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Field Performance of Timber Bridges

14. Dean, Hibbsville, and Decatur Stress-Laminated Deck Bridges

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Michael A. Ritter
Steve Golston
Keith Hinds

Abstract

The Dean, Hibbsville, and Decatur bridges were constructed in southern Iowa during 1994. Each bridge is a simple-span, stress-laminated deck superstructure, approximately 7.3 m (24 ft) long, constructed from eastern cottonwood lumber. The performance of each bridge was monitored for approximately 2 years, beginning shortly after installation. Monitoring involved collecting and evaluating data pertaining to the moisture content and vertical creep of the wood decks, the force level of the stressing bars, and the behavior of the bridges under static load conditions. In addition, comprehensive visual inspections were conducted to assess the overall condition of the structure. Based on field evaluations, the bridges are performing well with minor serviceability deficiencies.

Keywords: Timber, bridge, cottonwood, wood, stress laminated, performance.

Acknowledgments

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Introduction

In 1988, the U.S. Congress passed legislation known as the Timber Bridge Initiative (TBI). The objective of the TBI was to establish a national timber bridge program to encourage the effective and efficient use 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. Three emphasis areas were identified: technology transfer, demonstration bridges, and research. The Forest Service National Wood in Transportation Information Center (NWITIC) (formerly the Timber Bridge Information Resource Center) in Morgantown, West Virginia, maintains the technology transfer program and administers the demonstration bridge program. The demonstration bridge program provides matching funds on a competitive basis to local governments for the construction of timber bridges that illustrate the use of innovative designs or previously underutilized wood products (S&PF 1995). In so doing, bridge designers and users become more aware of the attributes of wood as a bridge material and new, economical, structurally efficient timber bridge systems should result. In addition, it is expected that timber use in bridges will expand to include abundant but underutilized wood species.

Responsibility for the research portion of the TBI was assigned to the USDA Forest Service, Forest Products Laboratory (FPL), a national wood utilization research laboratory, in Madison, Wisconsin. As part of the research program, FPL assumed a lead role in assisting local governments in evaluating the field performance of demonstration bridges, many of which use design innovations or materials that have not been previously evaluated. Through such assistance, FPL is able to collect, analyze, and distribute information on the field performance of timber bridges, thus providing a basis for validating or revising design criteria and subsequently improving efficiency and economy in bridge design, fabrication, and construction.

In addition to the TBI, Congress passed the Intermodal Surface Transportation Efficiency Act (ISTEA) in 1991, which included provisions for a timber bridge program aimed at improving the utilization of wood transportation structures. Responsibility for the development, implementation, and administration of the ISTEA timber bridge program was assigned to the Federal Highway Administration (FHWA) and included demonstration timber bridge, technology transfer, and research programs. Many aspects of the FHWA research program paralleled those underway at FPL; therefore, a joint effort was initiated to combine the respective research of the two agencies into a central research program. As a result, FPL and FHWA merged resources to jointly develop and administer a national timber bridge research program.

This report, fourteenth in a series documenting field performance of timber bridges, describes the development, design, construction, and performance of three eastern cottonwood stress-laminated deck bridges constructed in southern Iowa: the Dean and Hibbsville bridges in Appanoose County and the Decatur bridge in Decatur County. The bridges, built in 1994, are simple-span, stress-laminated deck superstructures, approximately 7.3 m (24 ft) long. The decision to construct this type of bridge was a direct result of the success of the Cooper Creek bridge, the first eastern cottonwood stress-laminated deck, located in Centerville, Appanoose County, Iowa (Ritter and others 1995b). The characteristics of each bridge are summarized in the Appendix.

Background

Each bridge is located on an unpaved, low-volume county road in southern Iowa (Fig. 1). The Dean bridge is on a double-lane, gravel roadway, approximately 8.0 km (5 miles) southwest of Moulton, Iowa, and provides access for farm vehicles and local traffic. The Hibbsville bridge is located on a single-lane, dead-end gravel roadway, approximately 3.2 km (2 miles) southwest of Numa, Iowa, and provides access to a cemetery. The Decatur bridge is located on a field entrance roadway, approximately 3.2 km (2 miles) southwest of Davis City, Iowa, and provides access for farm vehicles and machinery.

The original Dean and Hibbsville bridges were constructed in the 1930s. The Dean bridge was a timber plank deck supported by steel I-beam stringers. The Hibbsville bridge was a steel pony truss with a timber plank deck. Inspection of these bridges revealed severe corrosion of the steel components and deterioration of the timber plank decks, indicating that the bridges were structurally deficient and required major rehabilitation or replacement. The original crossing at the Decatur bridge site was a 1.5-m- (60-in.-) diameter corrugated steel culvert that had to be replaced because it washed away during a period of high water in July 1993.

As mentioned previously, the decision to construct eastern cottonwood stress-laminated deck bridges was a direct result of the success of the Cooper Creek bridge. To advance the utilization of cottonwood in transportation structures, the Chariton Valley Resource Conservation and Development Council (RC&D) provided the lamination material for the construction of the three bridges. Becasue the use of cottonwood in stress-laminated decks was still relatively new, FPL was contacted by Chariton Valley RC&D officials to provide technical advice to the counties and monitor the field performance of each bridge. As a result, the three bridges were included in the FPL/FHWA bridge monitoring program.

Objective and Scope

The objective of this project was to evaluate the field performance of the Dean, Hibbsville, and Decatur bridges for approximately 2 years, beginning shortly after bridge installation. The scope included data collection and analysis related to the moisture content of the deck, stressing bar force, vertical creep, bridge behavior under static load, and general structural performance. The results of this project will be evaluated with similar monitoring projects in an effort to formulate recommendations for design and construction of future stress-laminated deck bridges.

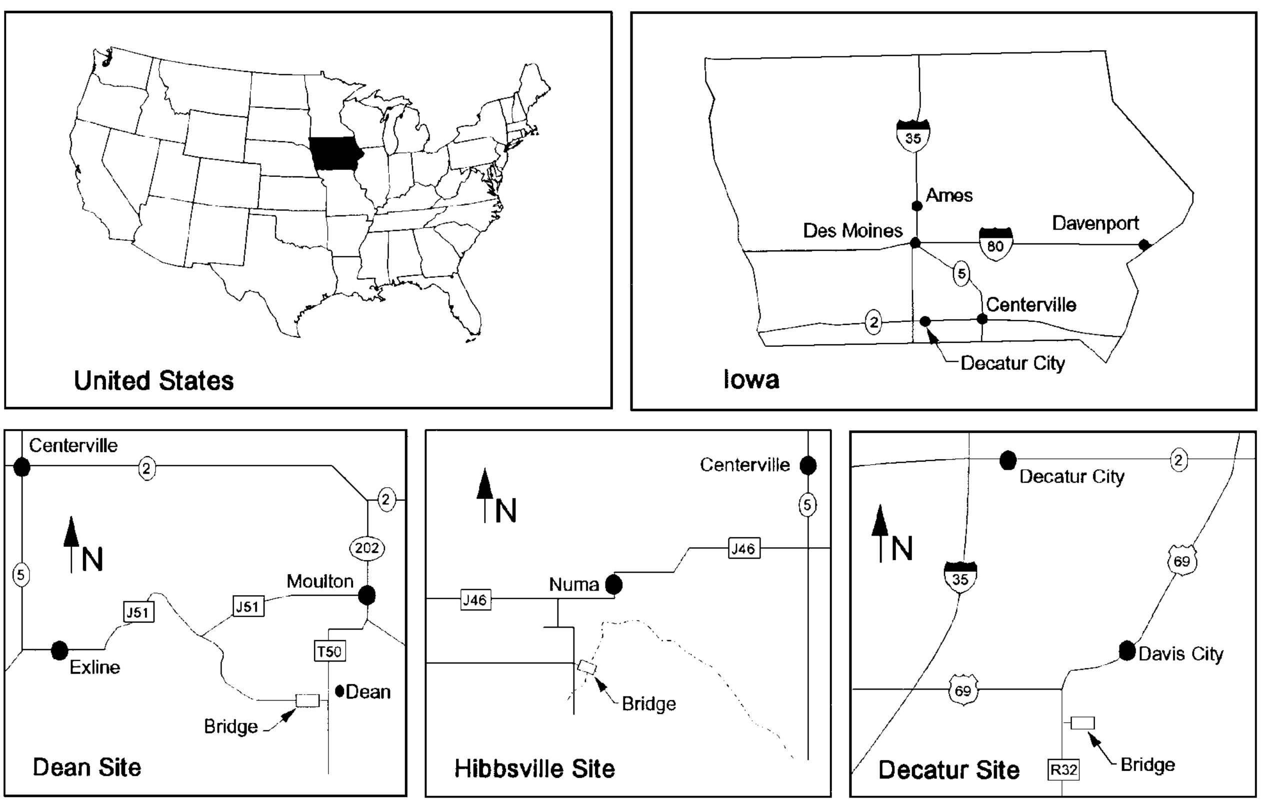


Figure 1—Location of the Dean, Hibbsville, and Decatur bridges.

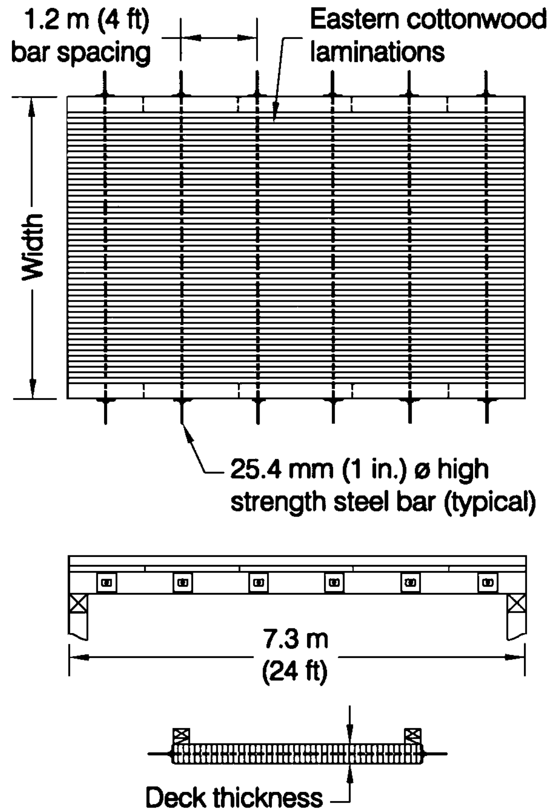
Design and Construction

Based on the design of the Cooper Creek bridge, design of the Dean and Hibbsville bridges was completed by the Appanoose County Engineering Department. Design of the Decatur bridge was based on the Appanoose County designs. Construction was completed by the respective county construction crews. An overview of the design and construction process for the bridge superstructures follows.

Design

The bridges were designed for American Association of State Highway and Transportation Officials (AASHTO) HS 20-44 truck loading (AASHTO 1989). Each bridge was designed to be a 7.3-m- (24-ft-) long, stress-laminated superstructure composed of eastern cottonwood sawn lumber laminations (Fig. 2). Each deck was designed with butt joints in the deck laminations placed in every fourth lamination transversely, with a 1.2-m- (4-ft-) longitudinal spacing between butt joints in adjacent laminations (Fig. 3). The resulting deck width for the Dean, Hibbsville, and Decatur bridges was 7.0, 5.2, and 6.4 m (23, 17, and 21 ft), respectively, with a deck thickness of 381, 356, and 356 mm (15, 14, and 14 in.), respectively. All lumber components were treated with creosote in accordance with the American Wood Preservers' Association (AWPA) Standard C14 (AWPA 1990). The Dean and Decatur bridges have an approximate 51-mm (2-in.) gravel wearing surface. The Hibbsville bridge has no wearing surface.

The stressing system for each bridge was designed to provide a uniform interlaminar compressive stress of 689.5 kPa (100 lb/in^2), which corresponds to a design bar force of 320 kN (72,000 lb) for the Dean bridge and 299 kN (67,200 lb) for the Hibbsville and Decatur bridges. For each bridge, six 25.4-mm- (1-in.-) diameter, high strength steel stressing bars, complying with the requirements of ASTM A722 (ASTM 1988), were spaced 1.2 m (4 ft) on center. The bar anchorages are discrete plate systems that are similar in design, although the actual number, type, and dimensions of the plates varied (Fig. 4). To provide protection from deterioration, the high strength steel bars and nuts were galvanized. The steel bearing and anchor plates were not galvanized. A unique feature of the anchorage configuration for the Dean and Decatur bridges is an untreated, white oak plate between the exterior lamination and the steel bearing plate. Full-length dense hardwood exterior laminations are sometimes used in stress-laminated bridges because they have stronger compression perpendicular-to-grain characteristics and are less likely to experience crushing beneath the anchorage plates. The white oak plates were added to the anchorage system in lieu of full-length dense hardwood laminations.



	Width	Deck Thickness
Dean	7.0 m (23 ft)	381 mm (15 in.)
Hibbsville	5.2 m (17 ft)	356 mm (14 in.)
Decatur	6.4 m (21 ft)	356 mm (14 in.)

Figure 2—Design configuration of the three eastern cottonwood bridges.

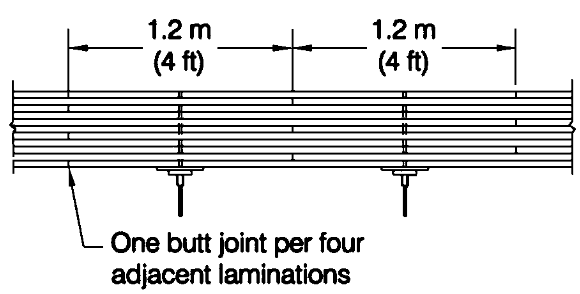
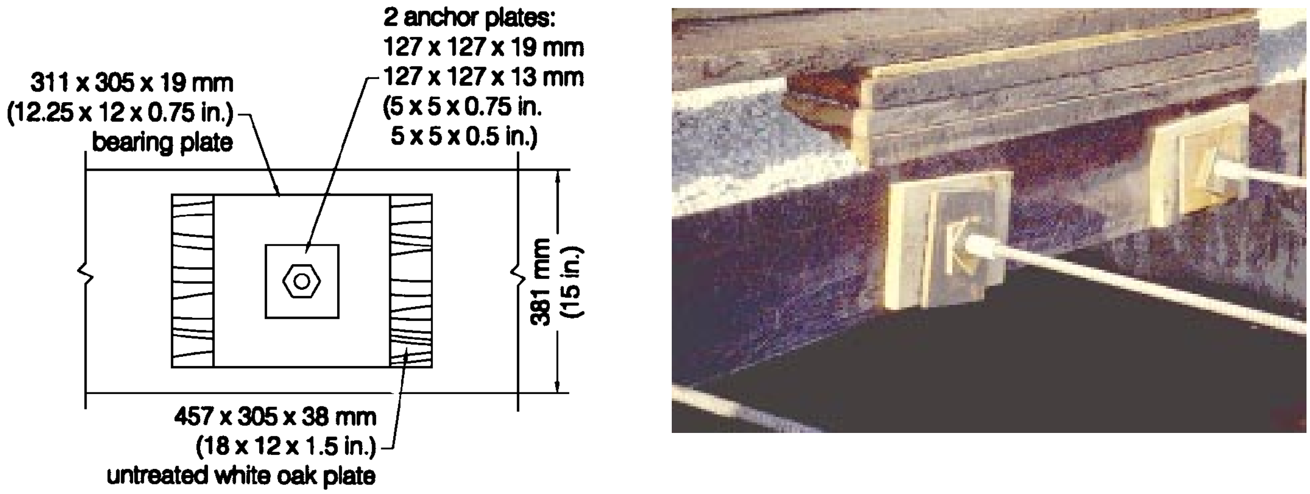
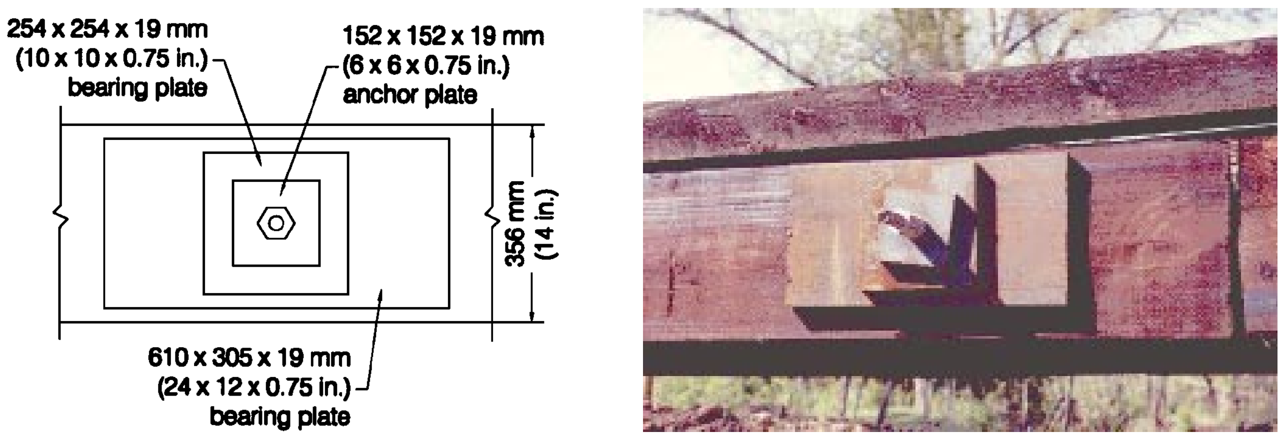


Figure 3—Butt joint configuration used for each bridge. Butt joints were placed transverse to the bridge span in every fourth lamination. Longitudinally, butt joints in adjacent laminations were separated by 1.2 m (4 ft).

Dean



Hibbsville



Decatur

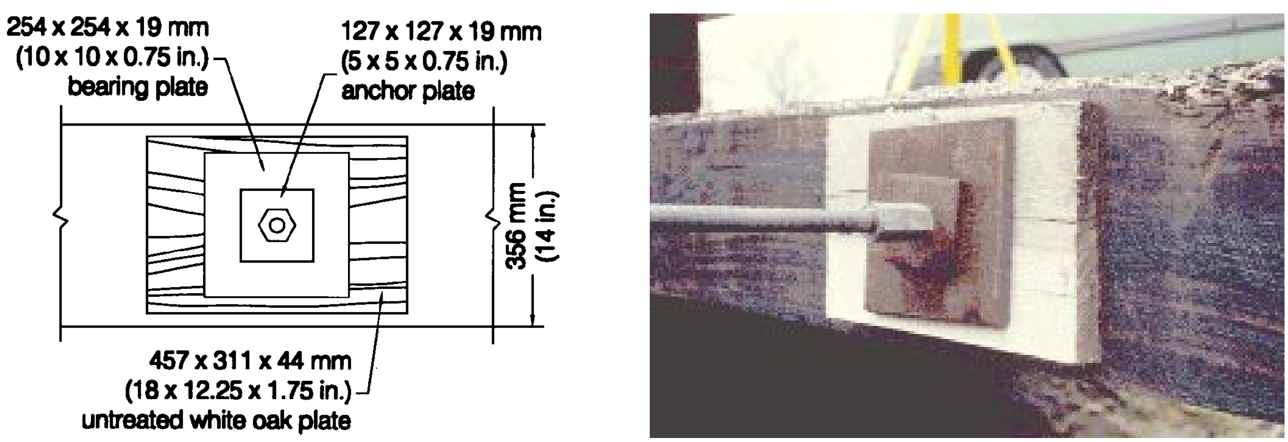


Figure 4—Details of the Dean (top), Hibbsville (middle), and Decatur (bottom) discrete plate anchorage configurations.

Construction

Construction of the Dean, Hibbsville, and Decatur bridges was completed by county construction crews in October 1993, January 1994, and June 1994, respectively. Following completion of the abutments and wingwalls, each bridge deck was assembled and placed in 1 day. The bridge railing and curb systems were installed shortly thereafter.

Concrete abutments and wingwalls were constructed for the Dean bridge. The Hibbsville bridge abutments were composed of timber piling with backwall planks and timber pile caps. The Decatur bridge abutments were also composed of timber piling with backwall planks, but steel W-beams were used for the pile caps. Each deck of the three bridges was constructed by assembling the laminations on temporary supports located on the approach roadway. Steel stressing bars were inserted through holes in the laminations; bearing and anchor plates were installed; nuts were hand tightened. The bars were then tensioned to the required force using a single hydraulic jack. After tensioning the bars, the entire Dean superstructure was lifted onto the abutments by a small crane, and the Hibbsville deck was dragged onto the abutments using a makeshift skid. The Decatur superstructure was dragged onto temporary steel beams that had been placed on the abutments. A crane then lifted the deck while the steel beams were removed, and the deck was lowered into place.

Approximately 2 weeks after the initial tensioning, the steel bars were tensioned to the design force a second time to compensate for losses in bar force (Ritter 1990). The bars were tensioned a third time at the first load test, approximately 7, 4, and 2 months after construction of the Dean, Hibbsville, and Decatur bridges, respectively. The first bridge inspections were also completed at the first load test. Inspection revealed that the as-built bridge configurations varied slightly from the design configurations (Fig. 2). For the Dean, Hibbsville, and Decatur bridges, the average width of the out-to-out bridge measured 6.98, 5.24, and 6.46 m (22.9, 17.2, and 21.2 ft), respectively. The length of the bridge measured 7.35, 7.19, and 7.22 m (24.1, 23.6, and 23.7 ft), respectively, and the span of the bridge, center-to-center of bearings, measured 6.80, 7.04, and 6.92 m (22.3, 23.1, and 22.7 ft), respectively. It was also noted that the laminations used for the Dean bridge were uneven in width and resulted in gaps between adjacent laminations near the buttjoints. The completed bridges are shown in Figure 5.

Evaluation Methodology

To evaluate the structural performance of the Dean, Hibbsville, and Decatur bridges, Chariton Valley RC&D officials contacted FPL for assistance. As a result, the bridges were included in the FPL/FHWA timber bridge monitoring program. Through mutual agreement, a bridge monitoring plan was developed and implemented as a cooperative effort with the Chariton Valley RC&D. The plan called for performance monitoring of the moisture content, stressing bar force, vertical creep, load test behavior, and condition assessments of each structure for approximately 2 years. The evaluation methodology utilized procedures and equipment previously developed by FPL (Ritter and others 1991).

Moisture Content

Changes in the moisture content of stress-laminated timber decks can significantly affect bar force, deck stiffness, vertical creep, and transverse stress relaxation. The moisture content of the deck of the Dean, Hibbsville, and Decatur bridges was measured using an electrical-resistance moisture meter with 76 mm (3 in.) insulated probe pins in accordance with ASTM D4444-84 (ASTM 1990). Measurements were obtained by driving the pins into the underside of the deck at depths of 25 to 76 mm (1 to 3 in.), recording the moisture content values, and adjusting the values for temperature and wood species (FORINTEK 1984). Measurements for the Dean and Decatur bridges were obtained by FPL personnel at the beginning and end of the monitoring period. Appanoose County personnel obtained measurements on a bimonthly basis for the Hibbsville bridge.

Bar Force

For stress-laminated bridges to perform properly, adequate bar force and interlaminar compression must be maintained. To monitor bar force, load cells developed by FPL were installed on two stressing bars of each bridge. Load cell measurements for the Dean bridge were obtained on an hourly basis through a remote data acquisition system. Load cell measurements for the Hibbsville and Decatur bridges were obtained by county personnel, using a portable strain indicator, daily for several days following installation, then on a monthly basis. For each bridge, load cell strain readings were converted to bar tensile force by applying a laboratory conversion factor. At the end of the monitoring period, the load cells were unloaded and checked for zero balance shift, and the measurements were adjusted accordingly.

Vertical Creep

As a structural material, wood can deform permanently or creep, as a result of long-term sustained loads. For stress-laminated bridges, vertical creep as a result of structure dead load is an important consideration because excessive creep can result in a sag of the superstructure (Ritter and others 1990). Creep of the Dean, Hibbsville, and Decatur bridges was measured along the edges of each bridge by attaching a stringline to the bearings to create a horizontal benchmark and measuring the deck elevation at midspan relative to the benchmark with a calibrated rule.

Figure 5—Completed bridges: Dean (top), Hibbsville (middle), and Decatur (bottom).

Load Test Behavior

Static load test results were used to assess overall bridge performance. Results will eventually be compared with load test results from other bridges to refine and improve design procedures and evaluate the effects of design variables on bridge performance. To determine the load test behavior of the bridges, static load tests were conducted twice during the monitoring period. The load tests occurred May 11, 1994, and May 16, 1996, for the Dean and Hibbsville bridges and August 3, 1994, and May 15, 1996, for the Decatur bridge. The interlaminar compression was approximately 437.1, 535.7, and 490.2 kPa (63.4, 77.7, and 71.1 lb/in^2) at the time of load test 1 and 529.5, 448.2, and 492.3 kPa (76.8, 65, and 71.4 lb/in^2) at the time of load test 2 for the Dean, Hibbsville, and Decatur bridges, respectively.

For each load test, fully loaded trucks were transversely positioned on the bridge in centric and eccentric load positions and longitudinally with the rear axles centered about the midspan (Figs. 6 and 7). As a result of the extremely muddy road condition, only load position 1 was tested for the second Hibbsville bridge load test. For each load position, resulting deflections were measured at a series of locations along the midspan of the bridge. Measurements of deck deflections were taken prior to testing (unloaded), for the load positions (loaded), and at the conclusion of testing (unloaded). Measurements of bridge deflections were obtained by suspending calibrated rules from the underside of the deck and reading values to the nearest 0.1 mm (0.004 in.) with a surveyor's level (Fig. 8). The accuracy of measurements was estimated to be ±0.1 mm (±0.004 in.).

Analytical Evaluation

Following completion of the load tests, analytical assessments were completed to determine the theoretical bridge response. Previous research showed that stress-laminated decks can be accurately modeled as orthotropic plates (Ritter and others 1995a). To further analyze the theoretical behavior of the Hibbsville, Dean, and Decatur bridges, an orthotropic plate computer model, currently being developed at FPL, was used to analyze the load test results and predict the maximum bridge deflection for AASHTO HS 20-44 loading. An edgewise modulus of elasticity (MOE) value, adjusted for wet-use conditions, of 8,563 MPa (1,242,000 lb/in^2) was used for modeling. This edgewise MOE value is based on an unpublished study that tested 130 of the cottonwood bridge laminations and resulted in a mean flatwise MOE for eastern cottonwood of 9,860 MPa (1,430,000 lb/in^2). Adjustment from flatwise to edgewise MOE was accomplished by applying a conversion factor of 0.965 (Williams and others 1994).

Condition Assessment

The general condition of each bridge was assessed at the time of each load test, which corresponded with the beginning and end of the monitoring. The condition assessment of the bridges involved visual inspections, measurements, and photographic documentation. Items of specific interest included the geometry of the bridge and the condition of the timber deck, rail system, wearing surface, stressing bars, and anchorage systems.

Results and Discussion

Performance monitoring of the Dean and Hibbsville bridges covered approximately 24 months, from May 11, 1994, through May 16, 1996. Performance of the Decatur bridge was monitored for approximately 22 months, from August 3, 1994, through May 15, 1996. Monitoring results follow.

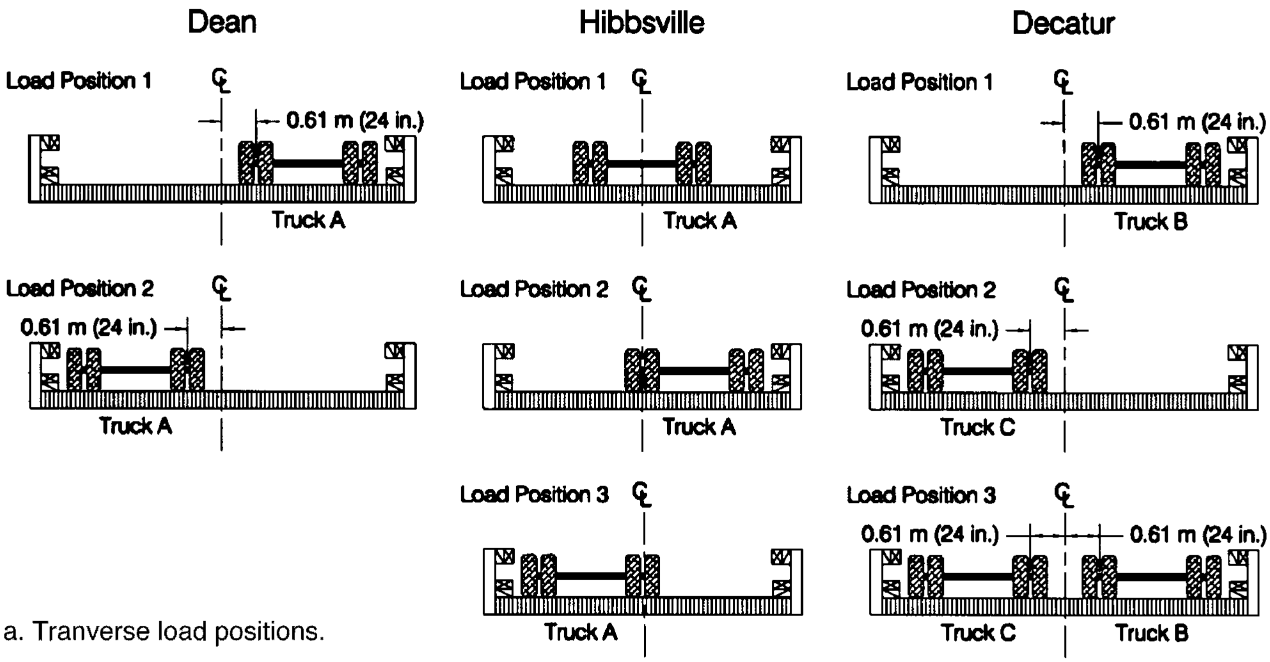
Moisture Content

The average lamination moisture content of the Hibbsville and Dean bridges was approximately 24% at the beginning and end of monitoring. The average lamination moisture content of the Decatur bridge was approximately 21% at the beginning and 26% at the conclusion of monitoring. Measurements from the Hibbsville bridge indicated that the moisture content level remained relatively stable throughout the monitoring period, although there were fluctuations of 2% to 3% in the measurement zone as a result of seasonal climatic changes. Although monthly data were not collected for the Dean and Decatur bridges, it is suspected that moisture content fluctuations for these bridges were similar to the those of the Hibbsville bridge.

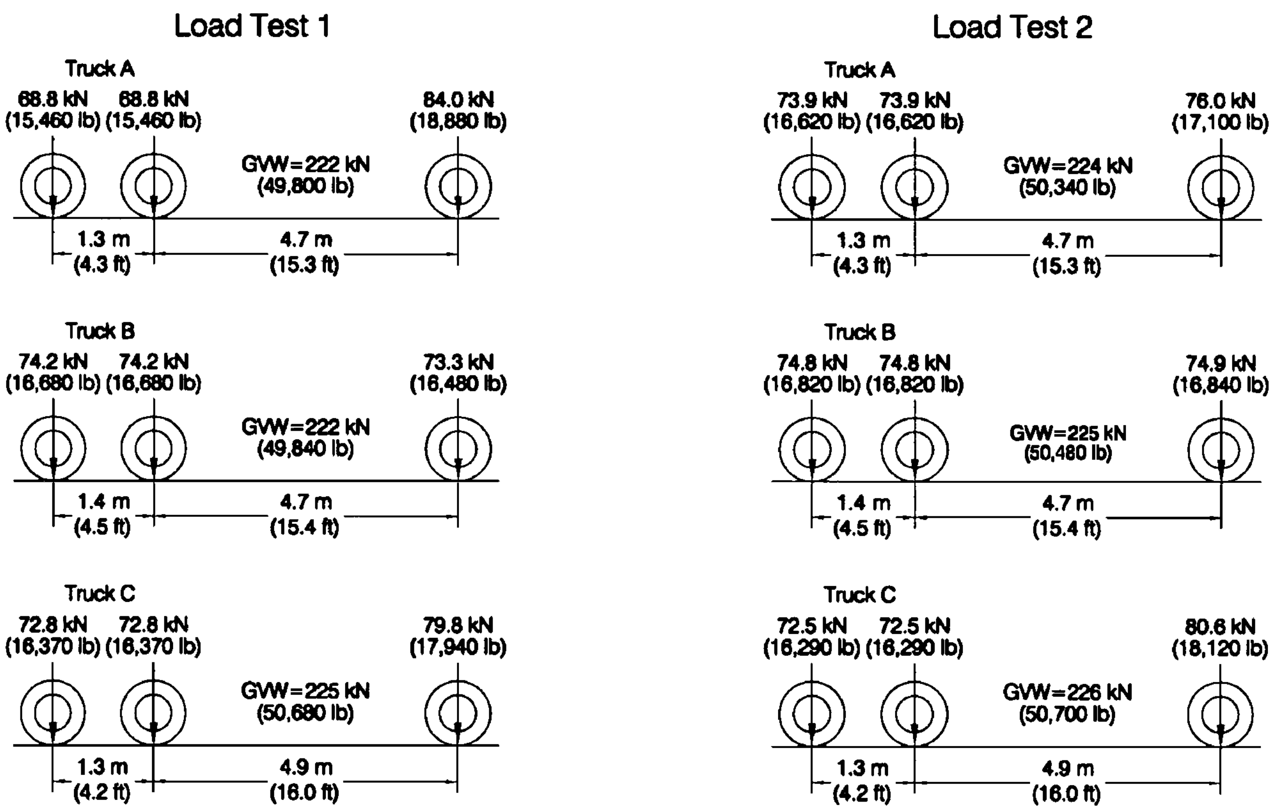
The primary contributing factor to the continued high moisture content of each bridge was the absence of a watertight membrane over the surface of the deck. For the Hibbsville bridge, water directly contacts the timber deck, and for the Dean and Decatur bridges, water permeates the gravel wearing surface and subsequently comes into contact with the timber deck. Furthermore, drying of the deck is inhibited by the gravel wearing surface of the Dean and Decatur bridges, and dirt and debris accumulations on the Hibbsville deck.

Bar Force

The average bar force trend for the Dean, Hibbsville, and Decatur bridges is illustrated in Figure 9. These figures also indicate a bar force of 128, 120, and 120 kN (28,800, 26,888, and 26,888 lb), which represents the minimum recommended interlaminar compression level of 276 kPa (40 lb/in^2) for the Dean, Hibbsville, and Decatur bridges, respectively (Ritter 1990). In each case, data begin at the



a. Tranverse load positions.



b. Vehicle configurations.

Figure 6—(a) Transverse load positions used for load tests (looking west). Only load position 1 was conducted at the second Hibbsville bridge load test. (b) Test vehicle configurations and axle loads for load test 1 and 2 (right single axle is vehicle front). The transverse vehicle track width, measured center-to-center of the rear tires, was 1.8 m (6 ft).

Figure 7—(a) Load position 2 used for the first Dean bridge load test (looking north). (b) Load position 1 used for the second Hibbsville load test (looking west).

Figure 8—Load test deflection measurements were obtained by reading values from calibrated rules suspended from the underside of the deck with a surveyor's level. Load test 2 of the Hibbsville bridge is shown.

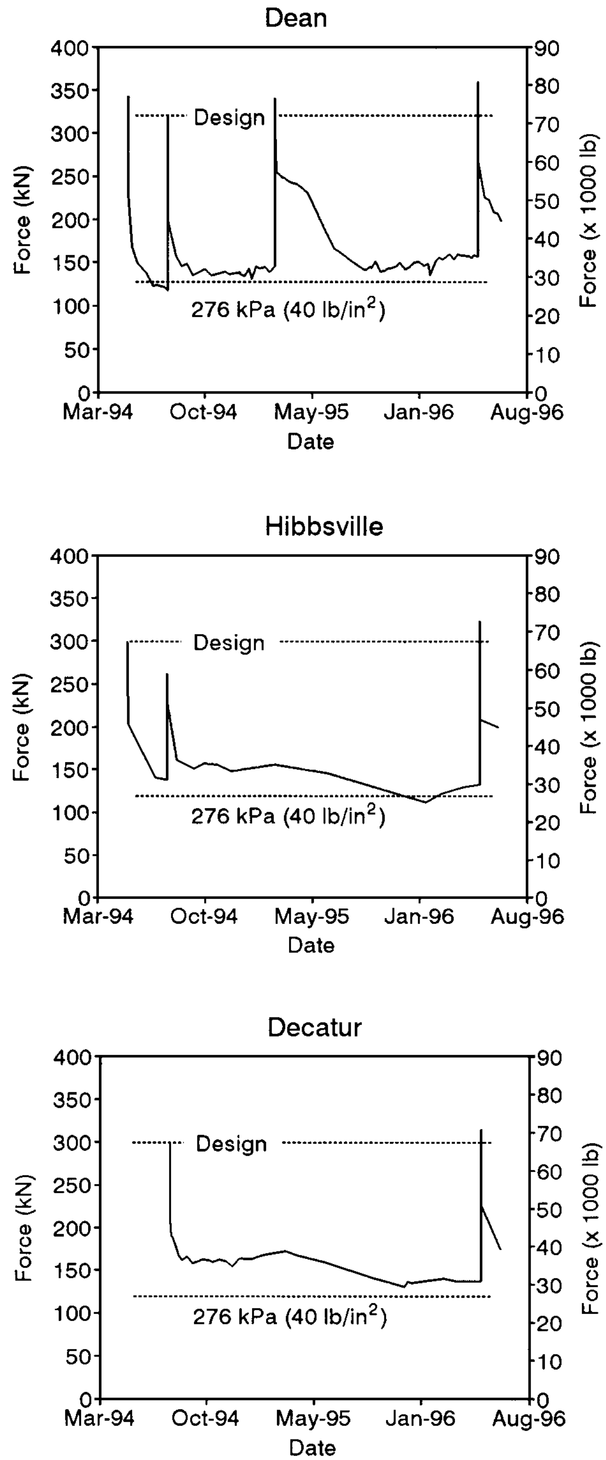


Figure 9—Average trend in bar force for the Dean, Hibbsville, and Decatur bridges.

time of the first load test when the bars were tensioned to the approximate design force of 320 kN (72,000 lb) for the Dean bridge and 299 kN (67,200 lb) for the Hibbsville and Decatur bridges. Data end shortly after the second load test, at which time the bars were tensioned to approximately 110% of the design force. Additional bar tensionings occurred during the monitoring period for the Dean and Hibbsville bridges. As indicated, initial force loss after bar tensioning occurred rapidly and ranged from approximately 15% to 40% within several days. The Dean bridge experienced the largest initial loss. Typically, rapid force loss continued at a decreasing rate until the bar force stabilized at approximately 138, 133, and 160 kN (31,000, 30,000, and 36,000 lb), which corresponds to interlaminar compression levels of 296, 310, and 372 kPa (43, 45, and 54 lb/in^2) for the Dean, Hibbsville, and Decatur bridges, respectively.

The majority of bar force loss is attributed to stress relaxation of the lumber laminations caused by the applied compressive force. For these bridges, the effects of stress relaxation were greatly augmented by the high moisture content of the cottonwood laminations.

Vertical Creep

At the time of the second Dean bridge load test, 22 mm (0.875 in.) of negative camber was measured at both edges of the deck. However, vertical creep could not be determined because camber measurements were not obtained at the first load test. The average positive camber of the Hibbsville bridge was 41 mm (1.625 in.) at the first load test. At the time of the second load test, 38 mm (1.5 in.) of negative camber was measured, indicating 79 mm (3.125 in.) of vertical creep during the monitoring period. For the Decatur bridge, 38 mm (1.5 in.) of positive camber was measured along the edge of the south deck at both load tests, indicating no appreciable vertical creep. Measurements could not be obtained along the north edge of the Decatur bridge because several laminations protruded below the underside of the deck, making reference to the bottom of the bridge impossible.

Load Test Behavior

Static load test results with locations and magnitudes of the maximum measured defections are presented in Figures 10, 11, and 12 for the Dean, Hibbsville and Decatur bridges, respectively. All transverse deflections are shown at the midspan of the bridge, as viewed from the east end looking west. For each load test, no permanent residual deformation was measured at the conclusion of the testing, and no movement was detected at the abutments. The deflections for each load position are typical of the orthotropic plate behavior of stress-laminated bridges (Ritter and others 1990).

From the static load test results, the following observations are noted. For load positions with a single test vehicle positioned close to the edge of the bridge, the resulting maximum deflection, generally occurred under the outside wheel line. When both vehicles were positioned on the Decatur bridge, the absolute maximum deflection for each load test occurred at or near centerline.

Assuming uniform material properties, loading, and linear elastic bridge behavior, deflections of the bridge resulting from a single test vehicle placed in symmetrical load positions should be a mirror image. This is illustrated in Figure 13 for the Dean and Hibbsville bridges. Actual deflections for one load position are compared with a mirror image of deflections for the corresponding symmetrical load position. For the Dean bridge, minor differences exist between the deflections, but the plots are essentially identical. Slightly greater variations between the deflections exist for the Hibbsville bridge.

For the Decatur bridge, different results were expected from the symmetrical load positions because two different test vehicles were used. For each load test, the combined rear axle weight of truck B was greater than that of truck C: 2,758 and 4,715 N (620 and 1,060 lb) for load tests 1 and 2, respectively. The plots should be similar in shape but the deflections of load position 1 that resulted from the heavier truck, Truck B, should be slightly greater than those of load position 2 when Truck C was used. Inspection of Figure 14 indicates the reverse is true. The measured deflections of load position 2 exceeded those of load position 1, indicating that the longitudinal stiffness of the north side of the bridge is greater than that of the south side.

The summation of deflections resulting from two separately applied truck loads should equal the deflection of both trucks applied simultaneously, if uniform material properties, proper vehicle placement, and accurate deflection measurements are assumed. This is illustrated in Figure 15 for the Decatur bridge, where the sum of the measured deflections for load positions 1 and 2 of the Decatur bridge load tests are compared with those of load position 3. For each load test, the plots are virtually identical, with minor variations within the accuracy of the measurement methods, indicating that the behavior of the bridge is within the linear elastic range under the applied loads.

Analytical Evaluation

Comparisons of the measured load test deflections to the theoretical bridge response in Figures 10–12 indicate that the theoretical deflections are generally similar to those measured. The theoretical response for the Dean bridge is nearly identical to the actual response with minor variations at the edges of the bridge. For the Hibbsville bridge, the load test

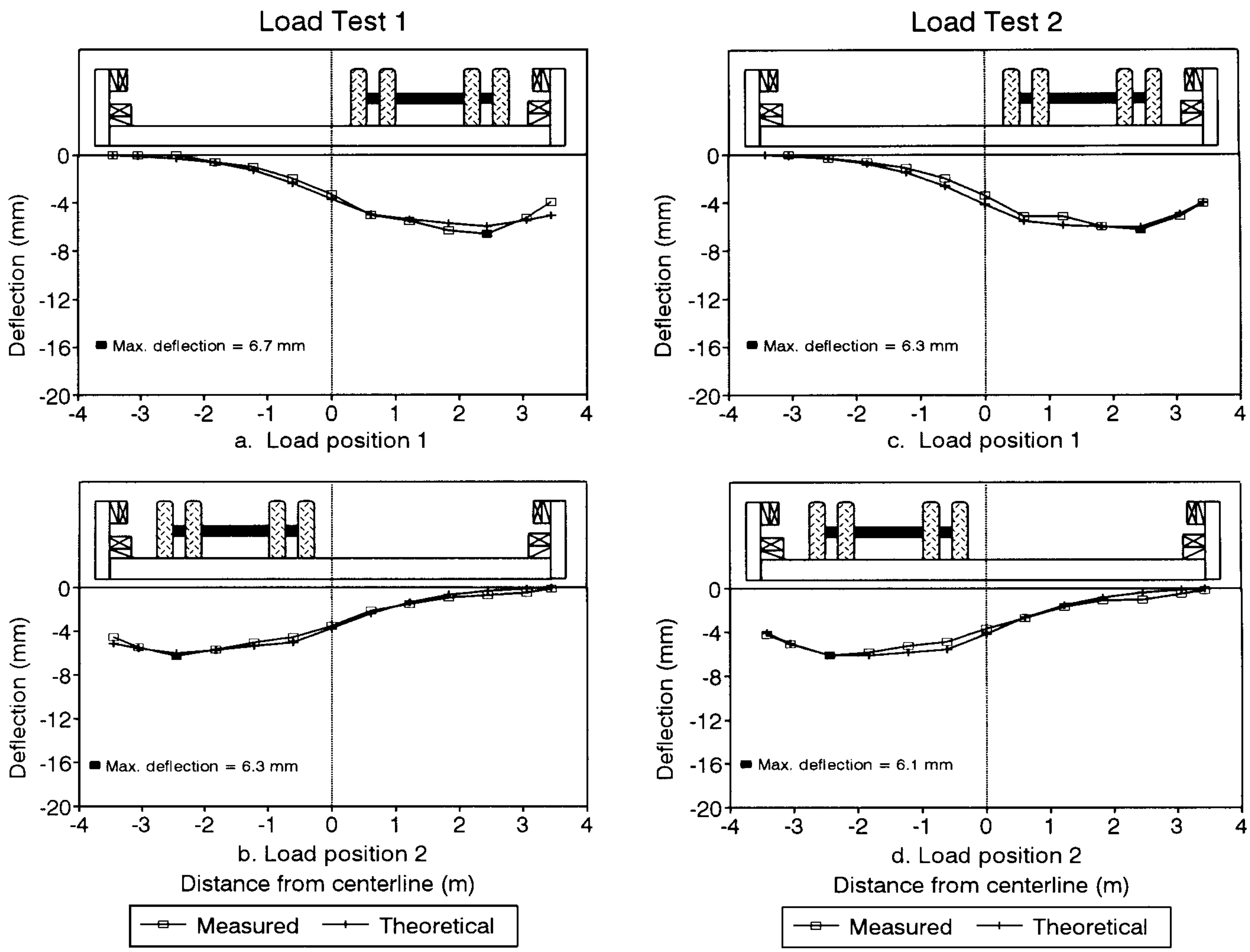


Figure 10—Transverse deflections for the Dean bridge load tests (looking west), measured at th midspan. Bridge cross-section and vehicle positions are shown to aid interpretation and are not to scale.

theoretical response fairly represents the measured deflections for load positions 1 and 3, also with variations along the edges of the bridge. Load test 1 theoretical deflections for load position 2 do not accurately represent the measured deflections, indicating an irregularity in the applied load or the behavior of the bridge. Conclusions regarding irregularities in bridge behavior cannot be made because such behavior was not demonstrated in other load positions, and eccentric load positions could not be retested during load test 2. For load test 2, the theoretical deflections overpredict the measured deflections. Only one load position could be conducted at this load test; therefore, it is not possible to determine why the theoretical deflection values were greater than the measured values.

For each load test of the Decatur bridge, the theoretical response accurately predicted the behavior of the bridge for load position 2 and overestimated the deflections of load position 1. The difference between the measured and theoretical deflections for load position 1 was anticipated, because the orthotropic plate model assumes constant bridge properties, and as mentioned previously, the longitudinal stiffness of the north (right) side of the bridge appears to be greater than that of the south side. For load position 3, the theoretical deflection closely matched the absolute maximum measured deflections, but greater variation between the deflections was again evident at points away from the centerline on the north (right) side.

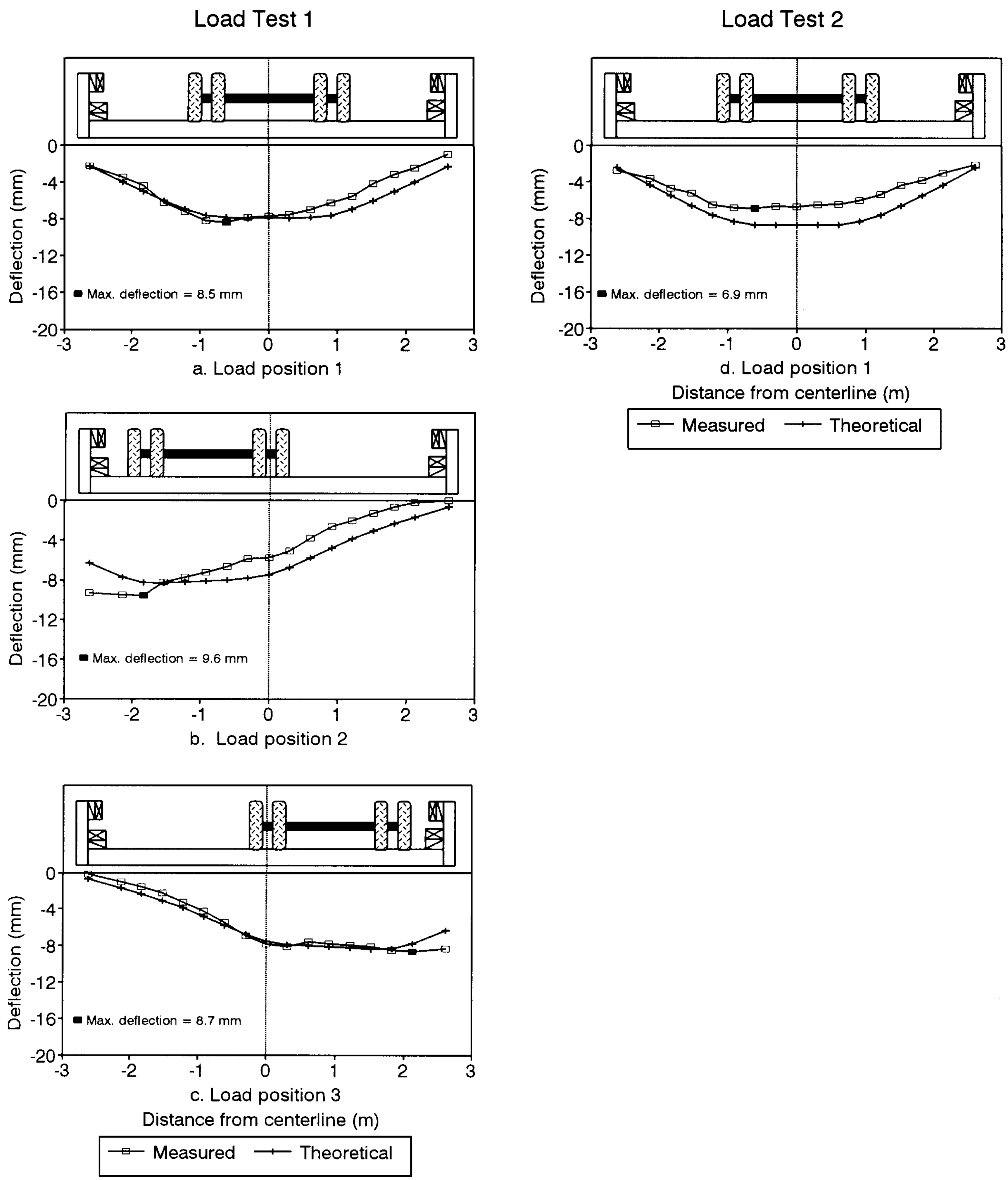


Figure 11—Transverse deflections for the Hibbsville bridge load tests (looking west), measured at the midspan. Load positions 2 and 3 could not be conducted during load test 2 because of extremely muddy road conditions. Bridge cross-sections and vehicle positions are shown to aid interpretation and are not to scale.

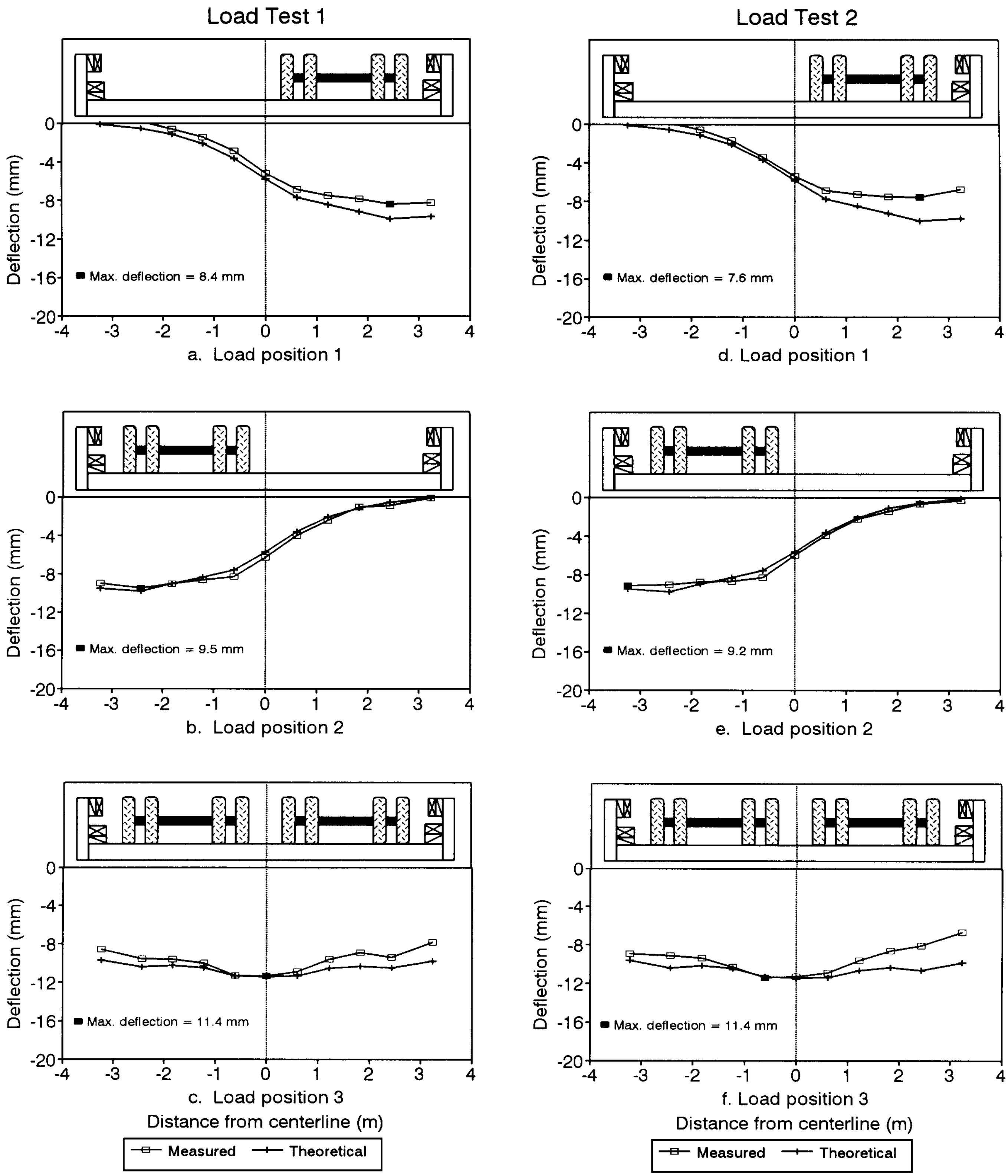


Figure 12—Transverse deflections for the Decatur bridge load tests (looking west), measured at the midspan. Bridge cross-sections and vehicle positions are shown to aid interpretation and are not to scale.

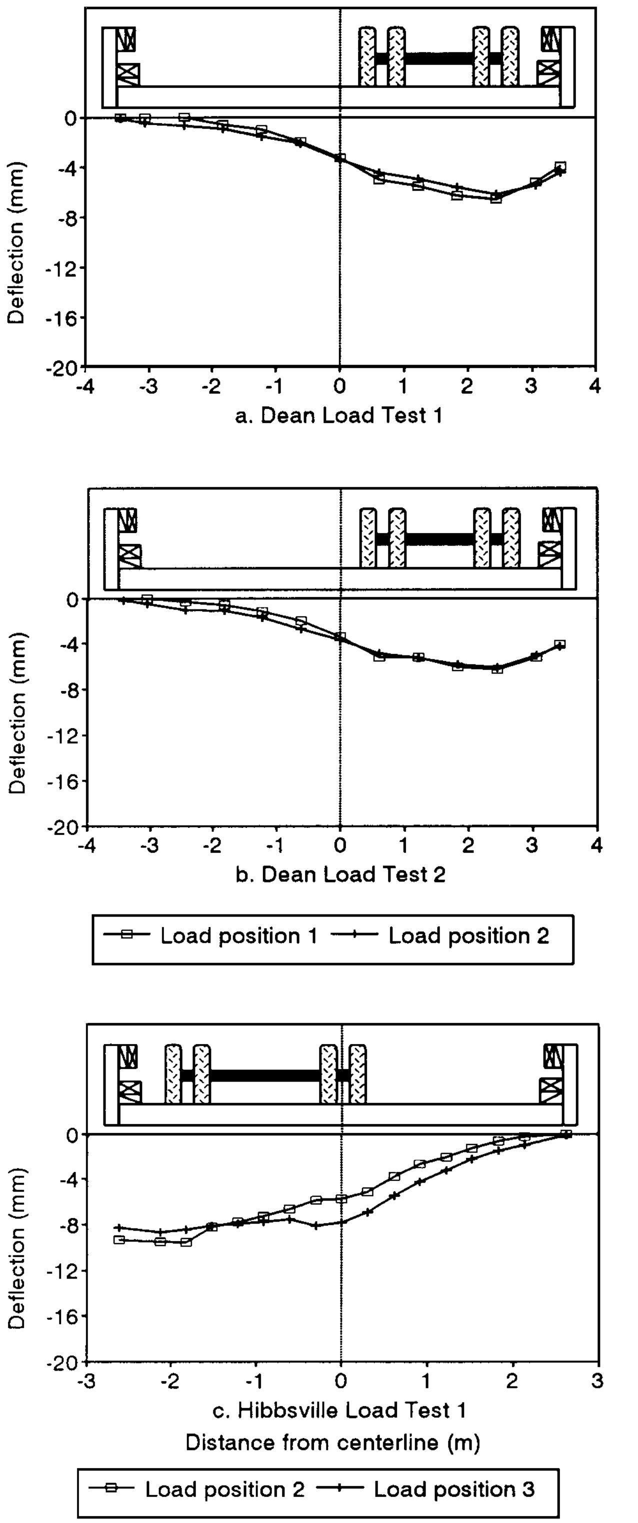


Figure 13—Comparison of measured deflections for the Dean bridge load tests, showing the actual deflection for load position 1, the mirror image of load position 2, and the measured load test 1 deflections for the Hibbsville bridge, showing the actual deflection for load position 2 and the mirror image of load position 3 (looking west).

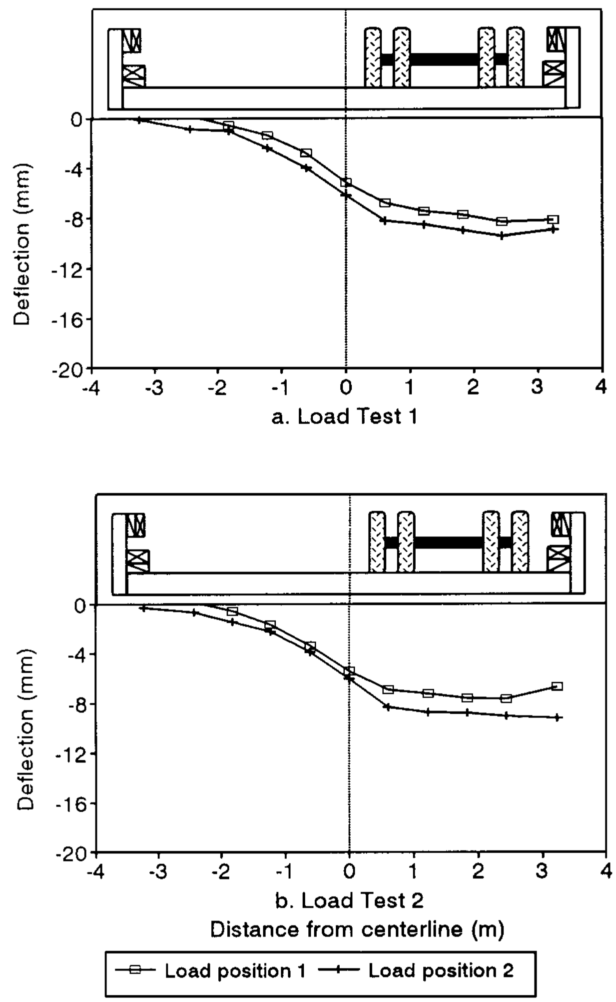


Figure 14—Comparison of measured deflections for the Decatur bridge load tests, showing the actual deflection for load position 1 and the mirror image of load position 2 (looking west).

Employing the same analytical parameters used for determining the theoretical bridge response for each of the load tests, the theoretical maximum deflections for AASHTO HS20–44 truck loading is shown in Figures 16, 17, and 18 for the Dean, Hibbsville, and Decatur bridges, respectively. For the Dean bridge, the theoretical maximum deflection occurred under the outside wheel line when the vehicle was placed eccentrically on the bridge. For the Hibbsville bridge, the theoretical maximum deflection occurred between the wheel lines. The theoretical maximum deflection for the Decatur bridge occurred at the centerline of the bridge when two design vehicles were positioned on the bridge. The maximum deflections for each load test are presented numerically and as a fraction of span length in Table 1.

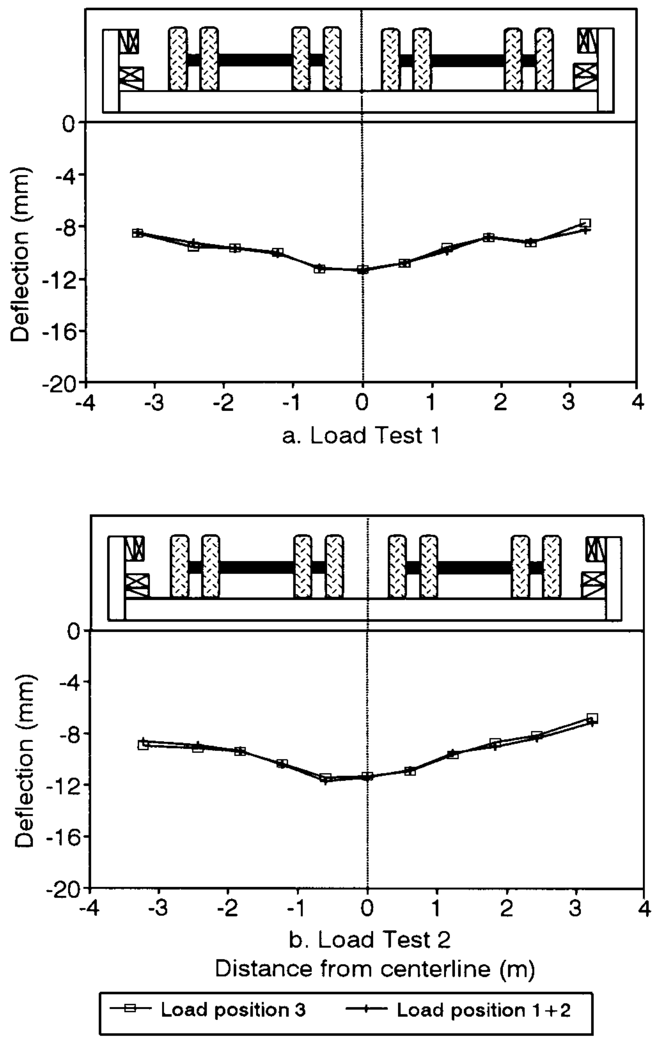


Figure 15—Transverse deflections of the Decatur bridge load tests, measured at the bridge midspan, comparing the sum of measured deflections from load positions 1 and 2 to position 3 (looking west).

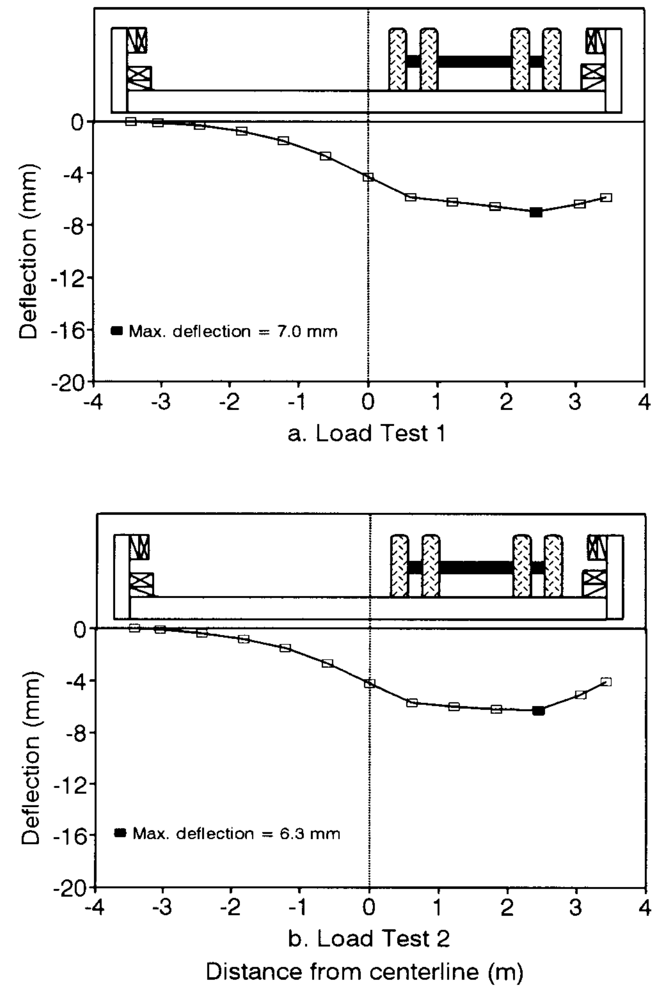


Figure 16—Maximum theoretical midspan deflection profiles for AASHTO HS 20-44 truck loading on the Dean bridge (looking west).

Assuming constant bridge properties, the same theoretical bridge deflection would be expected for both load tests because the same AASHTO HS 20-44 loading was applied in each case. However, for stress-laminated bridges with butt joints, it is known that an increase or decrease in interlaminar compression results in a corresponding increase or decrease in longitudinal bridge stiffness (Ritter and others 1995a). Table 1 also presents the interlaminar compression for each bridge at the time of the load tests and the corresponding change in stiffness. The Dean bridge experienced an increase in interlaminar compression from the time of load test 1 to the time of load test 2, and a subsequent 10% increase in longitudinal bridge stiffness. Theoretically, as illustrated in Table 1, the Hibbsville bridge experienced a 3% decrease in longitudinal stiffness. Although other factors may have contributed, it is likely that the change in stiffness was due primarily to a decrease in interlaminar compression. In addition, the theoretical change in stiffness for the Hibbsville bridge may not accurately represent the actual change because only one load position was conducted at the second load test, and as mentioned previously, the theoretical deflections overestimated the actual deflections. For the Decatur bridge, the interlaminar compression level remained virtually unchanged from the time of load test 1 to load test 2 and the corresponding change in stiffness was negligible.

Condition Assessment

Condition assessments of the Dean, Hibbsville, and Decatur bridges indicate that structural performance is acceptable with minor serviceability deficiencies. Inspection results for specific items follow.

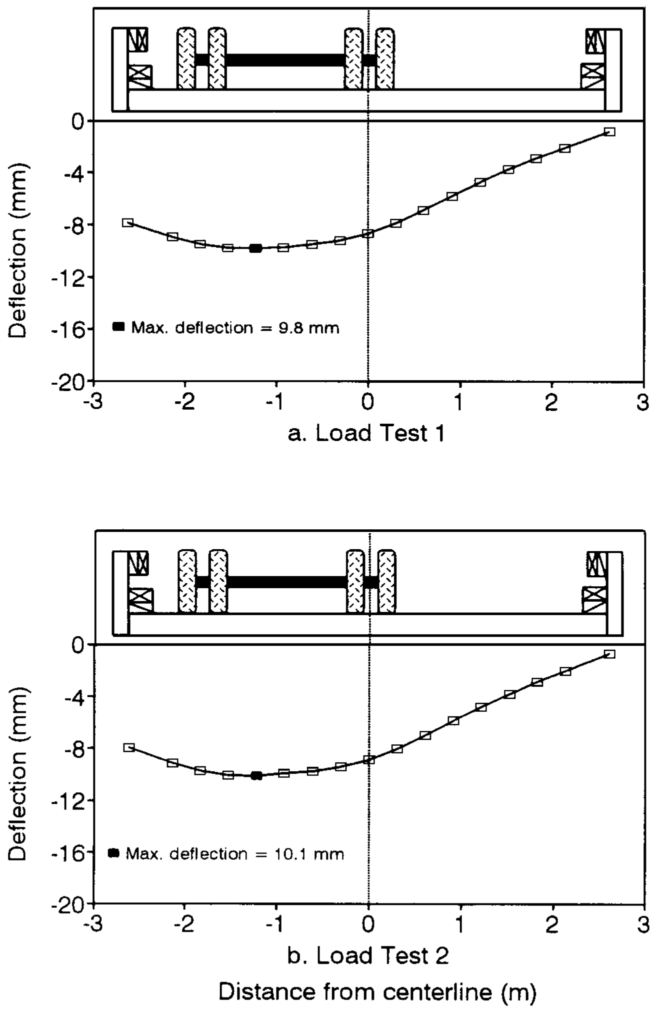


Figure 17—Maximum theoretical midspan deflection profiles for AASHTO HS 20-44 truck loading on the Hibbsville bridge (looking west).

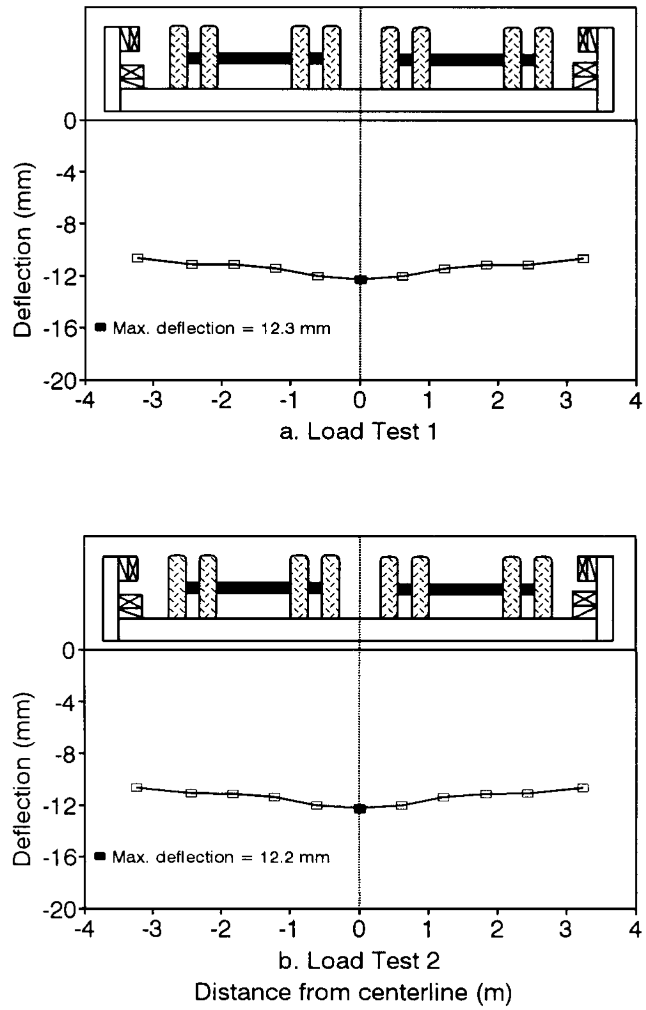


Figure 18—Maximum theoretical midspan deflection profiles for AASHTO HS 20-44 truck loading on the Decatur bridge (looking west).

Table 1—Results from orthotropic plate analysis for AASHTO HS20–44 design loading

	Maximum deflection (mm) (in.)		Deflection as fraction of span		Interlaminar compression (kPa) (lb/in^2)		Change in stiffness (%)
Bridge	Load test 1	Load test 2	Load test 1	Load test 2	Load test 1	Load test 2	from load test 1 to load test 2
Dean	7.0 (0.28)	6.3 (0.25)	1/971	1/1079	437.1 (63.4)	529.5(76.8)	+10
Hibbsville	9.8 (0.39)	10.1 (0.40)	1/718	1/697	535.7 (77.7)	448.2 (65.0)	–3
Decatur	12.3 (0.48)	12.2 (0.48)	1/563	1/567	490.2 (71.1)	492.3 (71.4)	+1

Bridge Geometry

Inspection of the geometry of the Dean, Hibbsville, and Decatur bridges revealed diverse conditions. The largest deck distortion occurred in the Dean bridge. At the time of the first load test, measurements of the width of the bridge indicated that the south deck edge was essentially straight and the north deck edge was approximately 216 mm (8.5 in.) narrower in width at midspan than at the abutments. This distortion was remeasured at the time of the second load test, revealing no additional reduction in width (Fig. 19). It is suspected that this distortion was largely attributable to the layout of the cottonwood laminations, which varied in width as much as 13 mm (0.5 in.) at the butt joints along the north third of the bridge width. This could have been prevented had the laminations been surfaced to a uniform thickness. In addition to the width distortion of the Dean bridge, only three laminations were bearing on the south end of the east abutment along the first 1.8 m (6 ft) of bearing.

Examination of the Decatur bridge revealed a bearing condition somewhat similar to that of the Dean bridge. Both corners of the deck along the west abutment curled upward approximately 38 mm (1.5 in.) (Fig. 20). At both edges, the displacement of the deck relative to the abutment gradually decreased until at 457 mm (18 in.) from each deck edge, the deck made contact with the abutment. This condition remained unchanged throughout the monitoring period. In addition to this distortion, the first bar at the southeast corner was observed to be angling upward (Fig. 21).

Figure 19—Distortion of north edge of Dean bridge. The width is approximately 216 mm (8.5 in.) narrower at the midspan than at the abutments.

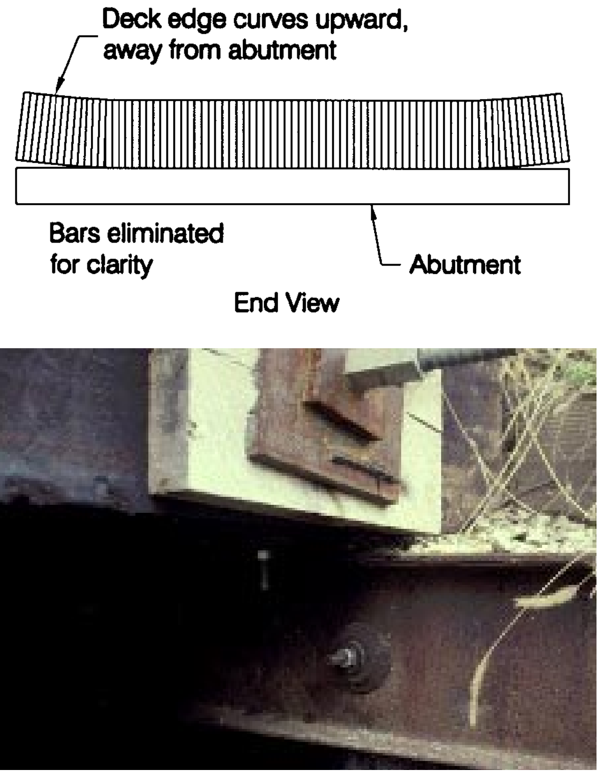


Figure 20—(top) Schematic of west abutment of Decatur bridge. The edges of the deck curve upward away from abutment; (bottom) gap between abutment and deck at the northwest corner of the Decatur bridge.

Figure 21—Stressing bar angles upward at southeast corner of Decatur bridge.

The uneven bearing contact and the bar distortion are probably a result of constructing the deck on uneven temporary supports.

Inspection of the geometry of the Hibbsville bridge revealed no deficiencies, although it was noted that the depth of the laminations varied, resulting in an uneven deck surface (Fig. 22).

Wood Condition

For each of the bridges, inspection of the wood components showed no signs of deterioration, although minor checking was evident in timber members that had been exposed to wet–dry cycles. At the Decatur bridge, creosote dripped from the deck onto the concrete abutments and rocks below the bridge. Creosote also exuded from the deck when probe pins were driven for moisture content measurements. This indicates that the laminations may have been treated to a higher than necessary level of creosote. There was no evidence of wood preservative loss at the Dean or Hibbsville bridges.

Wearing Surface

Both the Dean and Decatur bridges have a well-compacted gravel wearing surface, approximately 51 to 76 mm (2 to 3 in.) thick. The absence of a watertight membrane on the deck surface allows moisture to directly contact the bridge surface and drain through the bridge. This was illustrated at the time of the second Decatur bridge load test, when water was observed dripping from the underside of the deck at the butt joint locations. In terms of material deterioration, the presence of water should not be a problem as long as the wood deck continues to be protected by the gravel wearing surface and the preservative envelope is not broken. However, because the laminations are repeatedly allowed to become wet and the gravel surface prevents rapid drying, the deck will maintain a high moisture content level, thereby enhancing the stress relaxation of the laminations.

The Hibbsville bridge has no wearing surface. At the time of the first load test, gravel and debris were present on the bridge deck. At the second load test it was raining, and the surface of the deck and the approaches were extremely muddy (Fig. 7b). Erosion of backfill was also apparent in several locations, exposing the lamination ends, and water was observed to be dripping from the underside of the deck. The gravel and debris tracked onto the bridge by traffic and the uneven surface of the deck (Fig. 22) are factors that will accelerate deterioration of the timber deck. Exposure of the ends of the deck provides a location for vehicle tires to impact and damage the deck. However, because of the low volume of traffic, erosion of the surface of the deck and subsequent penetration of the preservative envelope have not occurred. As with the other bridges, the absence of a watertight membrane permits the laminations to become wet. The presence of mud greatly hinders drying of the deck, and a high lamination moisture content is preserved, thereby enhancing the stress relaxation of the laminations.

Anchorage System

The second condition assessment of each bridge revealed that the discrete plate anchorage system was performing as designed. Crushing of the steel and white oak bearing plates into the outer deck laminations at each bridge was negligible. No distortion was exhibited by the steel plates, but the white oak plates did show slight deformation, resulting from the stressing bar force (Fig. 23). The ungalvanized steel plates showed signs of corrosion, but the galvanized stressing bars did not. At this time, corrosion of the plates was minor and not affecting system performance. The rate of corrosion is expected to be slow because these roads are not salted during the winter. However, the plates may eventually deteriorate to the point where they are no longer structurally sound. Such deterioration is preventable by galvanizing all steel components prior to construction.

Figure 22—Uneven deck surface of the Hibbsville bridge as a result of unequal depth of the laminations.

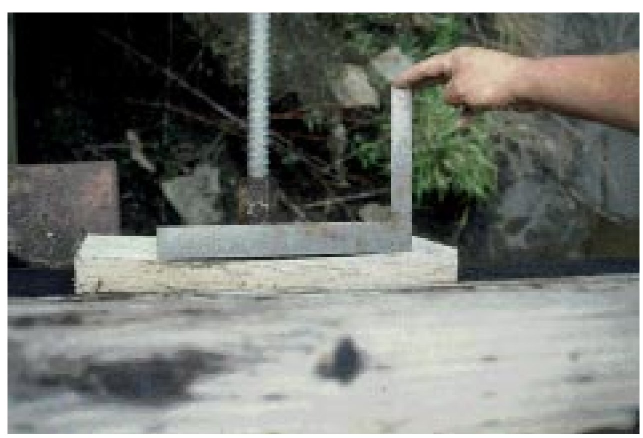


Figure 23—Top view of white oak bearing plate deformation in the Decatur bridge.

Inspection of the anchorage systems at the Dean and Decatur bridges revealed checks in the white oak plates (Fig. 4). Currently, the oak plates are not adversely affected by the checks and are performing as designed. However, because the plates are untreated and the checks provide avenues for moisture penetration, it is expected that the plates will eventually deteriorate and have a negative effect on the bar force. This can be prevented by treating the plates with a wood preservative prior to installation or substituting galvanized steel plates for the oak plates. In addition, conclusive evidence does not exist that the white oak plates perform the same function as do the full-length, dense hardwood exterior laminations. Bar force loss occurred rapidly; therefore, the anchorage systems were not required to perform under the full design force for an extended period. The reduced compression beneath the bearing plates explains the absence of exterior lamination crushing.

Figure 24—Two steel plates butt jointed and welded together to form bearing plate at the Dean bridge.

During the anchorage inspection at the Dean bridge, it was noted that one steel bearing plate was composed of two solid-steel plates, butt jointed at the centerline and welded together (Fig. 24). At present, the plate is performing properly; however, this modification is not recommended because it produces a bearing plate more susceptible to bending deformation.

Conclusions

After approximately 2 years of service, the three eastern cottonwood bridges are performing well with minor serviceability deficiencies. Based on the monitoring conducted for each bridge, the following conclusions are given:

- It is practical and feasible to construct stress-laminated decks using eastern cottonwood, a species not typically used in structural applications.

- The average lamination moisture content was approximately 24% for the Dean and Hibbsville bridges at the beginning and end of the monitoring, and 21% and 26% for the Decatur bridge at the beginning and end of the monitoring, respectively. The moisture content of the Hibbsville bridge remained relatively stable throughout this time, with minor fluctuations of 2% to 3% in the measurement zone as a result of seasonal climatic changes. The high moisture content level has not adversely affected the structural integrity of the bridges, although it has contributed to vertical creep and a high level of stress relaxation. A lamination moisture content less than or equal to 19% at installation is recommended.

- For each bridge, the bars were initially tensioned to approximately the design force of 320 kN (72,000 lb) for the Dean bridge and 299 kN (67,200 lb) for the Hibbsville and Decatur bridges. For these bar tensionings as well as subsequent tensionings, initial force losses occurred rapidly and continued for approximately 2 months at which time the rate of loss decreased. The decline in bar force is attributed to stress relaxation of the lumber laminations enhanced by the high moisture content. Future bridge inspections should verify bar forces to ensure adequate interlaminar compression, and bars should be retensioned as required.

- The Hibbsville bridge deck experienced vertical creep, resulting in a slight sag at the conclusion of the monitoring. The high moisture content of the cottonwood laminations probably influenced the vertical creep of this bridge. The Decatur bridge exhibited no vertical creep during monitoring.

- Static load tests and analysis indicate that each of the three bridges is performing as a linear elastic orthotropic plate 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 increased approximately 10% for the Dean bridge and decreased approximately 3% for the Hibbsville bridge. Although other factors may have contributed, it is likely that the changes in stiffness were due primarily to changes in interlaminar compression. For the Decatur bridge, the interlaminar compression level remained virtually unchanged from the time of load test 1 to load test 2 and the corresponding change in stiffness was negligible.

- Static load test results for the Decatur bridge indicate that the longitudinal stiffness of the north side of the bridge is greater than the south side.

- Lamination width in a stress-laminated bridge should be uniform to ensure full contact between laminations and prevent gaps at the butt joints that, as illustrated by the Dean bridge, result in deck distortion.

- Uneven bearing contact of the Dean and Decatur bridges was probably caused by constructing the deck on uneven temporary supports.

- The timber components of each bridge exhibit no signs of deterioration.

- The bridges are either unsurfaced or have a gravel wearing surface. The gravel and debris tracked onto the bridge trap moisture and do not allow the deck to dry. The addition of an asphalt wearing surface and an underlying asphalt impregnated geotextile fabric would help keep the wood decks dry and prevent water from dripping through the laminations.

- The unprotected deck surface of the Hibbsville bridge shows no signs of damage or deterioration. However, the presence of gravel and debris on the deck of the bridge, the various depth of the laminations that resulted in an uneven wearing surface, and the erosion of backfill are all factors that accelerate deterioration of the lumber laminations caused by vehicle wear.

- The anchorage system of each bridge is performing as designed. Crushing of the exterior lumber laminations is negligible. The exposed galvanized steel stressing bars and nuts show no visible signs of corrosion or other distress. No distortion is exhibited by the steel plates, although the oak plates display slight deformation. The ungalvanized steel bearing and anchor plates should be galvanized to prevent corrosion, and the white oak plates should be treated with a wood preservative to prevent decay.

- The white oak bearing plates appear to perform the same function as do full-length dense hardwood exterior laminations when initially installed. However, the deformation as a result of bending and splits observed in the plates during condition assessments indicates that, over time, the discrete hardwood plates do not perform the same function as the full-length laminations. It is expected that deterioration of the white oak plates will negatively affect bar force.

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Appendix—Bridge Characteristics

	Dean bridge	Hibbsville bridge	Decatur bridge
General			
Location	8 km (5 miles) southwest of Moulton, Iowa	3.2 km (2 miles) southwest of Numa, Iowa	3.2 km (2 miles) southwest of Davis City, Iowa
Date of Construction	October 1993	January 1994	June 1994
Owner	Appanoose County, Iowa	Appanoose County, Iowa	Decatur County, Iowa
Design Configuration			
Structure Type	Stress-laminated deck with butt joints	Stress-laminated deck with butt joints	Stress-laminated deck with butt joints
Butt Joint Frequency	1 in 4 laminations transverse with joints in adjacent laminations separated 1.2 m (4 ft) longitudinally	1 in 4 laminations transverse with joints in adjacent laminations separated 1.2 m (4 ft) longitudinally	1 in 4 laminations transverse with joints in adjacent laminations separated 1.2 m (4 ft) longitudinally
Total Length (out-out)	7.35 m (24.1 ft)	7.19 m (23.6 ft)	7.22 m (23.7 ft)
Skew	None	None	None
Number of Spans	1	1	1
Span Length (center-to center of bearings)	6.80 m (22.3 ft)	7.04 m (23.1 ft)	6.92 m (22.7 ft)
Width (out-out)	6.98 m (22.9 ft)	5.24 m (17.2 ft)	6.46 m (21.2 ft)
Number of Traffic Lanes	2	1	2
Design Loading	AASHTO HS20–44	AASHTO HS20–44	AASHTO HS20–44
Wearing Surface Type	Gravel	none	Gravel
Material and Configuration			
Timber:			
Species	Eastern Cottonwood	Eastern Cottonwood	Eastern Cottonwood
Size (actual)	381 mm (15 in.) deep	356 mm (14 in.) deep	356 mm (14 in.) deep
Moisture content shortly after installation	24%	24%	21%
Preservative Treatment	Creosote	Creosote	Creosote
Stressing Bars:			
Type	High strength steel bar with coarse right-hand thread, conforming to ASTM A722	High strength steel bar with coarse right-hand thread, conforming to ASTM A722	High strength steel bar with coarse right-hand thread, conforming to ASTM A722
Diameter	25.4 mm (1 in.)	25.4 mm (1 in.)	25.4 mm (1 in.)
Number	6	6	6
Design Force	320 kN (72,000 lb)	299 kN (67,200 lb)	299 kN (67,200 lb)
Spacing	1.2 m (4 ft)	1.2 m (4 ft)	1.2 m (4 ft)
Anchorage Type and Configuration:			
White Oak Bearing Plates	457 by 305 by 38 mm (18 by 12 by 1.5 in.)	none	457 by 311 by 44 mm (18 by 12.25 by 1.75 in.)
Steel Bearing Plates	311 by 305 by 19 mm (12.25 by 12 by 0.75 in.)	610 by 305 by 19 mm 254 by 254 by 19 mm (24 by 12 by 0.75 in. 10 by 10 by 0.75 in.)	254 by 254 by 19 mm (10 by 10 by 0.75 in.)
Steel Anchor Plates	127 by 127 by 19 mm 127 by 127 by 13 mm (5 by 5 by 0.75 in. 5 by 5 by 0.5 in.)	152 by 152 by 19 mm (6 by 6 by 0.75 in.)	127 by 127 by 19 mm (5 by 5 by 0.75 in.)