Experimental Testing of Composite Wood Beams for Use in Timber Bridges

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The use of new high-performance materials can play an important role in the search for ways to rebuild and rehabilitate the nation's deteriorating bridges. Innovations in the area of engineered wood products provide new possibilities for the design of timber bridges. Bonding high-strength fiber-reinforced plastic (FRP) composite plates to the tension face of wood beams can improve stiffnesses and strengths. By further incorporating a concrete compression flange, an even more versatile and efficient structural member can be created. The use of concrete-wood-FRP composite beams for timber bridges is investigated. The criteria involved in designing timber beams for bridges are presented. Then, details and results of an experimental study aimed at addressing design-related issues for concrete-wood-FRP composite beams are discussed. In the study, a series of wood, wood-FRP, concrete-wood, and concrete-wood-FRP composite beams were tested. The wood used was a laminated veneer lumber; the reinforcement consisted of carbon FRP plates. Composite action between the concrete flange and the wood web, bond strength between the FRP plates and the wood, and stiffness and ultimate strength properties of the composite beams were evaluated. Results of the tests indicate that composite wood beams show promise for use in timber bridges.

The rebuilding of the infrastructure is a major challenge facing the nation today. The U.S. Department of Transportation's 10th Biennial Report to Congress on *The Status of the Nation's Highways and Bridges: Conditions and Performance*, concluded that approximately 16 percent of Interstate bridges and 42 percent of all bridges are in need of repair (*1*). The cost of replacing all of the nation's deficient bridges is prohibitive. With limited financial resources, the current technology will not solve the problem. "High-tech solutions must be investigated" (*2*) so that innovative new technologies and materials can be used to rebuild the nation's infrastructure.

Both engineered wood and advanced composite materials offer intriguing opportunities in the area of infrastructure rehabilitation. Engineered wood products offer better structural properties than solid sawn lumber, and they are a renewable resource. Furthermore, some of these products can be made using species of trees not usually considered for structural lumber. Engineered wood products are already being used in modern structures. Composite materials, on the other hand, are relatively new to the construction industry. They have many beneficial characteristics, such as a high strength-to-weight ratio and corrosion resistance, but research still needs to be conducted before they can be safely and reliably used. The work reported herein involves an attempt to improve the performance of engineered wood by using it in combination with both advanced composite materials and concrete.

BACKGROUND

Recent advances in the production of engineered wood products have caused an increased interest in research on their use. Recent papers by Ritter (*3*) and Wipf et al. (*4*) have cited a growing interest in timber bridges. Wipf et al. state that the use of locally available timber "can stimulate local economies and enhance rural transportation systems." They go on to say that "for this to occur, additional research is needed to further develop timber as a material for transportation structures." Ritter points out that whereas most older bridges were made of timber, only 10 percent of current bridges are. He states that "one of the primary reasons for the decline in timber bridges has been a lack of research and development to advance timber bridge technology to meet modern needs." Most of the structurally deficient bridges are on secondary and rural roads, where spans are short (*5*). In a recent article, Brungraber et al. (*6*) describe how timber can play an important role in the replacement and rehabilitation of bridges in rural America. Similarly, Gutkowski and McCutcheon (*7*) and Behr et al. (*8*) have described how timber can be a cost-effective solution for short-span bridges. New and innovative designs of highway bridges using wood present additional solutions in the repair and replacement of these bridges (*5*). One application being tested is the use of stress-laminated bridge systems, as described by Oliva and Dimakis (*9*) and Moody et al. (*10*). These systems include the use of stress-laminated slab decks, parallel-chord truss systems, and T- and box sections. In addition to novel design concepts, newly created engineered wood products offer even more possibilities (*11*). The new types of engineered wood include laminated veneer lumber (LVL), parallel-strand lumber, and laminated-strand lumber. These products are manufactured by bonding together wood veneers or strands of Douglas fir, southern pine, and other species under heat and pressure using thermal-setting resins such as phenol-formaldehyde. The process produces a piece of wood typically 38 mm thick, 0.6 or 1.2 m wide, and up to 25 m long (*11*). The use of engineered wood to date has been limited in bridge applications, but continued research, along with the recently written timber bridge manual (*12*), will help to change this trend.

The use of fiber-reinforced plastic (FRP) composite materials in new structures, and in the rehabilitation of deteriorating ones, also shows great promise. Composite materials have the beneficial characteristics of being noncorrosive and generally resistant to chemicals, having a high strength-to-weight ratio, and being nonmagnetic and nonconductive. Several researchers have recently discussed possible applications of composite materials for civil structures (*13–24*).

Of particular relevance to this paper is past work on the bonding of composite plates to the tension face of wood beams (*25–28*). Triantafillou et al. (*25,26*) have shown experimentally that both nonprestressed and prestressed composite plates bonded to the tension face of plain wood beams significantly increases their stiffness and strength. A carbon fiber–reinforced plastic (CFRP) plate having an area of 1 percent of the wood member led to a 60 percent increase in stiffness. Triantafillou et al. also presented an analytical method for computing the maximum safe prestress that can be used. In the work by Davalos et al. (*27,28*), glass fiber–reinforced plastic plates were bonded to glulam beams and tested. The reinforced beams displayed higher strengths along with a 30 percent increase in stiffness compared with unreinforced beams. In fact, composite reinforced glulam beams were used in the recently constructed Taylor Lake pedestrian bridge in The Dalles, Oregon (*29*). Comparison of the design to one involving standard materials showed the composite glulams to be an economical solution.

In addition to the work on FRP-wood beams, a few studies have involved nonengineered wood-concrete beams. Richart and Williams (*30*) and McCullough (*31*) concluded that the concrete would provide an excellent wearing surface (road deck) and that smaller-dimensioned lumber beams could be used. In the tests performed, the composite beams exhibited satisfactory behavior, with some slippage occurring along the wood-concrete interface. A variety of shear connectors were studied, and triangular plate units were found to work best. Neither shrinkage nor repeated loads proved to be a problem. Later work by Pincus (*32,33*) involved the study of T-beams made of concrete flanges and wood webs. The effectiveness of both mechanical shear connectors and epoxy adhesives was studied and found to be sufficient to provide full composite action. Finally, recent work by Ahmadi and Saka (*34*) has investigated the behavior of timber-concrete floors. This work, like that of Pincus, indicates that it is possible to use full composite action to get stiffer and stronger beams. Ahmadi and Saka suggest that the increase in strength and stiffness can result in a 50 percent savings in timber joist costs.

The research on both wood-FRP and wood-concrete composite beams shows promising results, but no work appears to have been done involving the combination of all three materials. The rest of this paper addresses the potential use of concrete-engineered wood-FRP beams for timber bridges.

Design of Timber Bridge Beams

A complete manual treating the design, construction, inspection, and maintenance of timber bridges was written recently (*12*). When AASHTO *LRFD Specification for Highway Bridge Design* (*35*) was written, the timber bridge manual was used as a guideline. The remainder of this paper will refer to guidelines detailed in the timber bridge manual (*12*). For design, the allowable stress methodology is followed. Beams are designed for bending, deflection, shear, and bearing. As a result, values for allowable bending stress, F_b; shear stress, F_v; bearing stress, $F_{c\perp}$ (compression perpendicular to the grain); and modulus of elasticity, E (flexural modulus) are needed. Typical values of F_b, F_v, $F_{c\perp}$, and E for southern pine are 10.7 MPa, 0.62 MPa, 1.4 MPa, and 11.0 GPa, respectively; for LVL they are 20.2 MPa, 2.0 MPa, 5.2 MPa, and 13.8 GPa, respectively.

With regard to bending, the applied bending stress (f_b) must be lower than the adjusted allowable bending stress, $F_{b'}$. In determining $F_{b'}$, the allowable bending stress for the wood (F_b) is modified by several factors to account for load duration, moisture content, fire retardance, temperature, and size effects. In addition, the adjusted allowable stress also takes into account lateral stability and beam slenderness.

In designing wood beams for deflection, both short- and long-term deformations are considered. For both cases, deflections are computed using standard beam equations derived for linear-elastic, isotropic materials (e.g., $\Delta = 5wL^4/384E'I$ for a simply supported beam with a uniformly distributed load w and length L). In computing an adjusted modulus for the wood (E'), modifications are made for moisture content, fire retardance, and temperature. Although the standard equations are relatively accurate for short-term elastic deflections, they cannot account for the long-term effects of creep. To do so, it is common to increase the dead-load deflection by anywhere from 50 percent for engineered wood to 100 percent for unseasoned lumber. Ritter (*12*) recommends limiting short-term deflections to $L/360$ and long-term deflections to $L/240$.

Beams must be designed not only for normal bending stresses f_b, but also for shear stresses, f_v. For timber beams, horizontal as opposed to vertical shear will always govern (*12*). As with bending, design for shear requires that the maximum applied shear stress f_v be less than the adjusted allowable value, $F_{v'}$. In modifying the allowable shear stress, F_v, effects of moisture content, fire retardance, and temperature are accounted for.

The final design criterion for timber beams is bearing. The designer must ensure that a beam's bearing area is large enough to prevent excessive deformation. The procedure is similar to design for shear in that the applied bearing stress $f_{c\perp}$ must be less than the adjusted allowable bearing stress, $F_{c\perp'}$, which is modified for effects of moisture content, fire retardance, and temperature.

All of these criteria are intended for use in designing wood beams (engineered and nonengineered). The idea of designing concrete-wood-FRP composite beams raises the following questions:

1. Can FRP plates be bonded effectively to the tension face of the wood?
2. Can concrete flanges be made to act compositely with a wood web, and what is the maximum effective flange width?
3. What effect will the concrete and FRP have on the flexural stiffness of the beams, and how can accurate deflections be computed?
4. What effect will the concrete and FRP have on the ultimate load-carrying capacity and failure mode of the beams?
5. Which material will govern design for bending and shear, and how can stresses in these materials be computed?
6. How will varying the stiffness and strength of the FRP plates and concrete affect beam behavior?
7. Will concrete flanges, being part of a continuous deck, eliminate lateral stability problems once the concrete has cured, and what amount of bracing is necessary before curing?
8. How will the composite beams behave under sustained loads?
9. What will be the long-term durability of the composite beams?
10. Do the factors used for adjusting allowable wood stresses need to be reevaluated for use with composite beam design?

To gather information about concrete-wood-FRP beam behavior needed to help answer some of these questions, an experimental study was conducted.

Experimental Study Involving Composite Wood Beams

The experimental study included testing thirteen 1.83-m-long beams to failure. The beams, consisting of different combinations of engineered wood, CFRP plates, and concrete, included three plain wood beams (W1–W3), three wood beams with a CFRP plate attached to the tension face (WF1–WF3), four wood beams with a concrete compression flange attached (CW1–CW4), and three concrete-wood-CFRP beams (CWF1–CWF3). Dimensions for a typical CWF beam are shown in Figure 1. The cross-sectional dimensions of the other

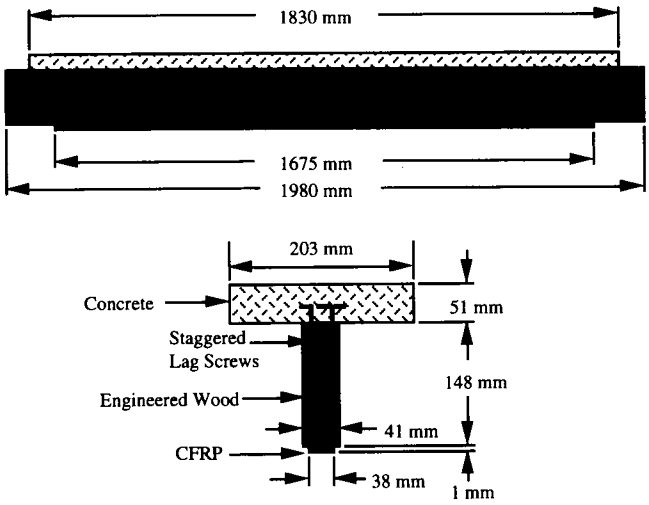

FIGURE 1 Typical CWF beam.

beams were identical to the CWF beams, with the concrete flange or the CFRP plate absent.

Beam Fabrication

To create composite beams, concrete flanges needed to be added to the CW and CWF beams and CFRP plates needed to be attached to the WF and CWF beams. To form the 51-mm-deep compression flanges, concrete having a water-to-cement ratio of approximately 0.40 by weight was cast directly onto the wood beams. A 100-mm^2 welded wire fabric was used as nominal reinforcement. Three batches of concrete were used to fabricate the CW and CWF beams, and standard 6-in.-diameter concrete test cylinders were poured with each batch. The concrete beams and cylinders were allowed to cure for 28 days before being tested. Results of the compression tests performed on the cylinders are presented in a subsequent section.

In attempts to achieve composite action, mechanical shear connectors were used. The connectors consisted of Grade 2 lag screws having a yield strength of 262 MPa. For three of the CW beams (CW1–CW3), 6.35-mm-diameter screws, staggered and longitudinally spaced at 76.2 mm, were used. For one CW beam (CW4) and all three CWF beams, 9.53-mm-diameter screws, staggered and longitudinally spaced at 50.4 mm, were used. The second arrangement was adopted after tests of beams CW1–CW3 indicated that composite action was lost before beam failure.

The CFRP plates were attached to the wood using a resorcinol-phenolformaldehyde adhesive in combination with a paraformaldehyde hardener. The bonding technique involved cleaning the wood and CFRP surfaces with acetone, applying a thin layer of adhesive to the wood, positioning the CFRP strip on the wood, and applying a uniform clamping pressure. The adhesive was then allowed to cure at room temperature for 24 hr before the pressure was removed.

Bond tests were performed to determine the resulting ultimate bond strength of the wood-CFRP connection. The tests consisted of loading three double-lap butt joints in tension to failure. From the tests, the average ultimate shear stress of the bond was found to be 3.95 MPa.

Material Properties

The CFRP plates used were made from unidirectional laminates. Each plate had a cured thickness of 1.0 mm, a width of 38 mm, and a length of 1675 mm. Carbon fiber–reinforced plates were chosen instead of glass or aramid fiber–reinforced plates because of the higher stiffness of the CFRP material.

To find the tensile properties of the CFRP plates, three tension samples were prepared and tested in accordance with ASTM D3039/3039M-93. The CFRP plates behaved linearly to failure with an average tensile modulus of 124 GPa and an ultimate strength of 1500 MPa.

The LVL used was made of southern pine. Because of the orthotropic nature of wood, its properties are different in the three mutually perpendicular directions; for wood, these directions are parallel to the grain, normal to the growth rings, and tangential to the growth rings. The second two directions are considered perpendicular to the grain. To determine values for the elastic moduli, ultimate shear stress, and ultimate bearing stress of the LVL, four types of wood samples were tested in accordance with ASTM D143–83. Test results indicated a uniaxial tensile modulus of 17.8 GPa (ultimate tensile strains ranged from 0.0025 to 0.0035 mm/mm), a uniaxial compression modulus of 8.3 GPa, an ultimate shear stress of 8.4 MPa, and an ultimate bearing stress of 28.2 MPa. Bending characteristics of the wood (flexural modulus, E, and flexural stiffness, EI) were assessed on the basis of the behavior of beams (W1–W3). The results (presented later) indicate a flexural modulus for the wood of 14.6 GPa. Finally, it should be noted that the approximate moisture content of the wood was 8 percent.

As mentioned previously, standard concrete test cylinders were cast when the CW and CWF beams were constructed. Results of compression tests on the cylinders indicate ultimate compressive strengths (f'_c) of 42.7 MPa for CW1–CW3, 42.0 MPa for CW4, and 29.6 MPa for the three CWF beams. The lower concrete strength associated with the CWF beams was a result of a slightly higher water-cement ratio.

Beam Test Procedure

All 13 beams were tested in four-point bending with the loads applied at the one-third points. The beams were loaded monotonically to failure using an 890-kN Universal testing machine. During the test, deflections were measured at midspan and beneath the loads. Gauges were used to record longitudinal strains through the beam's depth at both midspan and in the shear span. For the beams with CFRP plates, care was taken to ensure that the plates were not clamped beneath the end supports. The test setup and instrumentation plan is shown schematically in Figure 2.

Although slip between the CFRP and the wood, or between the wood and concrete, was not measured directly, the distribution of strains measured in the shear span allowed slip to be identified. Furthermore, glass slides were bonded to the side of the wood web and the underside of the concrete flange. By doing this, relative slip caused the glass slide to break.

Test Results

Results of the 13 beam tests are presented in Table 1. A detailed discussion of the test results is presented in the sections that follow.

Load-Displacement Behavior and Failure Mode of Test Beams

All three wood beams (W1–W3) displayed virtually identical linear load-deflection behavior to failure (Figure 3*a*). Furthermore, linear strain distributions through the depth of the members at midspan were recorded at the load, causing $\Delta = L/360 = 5.1$ mm (serviceability limit), and at 95 percent of the failure load (Figure 4). All wood beams failed in flexure due to splitting or snapping of wood fibers in the tension zone. At failure, outer-fiber tensile strains averaged 0.0033 mm/mm.

Like the wood beams, the three WF beams displayed both linear load-deflection behavior to failure (Figure 3*b*) and linear strain distribution at midspan (Figure 4). Flexural failure of these beams was caused by the tensile failure of the wood. For all three beams, the CFRP plates remained bonded to the wood up to failure. The primary difference between the W and WF beams was that the CFRP plate enabled the WF beams to reach an average outer-fiber tensile strain at failure of 0.0048 mm/mm (compared with 0.0033 mm/mm for the W beams).

Initially, both the CW and CWF beams behaved in a similar manner. They exhibited linear response, in terms of displacements and strain profiles at midspan, beyond the serviceability limit of $\Delta = 5.1$ mm (Figures 3*c*, 3*d*, and 4*a*). With regard to the strain profile, it can be seen that the concrete flange caused a significant upward shift of the neutral axis as compared with the W and WF beams. Upon further loading, the beams eventually began to lose composite action (at this point in the loadings, breakage of the glass slides was observed). Loss of

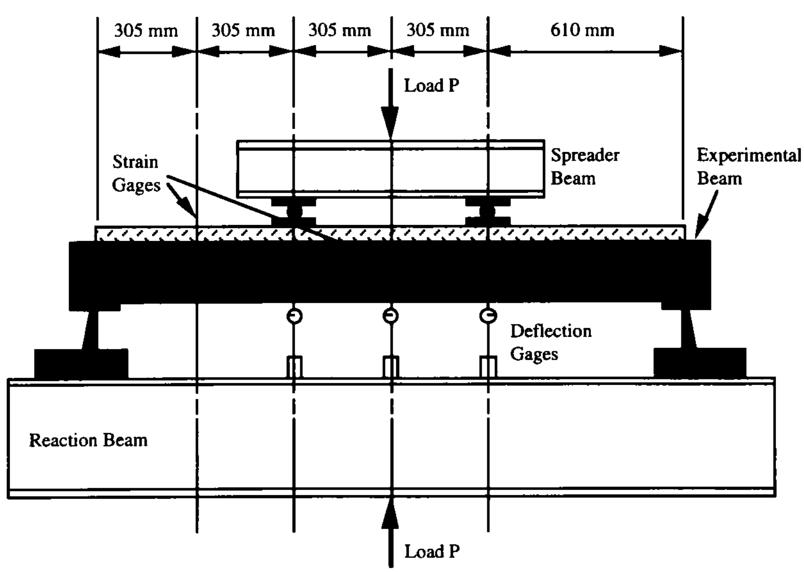

FIGURE 2 Typical beam test setup.

TABLE 1 Test Results for Each Beam

Beam	Concrete Strength, MPa	Mode of Failure	Load to Cause Δ=L/360, N	Ultimate Beam Strength, N
W1	NA[a]	Flexural Failure of Wood	6,715	27,812
W2	NA[a]	Flexural Failure of Wood	6,420	27,812
W3	NA[a]	Flexural Failure of Wood	6,882	28,925
WF1	NA[a]	Flexural Failure of Wood	7,679	43,387
WF2	NA[a]	Flexural Failure of Wood	8,287	37,825
WF3	NA[a]	Flexural Failure of Wood	8,147	37,825
CW1[b]	42.7	Flexural Failure of Wood	26,216	57,850
CW2[b]	42.7	Flexural Failure of Wood	28,814	56,737
CW3[b]	42.7	Flexural Failure of Wood	28,925	62,300
CW4[c]	42.0	Shear Failure of Wood	27,145	75,650
CWF1[c]	29.6	Shear Failure of Wood	27,456	73,425
CWF2[c]	29.6	Shear Failure of Wood	28,419	84,550
CWF3[c]	29.6	Shear Failure of Wood	28,244	80,100

[a] Not applicable.

[b] 6.35 mm diameter lag screws spaced at 76.2 mm.

[c] 9.53 mm diameter lag screws spaced at 50.4 mm.

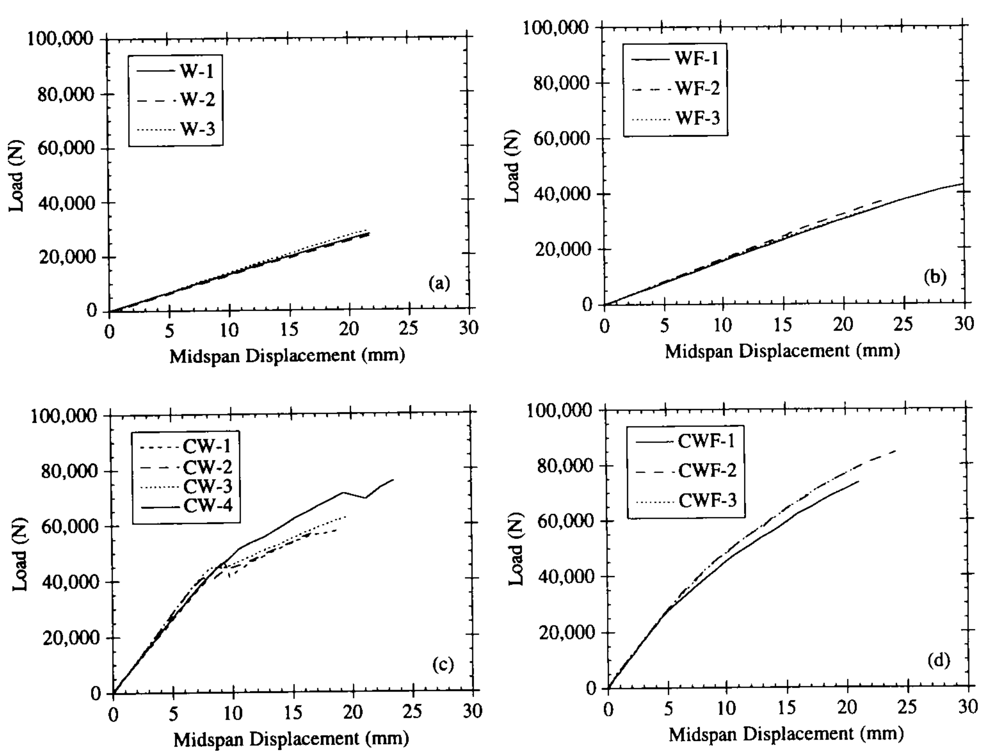

FIGURE 3 Load versus midspan deflection behavior: (*a*) W beams, (*b*) WF beams, (*c*) CW beams, (*d*) CWF beams.

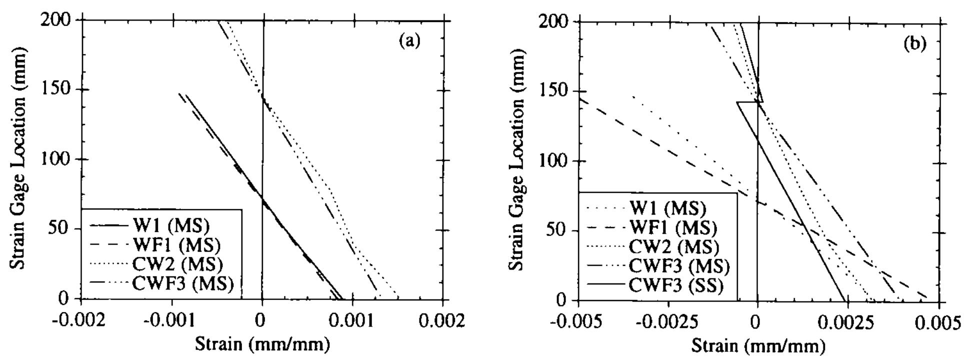

FIGURE 4 Strain profiles at midspan (MS) and in shear span (SS): (*a*) service load region ($\Delta = L/360$) and (*b*) at 95 percent of ultimate load.

composite action results in the reduced stiffness exhibited in the load-deflection response (Figures 3*c* and 3*d*) and in a disjointed shear-span strain distribution (typical) of Beam CWF3 (Figure 4*b*). Although the increased size and decreased spacing of lag screws used in Beams CW4 and CWF1–CWF3 appeared to eliminate the abrupt loss of composite action seen in beams CW1–CW3, some loss of composite action still occurred before failure.

The initial displacement of the CW and CWF beams was similar, but the presence of the composite plate caused the failure modes to differ. All of the CWF beams experienced longitudinal shearing of the wood web. As can be seen in Figure 4*b*, when Beam CWF3 failed, the midspan tensile strain at the outer fiber had not reached the level achieved by the WF beams (0.0048 mm/mm). In fact, the average tensile strain at the outer fiber at midspan was 0.0042 for the CWF beams. Unlike the CWF beams, failure of Beams CW1–CW3 was caused by tensile failure of the wood. Without the presence of the CFRP plate, these beams failed when the wood at midspan reached tensile strains at the outer fiber averaging 0.0032 mm/mm. Of the CW beams, only CW4 experienced a shear failure of the wood. For this beam, tensile failure of the wood did not occur even though an outer-fiber tensile strain of 0.0040 mm/mm was reached.

Changes in Stiffness and Strength of Test Beams

Using deflections recorded along the constant moment region, moment-curvature plots for the various beams were computed. By comparing both moment-curvature and load-deflection plots in the service load region (Figure 5), it is seen that both the CFRP plate and the concrete flange led to increases in stiffness as compared with the wood beams. A comparison of average slopes (*EI*) of the moment-curvature response of the WF, CW, and CWF beams with the slope of the W (control) beams reveals increases in flexural stiffness of 21, 487, and 533 percent, respectively (Table 2). By dividing the flexural stiffness of the beams by their moments of inertia, one can find values for the flexural modulus. Doing so for the W beams results in a modulus of 14.6 GPa, which is within 6 percent of the design value of

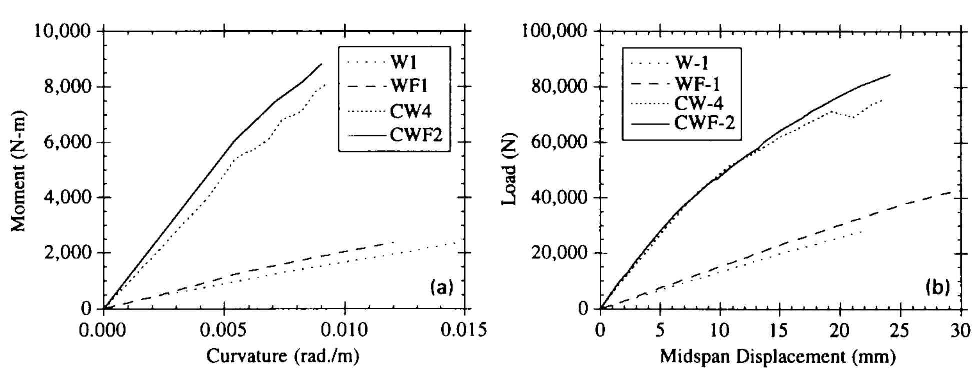

FIGURE 5 Comparison of typical (*a*) moment-curvature behavior in service load region ($\Delta \leq L/360$) and (*b*) load versus midspan deflection behavior to failure.

TABLE 2 Stiffness Increase for Beams in Service Load Region ($\Delta \leq L/360$)

Beams	Flexural Stiffness $(EI)^a$, N-m^2	Flexural Stiffness of Wood Beams $(EI)^a$, N-m^2	Percentage Increase
WF1, WF2, WF3	195,282	161,492	+21
CW1, CW2, CW3, CW4	947,460	161,492	+487
CWF1, CWF2, CWF3	1,021,529	161,492	+533

[a]All values are averages of like beams.

13.8 GPa. Converting the CFRP plate and concrete flange into equivalent amounts of wood produces flexural moduli of 15.2, 14.9, and 15.0 GPa for the WF, CW, and CWF beams, respectively. These values are all within 7 percent of the measured flexural moduli of the W beams, which indicates that accurate short-term service-load deflections can be computed by first converting the composite cross section to equivalent amounts of wood, and then using the wood's flexural modulus.

As for ultimate strengths, the WF, CW, and CWF beams show increases over the W beams of 40, 168, and 181 percent, respectively (Table 3). In addition, all three beam types exhibited larger deflections at failure than did the wood beams (Figure 5*b*). The larger deflections translated are consistent with the observed 44.7 percent increase in ultimate tensile strain of the wood at failure between the W and WF beams (0.0033 to 0.0048 mm/mm). Finally, it is interesting that the increases in stiffness (21 percent) and strength (40 percent) of the WF beams compared with the W beams were produced by adding a CFRP plate that has a cross-sectional area that is only 0.6 percent of the wood member.

Conclusions and Design Implications

Thirteen beams were tested to assess the use of composite beams made of various combinations of concrete, engineered wood, and CFRP materials in timber bridge design. Although additional research will be needed to answer fully all of the important questions surrounding the design of such beams, the research results presented herein provide the following insights:

- CFRP plates can be bonded effectively to the tensile face of engineered wood beams.
- Although concrete compression flanges were made to act compositely with wood webs beyond the serviceability limit, some loss of composite action was experienced before failure. Improved methods for developing full composite action should be sought.
- The incorporation of CRFP plates and concrete flanges led to significant increases in flexural stiffness and ultimate strength over plain engineered wood beams.
- CFRP plates adhered to the tension face of engineered wood beams led to increases in the tensile strain capacity of the wood.
- For all beams tested, failure of the wood (either in flexure or shear) initiated the overall failure. For the W and WF beams, flexural failure of the wood occurred. However, the large increase in flexural capacity associated with the CW and CWF beams changed the mode of failure from flexure to shear for some beams.
- Accurate service-load deflections can be computed by converting the concrete and CFRP plate into equivalent amounts of wood.

In summary, the test results indicate that adding concrete and CFRP to engineered wood beams can improve significantly their overall flexural behavior. However, before final design criteria are developed, several issues still

TABLE 3 Ultimate Strength Increase for Beams

Beams	Ultimate Beam Strength[a], N	Wood Beam Strength[a], N	Percentage Increase
WF1, WF2, WF3	39,679	28,183	+40
CW1, CW2, CW3, CW4	75,650	28,183	+168
CWF1, CWF2, CWF3	79,358	28,183	+181

[a]All values are averages of like beams.

must be addressed, including the behavior of composite beams under sustained and repeated loads, the long-term durability of the bonded composite plates, the behavior of full-scale beams, and the identification of appropriate methods of analysis needed to predict ultimate beam strength as well as behavior at service loads.

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REFERENCES

1. Cooper, J. D. A New Era in Bridge Engineering Research. *Proc., 2nd Workshop on Bridge Engineering Research in Progress*, National Science Foundation, Reno, Nev., 1990, pp. 5–10.
2. Scalzi, J. B. Looking Ahead to the 21st Century. *Proc., 2nd Workshop on Bridge Engineering Research in Progress*, National Science Foundation, Reno, Nev., 1990, pp. 1a–1b.
3. Ritter, M. A. *Current Timber Research and Development in the United States*. Forest Products Laboratory, U.S. Department of Agriculture, Madison, Wisc., 1993.
4. Wipf, T. J., M. A. Ritter, S. R. Duwadi, and R. C. Moody. *Development of a 6-Year Research Needs Assessment for Timber Transportation Structures*. Report FPL-GTR-74. Forest Products Laboratory, U.S. Department of Agriculture, Madison, Wisc., 1993.
5. Oliva, M. G., R. L. Tuomi, and A. G. Dimakis. New Ideas for Timber Bridges. In *Transportation Research Record 1053*, TRB, National Research Council, Washington, D.C., 1986, pp. 59–64.
6. Brungraber, R., R. Gutkowski, W. Kindya, and R. McWilliams. Timber Bridges: Part of the Solution for Rural America. In *Transportation Research Record 1106*, TRB, National Research Council, Washington, D.C., 1987, pp. 131–139.
7. Gutkowski, R. A., and W. J. McCutcheon. Comparative Performance of Timber Bridges. *Journal of Structural Engineering*, ASCE, Vol. 113, No. 7, July 1987, pp. 1468–1485.
8. Behr, R. A., E. J. Cundy, and C. H. Goodspeed. Cost Comparison of Timber, Steel, and Prestressed Concrete Bridges. *Journal of Structural Engineering*, ASCE, Vol. 116, No. 12, Dec. 1990, pp. 3448–3456.
9. Oliva, M. G. and A. G. Dimakis. Behavior of Stress-Laminated Timber Highway Bridge. *Journal of Structural Engineering*, ASCE, Vol. 114, No. 8, Aug. 1988, pp. 1850–1869.
10. Moody, R. C., M. A. Ritter, and H. GangaRao. Applications of Wood Materials for Innovative Bridge Systems. *Proc., 1st Materials Engineering Congress*, Denver, Colo., 1990, pp. 423–432.
11. Coger, T., ed. Engineered Wood Products Manufactured by Trus Joist MacMillan. *Crossings*, USDA Forest Service, Morgantown, W.Va., 1993, pp. 1–2.
12. Ritter, M. A. *Timber Bridges: Design, Construction, Inspection, and Maintenance*. Report EM 7700-8. U.S. Department of Agriculture, 1992.
13. Ballinger, C. A. Development of Composites for Civil Engineering. *Advanced Composite Materials in Civil Engineering Structures*, ASCE, Las Vegas, Nev., 1991, pp. 288–301.
14. Ballinger, C. A. Development of Fibre-Reinforced Plastic Products for the Construction Market—How Has and Can It Be Done. *Advanced Composite Materials in Bridges and Structures*, Canadian Society for Civil Engineering, Sherbrooke, 1992, pp. 3–13.
15. Bank, L. C. Questioning Composites. *Civil Engineering*, ASCE, Vol. 63, No. 1, Jan. 1992, pp. 64–65.
16. Chajes, M. J., T. A. Thomson, T. F. Januszka, and W. W. Finch. Flexural Strengthening of Concrete Beams Using Externally Applied Composite Materials. *Construction and Building Materials*, Vol. 8, No. 3, Sept. 1994, pp. 191–201.
17. Chajes, M. J., T. F. Januszka, T. A. Thomson, W. W. Finch, and D. R. Mertz. Shear Strengthening of Reinforced Concrete Beams Using Externally Applied Composite Fabrics. *ACI Structural Journal* (in press).
18. Head, P. R. Design Methods and Bridge Forms for the Cost-Effective Use of Advanced Composites in Bridges. *Advanced Composite Materials in Bridges and Structures*, Canadian Society for Civil Engineering, Sherbrooke, 1992, pp. 15–30.
19. Hoa, S. V. Design of a Composite Box Beam for Building Applications. *Advanced Composite Materials in Bridges and Structures*, Canadian Society for Civil Engineering, Sherbrooke, 1992, pp. 501–512.
20. McCormick, F. C. Advancing Structural Plastics into the Future. *Journal of Structural Engineering*, ASCE, Vol. 114, No. 3, March 1988, pp. 235–243.
21. Measures, R. M. Smart Structures—A Revolution in Civil Engineering. *Advanced Composite Materials in Bridges and Structures*, Canadian Society for Civil Engineering, Sherbrooke, 1992, pp. 31–59.
22. Meier, U., and H. Kaiser. Strengthening of Structures with CFRP Laminates. *Advanced Composite Materials in Civil Engineering Structures*, ASCE, Las Vegas, Nev., 1991, pp. 224–232.
23. Saadatmanesh, H., and M. R. Ehsani. Fiber Composite Plates Can Strengthen Beams. *Concrete International*, Vol. 12, No. 3, March 1990, pp. 65–71.
24. Sotiropoulos, S. N., and H. GangaRao. Bridges Systems Made of FRP Components. *Proc., 2nd Workshop on*

Bridge Engineering Research in Progress, National Science Foundation, Reno, Nev., 1990, pp. 295–298.

25. Triantafillou, T. C., N. Plevris, and N. Deskovic. Nonprestressed and Prestressed FRP Sheets as External Reinforcement of Wood Members. *Proc., Materials Engineering Congress*, ASCE, Atlanta, GA., 1992, pp. 710–717.
26. Triantafillou, T. C., and N. Deskovic. Prestressed FRP Sheets as External Reinforcement of Wood Members. *Journal of Structural Engineering*, ASCE, Vol. 118, No. 5, May 1992, pp. 1270–1284.
27. Davalos, J. F., H. A. Salim, and U. Munipalle. Glulam-GFRP Composite Beams for Stress-Laminated T-System Bridges. *Advanced Composite Materials in Bridges and Structures*, Canadian Society for Civil Engineering, Sherbrooke, 1992, pp. 455–464.
28. Davalos, J. F., E. Barbero, and U. Munipalle. Glued-Laminated Timber Beams Reinforced with E-Glass/Polyester Pultruded Composites. *Proc., 10th Structures Congress*, ASCE, San Antonio, Tex., 1992, pp. 47–50.
29. Leichti, R. J., P. C. Gilham, and D. A. Tingley. The Taylor Lake Bridge: A Reinforced-Glulam Structure. *Wood Design Focus*, Vol. 4, No. 2, 1993.
30. Richart, F. E., and C. B. Williams, Jr. *Tests of Composite Timber and Concrete Beams.* Bulletin Series 343. University of Illinois Engineering Experiment Station, 1943, pp. 1–62.
31. McCullough, C. B. Oregon Tests on Composite (Timber-Concrete) Beams. *Proceedings*, ACI, Vol. 39, No. 4, 1943.
32. Pincus, G. Bonded Wood-Concrete T-Beams. *Journal of the Structural Division*, ASCE, Vol. 95, No. 10, Oct. 1969, pp. 2265–2279.
33. Pincus, G. Behavior of Wood-Concrete Composite Beams. *Journal of the Structural Division*, ASCE, Vol. 96, No. 10, Oct. 1970, pp. 2009–2019.
34. Ahmadi, B. H., and M. P. Saka. Behavior of Composite Timber-Concrete Floors. *Journal of Structural Engineering*, ASCE, Vol. 119, No. 10, Oct. 1993, pp. 3111–3130.
35. *LRFD Specification for Highway Bridge Design.* AASHTO, Washington, D.C., 1994.