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Field Performance of Timber Bridges

6. Hoffman Run Stress-Laminated Deck Bridge

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Abstract

The Hoffman Run bridge, located just outside Dahoga, Pennsylvania, was constructed in October 1990. The bridge is a simple-span, single-lane, stress-laminated deck superstructure that is approximately 26 ft long and 16 ft wide. It is the second stress-laminated timber bridge to be constructed of hardwood lumber in Pennsylvania. The performance of the bridge was monitored continually for approximately 32 months, beginning shortly after installation. Performance monitoring involved gathering and evaluating data relative to the moisture content of the wood deck, the force level of steel stressing bars, the deck vertical creep, and the behavior of the bridge under static-load conditions. Furthermore, comprehensive visual inspections were executed to assess the overall condition of the structure. Based on field evaluations, the bridge is performing properly with no structural deficiencies, although with respect to serviceability, the bridge has developed a slight sag at midspan.

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6. Hoffman Run Stress-Laminated Deck Bridge

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Introduction

In 1988, Congress passed legislation known as the Timber Bridge Initiative (TBI). The intent of this legislation was to advance development and expand the use of timber as a structural material for highway bridges. Responsibility for administering the TBI was delegated to the USDA Forest Service and included a demonstration timber bridge program managed by the Timber Bridge Information Resource Center (TBIRC) in Morgantown, West Virginia, and a research program at the USDA Forest Service, Forest Products Laboratory (FPL), in Madison, Wisconsin (USDA 1994). In addition, the Forest Service National Forest System, which manages the national forests of the country, made a commitment to demonstrate new technology in timber bridge design and construction. Because of this commitment, many local municipalities became aware of the technology and found it to be a viable method for renewing aging infrastructures.

This report, the sixth in a series of bridge performance documents, describes the development, design, construction, and field performance of the Hoffman Run bridge near Dahoga, Pennsylvania, Elk County, Jones Township. The Hoffman Run bridge was constructed in October 1990 and was a direct result of the success of the Little Salmon Creek bridge (Ritter and others 1996) on the Allegheny National Forest in Pennsylvania. The Hoffman Run bridge is a single-lane, simple-span, stress-laminated deck that is approximately 26 ft long and 16 ft wide. (See Table 1 for metric conversion factors.) It is the second stress-laminated deck superstructure constructed of hardwood lumber in Pennsylvania. Characteristics of the bridge are summarized in the Appendix.

Background

The Hoffman Run bridge is located just outside Dahoga, Pennsylvania (Fig. 1). It is on County Road T364, a double-lane, gravel roadway that provides access to local residences and recreation areas on the Allegheny National Forest.

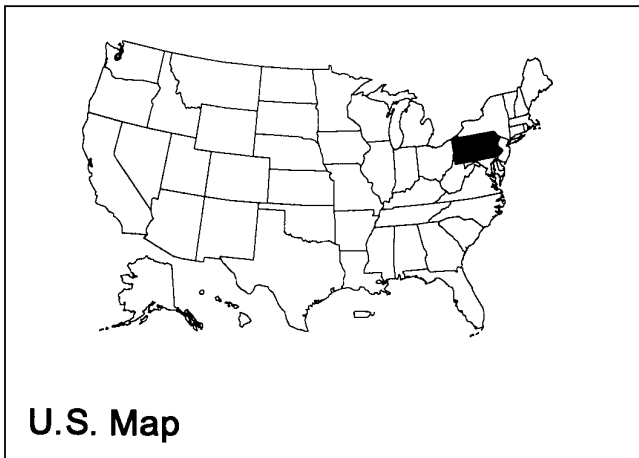
Table 1—Factors for converting English units of measurement to SI units

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

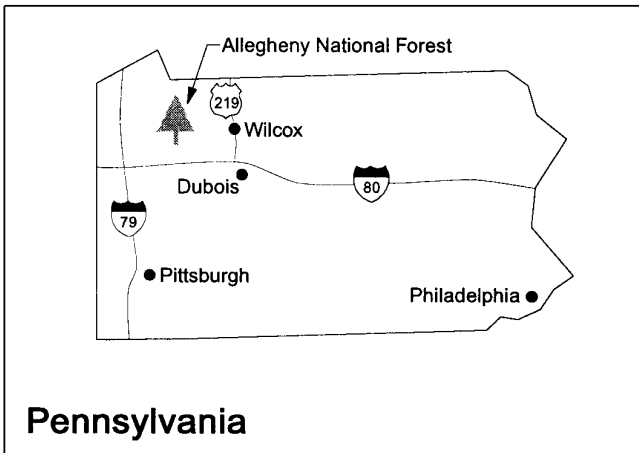
Traffic consists of light-passenger vehicles, recreational vehicles, school busses, and occasionally logging trucks. The average daily traffic is estimated between 40 and 50 vehicles per day.

The original Hoffman Run bridge was constructed in the early 1930s and consisted of a timber plank deck supported by steel girders (Fig. 2). Inspection of the bridge in the 1980s indicated that the steel girders were in poor condition and insufficient to carry modern traffic loads. Because of these deficiencies, a decision was made by Jones Township to replace the bridge.

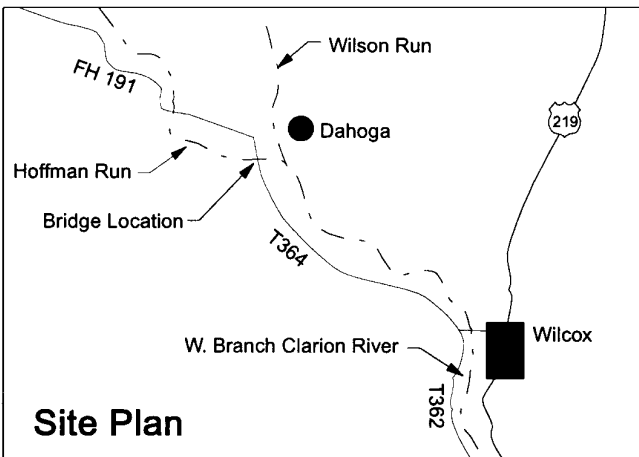
Based on the success of the Little Salmon Creek bridge built on the Allegheny National Forest in 1988, Jones Township decided to replace the Hoffman Run bridge with a stress-laminated timber bridge using mixed hardwood lumber. This type of timber bridge is an attractive alternative because it is inexpensive and can be built using local lumber and a township construction crew. At the time of planning for the bridge, stress-laminated timber bridges were relatively new in the United States. To evaluate bridge performance after construction, both Jones Township and the Pennsylvania Department of Transportation determined that the field performance of the bridge should be monitored to provide assurance that the performance of the system was acceptable. Subsequently, FPL and Jones Township entered into an agreement to complete structural monitoring of the bridge.



U.S. Map



Pennsylvania



Site Plan

Figure 1—Location and site layout of the Hoffman Run bridge.

Objective and Scope

The objective of this project was to ascertain the field performance characteristics of the Hoffman Run stress-laminated bridge by monitoring the bridge for approximately 2-1/2 years, beginning shortly after bridge installation. The project scope included data collection and analysis related to the wood moisture content, stressing bar force, vertical bridge creep, behavior under static truck loading, and general structure condition. The results of this project will be considered with similar monitoring activities in an effort to improve design and construction methods for future stress-laminated timber bridges.

Design and Construction

Design

The design of the Hoffman Run bridge was based on that of the Little Salmon Creek bridge, located in the Allegheny National Forest. The bridge was designed before a nationally recognized design procedure for stress-laminated timber bridges was available in the United States. The design criteria for the bridge aspects relating directly to stress laminating were based on a draft version of *Timber Bridges: Design, Construction, Inspection, and Maintenance* (Ritter 1990). The remainder of the design was based on *Standard Specifications for Highway Bridges*, published by the American Association of State Highway and Transportation Officials (AASHTO 1983).

The Hoffman Run bridge was designed for AASHTO HS 20-44 loading with a span length of 25 ft center-to-center of bearings, a width of 16 ft, and a nominal deck thickness of 12 in. (Fig. 3). Mixed hardwoods, mostly beech, visually graded No. 2 and better, were selected as the material for the bridge. The design of the deck was based on nominal 2- by 12-in. mixed hardwood laminations, pressure treated with creosote. The length of the laminations varied from 3 to 16 ft in length, and butt joints in the deck laminations were placed transverse to the bridge span in every fourth lamination, with a 4-ft longitudinal spacing between butt joints in adjacent laminations (Fig. 4). The stressing system was designed for seven 1-in.-diameter, high strength, threaded steel bars conforming to the requirements of ASTM A722 (ASTM 1988). Average bar spacing was 45 in. on-center, beginning 18 in. from the bridge ends. The design bar tension force was 65,000 lb, resulting in 120 lb/in² of inter-laminar compression, which is approximately 10 percent greater than that of the Little Salmon Creek bridge. A discrete plate bar anchorage system was used, consisting of two steel plates (Fig. 5). The bridge railing consisted of a sawn lumber curb, post, and rail. Because of the low traffic volume, no wearing surface was specified.

Construction

Construction of the Hoffman Run bridge was completed by the Jones Township Road Department. Construction began by removing the existing superstructure and replacing the original abutments with concrete abutments and



Figure 2—Original Hoffman Run bridge: (top) side view; (bottom) underside of bridge and abutment.

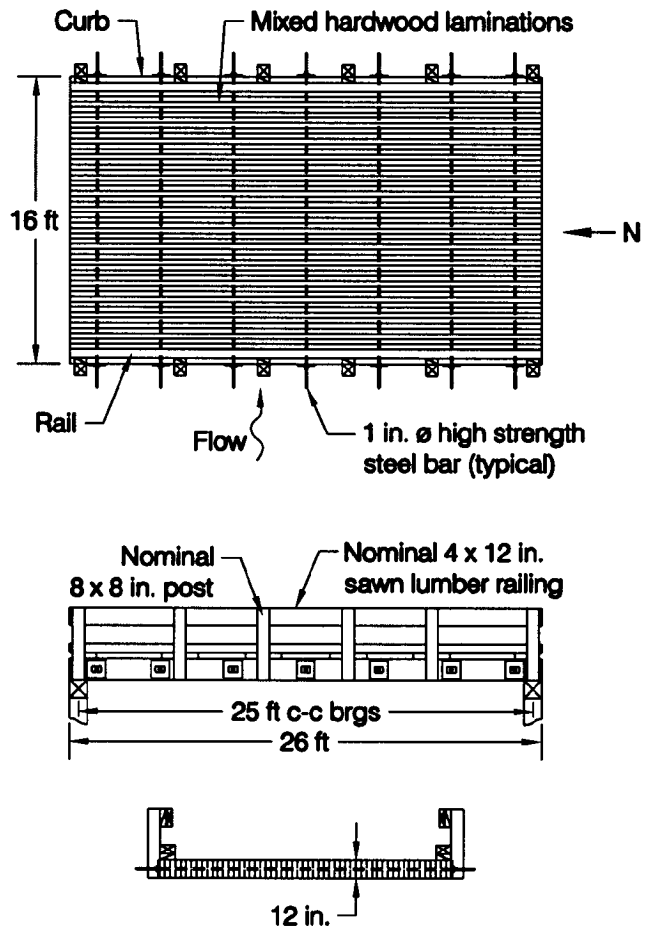


Figure 3—Design configuration of the Hoffman Run bridge.

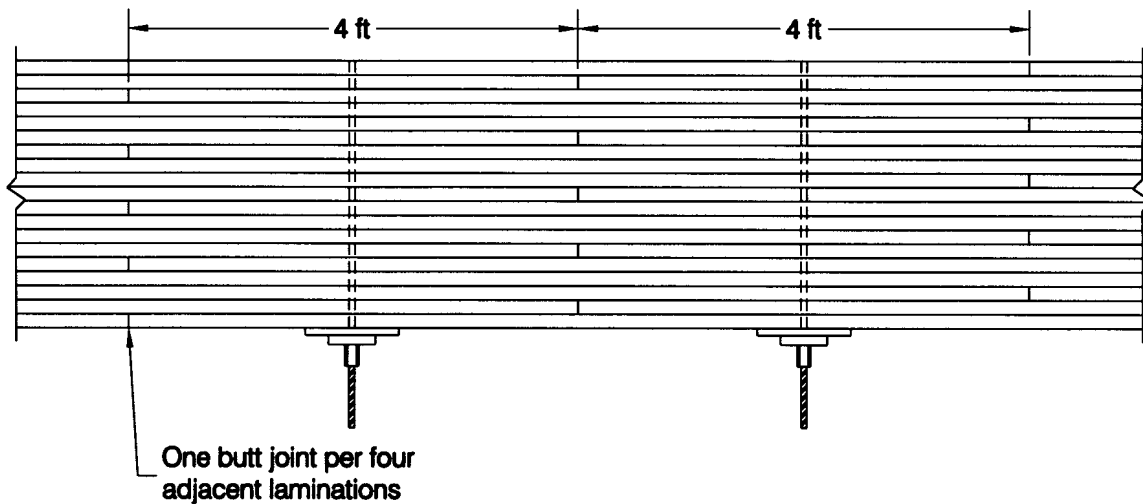


Figure 4—Butt joint configuration used for the Hoffman Run bridge. A butt joint was placed transverse to the bridge span in every fourth lamination. Longitudinally, butt joints in adjacent laminations were separated by 4 ft.

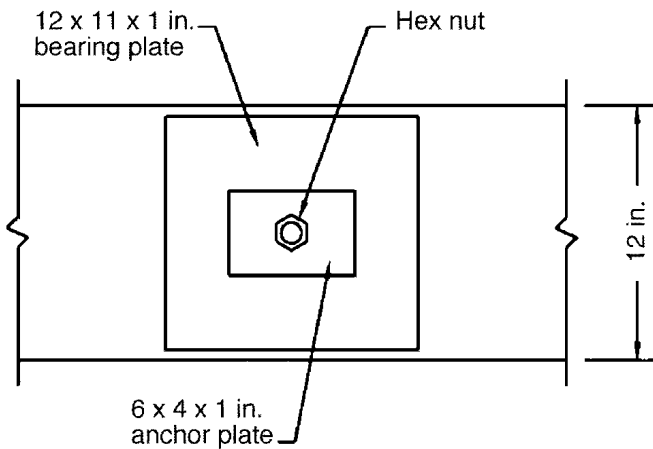


Figure 5—Details of the discrete plate bar anchorage configuration.

wingwalls (Fig. 6). Superstructure assembly began at the treating plant where treated deck laminations were nailed together to form two deck sections, each half the bridge width. Temporary steel bars were inserted through the laminations at several locations so that the deck sections could be transported to the bridge site.

Upon completion of the concrete abutments and wingwalls, the deck sections were loaded on a flatbed truck at the treating plant and transported to the bridge site (Fig. 7). At the bridge site, the sections were unloaded from the truck and placed side-by-side on temporary supports situated on the approach roadway (Fig. 8). The temporary steel bars were removed, and the stressing bars were inserted through both sections (Fig. 9). A single hydraulic jack was used to tension the bars to the required 65,000 lb (Fig. 10). Following bar tensioning, the entire superstructure was lifted as a unit by a small crane and placed atop the abutments (Fig. 11).

The installation of the bridge superstructure was completed in 1 day. After the initial bar tensioning, the bars were retensioned to 65,000 lb, approximately 1 week and 8 weeks after installation. The curb and bridge railing were installed shortly after the second bar tensioning. The completed bridge is shown in Figure 12.

The as-built configuration of the Hoffman Run bridge varied slightly from the design configuration shown in Figure 3. After the final stressing, the width of the bridge measured 16.1 ft at the abutments and midspan.

Evaluation Methodology

Because of the experimental nature of the Hoffman Run bridge, Jones Township representatives contacted FPL for assistance in evaluating the structural performance of the bridge. Through mutual agreement, a bridge monitoring plan was developed by FPL and implemented as a cooperative effort with Jones Township. The plan called for performance monitoring of the deck moisture content, bar force in



Figure 6—Completed concrete abutments and wingwalls for the Hoffman Run bridge.



Figure 7—Two deck sections for the Hoffman Run bridge were transported from the treating plant to the bridge site on a flatbed truck.



Figure 8—At the bridge site, the two deck sections were placed side-by-side atop temporary abutments located on the approach roadway.



Figure 9—The temporary steel bars in the two deck sections were removed, and stressing bars were inserted through both deck sections.



Figure 10—Stressing bars were tensioned with a single hydraulic jack.

stressing bars, vertical bridge creep, load test behavior, and condition assessment of the structure. The evaluation methodology used procedures and equipment previously developed by FPL (Ritter and others 1991).

Moisture Content

The moisture content of the Hoffman Run bridge was measured using an electrical-resistance moisture meter with 3-in. probe pins in accordance with ASTM D4444-84 procedures (ASTM 1990). Measurements were obtained by driving the pins into the underside of the deck at depths of 1 to 2 in., recording the moisture content from the unit, and adjusting the values for temperature and wood species. Moisture content measurements were taken at the time of bridge

installation, periodically during the monitoring, and at the conclusion of the monitoring.

Bar Force

Bar force for the Hoffman Run bridge was measured using load cells developed by FPL. The load cells were installed between the bearing plate and anchor plate on the second and fourth stressing bars from the north abutment on the downstream side of the bridge. Load cell measurements were obtained by Jones Township Road Department personnel using a portable strain indicator. Strain measurements were converted to units of bar tensile force by applying a laboratory calibration factor to the strain indicator reading. Bar force measurements were taken on a biweekly basis for several months following load cell installation and approximately monthly thereafter.

Vertical Creep

Vertical creep was measured at the beginning and end of the monitoring period. Measurements were obtained at midspan on the upstream deck edge using a stringline between bearings and a calibrated rule. The stringline served as a horizontal benchmark, and the relative deck elevation at midspan was measured with the rule.

Load Test Behavior

Static-load testing of the Hoffman Run bridge was conducted at the beginning and end of the monitoring period to determine the response of the bridge to truck loading. Each test consisted of positioning a fully loaded truck on the bridge deck and measuring the resulting deflections at a series of transverse locations at midspan. Measurements of bridge deflections were taken prior to testing (unloaded), for each load position (loaded), and at the conclusion of testing (unloaded). In addition, analytical assessments were conducted to determine the theoretical bridge response.

Load Test 1

The first load test was completed December 12, 1990, approximately 2 months after bridge installation. The bridge interlaminar compression at the time of the test was 114 lb/in². The test vehicle consisted of a fully loaded, two-axle dump truck with a gross vehicle weight of 30,120 lb (Fig. 13). The vehicle was positioned longitudinally on the bridge so that the rear axle was centered at midspan. Transversely, the vehicle was placed for three load positions (Fig. 14). For load position 1, the vehicle was centered on the bridge width. For load position 2, the vehicle was positioned on the downstream side with the center of the inside wheel line over the bridge centerline. For load position 3, the vehicle was positioned on the upstream side with the center of the inside wheel line over the bridge centerline. Measurements of the bridge deflection from an unloaded to loaded condition were obtained by placing calibrated rules on the deck underside and reading values with a surveyor's level to the nearest 0.06 in. (Fig. 15).



Figure 11—Completed deck placed on abutments: (a) bar tensioning completed, (b) deck lifted from temporary abutments by small crane, (c) deck positioned over Hoffman Run, (d) superstructure placed on abutments.

Load Test 2

The second load test was completed on July 19, 1993, approximately 34 months after bridge installation. At the time of the test, the interlaminar compression was approximately 70 lb/in². The test vehicle consisted of a fully loaded three-axle dump truck with a gross vehicle weight was of 46,450 lb (Fig. 16). Longitudinally, the vehicle was positioned so that the two rear axles were centered at midspan and the front axle was off the bridge span. Transversely, the vehicle was positioned for the same three load positions used for load test 1 (Fig. 14 and 17). Measurements of the bridge deflection from an unloaded to a loaded condition were taken at midspan using string potentiometers and an electronic data acquisition system (Fig. 18). The accuracy of this method for repetitive readings is estimated to be ±0.005 in.

Analytical Evaluation

Past research and investigations have indicated that stress-laminated decks can be accurately modeled as orthotropic

plates (Oliva and others 1990). To further analyze the theoretical behavior of the Hoffman Run bridge, an orthotropic plate computer model developed at FPL was used to analyze the load test results and predict the bridge deflection for AASHTO HS 20-44 truck loading. A modulus of elasticity value of 1,900,000 lb/in² was used for modeling based on estimates from unpublished in-grade lumber testing previously completed by FPL.

Condition Assessment

The overall condition of the Hoffman Run bridge was assessed on two separate occasions during the monitoring period. The evaluations occurred at the time of the load tests, which coincided with the beginning and end of the monitoring, and entailed visual inspections, measurements, and photographic documentation of the bridge condition. Items of specific interest include bridge geometry, condition of the timber deck and rail system, and condition of the stressing bars and anchorage systems.



Figure 12—Completed Hoffman Run bridge: (top) side view; (bottom) end view.

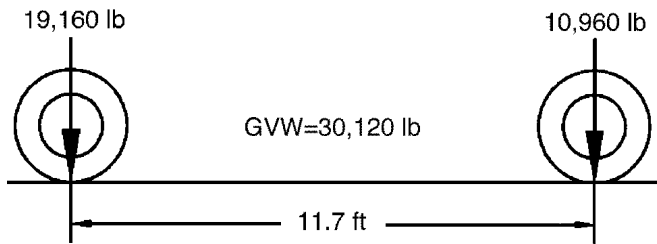


Figure 13—Load test 1 truck configuration and axle loads. Lighter axle is vehicle front. The transverse vehicle track width, measured center-to-center of the rear tires, was 6 ft.

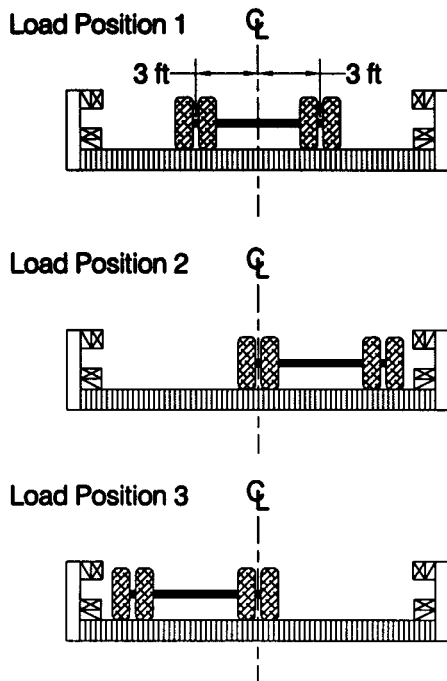


Figure 14—Load test 1 and 2 transverse load positions (looking north). For all load positions, the rear axle was centered over the bridge midspan.

Results and Discussion

The performance monitoring of the Hoffman Run bridge extended from December 1990 through July 1993. Results and discussion of the performance data follow.

Moisture Content

Average lamination moisture content of the Hoffman Run bridge was approximately 42 percent in December 1990, 2 months after installation. From that time, the bridge experienced a gradual decrease in moisture content to approximately 32 percent at the conclusion of the monitoring. The 10-percent decrease in moisture content occurred at a relatively constant rate during the 32 months, although there were fluctuations of 2 to 3 percent in the measurement zone as a result of seasonal climate changes. It is expected that the moisture content in the interior of the laminations was



Figure 15—Load test 1 bridge deflections were measured by suspending calibrated rules from the deck underside and reading displacement values with a surveyor's level.

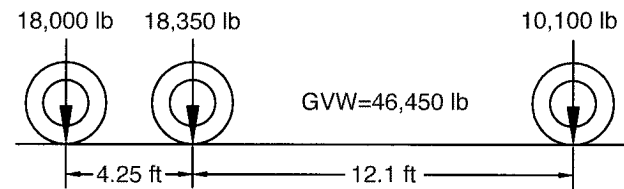


Figure 16—Load test 2 truck configuration and axle loads. The transverse vehicle track width, measured center-to-center of the rear tires, was 6 ft.

greater than the values obtained in the measurement zone as a result of slower moisture migration through the lamination depth.

Because of the high moisture content at the time of installation, the deck moisture content remained above the fiber saturation moisture content, which is approximately 25 to 30 percent. Above this level, changes in the moisture content do not cause dimensional changes in the wood laminations. In time, the moisture content will gradually decrease and eventually stabilize below the fiber saturation point. When the moisture content of the bridge falls below fiber saturation, subsequent moisture loss will result in shrinkage of the laminations and could significantly influence bar force retention.

Bar Force

The trend in average bar tension force for the Hoffman Run bridge is shown in Figure 19. The final bar tensioning, which occurred December 1990, approximately 2 months after installation, was approximately 65,600 lb, or 101 percent of the design force. After the final bar tensioning, the bar force began to decline. The decline continued throughout the monitoring, although the rate of loss decreased with time. At the conclusion of the monitoring, the average bar force was approximately 38,000 lb, or 59 percent of the design force, which corresponds to an average interlaminar compression of



Figure 17—Load test 2 transverse load positions (looking north): (top) load position 1; (middle) load position 2; (bottom) load position 3. For all load positions, the two rear axles were centered over the bridge midspan with the front axle off the span.



Figure 18—Load test 2 was measured with string potentiometers and an electronic data acquisition system.

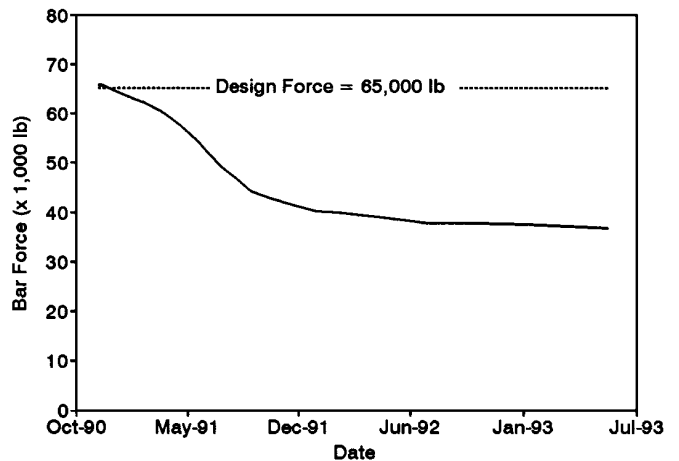


Figure 19—Average trend in bar tension force obtained from load cells installed on two stressing bars.

approximately 70 lb/in² and exceeds the recommended minimum interlaminar compression of the 40 lb/in² (Ritter 1990). Because the moisture content of the deck laminations remained greater than the fiber saturation point, there was no lamination shrinkage due to moisture loss. Thus, the decline in bar force is primarily attributed to accelerated stress relaxation in the lumber laminations as a result of the high moisture content level.

Vertical Creep

The Hoffman Run bridge was constructed without camber and was approximately level between the abutments when installed. In July 1993, at the conclusion of the monitoring period, negative camber measured 0.5 in. on the upstream bridge edge. The slight sag in the bridge is due to creep and is not typical of most stress-laminated timber bridges. It is likely that the high moisture content of the laminations and cyclic moisture changes in the exposed deck surface contributed significantly to the creep.

Load Test Behavior

Results for both load tests are presented in this section. In each case, transverse deflection plots are shown at the bridge midspan, as viewed from the south end (looking north). For each load test, no permanent residual deformation was measured at the conclusion of the testing. Furthermore, there was no detectable movement at either of the abutments.

Load Test 1

Transverse deflection values for load test 1 are shown in Figure 20. For each of the three load positions, the deflection results are typical of the orthotropic plate behavior of stress-laminated bridges (Ritter and others 1990). For load position 1 (Fig. 20a), the maximum measured deflection of 0.25 in. occurred at the bridge centerline, at 1.5 ft either side of the centerline, and beneath the wheel line on the downstream (right) side. For load position 2 (Fig. 20b), the maximum deflection of 0.25 in. was measured at the bridge centerline and at all data points up to and including the point beneath the downstream wheel line. For load position 3 (Fig. 20c), the maximum deflection of 0.25 in. was measured beneath both wheel lines and at 1.5 and 3 ft from the centerline on the upstream side of the bridge. For each case, it was impossible to determine the exact maximum deflection location given the accuracy of the readings.

Load Test 2

Transverse deflection values for load test 2 are shown in Figure 21. As with load test 1, the deflections are typical of orthotropic plate behavior. For load position 1 (Fig. 21a), the maximum measured deflection of 0.50 in. occurred beneath the downstream wheel line and 1.5 ft from the bridge centerline on the downstream (right) side. For load position 2 (Fig. 21b), the maximum measured deflection of 0.60 in. occurred beneath the downstream wheel line and the adjacent upstream data point, 17 in. from the wheel line. For load position 3 (Fig. 21c), the maximum measured deflection of 0.52 in. was under the upstream (left) wheel line. Given the accuracy of the measurement method, it is likely that the deflections accurately represent the actual bridge behavior.

Load Test Comparison

A comparison of the load position deflections for load tests 1 and 2 is shown in Figure 22. For both load positions, the plots are similar, although load test 2 deflections exceed those of load test 1. Maximum measured deflections for both load tests occurred at the same location for the respective load positions and differed by 0.25 in. for load position 1, 0.35 in. for load position 2, and 0.27 in. for load position 3. The difference in the deflections is attributable to the 17,190 lb additional rear axle load for load test 2 and the change in longitudinal bridge stiffness as a result of the difference in prestress levels at the time of load testing (discussed in Analytical Evaluation section).

Assuming linear elastic behavior and uniform material properties and loading, the bridge deflections for load positions 2 and 3 should be a mirror image. The actual load

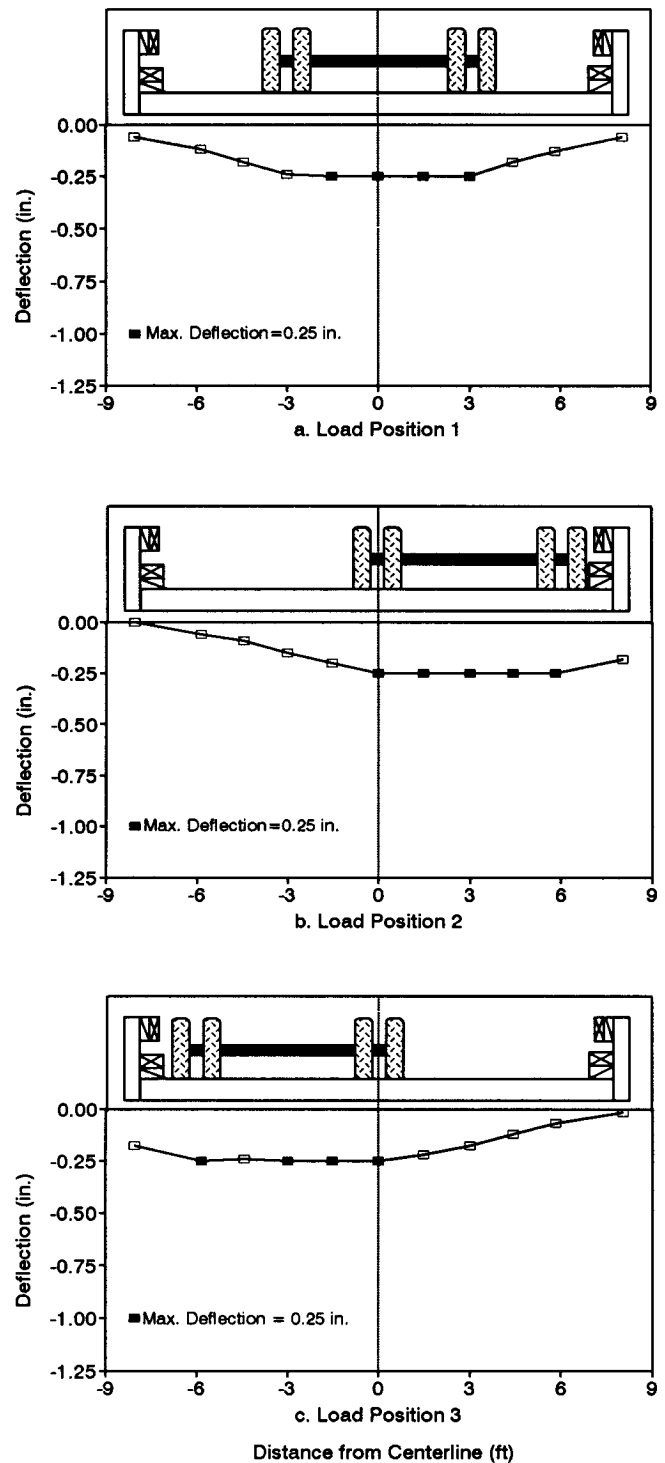


Figure 20—Transverse deflection for load test 1, measured at the bridge midspan (looking north). Bridge cross sections and vehicle positions are shown to aid interpretation and are not to scale.

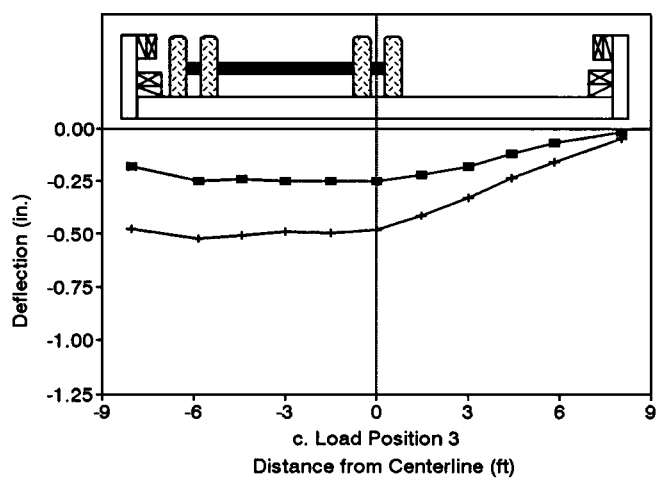
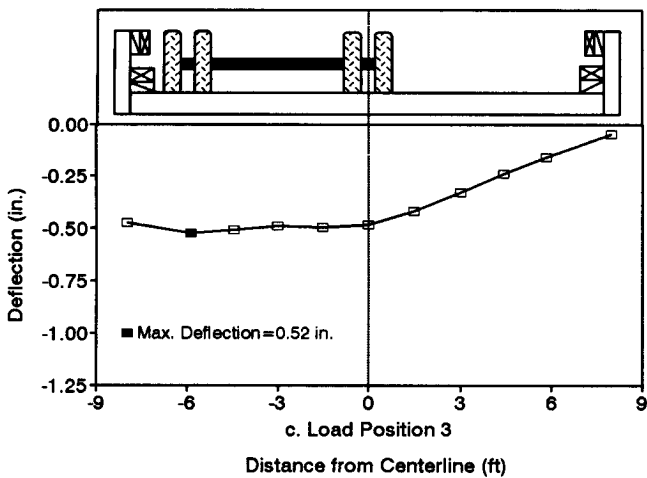
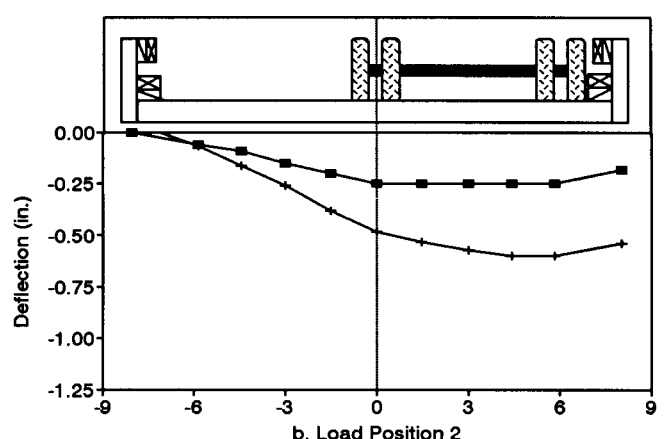
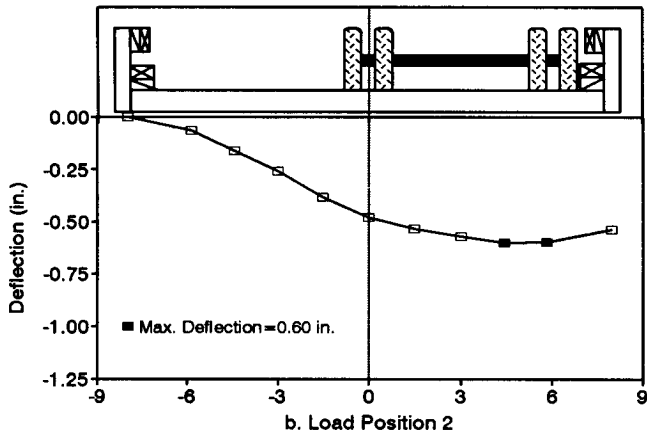
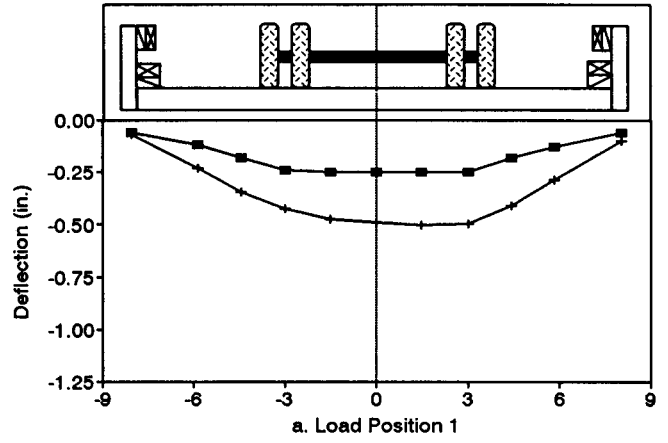
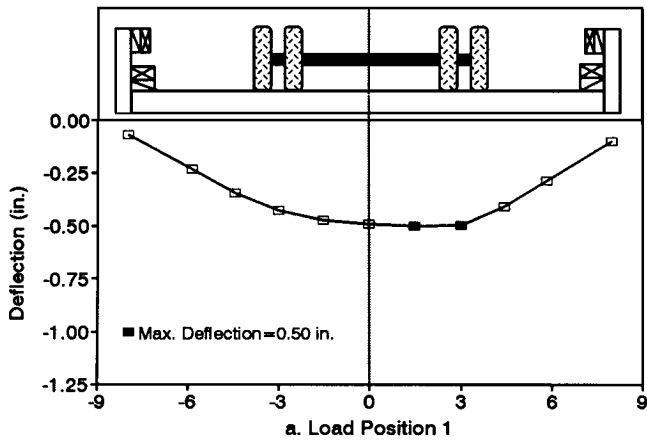


Figure 21—Transverse deflection for load test 2, measured at the bridge midspan (looking north). Bridge cross sections and vehicle positions are shown to aid interpretation and are not to scale.

Figure 22—Comparison of the measured deflections for load tests 1 and 2.

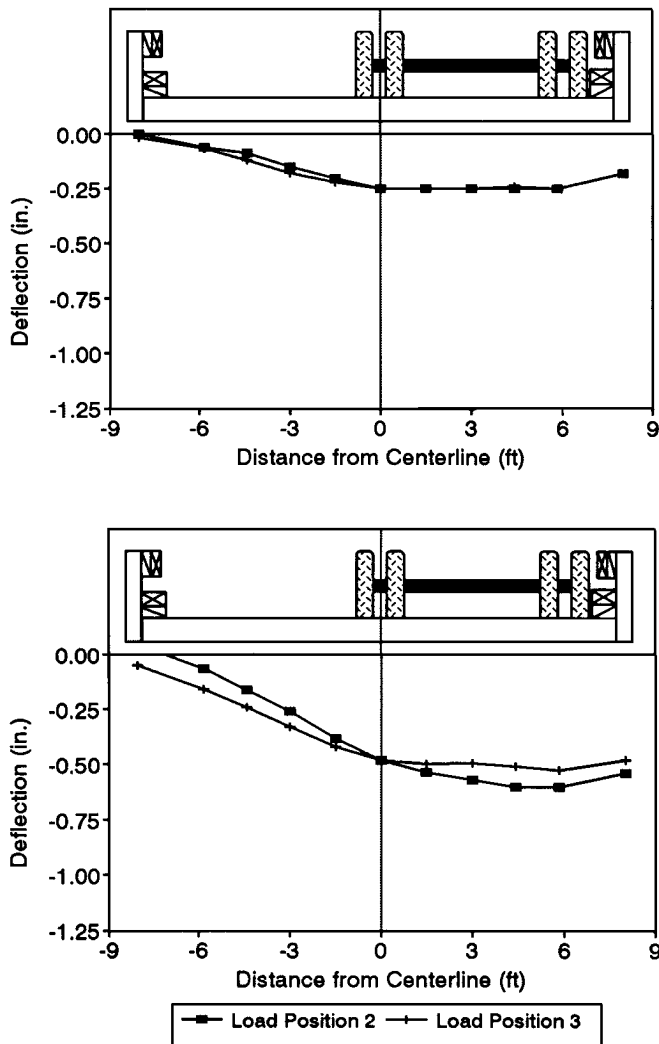


Figure 23—Comparison of measured deflections for load test 1 (top) and 2 (bottom), showing the actual deflection for load position 2 and the mirror image of load position 3 (looking north).

position 2 deflections and the mirror image of load position 3 deflections are shown in Figure 23 for each load test. As shown for load test 1, there are minor differences between the deflections, ranging from 0.01 to 0.03 in. at the bridge edges, but the plots are essentially the same. For load test 2, deflection plots vary somewhat in shape. The differences in deflection for the two load positions are probably the result of a slight difference between the wheel line loading caused by eccentric loading of the test vehicle. Load position 1 (Fig. 21) indicates that the downstream wheel line load is the greater of the two. When the heavier wheel line is placed towards the edge of the bridge, as in load position 2, it results in a larger edge deflection. For load position 3, the heavier wheel line is placed at the centerline and the edge deflection is not as large.

Analytical Evaluation

Comparisons of the measured load test results to the theoretical bridge response are shown in Figure 24. As illustrated, the theoretical bridge deflection is very close to that measured. Using the same load test analytical parameters, the theoretical deflection for AASHTO HS 20-44 truck loading is shown in Figure 25. Based on this analysis, the predicted maximum AASHTO HS 20-44 static deflection is 0.56 in. or approximately 1/536 of the bridge span for load test 1, and 0.60 in. or approximately 1/500 of the bridge span for load test 2. The theoretical maximum for each load test occurs when the truck is positioned eccentrically at the point between the wheel lines, adjacent to the eccentric wheel line.

Assuming constant bridge properties, the same bridge deflection would normally be expected for the same loading. However, it is known that an increase in interlaminar compression in stress-laminated bridges with butt joints results in an increase in longitudinal bridge stiffness (Oliva and others 1990). The difference in the maximum HS 20-44 deflection for the two load tests is attributable to an approximate 7 percent decrease in bridge stiffness at load test 2, as a result of the decreased level of interlaminar compression of 70 lb/in² for load test 2 compared with 114 lb/in² for load test 1.

Condition Assessment

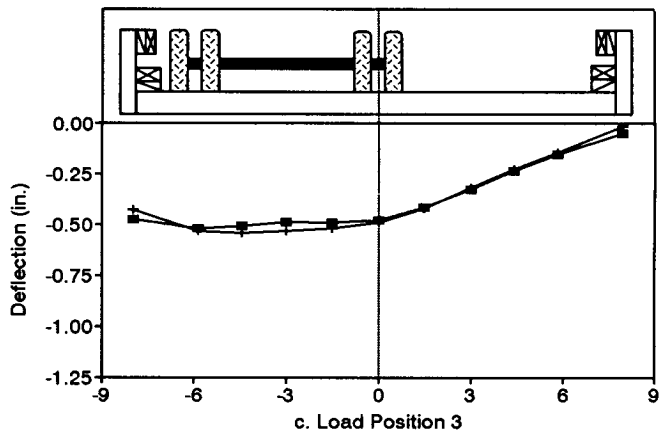
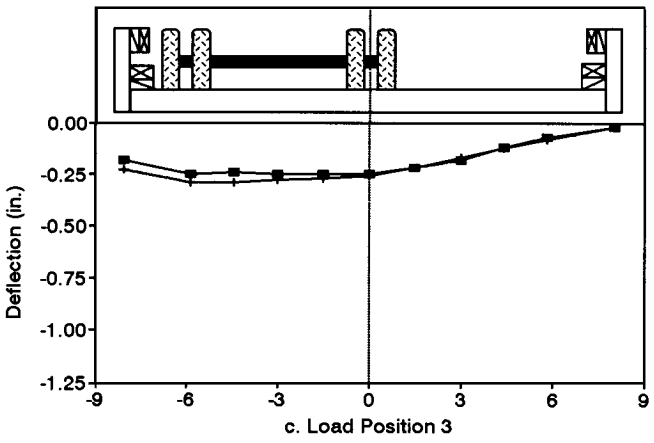
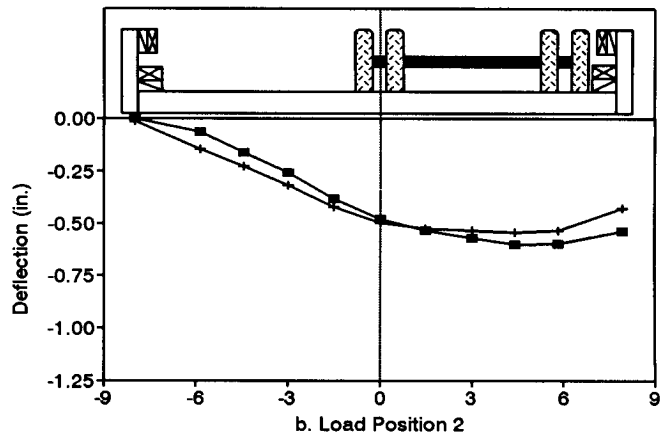
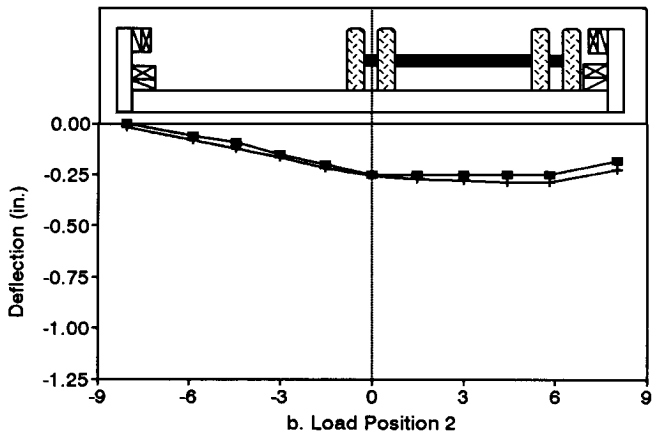
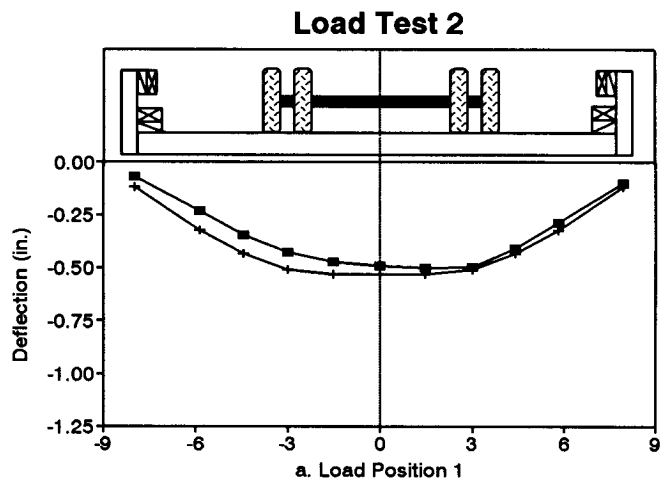
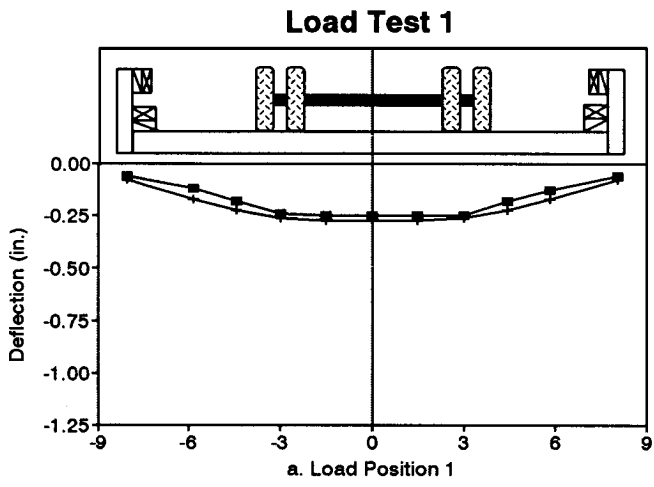
Condition assessment of the Hoffman Run bridge indicates that structural and serviceability performance are acceptable. Inspection results for specific items follow.

Bridge Geometry

Bridge measurements during the monitoring indicated that the width remained relatively stable, narrowing approximately 0.1 ft. The change in bridge width is probably the result of stress relaxation in the laminations. Additional reductions in bridge width can be expected as stress relaxation continues and the lamination moisture content falls below the fiber saturation point.

Wood Condition

Inspection of the wood components of the bridge showed no signs of deterioration, although some checking was evident on rail members exposed to wet-dry cycles. Shortly after rail installation, truss plates were attached to the ends of the rail and the railposts to hinder checking (Fig. 26). The top of the bridge rail showed minor checking, but the depth of the checks did not appear to penetrate the preservative treatment envelope. Checking was most pronounced in the end grain of the timber rail posts. This would have likely been prevented if an end grain sealer had been applied at the time of construction. There was no evidence of wood preservative loss and no preservative or solvent accumulations on the wood surface.



Measured
 Theoretical

Measured
 Theoretical

Figure 24—Comparison of the measured deflections for load tests 1 and 2 to the theoretical deflection, based on orthotropic plate analysis (looking north).

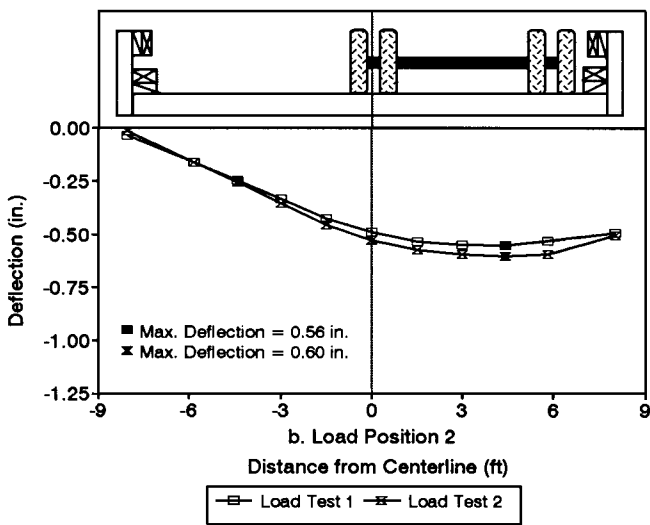
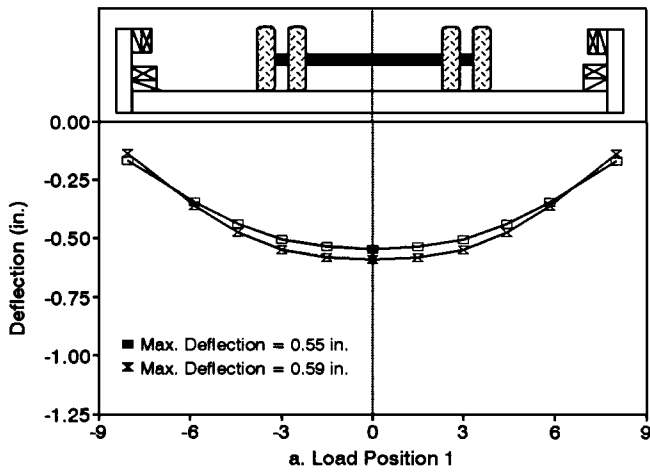


Figure 25—Maximum predicted deflection profile at the bridge midspan for AASHTO HS 20–24 truck loading.

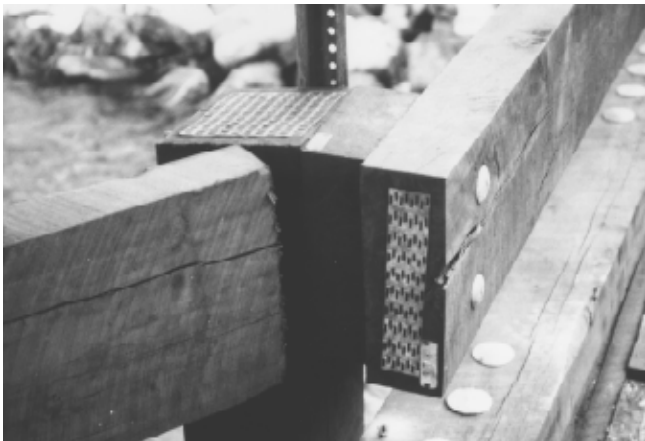


Figure 26—Truss plates attached to end of bridge rail and rail post to impede checking.

Wearing Surface

The Hoffman Run bridge was designed and constructed without a wearing surface; hence, the vehicles ride directly on the treated-wood deck. At the time of each load test, heavy accumulations of gravel and other debris from the unpaved approach roadway were present on the surface of the deck (Fig. 27). Deck wear from vehicle tracking and debris abrasion was noted in several locations, although the level of wear was slight and did not significantly reduce the deck section or extend beyond the preservative treatment. Over time, deck wear will continue, potentially causing a reduced deck section as well as accelerating bridge deterioration as the preservative treatment envelope wears away. In addition, the large volume of debris is capable of trapping moisture on the surface of the deck, which creates an environment more suitable to biological attack of the wood.

Bar Anchorage System

The stressing bar anchorage system has performed as designed with no significant signs of distress. There are no indications of the discrete plate anchorage crushing into the exterior laminations or measurable distortion in the bearing plate. The exposed steel stressing bars, hardware, and steel plates showed no visible signs of corrosion or other distress.

Conclusions

After approximately 2-1/2 years in service, the Hoffman Run bridge is exhibiting acceptable performance, although several serviceability deficiencies are noted. These deficiencies are primarily attributed to the high moisture content of the lumber laminations at the time of construction and throughout the monitoring. Based on monitoring conducted since bridge construction, the following conclusions are made:



Figure 27—Debris accumulation on the bridge deck noted during condition assessment.

- It is feasible to construct stress-laminated decks using mixed hardwood lumber.
- Prefabricating panels for stress-laminated decks and joining them at the construction site is a viable method of bridge construction. Assembling the panels at the construction site and lifting the entire superstructure into place with a small crane reduces field work, minimizes traffic disruption, and may be an economical alternative to field construction.
- The average moisture content in the outer 1 to 2 in. of the laminations of the Hoffman Run bridge decreased from 42 percent at the time of the first load test to 32 percent at the end of the monitoring, but remains above the fiber saturation moisture content. It is anticipated that the moisture content will continue to decrease and will eventually fall below the fiber saturation point, resulting in lamination shrinkage and bar force loss. For stress-laminated bridges constructed in the future, it is recommended that the average moisture content of the lumber laminations not exceed 19 percent at time of construction.
- During the monitoring, the average bar force for the Hoffman Run bridge decreased from 65,000 lb (120 lb/in² interlaminar compression) to 38,000 lb (70 lb/in² interlaminar compression). The decline in bar force is greater than expected and is primarily a result of accelerated stress relaxation in the lumber laminations caused by the high moisture content.
- Vertical creep during the monitoring period resulted in a sag at the superstructure midspan of 0.5 in. along the upstream bridge edge. This accelerated creep is again attributable to the high lamination moisture content. It is doubtful that the small sag will adversely affect the serviceability of the bridge.
- Load testing and analysis indicate that the Hoffman Run bridge exhibits linear elastic orthotropic plate behavior when subjected to static truck loading. Based on an analytical comparison of load test results at different levels of interlaminar compression, the longitudinal bridge stiffness decreased approximately 7 percent when the interlaminar compression decreased from 114 to 70 lb/in². The maximum predicted bridge deflection as a result of AASHTO HS 20-44 static truck loading is estimated to be 0.56 in. (L/536) at 114 lb/in² interlaminar compression and 0.60 in. (L/500) at 70 lb/in² interlaminar compression.
- Visual inspections of the bridge indicate that the performance of wood and steel components is satisfactory. The exposed steel stressing bars and hardware show no visible signs of corrosion or other distress, and the discrete plate bar anchorage is not distorted or crushing into the lumber laminations. The lack of a wearing surface has resulted in deck wear from vehicle tracking and debris abrasion. Although the level of wear is minimal and does not significantly reduce the deck section or extend beneath the preservative treatment, further deterioration is anticipated.

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Appendix—Information Sheet

General

Name: Hoffman Run bridge

Location: County Road T364, Dahoga, Pennsylvania

Date of Construction: October 1990

Owner: Jones Township

Design Configuration

Structure Type: Stress-laminated deck with butt joints

Butt Joint Frequency: Every 4th lamination transversely
Every 4 ft longitudinally in
adjacent lamination

Total Length (out-out): 26 ft

Skew: None

Number of Spans: 1

Span Lengths (center-to-center bearings): 25 ft

Width (out-out): 16 ft

Width (curb-curb): 14.5 ft

Number of Traffic Lanes: 1

Design Loading: AASHTO HS 20-44

Wearing Surface Type: None

Material and Configuration

Timber:

Species: Mixed hardwoods, mostly beech

Size (actual): 1-1/2 to 2 in. wide; 12 in. deep

Grade: No. 2 and better, visually graded

Moisture Condition: 42 percent average 2 months
after installation at 1 to 2 in.
depth

Preservative Treatment: Creosote

Stressing Bars:

Diameter: 1 in.

Number: 7

Design Force: 65,000 lb

Spacing: 45 in. average center-to-center beginning
18 in. from bridge ends

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

Anchorage Type and Configuration:

Steel Plates: 12 by 11 by 1 in. bearing
6 by 4 by 1 in. anchor