Development and Field Testing of the Camp Arrowhead Modular Stress-Laminated T-System Timber Bridge

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The development and field testing of a new modular stress-laminated T-system timber bridge, which is expected to become cost-competitive with precast concrete bridges, is presented. The new modular design can increase the quality of the product and decrease fabrication and installation efforts. The analysis methods used to predict the bridge response include orthotropic finite element (FE) modeling, a macrosolution for a stiffened plate, and a design method based on the FE modeling and macroanalyses. Details of the fabrication and installation procedures are presented. The Camp Arrowhead bridge is tested under a 231-kN (52-kip) loaded truck, and the measured and predicted responses are compared and discussed.

Modern timber bridges, recently built in West Virginia, consist of hardwood stress-laminated decks combined with softwood glued-laminated beams compressed together by high-strength steel bars to form T- and Box-systems. Current design codes do not provide standards for stress-laminated T-system timber bridges. Therefore, the Constructed Facilities Center (CFC) at West Virginia University (WVU) has developed design procedures, called the WVU method, for the design of these systems (*1*). The method has been adopted by the West Virginia Department of Transportation (WVDOT). However, higher-than-expected costs and inefficient construction practices have brought about a need for innovative design for stress-laminated T-system timber bridges. In response, a modular stress-laminated T-system timber bridge, which is expected to become competitive with precast concrete bridges, has been developed.

Modular T-system timber bridges developed in West Virginia consist of cells or modules, each about 122 to 152.4 cm (4 to 5 ft) wide and extending the full length of the bridge. Two glued-laminated timber (glulam) beams are transversely stress-laminated to approximately 30 deck lumber planks [3.8 cm (1.5 in.) thick] to form a single module. The depth of the beams depends on the bridge span and loading. Figures 1 and 2 show the cross section details of the Camp Arrowhead timber bridge, which was designed using the WVU method.

This paper presents an overview of the design and details of the construction, testing, and response evaluation of the two-lane, 18.9-m (62-ft) span Camp Arrowhead bridge built in Cabell County, West Virginia, in 1992. Response predictions are obtained by finite element (FE) analysis, a macrosolution, and the simplified WVU design method. The bridge is tested using a 231-kN (52-kip) double-axle truck, and deflections and strains are recorded and compared with predicted values.

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ANALYSIS AND DESIGN OVERVIEW

The design of stress-laminated T-system timber bridges is based on a macroflexibility solution of a stiffened orthotropic plate. A one-term approximation of this solution is used to define a wheel load distribution factor, which reduces the design of the structure to the design of a T-beam section. To analyze this T-beam section using beam theory, an effective flange width is computed from expressions obtained from a parametric FE analysis. The overview presented in this section describes the longitudinal global response, the transverse local effects in a deck section between two adjacent stringers, and design considerations.

Longitudinal Global Response

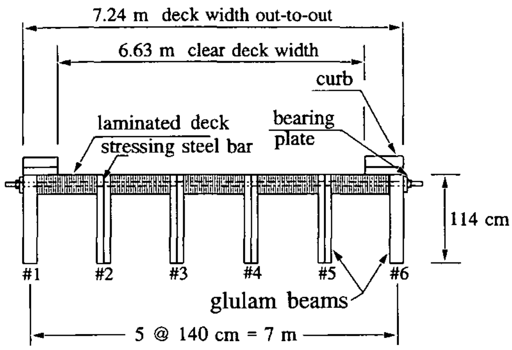
A generalized deflection function of a simply supported orthotropic plate stiffened by longitudinal stringers is obtained by a macrosolution (*1*,*2*). Using a first-term approximation, a transverse wheel load distribution factor W_f is defined as the ratio of the interactive forces acting on a stringer to the sum of interactive forces acting on all stringers (*3*). Using this concept, the maximum wheel load distribution factor for a symmetric load on an interior stringer can be written as

$$W_f = \frac{1 + C_0}{(n + 1)C_0 + \frac{2}{\pi} n} \tag{1}$$

where

C_0 = edge deflection coefficient = $\frac{b}{\pi}\frac{D_y}{D_e}\left(\frac{1 + 8\gamma^2}{\gamma^4}\right)$,
D_y = transverse bending stiffness of deck,
D_e = bending stiffness of composite edge stringer,
γ = aspect ratio = bridge width/bridge length, and
n = number of stringer spacings.

Equation 1 is presented to illustrate that the wheel load distribution factor W_f accounts for the orthotropic property of the system and simulates the portion of the actual truck loading carried by an interior stringer. Therefore, the defi-



Note: 1 ft = 0.305 m; 1 in = 2.54 cm

FIGURE 1 Cross section of the Camp Arrowhead bridge.

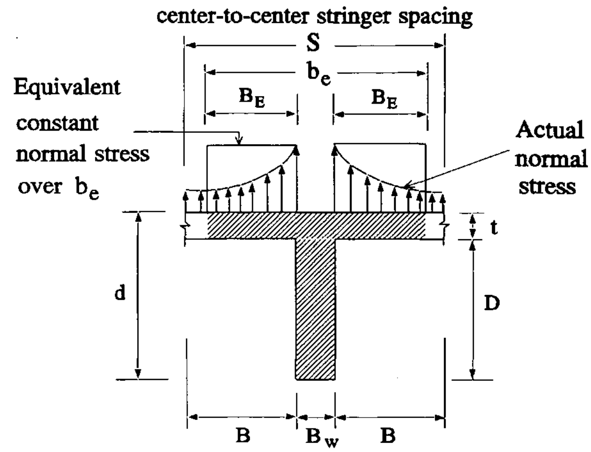


FIGURE 3 Isolated T-beam.

nition of W_f reduces the design of the system to the design of a composite T-beam. However, since the normal stress distribution in the flanges of the T-beam is nonlinear (Figure 3), we define an equivalent effective flange width over which the normal stresses can be assumed to be constant consistent with beam theory. The effective overhanging flange width B_E is computed from

$$\frac{B_E}{B} = 0.4586 + \frac{1}{198}\left(\frac{L}{B}\right)\left(\frac{D}{t}\right)\left(\frac{E_s}{E_d}\right) \tag{2}$$

where L is the bridge span, E_s and E_d are, respectively, the longitudinal elastic moduli of the stringer and the deck, and D and t are defined in Figure 3. Then, $b_e \leq S$.

In the WVU design method, the wheel load distribution factor (Equation 1) and the effective flange width (Equation 2) permit the designer to isolate a deck-and-beam portion of the bridge (Figure 3) and to design it as a T-beam. The T-beam is loaded at the center by an equivalent concentrated load P_d that produces a maximum moment equal to the maximum AASHTO lane moment (*4*), modified for wheel load distribution and number of lanes. Using Equation 1, the maximum live load moment, M_{LL}, and deflection, δ_{LL}, for an interior stringer are computed from (*1*)

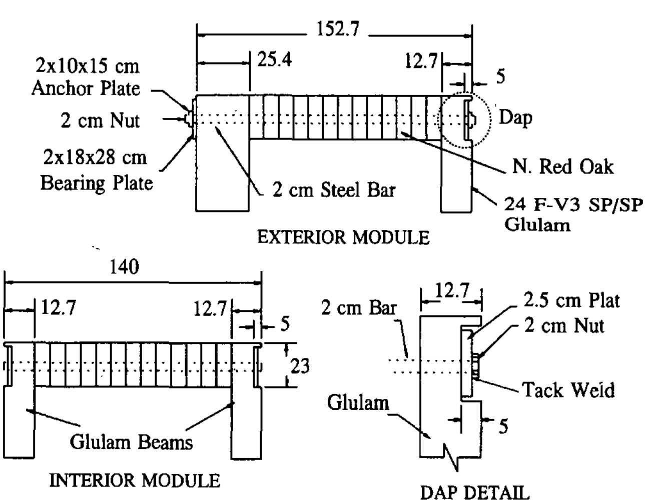
$$M_{LL} = \frac{P_d L}{4} \tag{3}$$

$$\delta_{LL} = \frac{P_d L^3}{48\, E_s I_c} \tag{4}$$

where

P_d = (AASHTO lane moment) $\left(\frac{4}{L}\right)(N_L)(W_f)$,
N_L = number of lanes,
E_s = modulus of elasticity of the stringer, and
I_c = transformed composite moment of inertia of the effective T-section (Figure 3).

To analyze the global longitudinal response of the bridge, the total normal stresses and longitudinal deflections are computed from beam theory using the properties of the transformed T-beam section.



Dimensions in cm.
Note: 1 in = 2.54 cm

FIGURE 2 Detail of Camp Arrowhead modular sections.

Transverse Local Response

For a trial deck thickness, the spacing of the stringers S is designed by computing the maximum transverse local deflection δ_{max} and stress σ_{max} (local effects) directly under a wheel load applied at the midspan of a deck section between stringers. The limits on local deflection can be based on human response and pavement cracking considerations. The local effects are computed from (*1*)

$$\delta_{max} = \frac{PS^3}{4\alpha E_T t^4} \qquad \sigma_{max} = \frac{3PS}{2\beta t^3} \tag{5}$$

where

$$\alpha = -10.9 + 7.8\left(\frac{S}{t}\right) + 0.27\left(\frac{E_L}{E_T}\right),$$

$$\beta = 3.0 + 3.1\left(\frac{S}{t}\right) + 0.152\left(\frac{E_L}{E_T}\right),$$

E_L, E_T = longitudinal and transverse elastic moduli of the deck,
P = resultant of a rear wheel load of an AASHTO truck [89 kN (20 kips) for HS-25 loading], and
t = thickness of the deck.

Design Considerations

The design procedure described in this section applies to multiple-lane, single-span, simply supported, stress-laminated T-system bridges under HS-20 or HS-25 truck loading. The required material properties are longitudinal modulus of elasticity of the deck (*5*) modified by a butt joint factor (*6*), transverse modulus of elasticity and in-plane shear modulus of the deck (*7,8*), and longitudinal modulus of elasticity of the glulam stringers (*5*). The allowable bending strength of the deck planks (*5*) is multiplied by a factor of 1.3 or 1.5 (*9,10*), depending on the lumber grade.

The objectives of the design are (*a*) to determine the maximum spacing of the stringers on the basis of guidelines for transverse local effects and (*b*) to determine the optimum dimensions of the stringer and the deck on the basis of guidelines for global bending effects. In most cases, the bending strength of the glulam stringers and the maximum local deflection of a deck section (relative to the stringers) control the design. The interior stringer is designed for symmetric AASHTO truck loading, as described previously, and the exterior stringer is designed for asymmetric loading, as shown elsewhere (*1*).

The configuration, construction, and testing of the Camp Arrowhead bridge are presented next.

BRIDGE CONFIGURATION

The design is for a bridge of two lanes, 18.9-m (62-ft) span, and 7.24-m (23.75-ft) out-to-out width subjected to an AASHTO HS-20 truck loading. The bridge consists of five cells or modules. The interior modules are 140 cm (55 in.) out-to-out, and the exterior modules are 152.7 cm (60 in.) out-to-out (Figures 1 and 2). The deck is built with 3.8- × 23-cm (1.5- × 9-in.) northern red oak lumber (No. 2 grade), and the glulam stringers are 24F-V3, Southern Pine/Southern Pine (*5*). The width of the exterior stringers is 25.4 cm (10 in.) and of the interior stringers is 12.7 cm (5 in.); the interior stringers are placed side by side to form a beam 25.4 cm (10 in.) wide. On the basis of the WVU design method, the required depth of the stringers is 114 cm (45 in.).

BRIDGE CONSTRUCTION

The modular construction technique used for this bridge is an attempt to reduce the total cost of the structure by decreasing the on-site construction time, reducing the crane size requirement, and reducing to one the on-site stressing operations. Modular construction, which permits replacing on-site efforts with fabrication-shop efforts, can reduce labor costs and significantly improve the quality control of the product. The authors believe that modular construction of stress-laminated timber bridges using standardized modules can lead to off-the-shelf, mass-production of bridges at competitive prices. The first T-system timber bridge built in the United States was the 22.3-m (73.25-ft) Barlow Drive bridge, which was fabricated in two halves and installed in 1988 in Charleston, West Virginia (*11*). The total superstructure/installation cost for this bridge was \$850/m^2 (\$79/ft^2). In contrast, the construction of the Camp Arrowhead bridge was much more efficient, and the total superstructure/installation cost was \$570/m^2 (\$53/ft^2). The fabrication of a modular T-system timber bridge involves the following activities: material procurement and dimensioning, preservative treating, assembling, and stressing.

Material procurement and dimensioning processes for modular bridges are the same as for other stress-laminated bridges. To prevent creosote bleeding, a steaming cycle after creosote treatment is now required for bridge components, except for northern red oak. Initially, a minimum level of creosote retention of 128 to 192 kg/m^3 (8 to 12 lb/ft^3), depending on the species, was specified by WVDOT; at present, a maximum retention level of 160 to 224 kg/m^3 (10 to 14 lb/ft^3) is also being required. Use of fresh creosote and periodic removal of the insolubles from the creosote solution produce a much better finished product; the beams in the Camp Arrowhead bridge appear quite clean and are dry to the touch, even on the hottest summer days. Dripping of creosote from the bridge superstructure has been practically eliminated. Details on construction and quality control aspects are given elsewhere (*10,12*).

Assembly of the modular T-system bridge differs greatly from the assembly of the earlier bridges. Previously, the West Virginia timber bridges were constructed in half-width modules, which were shipped to the bridge site and stressed several times on site over a period of 6 weeks (*11*). The new T-system modular construction method can be briefly described as follows:

1. Each module consists of two full-length glulam beams and approximately 30 deck planks 3.8 cm (1.5 in.) thick (Figure 1).
2. Stressing bars are located on 61-cm (2-ft) centers with an additional set of fabrication bars located at 183-cm (6-ft) centers.
3. The fabrication bars have anchor plates that are inserted into daps cut into the glulam beams 12.7 cm (5 in.) wide, so that no hardware protrudes past the face of the beam (Figure 2).
4. The bars on 61-cm (2-ft) centers and the fabrication bars are tensioned three times at the fabrication shop, which completes the process required to minimize bar force losses during the expected life of the bridge.
5. Curbs and guiderail components are added to the exterior modules at the fabrication shop.
6. The modules are shipped to the bridge site, and the bars at 61-cm (2-ft) centers are removed, whereas the bars on 183-cm (6-ft) centers are left in place.
7. The modules are lifted into position on the prepared bridge seats.
8. The full-length stressing bars are pushed through the vacated holes on 61-cm (2-ft) centers, and the anchorage hardware is installed to tension the bars one time only.

9. Finally, the bridge is fastened to the abutments, an asphalt overlay is applied, and the bridge is ready for vehicular traffic.

The time required for the entire fabrication process still takes approximately 8 weeks, but the new modular process allows the on-site work to be done in 1 day rather than the 3 or 4 weeks required of our previous construction methods.

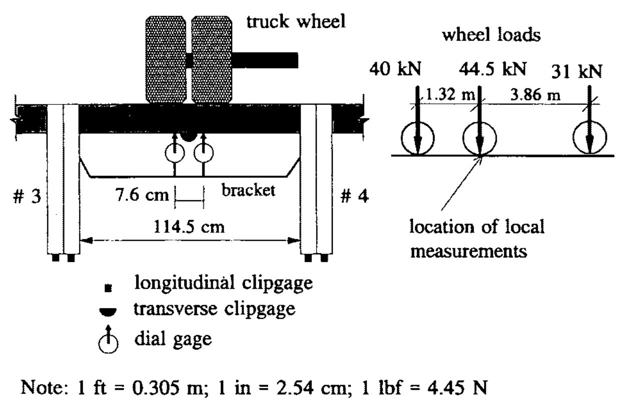
BRIDGE TESTING

The monitoring program of the Camp Arrowhead bridge includes measurements of bar force levels, moisture levels, live load deflections, and strains. The Camp Arrowhead bridge has been in service for only 4 months, but it appears that the bar forces are stable. The average prestress level of two of the prestressing bars is between 689 and 345 kPa (100 and 50 psi), which is considered acceptable. The average moisture content (MC) level in the bridge measured over a period of 4 months is between 17 and 21 percent; the assumed MC level in design is greater than or equal to 19 percent. The 231-kN (52-kip) loaded truck used to test the bridge was less than the design AASHTO HS-20 loading. Therefore, the actual truck load was used in the analysis to compare the predictions with the experimental field results. The location of the loaded truck, deflection measurements, and strain measurements are explained next.

Location of the Loaded Truck

A double-axle, 231-kN (52-kip) loaded truck was placed over the bridge, and the response of the superstructure was tested for three load conditions:

1. The truck was placed on the downstream lane of the bridge, facing the traffic direction (north); the center of gravity of the truck coincided with the midspan of the bridge.
2. In a similar manner, the truck was placed facing south on the upstream lane of the bridge.
3. The interior rear wheel of the truck was placed right at the midspan of the deck section of the middle module of the bridge (see Figure 4, which shows the deck section between Stringers 3 and 4 denoted in Figure 1).



Note: 1 ft = 0.305 m; 1 in = 2.54 cm; 1 lbf = 4.45 N

FIGURE 4 Measurement of beam strains and local effects.

Load Cases 1 and 2 were used to simulate the global response of the bridge under asymmetric loading, and, by superposition, these two load cases were also used to study the symmetric behavior of the bridge. Load Case 3 was used to study the local response of the middle deck section between two adjacent stringers.

Deflection Measurements

To measure the global live-load deflections with an engineer's level, sight rods were attached to the bottom of each of the six glulam beams. The rods were placed at quarter points along the span, which resulted in a total of 18 elevation reading points. To measure the local deflections in the middle deck section, relative to Stringers 3 and 4, two dial gauges were mounted on a steel angle bolted to the stringers (Figure 4).

Strain Measurements

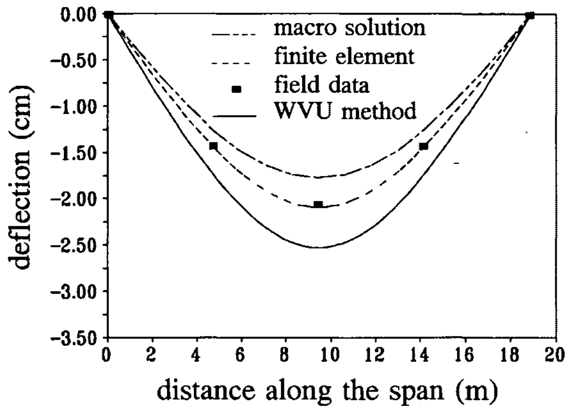
In general, to measure strains in wood with bonded strain gauges is a difficult task (*13*). Moreover, bonded gauges cannot be used in creosote-treated wood. Therefore, to measure strains in the Camp Arrowhead bridge, we used laboratory-built, clip-on strain transducers (*14*). Stringers 3 and 4 were each instrumented at the midspan with two clip gauges, which were placed on the bottom faces of the beams. Similarly, a clip gauge was placed transversely at the bottom face of the middle deck section (see Figure 4) to measure the transverse strain at the midspan of the deck.

BRIDGE RESPONSE EVALUATION

For comparative purposes, the Camp Arrowhead bridge is analyzed by an FE method, a macrosolution for a stiffened plate (*2*), and the WVU design method. A special FE formulation (*15*) modified for T-systems (*1*) is used in the analysis, and the deck is modeled with nine-node, orthotropic shell elements that include shear deformations. The beams are modeled with three-node, transversely isotropic (*16*), three-dimensional beam elements that include shear deformations. The comparisons of the global and local deflections and stresses are discussed in this section.

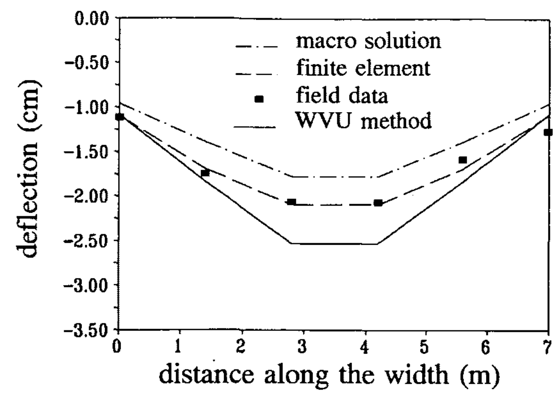
The longitudinal deflection profile of the middle stringer is shown in Figure 5. The FE maximum deflection prediction is within 2 percent of the experimental results, and the WVU design predictions are within 22 percent of the experimental results. The macrosolution prediction for a one-term approximation is within 13 percent of the experimental results. The transverse deflection profile is shown in Figure 6. Considering that the accuracy of the deflection measurements in the field is ±1.6 mm (±1/16 in.), the WVU design predictions are reasonably accurate.

The local transverse deflection in the deck section was measured under the interior rear wheel of the truck (see Figure 4). In the FE analysis, the deflections due to the interior and exterior rear wheel loads, at the location of the interior wheel, were computed separately. In the WVU method, only



Note: 1 ft = 0.305 m; 1 in = 2.54 cm

FIGURE 5 Deflection profile of interior Stringer 3.



Note: 1 ft = 0.305 m; 1 in = 2.54 cm

FIGURE 6 Transverse bridge deflection at midspan.

the transverse deflection exactly underneath the wheel load can be computed; therefore, the FE solution for the deflection due to the exterior rear wheel [1.31 m (4.3 ft) away] is added to the WVU method prediction and compared with the measured deflection. The same approach was used to compute the local transverse stresses. The experimental strains are converted to stresses by multiplying them by the transverse modulus of elasticity of the deck, which is assumed to be 172.25 MPa (25,000 psi) for a transverse prestress level of 345 kPa (50 psi) (*8*). The results, summarized in Table 1, show that the deflection predictions compare well with the field measurements. The predicted transverse stresses are 17 percent higher than the field data; this percent difference for structural timber is considered reasonable and acceptable (*13*).

The longitudinal global strains in the bottom surfaces of Stringers 3 and 4 were measured with clip-on gauges. The strains were converted to stresses by assuming a modulus of elasticity of 13.78 GPa (2.0×10^6 psi) for the tension laminae of the stringers. The global stresses and deflections for Stringers 3 and 4 are compared in Table 2. The WVU design deflection prediction is 12 percent higher than the FE prediction and only 3 percent higher than the field measurement. The FE stress predictions are only 8 percent higher than the measured values. However, the WVU design stresses are 13 percent higher than the FE solution and 22 percent higher than the measured value. Since the design method is expected to be conservative, the stress predictions are reasonably acceptable.

A visual inspection of the Camp Arrowhead bridge and an assessment of the MC levels, bar tension levels, and load response of the bridge indicate that the overall performance of the bridge is satisfactory and within the design expectations.

CONCLUSIONS

The development and load testing of a new stress-laminated, modular, T-system timber bridge are presented. This modular

TABLE 1 Local Response in Deck Section Between Stringers 3 and 4

	Finite Element			WVU method			Field data
	wheel			wheel			
	interior	exterior	total	interior	exterior	total	
deflection (cm)	0.134	0.075	0.209	0.136	0.075[a]	0.211	0.214
stresses (kPa)	209	106	315	275	106[a]	381	324

[a]From FE analysis
Note: 1 in = 2.54 cm; 1 psi = 6.89 kPa

TABLE 2 Global Deflections and Stresses of Interior Stringer

	Finite Element	Macro solution	WVU method	Field Data #3	Field Data #4
max. Deflection (cm)	2.10	1.79	2.53	2.06	2.06
max. stresses (MPa)	5.72	4.44	6.47	5.01	5.27

Note: 1 in = 2.54 cm; 1 psi = 0.00689 MPa system reduces fabrication, transportation, and installation efforts and allows for better quality control. The WVU design method described in this paper, based on rigorous closed-form and FE analyses, is sufficiently simple and reasonably accurate to predict deflections and stresses of stress-laminated T-system timber bridges. The comparisons with the test results indicate that the WVU design method predicts quite well the response of the Camp Arrowhead bridge and can be used for the design of stress-laminated T-system timber bridges. The Camp Arrowhead bridge is performing well in relation to its original design.

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REFERENCES

1. Davalos, J. F., and H. A. Salim. *Analysis and Design of Stress-Laminated T-System Timber Bridges*. Report CFC-92-TR153. Constructed Facilities Center, West Virginia University, Morgantown, 1991.
2. GangaRao, H. V. S., A. A. Elmeged, and V. K. Chaudhry. Macroapproach for Ribbed and Grid Plate Systems. *Journal of the Engineering Mechanics Division*, ASCE, Vol. 101, No. 1, 1975, pp. 25–43.
3. GangaRao, H. V. S., and P. R. Raju. Transverse Wheel Load Distribution for Deck-Stringer Bridges. *Proc., Third NSF Workshop on Bridge Engineering Research in Progress*, University of California, La Jolla, Nov. 1992, pp. 109–112.
4. *Standard Specifications for Highway Bridges* (14th edition). American Association of State Highway and Transportation Officials, Washington, D.C., 1989.
5. *National Design Specifications for Wood Construction*. National Forest Products Association, Washington, D.C., 1991.
6. Davalos, J. F., and D. A. Kish. Longitudinal Bending Stiffness of Stress-Laminated Timber Decks. *Journal of Structural Engineering*, ASCE, Vol. 119, No. 5, May 1993.
7. Taylor, R. J., B. DeV. Batchelor, and K. V. Dalen. *Prestressed Wood Bridges*. Structural Research Report SRR-83-01. Ministry of Transportation and Communications, Ontario, Canada, 1983.
8. Davalos, J. F., and J. T. Brokaw. System Stiffness and Strength Properties of Stress-Laminated Timber Bridge Decks. *ASAE 1992 International Winter Meeting*, Paper 924560, Nashville, Tenn., Dec. 1992.
9. Wolfe, R. W., and R. C. Moody. *Bending Strength of Vertically Glued Laminated Beams With One to Five Plies*. FPL-RP-333. U.S. Forest Products Laboratory, Madison, Wis., 1979.
10. Ritter, M. A. *Timber Bridges: Design, Construction, Inspection, and Maintenance*. USDA Forest Service, 1990.
11. Dickson, B., and H. V. S. GangaRao. Development and Testing of an Experimental Timber T-Beam Bridge. In *Transportation Research Record 1275*, TRB, National Research Council, Washington, D.C., 1990, pp. 67–75.
12. Davalos, J. F., M. Wolcott, B. Dickson, and J. Brokaw. *Quality Assurance and Inspection Manual for Timber Bridges*. Pennsylvania Department of Transportation, 1991.
13. Yadama, V., J. F. Davalos, J. R. Loferski, and S. M. Holzer. Selecting a Gauge Length To Measure Parallel-to-Grain Strain in Southern Pine. *Forest Products Journal*, Vol. 41, No. 10, 1991, pp. 65–68.
14. Loferski, J. R., J. F. Davalos, and V. Yadama. A Laboratory-Built Clip-On Strain Gauge Transducer for Testing Wood. *Forest Products Journal*, Vol. 39, No. 9, 1990, pp. 45–48.
15. Liao, C. L., and J. N. Reddy. *An Incremental Total Lagrangian Formulation for General Anisotropic Shell-Type Structures*. Research Report VPI-E-87.22. Department of Engineering Science and Mechanics, Virginia Polytechnic Institute and State University, Blacksburg, 1987.
16. Davalos, J. F., J. R. Loferski, S. M. Holzer, and V. Yadama. Transverse Isotropy Modeling of 3-D Glulam Timber Beams. *Journal of Materials in Civil Engineering*, ASCE, Vol. 3, No. 2, 1991, pp. 125–139.