Experimental behavior of a continuous metal connector for a wood-concrete composite system

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Abstract

The benefits of using shear connectors to join wood beams to a concrete slab in a composite floor or deck system are many. Studies throughout the world have demonstrated significantly improved strength, stiffness, and ductility properties from such connection systems as well as citing practical building advantages such as durability, sound insulation, and fire resistance. In this study, one relatively new shear connector system that originated in Germany has been experimentally investigated for use with U.S. manufactured products. The connector system consists of a continuous steel mesh of which one half is glued into a southern pine Parallam® Parallel Strand Lumber beam and the other half embedded into a concrete slab to provide minimal interlayer slip. A variety of commercial epoxies were tested for shear strength and stiffness in standard shear or "push out" tests. The various epoxies resulted in a variety of shear constitutive behaviors; however, for two glue types, shear failure occurred in the steel connector resulting in relatively high initial stiffness and ductility as well as good repeatability. Slip moduli and ultimate strength values are presented and discussed. Full-scale bending tests, using the best performing adhesive as determined from the shear tests, were also conducted. Results indicate consistent, near-full composite action system behavior.

Wood-concrete composites, used commonly in decking applications, are structurally efficient building materials that are engineered to capitalize on the best qualities of both components. These composites exhibit desirable structural properties not achievable by either the wood or the concrete alone. Ideally, the concrete (in the form of a slab) has relatively little tensile capacity and is used predominantly in compression, where it has superior performance in terms of strength and stiffness. The wood (in the form of a glue-laminated timber or structural composite lumber beam) is used predominantly in tension, which provides exceptional strength and stiffness relative to added weight when compared to an equivalent all-concrete section. The strength and stiffness advantages occur as a result of composite action (provided that the wood and concrete components work together as one unit with minimal interlayer slip). The end result is a strong, rigid, and lightweight decking system.

In addition to providing superior strength and stiffness, the advantages of these systems are many. Perhaps most important is improved durability. A reinforced concrete slab, for example, has the potential for steel corrosion whereby moisture can enter through tensile cracks, significantly reducing the lifetime of the structure. Similarly, a timber bridge deck can suffer enhanced deterioration from exposure to rainwater and road salts. In a wood-concrete composite, the concrete can avoid tensile cracking through predominantly compressive loading while effectively protecting the timber beams beneath. Other advantages, especially for houses, schools, and public buildings, include enhanced vi-

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Figure 1. — Structural upgrade of a historic home using a continuous steel mesh connector system.

Figure 2. — Newly constructed commercial floor using a continuous steel mesh connector system.

brational damping and improved sound insulation as well as fire resistance when compared to an all-wood floor system.

These advantages have prompted an increased use of wood-concrete composite systems in residential, commercial, and industrial applications throughout the world (Ahmadi and Saka 1993, Natterer et al. 1996, Bathon and Graf 2000, Jamnitzky 2001, Yttrup 2002). They are ideally suited to improve strength and deflection properties of existing wood decks and floors as well as providing a structurally efficient solution for new construction. **Figure 1** illustrates a recent structural upgrade of the attic space in a historic home using a wood-concrete system in Aschaffenburg, Germany. The wood structure has been renovated to strengthen the floor by incorporating continuous metal plates (seen protruding from the wood beams that are covered with a black moisture barrier) to create a rigid shear connection to the concrete to be poured on top. **Figure 2** shows a newly constructed commercial floor in Darmstadt, Germany. The new floor uses the same continuous steel mesh connector system.

The system illustrated in **Figures 1** and **2** is currently being used only in Germany. It is the intent of this paper to introduce this system to the United States through an investigative study of the system's performance using products made in the United States.

Overview of existing shear connectors

The shear connector is a critical component of the system and, to a great extent, determines the system's level of performance. The function of the shear connector is to effectively transfer the shear between the wood beam and the concrete deck to promote composite action. There are two bounds of composite action: a lower bound where the layers are not connected and work independently (no composite action) and an upper bound where the two layers are rigidly connected with no interlayer slip (full composite action). Ideally, one would desire the latter; however, no slip is difficult to achieve and most connectors produce at least some interlayer slip (partial composite action). Minor slippage can be beneficial, allowing distribution of shear stresses along the shear connectors.

When compared to no composite action, bending capacity and stiffness increases of over 40 and 200 percent, respectively, are commonly reported with partial composite systems (Mantilla-Carrasco and Oliveira 1999). In order to design composite sections, it is critical to know the shear connector capacity and failure mode. Due to the relatively non-ductile failure modes of the constituent concrete (compression) and wood (tension), ductile failures in the shear connector may provide increased overall ductility and reliability. The resulting composite system would realize increased strength, stiffness, ductility, reliability, and long-term performance as compared to similar non-composite sections.

A multitude of shear connection systems exist. They are perhaps best categorized into four groups: dowel-type, tubular-type, and shear key/anchor connectors (all of which are discontinuous) and glued-in plate connectors (which can be either continuous or discontinuous). The four groups are illustrated in **Figure 3**.

Group 1 connectors have the advantage of being inexpensive and uncomplicated to install; however, they are generally considered to be the least rigid of all systems (Ceccotti 1995) and have been substantiated as such in comparative studies (Blaß and Schlager 1996, Avak and Glaser 2002). Nevertheless, a study by Ahmadi and Saka (1993) investigated various configurations of high-strength nails, screws, and bolts and found increases in static 4 point flexural strength of 55 percent and reductions in mid-span deflections in the order of 2.3 times when compared to tests with no shear connectors.

Group 2 connectors have been shown to possess higher rigidity, ductility, and ultimate strength than dowel-type connectors (Blaß and Schlager 1996, Mungwa et al. 1999). The reason attributed to this is that nails and screws typically

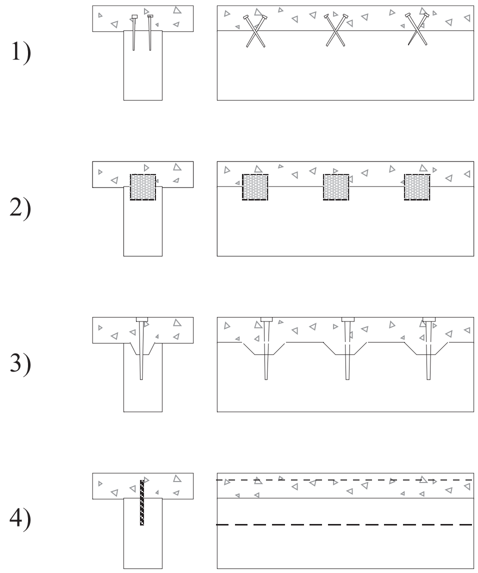

Figure 3. — Shear connector groups: 1) dowel; 2) tubular or metal plate; 3) shear key with anchor; and 4) (dis)continuous glued-in-wood plate.

Figure 4. — Shear connectors being glued into rout in PSL member for shear specimens.

cause wood splitting failure, whereas tubular-type connectors (rigid ring connectors, in particular) can cause wood shear plug failure, which is a stronger, more rigid mode of failure. Hollow cylinders as well as bent sheet metal connectors (Gauthier 1996, Mantilla and Carrasco 1999), which fall into this category, are also apt to provide good ductility, as the common failure modes are wood embedment, steel shear, or concrete crushing.

Group 3 connectors, where notches (shear keys) have been cut into the wood and reinforced with an anchoring device such as a post-tensioned bolt or lag screw, have been shown to have similar to moderately better strength and slip resistance than group 2 connectors (Blaß and Schlager 1996, Avak and Glaser 2002). The horizontal shear forces are transmitted through the shear key with little interlayer slip, while the dowels work in traction to resist the vertical load component. Gutkowski et al. (2000) reported the load-slip response to be initially linear with abrupt partial failure (presumably of the concrete) with a modest residual ductile behavior thereafter (presumably from yielding of the dowel).

Group 4 connectors are generally considered to possess the greatest rigidity (Ceccotti 1995). The observed failure modes from experimental shear tests are reported as being primarily wood shear failure or steel plate failure (Bathon and Graf 2000). For design purposes, interlayer slip is often considered negligible and in the case of bending, the assumption of plane sections remaining plane is generally accepted. Consequently, design calculations for group 4 connectors may be easily done through transformed sections. Groups 1 through 3, on the other hand, are semi-rigid composites, subject to varying levels of slip, which require more complex design procedures. Analytical solutions as well as finite element based solutions for partial composite action have been offered by various investigators (Ahmadi and Saka 1993, Girhammer and Gopu 1993, Mantilla-Carrasco and Oliveira 1999, Davids 2001). In this regard, glued-in plate connectors present a design advantage over other connector types.

The objective of the current study was to investigate the structural behavior of a continuous glued-in steel mesh connector. A similar study was conducted by Bathon and Graf (2000) using European species glulam and adhesives. The current study adapts the system to products manufactured in the United States. This was done through a two-phase test program. In the first phase, various bonding agents were assessed in terms of shear performance using standard shear or "push-out" tests. The best performing adhesive was then used in the second phase to evaluate full-scale bending performance.

Experimental shear test program: Phase 1

Experimental tests were performed to establish the capacity and ductility of a continuous glued-in steel mesh connector using southern pine Parallam® Parallel Strand Lumber (PSL). Five adhesives were evaluated in terms of performance and ease of application. All adhesives evaluated during this experimental stage were two-part room-temperature curing epoxies to meet application and end-use expectations. Two-part epoxies were used exclusively as they are generally accepted as having excellent bond strength between wood and steel. With thousands of glues on the market and in consideration of future field use, they were also chosen based on availability, ease of use (minimum risk of misuse and failure), and economics.

Figure 5. — Shear specimen formwork.

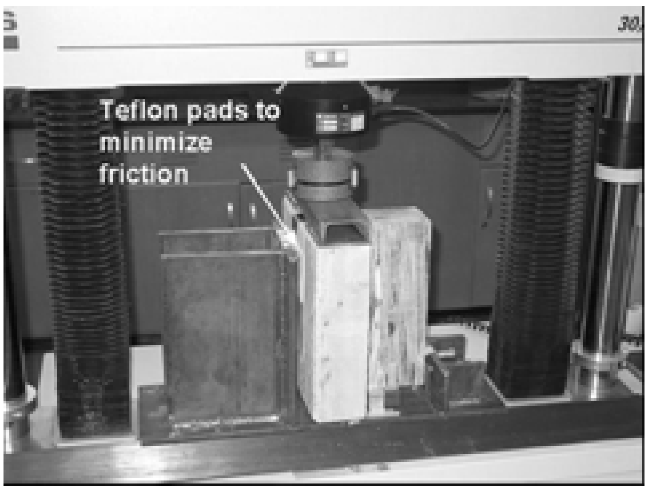

Figure 6. — Shear test set-up.

Specimen fabrication

Specimens consisted of an 89- by 133- by 305-mm Parallam® section attached to an 89- by 305- by 305-mm concrete slab with a 2.5-mm-thick meshed steel shear connector. The connector is in the physical form of a steel lattice such that the glue would gain mechanical advantage through interlocking in the lattice voids (**Fig. 4**). The shear connector had a length of 305 mm and a width of 100 mm, with 55 mm embedded in the concrete and 45 mm extending into the wood. The shear connector was split at the center with each side bent in alternating directions within the slab. A 45-mm-deep rout with width of 2.8 mm was provided centered widthwise on the top of the Parallam® sections.

Glues (properties provided in **Table 1**) were then filled into the rout using manufacturer-approved dispensing systems. For all but type B glues, an applicator gun with a duo-pak cartridge and a mix nozzle was used. For type B, the glue was premixed in accordance with manufacturer recommendations and dispensed into the rout through a plastic funnel applicator. A continuous bead of glue was dispensed into the rout with several passes until the rout was filled. In this manner, it was ensured that a comparable volume of glue was used for each adhesive. Shear connectors were then pressed firmly into the rout. It was common and desirable to observe excess glue overflow, which was simply removed. This glue "squeeze–out" was considered an indication that an adequate volume of adhesive was applied. This approach is considered to be a realistic and simple method to ensure adequate glue coverage when installing connectors *in situ*. Glues were then cured for a minimum of 7 days under ambient conditions.

Sections were inverted and placed on formwork (**Fig. 5**). A minimal reinforcement mesh (50 by 50 mm) was used, which was located directly on top of the steel connector. Ready-mix concrete was placed and vibrated to remove air pockets. The concrete had an average compressive strength of 22 MPa on the days of testing.

Shear test setup

Nineteen specimens were subjected to a one-sided "push-out" test under ambient laboratory conditions (approximately 16°C, 40% relative humidity). A photo of the test setup is provided in **Figure 6**. As no standard currently exists for composite wood-concrete sections, American Society for Testing and Materials standard ASTM D 5652 was used as a general guideline to establish rate of loading and general apparatus requirements (ASTM 2000). The test assembly was similar to that used by previous researchers (Blaß and Schlager 1996, Mantilla-Carrasco and Oliveira 1999, Bathon and Graf 2000) for comparative purposes. A steel test frame was designed and fabricated to withstand lateral forces resulting from the offset load and reaction points. A schematic of the test frame is shown in **Figure 7**. Load was applied by a 150-kN capacity Material Testing System (MTS) under deflec-

Table 1. — Glue properties.

Glue type	Published overlap shear strength	Working time	Application method	Manufacturer
	(MPa)	(min.)		
A	19.3	10	Applicator gun	Ace Super Strength Epoxy Gel
B	17.2	90	Premixed/plastic dispenser	3M™ Scotch-Weld™ 2216 B/A Gray
C	15.5	10	Applicator gun	Devcon® 2-Ton Epoxy
D	17.2	5	Applicator gun	Devcon® 5-Minute Gel
E	20.7	6 to 8	Applicator gun	Dayton Superior® Sure-Anchor (J-51)

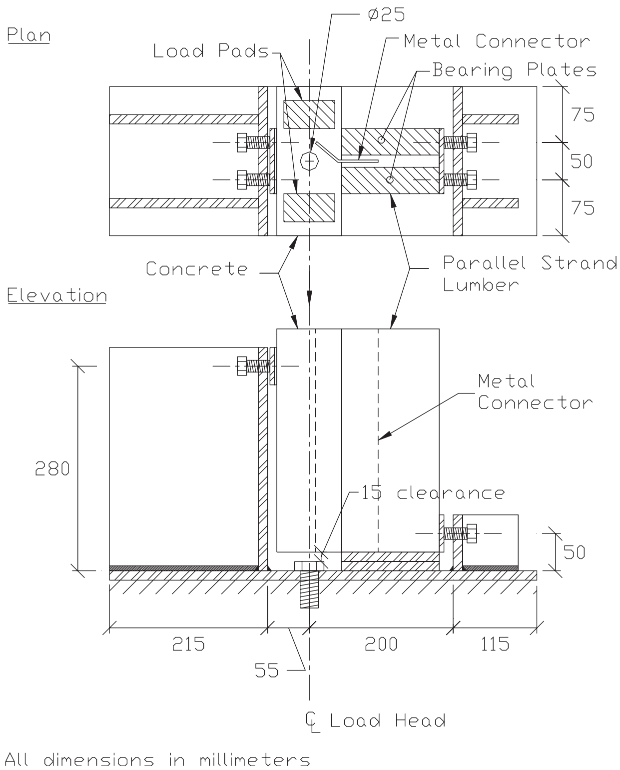

Figure 7. — Shear test frame schematic.

tion control mode at a rate of 0.8 mm/min. with computerized data acquisition. To avoid multi-axial stresses due to friction at the point of horizontal contact in the specimen, Teflon® pads were used between the concrete and the steel frame.

Shear test results

Ultimate failure load, corresponding shear stress, slip modulus, and failure mode are summarized in **Table 2**. Shear stress, determined by the ratio of maximum load over gross area of the steel to the wood, was assumed uniformly distributed over the shear plate for simplicity. It is noted that in this test the stress state in the glue is actually three dimensional with mechanical interlocking occurring in the steel voids. As a result, the calculated ultimate shear stress differs significantly from the published overlap shear strength of the adhesive provided in **Table 1**.

Slip modulus was calculated as the slope of the linear portion of the load/extension curve. In determining slip modulus, specimen extension was based on crosshead extension. Ideally, however, slip modulus is based on extension measured by separate gages (e.g., potentiometers) to minimize inaccuracies in the data from machine compliance. For this study, crosshead extension was deemed adequate since the objective was to compare the individual glue systems' performance. The initial portion of the curve shows a soft response due to specimen seating in the fixtures and was thus neglected.

For a visual depiction of shear behavior, applied load versus crosshead motion plots for each respective glue type are presented in **Figures 8a** to **8e**. Performance of glue types A and D (**Figs. 8a** and **8d**) are clearly unacceptable as the curves are highly variable (ultimate load coefficient of variation [COV] for A = 46.4%, and D = 22.1%) and lower

Table 2. — Tests results.

Glue type	Specimen	Ultimate load	Ultimate shear stress	Slip modulus	Failure mode
		(kN)	(MPa)	(kN/mm)	
A	A1	12.5	0.46	9.6	Glue shear
	A2	42.3	1.54	13.6	Glue bond at steel surface
	A3	52.0	1.89	20.7	Glue bond at steel surface
	A4	38.7	1.41	21.0	Glue bond at steel surface
B	B1	54.3	1.98	22.3	Steel shear
	B2	52.0	1.89	21.2	Steel shear
	B3	52.5	1.91	22.8	Steel shear
	B4	49.4	1.80	19.1	Steel shear/bond at steel surface
C	C1	52.5	1.91	19.5	Steel shear
	C2	49.4	1.80	21.9	Steel shear
D	D1	36.5	1.33	19.2	Glue shear
	D2	28.5	1.04	17.4	Glue shear
	D3	24.5	0.89	16.3	Glue shear
	D4	20.0	0.73	11.0	Glue shear
	D5	29.8	1.09	14.5	Glue shear
E	E1	48.9	1.78	13.5	Glue bond at wood surface
	E2	49.8	1.81	15.1	Steel shear
	E3	48.5	1.77	16.1	Bond at wood surface/steel shear
	E4	0.4	0.01	1.8	Glue did not cure

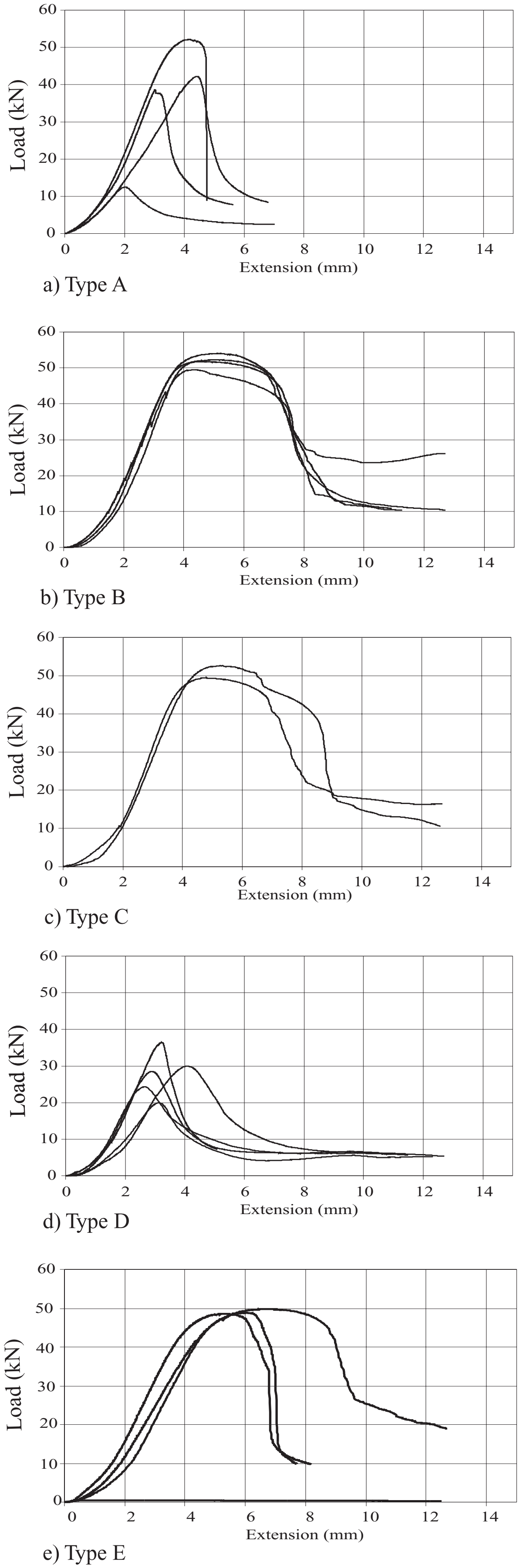

Figure 8. — Load vs. crosshead extension for shear tests.

strength and ductility are visibly apparent for most specimens. Type B and C glues (**Figs. 8b** and **8c**) however, produced excellent repeatability (ultimate load COV for B = 3.9% and C = 4.3%) as well as very high strength and ductility. Type E glue provided acceptable strength and consistency (except one specimen, due to poor mixture of components resulting in the glue not curing) but inferior ductility as compared to types B and C.

These results are consistent with the corresponding modes of failure reported in **Table 2**. It was noted that failure of the optimally performing glues (types B and C) was attributed almost exclusively to shear failure in the steel connector as shown by the sectioned specimen in **Figure 9**. In these cases, the favorable behavior (high strength and consistency as well as high ductility) is a result of yielding in the steel shear connector. The substandard glues (types A, D, and E) failed either in the glueline where glue residue was left on both wood and steel surfaces or at a wood or steel interface where the sectioned specimens revealed a clean surface as shown in **Figure 10**. It was therefore concluded that, as long as the bonding agent had sufficient strength to force failures into the steel shear connector, optimal connector performance could be achieved.

It was observed that the working time of the adhesive played an important role in the performance of the glues. Although care was taken to apply the glues within the working time given, glues with shorter working time (types D and E, in particular) were more difficult to work with. The inconsistent behavior of type E epoxy was likely due to inconsistent mixing in the duo-cartridge gun and thus poor curing of the epoxy. This is highlighted by specimen E4 in which essentially no capacity was realized prior to glue failure. For commercial applications, involving larger composite sections and more adhesive, long working time of the adhesive would certainly be required, giving a distinct advantage to type B.

Experimental bending test program: Phase 2

Specimen fabrication

Bending tests were conducted on two full-scale specimens, typical of that used in commercial applications, to evaluate overall system performance in bending.

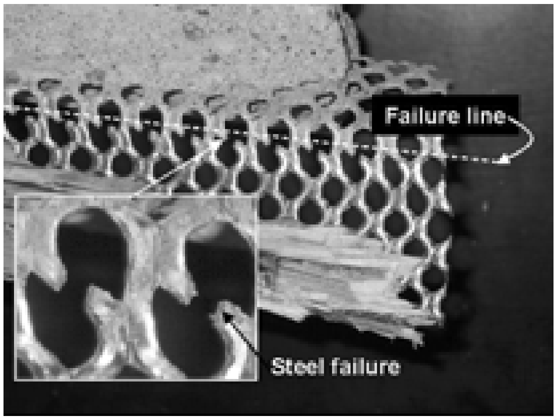

Figure 9. — Steel shear failure.

Figure 10. — Gluebond failure at steel surface.

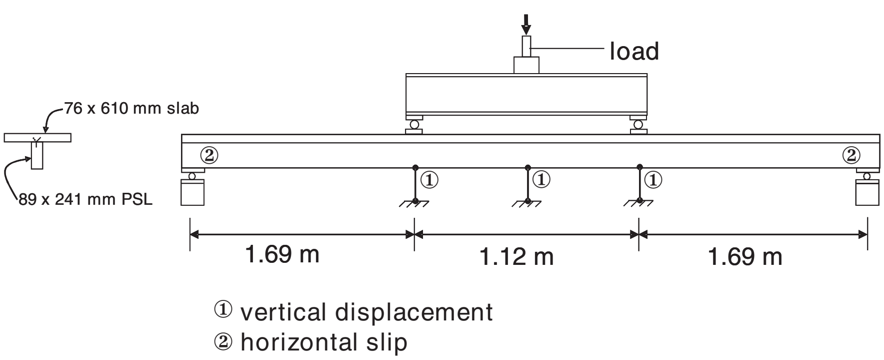

Figure 11. — Bending test set-up.

As illustrated in **Figure 11**, specimens consisted of an 89- by 241-mm southern pine PSL beam conjoined to a 76- by 610-mm concrete slab using one continuous row of steel mesh shear connectors. The connectors were identical in form to those used in the shear tests (with the exception that the length of the connector was 610 mm) and they were placed in the system in a similar manner. The longer connector was used to hasten installation and was not expected to influence final system capacity. Since the shear tests revealed type B adhesive to possess the best characteristics of the five glues investigated, the connectors were glued into a continuous rout in the PSL beam using type B adhesive.

Bending test setup

The center-to-center span was 4.5 m, as shown in **Figure 11**. The supports were pins with a bearing length of 90 mm. Two load points were applied 1.12 m apart centered about mid-span using a 445-kN capacity hydraulic actuator mounted to a steel spreader beam. The load was gradually stepped to failure within a timeframe of 20 to 30 minutes. Vertical beam displacement was monitored at mid-span and under both load points, and horizontal slip between the concrete and timber was measured over bearings using linear potentiometers.

Bending test results

The load/mid-span displacement curves of both beams are depicted in **Figure 12**. The results are notably consistent in terms of stiffness (EI_{beam1} = 7.2×10^9 kN·mm^2; EI_{beam2} = 7.7×10^9 kN·mm^2) and ultimate capacity ($P_{MAX\,beam1}$ = 119.1 kN; $P_{MAX\,beam2}$ = 118.3 kN). Observed behavior of the two specimens was also consistent. Horizontal slippage between the PSL and concrete was visible in the shear zone of the beam prior to ultimate failure. It was speculated and later confirmed, as depicted in the sectioned specimen in **Figure 13**, that plastic yielding occurred in the steel shear connectors. Neither steel-to-concrete failure nor adhesive failure was observed at any location. This was the desired result. As demonstrated in the shear tests, failure in the steel connector (as opposed to the concrete or wood) results in a favorably ductile response with optimum capacity. Also, the consistent constitutive behavior of steel is credited for the consistency of the results. Ultimate failure was brought on by tensile bending stresses in the PSL (**Fig. 14**).

The degree of composite action obtained in these tests is demonstrated in **Figure 12**. Theoretical values of 13.8 GPa and 27.9 GPa for Young's Modulus for PSL and concrete, respectively, were assumed. The lower bound line depicts beam performance assuming no composite action. In this case, it is assumed that the concrete is unable to carry bending stresses and is therefore ineffective. A transformed analysis was used to determine the upper bound line showing full composite action. Failure was assumed to be linear elastic and to occur in the PSL at an ultimate stress of 48 MPa. It is apparent from the curves that full composite action is possible using this system. Slight discrepancies in calcu-

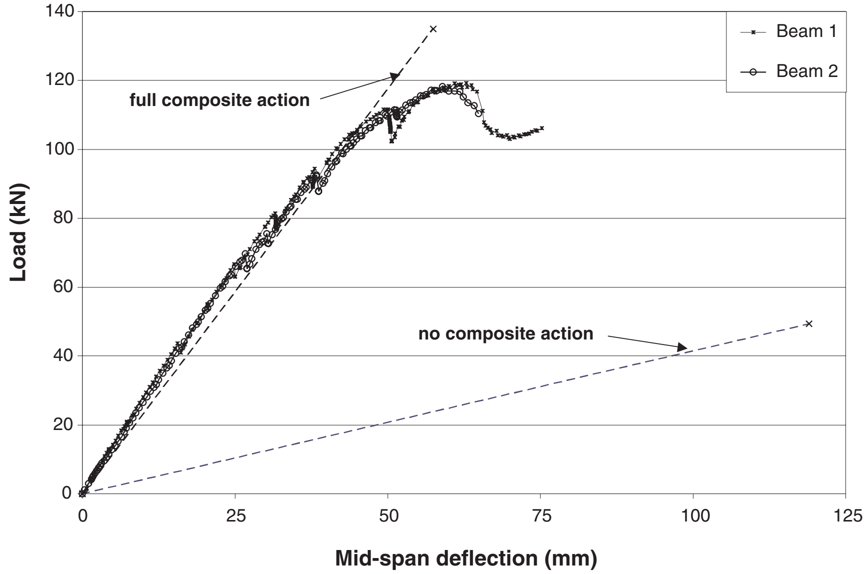

Figure 12. — Load versus mid-span displacement of composite beam specimens.

Figure 13. — Shear failure in steel mesh connector in composite beam specimen.

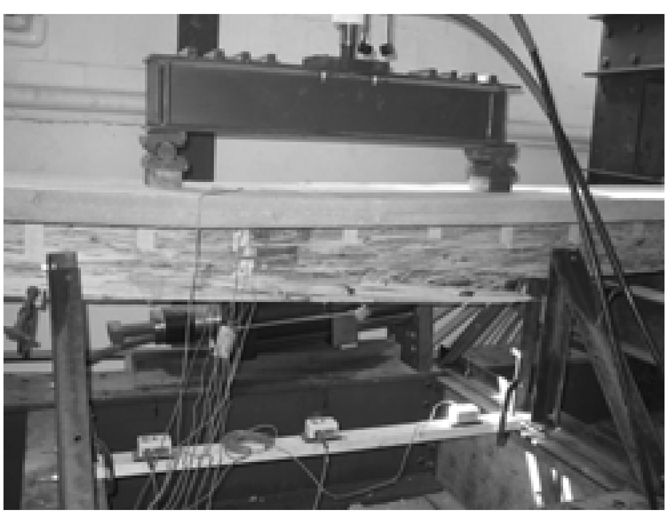

Figure 14. — Ultimate bending failure of wood-concrete composite beam.

lated versus attained capacities are likely due to actual versus nominal properties of materials.

Conclusions

This paper presents an experimental test program to evaluate a new continuous glued-in metal shear connector for wood-concrete composite floor and deck systems. The connector has recently received code approval for use in Europe. It consists of a steel mesh of which one half is glued into a slot in southern pine Parallam® PSL, while the other half is embedded into concrete. Five bonding agents were evaluated for this system in terms of performance and ease of application. One-sided push-out tests were conducted to yield ultimate shear strength and slip modulus for each glue type.

Overall, optimal performance was realized when failure was caused by yielding of the steel shear connector. This mode of failure resulted in the highest strength, stiffness, and consistency as well as the highest ductility relative to other modes of failure. Failure in the glueline or at a glue surface displayed poorer qualities. Two adhesive types gave consistent results and verified that shear can be effectively transferred between the PSL-concrete interface using meshed pulled metal plates.

Bending tests were also conducted on two full-scale specimens. Results from these tests were consistent in both measured and observed characteristics. Yielding initiated in the steel shear connectors in the shear zone of the beam, which ultimately led to bending tensile failure in the PSL. Comparing the experimental results to theoretical upper and lower bounds of composite action indicated that the system has near-full composite action behavior.

Further work needs to be conducted towards commercial development of this composite. Fundamental behavioral studies on areas such as durability, creep resistance, response to dynamic loading, as well as temperature and moisture fluctuations are necessary. Work in these areas is currently ongoing at the University of Massachusetts, Amherst.

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