United States
Department of
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Forest Service

Forest
Products
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National Wood
in Transportation
Information Center

Research
Paper
FPL–RP–570

In cooperation
with the

United States
Department of
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Federal
Highway
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Field Performance of Timber Bridges

16. North Siwell Road Stress-Laminated Bridge

James A. Kainz

Abstract

The North Siwell Road bridge was constructed during December 1994 in Hinds County, Mississippi. The bridge is a single-span, stress-laminated T-beam structure that measures 9.1 m (30 ft) long and 8.7 m (28.5 ft) wide. Performance of the bridge was monitored for 24 months, beginning at the time of installation. Monitoring involved gathering and evaluating data relative to the moisture content of the wood components, force level of stressing bars, and behavior of the bridge under static load conditions. In addition, comprehensive visual inspections were conducted to assess the overall condition of the structure. Based on field evaluations, the bridge is performing well.

Keywords: Wood, bridge, stress-laminated T, timber, performance

Acknowledgments

We thank the following from the Hinds County Department of Public Works for assistance with field data collection, load tests, and visual inspection of the North Siwell Road bridge: Wayne Williamson, Vonda Moore, and the bridge crew.

In addition, we express sincere appreciation to the following from the USDA Forest Service, Forest Products Laboratory: Mike Ritter, James Wacker, Paula Hilbrich Lee, and Lola Hislop for assisting in the field evaluation, data analysis, and report preparation; Vyto Malinauskas for fabricating load cells; and the Information Services Team for assistance in preparing this report.

March 1998

Kainz, James A. 1998. Field performance of timber bridges: 16. North Siwell Road stress-laminated bridge. Res. Pap. FPL–RP–570. Madison, WI: U.S. Department of Agriculture, Forest Service, Forest Products Laboratory. 17 p.

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Field Performance of Timber Bridges

16. North Siwell Road Stress-Laminated Bridge

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Introduction

In 1988, the U.S. Congress passed legislation known as the Timber Bridge Initiative (TBI) (USDA 1995). As part of an effort to revitalize rural America, the legislation established a national program emphasizing wood as a structural material for highway bridges. Recently, the emphasis of this program has shifted to include all timber transportation structures, such as sound barriers and bridges.

Responsibility for the development, implementation, and administration of the TBI program was assigned to the USDA Forest Service. The Forest Service established three primary program areas under the TBI: demonstration structures, technology transfer, and research. As part of the demonstration bridge program, the National Wood in Transportation Information Center (NWITIC) in Morgantown, West Virginia, awards annual grants for demonstration bridges on a competitive basis. Funds are awarded for design and construction of demonstration timber bridges with innovative designs and those that utilize locally available underutilized wood species. The NWITIC also maintains a technology transfer program to provide assistance and state-of-the-art information related to all aspects of timber bridges.

Responsibility for the research portion of the TBI program was assigned to the USDA Forest Service, Forest Products Laboratory (FPL). The FPL has established a broad research program to conduct a variety of timber bridge studies under laboratory and field conditions, including a nationwide bridge monitoring program. Through the bridge monitoring program, FPL is able to collect, analyze, and distribute information on the field performance of timber bridges to provide a basis for validating or revising design criteria and further improving efficiency and economy of timber bridge design, fabrication, and construction.

This report is 16th in a series of reports that documents the field performance of timber bridges. It describes the development, design, construction, and field performance of the North Siwell Road bridge located in Hinds County, Mississippi. The structure is a 9.1-m- (30-ft-) long, double-lane, stress-laminated T-beam bridge. Built in 1994, the North Siwell Road bridge was funded jointly by a competitive grant from the NWITIC and matching funds from Hinds County, Mississippi. An information sheet on the characteristics of the North Siwell Road bridge is provided in the Appendix.

Background

The North Siwell Road bridge is located in Hinds County, approximately 7 miles southwest of Jackson, Mississippi (Fig. 1). The bridge is owned by Hinds County and is located on a secondary road that provides access to local residences and schools. In 1994, the average daily traffic over this section of the road was 6,520 vehicles. Estimates by Hinds County Department of Public Works indicate that the average daily traffic is expected to increase to approximately 9,300 vehicles by the year 2012. The majority of this traffic consists of passenger vehicles and school buses.

The original North Siwell Road bridge was 7 m (23 ft) wide and 6.1 m (20 ft) long and consisted of prefabricated concrete deck panels on timber caps and piles. Based on an annual inspection in 1992, it was determined that the wood components were deteriorating and the concrete deck was in need of repair. Therefore, replacement of the bridge was deemed necessary to provide adequate access for school traffic, emergency vehicles, and commuters.

In December 1992, the Hinds County Department of Public Works submitted a proposal for partial funding of the North Siwell Road bridge replacement as a timber bridge under the NWITIC demonstration program. As part of this proposal, a preliminary bridge design was developed for a timber stress-laminated T-beam bridge. Southern Pine was selected as the primary material for the bridge because of economics and as a locally grown species in Mississippi. After review by a selection panel, funds were awarded and final design of the replacement bridge was initiated. Hinds County had little experience working with timber bridges; therefore, FPL was contacted during the final design phase of the project to evaluate the performance of the new North Siwell Road timber bridge. As a result, personnel from FPL and Hinds County Department of Public Works developed a performance monitoring plan that was initiated at installation.

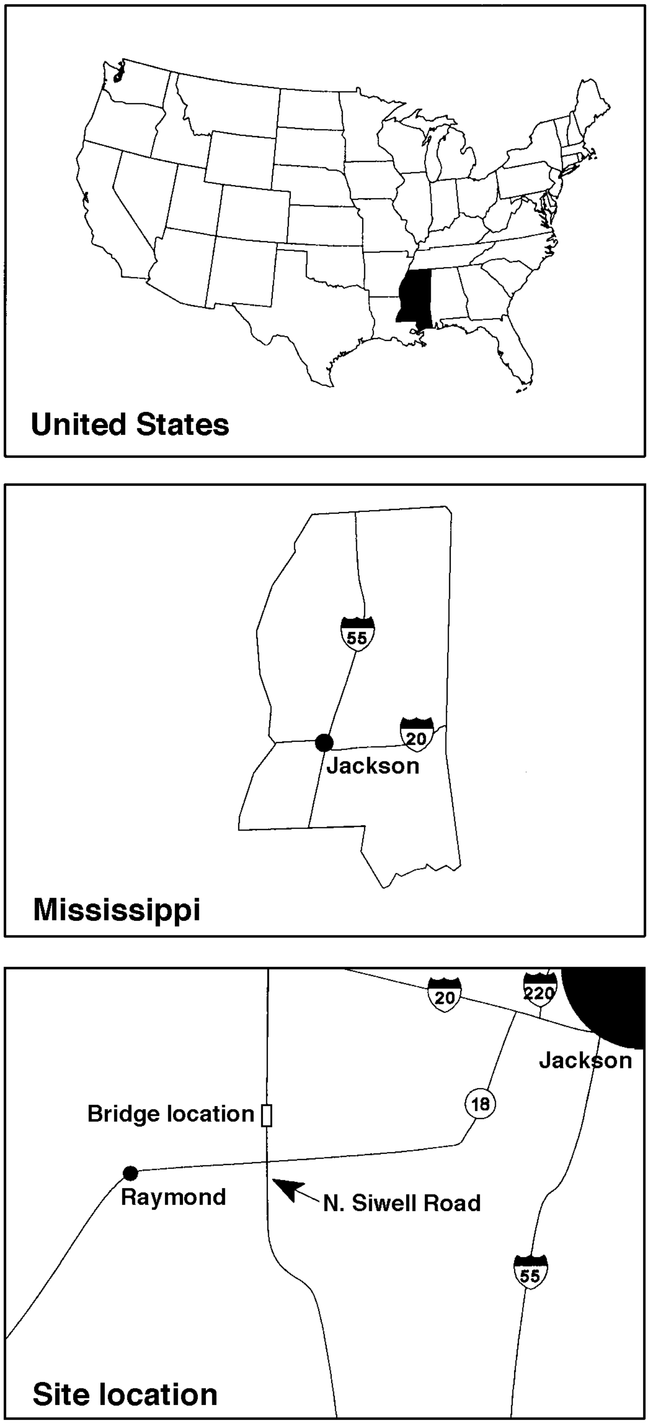

Figure 1—Location maps for the North Siwell Road bridge.

Objective and Scope

The objective of this project was to evaluate the field performance of the North Siwell Road bridge for 24 months, beginning at bridge installation. The scope of the project included data collection and analysis related to the wood moisture content, stressing bar force, bridge behavior under static truck loading, and general structure performance. The results of this project will be considered with the results of similar monitoring projects to improve design and construction for future stress-laminated T-beam bridges.

Design, Construction, and Cost

Design and construction of the North Siwell Road bridge were completed by several agencies and individuals. An overview of the design, construction, and cost of the bridge superstructure follows.

Design

A typical stress-laminated T-beam bridge consists of glued-laminated timber (glulam) webs and sawn lumber flanges that are connected by tensioned, high strength steel stressing elements (Fig. 2). On the North Siwell Road bridge, an American Association of State Highway and Transportation Officials (AASHTO) approved method for designing a stress-laminated T-beam structure was not available, so design criteria for the stress-laminated T-beam bridge was based on guidelines developed from research conducted at West Virginia University (Davalos and Salim 1993). All other aspects of design including loading, wood treatment, and wood strength were designed in accordance with the AASHTO *Standard Specifications for Highway Bridges* for two lanes of HS20-44 truck loading (AASHTO 1992).

Design and fabrication of the North Siwell Road bridge were completed by contract. The design geometry provided for a single span, simply supported structure, 9.1 m (30 ft) long and 8.7 m (28.5 ft) wide (Fig. 3). The new bridge was designed to be 3.1 m (10 ft) longer than the original bridge, because the increased span length allows additional hydraulic flow capacity that is required during periods of high water.

The North Siwell Road bridge configuration consists of Southern Pine glulam lumber webs and sawn lumber flanges. The glulam webs were 171.5 mm (6-3/4 in.) wide and 593.7 mm (23-3/8 in.) deep. Glulam design was based on material properties for combination 24F–V3 SP/SP (AASHTO 1992). Tabulated design values were 16.5 MPa (2,400 lb/in^2) for bending strength, 12.4 GPa (1,800,000 lb/in^2) for modulus of elasticity (MOE), 1.4 MPa (200 lb/in^2) for shear strength, and 3.9 MPa (560 lb/in^2) for compression strength perpendicular to grain. All design values were adjusted by appropriate wet-use factors per AASHTO requirements.

The sawn lumber flanges were constructed with standard 38- by 184-mm (nominal 2- by 8-in.) material that was surfaced on four sides to provide uniform contact between laminations. Butt joints were used in the flanges because

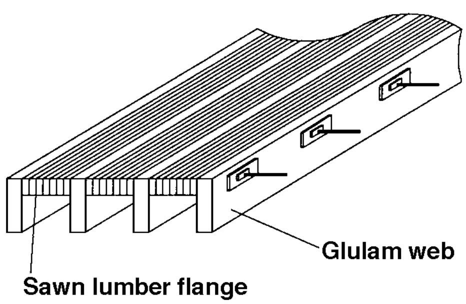

Figure 2—Typical configuration for a stress-laminated T system.

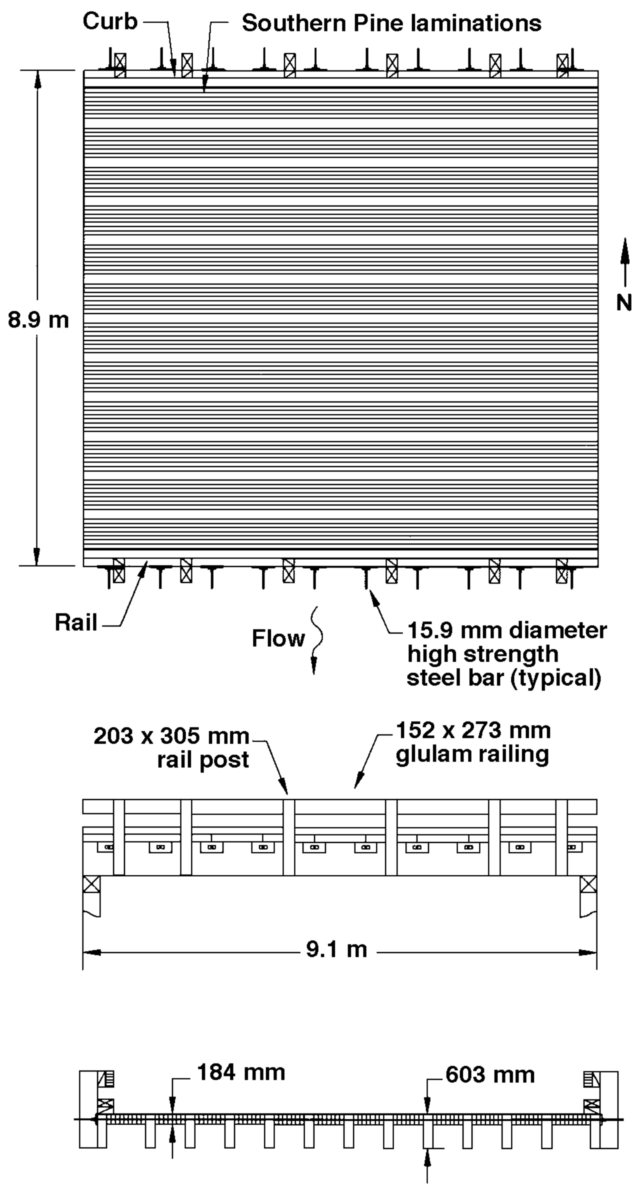

Figure 3—Design configuration of the North Siwell Road bridge.

material was not available in the full 9.1-m (30-ft) length. The butt joints were specified at an interval of one butt joint every four adjacent laminations, longitudinally spaced at 1.2 m (4 ft) (Fig. 4). The design values for the flange were based on AASHTO specifications for Southern Pine lumber visually graded No. 1 in accordance with Southern Pine Inspection Bureau rules (SPIB 1993). The tabulated design values were 10.3 MPa (1,500 lb/in^2) for bending, 11.7 GPa (1,700,000 lb/in^2) for MOE, 620 kPa (90 lb/in^2) for shear, and 3.9 MPa (565 lb/in^2) compression perpendicular to grain. All design values were adjusted by the appropriate wet-use factors, and laminations were specified to be at or below 19% moisture content prior to preservative treatment and bridge installation.

The stressing system used ten 15.9-mm- (5/8-in.-) diameter high strength steel bars that complied with the requirements of ASTM A722 (ASTM 1988) and provided a minimum ultimate tensile strength of 1.03 GPa (150,000 lb/in^2). The bars were inserted through oversized, predrilled holes located at the center of the sawn lumber flange and 92 mm (3-5/8 in.) from the top of the glulam webs (Fig. 3). The bars were spaced 914 mm (36 in.) on center, starting 457 mm (18 in.) from the ends of the bridge. Each bar required a tensile force of 156 kN (35,000 lb) to provide 924 kPa (134 lb/in^2) compression stress between the laminations. The value for interlaminar compression was based on the West Virginia University design method in which the recommended design interlaminar stress is 2.5 times the minimum interlaminar stress of 365 kPa (53 lb/in^2). The effect of this initial prestress is examined in detail in the following section on bar force. The bar tension was transferred into the deck using a discrete plate anchorage system, consisting of 184- by 394- by 25.4-mm (7.25- by 15.5- by 1-in.) bearing plates and a 50.8- by 127- by 25.4-mm (2- by 5- by 1-in.) anchor plate with a hexagonal nut (Fig. 5).

Design of the bridge rail and curb system was based on a crash-tested system for longitudinal spike-laminated decks (FHWA 1990). The bridge rail and curb consisted of a 267- by 152-mm (10.6- by 6-in.) sawn lumber timber rail and a 140- by 292-mm (5.5- by 11.5-in.) sawn lumber curb with 140- by 292-mm (5.5- by 11.5-in.) sawn lumber scupper blocks. The rail and curb were attached to six, 1,370- by 190- by 305-mm (54- by 7.5- by 12-in.) sawn lumber posts per side.

For protection from deterioration, all steel components, including stressing hardware, stressing bars, and anchorage plates, were galvanized in accordance with AASHTO M232 (AASHTO 1992). All wood components were preservative treated with creosote in accordance with American Wood Preservers' Association standard C14 (AWPA 1990). No asphalt wearing surface was specified for the bridge.

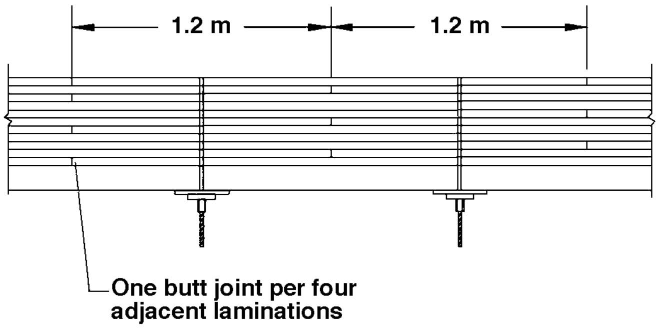

Figure 4—Butt-joint configuration used with Southern Pine sawn lumber flange laminations on the North Siwell Road bridge. A butt joint was placed transverse to the span in every fourth lamination. Longitudinally, butt joints in adjacent laminations were separated by 1.2 m (4 ft).

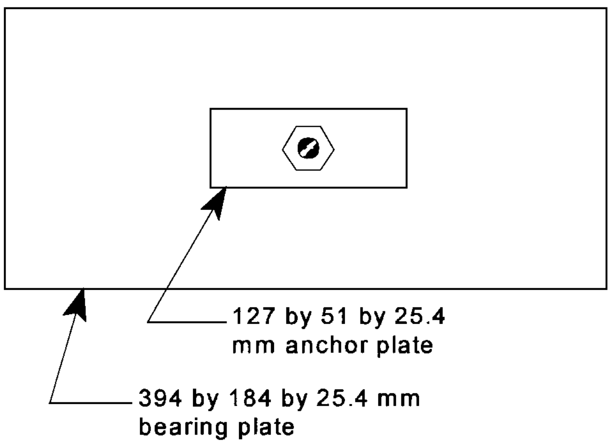

Figure 5—Discrete plate anchorage configuration consisting of a bearing plate, anchor plate, and hexagonal nut.

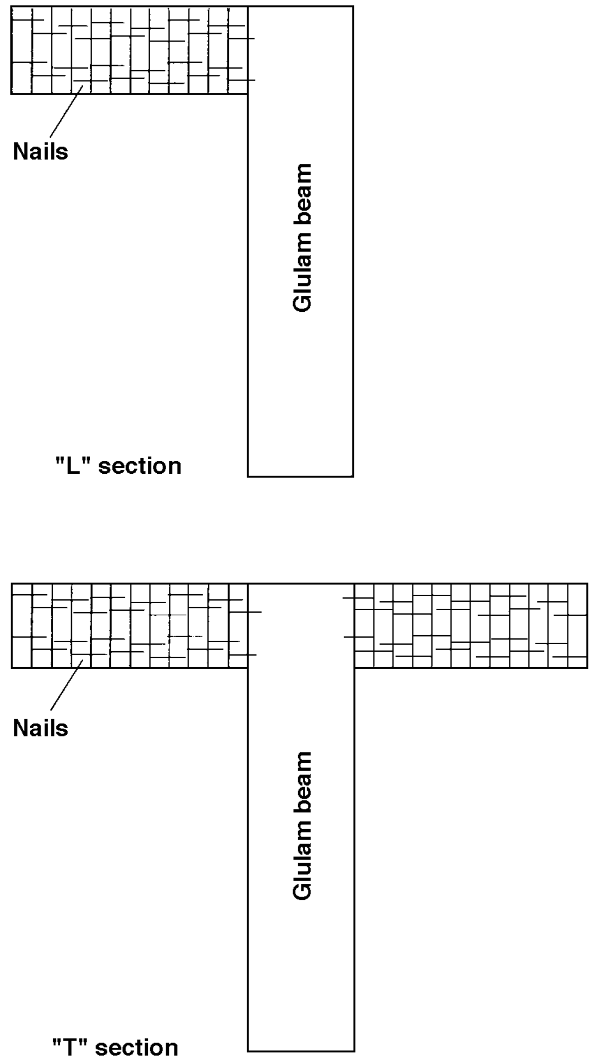

Figure 6—Assembled L and T sections used for transportation to bridge site and ease of installation.

Construction

Construction of the North Siwell Road bridge was completed in late 1994. Construction extended over several months and included assembly that was completed by the bridge manufacturer and installation that was completed by Hinds County personnel.

Assembly

The assembly process began with manufacturing the glulam beams and drying and surfacing of the sawn lumber flange material. Following manufacturing and drying, all sawn lumber and glulam members were prefabricated (cut and drilled) and pressure treated with creosote preservative. After preservative treatment, the treated wood was fabricated into sections to facilitate shipping and installation. These sections were made by nailing flange pieces to the glulam webs to form either an L or T module (Fig. 6). The L module was used on the exterior of the bridge, and the T configuration was used for the interior. After assembly, the L and T sections were loaded on a flatbed truck and shipped to the bridge site.

Installation

After demolition and removal of the existing bridge superstructure, a new substructure was installed, which consisted of timber piling, wing walls, and caps. The new substructure was installed behind the existing substructure to provide additional erosion protection for the new substructure. Approximately 0.5 to 0.7 m (2 to 3 ft) from the top, the existing substructure was removed to provide clearance for the new bridge.

On December 20, 1994, installation of the new superstructure commenced by lifting one assembled L section into place on the abutments with an overhead crane. After several T sections were in place next to the L section, the high strength steel bars were inserted through predrilled holes in the flange and webs. All sections were placed in a similar manner until the final L section was placed on the opposite end. After all sections were placed and the stressing bars were fully inserted, the steel bearing and anchorage plates were installed. A single hydraulic pump and jack was used to partially tension the steel stressing bars to bring the T and L sections into contact. After all sections were in contact, full design bar force was then introduced into the bridge. The Hinds County bridge crew began at one end and tensioned each bar along the length of the bridge (Fig. 7). After all bars were fully tensioned, the tensioning process was repeated to ensure that the interlaminar compression level was uniform and at the required design level. Seven days after this initial tensioning, a second design tensioning was introduced into the steel bars. The third tensioning was completed January 24, 1995, approximately 5 weeks after the initial stressing.

After the second bar tensioning, the bridge was attached to the substructure by connecting each glulam web to the timber cap with steel angles bolted to the webs and abutment cap (Fig. 8). At this time, the timber curb and rail system was also installed (Fig. 9). When the rail system was complete, a tack coat of asphalt and coarse rock was applied as a wearing surface and the bridge was opened for traffic. The bridge was posted with a reduced speed limit because the approach roadway was very rough. The completed bridge is shown in Figure 10.

Cost

The total cost of the North Siwell Road bridge superstructure was $46,190, which included design, fabrication, materials, and construction. To compare the cost of this bridge with others, the total cost of the bridge is divided by the total deck area to ascertain a cost per unit area. This equates to approximately $582/m^2 ($54/ft^2) for the North Siwell Road bridge.

Evaluation Methodology

To evaluate the structural and serviceability performance of the North Siwell Road bridge, Hinds County personnel contacted the FPL for assistance. Through a cooperative agreement with Hinds County, FPL, and Federal Highway Administration (FHWA), a 2-year bridge monitoring plan was developed and implemented. The plan included performance monitoring of the deck moisture content, stressing bar force, static load test behavior, and general bridge condition. The evaluation methodology employed procedures and equipment previously developed by FPL and used on other similar structures (Ritter and others 1991).

Figure 7—Bar tensioning using a single hydraulic pump and jack.

Figure 8—Bridge attachment to the sawn lumber substructure with steel angle saddles.

Figure 9—Attachment of bridge curb and rail system following the second bar tensioning.

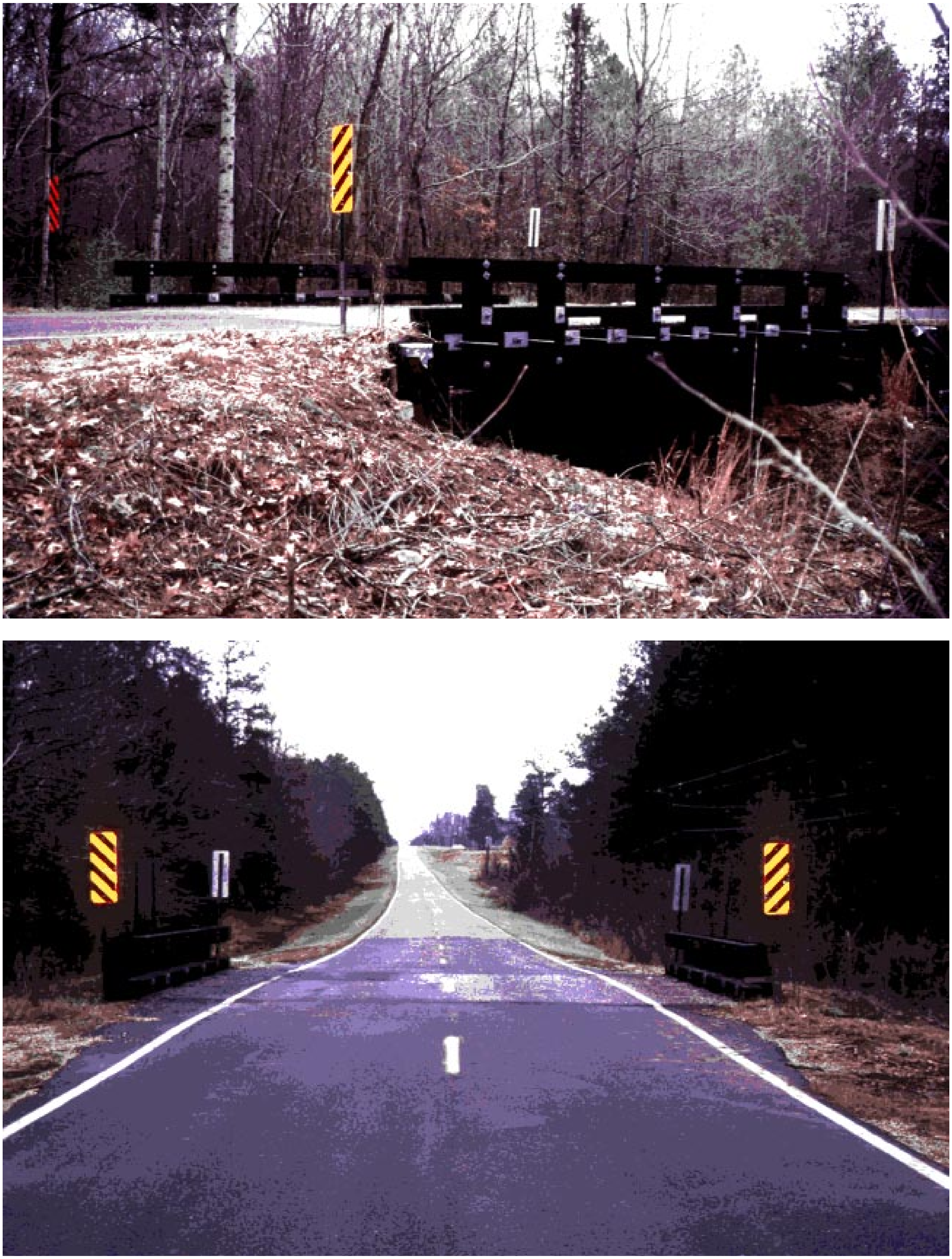

Figure 10—Completed North Siwell Road bridge: (top) side view, (bottom) end view.

Moisture Content

To characterize changes in moisture content, an electrical-resistance moisture meter was used to obtain wood moisture content readings on a quarterly basis. Moisture meter measurements were taken by Hinds County personnel from the bottom of glulam beams and the underside of the sawn lumber flanges and were assumed to be representative of the overall moisture content of the bridge. Measurements were obtained in accordance with ASTM D 4444–84 (ASTM 1992) by driving the insulated moisture pins into the underside of the bridge at depths of 50 to 75 mm (2 to 3 in.), recording the moisture content value from the unit, then adjusting the moisture content value for temperature and wood species.

Bar Force

To monitor stressing bar force, two calibrated load cells were placed on the stressing bars just prior to the third stressing. These cells were placed between the bearing and anchorage plates to monitor the bar forces based on the strain variations in the load cell. On a monthly basis, load cell measurements were obtained by Hinds County personnel with a portable strain indicator. The measurements were then converted to force, based on laboratory load cell calibrations, to determine the tensile force in the bar. At the conclusion of the monitoring period, the load cells were removed, adjusted for zero balance shift, and re-calibrated in the laboratory. In addition, hydraulic stressing equipment was used at the site visits to verify bar force levels obtained from the load cells.

Load Test Behavior

To determine the live-load behavior of the bridge under vehicle loading, two static load tests were conducted during the monitoring period. The first load test was completed immediately following the third stressing on January 24, 1995. The second load test was completed January 28, 1997, 2 years after the first load test.

The static load test consisted of positioning one or two fully loaded trucks on the bridge, then measuring the resulting deflections along the transverse centerline of the bridge. Deflection measurements from an unloaded to loaded condition were obtained by hanging calibrated rules on the underside of the deck and reading values with a surveyor's level. The accuracy of this method for repetitive readings is estimated to be ±1 mm (±0.04 in.).

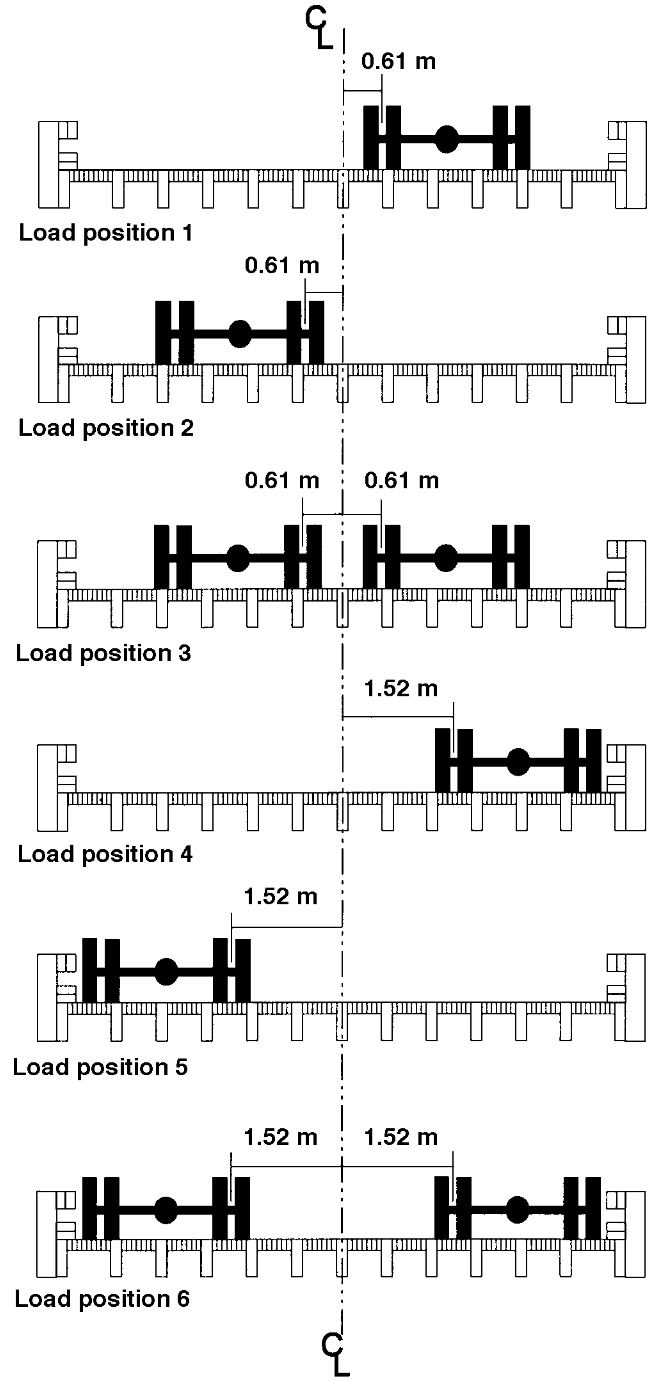

Figure 11—Transverse load positions (looking north) used for all static load tests on the North Siwell Road bridge.

For both load tests, the trucks were positioned for six transverse load positions: three centric and three eccentric (Fig. 11). The first and second load positions placed one vehicle on the bridge near the longitudinal centerline in opposite lanes. The third load position placed both vehicles on the bridge in the same positions as load positions 1 and 2. The fourth and fifth load positions placed one vehicle on the bridge near each rail. The sixth load position placed both vehicles on the bridge in the same positions as load positions 4 and 5 (Fig. 12).

Figure 12—Transverse load test positions used for both load tests: (a) load position 1, 0.6 m upstream of longitudinal center line; (b) load position 2, 0.6 m downstream of longitudinal center line; (c) load position 3, both trucks in same locations as load positions 1 and 2; (d) load position 4, 1.5 m upstream of longitudinal center line; (e) load position 5, 1.5 m downstream of longitudinal centerline; (f) load position 6, both trucks in same locations as load positions 4 and 5.

Load Test 1

The test vehicles were fully loaded, three-axle dump trucks with gross vehicle weights of 268.8 kN (60,440 lb) for truck 1A and 273.9 kN (61,570 lb) for truck 1B (Fig. 13). The vehicles were positioned longitudinally with the transverse centerline of the bridge, bisecting the tandem rear axles and the front axles off the bridge. Both trucks faced north for all load positions. Data points were positioned along the transverse centerline at each glulam web.

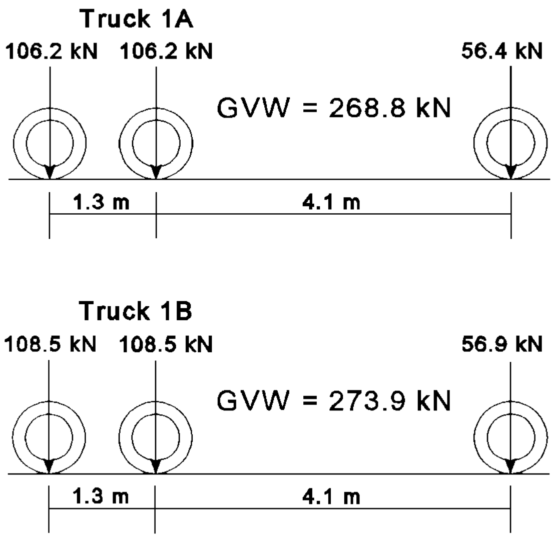

Figure 13—Load test truck configurations and axle loads for load test 1. All vehicle track widths, measured center–center of the rear tires, were 1.8 m (6 ft).

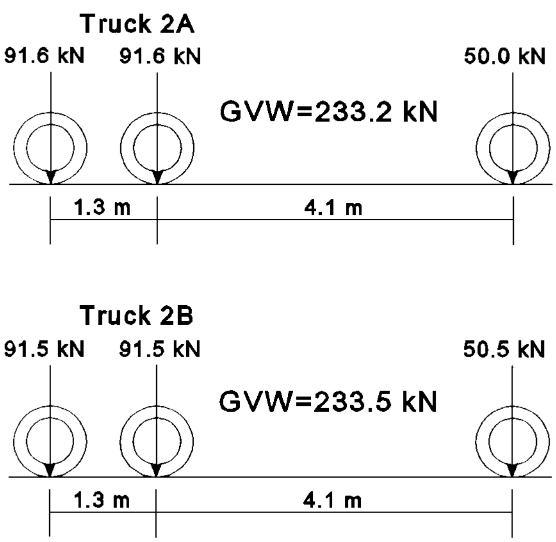

Figure 14—Load test truck configurations and axle loads for load test 2. All vehicle track widths, measured center–center of the rear tires, were 1.8 m (6 ft).

Load Test 2

The test vehicles were fully loaded, three-axle dump trucks with gross vehicle weights of 233.2 kN (51,950 lb) for truck 2A and 233.5 kN (52,490 lb) for truck 2B (Fig. 14). The vehicles were positioned longitudinally with the transverse centerline of the bridge, bisecting the tandem rear axles and the front axles off the bridge. Both trucks faced north for all load positions. Data points were positioned along the transverse centerline at each glulam web.

Predicted Deflection Analysis

At the conclusion of load testing, predicted deflections were calculated for AASHTO HS20–44 loading. The procedure was based on the measured load test deflection and a ratio of deflection coefficients (DCs) as determined through computer analysis (Murphy 1994). The following relationship was established to find the predicted deflection under HS20 loading:

$$\Delta_{\mathrm{HS20}} = \Delta_{\mathrm{Load\ test}} \left(\frac{DC_{\mathrm{HS20}}}{DC_{\mathrm{Load\ test}}} \right)$$

where

Δ_{HS20} is HS20 predicted deflection (mm),
$\Delta_{\mathrm{Load\ test}}$ maximum measured load test deflection (mm),
DC_{HS20} HS20 deflection coefficient ($\mathrm{kN{\cdot}m^4}$), and
$DC_{\mathrm{Load\ test}}$ load test vehicle deflection coefficient ($\mathrm{kN{\cdot}m^4}$).

Condition Assessment

The general condition of the bridge was assessed at the time of the two load tests. The assessments involved visual inspections, measurements, and photograph documentation of the bridge. Items of specific interest included geometry, wood condition, wearing surface, and stressing system.

Results and Discussion

Performance monitoring of the North Siwell Road bridge extended for 24 months, beginning in January 1995. Results and discussion of the performance data follow.

Moisture Content

The average trend in electrical resistance moisture content readings is shown in Figure 15. As shown, there are slight variations between the readings for the glulam webs and the sawn lumber flanges. At the beginning of the monitoring period, the glulam web moisture content was approximately 12.5%. During the monitoring period, the moisture content exhibited seasonal changes of ±5%. At the end of monitoring, the moisture content was approximately 15%. The sawn lumber flange moisture content was slightly higher at the start of monitoring at approximately 17%. During the first year of the monitoring, the sawn lumber flange moisture content varied similarly at ±5%. Readings were not taken during the second half of the monitoring period except during the final quarter of 1996. Near the end of the monitoring period, the sawn lumber flange moisture content was approximately 12.5%. Both the glulam webs and the sawn lumber flange were installed at a moisture content less than the recommended value of 19%.

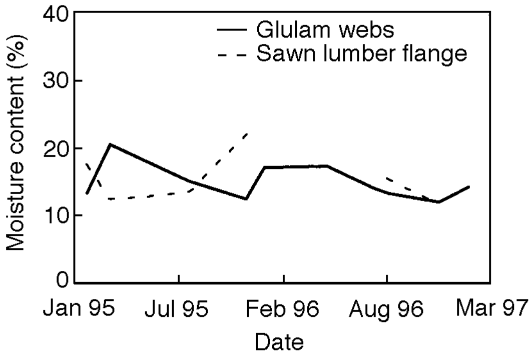

Figure 15—Average trend in electrical resistance moisture content readings.

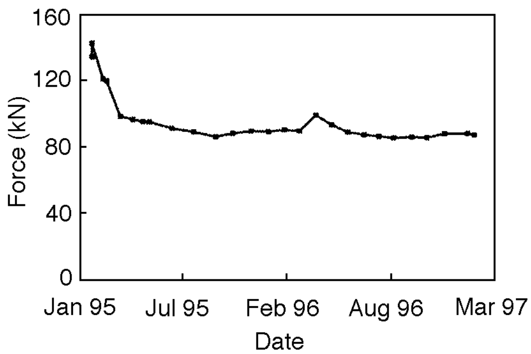

Figure 16—Average bar force from load cells installed after the third bar tensioning.

Bar Force

The average bar force based on load cell readings is shown in Figure 16. At the third stressing, all bars were tensioned to 142 kN (32,000 lb), which corresponds to a 848 kPa (123 lb/in^2) interlaminar stress. Two months later, the bar force decreased to 98.2 kN (22,100 lb) or 579 kPa (84 lb/in^2). The rate of loss declined, and the bar force then remained relatively stable for the remainder of the monitoring period. At the conclusion of monitoring, the average bar force was 87.6 kN (19,700 lb), which corresponds to a level of 490 kPa (71 lb/in^2) interlaminar compression stress.

The observed bar force loss is most likely the result of stress relaxation in the sawn lumber flange laminations. Stress relaxation is a time-dependent phenomenon caused by the long-term compressive force of the steel bars acting on the wood microstructure. In the design of stress-laminated bridges, it is assumed that the bridges will lose approximately 60% of the interlaminar compression over the life of the structure as a result of stress relaxation. The stress relaxation behavior of this bridge was similar to previous monitored bridges (Wacker and Ritter 1995; Ritter and others 1995, 1996). However, the amount of stress relaxation in the North Siwell Road bridge was affected by the wood moisture content. Recent monitoring completed on a bridge installed with a moisture content greater than 19% (Kainz and others 1996) showed that the bar force level decreased at a higher rate than did the North Siwell Road bridge where the wood moisture content was less than 19% at installation.

The amount of initial interlaminar compression stress can also affect the bar force performance. In this case, 47% of initial interlaminar compression stress was lost during the monitoring period, which is similar to other bridges. The initial interlaminar compression stress level was higher on the North Siwell Road bridge than other typical stress-laminated bridges. Thus, the final interlaminar compression stress level remained within acceptable levels.

Stress-laminated T-beam bridges can experience slip between the web and flange when the interlaminar compression drops below 345 kPa (50 lb/in^2). The final interlaminar compression stress of 490 kPa (71 lb/in^2) for the North Siwell Road bridge is well above this level. The North Siwell Road bridge has performed well during the 2 years of monitoring; therefore, it is assumed that with standard periodic maintenance (i.e., bar force checks, routine inspections), the bar force level should remain acceptable.

Load Test Behavior

In this section, results of the static load tests and analytical assessment of the North Siwell Road bridge are presented. For each load position, transverse deflection measurements are given at the transverse mid-span as viewed from the east end (looking west). No permanent residual deformation was measured at the conclusion of load testing, and there was no detectable movement at bridge supports. At the time of load test 1, the interlaminar compression stress was approximately 841 kPa (122 lb/in^2). For load test 2, the interlaminar compression stress was approximately 490 kPa (71 lb/in^2).

Load Test 1

Transverse deflections for load test 1 with the locations and magnitudes of the maximum measured deflections are shown in Figure 17. The maximum measured deflection was 9 mm (0.35 in.) for load position 1, 8.5 mm (0.34 in.) for load position 2, and 11.5 mm (0.45 in.) for load position 3. For the eccentric load positions, the maximum measured deflection was 8.5 mm (0.34 in.) for load positions 4 and 6 and 8.3 mm (0.33 in.) for load position 5. As shown in Figure 17, the maximum measured deflections occurred near the wheel lines of the test vehicles. These maximum measured deflections are symmetrical between similar load positions, such as load positions 1 and 2 and load positions 4 and 5. For the load positions with two vehicles, the deflection from load position 3 was greater than all other measured deflections, and the smaller deflection from load position 6 was similar to load positions 1, 2, 4, and 5. This was due to the transverse stiffness of the deck that was created by the interlaminar compression stress.

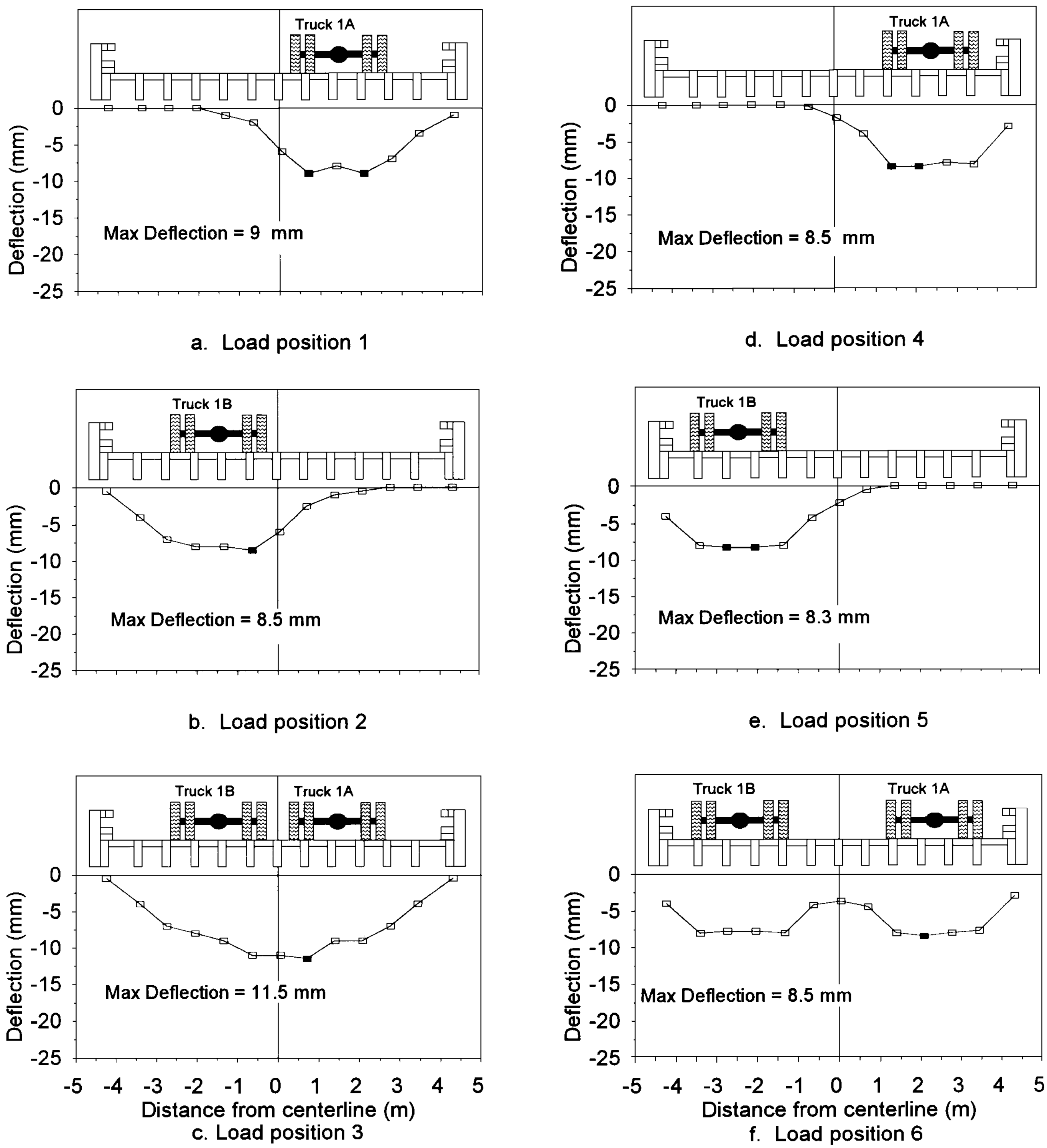

Figure 17—Transverse deflections measured at mid-span (looking north) for load test 1. Bridge cross-sections and vehicle positions are presented to aid interpretation and are not to scale.

Assuming uniform material properties, symmetric loading, and accurate deflection measurements, the summation of bridge deflections from two single-truck load positions should equal the deflections from the load position with both trucks. Figure 18 displays the summation of load positions 1 and 2 overlaid on load position 3. As shown, there are slight deflection variations near the longitudinal centerline, but the deflections are essentially the same.

Load Test 2

Load test 2 transverse deflections are shown in Figure 19. The maximum measured deflection was 7.4 mm (0.29 in.) for load position 1, 7.3 mm (0.29 in.) for load position 2, and 9.3 mm (0.37 in.) for load position 3. The maximum measured deflections for load positions 4 through 6 were 8.3 mm (0.33 in.) for load position 4, 7.2 mm (0.28 in.) for load position 5, and 7.6 mm (0.30 in.) for load position 6. Observations from load test 1 were also made for load test 2. The maximum measured deflections occurred near the wheel lines of the test vehicles. These maximum measured deflections were symmetrical between similar load positions, such as load positions 1 and 2 and load positions 4 and 5. For the load positions with two vehicles, the deflection from load position 3 was greater than all other measured deflections, and the smaller deflection from load position 6 was similar to load positions 1, 2, 4, and 5. Again, this was due to the transverse stiffness of the deck that was created by the interlaminar compression stress.

As for load test 1, the summation of bridge deflections for two single truck load positions should equal the deflections from the load position with both trucks. Figure 20 displays the summation of load positions 1 and 2 overlaid on load position 3 for load test 2. Aside from slight deflection variations, the deflections are essentially the same.

Load Test Comparison

An examination of the results from load tests 1 and 2 revealed similar maximum deflection locations for each load position (Fig. 21). As shown, the second load test exhibited smaller deflections than the first load test. The decreased deflections were due to 15% lighter loading on the second load test. If the deflections for load test 2 were factored to account for the 15% load reduction, they would appear very similar to the deflections for load test 1.

On most stress-laminated bridges with butt joints, there is a correlation between the transverse stiffness of the bridge and the level of interlaminar compression stress. It has been observed on several stress-laminated bridges that the transverse stiffness decreases when the level of interlaminar compression in the deck decreases (Wacker 1996). The change in interlaminar compression seemed to have little effect on the transverse stiffness of the North Siwell Road bridge.

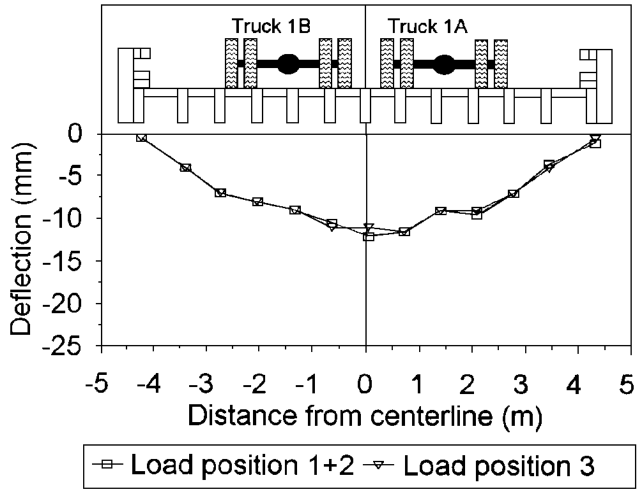

Figure 18—Comparison of measured deflections for load test 1, showing the actual deflection of load position 3 and the sum of load positions 1 and 2 (looking north).

Predicted Deflection Analysis

To compare the North Siwell Road bridge with other bridges, a theoretical deflection based on a standard HS20–44 truck was determined. Using the method previously described, maximum deflections of 11 mm (0.43 in.) for load test 1 and 10.4 mm (0.41 in.) for load test 2 were determined for load position 3. These deflections correspond to L/809 for load test 1 and L/849 for load test 2, which are significantly less than the minimum design deflection criteria of L/500 and the actual design deflection of L/579.

Condition Assessment

Condition assessments of the North Siwell Road bridge indicated that structural performance and serviceability were good. Inspection results for specific items follow.

Bridge Geometry

Measurements obtained during site inspections reveal that the bridge was slightly narrower at mid-span than at the abutments. This behavior, commonly called “hour glassing” is a result of reduction of lamination size caused by compressive deformation (creep) over time at mid-span and resistance to movement provided by the attachment of the webs at the abutments. Hour glassing does not affect the structural performance of the bridge but can have an adverse aesthetic effect, if severe. The hour glassing on the North Siwell Road bridge is minor and generally not noticeable.

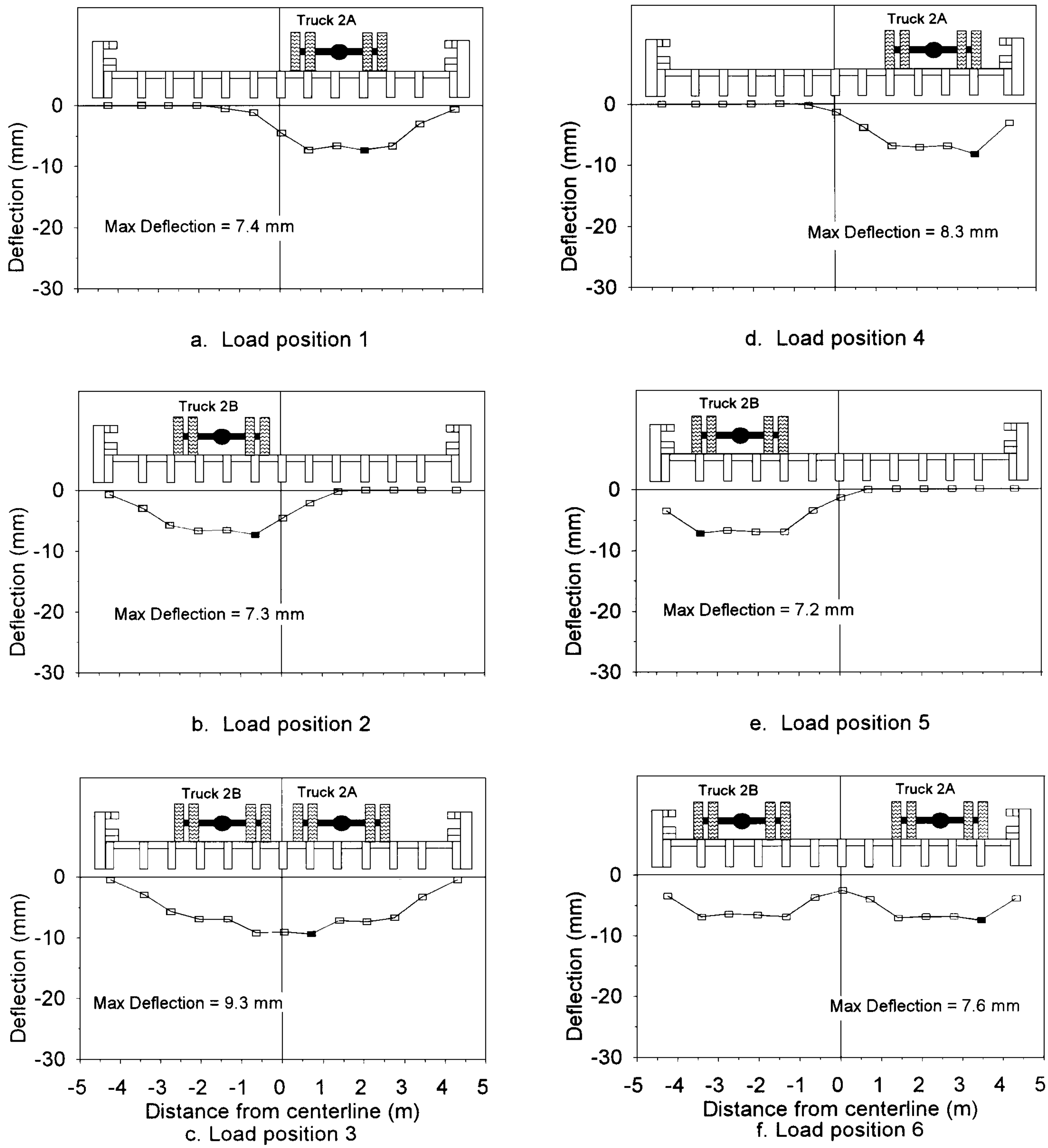

Figure 19—Transverse deflection measured at mid-span (looking north) for load test 2. Bridge cross-sections and vehicle positions are presented to aid interpretation and are not to scale.

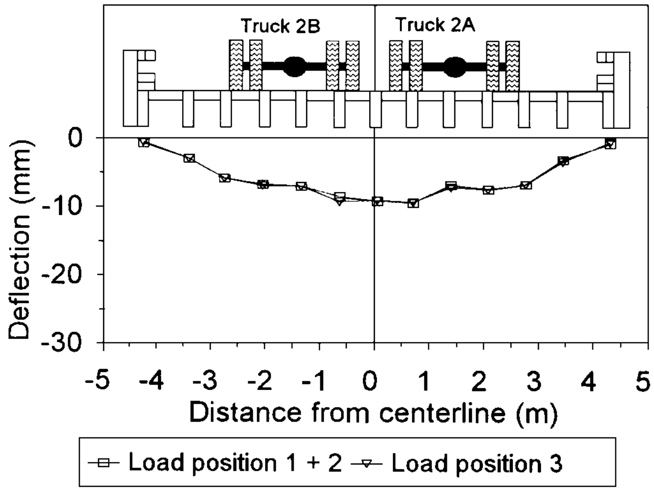

Figure 20—Comparison of measured deflections for load test 2, showing the actual deflection of load position 3 and the sum of load positions 1 and 2 (looking north).

Wood Condition

Inspection of the wood components of the bridge showed no sign of deterioration. Minor checking was evident on the end grain of the rail posts exposed to rapid wet/dry cycles. This checking could have been alleviated through the use of a sealer on the post end grain.

Preservative Treatment

On the tension side of several webs, the preservative treatment appeared to be missing or spotty (Fig. 22). This may be a result of banding blocks left in place during the preservative treatment process. The flange and web members exhibited no signs of excess preservative treatment and no leaching was observed. However, excess preservative has migrated to the surface of the Douglas Fir posts and rails.

Wearing Surface

The approach roadways were completed approximately 6 months after bridge installation by applying a new wearing surface that was feathered over abutments but was not extended across the length of the bridge. Although there appears to be no cracks and degradation of the original tack coat wearing surface, the surface allows water to penetrate the flange and upper portions of the webs. During the final site visit, water was observed flowing through the butt joints while it was raining. The increased water in the laminations could lead to premature deterioration in these areas.

Anchorage System

The stressing bar anchorage system has performed adequately. No measurable distortion in the bearing plates was observed, and the exposed galvanized steel stressing bars, hardware, and anchorage plates showed no visible sign of corrosion or other deterioration. There was no apparent crushing of the bearing plates into the Southern Pine glulam exterior laminations.

Conclusions

After 24 months in service, the North Siwell Road bridge is performing satisfactorily. Based on extensive bridge monitoring during this period, we make the following observations and recommendations:

- The glulam and sawn lumber components of the North Siwell Road bridge were initially installed at 12.5% and 17% moisture content, respectively. The average trend in moisture content indicates that the glulam webs and sawn lumber flanges have experienced little variation during the monitoring period but had some fluctuation caused by seasonal change.

  Following the final design bar tensioning, the bridge lost approximately 47% of the bar force introduced to end at 87.6 kN (19,700 lb), which corresponds to 490 kPa (71 lb/in^2) interlaminar compression stress. This decrease in bar force was most likely attributable to transverse stress relaxation in the wood laminations. The acceptable bar force level at the end of monitoring is a result of retardation of the stress relaxation rate caused by the low moisture content in the wood laminations. The introduction of a high initial bar force level that resulted in a high interlaminar compressive stress also contributed to the acceptable ending bar force performance of this bridge. To maintain this performance, bar force should be checked as part of the routine inspection program.
- Load testing and analysis indicate that the North Siwell Road bridge is performing in a linear elastic manner when subjected to truck loading. The simulated HS20–44 truck loading conditions produced maximum deflections of 11 mm (0.43 in.) for load test 1 and 10.4 mm (0.41 in.) for load test 2. The similar deflections for load tests 1 and 2 correspond to approximately L/830 and are based on center–center bearing lengths of the span. The deflections are substantially smaller than the target design deflection of L/500 for this span.
- Wood checking is evident in the exposed end grain of the rail posts. It is likely this would not have occurred if a sealer had been placed over the end grain at the time of construction. A sealer could be applied at this time to prevent additional damage in the post end grain.
- There are no indications of corrosion on the galvanized steel stressing bars, hardware, or plates.

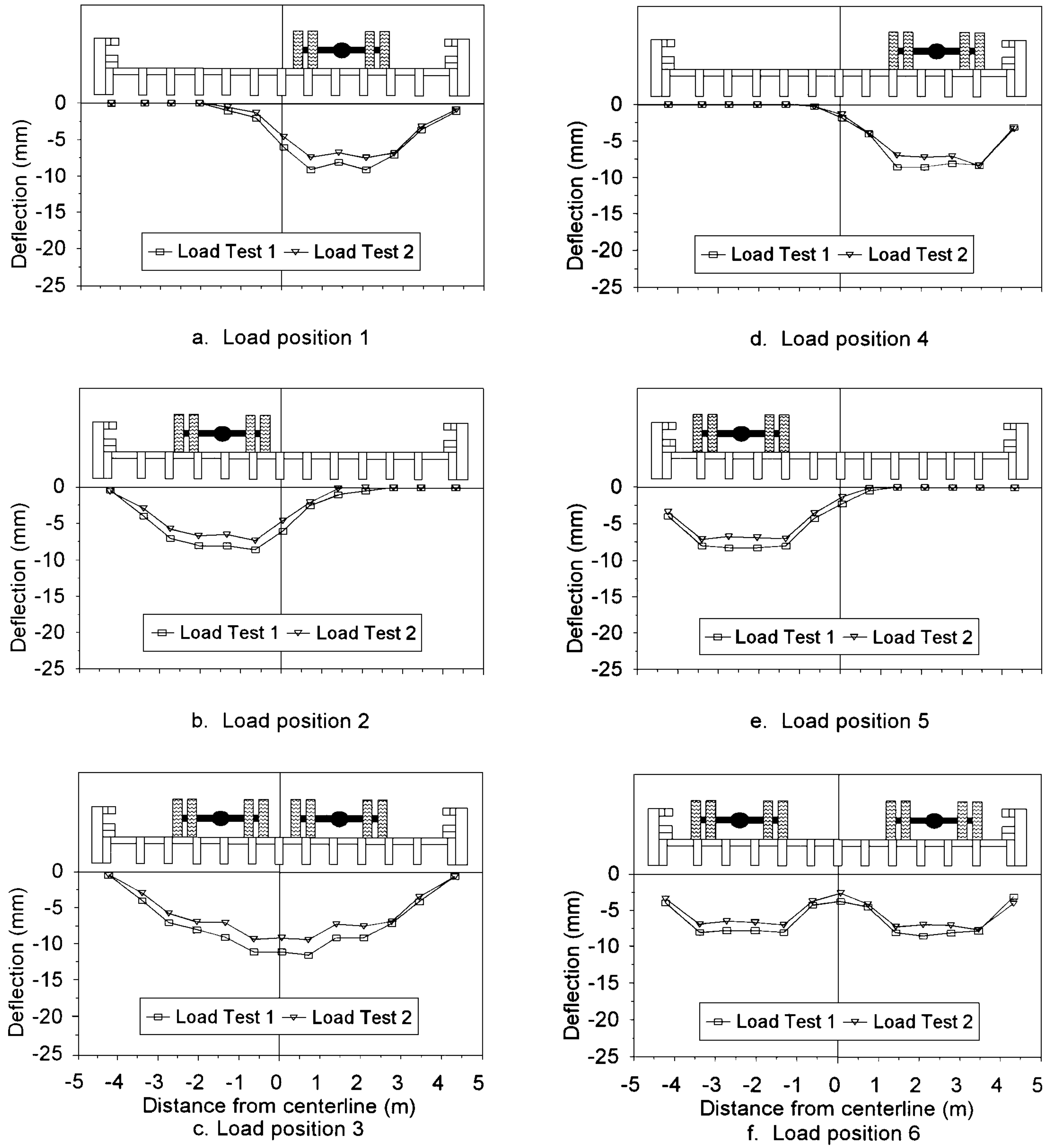

Figure 21—Comparison of load tests 1 and 2 at all load positions.

Figure 22—Area with no preservative penetration, probably a result of material banding during treatment.

- The asphalt tack coat wearing surface provides little or no moisture protection for the deck. During the second load test, water was observed flowing through the butt joints. Direct contact with water could increase the moisture content in these areas and could lead to premature deterioration. It is recommended that the bridge be resurfaced in the near future to prevent moisture flow onto the timber members.
- The preservative treatment of the bridge is adequate except for several areas on the underside of the flange that appear to be void of preservative treatment. The areas appear to be on the corners of the T or L sections and are most likely a result of banding. The small areas should pose no threat to the preservative envelope as a result of their location on the underside of the bridge. Eventually, these areas should be field treated with a preservative compound or a coating.

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Appendix—Information Sheet

General

Name: North Siwell Road bridge

Location: Hinds County, Mississippi

Date of Construction: December 1994

Owner: Hinds County

Design Configuration

Number of Spans: 1

Structure Type: Stress-laminated T with glulam webs and sawn lumber butt-jointed flanges

Butt-Joint Configuration: 1 in 4 transverse laminations with joints in adjacent deck flange laminations separated 1.2 m (4 ft) longitudinally

Total Length (out–out): 9.1 m (30 ft)

Skew: None

Span Lengths (center–center bearings): 8.5 m (28 ft)

Bearing Lengths: 305 mm (12 in.) full bearing, no overhang

Width (out–out): 8.8 m (28.7 ft)

Width (curb–curb): 8.2 m (26.8 ft)

Number of Traffic Lanes: 2

Design Loading: AASHTO HS20–44

Wearing Surface Type: Asphalt tack coating; 25.4- to 50.8-mm (1- to 2-in.) thickness

Material and Configuration

Flange Laminations:

Species: Southern Pine sawn lumber

Size: standard 38 by 184 mm (nominal 2 by 8 in.)

Grade: No. 1

Moisture Condition: Approximately 17% at installation

Webs:

Species: Southern Pine Glulam

Size (actual): 171.5 by 593.7 mm (6.75 by 23.375 in.)

Beam Designation: 24F–V3 SP/SP

Moisture Condition: Approximately 12.5% average at installation

Rails:

Species: Douglas Fir sawn lumber

Size (actual): 152 by 269 mm (6 by 10.6 in.)

Posts:

Species: Douglas Fir sawn lumber

Size (actual): 191 by 305 mm (7.5 by 12 in.)

Curb and Scupper:

Species: Douglas Fir sawn lumber

Size (actual): 133 by 292 mm (5.25 by 11.5 in.)

Grade: No. 1

Preservative Treatment: Creosote

Stressing Bars:

Type: High strength steel thread bar with coarse right-hand thread, conforming to ASTM A 722

Diameter: 15.9 mm (5/8 in.)

Number: 10

Design Force: 155.7 kN (35,000 lb)

Spacing: 914 mm (36 in.) center–center beginning 457 mm (18 in.) from ends of bridge

Anchorage Type and Configuration:

Discrete Plate: Anchor plate and nut: 51 by 127 by 25.4 mm (2 by 5 by 1 in.) plate and flat hex nut

Bearing plate: 191 by 393.7 by 25.4 mm (7.5 by 15.5 by 1 in.)