Development and Testing of an Experimental Stressed-Timber T-Beam Bridge

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New types of timber bridges are being designed and constructed on the basis of a new joining technique called laminating. At West Virginia University, designers have combined this new technology with a new timber product, laminated veneer lumber, to create a T-beam bridge. The process of designing this bridge required a limited research project because of the lack of design guides. The tests performed are described and many of the results of these tests are included. The bridge was constructed in May 1988, after which a monitoring program was initiated. These tests are also described and results tabulated. Short-term performance of the bridges appears excellent; the long-term performance remains unmeasured. Costs of this bridge are included, as are the authors' recommendations for future bridges of this type.

Timber has been rediscovered as an effective bridge-building material. The stamina and beauty of the early covered timber bridges, many of which remain in service today, are well known. Modern timber design and construction techniques are revitalizing the timber bridge industry and expanding the market for lower-grade structural hardwoods.

In West Virginia, a state blessed with an ample supply of hardwood timber but burdened with many lightly traveled bridges in poor condition, timber is a natural choice for new bridges and rehabilitation projects. A nationwide program, initiated by Congress and administered by the U.S. Department of Agriculture-Forest Service (USDA-FS), has been promoting the use of locally grown timber by funding a series of conferences on timber bridges across the country. Coupled with these conferences is the building of demonstration bridges, which, in most cases, are funded by local or state highway departments.

It was through this program that a modern timber bridge was built in Charleston, West Virginia. The site required a structure capable of spanning 75 ft with a 22-degree skew. The demonstrative function of the project required the use of locally grown timber (primarily hardwoods). The lack of design aids required experimentation, modeling, and field monitoring after construction.

STRESS-LAMINATED TIMBER

Originating in Canada over 10 years ago, stress-laminated timber has proved to be a practical method of combining relatively short and narrow timber planks (laminations) into a strong, resilient, and inexpensive bridge. To create a stressed-timber deck bridge (a bridge consisting only of a thick, laminated timber slab), multiple laminations are squeezed together by high-strength steel bars that pass through predrilled holes in the laminations. The bars are tensioned using a hydraulic jacking system similar to the systems used for posttensioning concrete. The compression within the timber laminations and the frictional resistance to sliding are the only mechanisms required to create a timber "plate," thus eliminating traditional mechanical fasteners. Either steel bearing plates or steel channels are required to distribute the high local compressive stress from the steel bars to the outer timber laminations. The compressive stress is then transmitted to the inner laminations. Figure 1 shows the configuration of a stressed-timber deck.

This method of construction has many advantages over conventional timber construction but unfortunately is limited to fairly short spans, usually less than 45 ft. Although there is great demand for short-span bridges, especially in a mountainous state such as West Virginia, the span requirements of the demonstration bridge exceeded the capacity of stress-laminated decks.

PROPOSED DESIGN: STRESS-LAMINATED TIMBER T-BEAMS

The demand for structurally reliable, long, wide timber components cannot be met by the solid sawn timber available today. This need has given rise to many "reconstituted" timber products, including plywood, glued laminated beams, and stress-laminated decks. Timber materials can be combined to use the material properties to maximum advantage. However, producing high-strength, high-reliability timber products is usually expensive and thus should be done only when these features are most necessary.

The T-beam concept was proposed because the material combination was very efficient. A high-strength, high-reliability stringer combined with a deck composed of lower-cost, lower-quality timber appeared to be the combination that best met the bridge site and demonstration requirements. High-strength steel bars not only compress the deck laminations together, as is done in stressed timber, but also connect the stringers and the deck. The longitudinal orientation of the deck planks should add substantially to the strength of the stringers while also serving as a riding surface.

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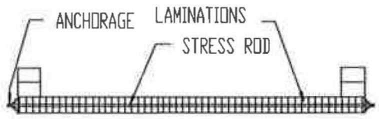


FIGURE 1 Stressed deck.

To span the 75 ft required, a glue-laminated timber product, laminated veneer lumber (LVL), was selected. LVL is a high-quality structural timber material composed of thin (1/10-in. thick), vertical laminations glued together to form beams up to 48 in. deep and 80 ft long. By adding laminations, any desired thickness can be created.

A stress-laminated timber T-beam is formed by using high-strength steel bars to join a stress-laminated deck and an LVL beam. Figure 2 shows the orientation of the components and how the high-strength steel bars tie the timber components together.

Assuming full composite action between the decking and the stringers, an effective flange width equal to half of the clear span between stringers, and a load-sharing fraction of 0.4, an initial design was prepared. A summary of the proposed design follows:

- Center of bearing to center of bearing, 73.25 ft, 17 ft width (one lane), 22-degree right forward skew;
- Seven LVL stringers, each 73.25 ft long, 42 in. deep, and 6 in. wide with 3 in. of camber;
- Northern red oak deck laminations between stringers, 18, each 9 in. deep, 1½ in. wide, and a maximum of 16 ft long;
- Steel bars, ¾-in. diameter, 150 ksi (ultimate stress) on 36-in. center;
- LVL diaphragms on 25-ft centers, each 6 in. wide, 31 in. deep, and 27 in. long;
- Timber guardrail system designed and crash tested by others.

The experimental program described in the following section was undertaken to test the design assumptions.

MODEL TESTING

Because the proposed design was new and untried, a model test program was performed at the Major Units Laboratory of West Virginia University. Time constraints restricted the test program to the most critical areas involving the safety and performance of the design: (a) composite action, (b) load sharing, and (c) creep of the system due to prestress forces.

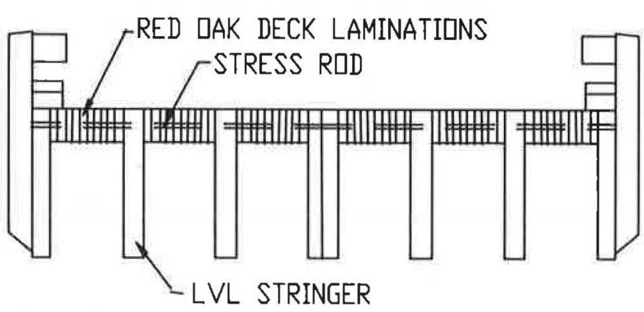


FIGURE 2 T-beam section.

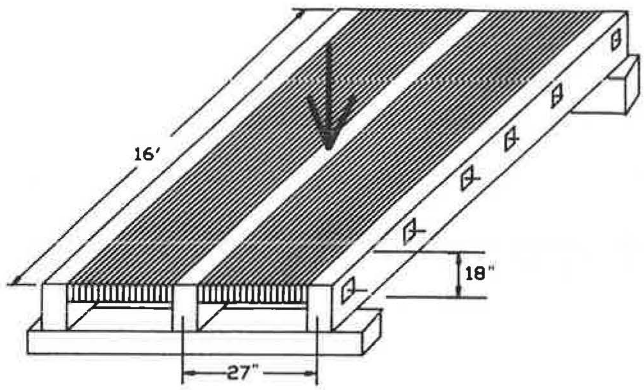


FIGURE 3 Experimental set-up.

A 16-ft-long, three-stringer model (see Figure 3) was built to simulate the full-scale structure. The model was not built to scale but rather was designed to simulate a worst case. The spacing of the model stringers was identical to that of the planned full-size structure, but the deck depth was greatly reduced, butt joints were located at every other lamination, low-quality red oak was used, and the laminations were left unnailed. The model was simply supported at the ends (15 ft 6 in. from center of bearing to center of bearing) and static loading was applied by a hydraulic jack at various locations.

Timber is usually modeled as an orthotropic material with vastly superior bending properties in the longitudinal axis than in the radial or tangential axes. In traditional timber bridge construction, the decking would be placed on top of the stringers and perpendicular to them; thus the stronger bending strength of the decking is perpendicular to the stringers. The traditional configuration gives good load transfer from stringer to stringer but decreases the live-load-carrying capacity of the stringers by adding dead load. However, the T-beam differs dramatically from this usual orientation; the load transfer from deck to stringer may depend only on shear forces between elements of an "articulated plate."

Composite Action

One reason for the choice of the T-beam configuration was to minimize the total amount of timber required. By orienting the deck timber laminations in the same direction as the stringers, the stronger bending strength of the timber decking acts in combination with the stringers to form a composite structure. Thus, the deck laminations serve as both a riding surface and a load-carrying component.

One test for composite action of a system is to measure the strains at the upper and lower surfaces of a stringer. If composite action exists, the strain diagram (see Figure 4) will indicate a neutral axis location higher than that which would be found for a simple rectangular cross section.

Bending tests performed on both the stringer alone and the stringer-deck combination indicated almost complete composite action for the T-beam unit tested. Figure 4 indicates that the experimentally determied neutral axis of the test model nearly coincides with the calculated neutral axis found

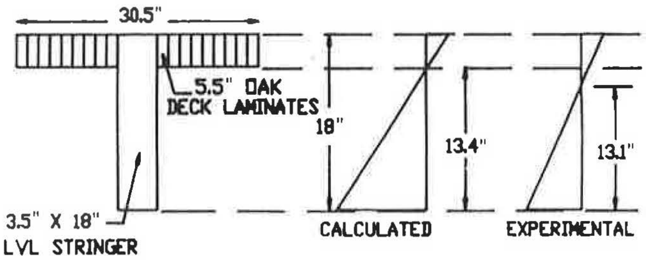


FIGURE 4 Location of neutral axis.

using the geometric properties of a T-beam section with flange width equal to 30.5 in.

Load Sharing

The gain realized by orienting the decking parallel to the stringers has the disadvantage of decreasing the ability of the deck to transfer load to neighboring unloaded stringers. The bridge design code of AASHTO and the Ontario Highway Bridge Design Code (OHBDC) (*1*,*2*) do not offer any guidelines for this type of structure; thus, a test program was necessary.

This stressed deck system can be considered an "articulated" system (i.e., load sharing is accomplished through shear transfer only) or a plate with both shear and moment transfer. Both the moment and shear transfer ability are closely linked to the tension force levels of the posttension system.

The limited test program included three types of testing: (a) the "laminate slip" test (see Figure 5) to establish the normal force required to prevent slipping of laminates relative to neighboring laminates, (b) the "laminate gapping" test (Figure 6) to establish to what degree the prestressing bars prevent the opening of gaps between laminations on the tension side of the deck, and (c) the "load sharing" test in which the deflection of loaded stringers and the simultaneous deformation of neighboring, unloaded stringers were measured (Figure 7). From these tests the load distribution response of the system was established.

The test results accentuated the importance of maintaining posttension force levels. The time limitations of the test program precluded the comprehensive testing required to establish accurate, quantitative information necessary to determine the exact minimum levels of posttension force required, but the trend was quite clear. The observed results correlated well with the OHBDC (*2*), in which a posttension force equal to 12 percent of the allowable compressive strength of the timber laminates is the expected permanent force level. Testing performed at this posttension force level, under loads equal to an HS-20 wheel load, indicated excellent shear transfer (Figure 5) and only minor gapping (Figure 6). Measurements of deflections (Figure 7) indicated that the load transfer to adjacent components decreased as the load increased; at the maximum applied load of 16 kips the fraction of the load carried by the directly loaded stringer was approximately two-thirds of the total load.

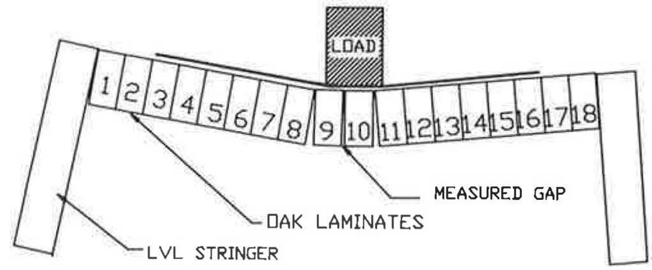


FIGURE 6 Gap measurement test.

Creep from Prestressing

Creep is the time-dependent deformation of a body under constant load. Timber is particularly sensitive to this phenomenon, and because the performance of a stressed timber deck relies so heavily on the force level holding the laminates together, understanding the creep behavior of the system due to prestressing is of vital importance to the structural integrity of prestressed timber bridges.

In stressed timber decks and stressed T-beam systems, the tensioned, high-strength steel bars and bulkhead system exert a constant compressive force on the timber laminations. When the system gradually loses compressive strength over some period of time, the timber loses some amount of width and

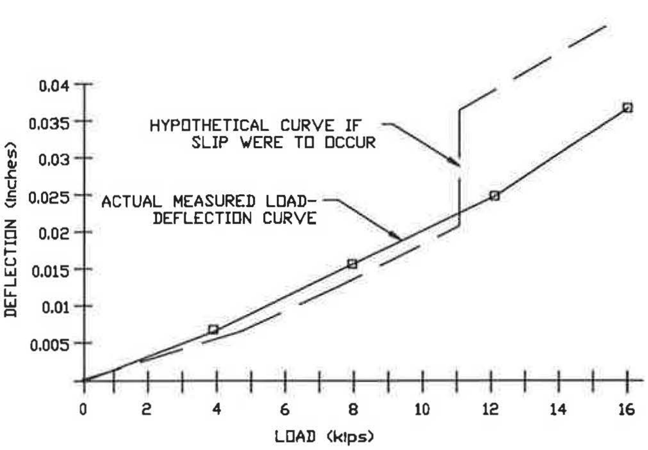


FIGURE 5 Load deflection (of deck planks).

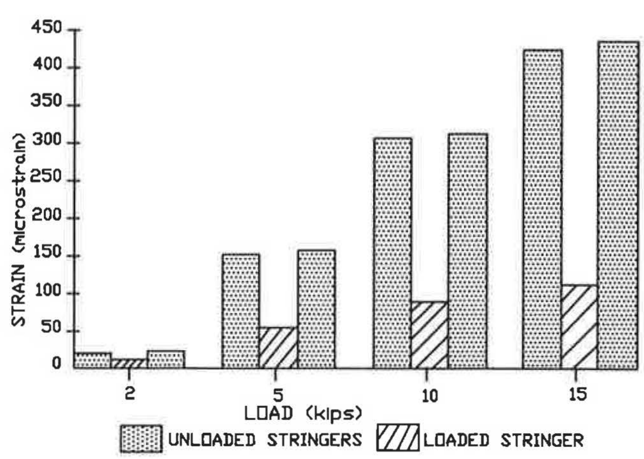


FIGURE 7 Load sharing between stringers.

the force in the steel bars is then reduced. The rate of creep loss is quite high in timber early in the load history (as it is in most materials) and then decreases as time passes.

The OHBDC design methodology includes provisions to compenstate for the loss of compressive strength when timber bridges are designed using several species of softwoods, but data for hardwoods are not included. In this study simple experiments were performed to gauge the creep effects in red oak. In the model, tension bars were loaded to a force level (18 kips) that created a compressive stress of 107 psi within the timber. Load cells were placed on two rod ends between the anchor plates and the bulkhead. Force level readings were taken for a 32-day period.

The data in Figure 8 show a loss of force over 32 days. Unfortunately, the timber deck of the model was found to be losing moisture (drying) as well as compressive strength; thus, it was impossible to determine which losses caused what share of the total. The test results did, however, emphasize the absolute necessity of checking the tensile bar force levels of the steel bars at regular intervals. On the basis of the experimental results, it was decided to follow OHBDC recommended stressing procedures and sequences and to monitor the bar forces on the actual bridge after installation.

ANALYTICAL PROCEDURES

The information accumulated during the model-testing program is only useful if it can be transformed to design values and guidelines. To accomplish this transformation, two mathematical methods were employed: a finite-element computer program and a system of generalized plate equations. Both mathematical procedures varied the ratio of the longitudinal modulus to the transverse modulus of elasticity until the deformations of the mathematical models approximated the deformations of the scale model. The most representative E_L/E_T ratio values then served as the final design material constant.

For this design project, the authors concentrated on two modeling methods: finite-element (FE) methods using the ANSYS computer program and generalized plate equations developed by GangaRao (*3*). Results from both methods correlated quite well with experimental data. The analytical models provided accurate predictions of moments in both the longitudinal and the transverse directions as well as deflections and distribution width.

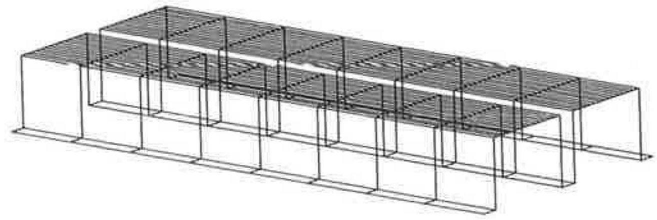


FIGURE 9 Finite-element configuration.

Finite-Element Method

Several attempts were made before an accurate FE model was found. The final model utilizes quadrilateral shell elements with both membrane and bending capabilities. In addition, to create compatibilty between the deck elements and the stringer, a thin "interface" element was introduced. Figure 9 shows the arrangement of model elements.

The model bridge was divided into a total of 800 elements, with each node (four nodes per element) having 6 degrees of freedom. Reduction of the number of simultaneous equations was possible by constraining the displacements in the X- and Y-directions as well as rotation about the Z-axis. Symmetry of the model and support boundary conditions further reduced the number of equations.

The assumed orthotropy of timber requires nine separate elastic constants to calculate the complete status of stresses and deflections. The FE analysis with ANSYS required only four elastic constants because only these values affect the calculation needed. E_L for this model was approximated at 1.7×10^6 psi for the red oak and 2.0×10^6 psi for the LVL. E_T for red oak was varied from $\frac{1}{10}E_L$ to $\frac{1}{25}E_L$, and G_{LT} was set at $0.06E_L$. These values are lower than the elastic constants for the individual members, to compensate for the effects of joints. Figure 10 compares the FE displacement profile with the experimental response and the plate-solution predictions. Although the maximum value of deflection, near the center of the loaded span, is not quite as large as the experimental value (0.27 versus 0.38 in.), the shape of the profile and the

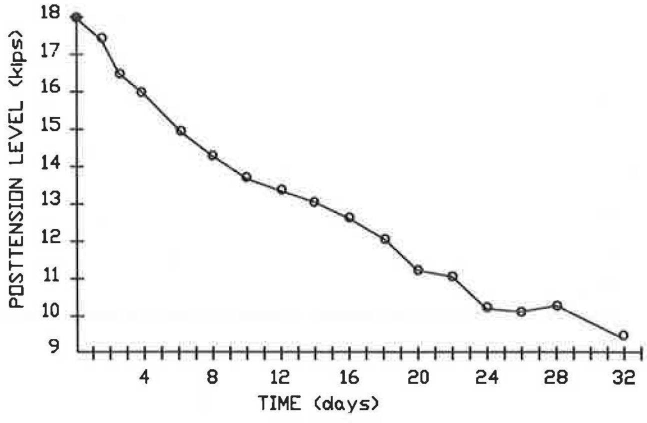


FIGURE 8 Creep losses.

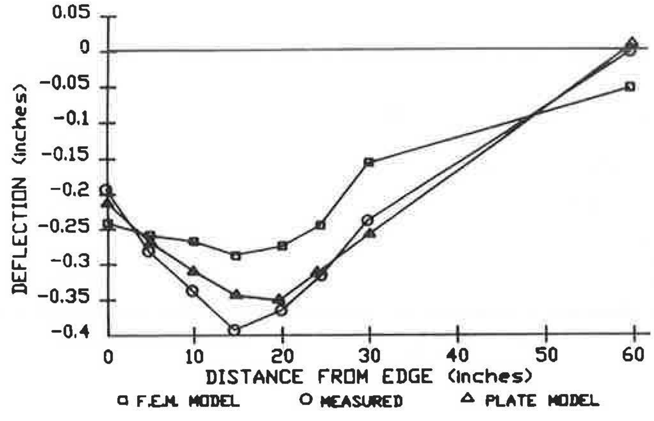


FIGURE 10 Transverse deflected profiles.

deflection of the stringers match very well. Because the deflections of the FE analysis (with $E_T = \frac{1}{20}E_L$) are less than the experimental values, the conclusion can be reached that the transverse stiffness of the deck is less than the estimated stiffness of the individual planks.

Generalized Plate Equations

A system of design equations was developed by GangaRao specifically to analyze steel grid decks, but it can be applied to any orthotropic bridge deck (*3*). These design equations account for most of the parameters affecting the plate behavior of a bridge, such as the aspect ratio of the deck, the orthotropic properties of the deck, loading conditions, number of lanes, and stringer spacing.

Very briefly, the generalized design equations are based on the premise that the bridge can be modeled as an orthotropic plate having two edges simply supported and two edges stiffened by edge beams. The solution form of the governing plate-bending equation is developed for a general load using Fourier and polynominal series; compatibility of deflections of the stringer and decking is assumed to analyze the system. The output of the analytical model includes both moments and deflections at any desired location for any applied load. To test the accuracy of this analytical method, the same input was used as that of the FE model discussed earlier except E_T, which was taken as $\frac{1}{25}E_T$. Because the applied load on the model structure simulated only one rear wheel (the model bridge was not wide enough to accommodate the standard 6-in. spacing of wheel loads), the loading of the analytical model was a combination of symmetric and antisymmetric cases. Superposition of the symmetric and antisymmetric loads produced the deflected profile shown in Figure 10. As in the FE model, the input value of E_T for the red oak in the transverse direction was varied to produce the best correlation with the experimental value. The best correlation of experimental deflections and analytically determined deflections was achieved when a value of $E_T = \frac{1}{25}E_L$ was used.

REVISED DESIGN

The second (and essentially the final) design was based on the test program data and observations and on the information generated by the analytical models. The T-beams were sized by working stress methods using simple beam-bending theory and assuming full composite action. The applied moments were computed for AASHTO loading and then reduced by the load-sharing factor found experimentally (and verified analytically). The live-load moments were not adjusted for impact, because AASHTO does not require this adjustment for timber structures.

For ease of fabrication, transportation, and erection, the bridge was designed to be constructed in two symmetric halves. The stressing system was designed with safety as the primary consideration. High-strength ⅝-in.-diameter steel bars were spaced on 24-in. centers and checked for their ability to prevent a catastrophic failure as well as their capacity to provide the normal force required for the timber laminations to transmit stresses.

Diaphragms, built of LVL timber like the stringers, were located at 18-ft centers. These were designed to provide lateral stability to the T-beam system and also serve as a backup load transfer mechanism in the event of a failure of the stressing system.

The guardrail system was based on a 10-kip crash-tested timber system developed by Wheeler Consolidated. Coal tar creosote preservative was specified for all wood components, and all steel items were galvanized. The final design is as follows:

- Six LVL stringers, each 73.25 ft long, 45 in. deep, and 6 in. wide with 3½ in. of camber;
- One LVL double wide stringer consisting of two stringers, each 73.25 ft long, 45 in. deep, and 4½ in. wide with 3½ in. of camber;
- Northern red oak deck laminations between stringers, 18, each 9 in. deep, 1½ in. wide, and a maximum of 16 ft long;
- Steel bars ⅝-in. diameter, 150 ksi (ultimate stress) on 24-in. centers;
- 8 × 20 steel channel bulkheads with 6 × 6 × ½ in. anchor plates;
- LVL diaphragms on 18-ft centers, each 6 in. wide, 36 in. deep, and 27 in. long;
- Timber guardrail system designed and crash tested by others.

FABRICATION AND ERECTION

Fabrication of the T-beam bridge was by Burke Parson and Bowlby (BPB) of Spencer, West Virginia. LVL stringers, purchased from Trus-Joist, Inc., of Boise, Idaho, were predrilled, preservative treated, and then shipped by rail and truck to the fabrication plant at Spencer. The deck planks, which were also predrilled and preservative treated, were nail laminated to each other and to the stringers and then temporarily posttensioned with mild steel rods in every second hole. Three 6-in.-wide stringers and one 4½-in.-wide stringer were assembled with the required decking to create half of the bridge. The process was repeated to create the other half, and the two halves were trucked to the bridge site.

At the bridge site, the abutments from the previous structure were repaired and modified to accept the new timber T-beams. A 140-ton crane and an 80-ton crane were located on either side of the span; the 140-ton crane lifted one half of the bridge from the truck trailer and placed it in position on the abutments. The smaller crane was used as an emergency backup in the event that the long reach required of the primary crane caused it to overturn. Although the smaller crane proved to be unnecessary, it was useful in positioning the end of the bridge half.

After the second half-section was craned to its position on the abutments, full-length, high-strength rods were inserted; then the temporary rods were removed and the entire structure was posttensioned. Installation of guardrails and concrete abutment backwalls was followed by the restressing operations; paving the surface with asphalitc concrete completed the job.

FIGURE 11 Completed structure.

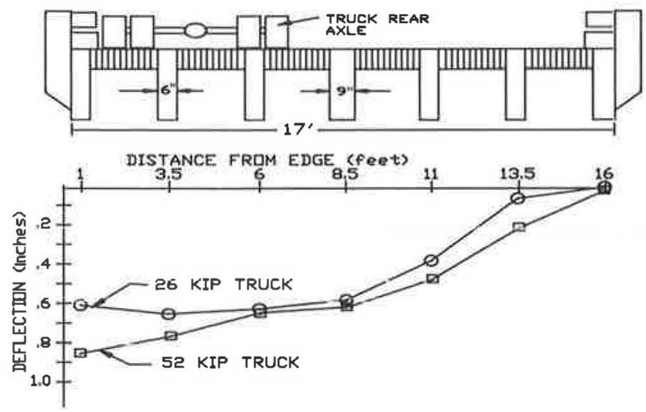


FIGURE 12 Transverse deflection profiles.

MONITORING PROGRAM

Figure 11 shows the completed timber T-beam structure. The bridge has been in service for 1 year and is apparently functioning well. It is, however, still an experimental structure and as such requires careful monitoring.

A short-term monitoring program (July 1, 1988, to December 30, 1988) consisted of six live-load tests and measurements of several of the bridge's responses. The live-load tests were designed to determine (a) the load deflection response of the stringers and the deck, (b) the stresses in the exterior stringer, (c) slip between laminates, and (d) skew effects. Other measured performance characteristics were stress level within the posttension system, dead-load deflection changes with time (creep), and moisture content fluctuations. In addition to these measurements and tests, simple visual observations and photographs were compiled to record the weathering of the exposed surfaces and general condition of the structure. The most recent visit to test posttension levels was made in May 1989.

MONITORING RESULTS

The short-term test program has provided a large body of data both for analysis of the bridge performance and as a datum from which long-term behavior can be measured. In addition to the data accumulated, valuable experience was gained in field monitoring techniques and in the unique problems of on-site experimentations. The test program was not completely successful, however. Portions of the results from several of the tests were clearly erroneous; the cause of these errors has been determined and changes will be made in future testing to eliminate these mistakes.

Stringer Deflection

Four separate load tests were performed in which a loaded truck was parked with its wheels on an outside stringer. Figure 12 shows transverse deflected profiles for two of these tests. There are some inconsistencies in the data, which show unreasonably lower deflections immediately below wheel loads. For example, the deflection 1 ft from the edge (see Figure 12) is directly beneath one of the wheel loads and should be greater than the deflection 3.5 ft from the edge. Taking deflection readings with a transit from the uneven top surface of the deck, particularly at the location of the test vehicle, was difficult and led to inaccuracies.

Maximum deflections generally occurred at the loaded exterior stringer, as is expected. The maximum deflection of 0.84 in. was due to the largest load of the test series (52,100 lb), which is approximately 83 percent of the AASHTO HS-20 truck load. The maximum deflection is quite small (equal to *L*/865) and much lower than was expected for a timber highway bridge. Part of the unforeseen stiffness is undoubtedly due to the double center stringer (9 in. thick), which was added to allow the bridge to be constructed in two symmetric halves. The large (8- × 12-in.) wheel rail and guardrail probably also contribute to the stiffness of the bridge.

Deck Deflection

Results of two deck deflection tests are shown in Figure 13. The data indicate a maximum deflection in both tests of only 0.02 in. for wheel loads of 8 and 10 kips.

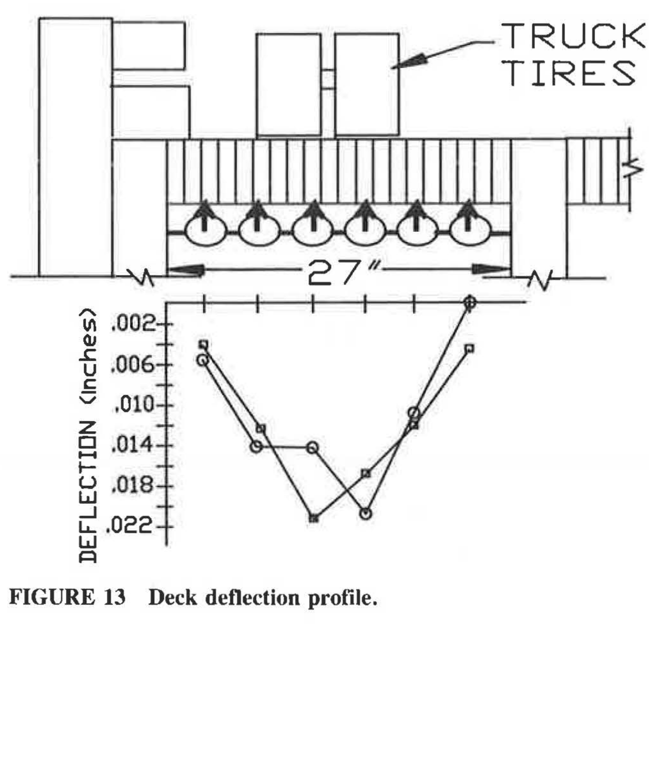


FIGURE 13 Deck deflection profile.

One benefit of this high deck stiffness is the ability of the deck to transfer the live load to the stringers more directly and to then transfer load from one stringer to another, providing a more even load distribution. A second benefit is that a small deformation is less likely to cause cracking of an overlay (asphaltic concrete). However, this degree of stiffness is probably more than is necessary and, of course, comes at some increased cost.

Stringer Stress Level

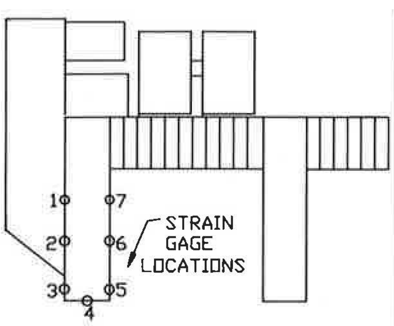
Strain measurement in the exterior stringer was one of the most difficult procedures to perform in the field and, unfortunately, the results cannot be considered very reliable. Figure 14 shows the data gathered for two live-load tests and includes the maximum calculated stress value. The readings show a strain increase in the gauges closer to the bottom of the stringer, as was expected, but in both tests the No. 4 gauge, located at the extreme lower face, exhibited a decrease in strain.

Despite the inconsistencies of the test results, the low strain measured by the six other gauges is indicative of a reasonable stress level in the most heavily loaded stringer. Simple calculations to convert the maximum measured strain to stress indicate a stress level of about 700 psi. The approximate load on that stringer was 15,000 lb. The problem of bonding strain gauges to creosote-treated timber can be overcome by use of clip-on strain gauges. A reliable, economical, and reusable strain transducer for field measurements [developed by Laferski et al. (*4*)] will be used in future monitoring projects.

Laminate Slip

The same testing done for deck load deflections was also used to locate any laminate slip. Although the graph in Figure 13 shows abrupt changes in deflection, the scale of the graph is highly distorted, with very small divisions on the ordinate compared with divisions on the abscissa. A graph with identical *X*- and *Y*-scales would show only a very slight curvature with no discontinuities.

Resistance to slip is dependent on the maintenance of a minimum posttension level. There is no evidence of laminate slip from either load tests or visual observations. After the load tests, all deflected deck planks returned to their unloaded positions.



MEASURED STRAIN

TEST 1		TEST 2	
#1	132 ue	#1	113 ue
#2	168 ue	#2	
#3	245 ue	#3	225 ue
#4	181 ue	#4	183 ue
#5	219 ue	#5	360 ue
#6	168 ue	#6	
#7	26 ue	#7	282 ue

MAXIMUM STRESS = 720psi

FIGURE 14 Strain measurements.

As a field check, the tension in one of the high-strength steel bars was released; the bar was able to move freely through the laminations indicating that the original hole clearance was intact and that no slippage had occurred.

Skew Effects

Due to the skewed bridge geometry, the possibility of an uplift force exists when a diagonally opposite corner is loaded. To test for this possibility, a truck load was placed at one corner and a dial gauge located below the opposite corner was monitored as the truck crossed the bridge. If the bridge skew had been an influence on the deflection response, an initial upward deflection would have been apparent. However, the dial gauge showed an immediate downward deflection of 0.002 in. when load was applied at the corner diagonally opposite. This deflection remained consistent as the truck traveled the length of the bridge; thus, no evidence of uplift was found.

Posttension Levels

Because the tension level in the posttension system is such an important factor in the response of the bridge, this testing was perhaps the most important of the monitoring program. Unfortunately, problems with equipment used to measure the tension level during the initial testing period reduced the amount of data. However, the amount of data gathered during the initial 6-month monitoring period combined with the data collected 1 year after the installation of the structure were sufficient to establish the prestressing system behavior.

Erection of the structure on May 7 and 8 was followed by an initial stressing on May 10, 1988. Information from the contractor indicated an initial tensile force of approximately 32 kips in the high-strengh steel bars. The second and third stressings were performed on May 18 and July 8, 1988, at which time the tensile force applied was again about 32 kips. Force levels from one load cell on one bar were measured from October to December 1988. In addition, an average tensile force from 15 bars tested in May 1989 was estimated by tensioning the rods until the anchor nuts were free to turn, rather than by using a load cell.

The early force levels from the single load cell, installed in October 1988, 5 months after the bridge was first stressed, are lower than expected (see Figure 15). The average force measured on May 1989 was 21.8 kips, or 67 percent of the jacking force. An extended period of rain before the test date may have caused some moisture-induced swelling and, consequently, an increased tension force level in the rods. If this was the case, another testing of the rods after a dry period should show a reduced force level. This procedure was tentatively planned for late summer 1989.

The long-term ability of the system to maintain the necessary tension level is a crucial element in the structural and economic performance of the bridge. Should the tension level fall below an acceptable level, the rods can be retensioned, but the intention of the design is to reduce this maintenance expense to a minimum. The structural consequences of inadequate tension level are serious. Complete loss of tension in one rod should not cause catastrophic failure (a safety feature

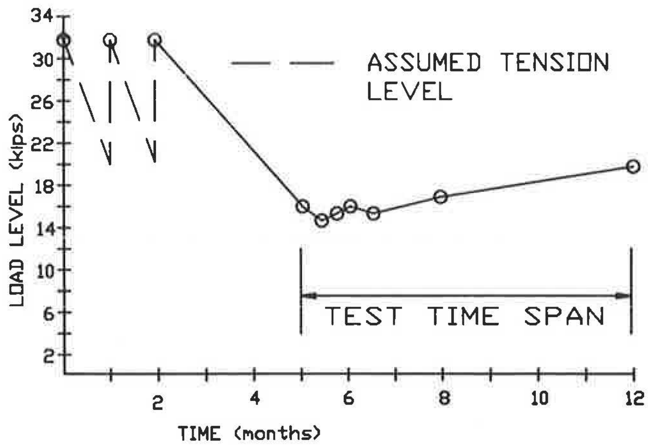


FIGURE 15 Posttension system losses.

of the design), but the ability to distribute load to the laminates and stringers can be seriously decreased.

Creep Deflection

Data for the creep deflection test were accumulated over the 6-month period of the monitoring program. Six months is a relatively short time span in the intended life of the structure, and there is, at this time, only slight evidence of loss of camber through creep losses.

Figure 16 indicates the loss of 0.03 ft (0.36 in.) of midspan elevation over 5 months. This is the maximum loss of the seven locations measured (top of each stringer at the bridge center line); the average value is 0.02 ft (0.24 in.). As in the previous tests, these data are not completely consistent, but the trend is well established. All of the stringers showed some loss of elevation from the first to the last reading. Each of the stringers maintained an upward camber, but the readings reveal that the bridge has an unequal camber when viewed across the section; that is, the center-line elevation of the center stringers is lower than that of the edge stringers.

The relatively short time span of this creep test prevents an accurate prediction of the rate of creep loss over the life span of the structure. Most materials exhibit a more rapid rate of loss early in the life span, which then levels off with time. It remains to be seen if this bridge will behave similarly.

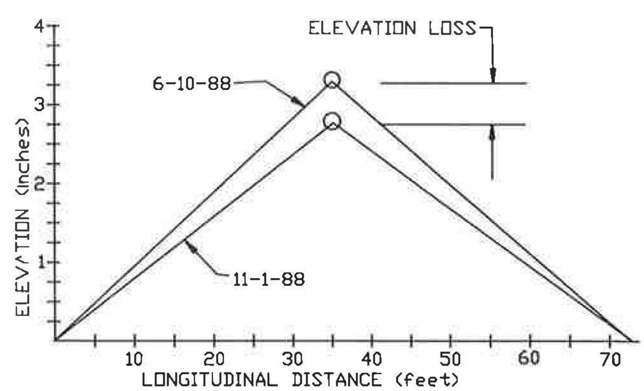


FIGURE 16 Creep deflection.

Moisture Levels

The timber and LVL materials used in the bridge were kiln dried to a moisture content of 19 and 8 percent, respectively, and then creosote treated. Exposure to the environment has increased the moisture content, as expected, with the greatest effect on the riding surface of the deck planks and LVL stringers.

At the time of this test, the timber deck surface was exposed to the weather; since that time, the surface has been covered with asphaltic concrete. Moisture levels were tested on the guardrail, top surface of decking, top surface of stringers, inside surfaces of stringers, and underside of deck planks. The variation of moisture levels was from 18 to nearly 40 percent, with the greatest variation in the LVL stringers. The highest readings were found on the upper surface of the stringers, which were subjected to the most adverse weather conditions. The moisture level in the oak decking was more consistent, averaging 22 percent. There did not appear to be a significant change in moisture levels from one test to the next, but the time span between tests was inadequate to show any relationship.

Weathering and General Condition

Visual observation showed very little noticeable weathering of the deck or stringers. However, the large timbers that make up the curb and the guardrail checked and split as they dried; some of these splits are extensive and much deeper than the penetration of the creosote.

Only minimal corrosion of the hardware has been observed. This insignificant corrosion was found on the high-strength bars where the couplers used in the stressing operation rubbed off some of the galvanizing material. A comprehensive inspection was not done, but the one rod that was removed and reinstalled was in excellent condition.

The galvanized steel pipe and neoprene washers used as a weatherproof covering for the anchoring hardware appear to be effective when installed correctly. Unfortunately, a large number of the installations were done incorrectly, and moisture has entered the covering devices. No serious corrosion was noted, however, even on the poorly protected anchors. These covering devices can do more harm than good, because they may retain moisture around the anchoring hardware rather than prevent its entrance.

Visual observation of the two exterior stringers indicated them to be off the vertical (not perpendicular to the abutments) when viewed from the abutments. By measuring this misalignment with a carpenter's 4-ft level, the maximum distance from the vertical was found to be 2¼ in. Discussions with the contractor indicate that the cause of this substantial distortion was either the erection procedure, in which one half of the bridge was "pulled" a small distance to mate with the previously anchored first half, or the retensioning operations, in which the top portion of the structure was squeezed together while the lower portion remained stationary.

COSTS

Information from the West Virginia Department of Highways shows the cost of the 73-ft timber T-beam bridge to be as follows:

Item	*Cost ($)*
Timber contractor, fabrication and delivery	78,800.00
Crane rental	10,125.00
Labor for erection	14,400.00
Equipment for erection	2,400.00
Materials for abutment alterations	1,185.00
Total	106,910.00

Although a breakdown of the contractor costs is not available, a large portion of the $78,800 was for the LVL beams and their transportation to the fabrication site. Labor and equipment for erection costs are for a 16-day period.

SUMMARY AND RECOMMENDATIONS

Results of the field monitoring program indicate that the stressed-timber T-beam bridge is performing very well. Most important, the force level in the high-strength steel bars is being maintained at a safe level. The deck deflection is minor and the stringer deflection appears to be acceptable. Weathering of the decking and stringers is, at this time, negligible.

From a performance perspective, the dead-load deflection (creep) is the most apparent problem and may not be significant over the long term. This will require future monitoring (as will the posttension force level). From a construction and fabrication perspective, the misalignment (not perpendicular to the abutment) of the stringers and the less cambered central stringer are serious problems that must be addressed in future T-beam projects. Several other minor construction problems can be easily rectified for future bridges.

The problem of high costs is not so easily resolved. This particular structure had very high crane rental costs and transportation charges. LVL is an excellent product with very good performance characteristics, but it is an expensive component. Much of the LVL costs were for transportation; if this cost could be reduced, the use of this product would be more cost-competitive.

The design of the structure also played a large role in the high cost. Because there were no guidelines to follow, the design had to be very conservative to guarantee public safety. There is little doubt that with the information now available, a more economical structure can be designed.

Recommendations for designers and fabricators of timber T-beam bridges include the following:

1. Use for shorter spans (40 to 60 ft). Although there is no performance problem, the high cost of transporting long components to treatment plants and then to the fabricator becomes prohibitive. Also, costs for large cranes are much higher than those for smaller cranes.
2. Increase the spacing distance between stringers to about 36 in. The deck stiffness is sufficient to transfer the live load more than the 27 in. span of this structure. The stringer capacity is not being utilized as fully as possible by the 27-in. spacing.
3. Choose stringer and deck material by geographic location as well as by load capacity and preservative treatability. Other possible materials (glued laminated timber, for example) may be less expensive and may be located closer to treatment facilities and fabricators.
4. Use a different rigging method when craning the bridge halves. This erection method used spreader beams to support the bridge halves; an "eye hook" rigging method would allow the crane operator to place the bridge segment exactly where required.
5. Do not anchor bridge halves to abutments until after second stressing sequence. Once this second stressing is complete, almost all of the compression of the timber deck will have taken place and there should be no more appreciable distortion.

With some additional monitoring, analytical work, and cost analysis, the timber T-beam can become a successful alternative to conventional bridge construction. A combination of low-cost local timber for the decking, high-reliability manufactured timber products for the stringers, and precision fabrication and erection techniques should provide a safe, long-lasting, and economical structure.

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