Fatigue Testing of Wood-Concrete Composite Beams

Dr. Jeno Balogh

Department of Engineering Technology
Metropolitan State University of Denver
Denver, CO 80217

Dr. Rebecca Atadero

Department of Civil and Environmental Engineering
Colorado State University
Fort Collins, CO 80523

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ABSTRACT

Currently, wood-concrete composite structural members are usually applied in building structures. There are a relatively small number (in the low 100s) of known bridge applications involving wood-concrete composites. A problem with using these novel composite members in bridges with high traffic is that the fatigue behavior of the composite member under long-term repeated loading is not known. This report describes research performed in coordination with work at the University of Stuttgart, attempting to establish the S-N curve for fatigue loading of notched wood-concrete connections based on low/high-cycle, repeated loading tests. Experimental results are obtained on fourteen 1524 mm span composite beam specimens in which the wood and concrete are interconnected by embedded anchor screws at the notch locations. Five specimens are loaded statically while the others are cycled to failure with a maximum to minimum cyclic load ratio of 10. Points on the S-N curve are determined for three levels of the maximum load as a function of the average static failure load. Typical observed failure modes are block-shear of the wood at the notch and tension failure of the wood at mid-span. As a result, the obtained S-N curve could be proposed for future consideration in drafting design codes addressing the timber-concrete composite structures for bridges.

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EXECUTIVE SUMMARY

The behavior of wood concrete beam members under high-cycle repeated loading was investigated by the means of laboratory testing, to study the adequacy of wood-concrete composite beam systems to possible bridge applications.

Fourteen wood-concrete composite beam specimens were built. The wood and concrete layers were interconnected by embedded anchor screws placed in notches cut into the wood. In the experimental program, five specimens were loaded statically while the others were cycled, under pulsating load, to failure.

Points on the S-N curve were determined for three different levels of the maximum load as a function of the average static failure load. Typical observed failure modes were block-shear of the wood at the notch followed by tension failure of the wood at mid-span. The S-N curve obtained was compared to the Eurocode (EC-5) specifications for wood. It was found that for the shear-block failure mode of the wood within a composite member, the existing wood fatigue specifications could be used.

1. INTRODUCTION

Wood-concrete composite systems are constructed by interconnecting a wood layer with a concrete layer placed atop. In this way it is possible to exploit the benefits of both materials since the wood is subjected to tension coupled with bending, and the concrete slab is mainly compressed. The composite system, in fact, exhibits larger stiffness and strength, enhanced performance with respect to vibration, better seismic performance, and better appearance. In addition, there is the possibility to use the wood layer as a permanent formwork for the concrete slab (Ceccotti 1995), compared with non-composite systems.

A notched interlayer connection detail (Natterer 1996) was adopted in this study, with a vertical bearing surface (Kuhlmann and Schanzlin 2000, Natterer et al. 1996). The interconnection between wood and concrete is achieved by direct bearing of the concrete in the notch on the wood surface. Advantages of the notch detail include the higher composite action (i.e., stiffness of the composite system) and larger achievable load capacity (Balogh et al. 2008, Clouston et al. 2005, Dias 2005, Fast 2003, Kuhlmann and Schanzlin 2000, Natterer 1997) compared with mechanical connector detail.

The behavior of such members under high-cycle repeated loading is not yet well known due to the limited test data available (Kuhlmann and Aldi 2008, Kuhlmann and Aldi 2009, Miller 2009, Mueller and Rautenstrauch 2011). Knowing the fatigue performance of these members is essential in bridge applications. Nevertheless, a number of bridges have been built (Jutila 2010, Dias 2011), showing the adequacy of the wood-concrete composite system to bridge applications.

2. EXPERIMENTAL SETUP

2.1 Geometry

Fourteen 1626 mm (64 in) long wood-concrete composite beam specimens were built. The width of the specimens was 191 mm (7.5 in) and each had a 89 mm (3.5 in) thick wood layer consisting of five 2 in x4 in boards with nominal size of 38 mm x 89 mm (1.5 in x 3.5 in), as shown in Figure 2.1 and Figure 2.2, and a 63 mm (2.5 in) concrete layer. The wood and concrete layers were interconnected by embedded anchor screws, of 12 mm (0.5 in) diameter and 100 mm (4 in) long with an embedding depth of 50 mm (2 in), at the notch locations, as shown in Figure 2.3. The notches were cut into the wood 25 mm deep (1 in) and were 150 mm (6 in) long over the entire width of the specimen. The wood boards were attached to each other by 4 in deck screws 150 mm (6 in) apart. The wood layer was coated with a sealer paint (shown as white in the picture) to reduce the moisture entering the wood during the casting of the concrete layer. The paint was applied to the end of the wood boards as well.

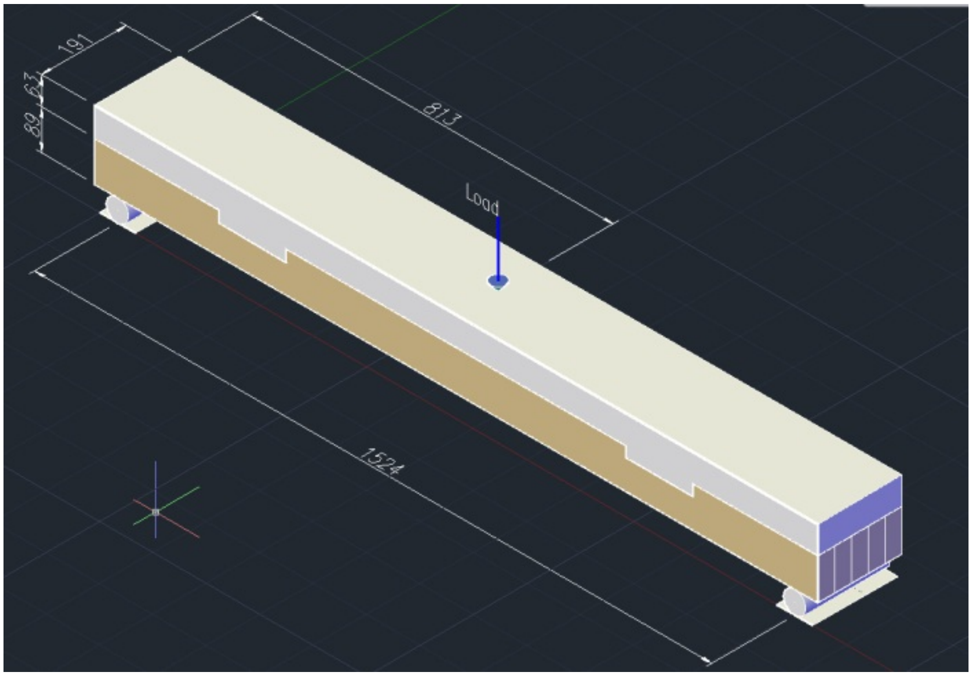

Figure 2.1 Specimen configuration [mm]

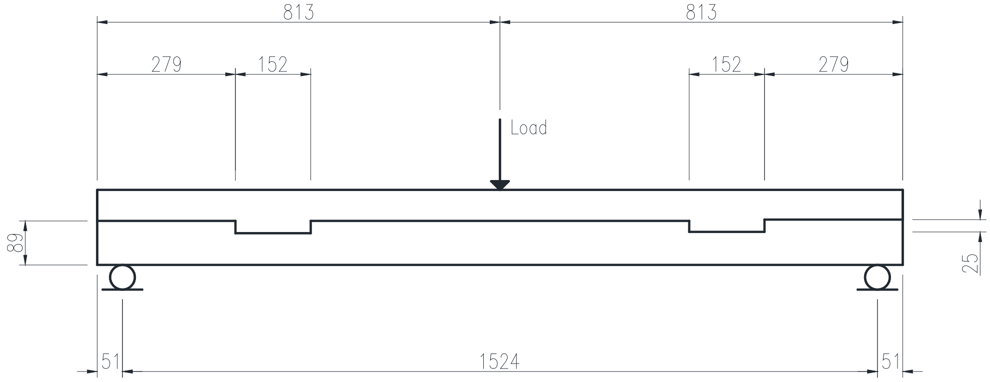

Figure 2.2 Specimen geometry [mm]

Figure 2.3 Specimens under construction with exposed notch detail

2.2 Material Properties

The wood material was premium-grade (No. 2 or better WWPA) kiln dried Hem-Fir with an average modulus of elasticity of 7.65 GPa (1110 ksi) at a measured average 6.3% moisture content, which is lower than the value specified by the NDS-2005 of 8.96 GPa (1300 ksi). The MOE value was determined by testing 28 wood samples according to ASTM D143 using an Instron 5569 device, while the moisture content was determined with a Delmhorst 11212 moisture mapping meter. The dry conditions are typical of the Colorado climate (the tests were conducted between October and March). The concrete material was a 21 N/mm^2 (3000 psi) type with glass fibers. Two 10 mm (3/8 in) longitudinal rebars were used.

2.3 Test Setup

The specimens were configured as simply supported with a span of 1524 mm (60 in). Loading was applied at the mid-span (three-point loading) using an MTS hydraulic loading and digital data acquisition system, as shown in Figure 2.4. Each support was configured as a roller to avoid horizontal forces straining the actuator. This setting resulted in a symmetric configuration (see Figure 2.2).

Some of the specimens were subject to static loading while others to cyclic pulsating loading. All loading was displacement controlled. The controlled displacement component was the deflection of the member at the point of the load application (mid-span). During the cyclic loading, the load was continuously monitored, and adjustments to the controlling displacement were made such to maintain the load range within +/-5% of its target value.

Figure 2.4 Typical test setup

3. RESULTS

3.1 Static Test Results

In order to determine the average static failure load, five specimens were tested using a procedure suggested by EN 26891, a European standard describing principles for determining strength and deformation characteristics of mechanical joints in timber structures. A 44.5 kN (10 kip) reference maximum load was assumed (which was found to be about a 21% under-estimation, which resulted in a slightly lower actual loading speed compared with the value suggested in EN 26891) during the conversion of the mostly load controlled curve provided by EN 26891 to a displacement controlled one. A typical load-deflection characteristic recorded is shown in Figure 3.1 (depicting the behavior of specimen B6).

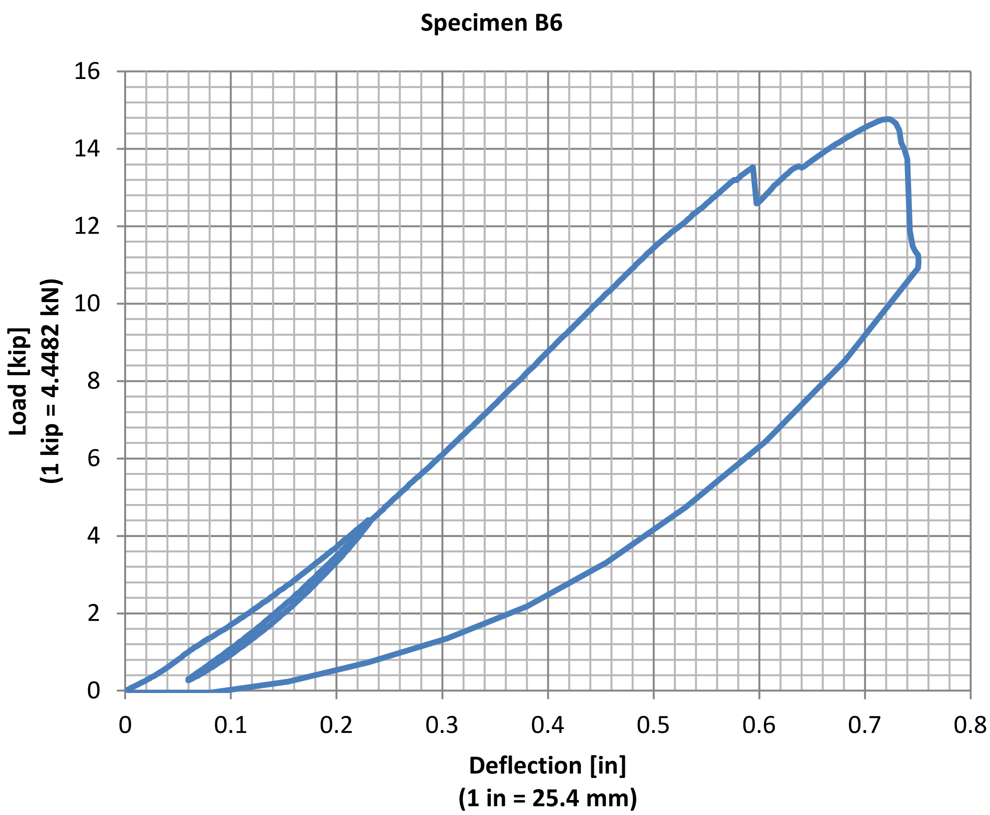

Figure 3.1 Typical load-deflection curve (Specimen B6)

The static test results are summarized in Table 3.1. The total failure load is the maximum load resisted by the specimen, and the initial failure load corresponds to the first failure (indicated by a drop in the load-deflection curve) noticed during the ramp loading to failure.

Table 3.1 Static test results

Specimen No.	Total Failure Load	
	(kN)	(kip)
B2	59.6	13.4
B3	52.0	11.7
B4	53.8	12.1
B5	50.7	11.4
B6	65.8	14.8
Avg.	56.4	12.7

In the following, $P_{s,max}$=56.4 kN (12.7 kip) will denote the average static total failure load. An initial failure in all specimens occurred as a block shear failure of a 2 in x4 in board, which was followed by other block-shear failures until a tension failure developed, always in the board with most defects (knots) in the mid-span region.

3.2 Fatigue Test Results

The load was cycled sinusoidally, at a frequency of 1 Hz, maintaining a maximum load, P_{max}, to minimum load, P_{min}, ratio of $R=P_{min}/P_{max}$=0.1, as shown in Figure 3.2.

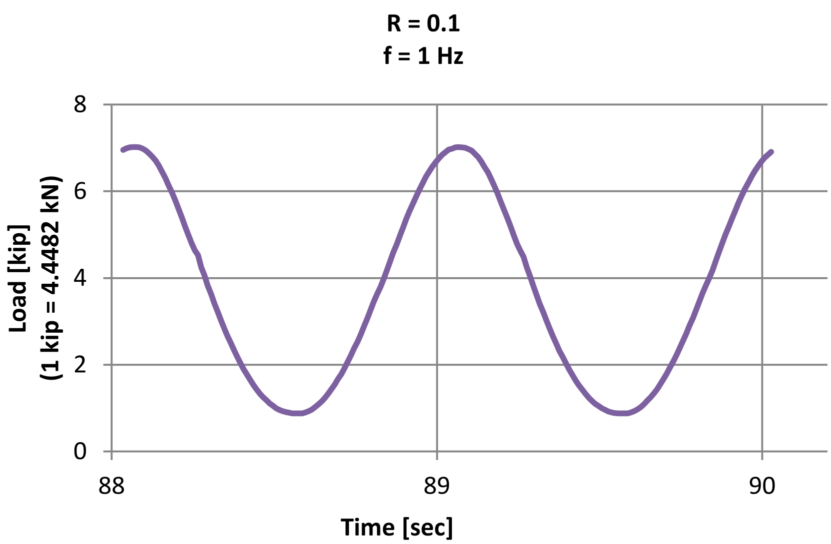

Figure 3.2 Typical pulsating load (Specimen F10)

Typical cyclic behavior of the test specimens observed corresponding to the loading shown in Figure 3.2 is depicted in Figure 3.3.

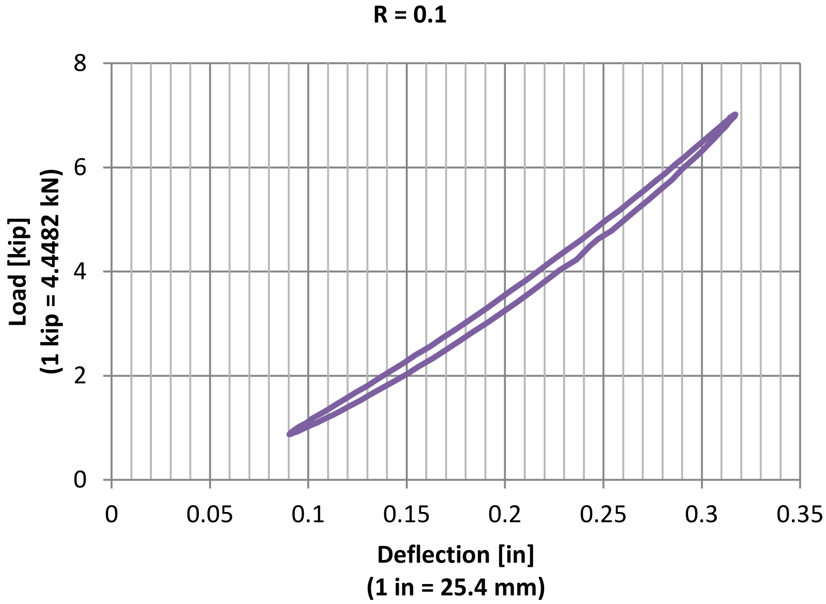

Figure 3.3 Typical behavior for a load cycle (Specimen F10)

Points on the S-N curve are determined for three levels of the maximum load as a function of the average static failure load, $P_{max}/P_{s,max}$ of 0.8, 0.7, and 0.6. The results are summarized in Table 3.2, Table 3.3, and Table 3.4, respectively.

Table 3.2 Fatigue Test Results for P_{max}=0.8$P_{s,max}$

Spec. No.	Initial Failure (cycles)	Total Failure (cycles)	Min Load P_{min} (kN)	(kip)	Max Load P_{max} (kN)	(kip)
F14	$10^{0.6}$	$10^{1.04}$	4.49	1.01	44.9	10.1
F15	$10^{2.99}$	$10^{3.5}$				
F19	$10^{0.9}$	$10^{2.1}$				

Table 3.3 Fatigue Test Results for P_{max}=0.7$P_{s,max}$

Spec. No.	Initial Failure (cycles)	Total Failure (cycles)	Min Load (kN)	(kip)	Max Load (kN)	(kip)
F11	$10^{4.2}$	$10^{4.2}$	3.96	0.89	39.6	8.9
F12	$10^{4.05}$	$10^{4.05}$				
F13	$10^{4.12}$	$10^{4.12}$				

Table 3.4: Fatigue Test Results for P_{max}=0.6$P_{s,max}$

Spec. No.	Initial Failure (cycles)	Total Failure (cycles)	Min Load (kN)	Min Load (kip)	Max Load (kN)	Max Load (kip)
F17	$10^{4.25}$	$10^{4.53}$	3.38	0.76	33.8	7.6
F18	$10^{4.88}$	$>10^{5}$				

In addition, specimen F10 was investigated at a load level P_{max}=0.5$P_{s,max}$, of 28.2 kN (6.35 kip) and no signs of failure were detected in the first 10^5 cycles.

A typical observed block-shear failure mode is presented in Figure 3.4, visible on the second and third boards from the left.

Figure 3.4 Typical block-shear failure (Specimen F11)

Typical observed tension failure (of the wood in bending) mode is presented in Figure 3.5. In all instances, block-shear of the wood at the notch occurred first.

Figure 3.5 Typical tension failure (Specimen F11)

Based on the fatigue test results, the mean S-N line is approximated in Figure 3.6, on a logarithmic scale.

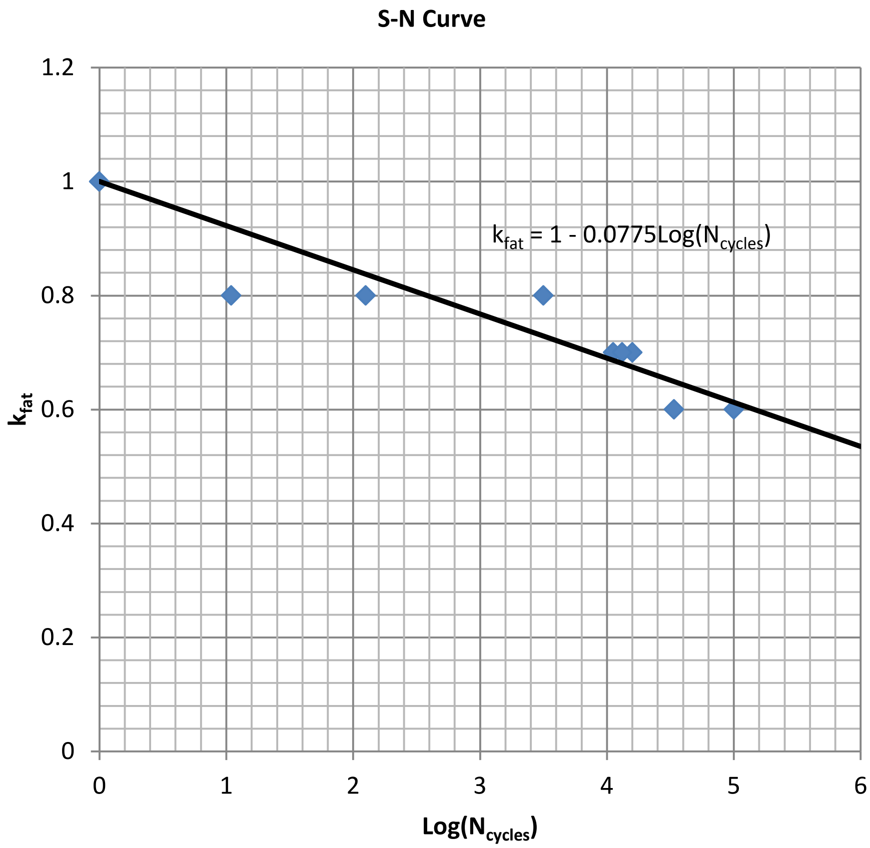

Figure 3.6 Mean S-N Line Approximation

4. CONCLUSIONS

This report presents the results of research at the Metropolitan State College of Denver, performed in coordination with work at the University of Stuttgart, attempting to establish the S-N curve for fatigue loading of notched wood-concrete connections based on low/high-cycle, repeated loading tests. Experimental results were obtained on 14 composite beam specimens. Five specimens were statically loaded while the others were cycled to failure.

Points on the S-N curve were determined for three levels of the maximum load as a function of the average static failure load. Typical observed failure modes were block-shear of the wood at the notch in some cases followed by tension failure of the wood at mid-span. The obtained S-N mean line allows for comparison with the Annex A of Eurocode 5: Design of Timber Structures EN 1995-2:2004, which describes verification of fatigue strength in timber bridges. Assuming the coefficient b = 1.3 (corresponding to wood in shear from Table A.1 of the Annex A) the coefficient a is found to be a = 9.68, which is more conservative than the value provided for the coefficient $a = 6.7$ in the Annex A for wood in shear.

Therefore, the results tentatively indicate that the provisions of EC5 EN 1995-2:2004 for fatigue verification could be used for the type of composite member studied herein, provided the expected failure mode is by shear of the wood at the notches. This is consistent with findings by Kuhlmann and Aldi [7].

Based on these results, the wood-concrete system could be considered for short-span, low-traffic secondary road bridges. This conclusion is tentative only, due to the small number of tests conducted in the study.

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