Push-Out Test Results

Test	Ultimate load (kN)	Displacement (mm)	Shear stress (MPa)	Slip modulus (kN/mm)
B1	120.97	1.74	3.02	353.87
B2	114.89	1.40	2.87	367.65
B3	103.53	1.34	2.59	371.75
B4	102.53	1.40	2.56	331.13
B5	116.88	1.52	2.92	410.96
B6	110.96	1.25	2.77	657.42
Average	111.62	1.44	2.79	415.46
Standard	7.41	0.17	0.19	121.38
Coefficient of variation [%]	6.64	11.81	6.81	29.22

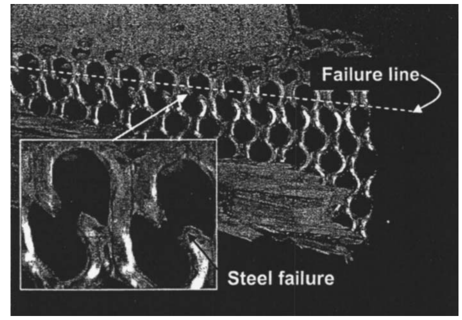

Fig. 6. Shear failure of steel connector of push-out specimen

Push-Out Test Results

Test results and corresponding descriptive statistics are shown in Table 1. They include ultimate failure load, corresponding displacement, corresponding shear stress distributed uniformly over the shear connector [i.e., ultimate load divided by gross area of the steel to the wood (400×100 mm^2)], and slip modulus K as determined by the slope of the linear-elastic portion of the load–displacement curve.

The failure mode for each specimen was determined to be yielding followed by rupture of the steel shear connector. Fig. 6 shows a push-out specimen with concrete and wood removed to reveal the failure line in the steel beneath. Along the shear failure line, at the original concrete and wood interface, the connector looks and performs similarly to that of truss web members. The steel mesh forms small cross-links along this plane and the links undergo either tension or compression stress. Upon close inspection, it appears that the links under compression yield and buckle while the links under tension yield and subsequently rupture. No failure was observed in either the concrete or the wood. Also, adhesive failure was not visible at any location.

The failure mode of the shear specimens was also evident through examination of the test data. Fig. 7 shows the resulting load/displacement curves from the push-out tests. The curves have a rigid linear-elastic region and a well developed plastic region (i.e., where ultimate load is sustained along a plastic plateau) which is indicative of steel failure. Furthermore, the curves

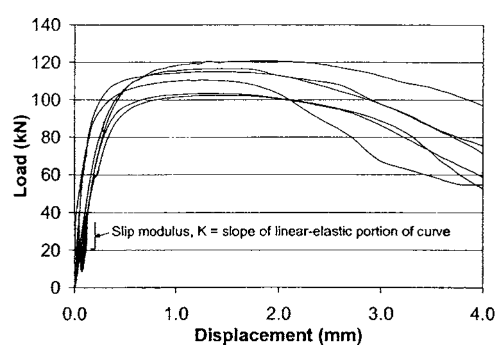

Fig. 7. Load–displacement curves of push-out test specimens

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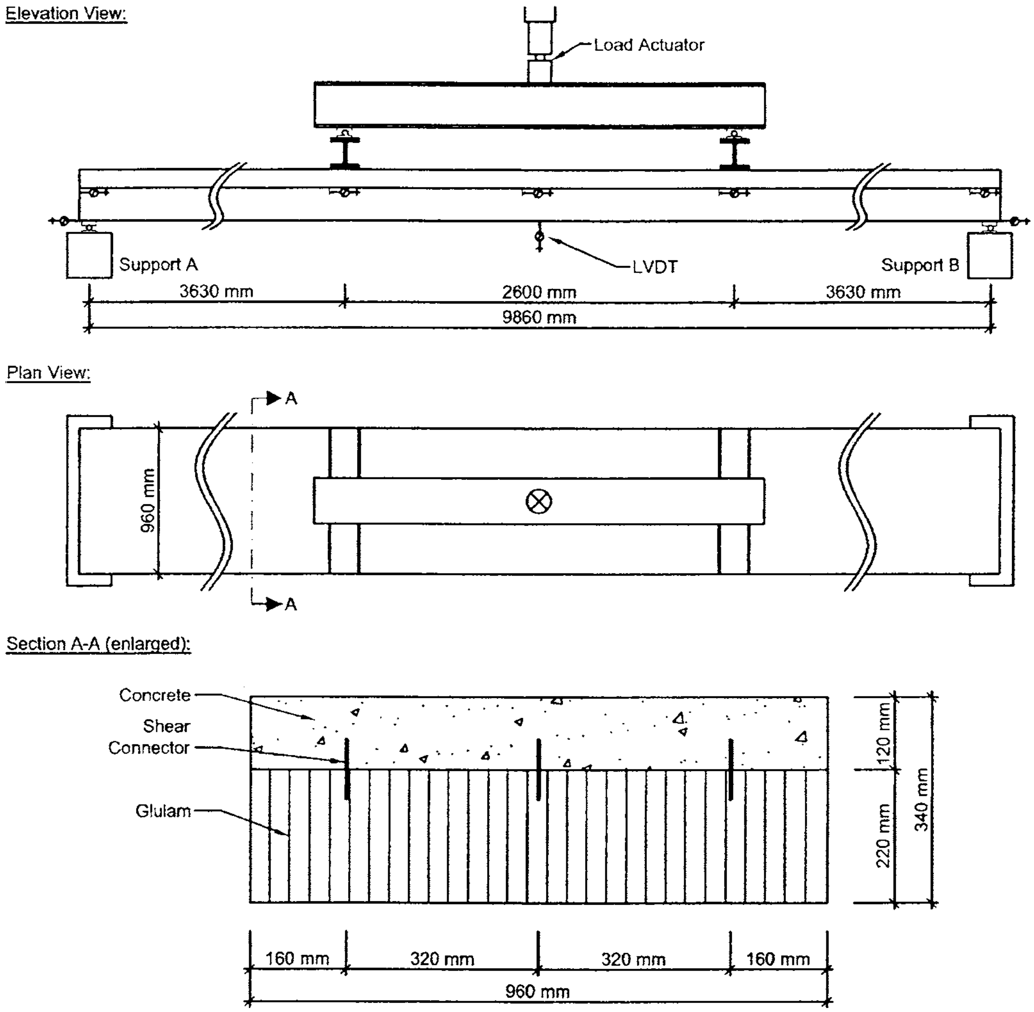

Fig. 8. Beam specimen setup

have a relatively close fit (showing consistent overall response) as well as a low coefficient of variation (COV) for ultimate load (COV=6.6% with minimum 102.5 kN and maximum 121 kN). The typical COVs for concrete and wood strength are comparatively higher at approximately 10–15 and 15–25%, respectively. Hence, the consistency of the results also indicates that failure occurred in the steel connector and not the other materials. Low strength variability is a highly desirable attribute and particularly noteworthy for a composite system using three different materials.

The slip modulus varies between 331.1 and 657.4 kN/mm with an average of 415.5 kN/mm. The variability is expectedly higher than that for ultimate load (29.22 versus 6.64%) since the slip modulus is influenced by slippage occurring in three different slip planes and can be accentuated by small variations in concrete porosity or glue spread. Also, friction between the wood and the concrete was noted to play a role in this. Specimens with a small gap between the wood and the concrete showed a lower slip modulus than those that were tightly fit indicating the importance of quality control during fabrication.

Bending Tests

A full scale timber plate system was fabricated and tested to investigate performance under static bending. As shown in Fig. 8, the specimen consisted of a 10 m long, 220×960 mm solid glulam plate, a 120×960 mm concrete slab, and three rows of shear connectors in cross section. The shear connectors consisted of the same steel mesh material that was tested in the push-out test with the exception that each connector was 1 m long (as opposed to 400 mm long) and butted end to end. The center-to-center span was 9.86 m with a bearing length of 96 mm at both supports. The glulam beam was supplied with a camber of 33 mm at mid-span to correct for dead load deflection. Two point loads were applied 2.60 m apart centered about mid-span using a 2,000 kN capacity hydraulic actuator, mounted to a system of steel spreader beams. Vertical beam displacement was monitored at mid-span; horizontal slip between the concrete and timber was measured at mid-span, under load points and over bearings; and horizontal displacement of the beam at bearings was recorded.

During beam construction, a moisture barrier film was placed between the glulam and the concrete to protect the glulam from excessive moisture. The film appears as a high gloss on the wood in Fig. 9. Ample glue squeeze out—reassurance that the steel/wood bond is adequate—is evident in this photo. A reinforcing mesh (131 mm^2/m) was then placed directly on top of the shear connectors prior to pouring of the concrete. Fig. 10 shows the prepared beam in the test frame.

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Fig. 9. Bending specimen prior to installation of wire fabric and concrete

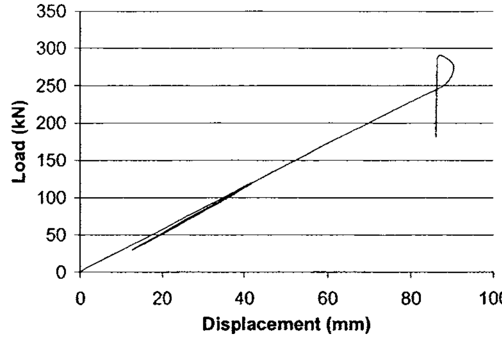

Fig. 11. Load versus mid-span displacement of bending specimen

Bending Test Results

The load/mid-span displacement curve of the beam is depicted in Fig. 11. Two aspects of the curve are particularly noteworthy: (1) the curve is atypical showing decreasing displacement upon failure and (2) the curve does not show the same level of plastic yielding (i.e., plateau on the load–displacement curve) as does Fig. 7. The reason for these unexpected results was found to be torsional failure of the beam leading to abnormal data acquisition. At peak load, the only external visible signs of beam failure were slippage cracks in the concrete near the interface of the wood and concrete between the left end support and the neighboring load point. It was determined that at incipient yield, at approximately 250 kN, one row of shear connectors near the left end support began to fail before the others (evidenced by progressive degrees of steel deformation and rupture upon investigation), leading to out-of-plane deformation. The load apparatus also tipped out of plane, rotating toward the yielded connectors. Data acquisition was load controlled and as the load continued to increase, the twisted beam measured decreasing deflection at mid-span until the test was manually stopped. Thus—and unfortunately—the load/mid-span displacement curve did not capture the plastic response of the beam. This does not mean that the shear connectors did not undergo plastic yielding, however. Sensors measuring horizontal displacement (slip) between the timber and concrete, placed over the bearing where failure initiated, showed high ductility as in the shear tests (see Fig. 12). Also, upon inspection of the connectors, yielding and ultimate fracture—as in the push-out tests—was apparent (see Fig. 13). Adhesive failure was not visible at any location.

Torsional failure is not common in beam tests using homogeneous materials but is, in hindsight, feasible for this composite beam construction where material variability can lead to failure in one row of connectors before another. Although the load apparatus was placed concentrically, the relatively narrow configuration (length to width ratio of 10:1) of the beam may have contributed to the torsional failure mode. It is speculated that this would not be a concern for full floor applications, since the full floor width would provide torsional stability to the system; however, this is an area of study which requires further investigation.

Design Methodology

To date, no guidelines are available in the United States for design of wood–concrete composites. However, formulas to estimate design parameters of a bimaterial system with semirigid shear connectors are provided by the European Standard for Timber Design, *Eurocode 5* (DIN 1994). These formulas estimate effective bending stiffness of the overall composite and maximum bending stresses (i.e., maximum tensile and compressive stresses in both the wood and the concrete). The formulas derive from the material mechanics, specifically elastic composite beam theory, and are independent of design method. That is, the formulas can be integrated into either an allowable stress design or a reliability based design (i.e., load and resistance factored design) method in a manner similar to that done for modulus of elasticity and modulus of rupture for homogeneous materials.

In accordance with this code, the effective composite bending stiffness $(EI)_{ef}$ is calculated as

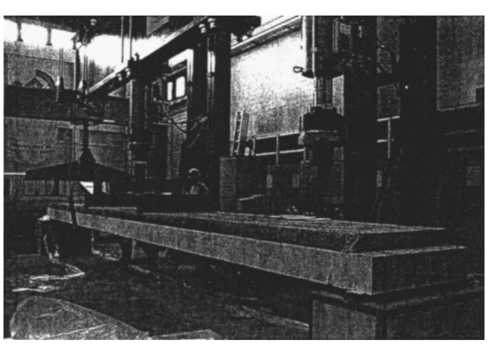

Fig. 10. Bending test setup

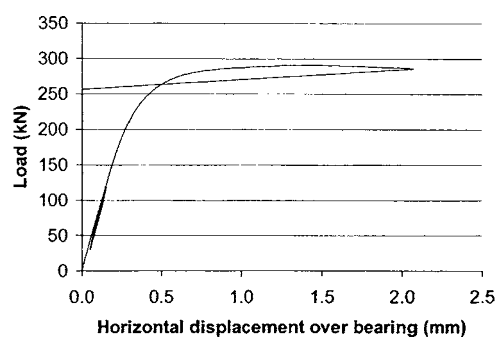

Fig. 12. Load versus horizontal displacement over bearing of bending specimen

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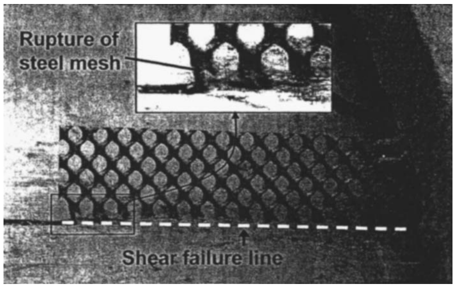

Fig. 13. Shear failure in steel connector of beam specimen

$$(\mathrm{EI})_{\mathrm{ef}} = \sum_{i=1}^{2} (E_i I_i + \gamma_i E_i A_i a_i^2) \tag{1}$$

where, referencing Fig. 14: subscripts 1 and 2 refer to the respective component; E=modulus of elasticity; I=moment of inertia; A=cross-sectional area; a=distance from centroid of respective component to overall neutral axis; and γ=shear connector reduction factor. The values of γ range between 0 (signifying no composite action) and 1 (full composite action) and are calculated as

$$\gamma_1 = \frac{1}{1 + \dfrac{\pi^2 E_1 A_1}{K' L^2}}$$

$$\gamma_2 = 1 \tag{2}$$

where K'=slip modulus K divided by the length of the connector in the push-out test; and L=beam span. For discontinuous connectors (i.e., nails or screws), K'=slip modulus K divided by the spacing of the connectors.

The distance between the wood member and the overall neutral axis a_2 is dependent on the shear reduction factor γ_1 and is calculated as

$$a_2 = \frac{\gamma_1 E_1 A_1 (h_1 + h_2)}{2\sum_{i=1}^{2} \gamma_i E_i A_i} \tag{3}$$

where h=height of the respective member. The distance a_1 is determined from the geometry of the composite beam and Eq. (3).

The basic mechanical behavior of a composite beam is assumed to be elastic. Referring to Fig. 14, the normal stress distribution in each component of the composite is determined by algebraic addition. It is designed such that the concrete layer undergoes compression and bending, the wood member undergoes tension and bending, and the connector experiences shear. The maximum tensile stress in the wood is calculated as

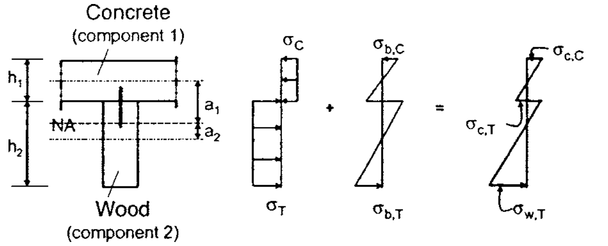

Fig. 14. Stress distribution within wood concrete composite T-beam

$$\sigma_{w,T} = \sigma_T + \sigma_{b,T} = \frac{M_{\max}}{(\mathrm{EI})_{\mathrm{ef}}} E_2 a_2 + \frac{M_{\max}}{(\mathrm{EI})_{\mathrm{ef}}} \frac{h_2}{2} E_2 \tag{4}$$

where $\sigma_{w,T}$=total normal stress in the wood; σ_T=tensile stress in the wood as a result of the force couple in the composite section; $\sigma_{b,T}$=bending stress in the wood as a result of force couple about the wood section; and $M_{\max}$=maximum bending moment in the beam.

The maximum compressive stress in the concrete is calculated in a similar manner as

$$\sigma_{c,C} = \sigma_C + \sigma_{b,C} = \frac{M_{\max}}{(\mathrm{EI})_{\mathrm{ef}}} \gamma_1 E_1 a_1 + \frac{M_{\max}}{(\mathrm{EI})_{\mathrm{ef}}} \frac{h_1}{2} E_1 \tag{5}$$

where $\sigma_{c,C}$=total normal stress in the concrete; σ_C=compressive stress in the concrete as a result of the force couple in the composite section; $\sigma_{b,C}$=bending stress in the concrete as a result of force couple about the concrete section; and $M_{\max}$=maximum bending moment in the beam.

The shear flow in the connector may be computed as

$$q = \frac{\gamma_1 E_1 A_1 a_1 V_{\max}}{(\mathrm{EI})_{\mathrm{ef}}} \tag{6}$$

where q=maximum shear flow in the connector and $V_{\max}$=maximum shear force in the beam.

Connector Efficiency

The strength and stiffness of a wood–concrete composite system depends on the efficiency of the shear connector. A comparative analysis of the efficiency of the proposed connector versus traditional connectors is provided in the following paragraphs. The analysis uses the design equations above and the dimensions of the timber plate system of the preceding bending setup. The results of the analysis are presented in Fig. 15 which shows the increase of effective bending stiffness $(\mathrm{EI})_{\mathrm{ef}}$ with γ_1. As γ_1 approaches 1, the system approaches full composite action. Conversely, when γ_1 is zero there is no composite action (i.e., the concrete and the timber act independently).

Given E_1=30,500 MPa; A_1=38,400 mm^2 (using the tributary width for one row of shear connectors, 320 mm); K=average result of six tests=415,460 N/mm/400 mm connector length and L=9,860 mm, a reduction factor γ_1=0.90 is obtained for the proposed connector using Eq. (2). Referencing Fig. 15, this is a

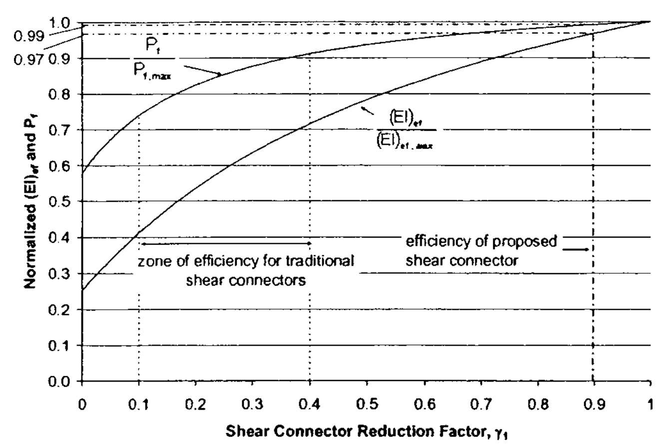

Fig. 15. Comparison of efficiency of proposed shear connector to traditional shear connectors

highly favorable result in light of the fact that for traditional connectors (nails, screws), the factor is typically between $0.1 < \gamma_1 < 0.4$ (Steinberg et al. 2003).

Using $\gamma_1 = 0.90$ together with Eq. (1), with $I_1 = 4.61 \times 10^7\ \text{mm}^4$; $I_2 = 2.84 \times 10^8\ \text{mm}^4$; $E_2 = 11{,}500$ MPa; $A_2 = 70{,}400\ \text{mm}^2$; $a_1 = 74.0$ mm; $a_2 = 96.0$ mm, we find that for the proposed connector $(EI)_{ef} = 1.79 \times 10^{13}\ \text{N mm}^2$. This value is 97% of (or 3% softer than) that of the most efficient wood concrete system (i.e., when $\gamma_1 = 1$) where $(EI)_{ef} = 1.85 \times 10^{13}\ \text{N mm}^2$.

Failure Prediction

Failure may occur in any one of the three materials—the wood, the concrete, or the connector. The point load at failure P_f may be estimated for the beam by analyzing each potential material failure and finding the governing case (i.e., the failure mode which yields the smallest P_f) as follows:

1. Wood tensile failure: The characteristic (i.e., 5th percentile test) strengths of the German Spruce glulam, grade BS11, are published as 17 MPa in tension and 24 MPa in bending. Assuming a lognormal distribution with a coefficient of variation of 14%, the average strengths are calculated to be 21 and 30 MPa, in tension and bending, respectively. Thus, given $\sigma_{Tens} = 21$ MPa; $\sigma_{Bend} = 30$ MPa; $h_2 = 220$ mm; $E_2 = 11{,}500$ MPa; $a_2 = 96.15$ mm, and $(EI)_{ef} = 5.37 \times 10^{13}\ \text{N mm}^2$ (using the full beam width), then referencing Eq. (4) and using a linear interaction formula for tension and bending

$$\frac{\sigma_T}{\sigma_{Tens}} + \frac{\sigma_{b,T}}{\sigma_{Bend}} \leqslant 1$$

then $P_f = 312$ kN.
2. Concrete compressive failure: The point load at failure assuming compression failure in the concrete is estimated using Eq. (5) together with the average concrete compression strength of 55 MPa. Given: $\gamma_1 = 0.90$; $h_1 = 120$ mm; $E_1 = 30{,}500$ MPa; $a_1 = 73.8$ mm, and $(EI)_{ef} = 5.37 \times 10^{13}\ \text{N mm}^2$ (using the full beam width), then $P_f = 422$ kN.
3. Connector shear failure: The point load at failure assuming shear failure in the connector is calculated using the shear flow capacity of the connector as determined in the push out tests. The capacity is the average ultimate force/connector length (111,620 N/400 mm = 279.1 N/mm). Given that there were three rows of connectors and assuming that the shear force was distributed among the rows (one row as a lower boundary and three rows as an upper boundary), the shear flow capacity of the beam would be between 1×279.1 N/mm = 279.1 N/mm and $3 \times 279.1 = 873.1$ N/mm. Using these boundary values in Eq. (6) and solving for ultimate load, $P_f = 2 \times V_{max}$, $P_{f(low)} = 128$ kN and $P_{f(high)} = 411$ kN.

The smallest P_f of the preceding analyses is from shear failure of one row of connectors (i.e., $P_f = 128$ kN). The actual maximum elastic load on the beam was 250 kN and ultimate failure occurred at 291 kN. This result indicates that failure likely initiated in one row of the connectors and that the shear force was not evenly distributed which, indeed, was apparent from the observed out-of-plane deformation as well as subsequent examination of the failed beam specimen.

In terms of structural efficiency of the system (referencing Fig. 15), one can compare the predicted point load at failure assuming wood failure ($P_{f,max} = 312$ kN) to that for full composite action ($P_{f,max} = 315$ kN). In this case, the beam's structural efficiency is 99%, or the connector system is 1% weaker than a fully composite system.

Conclusions

The objective of this work was to investigate the structural behavior of an innovative continuous wood concrete composite system. The system is well suited for residential and commercial floor systems and is easy-to-construct while providing a cost efficient assembly. Laboratory tests were performed to establish shear and flexural behavior of the system. Based on these tests, the following observations and conclusions can be made:

1. Failure consistently occurred in the steel mesh shear connector, as it was designed to do.
2. As a result of steel failure, as opposed to concrete or timber failure, specimens consistently displayed stiff, high strength yet ductile behavior. The consistency of the results was also credited to the reliability of the steel, in comparison to that of the other two materials.
3. The adhesive gave consistent results and verified that shear can be effectively transferred between the glulam–concrete interface using steel mesh plates.
4. The wood concrete composite beam behaves very nearly as would a full composite action system. The effective bending stiffness is 3% less stiff than that of a beam with full composite action and the computed design strength is 1% less than that of a fully composite beam. This level of structural efficiency signifies a tremendous achievement in the development of wood—concrete composites—to date, no other connector system has been shown to achieve this.

The proposed system is not only well suited for new buildings—as an alternative to concrete floors—but is also ideal for use in upgrading old buildings. Mill structures, with large timber beams for example, are particularly well suited for this technology. Use in bridge design is also possible, but requires further investigation in durability, creep resistance, temperature and moisture fluctuations, as well as response to dynamic loading. Work in these areas is currently ongoing at the Univ. of Applied Sciences, Wiesbaden, Germany and at the Univ. of Massachusetts, Amherst, Mass.

Acknowledgments

The writers gratefully acknowledge the donation of metal connectors and glued-laminated timber specimens by Zang and Bahmer GmbH. Also gratitude is extended to undergraduate student Holger Stumpf for help during preparation and testing of the specimens.

Notation

The following symbols are used in this paper:

- A = cross-sectional area;
- a = distance from centroid of component to overall neutral axis;
- E = modulus of elasticity;
- $(EI)_{ef}$ = effective bending stiffness;
- h = height of member;

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I = moment of inertia;
K = slip modulus;
L = beam span;
M_{max} = maximum bending moment in beam;
P_{allow} = maximum allowable point load;
P_{ult} = ultimate point load;
q = maximum shear flow in connector;
s = spacing of shear connectors;
V_{max} = maximum shear force in beam;
γ = shear connector reduction factor;
$\sigma_{b,C}$ = bending stress in concrete;
$\sigma_{b,T}$ = bending stress in wood;
σ_{C} = compression stress in concrete;
$\sigma_{c,C}$ = total normal stress in concrete;
σ_{T} = tensile stress in wood; and
$\sigma_{w,T}$ = total normal stress in wood.

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J. Struct. Eng. 2005.131:1404-1412.