

# LOAD TESTS OF LARGE WOOD-CONCRETE BEAMS

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# ABSTRACT

Research at Colorado State University in wood-concrete layered composites for structural applications has been in progress for more than a decade. Key features of the research are experimental and analytical work to analyze notched shear key interlayer connections. These connections are being used to achieve composite action by transferring interlayer forces by bearing in a notch cut out of one material layer. This report documents a series of experimental load tests on thick-layered wood-concrete composite beams. The research of thick-layered beams is a precursor to possible future research of thick-layered wood-concrete bridge deck systems. Experimental results are given for two beams referred to as the deep beam specimens (12' long x 13.5" deep x 7.25" wide), and four beams referred to as the wide beam specimens (20' long x 15" deep x 2' and 3' wide). The results of the load tests are presented in terms of the measured beam deflections and the horizontal slip between the wood and concrete layers. Some load tests were eliminated due to various physical reasons which compromised them. Results for the loadings show that a low to moderate percentage of potential composite action was achieved. For the deep beam specimens the degree of composite action observed ranged from 64.22 percent to 84.22 percent of that theoretically possible if the layers were fully bonded to each other. The average value was 72.31 percent. These results are similar to those found by past researchers for narrow, shallow beams. Before the first loading, the deep beams were free to creep for approximately one year. This high average efficiency suggests creep effect did not significantly affect the deep beams. For the wide beam specimens the degree of composite action observed ranged from 11.81 percent to 46.86 percent and the average value was 27.35 percent. This is an unexpected low level. Before the first loading, two of the wide beams were free to creep for about two months. Two other wide beams were free to creep for about 28 months. The latter exhibited the lowest efficiencies. It appears that creep significantly affected the wide beams. Because creep itself was not measured, the effect of increased beam width versus creep could not be distinguished.



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# EXECUTIVE SUMMARY

Our nation's bridges are predominately on rural secondary roads and are critical to the movement of agricultural and mineral products. Low tax bases make saving every possible bridge repair and replacement dollar important. About 20 percent of the bridges use timber decking on either steel or wood stringers. Typically, timber decks in older bridges are the first component to wear and exhibit reduced load capacity. The usual consequence is the deck is completely replaced with a new one. Otherwise, the bridge using the deck would have to be either posted for lower load limits or closed.

In Europe, an innovative concrete overlay technique was recently developed to strengthen solid wood floors in apartment and office buildings. Interlayer shear transfer, needed to affect composite behavior, is achieved by a notched shear key. Wood-to-concrete bearing and shear in the notch materials affect the interlayer force transfer. A vertical anchor is glued into the wood and can be turned to tighten the concrete-to-wood bearing surfaces after hydration of the concrete has taken place. The shear key concept has potential as either a retrofit strengthening technique for old timber bridge decks or as new construction. However, because of much higher loads involved in bridges, the mixed material deck system must be much deeper than in residential and commercial floor systems. Consequently, in this study a series of load tests were conducted on large composite beam specimens as a low-cost approach in advance of possible load tests of full decks.

Two specimens (termed “deep beams”) involved beams with a span and depth (overall and for each material) consistent with what might be required in a short bridge deck. One specimen used a glued anchor connection such as developed in Europe. The second specimen used an alternative mechanical anchor connection which passed through the wood and was anchored against the bottom of the wood member. A second set of specimens (termed “wide beams”) was intended to examine the effect of increased span and member width on the load resistance on layered deep beams. Two specimens used a glued anchor connection and two used the mechanical anchor connection.

The effectiveness (efficiency) of the specimens was examined by comparing the measured displacements with those based on each of two theoretical bounds. One bound was with the two layers fully bonded to each other, the other bound was with complete absence of interlayer bond. Efficiency is an indicator of how close the behavior is to that of the fully bonded bound. For the deep beams with glued connections the average efficiency was 64.91 percent which is consistent with past studies of shallower beams. For the deep beams with mechanical connections the average efficiency was 83.55 percent. Thus, in these specimens, the mechanical connection performed at a much higher efficiency as compared to the glued connections and past studies of shallower members. Overall their average efficiency was 72.31 percent. For the wide beams with a glued anchor connection the average efficiency was 28.68 percent. For the wide beams with mechanical connections the average efficiency was 35.24 percent. Thus, the wide beams were not as noticeably different for each connection method. However they both exhibited a significant drop in efficiency (overall average efficiency was 27.35%) compared to the Deep Beams. Significant creep deflection likely occurred in the Wide Beam specimens and likely influenced the results for those specimens. Creep deflection was not measured, thus could not be considered quantitatively in this study. Consistency of construction and possible flaws (such as poor concrete consolidation at some locations) were other intangible factors in the low results for the wide beam specimens.

The results suggest that with proper construction, and control of creep, for example by showing the mixed wood-concrete construction for deep beams, is feasible. This is evidenced by the 72.31 percent efficiency of the deep beams. However, the tests were limited to ramp loadings and the long-term effect of repeated loading and possible fatigue of the connection should be examined. The wide beam specimens were compromised by the effects of creep and other physical effects. Thus they should be redone and no dependable conclusion is possible. Overall, quantitative studies of the creep phenomenon should be undertaken.

# 1. INTRODUCTION

## 1.1 Introduction

This report was derived from ongoing research conducted by researchers at Colorado State University (CSU). The project focuses on the innovative use of layered wood-concrete members for bridge structures. Current research at CSU involves composite wood-concrete beams of various depths. These beams consist of a bottom layer of dimension lumber and a top layer of reinforced concrete. The two layers are interconnected by a shear key / anchor detail that provides inter-layer force transfer. A primary focus of the research is to quantify the degree of composite action achieved by the layered beam system with notch type connections. Figure 1.1 shows the common definitions of complete, partial, and zero composite action.

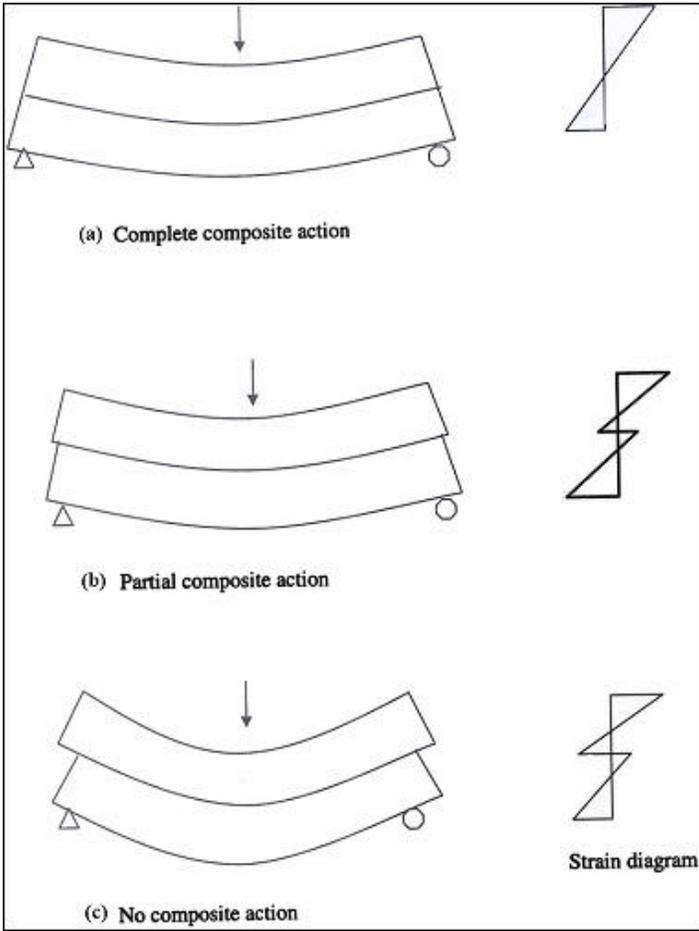


Figure 1.1 Composite Action Definitions

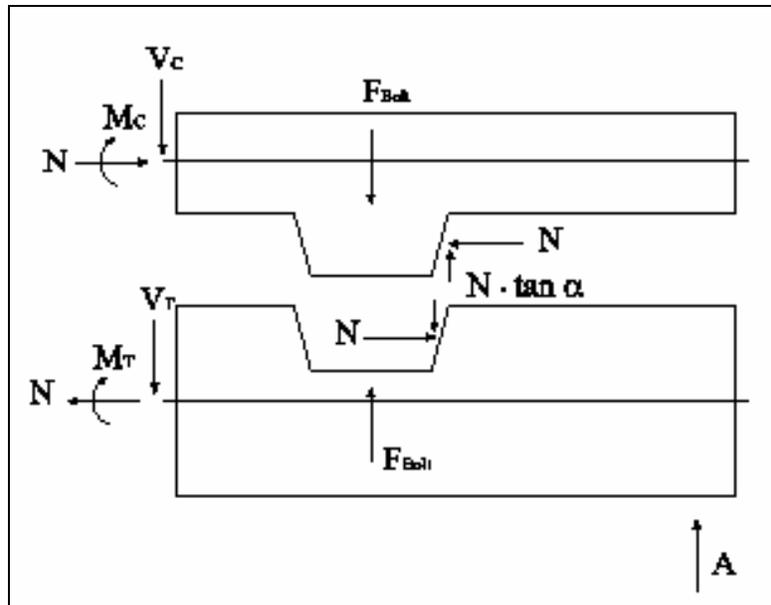
In this report, the degree of composite action is calculated for several composite wood-concrete beam specimens. A high degree of composite action is desired in layered wood-concrete beams because it leads to both reduced deflections and increased load-carrying capacity. In a simple bending member as shown in Figure 1.1, the bottom outermost fibers at mid-span are stressed in tension, whereas the top outermost fibers at mid-span are stressed in compression. The wood-concrete beam member is an effort to combine the compressive strength behavior of concrete with the tensile strength behavior of timber to provide an improved composite beam.

When complete composite action is realized the layered beam acts as a one-layer beam with mixed material properties. In this case the beam is stressed such that all or most of the concrete is in compression and all or most of the timber is in tension, depending on the depth of each material. Also there is complete transfer of stresses between the two layers on the layer interface, and no interlayer slip (relative horizontal movement) occurs (Figure 1.1 a). Complete composite action is the most efficient combination of the two materials in a layered beam configuration. Conversely, when the beam has no composite action, the behavior of the wood-concrete beam is that of an individual concrete beam deflecting on top of an individual timber beam. In this case, the concrete beam and the timber beam are both stressed in a combination of tension and compression. Furthermore in beams with no composite action there is no transfer of stresses between the two layers, and there is large relative movement of the concrete layer with respect to the wood layer, i.e. inter-layer slip occurs (Figure 1.1 c).

When connections are made between the concrete layer and the timber layer, partial composite action is developed. Although the different layers are stressed both in tension and compression, the situation is better than that for the case where there is no composite action. More of the concrete is stressed in compression, and more of the wood is stressed in tension. Interlayer slip does occur but it is smaller in magnitude than the slip developed in the beam with no composite action. Thus the case of partial composite action falls between the limits of no composite action (worst performance) and complete composite action (best performance). Throughout the remainder of this report the performance of the wood-concrete beams tested at CSU are discussed in terms of composite action and a closely related measure of composite efficiency.

The method of concrete-wood interconnection used in the CSU research consists of a notched shear key. The notched shear key transfers forces between the two layers via bearing contact on one side of the notch. The notch shape is trapezoidal, with inclined bearing surfaces to mitigate stress concentrations at the corners of the notch. As a result of the inclined bearing surfaces a non-zero vertical component of the bearing force develops. To equilibrate this vertical force component, and to close any gaps between the two layers in the notch, an anchor connection is used. Many types of anchor connections exist, but in this report only two types (glued-dowel, and mechanical) are examined.

On average, the glued dowel connectors were 3/8" diameter threaded rod with a length equal to the concrete depth plus the notch depth plus roughly half the wood layer depth plus roughly 1/2". The mechanical connectors were made of the same threaded rod with a rough length equal to the beam depth plus 1/2". The modulus of elasticity of the Hilti dowel is specified by the fabricators as  $2.1 \times 10^5$  N/mm<sup>2</sup> or  $30.46 \times 10^3$  kip/in<sup>2</sup>.



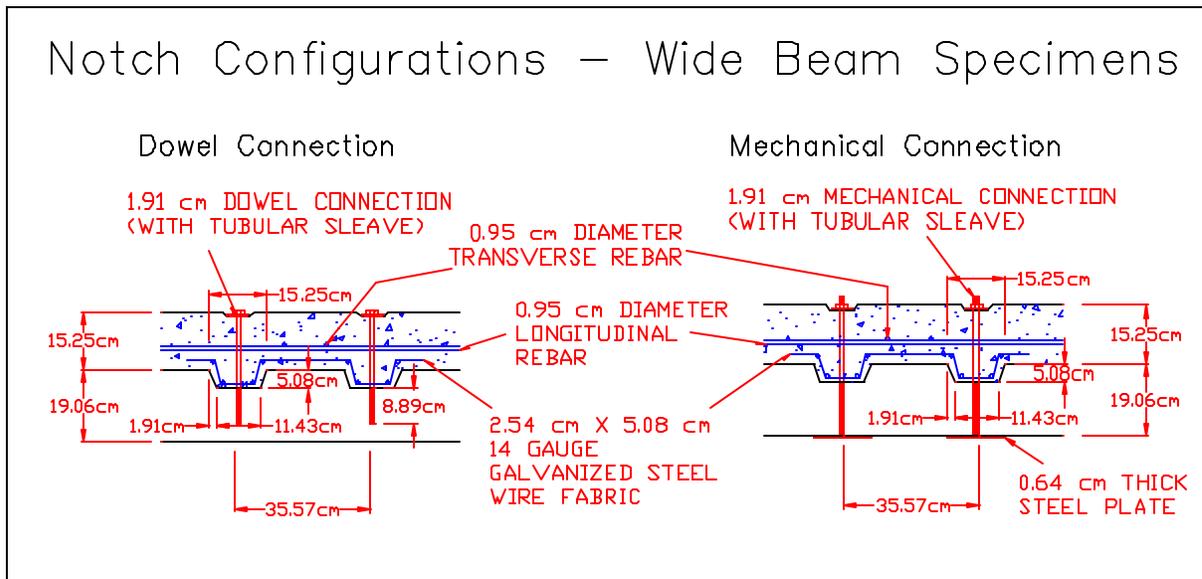
**Figure 1.2 Notched Shear Key Connection Free Body Diagram**

Some prior CSU research studies concentrated on the application of wood-concrete composites to floors in commercial buildings. These research studies included laboratory tests of a large number of small dimension beam specimens consisting of nominal 2" x 4" (3.81 x 8.89 cm dressed size) or 2" x 6" (3.81 x 13.98 cm dressed size) wood layers with a 5.08 cm – 7.62 cm (2" - 3") thick concrete layer. Tests of several thin-layered deck specimens of the same dimensions have also been completed at CSU. For bridge applications, higher flexural and shear resistance are required and deeper layered systems and stronger interlayer connections are needed. Originally a test of a full width bridge deck was planned, but it was determined that insufficient load carrying capacity was available in the existing test frame. Consequently, as a compromise a study was conducted of the behavior of wide beam specimens. These specimen dimensions were configured to be consistent with those that would be needed in a full bridge deck. Also, two much deeper beam specimens were prepared and tested for potential use in timber trestle bridge chords. These deep beam specimens involved nominal 8" x 8" (18.42 x 18.42 cm dressed size) solid sawn wood members and a 25.41 cm (10") thick concrete layer.

## 1.2 Overview of the Research

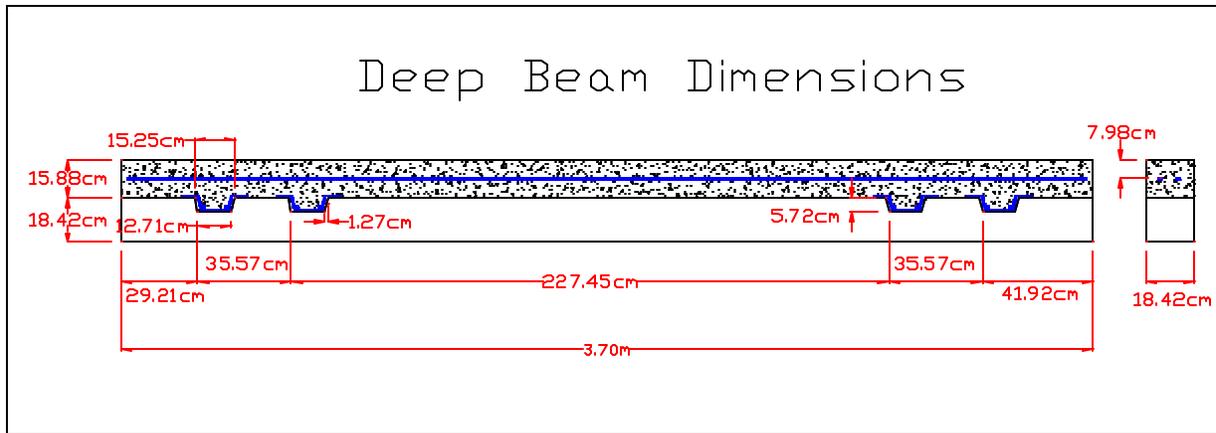
The study described was conducted from May 2001 to August 2003. Two different types of beams were constructed. The first beam type, referred to as a deep beam (DB), had a solid sawn Douglas-Fir wood layer. The second beam type, referred to as a wide beam (WB), had a bottom wood layer made from either 16 or 24, 5.08 cm x 25.41 cm (2" x 10") Hem-Fir No. 2 dimension lumber boards laterally nailed together. In all specimens the concrete layer was cast from premixed commercial concrete having a specified compressive strength of 2.41 kN/cm<sup>2</sup> (3500 psi). The concrete layer was reinforced according to the shrinkage and temperature effects given in ACI 318-99 concrete building code (ACI 1999). In all specimens the layers were interconnected using shear-key notches grooved out of the wood layer (Figure 1.3). For vertical anchorage either a dowel or mechanical connector was used (Figure 1.3). One type of connector (Glued-Dowel type) consisted of a short threaded rod glued into the bottom of the notch in the wood layer. The other type of connector (mechanical type) was a long threaded rod extending through a

hole drilled in the wood layer and attached to a steel bearing plate nailed to the bottom surface of the beam. In both cases a plastic sleeve was placed around the portion of the connector surrounded by the concrete layer. This was done to prevent bonding of the concrete with the steel connector, as it is necessary to tighten the connections after the concrete cures. The two different connector types are shown in Figure 1.3 with the notch dimensions of the wide beam specimens. The notch configurations for the deep beam specimens are identical to those shown in Figure 1.3 except that the depths of the wood and concrete layers are different.



### 1.3 Description of the deep beam specimens

Figure 1.4 shows the main longitudinal view, end view, and notch placement of the deep beam specimens. The beams had a span of 12.0 ft on simple supports. For each of the two deep beams the wood layer was comprised of a single surfaced dry 18.42 cm x 18.42 cm (7.25" x 7.25", i.e. nominal size 8" x 8") solid-sawn Douglas - Fir No. 1 timber. Each of the deep beams used a different type of notch anchor connection. Specimen 1, utilized the glued dowel type connection, and specimen 2 utilized a mechanical type connection. The moisture content of the timber was measured using a Delmhorst R-2000 electrical resistance moisture meter at a penetration depth of 3.81 cm (1.5"). Measured values were found to be less than 6 percent.



**Figure 1.4 Deep Beam Dimensions (cross-section and end-view)**

Two notches were made at each end of the beam, placed as shown in Figure 1.4. The glued-dowel notch connections utilized a 12.7 mm (1/2") diameter hole pre-drilled to a depth of 101 mm (4") into the wood layer of the notch. The hole was then tapped with a 15.98 mm tap and the dowel connectors were set in place after applying a HIT HY 150-epoxy adhesive into the tapped hole.

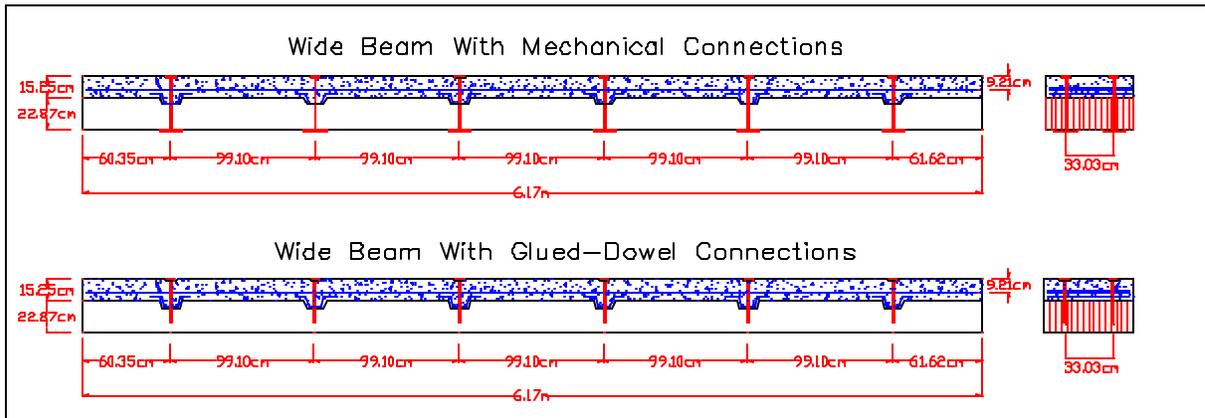
The mechanical dowel connection used a 12.7 mm (1/2") diameter hole pre-drilled through the entire depth of the wood layer. The dowel consisted of a threaded rod welded to a steel bottom plate. The dowel was put in place by inserting the threaded rod up through the bottom of the hole. Then, the dowel was fixed in place by screwing the steel plate to the bottom side of the beam using wood screws.

The construction of the deep beams was completed by casting a 15.88 mm (6.25") layer of reinforced concrete over each of the wood layers. The concrete was properly consolidated using a mechanical vibrator, with special attention focused at consolidating the concrete in the notches. Shrinkage and temperature steel reinforcement was placed to satisfy the requirements of the ACI 318-99 concrete building code (ACI 1999). Placement of the reinforcement for the notches of the Deep Beams is shown in Figure 1.3. Placement of the reinforcement for the Deep Beams is illustrated in Figure 1.4. After allowing the concrete to cure for 28 days a 70 N-m (50 lb-ft) torque was applied to all the connector nuts to re-tighten the notch connections.

## 1.4 Description of the wide beam specimens

The wide beams were similar to the deep beams, but had a longer span and thus larger dimensions. A drawing of the wide beam specimens is shown in Figure 1.5. The beams were 6.17 m (20 ft.) long with a 15.25 cm (6") deep concrete top layer and a 22.87 cm (9") wood bottom layer. The wood layer was constructed of 16 to 24 3.81 cm x 23.50 cm (nominal 2"x10") grade No. 2 Hem-Fir dimension lumber boards laterally nailed together. Two beams (wide beam No. 1 and wide beam No. 2) were 91.48 cm (3 ft.) wide, while the other two beams (wide beam No. 3, and wide beam No. 4) were 60.98 cm (2 ft.) wide. Each beam had six shear notches spaced evenly along the length of the beam. Notches were cut through the entire beam width, and notch dimensions were varied with respect to the type of inter-layer connection used in each notch (See Figure 1.3). Beams No. 1 and No. 4 had the glued dowel-type connections at the shear notch locations. Beams No. 2 and No. 3 had mechanical connections. The notches corresponding to the dowel connections were each 5.08 cm (2") deep with a 11.43 cm (4.5") bottom width and a 15.25 cm (6") top width. The notches corresponding to the mechanical connections were 3.81 cm (1.5") deep with a 12.71 cm (5") bottom width and a 15.25 cm (6") top width. A detail of the

notch configurations for the wide beams was previously shown in Figure 1.3. The placement of the notched shear keys in the wide beam specimens is shown in Figure 1.5. Note that two different widths were used for the wide beam specimens, namely 60.98 cm (2 ft.) and 91.48 cm (3 ft.). Figure 1.5 is representative of only the 60.98 cm (2 ft.) wide specimens. Except for their width, the 91.48 cm (3 ft.) wide specimens have all other dimensions identical to those shown in Figure 1.5. The lateral spacing of the notch anchor connections was 33.03 cm (13") for the 60.98 cm (2 ft.) wide specimens, and 49.55 cm (19.5") for the 91.48 cm (3 ft.) wide specimens.



**Figure 1.5 Wide Beam Dimensions (cross-sections and end-views)**

## 2. EXPERIMENTAL TEST PROGRAM

### 2.1 Experimental Test Program

The experimental test programs used for the wide beams and deep beams were similar. All of the beams were ramp loaded with a point load at mid-span using hydraulic actuators. Slip between the wood and concrete layers, was measured at the locations of the shear notches. Vertical deflections of the beam were measured at selected locations along the span. The deep beam load test setup is shown below in Figure 2.1. All of the measurements were taken using string potentiometer transducers (Figures 2.1 and 2.2). Both the wide beams and the deep beams were repeatedly ramped up to various load levels using the hydraulic actuators in stroke control.



**Figure 2.1 Deep Beam Test Setup**

Measurements were recorded at different levels along the ramp load tests. The 91.48 cm (3 ft.) wide beams with 6.10 m (20 ft.) spans (cast in March 2001) were tested in May and June of 2001. The 60.98 cm (2 ft.) wide beams with 6.10 m (20 ft.) spans (cast in March 2001) were tested in July and August of 2003. The deep beams (cast in March 2001) were tested in February and April of 2002.



**Figure 2.2 Wide Beam Test Setup**

## **2.2 Conduct of the Tests - Deep Beam Specimens**

Each of the two deep beam specimens was tested in same load frame using the same equipment. First, a simple support was built at each end of the 3.63 m (143") clear span. A cylindrical steel bar sandwiched between two steel plates was used to provide a roller support at one end. At the other end of the span a steel half-cylinder, with a thin steel plate on top, was bolted down to a rigid support to simulate a pin type support. The beams were rolled into place on the frame with care taken to not allow the beams to bend under their own weight. After configuring the beam to the load frame the specimen was then instrumented. Small holes were drilled into the concrete shear notches. Then a plastic anchor was fitted in each hole and glued in place with epoxy. Eye-screws were screwed into the plastic anchors and string potentiometers were attached to them. String potentiometers were then mounted on a bracket that was then attached to the wood layer of the beam with screws. For deflection measurements of the beam, string potentiometers were placed underneath the beam at the 1/3, 1/2, and 2/3 points along its span. String was used to attach the string potentiometer cables to eye-screws screwed into the bottom of the beam.

Before initiating the load tests all notch connectors were re-tightened to 70 N-m (51.63 lb-ft) using a torque wrench. This was done to close the gaps that had formed between the concrete and wood layers due to shrinkage. A single point load was applied through the use of two hydraulic rams connected with a distributor beam loading a steel half cylinder placed at mid-span of the simply supported 3.63 m (143") span. Table 2.1 includes the details of the tests conducted for each deep beam specimen. Deep beam 1 was subjected to six load tests, designated DB1.1 through DB1.6. In order of occurrence, these tests consisted of, five cycles each from 0 to 35.58 kN (8 kips) and back to zero; then 10 repetitions, each from 0 to 35.58 kN (8 kips) and back to zero; then 10 repetitions, each from 8.89 kN (2 kips) to 35.58 kN (8 kips) and back to zero; then a failure ramp load test from 0 to 61.55 kN (13.84 kips); and a final post-failure ramp load test from 0 to 81.22 kN (18.26 kips). Deep beam 2 was subjected to four load tests, designated DB2.1 through DB2.4. In order of occurrence these tests consisted of, five repetitions each from 0 to 35.58 kN (8 kips) and

back to zero; then eight repetitions from 8.89 kN (2 kips) to 35.58 kN (8 kips) and back to zero; then a failure ramp load test from 0 to 87.65 kN (19.71 kips); then a final post-failure ramp load test from 0 to 80.96 kN (18.2 kips).

**Table 2.1 Deep Beam Load Tests**

Beam	Date Tested	Load Test Designation	Connection Type	Number of Repetitions	Starting Load		Peak Load	
					kN	kips	KN	kips
Deep Beam 1	2/20/2002	DB 1.1	Glued-Dowel	5	0	0	35.58	8
Deep Beam 1	2/20/2002	DB 1.2	Glued-Dowel	10	0	0	35.58	8
Deep Beam 1	2/20/2002	DB 1.3	Glued-Dowel	10	8.89	2	35.58	8
Deep Beam 1	4/4/2002	DB 1.4	Glued-Dowel	5	0	0	35.58	8
Deep Beam 1	4/4/2002	DB 1.5	Glued-Dowel	0	0	0	61.55	13.84
Deep Beam 1	4/4/2002	DB 1.6	Glued-Dowel	1	0	0	81.22	18.26
Deep Beam 2	2/18/2002	DB 2.1	Mechanical	5	0	0	35.58	8
Deep Beam 2	2/18/2002	DB 2.2	Mechanical	8	8.89	2	35.58	8
Deep Beam 2	2/19/2002	DB 2.3	Mechanical	1	0	0	87.65	19.71
Deep Beam 2	2/19/2002	DB 2.4	Mechanical	1	0	0	80.96	18.2

### 2.3 Conduct of the Tests - Wide Beam Specimens

The four wide beam specimens were cast into place on the load frame in which they were tested. Figure 2.2 shows the setup of the load frame, string potentiometer transducers, and the position of the hydraulic actuator. The beams were moved into place for testing by carefully sliding the specimens along their supports. The simple supports for the beams were made using steel bars placed between two steel plates. A pin support was made at one end of the span by welding the steel bar in place, while at the opposite end the bar was left free to roll. The span for each of the Wide Beams was 6.1 m (20 ft.). Each beam was loaded at mid-span using the single actuator pressing against a thick square steel distributor plate. Instrumentation for the wide beams was installed in the same manner as for the deep beam specimens. The locations of the string-potentiometers varied between two sets of beams. For wide beams 1 and 2 deflection measurements of the beam were taken at the 1/4, 1/2, and 3/4 points along the span. For wide beams 3 and 4, deflection measurements were taken at the 1/8, 1/4, 3/8, 1/2, 5/8, 3/4, and 7/8 points along the span. For each of the four beams interlayer slip measurements were taken at all of the six notches on one side of the beams.

All the wide beams were subjected to two repeated ramp load tests, one starting at 0 load and the other starting from a non-zero load level. Before the tests all the notches were tightened with a torque wrench to 70 N-m (51.63 lb-ft). Table 2.2 shows the starting and ending load levels for each of the eight load tests. Wide beams 1 and 2 were subjected to five load repetitions from 0 to 177.92 kN (40 kips), then eight load repetitions from 44.48 (10 kips) to 177.92 kN (40 kips). Wide beams 3 and 4 were subjected to eight load repetitions from 0 to 88.96 kN (20 kips), then eight load repetitions from 22.24 kN (5 kips) to 88.96 kN (20 kips).

**Table 2.2 Wide Beam Load Tests**

Beam	Date Tested	Load Test Designation	Connection Type	Number of Repetitions	Starting Load		Peak Load	
					kN	kip	kN	kip
Wide Beam 1	5/17/2001	WB 1.1	Glued-Dowel	5	0	0	177.92	40
Wide Beam 1	5/21/2001	WB 1.2	Glued-Dowel	8	44.48	10	177.92	40
Wide Beam 2	5/17/2001	WB 2.1	Mechanical	5	0	0	177.92	40
Wide Beam 2	5/24/2001	WB 2.2	Mechanical	8	44.48	10	177.92	40
Wide Beam 3	7/29/2003	WB 3.1	Mechanical	8	0	0	88.96	20
Wide Beam 3	7/30/2003	WB 3.2	Mechanical	8	22.24	5	88.96	20
Wide Beam 4	8/4/2003	WB 4.1	Glued-Dowel	8	0	0	88.96	20
Wide Beam 4	8/5/2003	WB 4.2	Glued-Dowel	8	22.24	5	88.96	20

## 3. RESULTS OF THE TEST PROGRAM

### 3.1 Calculation of Efficiency

For each cycle of each load test the following results were either recorded or calculated: the maximum measured deflection at mid-span ( $\Delta$ ), the applied load at mid-span corresponding to the maximum deflection, the ratio of clear span to maximum mid-span deflection, the theoretical non-composite deflection ( $\Delta_{NC}$ ), the theoretical fully-composite deflection ( $\Delta_{FC}$ ), the available composite action, the composite efficiency, and the composite action observed. A complete presentation of the results is given in Appendix A. A group of selected results is described in the subsequent sections.

The theoretical non-composite deflection was calculated using a composite flexural stiffness of the beam equal to the sum of the EI value for the wood and concrete layers, where EI is the product of the modulus of elasticity ( $N/cm^2$ ) and the moment of inertia ( $cm^4$ ). The moment of inertia used to compute EI was simply calculated from the dimensions of the sections. Fully composite deflection values were computed by a transformed sections approach. In this approach the concrete layer was transformed into an equivalently stiff wood layer and the beam was analyzed as a one-layer homogenous wood beam. The analysis uses the E of the wood layer and the moment of inertia of the transformed section. For the transformed concrete layer the depth was the same as the original concrete layer, but the transformed width was equal to the original concrete width scaled by the modular ratio of the E of the concrete to the E of the wood.

It should be noted, however, that computed fully composite deflections are smaller than actual fully composite deflections. In the fully composite deflection calculation using transformed sections it is assumed that the two layers of the beam form a continuous bond along the length of the beam. However, because in the actual beams interlayer slip is only resisted at the notch locations the computed fully composite deflection is under-estimated. Furthermore, for both the non-composite and the fully composite cases the deflection at mid-span was calculated using elementary beam deflection formula for simply supported beams according to Equation 1 below. In this calculation shear deformation is neglected, which also introduces a slight error into the efficiency calculations.

$$\Delta = \frac{PL^3}{48EI} \quad \text{Equation 1}$$

The composite action available, composite efficiency, and composite action observed was calculated using the measured maximum deflection and load values for each load test cycle. The equations used are given below.

$$\text{Composite Action Available (CAA)} = \frac{\Delta_{NC} - \Delta_{FC}}{\Delta_{NC}} \quad \text{Equation 2}$$

$$\text{Composite Efficiency (EFF)} = \frac{\Delta_{NC} - \Delta}{\Delta_{NC} - \Delta_{FC}} \quad \text{Equation 3}$$

$$\text{Composite Action Observed} = \text{EFF} * \text{CAA} \quad \text{Equation 4}$$

Detailed test results for all the specimens are provided in Appendix A.

### 3.2 Deep Beam Test Results

As a typical example, Table 3.1 lists the results observed for load test DB1.2. All results refer to mid-span deflections, and each row of the table corresponds to one cycle of the load test. The second and third columns of the table give the measured maximum mid-span deflection and the load recorded at the maximum deflection reading respectively. The fourth column shows the ratio of beam length to maximum mid-span deflection. The fifth and sixth columns give the predicted fully composite and fully non-composite mid-span deflections calculated using Equation 1. The remaining three columns of the result tables give the composite efficiency calculations based on Equations 2, 3, and 4.

**Table 3.1 Deep Beam 1 Load Test 2**

Deep Beam 1 - Glued Dowel Connection								
Midspan Deflection Results - Load Test 2								
1	2	3	4	5	6	7	8	9
Repetition	Max. D (cm)	Load (kN)	L / D	D <sub>NC</sub> (cm)	D <sub>FC</sub> (cm)	Composite Action Available	Composite Efficiency	Composite Action Observed
1	0.90	34.9	405	1.74	0.46	73.76%	65.08%	48.00%
2	0.93	35.7	395	1.78	0.47	73.76%	64.79%	47.79%
3	0.94	36.6	388	1.82	0.48	73.76%	65.36%	48.21%
4	0.94	36.4	389	1.81	0.48	73.76%	65.23%	48.11%
5	0.94	36.2	390	1.80	0.47	73.76%	65.04%	47.97%
6	0.94	36.3	389	1.81	0.47	73.76%	65.00%	47.94%
7	0.95	36.6	386	1.82	0.48	73.76%	65.08%	48.00%
8	0.95	36.5	387	1.82	0.48	73.76%	64.91%	47.88%
9	0.95	36.4	387	1.81	0.47	73.76%	64.65%	47.69%
10	0.94	36.0	389	1.79	0.47	73.76%	64.33%	47.45%
<b>Average</b>	0.94					73.76%	64.95%	47.90%

The modulus of elasticity value for the deep beams was taken as 896.3 kN/cm<sup>2</sup> (1300 ksi) for the wood, and as 2190 kN/cm<sup>2</sup> (3177 ksi) for the concrete. The modulus of elasticity of the concrete was calculated from the average of six standard compressive strength test cylinders tested at 28 days after placement of the concrete. This was done in accordance with the ACI 318-99 standards. The modulus of elasticity of the wood was taken as the tabulated value in the American Forest & Paper Association’s 1997 edition of the National Design Specifications for Wood Construction.

From Table 3.1 it is evident that a gradual decrease in composite efficiency was observed for successive repetitions of the loading. This gradual decrease was observed for both deep beam specimens. It is attributed to the notch connection loosening as the number of load test repetitions increases. Further, it has been noted that re-tightening the notch connections is necessary after allowing wood-concrete composite beams to cure in place for extended periods of time. Unfortunately, because of unexpected delays due to equipment failure the Deep Beam 1 was left to creep under its own weight for nearly one year before load testing began.

Table 3.2 presents a summary of the results for all the deep beam load tests. Large increases in composite efficiency (23%) and composite action observed (17%) were found in load test DB1.2 with respect to load test DB1.1. These differences are attributed to the fact that the notch connections in deep beam 1 were not re-tightened before initiating load test DB1.1. This was noticed after load test DB1.1 and the notch connections were then re-tightened by applying a 70 N-m (51.63 lb-ft) torque to the connectors. Also, as

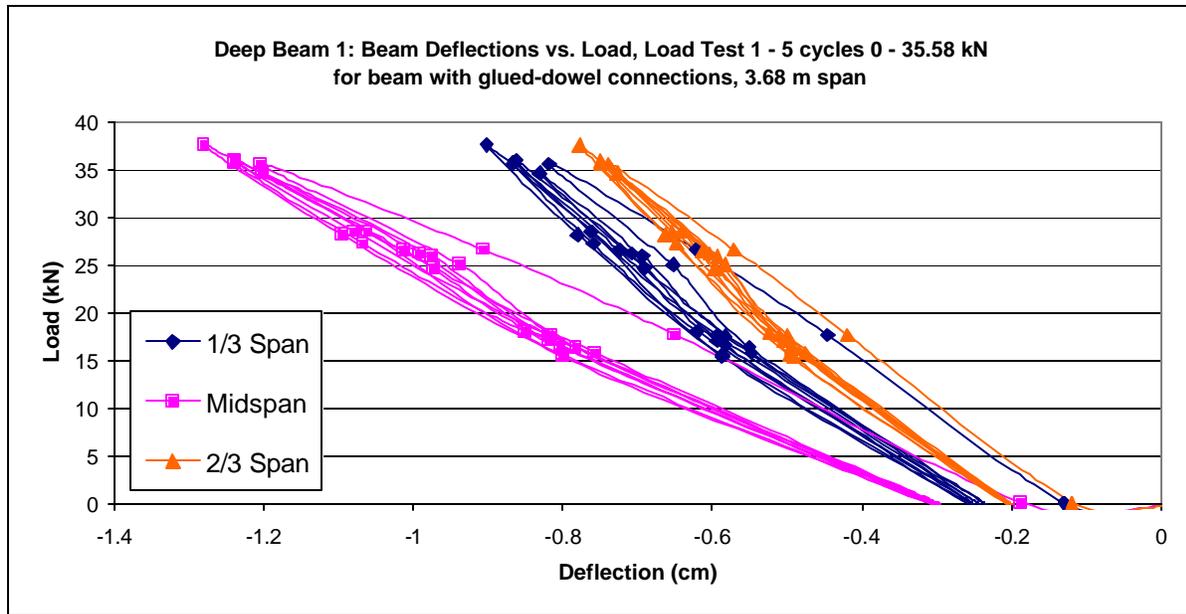
noted above, deep beam 1 was in place for nearly one year before testing. During that time creep deflections occurred. Thus load test DB1.1 shows the response for loosened notch connections after a long period of creep. The subsequent tests for deep beams, load tests DB1.2 – DB1.5, are more representative of the service behavior of deep beam 1 with tightened notch connections. However, about six weeks of creep occurred between tests DB1.3 and DB1.4. For deep beam 2, the connections were retightened before each load test, including DB2.1. Thus tightening was not an issue. However this specimen experienced creep from the time of casting until the load test of DB2.1. This initial period of creep was the same for deep beam 1, i.e. about one year.

**Table 3.2 Deep Beam Test Results for All Load Tests**

Beam	Date Tested	Load Test	Connection Type	Number of Repetitions	Composite Action Available	Composite Efficiency	Composite Action Observed
Deep Beam 1	2/20/2002	DB 1.1	Glued-Dowel	5	73.76%	41.86%	30.88%
Deep Beam 1	2/20/2002	DB 1.2	Glued-Dowel	10	73.76%	64.95%	47.90%
Deep Beam 1	2/20/2002	DB 1.3	Glued-Dowel	10	73.76%	65.79%	48.52%
Deep Beam 1	4/4/2002	DB 1.4	Glued-Dowel	5	73.76%	64.24%	47.38%
Deep Beam 1	4/4/2002	DB 1.5	Glued-Dowel	0	73.76%	59.36%	43.78%
Deep Beam 1	4/4/2002	DB 1.6	Glued-Dowel	1	73.76%	45.96%	33.90%
Deep Beam 1 Average (excluding failure load tests DB1.5 – DB1.6)					73.76%	59.21%	43.67%
Deep Beam 2	2/18/2002	DB 2.1	Mechanical	5	73.76%	82.87%	61.12%
Deep Beam 2	2/18/2002	DB 2.2	Mechanical	8	73.76%	84.22%	62.12%
Deep Beam 2	2/19/2002	DB 2.3	Mechanical	1	73.76%	69.71%	51.41%
Deep Beam 2	2/19/2002	DB 2.4	Mechanical	1	73.76%	-28.97%	-21.37%
Deep Beam 2 Average (excluding failure load tests DB2.3 – DB2.4)					73.76%	83.54%	61.62%

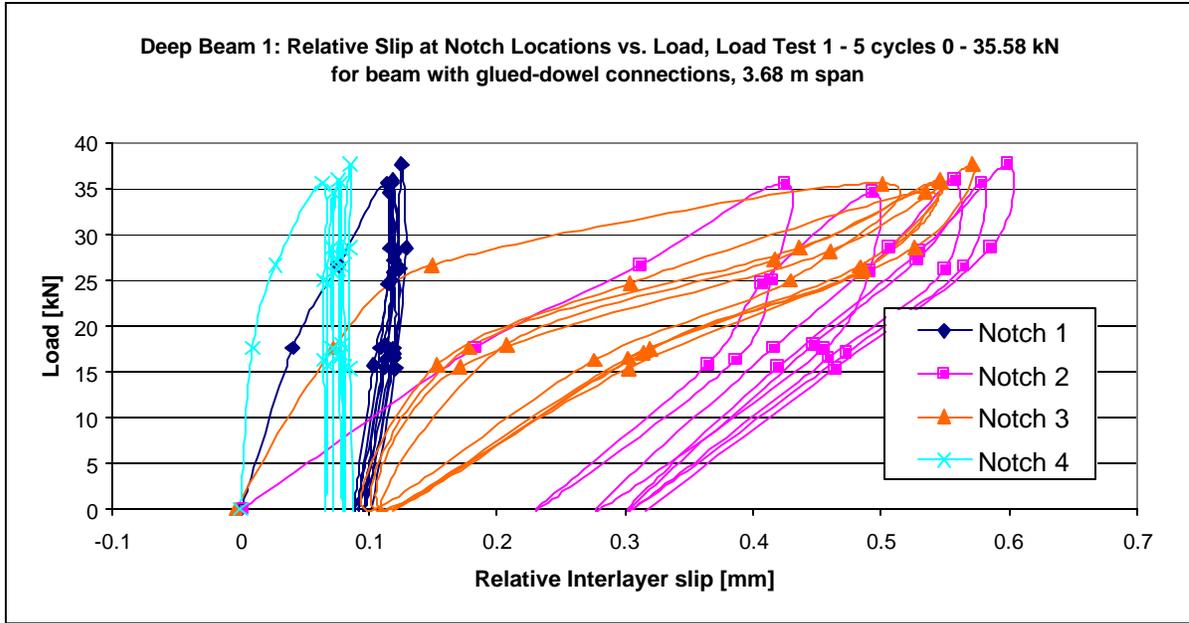
The small changes in composite action observed in going from load test DB1.2 to load test DB1.3 to load test DB1.4 for deep beam 1 seem to be the result of the gradual decrease in composite efficiency due to connector loosening. Load test DB1.6 shows sharply decreased values of composite efficiency and composite action. This is because this was a post failure load test conducted after the failure load test DB1.5. Similarly, a noticeable (14.5%) drop off in composite efficiency occurred in load test DB2.3 compared to DB2.4. This was because DB2.3 was a failure load test and DB2.4 was a post-failure load test. The negative efficiency value found from test DB2.4 simply means that the already failed specimen deflected beyond its theoretical fully non-composite limit.

Results for all of the load tests for deep beams 1 and 2 were similar to those shown in Table 3.1 and are provided in the Appendix A. Results for deep beam 2 showed a higher average composite efficiency (83.5%) as compared to deep beam 1 (59.2%). This is possibly due to the different connector type, but could also be from a number of other causes, e.g. better concrete consolidation in the notches or less knot defects or less cracks in the wood layer. The results of the failure load test (load tests DB2.3 and DB2.4) for deep beam 2 and for deep beam 1 (load tests DB1.5 and DB1.6) are not directly comparable. This is because deep beam 2 was forced to deflect beyond its theoretical fully non-composite limit, whereas deep beam 1 was not.



**Figure 3.1 Load Test DB1.1 Load vs. Mid-Span Deflection Response**

Figures 3.1 and 3.2 show the observed load-deflection and load-slip characteristics, respectively, for load test DB1.1. These plots are representative of the typical results for all the deep beam load tests. In general, after the first loading an approximately linear elastic load-deflection was observed. The typical load-slip relationship, as shown in Figure 3.2, exhibited much more non-linearity and in-elastic deformation than the load-deflection characteristic. The overall load-deflection and load-slip response of deep beam 2 had similar properties as those of deep beam 1. However, the similarity of the responses of deep beam 1 and deep beam 2 was not evident in the failure load tests. Deep beam 1 and deep beam 2 experienced completely different types of failure. Deep beam 1 failed in a mid-span flexural type failure mode in the wood, (Figure 3.3). Deep beam 2 failed due to cracks propagating in the wood layer behind the outside notch, effectively failing the notch (Figure 3.4).



**Figure 3.2 Load Test DB1.1 Load vs. Relative Interlayer Slip Response**



**Figure 3.3 Deep Beam 1 Failure at Mid-Span**



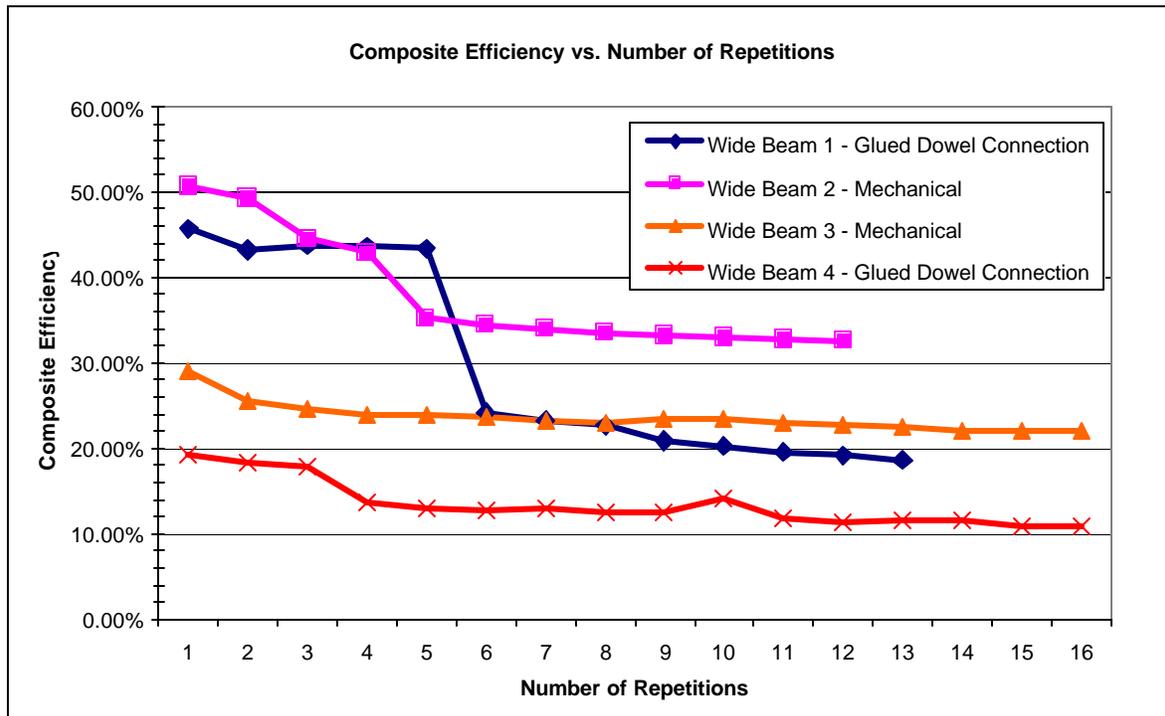
**Figure 3.4 Deep Beam 2 Failure at Notch Connections**

### **3.3 Wide Beam Test Results**

Again, the modulus of elasticity of the concrete was based on standard compressive strength test cylinders tested at 28 days. However load tests were conducted long after the 28-day cure period and thus the value used under estimates of the actual value at the time of the test. For the wide beams the modulus of elasticity of the wood was taken as  $1103 \text{ kN/cm}^2$  (1600 ksi) and the modulus of elasticity of the concrete was taken as  $2190 \text{ kN/cm}^2$  (3177 ksi). Note that after being constructed with a 15.25 mm (6 inch) layer of concrete placed on top of the wood layer, the wide beam specimens were un-shored until the load tests were conducted. However, the deflections due to creep were not measured. Deflection measurements recorded in the various load tests were due to the applied load only. The modulus of elasticity of the wood layer ( $1103 \text{ kN/cm}^2$ ) was again taken as the tabulated value in the American Forest & Paper Association's 1997 edition of the National Design Specifications for Wood Construction. The results of the load tests on the four wide beam specimens are shown in Table 3.3.

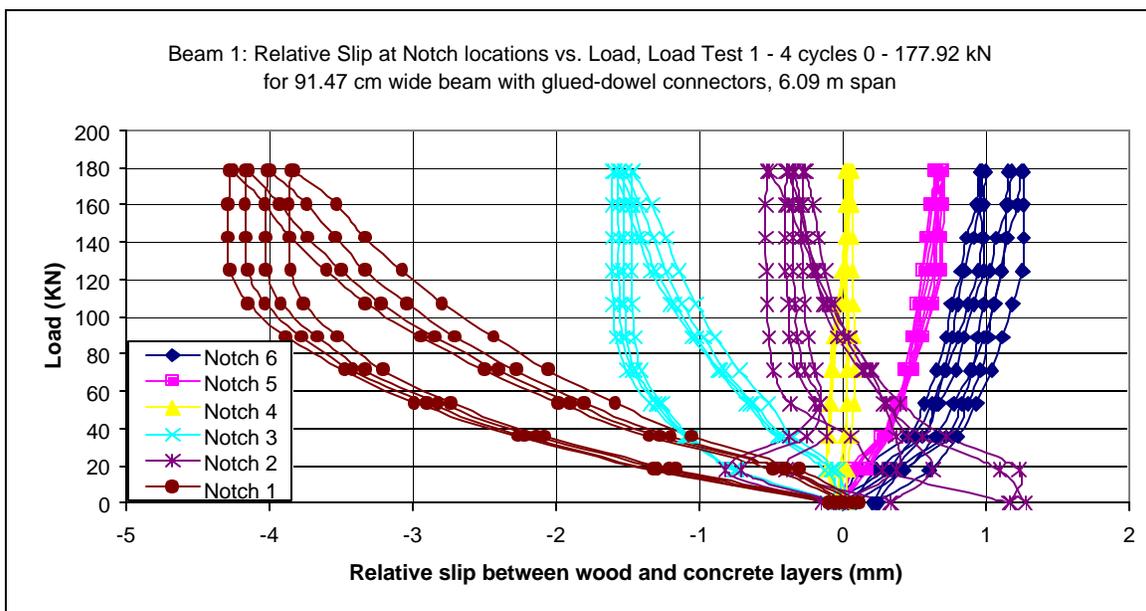
**Table 3.3 Wide Beam Load Test Results**

Beam	Date Tested	Load Test	Connection Type	Number of Repetitions	Composite Action Available	Composite Efficiency	Composite Action Observed
Wide Beam 1	5/17/2001	WB 1.1	Glued-Dowel	5	74.93%	43.92%	32.91%
Wide Beam 1	5/21/2001	WB 1.2	Glued-Dowel	8	74.93%	21.07%	15.79%
<b>Wide Beam 1 Average</b>					<b>74.93%</b>	<b>32.50%</b>	<b>24.35%</b>
Wide Beam 2	5/17/2001	WB 2.1	Mechanical	5	74.93%	46.86%	35.11%
Wide Beam 2	5/24/2001	WB 2.2	Mechanical	8	74.93%	33.60%	25.17%
<b>Wide Beam 2 Average</b>					<b>74.93%</b>	<b>40.23%</b>	<b>30.14%</b>
Wide Beam 3	7/29/2003	WB 3.1	Mechanical	8	74.98%	24.63%	18.46%
Wide Beam 3	7/30/2003	WB 3.2	Mechanical	8	74.98%	22.63%	16.97%
<b>Wide Beam 3 Average</b>					<b>74.98%</b>	<b>23.63%</b>	<b>17.72%</b>
Wide Beam 4	8/4/2003	WB 4.1	Glued-Dowel	8	74.98%	15.05%	11.28%
Wide Beam 4	8/5/2003	WB 4.2	Glued-Dowel	8	74.98%	11.81%	8.86%
<b>Wide Beam 4 Average</b>					<b>74.98%</b>	<b>13.43%</b>	<b>10.07%</b>



**Figure 3.5 Wide Beam Composite Efficiency Results**

In general the wide beam specimens showed a low degree of composite efficiency in general and compared to the deep beams. The range was 11.81 percent to 46.86 percent compared to 41.86 percent to 84.22 percent for the deep beams. Figure 3.5 shows the observed trend of gradually decreasing composite efficiency with respect to increasing number of load test repetitions. Wide beam 1 exhibited a significant drop in efficiency after load test WB1.1. However during load test WB1.2 the drop in efficiency was low and very gradual. Wide beam 2 exhibited a low and gradual drop in efficiency during load test WB2.1. However, similar to wide beam 1 a large drop in efficiency occurred after load test WB2.1 and a gradual drop in efficiency was observed during the loading sequences of load test WB2.2. The large drop in composite efficiency observed for wide beams 1 and 2 appear to be isolated cases. Considering the large (15-20%) and sudden drop, it is possible that unobservable failures occurred after the first load tests of wide beams 1 and 2. Figure 3.6 and Figure 3.7 show data which suggest this suspicion. Figure 3.8 shows the notch numbering scheme referred to in the preceding graphs and following discussion.



**Figure 3.6 Load Test WB1.1 Load vs. Deflection Response**

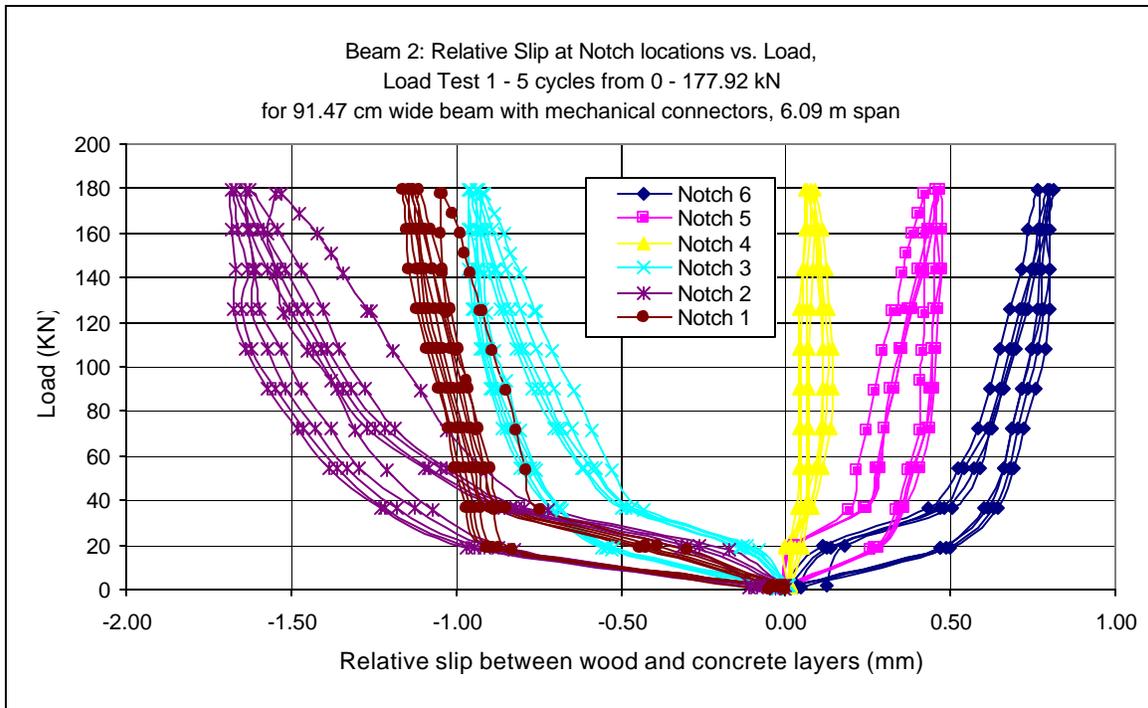
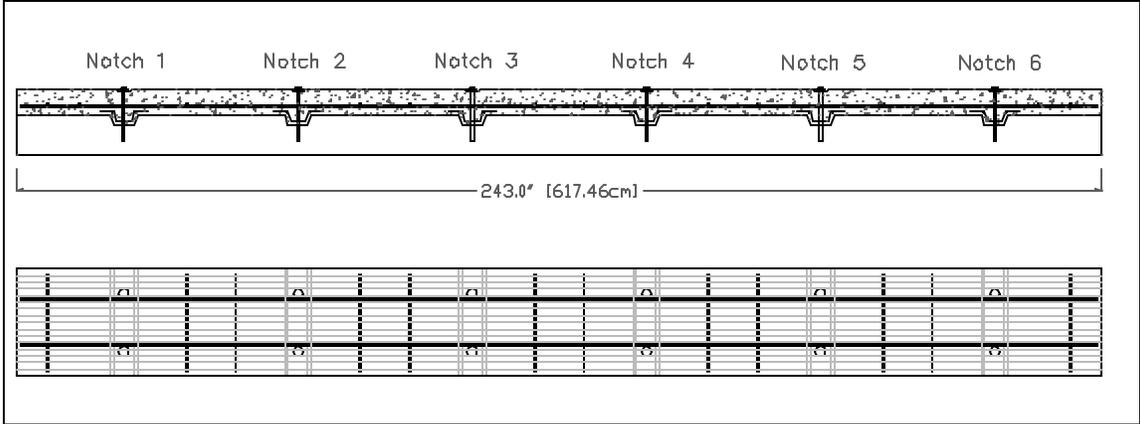
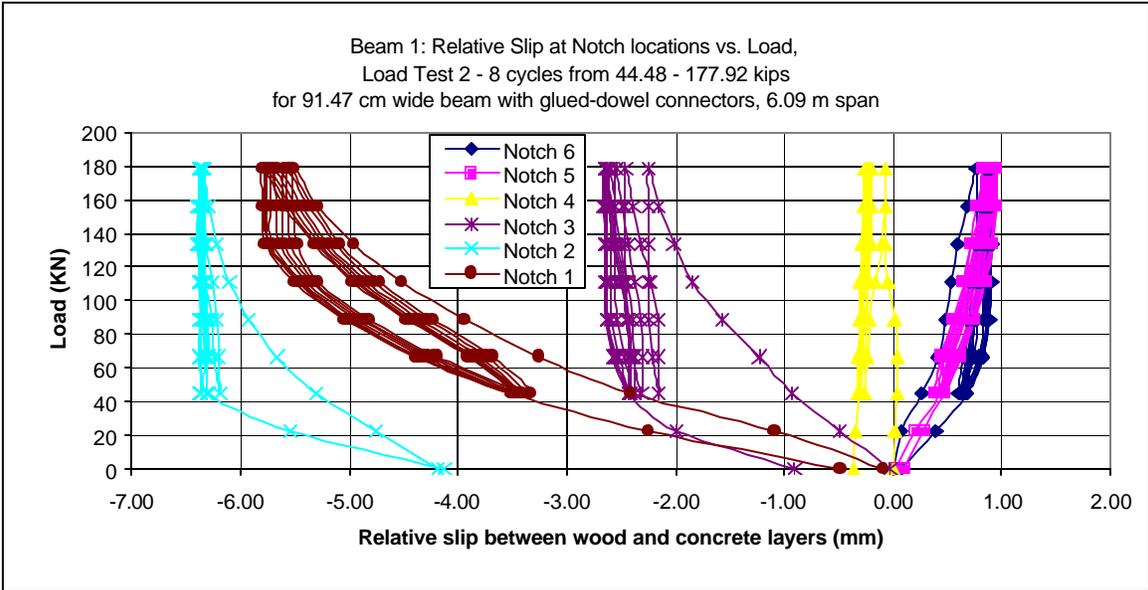


Figure 3.7 Load Test WB2.1 Load vs. Slip Response

In Figure 3.6 notch #1 of wide beam 1 undergoes a large relative slip between the wood and concrete layers, roughly four times the slip measured at the other five notches. If it is assumed that notches #1 and #6, closest to the supports, have identical forces in the notch, then Figure 3.6 suggests an ineffective or failed notch connection contributed to the behavior. However, Figure 3.6 does not plot notch force vs. slip, thus the presence of an ineffective notch is only speculation. As support for this hypothesis, there are a number of potential causes for the ineffective notch. One probable cause is that the dowel connector for notch #1 for load test WB1.1 was either not torqued sufficiently or was over-torqued, resulting in a large gap between the concrete layer and the wood layer at the notch. Figure 3.7 displays the load vs. relative interlayer slip results for WB2.1. The data suggests that wide beam 2 had one very inefficient notched connection as evident from the large relative slip (compared to the other notches) occurring at notch #2 in wide beam 2. Comparing Figures 3.6, 3.7, (and subsequently Fig. 3.9) it is seen that the gradually decreasing efficiency of wide beams 1 and 2 is likely predominantly due to gradually decreasing effectiveness of the notches #1 and #2 of these respective beams. However the gradually increasing slip at the ineffective notches does not explain the sudden and dramatic drop in composite efficiency observed for wide beams 1 and 2 in Figure 3.6.



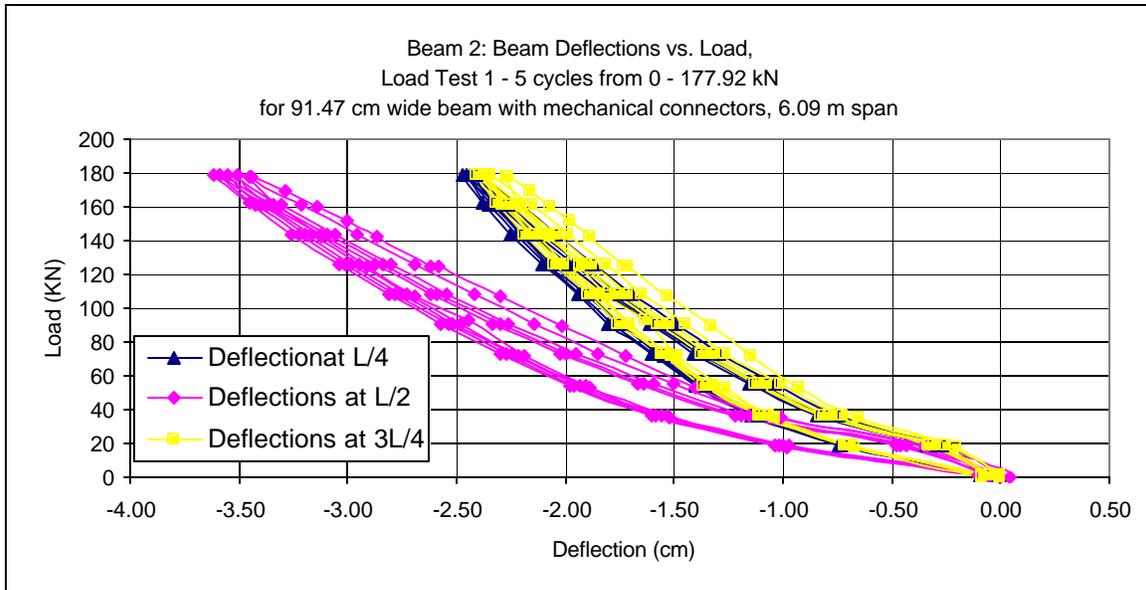
**Figure 3.8 Wide Beam Specimen Notch Numbering**



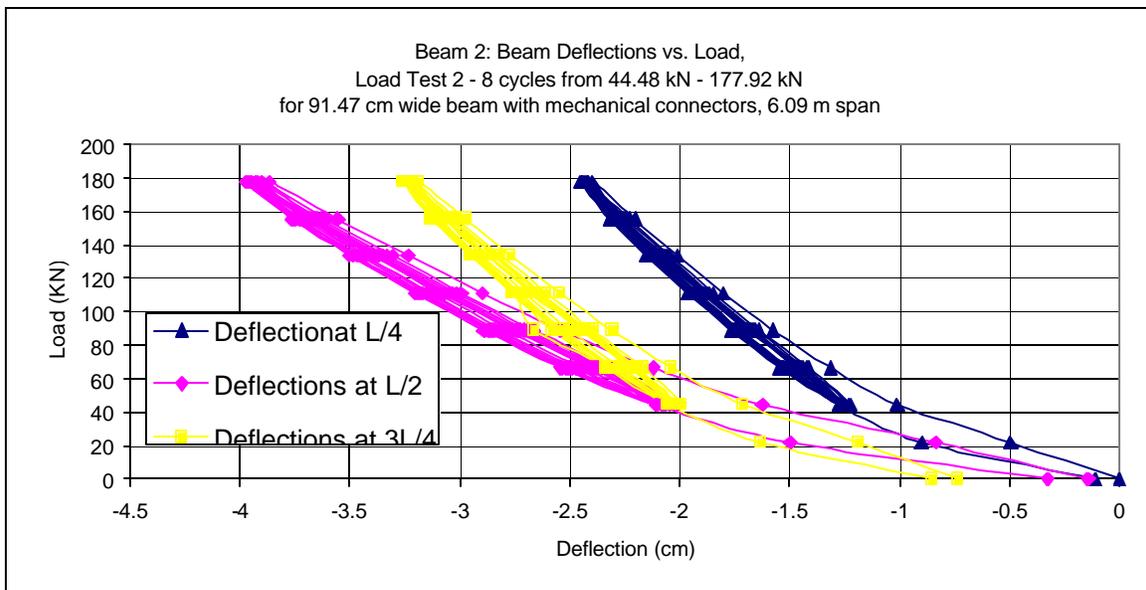
**Figure 3.9 Load Test WB1.2 Load vs. Deflection Response**

The large sudden drop in efficiency of wide beam 1 is likely from a connection failure. This is deduced from data in Figure 3.9 where notch #2 of wide beam 1 exhibits large interlayer slip behavior that was not displayed for the same notch in Figure 3.6. The onset of large slip at notch #2 from the beginning of load test 2 suggests that failure occurred sometime between Load Test WB1.1 and Load Test WB1.2. It is probable that the failure materialized, but was not recognized, during equipment re-calibration tests that were made in the time between load tests 1 and 2.

The sudden drop in efficiency of wide beam 2 appears to be due to an entirely different type of failure. The comparison of load vs. slip curves for load tests WB2.1 and WB2.2 do not suggest any failed notches. However, a comparison of the load vs. deflection plots for load tests WB2.1 and WB2.2 indicates that a possible flexural failure occurred in wide beam 2 sometime between the two different load tests. Specifically, no sudden drops in stiffness were observed during either load test WB2.1 or load test WB2.2 which suggests that no flexural failures occurred during the conduct of these load tests. Figures 3.10 and 3.11 show a large decrease in stiffness and a large increase in deflection at the  $\frac{3}{4}$  point location of the span of wide beam 2 in Load Test WB2.2 compared to load test WB2.1.

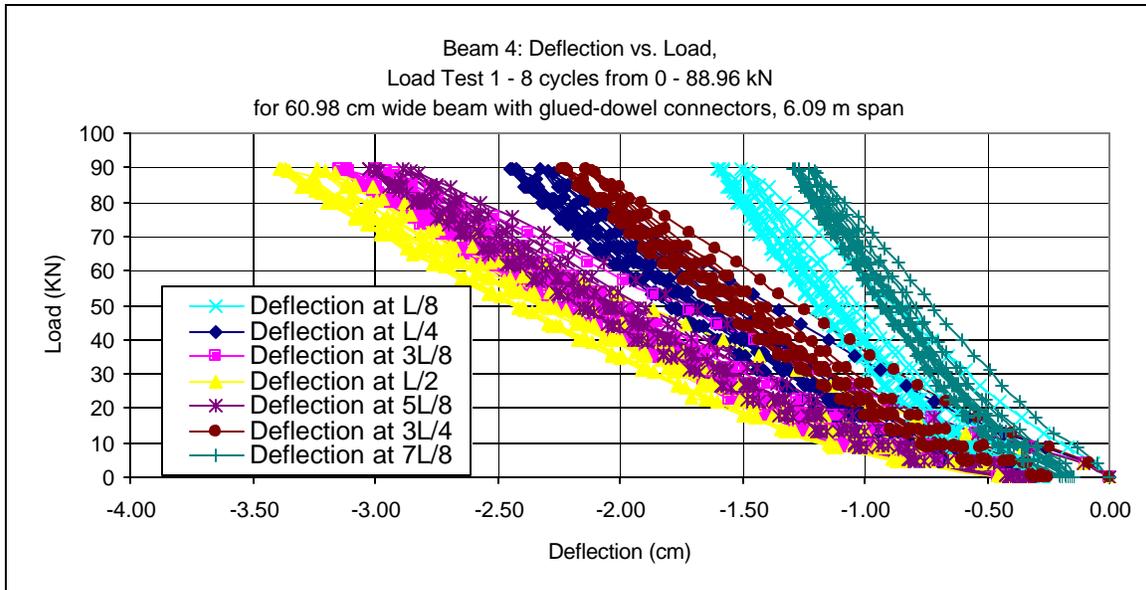


**Figure 3.10** Load Test WB2.1 Load vs. Deflection Response

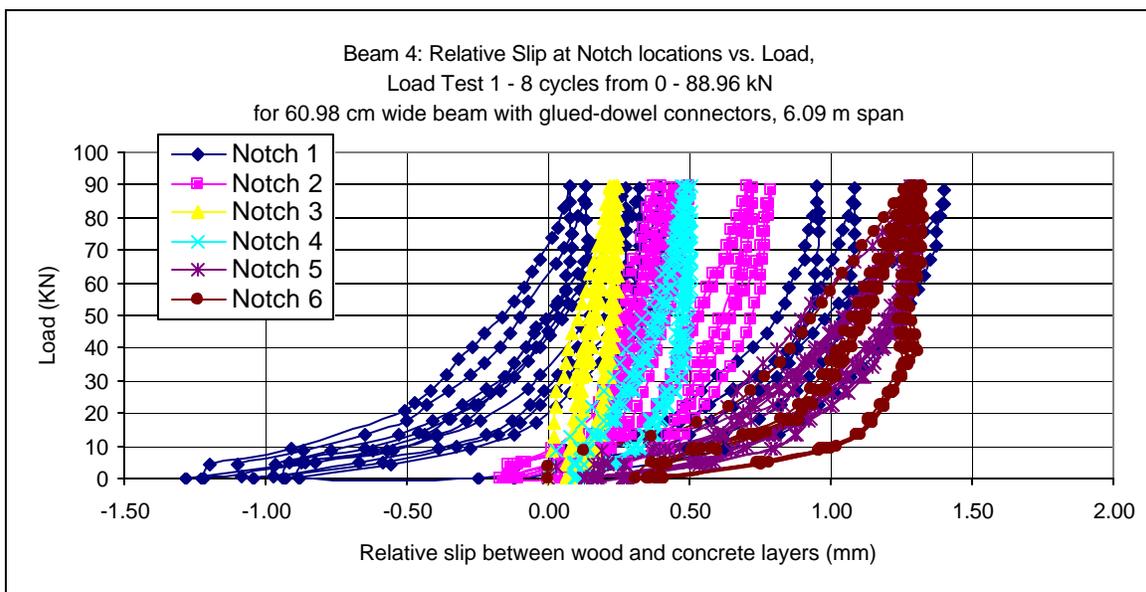


**Figure 3.11** Load Test WB2.2 Load vs. Deflection Response

Unlike the results for wide beams 1 and 2, the low composite efficiency results for wide beams 3 and 4 do not appear to be the result of prior failures. Table 3.3 shows that these beams experienced only a gradual decrease in composite efficiency as the number of load test repetitions increased. Further, wide beams 3 and 4 were found to have low composite efficiencies from the outset of their testing program. The load vs. deflection and load vs. slip responses of wide beams 3 and 4 changed very little in going from load test 1 to load test 2 for each of the two beams. Inspection of the load vs. deflection responses shows that both wide beam 3 and wide beam 4 had predominantly linear, elastic load-deflection responses with small relative slip measurements at the notch locations. Figures 3.12 and 3.13 are representative examples of these observations.



**Figure 3.12 Load Test WB4.1 Load vs. Deflection Response**



**Figure 3.13 Load Test WB4.1 Load vs. Relative Slip Response**

Figure 3.14 depicts the typical failure observed at a notch for wide beams 3 and 4. The failures of wide beam specimens 3 and 4 occurred at the notch locations, similar to deep beam specimen 2. However the mode of notch failure for the wide beam specimens differed from that observed for deep beam specimen 2. In the wide beams the concrete layer cracked across the top of the notch, parallel to the layer interface. Failure at a single notch occurred suddenly with subsequent progressive failures at other notches, followed by a flexural type failure near mid-span. It is notable that notch type failures occurred in wide beam specimens 3 and 4, which exhibited very little composite efficiency (11-25%). In general notch type failures require sufficient composite efficiency to generate the shear stresses needed to result in failure at the notch. That is flexural type failures are consistent with the expected failure of a fully non-composite beam (low composite efficiency beam), and notch type failures are consistent with the expected failure of a fully composite beam (high composite efficiency beam).



**Figure 3.14 Notch #4 Failure of Wide Beam Specimen #4**

One likely contributor to the failure response observed in beam 4 is the particular rebar arrangement for the notches of that beam. Unlike the other three beams, the bottoms of the notches of beam 4 were not reinforced with transverse rebar. Beam 4 also did not have wire mesh placed in the bottom of its notches. Figure 3.15 shows the reinforcement details for beams 4 and 3 pictured first and second from the left-hand side of the picture. Beam 4 is pictured on the far left-hand side of Figure 3.15; the absence of transverse rebar at the notch locations is apparent by comparing the picture of beam 4 with the picture of beam 3 just its right. The unusual failure characteristics of beams 3 and 4 are one supporting argument that further studies of the reinforcement details for wood-concrete composite beams (and decks) are needed.



**Figure 3.15 Rebar Placement for Beams 3 and 4**

Another possible cause for the failure response of wide beams 3 and 4 may be due to the stresses generated by slipping and creeping of the beams for 2 ½ years. It may be that the slip resistance lost during a long duration of static dead load is not completely recoverable via connector re-tightening. It should also be noted, as seen in Figure 3.14, that concrete in the notches of the wide beam specimens was not completely consolidated to fill the notch shown. The poor consolidation is surely detrimental to the performance of the notch, and this perhaps is the cause of the failure characteristics of wide beams 3 and 4. However, a creep analysis of wide beams 3 and 4 would be a pre-requisite to better explaining the link between their low composite efficiency behavior and notch type failure response.

## 4. OBSERVATIONS AND CONCLUSIONS

### 4.1 Observations

The beams tested in this study had a wide range of observed composite efficiency values. Much of this observed variation in efficiency was explained in terms of ineffective notch connections or material failures. For example, deep beam 1 had an average composite efficiency of 59 percent for four repeated load tests. However, in load test DB1.1 a much lower composite efficiency (42%) was observed due to the notch anchor connections not having been tightened before the loading was initiated. As another example, wide beam 2 had observed efficiencies of 47 percent and 34 percent, respectively, for load tests WB2.1 and WB2.2. However, WB2.2 was conducted after an undetected flexural failure in the wood layer of wide beam 2 in the prior loading.

Thus many of the composite efficiency results of this study were strongly influenced by some type of ineffective connection or material failure. Because the presence of ineffective connections and material failures are localized effects, albeit very important ones, they do not allow the overall performance of the wood-concrete beam system to be studied. Thus it may be better to assess the gross wood-concrete beam system performance based on composite efficiency results for only those beam specimens where ineffective connections and material failures were not observed.

Table 4.1 lists the adjusted average composite efficiency results for all of the load tests. The adjusted average composite efficiencies were computed after excluding all of the load test results where known or suspected connection or material failures occurred. Note that the results of wide beams 3 and 4 were not adjusted because no ineffective connector or material failure effects were observed for those tests. The load tests excluded from the adjusted average composite efficiency computation were omitted for the following reasons; DB1.1 had an ineffective (not tightened) notch; DB1.5 was a failure load test; DB1.6 was a post failure load test; DB2.3 was a failure load test; DB2.4 was a post failure load test; WB1.2 had an ineffective (failed) notch; and WB2.2 had a flexural failure.

**Table 4.1 Adjusted Average Composite Efficiency Results**

<b>Beam</b>	<b>Tests used to compute the adjusted average composite efficiency</b>	<b>Tests excluded from adjusted average composite efficiency computation</b>	<b>Adjusted Average composite efficiency</b>
Deep Beam 1	DB1.2, DB1.3, DB1.4	DB1.1, DB1.5, DB1.6	64.99%
Deep Beam 2	DB2.1, DB2.2	DB2.3, DB2.4	83.55%
Wide Beam 1	WB1.1	WB1.2	43.92%
Wide Beam 2	WB2.1	WB2.2	46.86%
Wide Beam 3	WB3.1, WB3.2	none	23.63%
Wide Beam 4	WB4.1, WB4.2	none	13.43%

The composite efficiency results in Table 4.1, although not appropriate for a statistical evaluation, show that the Deep Beam specimens had higher average composite efficiency than the Wide Beam specimens. There are many potential explanations for this observation as discussed in earlier sections. Another possible explanation is that the Wide Beams were significantly more difficult to construct than the Deep Beams. As a result the Wide Beams had many more construction flaws (e.g. poor consolidation of the notch) than the Deep Beams. Thus the Deep Beam and Wide Beam results in Table 4.1 may not be directly comparable considering that there was a large difference in the quality of the two beam types. Table 4.2 compares the geometries of the various specimens and other observed efficiencies.

**Table 4.2 Compilation of Specimen Geometries and Test Results**

<b>Specimen</b>	<b>Span (ft)</b>	<b>Width (in)</b>	<b>Total Depth (in)</b>	<b>Type of Connector</b>	<b>Initial Creep Time (months)</b>	<b>Average Composite Efficiency (%)</b>
Deep Beam 1	12	7.25	33.3	Glue	12	64.99
Deep Beam 2	12	7.25	33.3	Mechanical	12	83.55
Wide Beam 1	20	96.48	34.31	Glue	2	43.92
Wide Beam 2	20	96.48	34.31	Mechanical	2	46.86
Wide Beam 3	20	60.98	34.31	Glue	27	23.63
Wide Beam 4	20	60.98	34.31	Mechanical	28	13.43

As with the deep beam specimens, creep is an issue for wide beam specimens. Wide beam 1 and wide beam 2 (width = 96.48”) were in place and subject to creep for only about two months from the time of casting to the time of load testing. Connections in wide beam 1 were also not retightened before the first load test. Wide beam 3 and wide beam 4 (width = 60.98”) were in place and subject to creep for about 27 months and 28 months, respectively, from the time of casting to the time of loading. Despite retightening the connections before load testing, the long term creep is likely the dominant factor in the much lower composite efficiencies for wide beam 3 and wide beam 4 compared to wide beam 1 and wide beam 2. The difference in width may have been a factor, but there is no data basis to distinguish the effect of that variable. An additional factor in the results is the wide beam had a longer span (20’) than the deep beam (12’).

Specimens wide beam 1 and wide beam 2 had the same geometries and initial creep time. Thus the type of connector (glue vs. mechanical) was the only difference, and the specimens had similar efficiencies. Specimens wide beam 3 and wide beam 4 had the same geometries but were about two thirds the width of wide beam 1 and 2. They were also subject to much longer creep (27-28 months vs. 2 months). The efficiency of wide beams 3 and 4 were much lower than for wide beams 1 and 2, attributed in large part to the much longer creep time. However, the mechanically connected wide beam 4 had about 60 percent of the efficiency of glue connected wide beam 3. So the connector quality may have been a secondary contributor.

## 4.2 Conclusions

A very small sample of beams of different geometries was involved in the pilot test program, so statistical assessment is not meaningful. It is evident that creep of the specimens (all un-shored, except for the wood layer itself holding the concrete layer) was a significant phenomenon. For bridge applications this suggests shoring may be needed unless significant camber can be incorporated to offset the effect. In residential and commercial application, lightweight concrete could be considered.

Several loadings were involved, and when questionable or compromised loadings are eliminated the findings are not significantly altered from those of the overall set. The deep beam results suggest that the 1-year creep time may not have been a significant factor as reasonably high efficiencies resulted. The connections in these beams were retightened before loading, which may have contributed to the outcome. But without a test of specimens with no creep exposure, a definitive degree of effect of creep is not distinguishable. They may have had even higher efficiencies if creep had been prevented. Thus, high efficiencies are possible as evidenced by these beams even with one year of creep. The wide beams had definite creep effects compared to the deep beams. Because the wide beams were exposed to creep more than twice as long as the deep beams, the length of time was apparently a major factor. No definitive observation is possible for the comparison of glued connectors vs. mechanical connectors, as no consistent difference in efficiency was evident.

The results of this study show the complex behavior of composite wood-concrete beams. Composite efficiency is affected by many factors such as concrete consolidation in the notch, knots or other defects in the timber layer, the effectiveness of the notch anchor, and many others. Also, creep effects may have a strong effect on the composite efficiency of large wood-concrete composites. Thus it is recommended that the influence of all of these different factors on composite efficiency be studied in depth with a large number of specimens so as to allow parameter studies.



## 5. APPENDIX

### APPENDIX A: Compilation of Test Results

Wide Beam 1 - Glued Dowel Connection								
Midspan Deflection Results - Load Test 1								
Repetition	Max. D (cm)	Load (kN)	L / D	D <sub>NC</sub> (cm)	D <sub>FC</sub> (cm)	Composite Action Available	Composite Efficiency	Composite Action Observed
1	3.455	177.9	176	5.25	1.32	74.93%	45.71%	34.25%
2	3.55	177.9	172	5.25	1.32	74.93%	43.20%	32.37%
3	3.53	177.9	173	5.25	1.32	74.93%	43.76%	32.79%
4	3.54	177.9	172	5.25	1.32	74.93%	43.60%	32.67%
5	3.55	177.9	172	5.25	1.32	74.93%	43.35%	32.48%
<b>Average</b>	3.53	177.9	172.9	5.25	1.32	74.93%	43.92%	32.91%

Wide Beam 1 - Glued Dowel Connection								
Midspan Deflection Results - Load Test 2								
Repetition	Max. D (cm)	Load (kN)	L / D	D <sub>NC</sub> (cm)	D <sub>FC</sub> (cm)	Composite Action Available	Composite Efficiency	Composite Action Observed
1	4.30	177.9	142	5.25	1.32	74.93%	24.17%	18.11%
2	4.34	177.9	140	5.25	1.32	74.93%	23.24%	17.41%
3	4.36	177.9	140	5.25	1.32	74.93%	22.63%	16.95%
4	4.43	177.9	138	5.25	1.32	74.93%	20.90%	15.66%
5	4.46	177.9	137	5.25	1.32	74.93%	20.21%	15.15%
6	4.48	177.9	136	5.25	1.32	74.93%	19.58%	14.67%
7	4.50	177.9	135	5.25	1.32	74.93%	19.15%	14.35%
8	4.52	177.9	135	5.25	1.32	74.93%	18.66%	13.98%
<b>Average</b>	4.43	177.9	137.8	5.25	1.32	74.93%	21.07%	15.79%

Wide Beam 2 - Mechanical								
Midspan Deflection Results - Load Test 1								
Repetition	Max. D (cm)	Load (kN)	L / D	D <sub>NC</sub> (cm)	D <sub>FC</sub> (cm)	Composite Action Available	Composite Efficiency	Composite Action Observed
1	3.26	177.9	187	5.25	1.32	74.93%	50.66%	37.96%
2	3.31	177.9	184	5.25	1.32	74.93%	49.32%	36.95%
3	3.50	177.9	174	5.25	1.32	74.93%	44.54%	33.38%
4	3.57	177.9	171	5.25	1.32	74.93%	42.92%	32.16%
<b>Average</b>	3.41	177.9	179	5.25	1.32	74.93%	46.86%	35.11%

<b>Wide Beam 2 - Mechanical</b>								
<b>Midspan Deflection Results - Load Test 2</b>								
Repetition	Max. D (cm)	Load (kN)	L / D	D <sub>NC</sub> (cm)	D <sub>FC</sub> (cm)	Composite Action Available	Composite Efficiency	Composite Action Observed
1	3.87	177.9	158	5.25	1.32	74.93%	35.27%	26.43%
2	3.90	177.9	156	5.25	1.32	74.93%	34.44%	25.80%
3	3.92	177.9	156	5.25	1.32	74.93%	33.95%	25.44%
4	3.93	177.9	155	5.25	1.32	74.93%	33.57%	25.16%
5	3.95	177.9	154	5.25	1.32	74.93%	33.22%	24.89%
6	3.96	177.9	154	5.25	1.32	74.93%	32.99%	24.72%
7	3.96	177.9	154	5.25	1.32	74.93%	32.78%	24.57%
8	3.97	177.9	153	5.25	1.32	74.93%	32.56%	24.39%
<b>Average</b>	3.93	177.9	155	5.25	1.32	74.93%	33.60%	25.17%

<b>Wide Beam 3 - Mechanical</b>								
<b>Midspan Deflection Results - Load Test 1</b>								
Repetition	Max. D (cm)	Load (kN)	L / D	D <sub>NC</sub> (cm)	D <sub>FC</sub> (cm)	Composite Action Available	Composite Efficiency	Composite Action Observed
1	3.462	105.91	176	4.42	1.11	74.98%	28.95%	21.71%
2	3.01	89.18	202	3.72	0.93	74.98%	25.49%	19.11%
3	3.05	89.57	200	3.74	0.94	74.98%	24.71%	18.53%
4	3.07	89.51	199	3.74	0.94	74.98%	24.00%	17.99%
5	3.07	89.73	198	3.75	0.94	74.98%	23.97%	17.97%
6	3.09	89.97	197	3.76	0.94	74.98%	23.67%	17.75%
7	3.07	89.11	198	3.72	0.93	74.98%	23.17%	17.37%
8	3.09	89.44	197	3.73	0.93	74.98%	23.05%	17.28%
<b>Average</b>	3.08	89.55	198	3.74	0.96	74.98%	24.63%	18.46%

<b>Wide Beam 3 - Mechanical</b>								
<b>Midspan Deflection Results - Load Test 2</b>								
Repetition	Max. D (cm)	Load (kN)	L / D	D <sub>NC</sub> (cm)	D <sub>FC</sub> (cm)	Composite Action Available	Composite Efficiency	Composite Action Observed
1	3.08	89.53	198	3.74	0.94	74.98%	23.34%	17.50%
2	3.08	89.70	198	3.75	0.94	74.98%	23.54%	17.65%
3	3.08	88.96	198	3.71	0.93	74.98%	22.88%	17.16%
4	3.11	89.61	196	3.74	0.94	74.98%	22.66%	16.99%
5	3.08	88.64	198	3.70	0.93	74.98%	22.45%	16.83%
6	3.10	88.95	197	3.71	0.93	74.98%	22.12%	16.59%
7	3.10	89.05	197	3.72	0.93	74.98%	22.09%	16.57%
8	3.10	88.91	197	3.71	0.93	74.98%	21.96%	16.47%
<b>Average</b>	3.10	89.03	197	3.72	0.93	74.98%	22.63%	16.97%

Repetition	Max. D (cm)	Load (kN)	L / D	D <sub>NC</sub> (cm)	D <sub>FC</sub> (cm)	Composite Action Available	Composite Efficiency	Composite Action Observed
1	3.17	88.63	193	3.70	0.93	74.98%	19.30%	14.47%
2	3.20	88.81	191	3.71	0.93	74.98%	18.27%	13.70%
3	3.24	89.48	188	3.74	0.93	74.98%	17.78%	13.33%
4	3.36	89.72	181	3.75	0.94	74.98%	13.67%	10.25%
5	3.35	89.02	182	3.72	0.93	74.98%	13.06%	9.79%
6	3.36	88.81	182	3.71	0.93	74.98%	12.70%	9.52%
7	3.38	89.84	180	3.75	0.94	74.98%	13.05%	9.78%
8	3.39	89.64	180	3.74	0.94	74.98%	12.53%	9.39%
<b>Average</b>	3.37	89.41	181	3.73	0.93	74.98%	15.05%	11.28%

<b>Wide Beam 4 - Glued Dowel Connection</b>								
<b>Midspan Deflection Results - Load Test 2</b>								
Repetition	Max. D (cm)	Load (kN)	L / D	D <sub>NC</sub> (cm)	D <sub>FC</sub> (cm)	Composite Action Available	Composite Efficiency	Composite Action Observed
1	3.39	89.58	180	3.74	0.94	74.98%	12.56%	9.42%
2	3.51	94.05	174	3.93	0.98	74.98%	14.13%	10.59%
3	3.42	89.75	178	3.75	0.94	74.98%	11.68%	8.76%
4	3.40	89.01	179	3.72	0.93	74.98%	11.25%	8.44%
5	3.40	89.02	180	3.72	0.93	74.98%	11.54%	8.66%
6	3.42	89.67	178	3.74	0.94	74.98%	11.54%	8.65%
7	3.41	89.03	179	3.72	0.93	74.98%	10.91%	8.18%
8	3.42	89.17	178	3.72	0.93	74.98%	10.89%	8.16%
<b>Average</b>	3.41	89.18	179	3.72	0.94	74.98%	11.81%	8.86%

<b>Deep Beam 1 - Glued Dowel Connection</b>								
<b>Midspan Deflection Results - Load Test 1</b>								
Repetition	Max. D (cm)	Load (kN)	L / D	D <sub>NC</sub> (cm)	D <sub>FC</sub> (cm)	Composite Action Available	Composite Efficiency	Composite Action Observed
1	1.20	35.6	304	1.77	0.46	73.76%	43.30%	31.94%
2	1.20	34.6	304	1.72	0.45	73.76%	40.85%	30.13%
3	1.24	35.9	296	1.79	0.47	73.76%	41.58%	30.67%
4	1.24	35.6	295	1.77	0.47	73.76%	40.78%	30.08%
5	1.28	37.6	286	1.87	0.49	73.76%	42.81%	31.58%
<b>Average</b>	1.23					73.76%	41.86%	30.88%

Deep Beam 1 - Glued Dowel Connection								
Midspan Deflection Results - Load Test 2								
1	2	3	4	5	6	7	8	9
Repetition	Max. D (cm)	Load (kN)	L / D	D <sub>NC</sub> (cm)	D <sub>FC</sub> (cm)	Composite Action Available	Composite Efficiency	Composite Action Observed
1	0.90	34.9	405	1.74	0.46	73.76%	65.08%	48.00%
2	0.93	35.7	395	1.78	0.47	73.76%	64.79%	47.79%
3	0.94	36.6	388	1.82	0.48	73.76%	65.36%	48.21%
4	0.94	36.4	389	1.81	0.48	73.76%	65.23%	48.11%
5	0.94	36.2	390	1.80	0.47	73.76%	65.04%	47.97%
6	0.94	36.3	389	1.81	0.47	73.76%	65.00%	47.94%
7	0.95	36.6	386	1.82	0.48	73.76%	65.08%	48.00%
8	0.95	36.5	387	1.82	0.48	73.76%	64.91%	47.88%
9	0.95	36.4	387	1.81	0.47	73.76%	64.65%	47.69%
10	0.94	36.0	389	1.79	0.47	73.76%	64.33%	47.45%
<b>Average</b>	0.94					73.76%	64.95%	47.90%

Deep Beam 1 - Glued Dowel Connection								
Midspan Deflection Results - Load Test 3								
Repetition	Max. D (cm)	Load (kN)	L / D	D <sub>NC</sub> (cm)	D <sub>FC</sub> (cm)	Composite Action Available	Composite Efficiency	Composite Action Observed
1	0.91	36.7	402	1.82	0.48	73.76%	67.90%	50.08%
2	0.92	36.7	398	1.83	0.48	73.76%	67.37%	49.69%
3	0.93	36.9	395	1.83	0.48	73.76%	67.00%	49.42%
4	0.96	37.7	382	1.87	0.49	73.76%	66.25%	48.86%
5	0.96	37.3	382	1.85	0.49	73.76%	65.56%	48.35%
6	0.96	37.1	383	1.84	0.48	73.76%	65.30%	48.16%
7	0.96	37.0	382	1.84	0.48	73.76%	64.93%	47.89%
8	0.96	37.1	380	1.85	0.48	73.76%	64.83%	47.82%
9	0.96	36.8	380	1.83	0.48	73.76%	64.24%	47.38%
10	0.97	37.3	376	1.85	0.49	73.76%	64.49%	47.57%
<b>Average</b>	0.95					73.76%	65.79%	48.52%

<b>Deep Beam 1 - Glued Dowel Connection</b>								
<b>Midspan Deflection Results - Load Test 4</b>								
<b>Repetition</b>	<b>Max. D (cm)</b>	<b>Load (kN)</b>	<b>L / D</b>	<b>D<sub>NC</sub> (cm)</b>	<b>D<sub>FC</sub> (cm)</b>	<b>Composite Action Available</b>	<b>Composite Efficiency</b>	<b>Composite Action Observed</b>
1	0.90	34.5	406	1.72	0.45	73.76%	64.35%	47.46%
2	0.91	34.8	403	1.73	0.45	73.76%	64.47%	47.55%
3	0.92	35.0	399	1.74	0.46	73.76%	64.13%	47.30%
4	0.92	34.9	399	1.74	0.46	73.76%	64.00%	47.20%
5	0.93	35.6	393	1.77	0.46	73.76%	64.27%	47.40%
<b>Average</b>	0.91					73.76%	64.24%	47.38%

<b>Deep Beam 1 - Glued Dowel Connection</b>								
<b>Midspan Deflection Results - Load Test 5 and 6</b>								
<b>Load Test</b>	<b>Max. D (cm)</b>	<b>Load (kN)</b>	<b>L / D</b>	<b>D<sub>NC</sub> (cm)</b>	<b>D<sub>FC</sub> (cm)</b>	<b>Composite Action Available</b>	<b>Composite Efficiency</b>	<b>Composite Action Observed</b>
5	1.72	61.6	213	3.06	0.80	73.76%	59.36%	43.78%
6	2.67	81.2	137	4.04	1.06	73.76%	45.96%	33.90%
<b>Average</b>	2.20					73.76%	52.66%	38.84%

<b>Deep Beam 2 - Mechanical Connection</b>								
<b>Midspan Deflection Results - Load Test 1</b>								
<b>Repetition</b>	<b>Max. D (cm)</b>	<b>Load (kN)</b>	<b>L / D</b>	<b>D<sub>NC</sub> (cm)</b>	<b>D<sub>FC</sub> (cm)</b>	<b>Composite Action Available</b>	<b>Composite Efficiency</b>	<b>Composite Action Observed</b>
1	0.71	37.3	516	1.86	0.49	73.76%	83.78%	61.80%
2	0.69	35.9	527	1.78	0.47	73.76%	82.86%	61.11%
3	0.69	35.7	527	1.78	0.47	73.76%	82.59%	60.92%
4	0.69	35.7	527	1.77	0.47	73.76%	82.56%	60.89%
5	0.70	35.7	526	1.78	0.47	73.76%	82.55%	60.89%
<b>Average</b>	0.70					73.76%	82.87%	61.12%

Deep Beam 2 - Mechanical Connection								
Midspan Deflection Results - Load Test 2								
Repetition	Max. D (cm)	Load (kN)	L / D	D <sub>NC</sub> (cm)	D <sub>FC</sub> (cm)	Composite Action Available	Composite Efficiency	Composite Action Observed
1	0.66	35.6	554	1.77	0.47	73.76%	85.03%	62.71%
2	0.67	36.0	544	1.79	0.47	73.76%	84.65%	62.44%
3	0.66	35.5	551	1.76	0.46	73.76%	84.57%	62.38%
4	0.67	35.8	545	1.78	0.47	73.76%	84.41%	62.26%
5	0.67	35.4	547	1.76	0.46	73.76%	84.11%	62.04%
6	0.67	35.5	547	1.76	0.46	73.76%	84.14%	62.06%
7	0.67	35.5	545	1.77	0.46	73.76%	84.06%	62.00%
8	0.69	35.8	527	1.78	0.47	73.76%	82.76%	61.05%
<b>Average</b>	0.67					73.76%	84.22%	62.12%

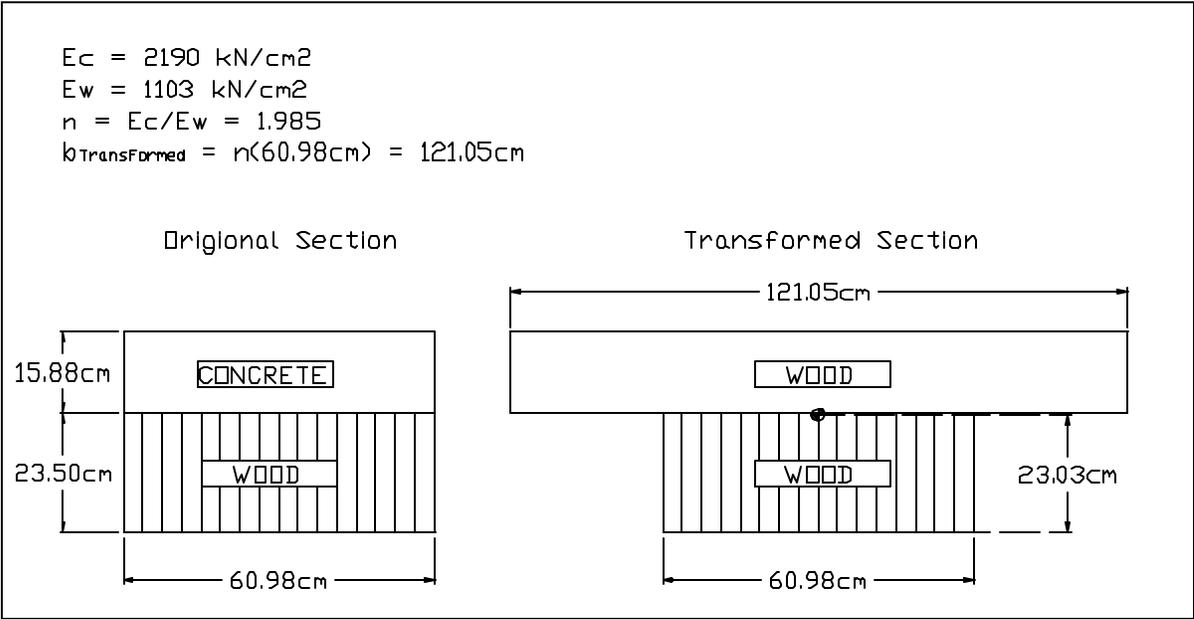
Deep Beam 2 - Mechanical Connection								
Midspan Deflection Results - Load Test 3 and 4								
Load Test	Max. D (cm)	Load (kN)	L / D	D <sub>NC</sub> (cm)	D <sub>FC</sub> (cm)	Composite Action Available	Composite Efficiency	Composite Action Observed
3	2.12	87.7	173	4.36	1.14	73.76%	69.71%	51.41%
4	4.89	81.0	75	4.03	1.06	73.76%	-28.97%	-21.37%
<b>Average</b>	3.50					73.76%	20.37%	15.02%

# APPENDIX B: Example Composite Efficiency Calculation

$\Delta = 3.165 \text{ cm}$

$P = 88.626 \text{ cm}$

Where, for example  $\Delta$  and  $P$  are the experimentally measured mid-span deflection and applied load for the first repetition of Load Test WB4.1. To compute the composite efficiency, the theoretical fully composite ( $\Delta_{FC}$ ) and fully non-composite ( $\Delta_{NC}$ ) deflections must be known. To compute the theoretical fully composite deflection, the method of transformed sections is used. The flexural rigidity  $EI$  of the transformed section is calculated by taking  $E_{TRANSFORMED}$  equal to  $E_{WOOD}$  and computing the transformed moment of inertia using the parallel axis theorem. The dimensions of the transformed section are shown in Figure B.1.



**Figure B.1 – Transformed Section Properties**

Using the dimensions of the transformed section the transformed moment of inertia is calculated as follows. Where the y-coordinate of the centroid (23.03 cm) for the transformed section was shown in Figure B.1.

$$I_{WOOD} = \frac{1}{12} (60.98cm)(23.03cm)^3 = 40396cm^4$$

$$I_{CONCRETE} = \frac{1}{12} (121.05cm)(15.88cm)^3 = 65949cm^4$$

$$I_{TRANSFORMED} = I_{WOOD} + A_W d_W^2 + I_{CONCRETE} + A_C d_C^2$$

$$I_{TRANSFORMED} = 40396cm^4 + (60.98cm)(23.50cm)(23.03cm - 23.50cm / 2)^2 + 65949cm^4$$

$$+ (121.05cm)(15.88cm)((23.50cm + 15.88cm / 2) - 23.03cm)^2$$

The results are;  $I_{TRANSFORMED} = 424640 \text{ cm}^4$ ,  $EI_{TRANSFORMED} = 4.684E+08 \text{ kN-cm}^2$ . Thus for  $P = 88.626 \text{ kN}$ , the theoretical fully composite mid-span deflection for a simply supported beam loaded at mid-span is:

$$\Delta_{FC} = \frac{Pl^3}{48EI} = \frac{(88.626kN)(617cm)^3}{48(4.684E + 08kN - cm^2)} = 0.93cm$$

The theoretical fully non-composite midspan deflection was computed using a flexural rigidity equal to the sum of the flexural rigidities of the two separate layers. This gives a  $EI_{NC} = 1.172E+08 \text{ kN-cm}^2$ .

$$EI_{NC} = EI_W + EI_C$$

$$EI_{NC} = (1102kN / cm^2)(65949 cm^4) + (2188kN / cm^2)(20350 cm^4)$$

$$EI_{NC} = 1.172E + 08kN - cm^2$$

The theoretical fully non-composite mid-span deflection for a simply supported beam loaded at mid-span is:

$$\Delta_{FC} = \frac{Pl^3}{48EI} = \frac{(88.626kN)(617cm)^3}{48(1.172E+08kN-cm^2)} = 3.70cm$$

Now, having computed the theoretical fully composite and fully non-composite mid-span deflections, the composite efficiency can be computed. The measured deflection is  $\Delta = 3.165$  cm. The composite efficiency is calculated as:

$$\text{Composite Efficiency} = \frac{\Delta_{NC} - \Delta}{\Delta_{NC} - \Delta_{FC}} = \frac{3.70cm - 3.165cm}{3.70cm - 0.93cm} = 19.31\%$$

It should be noted that the effects of shear deformation on the theoretical fully composite and fully non-composite deflections were neglected. However, the effects of shear deformation are included in the experimentally measured values. Thus the computed composite efficiency values are lower than what they would be if shear deformation effects were included in the theoretical fully composite and fully non-composite deflection computations.

Composite efficiency values are strongly affected by the definitions used to compute the theoretical fully composite and fully non-composite deflections. Because the layered wood-concrete beam specimens studied at CSU are connected only at discrete points along their span the theoretical fully composite stiffness value computed from the transformed section method can never be realized. As a result of the difference between the actual and the assumed (transformed section) theoretical fully composite deflection, the computed composite efficiency values are lower than what they would be for a more accurate definition of a fully composite beam.