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Field Performance of Timber Bridges

8. Lynches Woods Park Stress-Laminated Deck Bridge

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Michael A. Ritter
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Abstract

The Lynches Woods Park bridge was constructed during the summer of 1990 in Newberry, South Carolina. It is a single-span, single-lane, stress-laminated deck superstructure that measures approximately 30 ft long, 16 ft wide, and 14 in. deep. The bridge is unique in that is one of the first known stress-laminated deck bridges to be constructed of Southern Pine lumber treated with chromated copper arsenate. The performance of the bridge was continuously monitored for approximately 3 years, beginning 10 months after installation. Performance monitoring involved gathering and analyzing data relative to the wood moisture content, force level in the stressing bars, and behavior under static-load conditions. In addition, comprehensive visual inspections were conducted to assess the overall structure condition. Based on the field evaluations, the bridge is performing well with no structural or serviceability deficiencies.

Keywords: Timber, bridge, wood, stress laminated

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Introduction

In 1988, the U.S. Congress passed legislation known as the Timber Bridge Initiative (TBI). The objective of this legislation was to establish a national program to provide effective and efficient utilization of wood as a structural material for highway bridges. Responsibility for the development, implementation, and administration of the timber bridge program was assigned to the USDA Forest Service. Within the program, the Forest Service established three primary program areas: demonstration bridges, technology transfer, and research. The demonstration bridge program, which is administered by the Forest Service Timber Bridge Information Resource Center (TBIRC) in Morgantown, West Virginia, provides competitive matching funds to local governments to demonstrate timber bridge technology through the construction of demonstration bridges (USDA 1995). The TBIRC also maintains a technology transfer program to provide assistance and state-of-the-art information related to timber bridges. One objective of these programs is to encourage the use of new or previously underutilized wood products, bridge designs, and design applications.

Responsibility for the research portion of the TBI program was assigned to the USDA Forest Service, Forest Products Laboratory (FPL), a national wood utilization research facility. The FPL currently conducts research in numerous areas related to timber bridges and other wood transportation structures (Ritter and others 1994). As part of this research program, FPL has implemented an extensive program to assist local governments in evaluating the field performance of timber bridges, many of which employ design innovations or materials that have not been previously evaluated. Through this program, FPL is able to collect, analyze, and distribute information on the field performance of timber bridges, providing a basis for validating or revising design criteria and further improving efficiency and economy in bridge design, fabrication, and construction.

This paper is the eighth in a series that documents the field performance of timber bridges included in the FPL timber bridge monitoring program. It addresses the field performance of the Lynches Woods Park bridge built in Newberry, South Carolina, in 1990. The bridge was constructed through the TBI demonstration bridge program and is a single-span, single-lane, stress-laminated deck superstructure that measures approximately 30 ft long, 16 ft wide, and 14 in. deep. (See Table 1 for metric conversion factors.) Constructed of Southern Pine lumber, the Lynches Woods Park bridge is unique because it is one of the first known stress-laminated deck bridges to be constructed of lumber treated with chromated copper arsenate (CCA). An information sheet on the Lynches Woods Park bridge is provided in the Appendix.

Table 1—Factors for converting English units of measurement to SI units

English unit	Conversion factor	SI unit
acre	0.0040	square meter (m^2)
inch (in.)	25.4	millimeter (mm)
foot (ft)	0.3048	meter (m)
square foot (ft^2)	0.09	square meter (m^2)
pound (lb)	4.448	newton (N)
lb/in^2 (stress)	6,894	pascal (Pa)
mile	0.0016	meter (m)

Background

The bridge is located within the 268-acre Lynches Woods Park in the City of Newberry, South Carolina (Fig. 1). The bridge is on a single-lane, unpaved roadway where it crosses the Rocky Branch river, approximately 2.5 miles from the park entrance. This roadway provides the only vehicle access to the park and consists primarily of passenger cars and maintenance vehicles. The traffic volume across the bridge is estimated at 20 vehicles per day.

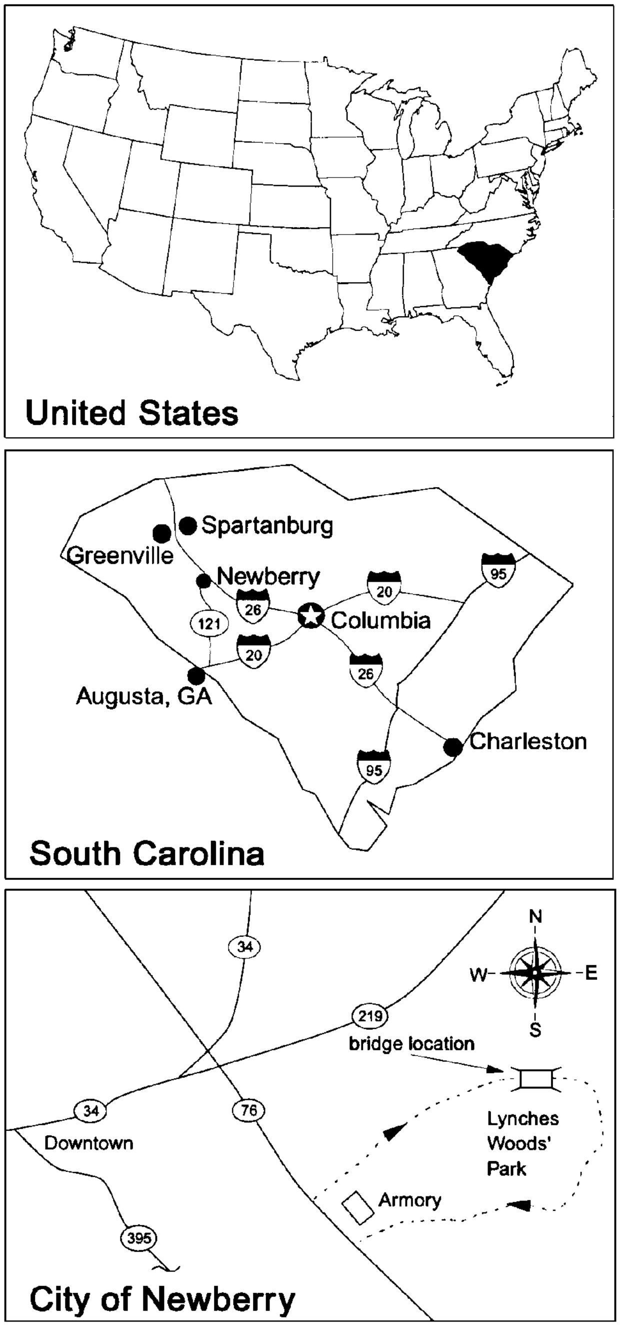


Figure 1—Location maps for Lynches Woods Park bridge.

Before replacement in 1990, the crossing at the Rocky Branch river consisted of a 20-ft-long, nail-laminated lumber deck, supported by a timber substructure. During the spring of 1988, the Rocky Branch river flooded and the bridge superstructure was washed out. Because of this washout and extensive damage to the timber abutments, a temporary low water crossing was installed until a new bridge could be designed and constructed (Fig. 2). Based on an evaluation of alternatives for the replacement structure, it was determined

Figure 2—Prior to bridge construction, the Lynches Woods Park bridge site was a low-water crossing.

that the new bridge would be a timber deck superstructure, supported by concrete abutments. It was further determined that the length and elevation of the replacement structure would be increased from those of the original structure to provide increased hydraulic capacity.

Through a cooperative effort between the Crossroads of History Resource, Conservation, and Development Council and Newberry County, a proposal was submitted to the USDA Forest Service for partial funding of the Lynches Woods Park replacement as a demonstration bridge under the Timber Bridge Initiative (USDA 1995). The project proposed a stress-laminated deck timber bridge constructed of Southern Pine lumber treated with CCA. In 1989, the project proposal was funded through the TBIRC, and plans for the design and construction of the Lynches Woods Park project were finalized.

In March 1991, approximately 8 months after the bridge was installed, the Newberry County Highway Department contacted FPL for assistance in monitoring the performance of the Lynches Woods Park bridge. Subsequently, FPL and the Newberry County Highway Department mutually developed a cooperative research and development agreement to complete structural monitoring of the bridge as part of the ongoing timber bridge monitoring program at FPL.

Objective and Scope

The objective of this project was to evaluate the field performance of the Lynches Woods Park bridge for a minimum of 2 years, beginning 10 months after construction. The scope of the project included data collection and analysis related to wood moisture content, stressing bar force, static truck loading, and general structure performance. The results of this project will be considered with similar monitoring projects in an effort to improve design and construction methods for future stress-laminated timber bridges.

Design, Construction, and Cost

The design and construction phases of this project involved a mutual effort among several agencies and individuals. The following presents an overview of the design, construction, and costs of the Lynches Woods Park bridge.

Design

Design of the Lynches Woods Park bridge was completed by the Newberry County Highway Department personnel with assistance from the engineering staff of the Francis Marion and Sumter National Forests. With the exception of those design aspects dealing with stress laminating, the bridge superstructure was designed according to American Association of State and Highway Transportation Officials (AASHTO) *Standard Specifications for Highway Bridges* for AASHTO HS20–44 loading requirements (AASHTO 1989). Specific requirements for stress laminating were based on a Forest Service, Eastern Region, standard stress-laminated bridge design.

The design geometry of the Lynches Woods Park bridge provided for a single-span, simply supported stress-laminated lumber deck, 30 ft long, 16 ft wide, and 14 in. deep (Fig. 3). Design calculations were based on a 29-ft center-to-center of bearing span length and a 14.3-ft clear roadway width. The lumber laminations for the stress-laminated deck were designated as nominal 4-in.-thick Southern Pine lumber, treated with CCA in accordance with AWPA C14 (AWPA 1989). The availability of 4- by 14-in. Southern Pine lumber at the full-span length was cost prohibitive. Thus, the deck was designed with butt joints to reduce the required lamination length. Butt joints were spaced so that only one would occur in every four adjacent laminations transversely and at 4-ft intervals in the longitudinal direction (Ritter 1990).

Design values for the Southern Pine laminations were based on the National Design Specification for Wood Construction for lumber visually graded No. 1 Dense Structural, in accordance with the Southern Pine Inspection Bureau (SPIB) grading rules (AFPA 1986, 1988). Tabulated design values for the Southern Pine laminations were 1,550 lb/in^2 for bending, 1,600,000 lb/in^2 for modulus of elasticity (MOE), and 440 lb/in^2 for compression perpendicular to grain.

The stress-laminating system for the Lynches Woods Park bridge utilized threaded steel bars placed through the laminations and tensioned to provide the required interlaminar compression and load transfer between the individual laminations. The design specified the use of 1-in.-diameter, high strength, galvanized steel bars with an ultimate strength of 150,000 lb/in^2, in accordance with the requirements of ASTM A722 (ASTM 1988). The bars were spaced 4 ft on-center, beginning 1 ft from the bridge ends. To prevent chemical interaction between the wood preservative and the bar galvanizing, the bars were encased in polyvinyl chloride (PVC) tubing. The design force for the bars was 72,000 lb, which resulted in an interlaminar compression of approximately 107 lb/in^2. Bar anchorage at the edges of the bridge was a discrete plate anchorage system, consisting of steel bearing and anchorage plates (Fig. 4).

Design of the bridge railing and curb system was based on AASHTO static-load requirements (AASHTO 1989). All rail and curb members were CCA-treated, Southern Pine sawn lumber, visually graded No. 2 Dense Structural (AFPA 1986, 1988). Rail members were specified as nominal 6 by 12 in., with a splice at the bridge midspan. The railposts were specified as nominal 8 by 8 in. and were spaced at a maximum of 6 ft on-center. The railing was offset from the posts with nominal 4-in.-thick offset blocks. All rail and curb members were surfaced on four sides (S4S) to ensure dimensional tolerance and uniform contact between adjacent members during construction.

A wearing surface was not specified for the Lynches Woods Park bridge because of the anticipated low traffic volume.

Construction

Construction of the Lynches Woods Park Bridge was completed in the summer of 1990 under the supervision of personnel from Newberry County and the Francis and Marion National Forests. Labor for bridge construction was provided by a local state prison. The construction process began with the removal of the existing low water crossing and the erection of reinforced-concrete abutments (Fig. 5). Lumber components for the bridge superstructure were then delivered to the bridge site on a flatbed truck. Assembly of the superstructure was completed in-place, with temporary scaffolding support between the abutments (Fig. 6). With scaffolding providing full support at butt joint locations, the individual laminations were hand-placed by the work crew and spiked together to ensure proper alignment of the bar holes. The PVC tubing was then placed through prebored bar holes to facilitate bar insertion and provide bar corrosion protection. Full-width stressing bars were inserted through the PVC tubing, and anchorage plates and nuts were attached.

The stressing bars for the Lynches Woods Park bridge were tensioned on four separate occasions. At each bar tensioning, the bars were fully tensioned to the design force of 72,000 lb using a single hydraulic jack. The initial bar tensioning was completed just prior to removal of the construction scaffolding by tensioning the first bar at one bridge end, then sequentially tensioning each successive bar along the bridge length. The bars were then retensioned several times using the same procedure until a uniform force was achieved in all bars. On the three subsequent bar tensionings, conducted at 1, 4, and 8 weeks after the initial tensioning, a similar bar tensioning procedure was followed (Fig. 7).

After the second bar tensioning, the deck was anchored to the concrete substructure, and the rail and curb system was installed. The deck-to-substructure connection was made with steel angle brackets that were attached to the deck underside and bolted to the concrete abutment walls (Fig. 8). The curb system was assembled for alignment and then bolted to the deck through field-drilled holes that were treated with wood preservative (Fig. 9). After the curb was in place, the railing system was attached and installation was completed. The completed Lynches Woods Park bridge is shown in Figure 10.

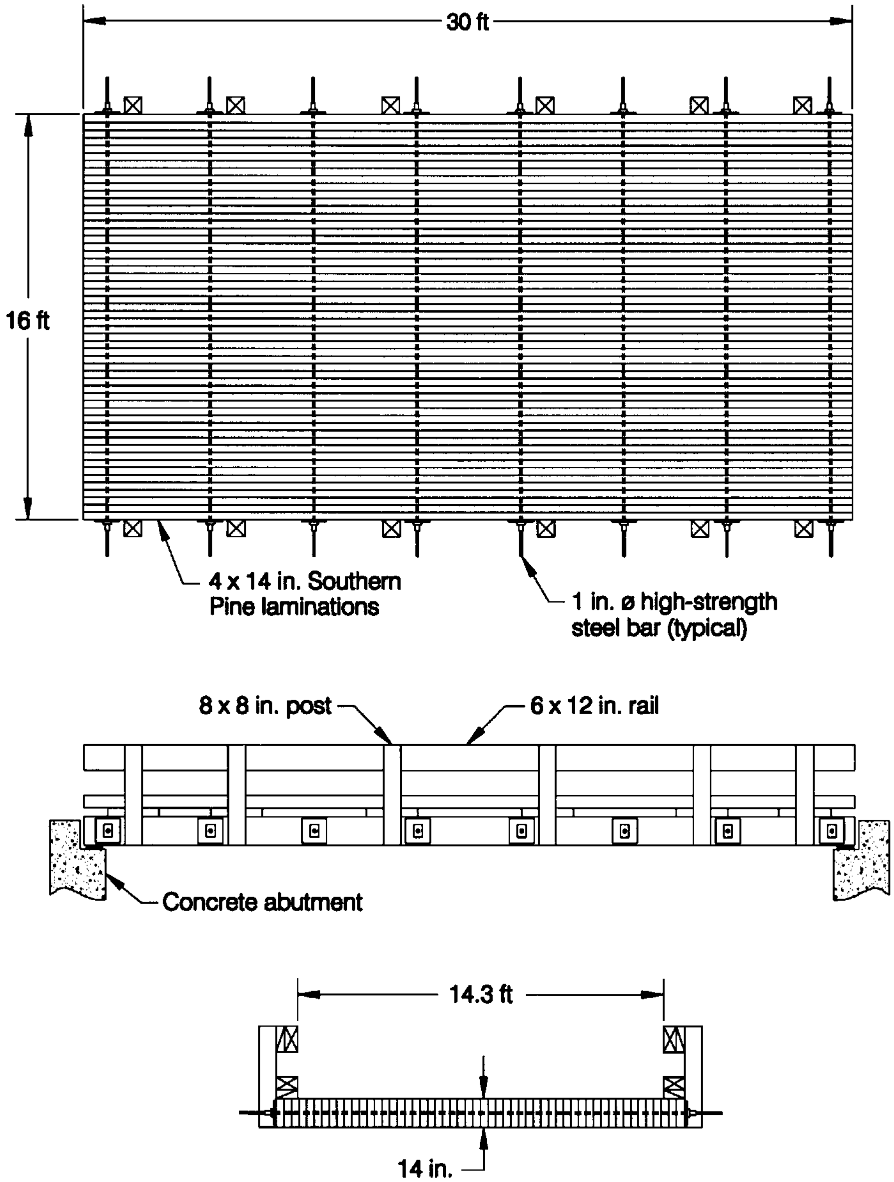


Figure 3—Design configuration of the Lynches Woods Park bridge.

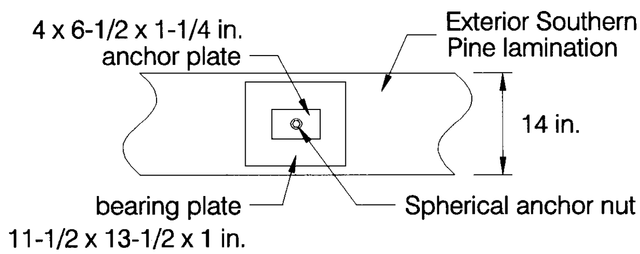


Figure 4—Design configuration of the stressing bar anchorage.

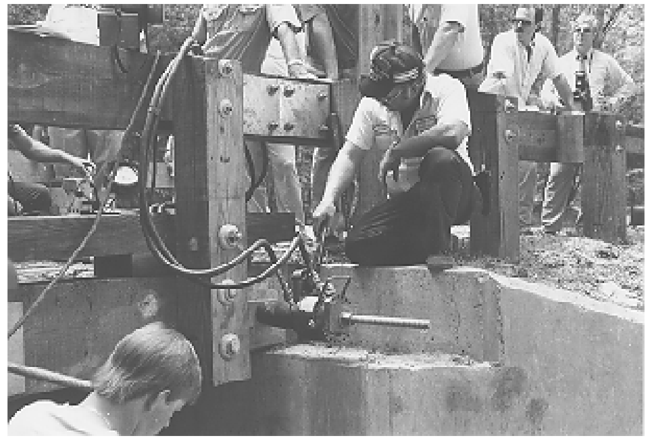


Figure 7—Bar tensioning, performed 8 weeks after construction, using a single hydraulic jack.

Figure 5—Installation of reinforced-concrete abutments.

Figure 8—Attachment of the superstructure to the substructure by bolting the deck to a steel angle that is bolted to the side of the concrete substructure backwall.

Figure 6—Temporary scaffolding was placed during construction to support laminations before the deck was stress laminated.

Figure 9—Attachment of the sawn-lumber curb to the bridge deck.

Figure 10—The completed Lynches Woods Park bridge, approximately 14 months after installation.

Cost

The total fabrication and construction cost for the Lynches Woods Park bridge superstructure including the railing system was $19,720. Based on a total deck surface area of 480 ft^2, the cost was approximately $41/$ft^2$.

Evaluation Methodology

To evaluate the structural performance of the Lynches Woods Park bridge, Newberry County personnel contacted FPL for assistance. Through mutual agreement, a 2-year monitoring plan was developed by the FPL and implemented through a cooperative research and development agreement with Newberry County. The plan called for monitoring several key performance indicators, such as moisture content, bar force, static-load behavior, and general structure condition. At the initiation of field monitoring, approximately 10 months after the bridge was constructed, an FPL representative visited the bridge site to install instrumentation and train Newberry County personnel in data collection procedures for moisture content and bar force measurements. Load tests and general condition assessments were completed by FPL personnel during site visits. The evaluation methodology utilized procedures and equipment previously developed by Ritter and others (1991) and is discussed in the following sections. In addition, information recorded by Newberry County personnel at bridge installation was used to augment FPL monitoring data.

Moisture Content

Moisture content readings were collected from the deck superstructure on a monthly basis throughout the monitoring period. The measurements were taken with an electrical-resistance meter, in accordance with ASTM standard requirements (ASTM 1990). The moisture content readings were obtained at a series of locations on the underside of the deck at a pin penetration of approximately 2 in. (Fig. 11). Field readings were adjusted for wood species and temperature to determine the actual moisture content values (Forintek 1984). In addition, several core samples were collected at the end of the monitoring period to measure the moisture content by the ovendry method (ASTM 1992).

Bar Force

Bar force data were obtained on a biweekly basis throughout the monitoring period. Measurements were obtained from load cells installed on two of the eight bars using a portable strain indicator (Fig. 12). The load cell strain readings were converted into equivalent units of bar force, based on load cell calibrations completed in the laboratory prior to bridge monitoring. At the end of the monitoring period, the accuracy of the load cells was verified with hydraulic force checks at the bridge site and load cell recalibration in the laboratory (Ritter and others 1991).

Figure 11—Electrical-resistance moisture readings were taken from the underside of the deck at a depth of approximately 2 in.

Figure 12—Load cell readings were obtained with a portable strain indicator.

Behavior Under Static Load

Static-load tests of the Lynches Woods Park Bridge were conducted at the beginning and ending of the monitoring period to determine the bridge response under static-loading conditions. In addition, an analytical assessment was completed to evaluate the bridge response using computer modeling.

Load Testing

Static-load testing of the Lynches Woods Park bridge consisted of positioning a loaded dump truck on the bridge and measuring the resulting deflections at a series of locations at midspan and near the abutments. For each load test, the vehicle was placed in three transverse positions (Fig. 13). Longitudinal vehicle placement was based on the truck configuration and was different for each load test. Measurements were obtained by suspending calibrated rules from the deck underside and reading their relative movement with a

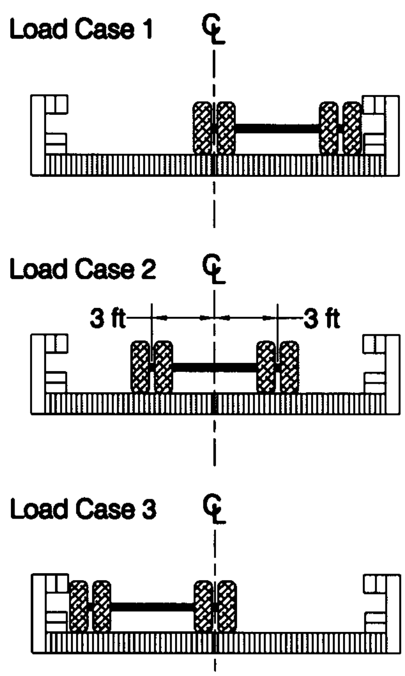


Figure 13—The three transverse load cases used for static-load tests.

surveying level. Deflection measurements were obtained prior to testing (unloaded), after placement of the test vehicle (loaded), and at the conclusion of testing (unloaded).

Load Test 1

The first load test was conducted on May 9, 1991, at the initiation of field monitoring. The bridge interlaminar compression at the time of the testing was approximately 40 lb/in^2. The test vehicle consisted of a fully loaded, two-axle dump truck with a gross vehicle weight of 32,960 lb (Fig. 14). In the longitudinal direction, the vehicle was positioned with the load centriod at the bridge midspan. Deflection measurements for this test were obtained to the nearest 0.06 in.

Load Test 2

The second load test was conducted on February 28, 1994, at the conclusion of field monitoring. For this load test, the interlaminar compression level was approximately 40 lb/in^2 or 90 lb/in^2 (after retensioning the stressing bars). The load test vehicle was a fully loaded, three-axle dump truck with a gross vehicle weight of 48,580 lb (Figs. 14, 15). In the longitudinal direction, the vehicle was positioned so that the rear axles were centered at midspan and the front axle was off the bridge. Deflection measurements for this test were obtained to the nearest 0.04 in.

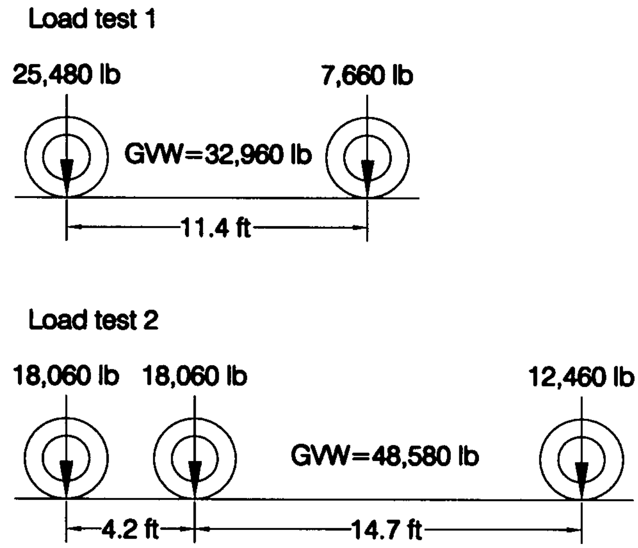


Figure 14—Axle weights and spacings for the trucks used in load test 1 (top) and load test 2 (bottom).

Analytical Evaluation

At the completion of load testing, the theoretical bridge behavior was modeled for actual truck loading and AASHTO HS20-44 loading using an orthotropic plate computer program developed at FPL.

Condition Assessment

The general condition of the Lynches Woods Park Bridge was assessed on three occasions during the monitoring period. The first assessment occurred at the beginning of the monitoring when instrumentation was installed and initial load testing was completed. The second assessment occurred approximately 3 months into the monitoring period. The final assessment occurred at the end of the monitoring period when final load testing was completed. These assessments involved visual inspections, measurements, and photographic documentation of the bridge's condition. Items of specific interest included deck camber, wood components, the stressing bar and anchorage system, and wood preservative.

Results and Discussion

Performance monitoring of the Lynches Woods Park Bridge was initiated in May 1991, approximately 10 months after bridge construction, and continued for 34 months until February 1994. A discussion of the monitoring results follows.

Moisture Content

The average moisture content trend for the monitoring period is shown in Figure 16. Based on data collected by Newberry

Figure 15—Vehicle positions for load test 2, as viewed from the south end of the bridge. From top to bottom, load cases 1, 2, and 3 are shown.

County personnel just prior to construction, the bridge was installed at an initial average moisture content of 26 percent. At the beginning of the monitoring period, approximately 10 months after installation, the initial electrical-resistance moisture content measurements indicated an average moisture content of 25 percent. Throughout the monitoring, climatic changes caused the average moisture content of the laminations to fluctuate from 20 to 25 percent. During an unusually dry period during the summer of 1992, the average

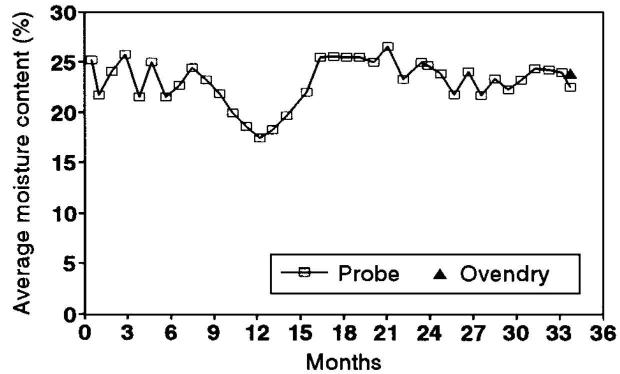


Figure 16—The average trend in moisture content readings taken at the lower 2–3 in. of the deck during monitoring, beginning 10 months after bridge installation.

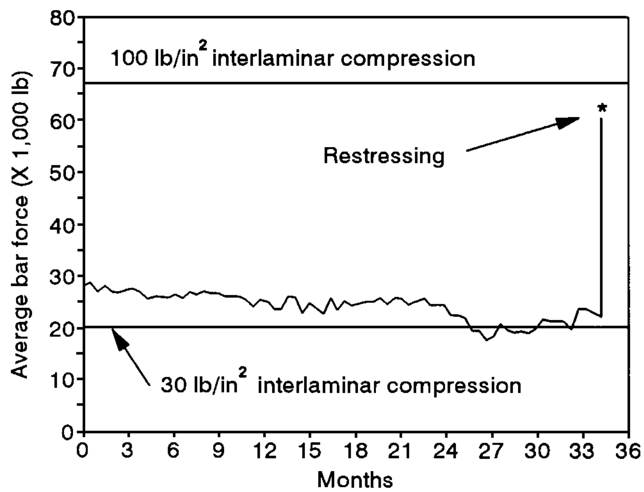


Figure 17—The average trend in bar force measurements since the initiation of monitoring, beginning 10 months after bridge installation.

moisture content briefly dropped below 20 percent. However, at the conclusion of monitoring, the average moisture content had rebounded to approximately 25 percent. In addition, ovendry moisture core samples obtained at the conclusion of monitoring indicated an average moisture content of 24 percent.

Although the moisture content of the Lynches Woods Park bridge was more than the recommended maximum of 19 percent at installation, it did not affect overall performance of the structure. In addition, rapid wetting and drying cycles did not occur as noted in previous reports on exposed structures treated with waterborne preservatives.

Bar Force

The average trend in bar force measurements during the monitoring period is shown in Figure 17. At the initiation of monitoring, the average measured bar force was approximately 29,000 lb. Thus, during the 8 months since the final bar tensioning, the average bar force decreased apthe final bar tensioning, the average bar force decreased approximately 60 percent (43,000 lb) prior to the initiation of monitoring. Periodic measurements during the monitoring showed that the average bar force was relatively stable at approximately 25,000 lb, or 37 lb/in^2 interlaminar compression. At the conclusion of the monitoring, the average bar force was approximately 24,000 lb, or 35 lb/in^2 interlaminar compression. Although not required for performance reasons, the bars were retensioned at the end of the monitoring period because of equipment availability and to allow load testing of the bridge at different interlaminar compression levels.

Although the overall bar force was relatively low at the initiation of monitoring, the bar force retention for the Lynches Woods Park bridge was satisfactory. Factors affecting the bar force were moisture content changes and stress relaxation. The moisture content changes were small and occurred in the outer portion of the laminations; therefore, the effect on bar force was minimal. The majority of bar force loss, both before and during the monitoring period, was due primarily to stress relaxation of the lumber laminations.

Behavior Under Static Load

Results of the static-load testing and analytical evaluation are presented. For each load case, transverse deflection measurements are given at the bridge midspan as viewed from the south end (looking north). For both load tests, no permanent residual deflection was measured between load cases or at the conclusion of testing. In addition, no measurable deflection was observed at the bridge supports.

Load Test 1

Transverse deflection for load test 1 is shown in Figure 18. For load case 1 (Fig. 18a) and load case 3 (Fig. 18c), the symmetry of the load cases resulted in the same maximum measured deflection of 0.50 in. under the wheel line nearest the bridge edge. For load case 2 (Fig. 18b), with the vehicle centered on the bridge cross-section, the maximum measured deflection of 0.44 in. occurred at the bridge centerline. In all cases, the deflection is typical of the linear elastic orthotropic plate behavior of stress-laminated bridges, and the locations of maximum measured deflection correspond to those observed in other similar bridges (Ritter and others 1991).

Load Test 2

Transverse deflection for load test 2 at interlaminar compression levels of 40 lb/in^2 and 90 lb/in^2 is shown in Figure 19. At the 40 lb/in^2 level, the maximum deflection for load case 1 (Fig. 19a) and load case 3 (Fig. 19c) occurred under the wheel line nearest the bridge edge and measured 0.50 and 0.55 in., respectively. For load case 2 (Fig. 19b), the maximum measured deflection of 0.45 in. occurred at the bridge centerline. For all load cases, the maximum measured deflections are very similar to those of load test 1, with no difference in location and only minor differences in magnitude. Such a small difference in magnitude was not expected because of the increased loading for load test 2, which should have resulted in a substantially greater deflection magnitude.

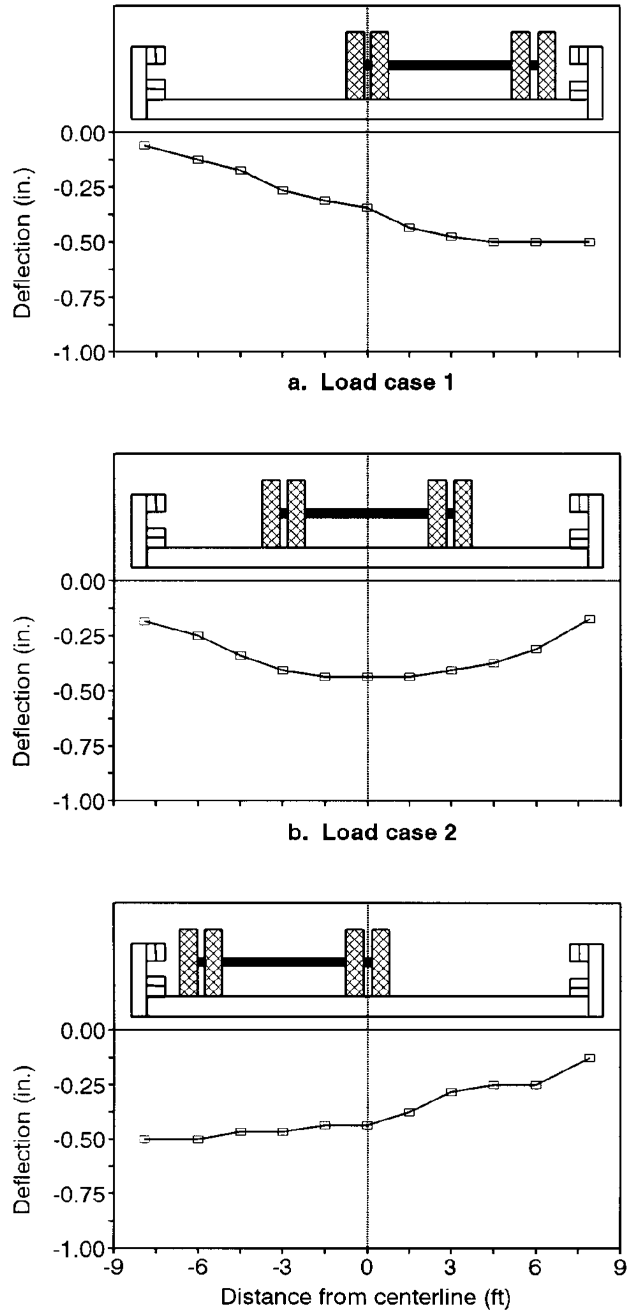


Figure 18—Transverse deflection for load test 1, measured at the bridge midspan (looking north). Bridge cross sections and vehicle positions are presented to aid interpretation only and are not to scale.

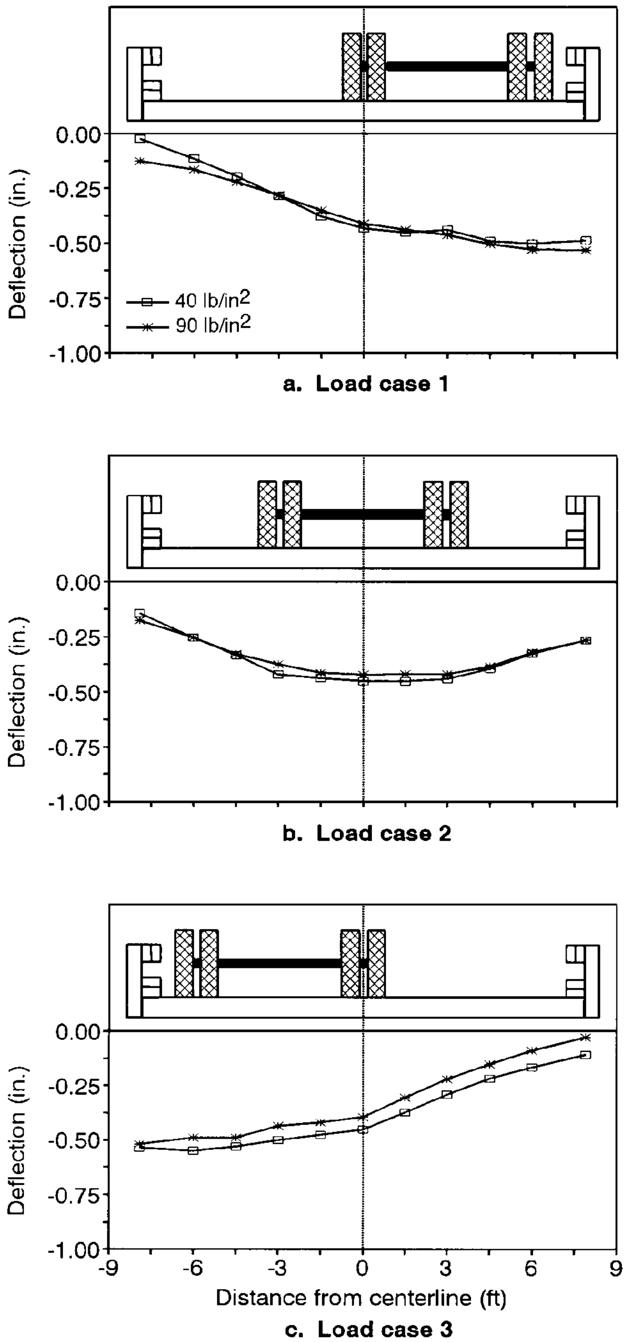


Figure 19—Transverse deflection for load test 2, measured at the bridge midspan (looking north), at interlaminar compression levels of 40 and 90 lb/in^2. Bridge cross sections and vehicle positions are presented to aid interpretation only and are not to scale.

This difference in magnitude is discussed further in the Analytical Evaluation section.

At 90 lb/in^2 interlaminar compression, the magnitude and location of the maximum measured deflections changed slightly from those measured at the 40 lb/in^2 level. For load case 1 (Fig. 19a) and load case 3 (Fig. 19c), the maximum measured deflection occurred at the bridge edge, rather than under the wheel line nearest the bridge edge, and measured 0.53 and 0.52 in., respectively. For load case 2 (Fig. 19b), the maximum measured deflection of 0.42 in. occurred at the bridge centerline, the same location measured for the 40 lb/in^2 level and for load test 1. Given the 50-lb/in^2 increase in interlaminar compression, we expected that the bridge stiffness would increase slightly (Oliva and others 1990). When comparing the two interlaminar compression levels, the relative deflection curves are very similar but an increase in stiffness, indicated by reduced deflection, is clearly evident only for load case 3. It is likely that a global increase in longitudinal bridge stiffness at the 90 lb/in^2 interlaminar compression existed for all load cases. However, it is not possible to draw definitive conclusions from these tests, given the accuracy of the deflection measurements.

Analytical Evaluation

A comparison of load test 1 measured deflections to the theoretical bridge response based on orthotropic plate analysis is shown in Figure 20. For modeling purposes, we assumed a deck lamination MOE of 1,600,000 lb/in^2 for the Southern Pine material, based on tabulated design values (AFPA 1988). As shown in Figure 20, the theoretical bridge deflection is very close to that measured, although minor differences between the measured and theoretical deflection are evident in several locations.

A comparison of the measured and theoretical deflections for load test 2 at interlaminar compression levels of 40 and 90 lb/in^2 are shown in Figure 21. In completing the analysis, we found that the assumed material MOE of 1,600,000 lb/in^2 used for load test 1 resulted in poor theoretical correlation with measured results and consistently overstated the deflections for load test 2. Based on these results, it was evident that a larger longitudinal stiffness was necessary for load test 2 to achieve a measured theoretical correlation similar to that obtained for load test 1. Given that both load tests 1 and 2 were conducted when the interlaminar compression was 40 lb/in^2, we eliminated bridge prestress effects on the transverse deck properties and butt joints as a possible cause of longitudinal stiffness change. In addition, the deck section and lamination moisture content were unchanged and did not significantly affect longitudinal stiffness. As shown in Figure 21, the correlation between the measured deflection and that computed using an MOE of 2,100,000 lb/in^2 is very good and similar to the load test 1 correlation with a material MOE of 1,600,000 lb/in^2. Although the exact cause of the apparent 30 percent increase in MOE is unknown at this time, it is suggested that a chemical interaction between the CCA treatment and the wood laminations could cause a modification to the wood structure that results in an

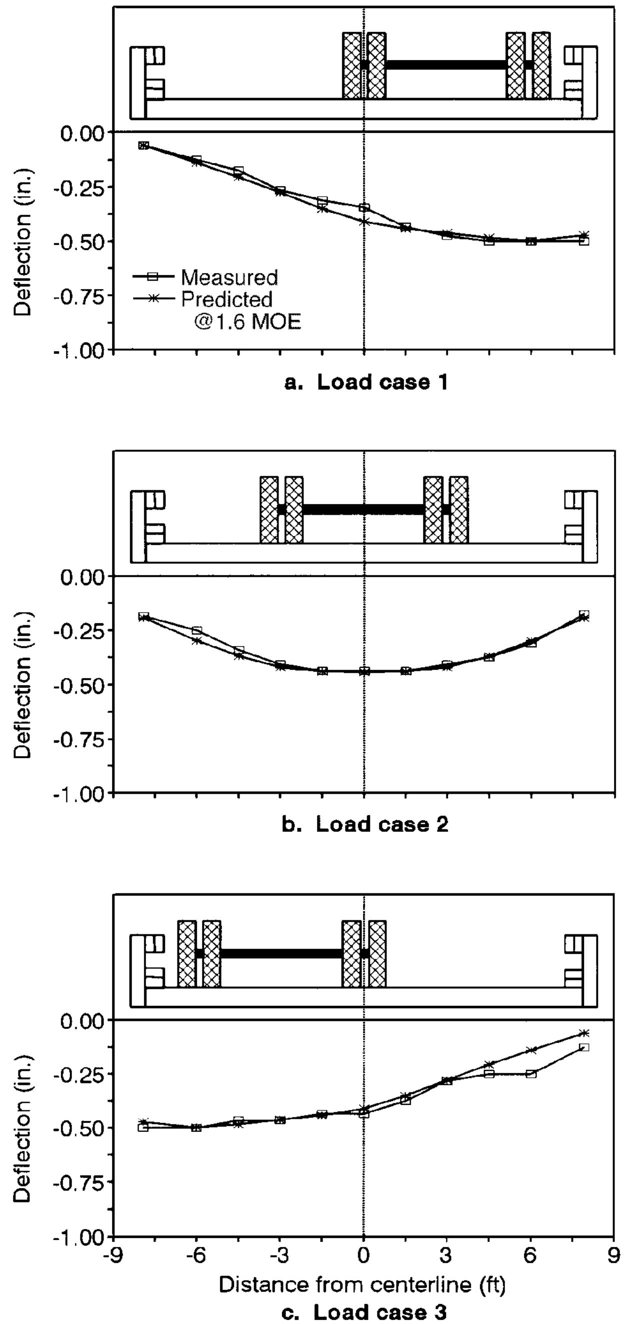


Figure 20—Comparison of load test 1 measured and predicted deflections using orthotropic plate analysis.

increase in MOE. An MOE change of this extent has not been observed in research studies of treated materials. However, the same type of phenomenon has been observed on other bridges built with CCA-treated lumber that are currently being monitored by FPL; conclusive findings will be forthcoming.

Theoretical deflection for AASHTO HS20-44 truck loading is shown in Figure 22 for load cases 1 and 2. (Load case 3 is a mirror image of load case 1.) Deflections correspond to the load test conditions previously discussed and are based on the same relative analytical input parameters for interlaminar compression and MOE. As shown for each set of load test conditions, the maximum HS20-44 theoretical deflection occurred for load case 1 rather than load case 2, and the location of maximum HS20-44 deflection for each load case corresponds to the location where maximum load test deflections were measured. As summarized in Table 2, the maximum theoretical HS20-44 deflection of 0.86 in., or L/404 as expressed as a fraction of the bridge span, occurred under load test 1 conditions. For load test 2, with an MOE of 2,100,000 lb/in^2, the maximum theoretical deflection of 0.69 in. or L/503 was obtained at an interlaminar compression of 40 lb/in^2. After bar tensioning to 90 lb/in^2, the load test 2 maximum theoretical HS20-44 deflection was 0.64-in. or L/543. Thus, the 50-lb/in^2 increase in interlaminar compression for load test 2 resulted in an approximate 7-percent decrease in the theoretical HS20-44 deflection.

Condition Assessment

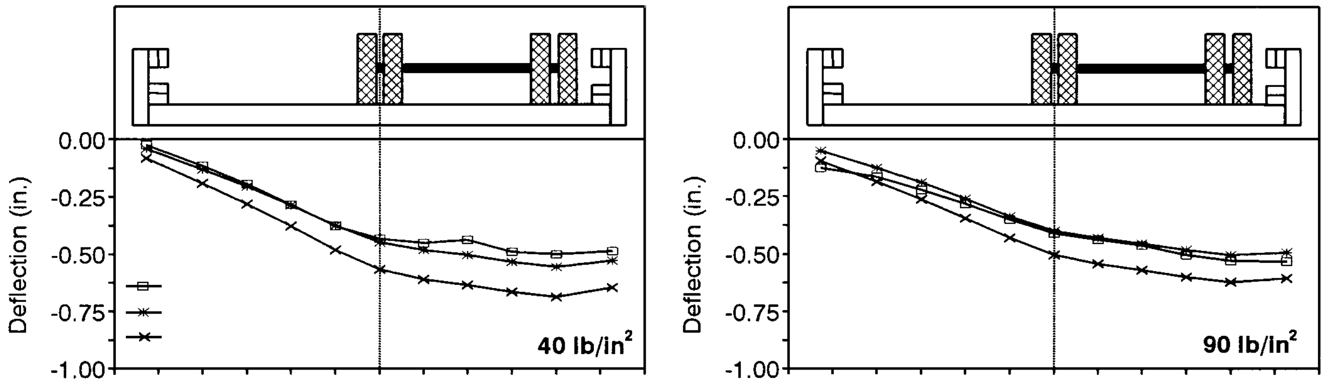
Condition assessment of the Lynches Woods Park bridge indicates that all structural and serviceability aspects of the structure are satisfactory. Results for specific areas follow.

Deck Camber

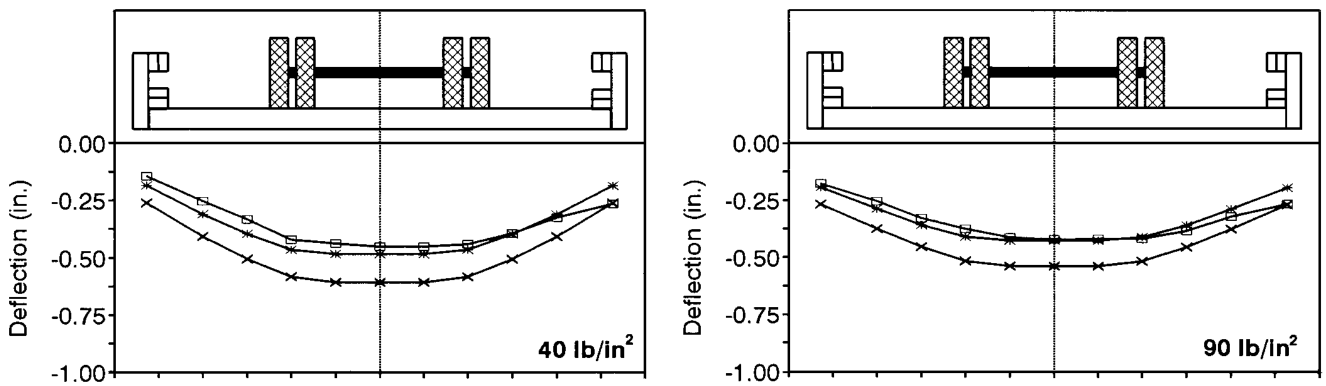
An initial deck camber of 1.25 in. was introduced during bridge construction by slightly raising the center scaffolding support with respect to the abutments. Measurements at the initiation of the monitoring period indicated approximately 0.50-in. positive deck camber at both the upstream and downstream edges. Thus, approximately 0.75 in. of positive camber was lost due to the bridge dead-load deflection and creep during the 10 months prior to the initiation of monitoring. Measurements at the conclusion of the monitoring period indicated no measurable change in the deck elevation, and approximately 0.50 in. of positive camber remained.

Wood Components

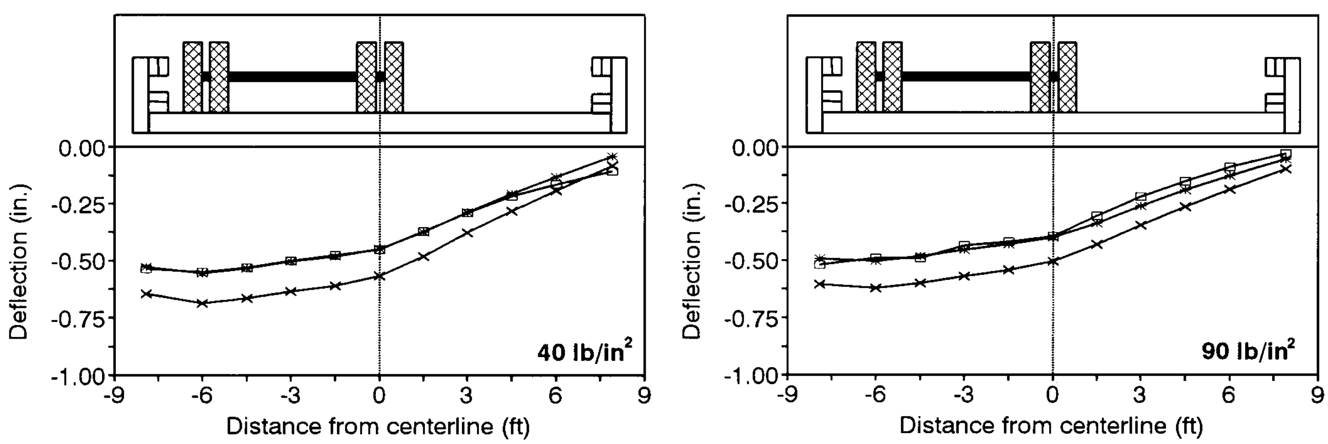
Visual inspection of the wood components in the Lynches Woods Park bridge indicated no sign of deterioration or damage. Because the location is sheltered by tall trees most of the year, drying of wood members was minimal, and components appeared to remain at a relatively high moisture content. Thus, end-grain and side-grain checking in rail and curb members, typical for most sawn lumber highway structures, was minimal for this bridge.



a. Load case 1



b. Load case 2



c. Load case 3

Figure 21—Comparison of load test 2 measured and predicted deflections using orthotropic plate analysis.

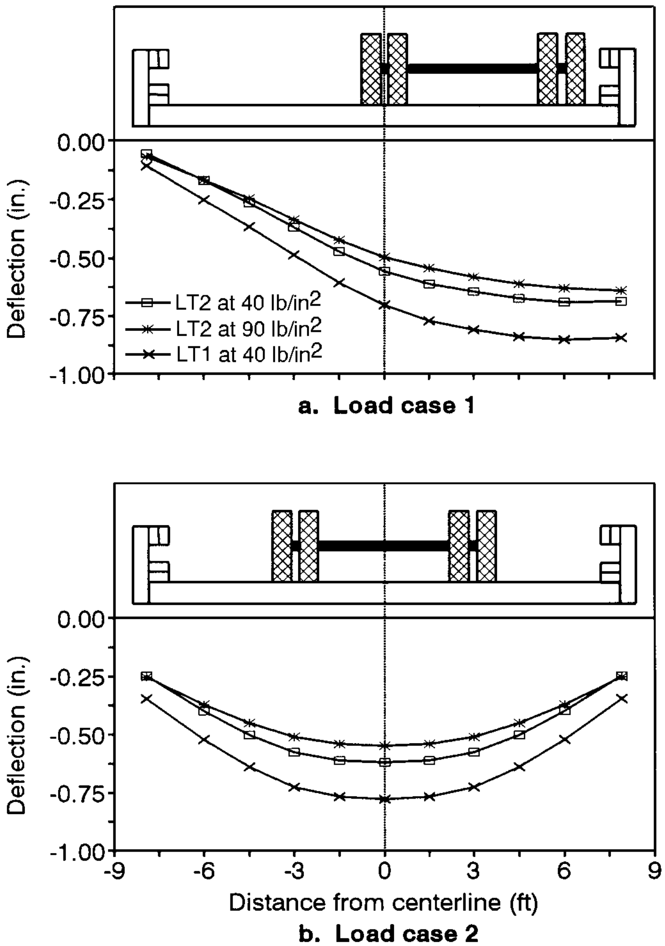


Figure 22—Predicted midspan deflection for load cases 1 and 2 using HS20-44 loading and orthotropic plate analysis.

Table 2—Maximum theoretical HS20-44 deflections

	Maximum theoretical HS20-44 deflection (in.)	
Load test/condition	Load case 1	Load case 2
Load test 1 40 lb/in^2 interlaminar compression MOE = 1,600,000 lb/in^2	0.86	0.78
Load test 2 40 lb/in^2 interlaminar compression MOE = 2,100,000 lb/in^2	0.69	0.62
Load test 2 90 lb/in^2 interlaminar compression MOE = 2,100,000 lb/in^2	0.64	0.55

Figure 23—Visual inspection of outer lamination after removal of bearing plate showed no signs of bar corrosion.

Figure 24—Gradual deformation of the laminations in the vicinity of the bar anchorages. Note the small gap between the aluminum straight edge and the deck edge.

Stressing Bar and Anchorage System

Inspections of the stressing bar system indicated satisfactory performance. Galvanized steel hardware that was exposed to the environment was in good condition, with no visible sign of corrosion or deterioration. At the conclusion of monitoring, several bearing plates were removed to allow visual inspection of the stressing bars and bearing plate areas (Fig. 23). The condition of several bars was examined inside the PVC tubing and signs of corrosion were not evident. In addition, there was no visible crushing of the discrete plate anchorage into the outside laminations. However, slight deformations were present in the outside laminations along each bridge edge. These deformations produced a gentle depression in the vicinity of the plate anchorages that measured from 1/8 to 1/16 in. (Fig. 24). These depressions were the result of stress relaxation of the deck laminations and did not affect the performance of the structure.

Wood Preservative

The performance of CCA preservative treatment used on sawn lumber components was satisfactory, and the integrity of the preservative envelope remained intact during the monitoring period. Dimensional stability of the treated wood was good, because the shaded and humid environment at the site prevented rapid wetting and drying cycles that leads to dimensional changes in exposed structures. Based on visual inspections, there was no notable difference between the performance of this bridge and other similar bridges treated with oil-based preservatives.

Conclusions

Based on the monitoring results from the Lynches Woods Park bridge, we make the following conclusions and recommendations:

- After 34 months in service, the performance of the Lynches Woods Park bridge is satisfactory. There are no visible signs of deterioration that would prevent the bridge from providing several additional years of acceptable service.
- It is both feasible and practical to design and construct stress-laminated timber decks using CCA-treated Southern Pine lumber.
- Stress-laminated decks can be constructed in-place by manually placing laminations and using temporary scaffolding to support the butt-jointed laminations prior to bridge stressing. This technique is most viable when large construction equipment is not readily available and manual labor is cost effective.
- The general trend in moisture content in the lower 2 in. of the laminations indicates no significant changes during the monitoring period. At the conclusion of monitoring, the lumber laminations remain at approximately 25 percent moisture content.
- The general trend in bar force measurements indicates that an adequate level of bar force was maintained throughout the monitoring period. With an initial tension of 72,000 lb, bar forces decreased approximately 43,000 lb (or 60 percent) during the 8 months prior to monitoring. During the monitoring period, bar force measurements were relatively stable at approximately 25,000 lb, or 37 lb/in^2 interlaminar compression. At the conclusion of monitoring, the stressing bars were tensioned to the 90 lb/in^2 interlaminar compression level.
- Loss of camber as a result of initial dead load and deck creep totaled 0.75 in. and occurred prior to the initiation of monitoring. No measurable camber loss was detected during the monitoring period. A positive camber of approximately 0.50 in. remained at the conclusion of monitoring.
- Load testing and analysis indicated that the Lynches Woods Park bridge is performing as a linear-elastic orthotropic plate when subjected to static-truck loading.
- Based on load test analyses, the bridge appears considerably stiffer after only 2-1/2 years in service. At the same level of 40 lb/in^2 interlaminar compression, the correlation between the actual and theoretical deflection was excellent using an MOE of 1,600,000 lb/in^2 for load test 1 and an MOE of 2,100,000 lb/in^2 for load test 2. Although the exact cause of the apparent increase in MOE is unknown at this time, the phenomenon has been observed on other bridges currently being monitored by FPL and conclusive findings will be forthcoming.
- The maximum theoretical deflections for a single lane of AASHTO HS20-44 loading were below the design limit of 1/360 for the span length in all cases. For load test 1 at 40 lb/in^2 interlaminar compression, the maximum deflection is estimated to be 0.86 in., which is approximately 1/404 of the span length measured center-to-center of bearings. For load test 2 at 40 lb/in^2 interlaminar compression, the maximum deflection is estimated to be 0.69 in., which is approximately 1/503 of the span length. For load test 2 at 90 lb/in^2 interlaminar compression, the maximum deflection is estimated to be 0.64-in., which is approximately 1/543 of the span length.
- Based on visual inspections, there are no indications of deterioration in the steel hardware or wood components. In addition, there was no notable difference between the performance of this CCA-treated bridge and other similar bridges treated with oil-based preservatives.

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Appendix—Information Sheet

General

Name: Lynches Woods Park bridge

Location: City of Newberry, South Carolina

Date of Construction: July 1990

Owner: Newberry County, South Carolina

Design Configuration

Structure Type: Stress-laminated deck with butt joints

Butt Joint Frequency: 1 in 4 laminations transverse with joints in adjacent laminations separated 4 ft longitudinally

Total Length (out–out): 30 ft, 29.9 ft (as-built)

Skew: None

Number of Spans: 1

Span Length (center–center of bearings): 29 ft
28.9 ft (as-built)

Width (out–out): 16 ft, 15.8 ft (as-built)

Width (curb–curb): 14.3 ft, 14.5 ft (as-built)

Number of Traffic Lanes: 1

Design Loading: AASHTO HS20–44

Wearing Surface Type: None

Material and Configuration

Timber:

Species: Southern Pine

Size: 3-7/8 by 14 in.

Grade: No. 1 Dense Structural

Moisture Condition: Approximately 26 percent at installation

Preservative Treatment: Chromated copper arsenate (CCA)

Stressing Bars:

Diameter: 1 in.

Number: 8

Design Force: 72,000 lb

Spacing (center–center): 4 ft, beginning 1 ft from bridge ends

Type: High strength, steel bar with course right-hand thread, conforming to ASTM A722

Rail and Curb System:

Design: AASHTO static loading

Species: Southern Pine

Grade: No. 2 Dense Structural

Member Sizes: Rails: nominal 6 by 12 in.
Posts: nominal 8 by 8 in.
Curbs: nominal 6 by 10 in.

Preservative Treatment: Chromated copper arsenate (CCA)

Anchorage Type and Configuration:

Discrete steel plates: 11.5 by 13.5 by 1 in. bearing
4 by 6.5 by 1.25 in. anchor