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Field Performance of Timber Bridges

9. Big Erick's Stress-Laminated Deck Bridge

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Abstract

The Big Erick's bridge was constructed during September 1992 in Baraga County, Michigan. The bridge is 72 ft long, 16 ft wide, and consists of three simple spans: two stress-laminated deck approach spans and a stress-laminated box center span. The bridge is unique in that it is one of the first known stress-laminated timber bridge applications to use Eastern Hemlock sawn lumber and a combination of stress-laminated decks and a stress-laminated box in a single bridge. Performance of the bridge was monitored for 35 months, beginning at the time of installation. Monitoring involved gathering and evaluating data relative to the moisture content of the wood components, the force level of stressing bars, and the behavior of the bridge under static-load conditions. In addition, comprehensive visual inspections were conducted to assess the overall condition of the structure. Based on field evaluations, the bridge is performing well, with only minor serviceability deficiencies.

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Introduction

In 1988, the U.S. Congress passed legislation known as the Timber Bridge Initiative (TBI) (USDA 1995). The objective of this legislation was to establish a national program to provide effective and efficient utilization of wood as a structural material for highway bridges. Responsibility for the development, implementation, and administration of a program based on the goals of the TBI legislation was assigned to the USDA Forest Service. The Forest Service established three primary program areas to effectively implement and administer this program: demonstration bridges, technology transfer, and research. The demonstration bridge program and technology transfer are administered through the Timber Bridge Information Resource Center (TBIRC) in Morgantown, West Virginia. As part of the demonstration bridge program, TBIRC has been awarding annual grants for demonstration bridges on a competitive basis since 1989. Funds are awarded for the design and construction of demonstration timber bridges with innovative designs and those that use locally available underutilized wood species. The TBIRC also maintains a technology transfer program to provide assistance and state-of-the-art information related to all aspects of timber bridges. Responsibility for the research portion of the TBI program was assigned to the USDA Forest Service, Forest Products Laboratory (FPL). The FPL established a broad research program to conduct a variety of timber bridge studies under laboratory and field conditions, including a nationwide bridge monitoring program. Through the bridge monitoring program, FPL is able to collect, analyze, and distribute information on the field performance of timber bridges to provide a basis for validating or revising design criteria and further improving efficiency and economy of timber bridge design, fabrication, and construction.

This report is ninth in a series that documents the field performance of timber bridges included in the FPL national timber bridge monitoring program. It describes the development, design, construction, and field performance of the Big Erick's bridge located in Baraga County, Michigan.

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)
square foot (ft^2)	0.09	square meter (m^2)
pound (lb)	4.448	Newton (N)
lb/in^2 (stress)	6,894	Pascal (Pa)

The bridge is a 72-ft-long, single-lane, three-span structure, consisting of stress-laminated deck approach spans and a stress-laminated box center span. (See Table 1 for metric conversion factors.) Built in 1992, the Big Erick's bridge was funded through a competitive grant from the TBIRC and matching funds from the Michigan Department of Natural Resources. The bridge is one of the first known applications that uses Eastern Hemlock lumber, a locally underutilized secondary softwood species, in a stress-laminated configuration. The bridge is also the first known application where a stress-laminated deck is combined with a stress-laminated box. An information sheet on the Big Erick's bridge is provided in the Appendix.

Background

The Big Erick's bridge is located in Baraga County, approximately 20 miles northeast of L'Anse, in the western Upper Peninsula of Michigan (Fig. 1). The bridge is owned by the Michigan Department of Natural Resources (MI–DNR) and is part of a state forest gravel road that provides access to the Huron River valley for recreation, private residences, logging traffic, and fire control equipment. The estimated average daily traffic for this section of the road is approximately 30 vehicles, of which 30 percent is logging traffic.

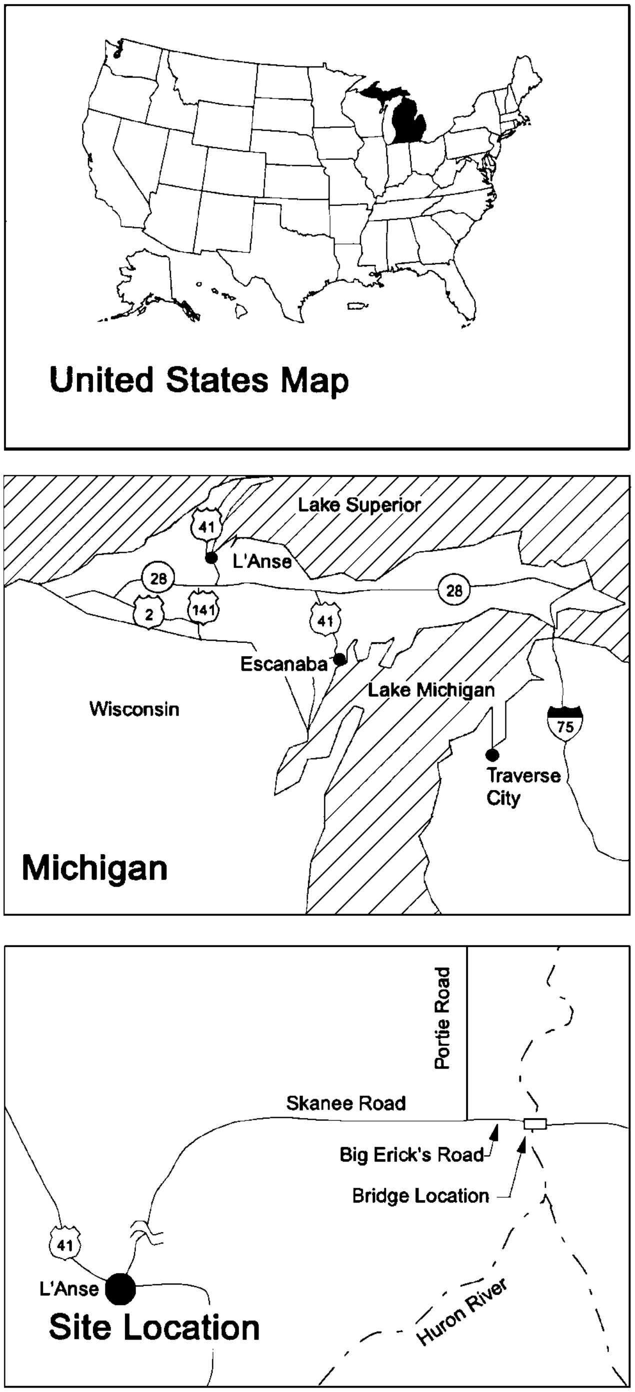

Figure 1—Location maps for the Big Erick's bridge.

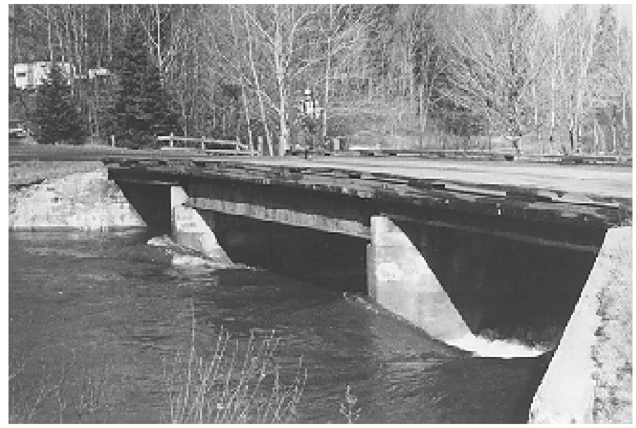

Figure 2—Big Erick's bridge prior to replacement (1980).

The original Big Erick's bridge was constructed in 1957 by the MI–DNR Fire Division to provide access for administrative vehicles, fire equipment, and logging trucks. This bridge was 20 ft wide and consisted of two 20-ft-long approach spans and a 32-ft-long center span (Fig. 2). Beneath two layers of timber plank deck, the approach spans were supported by untreated sawn lumber stringers and the center span was supported by steel stringers. Failure of the deck material in 1980 alerted MI–DNR officials to structural problems with the bridge. After the decking material was repaired, an annual inspection in 1989 revealed cracked and decayed wood stringers on the approach spans. Several options were examined to repair or replace the structure. As a result of uncertainty regarding the extent of rehabilitation required to restore full-load capacity, replacement of the bridge was deemed the most feasible solution.

As a result of the high volume of public recreation in the area, the MI–DNR chose a timber bridge as a replacement to blend with the natural aesthetics of the area. The Upper Peninsula Resource Conservation and Development Council was contacted to submit a proposal for partial funding for a replacement demonstration timber bridge under the Forest Service demonstration program. As part of this proposal, a preliminary bridge design was developed, consisting of two stress-laminated decks and one stress-laminated box. Eastern Hemlock was selected as a primary material for the bridge because it is an underutilized, locally grown species in the Upper Peninsula of Michigan. The proposal was submitted in January 1991 to the TBIRC for inclusion under the Timber Bridge Initiative (USDA 1995). After review by a selection panel, funds were awarded and final design of the replacement bridge was initiated. During the final design phase of the project, FPL was contacted to monitor the new Big Erick's timber bridge. As a result, FPL and MI–DNR developed a mutually agreeable monitoring plan that was initiated at installation.

Objective and Scope

The objective of this project was to evaluate the field performance of the Big Erick's bridge for 35 months, beginning at bridge installation. The project scope included data collection and analysis related to the wood moisture content, stressing bar force, bridge behavior under static truck loading, and general structure performance. The results of this project will be considered with similar monitoring projects to improve design and construction methods for future stress-laminated bridges.

Design, Construction, and Cost

The design and construction of the Big Erick's bridge was a cooperative effort involving several agencies and individuals. An overview of the design, construction, and cost of the bridge superstructure is presented.

Design

Design of the Big Erick's bridge was completed by a consulting engineer retained by MI–DNR. The design geometry provided for three simple spans with a total length of 72 ft and a width of 16 ft. The structure consists of two 20-ft-long, stress-laminated deck approach spans and one 32-ft-long, stress-laminated box center span (Fig. 3). The span lengths were designed to match existing reinforced concrete abutment and pier configurations, because the substructure was in serviceable condition. Each span was also designed for construction as preassembled half-width panels to improve transportation and on-site assembly.

Aside from those design aspects related to stress laminating, the bridge design was in accordance with the American Association of State Highway and Transportation Officials (AASHTO) *Standard Specifications for Highway Bridges* (AASHTO 1989) for a single lane of AASHTO HS25–44 truck loading. Design criteria for stress laminating were different for the deck and box spans and are discussed in detail in the following sections on approach span design and center span design. Common to both stress-laminated designs was the use of 0.625-in. coarse, right-hand thread, high strength steel bars, which comply with the requirements of ASTM A722 (ASTM 1988) and provide a minimum ultimate tensile strength of 150,000 lb/in^2.

Design of the bridge rail and curb system was based on AASHTO static-load requirements for vehicular and pedestrian traffic. The bridge rail and curb consisted of two 5.125- by 6-in. glued-laminated (glulam) timber rails and a 6- by 12-in. sawn lumber curb with 6- by 12-in. sawn lumber scupper blocks. The rail and curb were attached to 8.75- by 10.5-in. glulam posts, spaced 6 ft on center along the edges of the bridge (Fig. 3, Guardrail and Anchorage).

For protection from deterioration, all steel components, including stressing hardware, stressing bars, and anchorage plates, were galvanized in accordance with AASHTO M111 (AASHTO 1990). All wood components were preservative treated with creosote in accordance with American Wood Preservers' Association (AWPA) Standard C14 (AWPA 1990). To protect pedestrians from direct contact with creosote-treated lumber, the glulam rails were covered with CCA-treated dimension lumber facia boards. In addition, a 2.75-in. asphalt wearing surface was specified for the bridge and approach roadway as a means of protecting the deck surface from the premature deterioration resulting from direct exposure to traffic and the elements.

Approach Span Design

The stress-laminated deck approach spans were identical in design and configuration. Design criteria for stress laminating were based on a newly developed Forest Service design procedure (Ritter 1990). Each span was 20 ft long, 16 ft wide, and 11 in. deep (Fig. 3). The laminations consisted of 2- by 11-in. Eastern Hemlock sawn lumber that was surfaced on one side to provide uniform thickness. Eastern Hemlock was not available in 20-ft lengths; therefore, butt joints were used in the decks. Butt joints were spaced across the width and length at intervals of eight laminations and 4 ft, respectively (Fig. 4). Design values for the Eastern Hemlock laminations were based on the National Design Specification for Wood Construction (AFPA 1988) for lumber visually graded No. 1, in accordance with Northeastern Lumber Manufacturing Association (NELMA) rules (NELMA 1986). The tabulated design values for the Eastern Hemlock species combination were 1,300 lb/in^2 for bending strength, 1,200,000 lb/in^2 for modulus of elasticity (MOE), 85 lb/in^2 for shear strength, and 550 lb/in^2 for compression strength perpendicular to grain. All design values were adjusted with the appropriate wet-use factors, and laminations were specified to be at or less than 19 percent moisture content prior to preservative treatment and bridge installation.

The stressing system for the stress-laminated decks utilized nine high strength steel bars per span that were inserted through predrilled holes at the center of the laminations (Fig. 3, Section A-A). The bars were spaced 2 ft on center, starting 2 ft from the span end. Each bar required a tensile force of 26,400 lb to provide 100-lb/in^2 interlaminar compression between the laminations. Bar tension was trans- ferred into the deck using a discrete plate anchorage system, consisting of 6- by 12- by 0.875-in. bearing plates and a 2- by 5- by 1-in. anchor plate with a hexagonal nut (Fig. 3 Guardrail and Anchorage).

Center Span Design

Design requirements for the stress-laminated box center span were based on standard design criteria adopted by West Virginia Department of Highways (WV–DOH 1989). The span is 32 ft long, 16 ft wide, and 22.5 in. deep (Fig. 3). The box configuration consisted of Southern Pine glulam webs and Eastern Hemlock sawn lumber flanges. Glulam webs were 8.75 in. wide and 22.5 in. deep, except for the two center webs that were 5.125 in. wide and 22.5 in. deep. Glulam design was based on material properties for combination 24F–V3, as specified by the American Institute of Timber

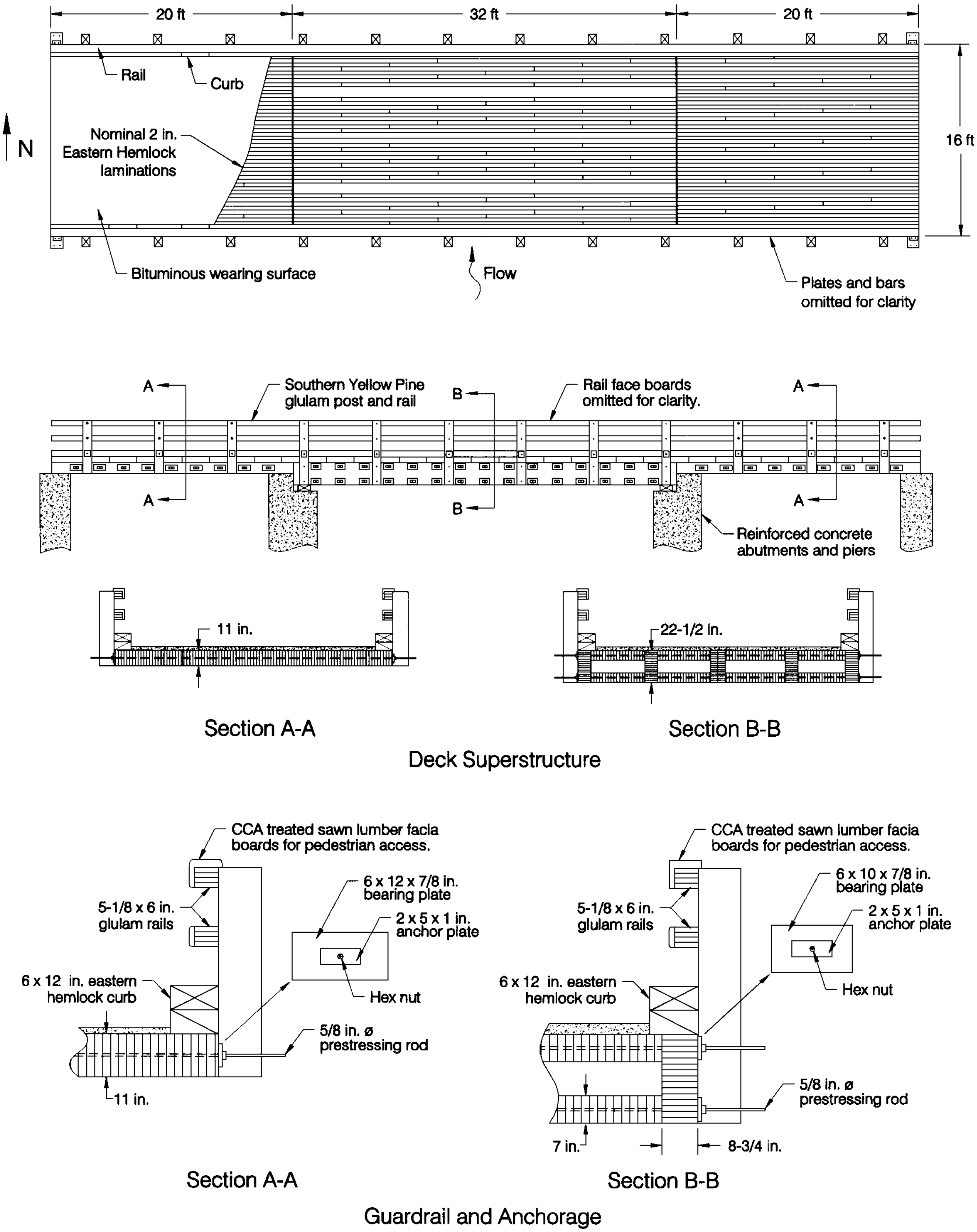

Figure 3—Design configuration of the Big Erick's bridge.

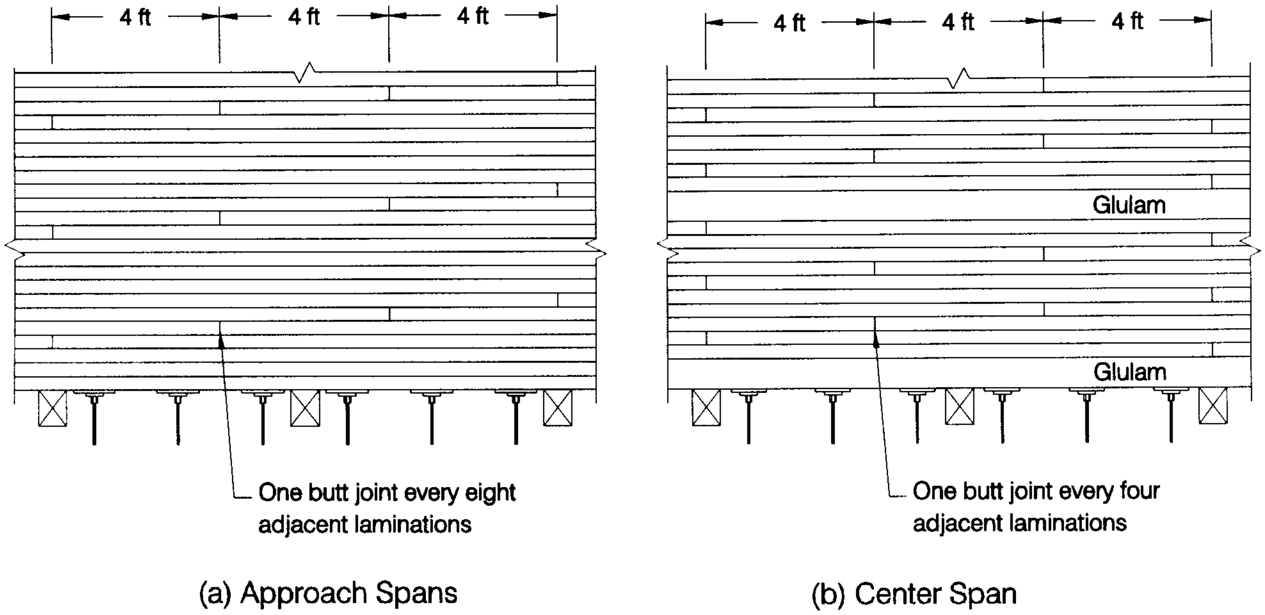

Figure 4—Butt-joint configuration used with Eastern Hemlock deck laminations for the (a) approach spans and (b) center span on the Big Erick's bridge. A butt joint was placed transverse to the span in every (a) eighth lamination on the stress-laminated decks and (b) fourth lamination on the stress-laminated box. Longitudinally, butt joints in adjacent laminations were separated by 4 ft.

Construction (AITC) *Standard Specifications for Structural Glued Laminated Timber for Softwood Species* (AITC 1987). Tabulated design values were 2,400 lb/in^2 for bending strength, 1,800,000 lb/in^2 for MOE, 200 lb/in^2 for shear strength, and 560 lb/in^2 for compression strength perpendicular to grain. All design values were adjusted by appropriate wet-use factors per AITC requirements. The sawn lumber flanges, 7 in. deep, were the same species and grade of Eastern Hemlock as was used for the approach spans. Butt joints in the sawn lumber flanges were specified at an interval of one butt joint every four adjacent laminations, spaced 4 ft longitudinally (Fig. 4).

The stressing system for the center span consisted of 30 high strength steel bars placed in pairs, 3.5 in. from the top and bottom of the glulam beams (Fig. 3, Section B-B). The bars were spaced 2 ft on center, starting 2 ft from the span ends, and were placed through the center of the top and bottom sawn lumber flange. A tensile force of 16,800 lb was required in each bar to provide 100 lb/in^2 of interlaminar compression. Bar tension was transferred into the box using a discrete plate anchorage system, consisting of 6- by 10- by 0.875-in. bearing plates and 2- by 5- by 1-in. anchor plates with a hexagonal nut (Fig. 3, Guardrail and Anchorage).

Construction

Construction of the Big Erick's bridge was completed by MI–DNR personnel and local prison labor in the summer and fall of 1992. Construction extended over several months and was completed in two phases: assembly and installation.

Assembly

As previously mentioned, the bridge was designed for preassembly in half-width sections at a location other than the bridge site. The process began with the harvesting and milling of Eastern Hemlock sawn lumber in the Upper Peninsula of Michigan. The material was then airdried and stored outside for approximately 5 months prior to preservative treatment (Fig. 5). During this period, the glulam beams were manufactured in Wisconsin. Following drying and manufacture, all sawn lumber and glulam members were prefabricated (cut and drilled) and sent to Indiana for pressure preservative treatment with creosote. When the preservative treatment was completed, the material was shipped to Baraga, Michigan, for assembly. During 3 weeks in July 1992, the sawn lumber and glulam beams were assembled into half-width panels. The approach span deck assembly was initiated by placing the laminations edgewise on temporary supports and nailing them together. Bars were then inserted through predrilled holes (Fig. 6). The center span box assembly proceeded in a similar manner. The exterior web was placed on temporary supports, and the bottom flange was nailed to the web. The next web was placed adjacent to the bottom flange, and the top flange was nailed in place. After the top and bottom flanges were in place, the bars were inserted through the predrilled holes (Fig. 6). Both assembly processes continued until 8-ft-wide panels were completed. After each panel was assembled, bearing plate anchorage systems were installed on all the bars. The stressing bars were then tensioned along the length of the half-width panel to a design load of 26,400 lb for the approach spans and 16,800 lb for the center span.

Figure 5—Eastern Hemlock lumber for the Big Erick's bridge awaiting preservative treatment.

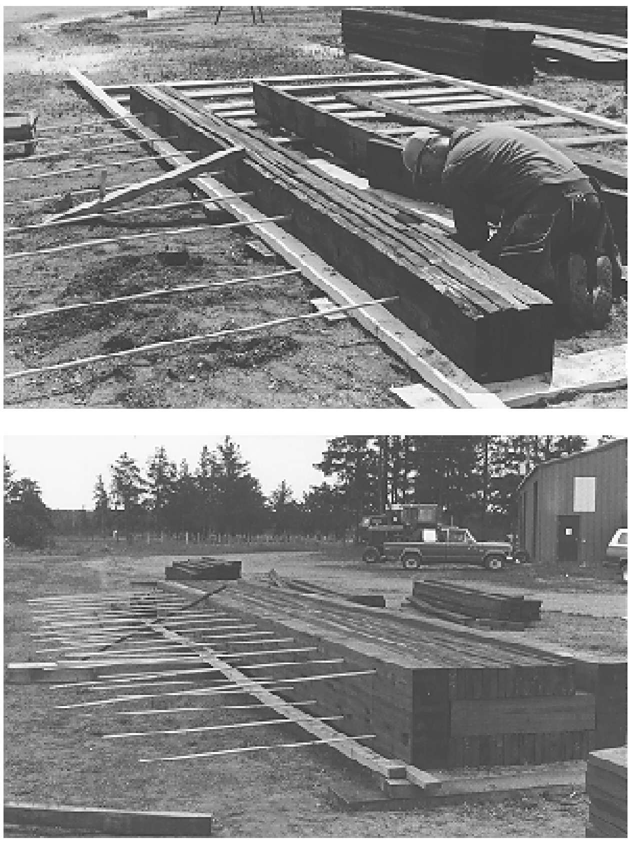

Figure 6—Prefabrication of the stress-laminated deck (top) and stress-laminated box (bottom) in half-width panels at an outdoor work site.

The stressing was accomplished with a hydraulic jacking system, which consisted of a hydraulic pump, a single hollow core jack, and a stressing chair (Fig. 7) (Wacker and Ritter 1992). After all panels were assembled and stressed, a second design load stressing was completed August 26, 1992, approximately 1 month after the initial stressing. The panels were then loaded on two flatbed trailers and transported to the bridge site in preparation for bridge installation (Fig. 8).

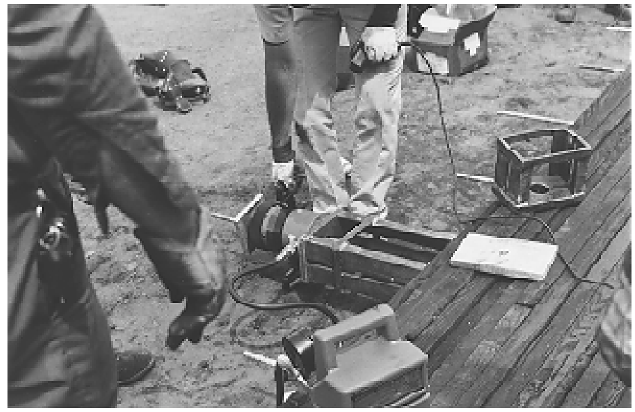

Figure 7—High strength steel bars were tensioned using a single hydraulic jack, electric pump, and steel chair.

Figure 8—Prefabricated half-width panels arriving at the bridge site prior to installation.

Installation

After demolition and removal of the existing bridge superstructure, an assessment of the reinforced concrete substructure revealed that the west abutment had sustained freeze-thaw damage. To remedy the problem, the damaged part of the west abutment was removed, and the remaining portion of the abutment was capped with new concrete. The remainder of the substructure was structurally adequate, and the complete substructure was coated with a sealer prior to the new bridge installation. Wood sleeper blocks, measuring 12.5 by 5.75 in., were added at the center span piers to raise the new bridge to the existing roadway grade. After the addition of neoprene bearing pads, additional modifications were not made to the abutments or piers.

When the substructure work was completed, the assembled half-width panels were lifted into place with a large overhead crane (Fig. 9). After the half-width panels were placed, each panel was connected to the adjacent panel with high strength steel couplers on the stressing bars (Fig. 10). After all bars were coupled, stressing bars were partially tensioned to bring the half-width panels into contact. Full-design bar force was then introduced into the bridge by utilizing two separate sets

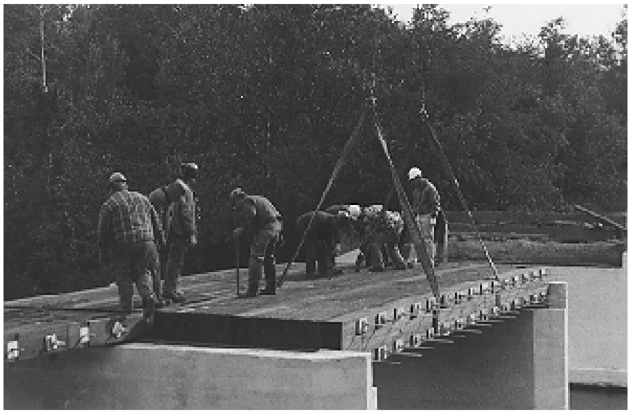

Figure 9—Placement of half-width panels with an overhead crane.

Figure 10—Connection betwen half-width panels was achieved by placing couplers on adjacent bars and stressing the entire deck together.

of hydraulic equipment. The MI–DNR crew began at opposite ends of each span and stressed each bar along the length of that span (Fig. 11). On the center span, each set of bars (top and bottom) was tensioned before moving along the span length to the next set of bars. After all bars were fully tensioned, the stressing process was repeated to ensure that the stress level was uniform and at the required design level.

After bar stressing, the bridge was attached to the substructure. The two approach spans were connected by steel angles, concrete anchors, and lag bolts located in the corners, at the abutments, and underneath the deck at the piers. The center span was connected by drift pins driven through the end of each glulam web to the sleeper block on the concrete pier.

Following substructure attachment, the timber curb and rail system was installed. As a result of expected pedestrian traffic from the adjoining campground, CCA-treated lumber facia boards were installed over the creosote rail members and a roofing sealer was applied to the curb. The asphalt wearing surface was applied approximately 2 weeks after bridge installation. The completed bridge is shown in Figure 12.

Figure 11—Two sets of deck stressing equipment were used to tension the stressing bars after deck placement.

Figure 12—Completed Big Erick's bridge: side view (top), end view (bottom).

Table 2—Final costs of bridge superstructure

Span	Deck area (ft^2)	Cost (US$) Materials	Labor	Total	Per ft^2
Approach[a]	640	12,390	15,391	27,781	43
Center	512	24,909	13,467	38,376	75

[a]Costs are based on total costs for west and east spans.

Cost

Total cost of the Big Erick's bridge superstructure was $66,157 and included fabrication, materials, labor, and construction. As a result of the different design methodologies used in this three-span bridge, costs for each span type were tabulated separately and are summarized in Table 2. The cost per square foot is based on the total cost of the span divided by the total deck area of that span. The $75/ft^2 for the center span was almost twice the $43/ft^2 for the approach spans. The increased center span cost is attributable to the greater quantity of material and longer installation time. The increased material cost of the center span is the result of using glulam timber for the webs; two rows of plates, bars, and nuts; and two layers of flange decking material.

Evaluation Methodology

To evaluate the structural and serviceability performance of the Big Erick's bridge, MI–DNR contacted FPL for assistance. Through mutual agreement, a 3-year bridge monitoring plan was developed by the FPL and implemented through a Cooperative Research and Development Agreement with MI–DNR. The plan included performance monitoring of the deck moisture content, stressing bar force, static-load test behavior, and general bridge condition. During the installation of the bridge, FPL representatives visited the bridge site to install instrumentation and train MI–DNR personnel in the data collection process for moisture content and bar force measurements. Static-load tests and general bridge condition assessments were conducted by FPL representatives during site visits. The evaluation methodology employed procedures and equipment developed by FPL and used on similar structures (Ritter and others 1991).

Moisture Content

To characterize changes in moisture content, an electrical-resistance moisture meter was used to obtain wood moisture content readings on a monthly basis. Moisture meter measurements were taken by MI–DNR personnel from the sawn lumber in the west approach span and were assumed to be representative of overall bridge moisture content. The west span was chosen because of safety concerns in accessing other bridge locations. Measurements were obtained in accordance with ASTM D4444–84 (ASTM 1990) by driving the moisture pins into the underside of the deck at depths of 2 to 3 in., recording the moisture content value from the unit, then adjusting the moisture content value for temperature and wood species, if necessary.

Figure 13—One of six load cells installed on stressing bars to measure changes in bar force.

Bar Force

To monitor stressing bar force, two calibrated load cells were placed on each span of the Big Erick's bridge at the time of bridge installation. The cells were placed on the stressing bar, between the bearing and anchorage plates, to monitor bar forces based on the strain variations in the load cell (Fig. 13). Load cell measurements were obtained by MI–DNR personnel with a portable strain indicator on a biweekly basis for the first year, and monthly thereafter. The measurements were then converted to force levels, based on laboratory load cell calibrations, to determine the tensile force in the bar. Approximately midway through the monitoring period, the load cells were unloaded and adjusted for zero balance shift. At the conclusion of the monitoring period, the load cells were removed, adjusted for zero balance shift, and recalibrated in the laboratory. In addition, hydraulic stressing equipment was used at the site visits during the monitoring period to verify bar force levels obtained from the load cells.

Load Test Behavior

To determine load behavior of the Big Erick's bridge, two static-load tests were conducted during the monitoring period. The first load test was completed on two separate occasions; the approach spans were tested 2 months after bridge installation and the center span was tested 11 months after installation. A second and final load test was completed 35 months after bridge installation and included tests on all three spans.

The static-load test consisted of positioning a fully loaded truck separately on each span and measuring the resulting deflections along the transverse midspan of the loaded span. For both load tests, the truck was positioned for three transverse load cases on each span (Fig. 14). The first load case centered the vehicle over the longitudinal centerline; the second load case located the vehicle adjacent to the upstream curb; the third load case located the vehicle adjacent to the

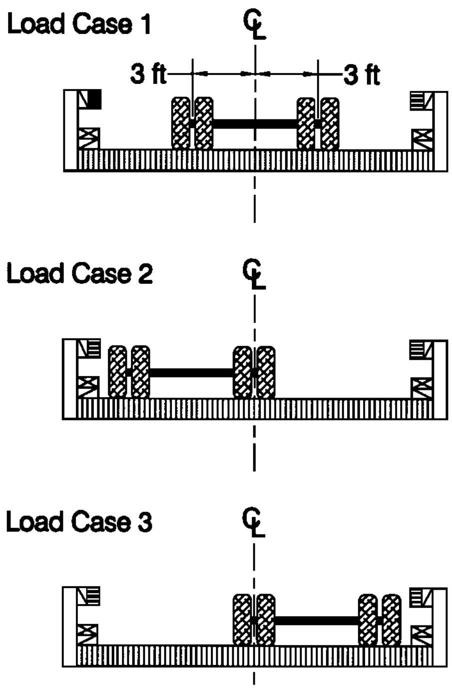

Figure 14—Transverse load cases (looking west) used for all static-load tests on the Big Erick's bridge.


Figure 15—Transverse load test cases used for all spans: (top) load case 1, centered on roadway; (middle) load case 2, adjacent to upstream curb; (bottom) load case 3, adjacent to downstream curb.

downstream curb (Fig. 15). Longitudinal vehicle placement differed, depending upon the span length and vehicle configuration. Measurements of bridge deflections were taken prior to testing (unloaded), for each load case (loaded), and at the conclusion of testing (unloaded). Measurements of bridge deflections from an unloaded to loaded condition were obtained by hanging calibrated rules on the underside of the deck and reading values with a surveyor's level. The accuracy of this method for repetitive readings is estimated to be ±0.04 in.

Load Test 1A

The first load test was November 19, 1992, and involved approach span testing only. The test vehicle was a loaded, three-axle dump truck with a gross vehicle weight of 54,760 lb (Fig. 16). The vehicle was positioned longitudinally, with the tandem rear axles bisecting the midspan with the front axle off the bridge. The truck faced west when testing the west span and east when testing the east span. Data points were transversely positioned along the midspan of each approach span at an interval of 2 ft, beginning at the longitudinal bridge centerline. It should be noted that the data points were not directly below the vehicle wheel lines during load case 1.

Load Test 1B

Load test 1B involved the center span only and was completed August 23, 1993,—9 months after the sawn lumber approach spans were tested. The test vehicle was a loaded, three-axle dump truck with a gross vehicle weight of

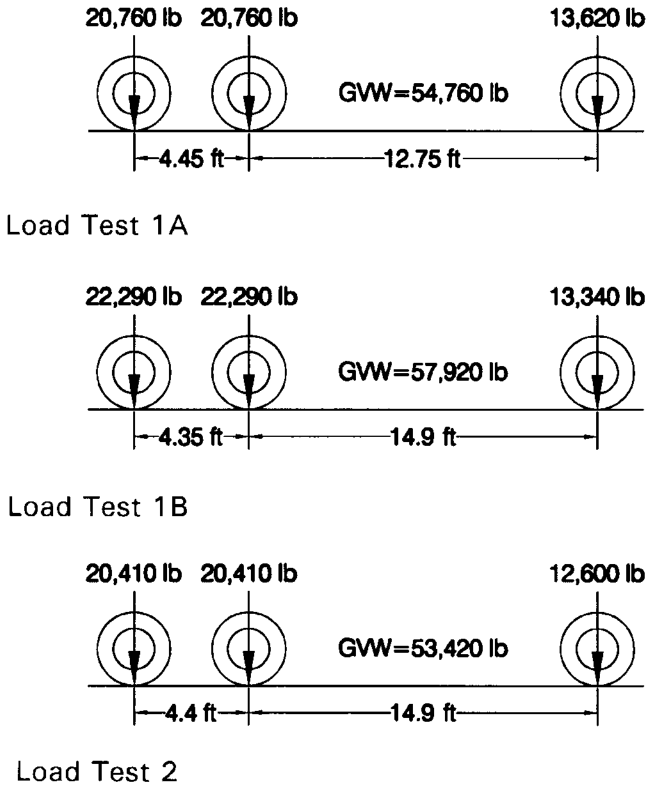

Figure 16—Load test truck configurations and axle loads for load tests 1A, 1B, and 2. All vehicle track widths, measured center–to–center of the rear tires, were 6 ft.

57,92 lb (Fig. 16). The vehicle was positioned longitudinally, facing east, so that the centroid of the vehicle aligned with the midspan. Data points were set at the centerline of each glulam beam at midspan.

Load Test 2

The final load test was completed July 18, 1995, and included testing on all three spans. The test vehicle was a loaded, three-axle dump truck with a gross vehicle weight of 53,420 lb (Fig. 16). The vehicle was positioned longitudinally on each span in the same positions used for load tests 1A and 1B. Data points were located for the approach spans at a transverse spacing of 1.5 ft so that the points aligned with the vehicle wheel lines. Data points for the center span were the same as those used for load test 1B.

Analytical Assessment

At the conclusion of load testing, the approach span behavior was modeled for load test and AASHTO HS 25–44 loading using an orthotropic plate computer program developed at FPL. The center span was not modeled.

Condition Assessment

The general condition of the bridge was assessed on four separate occasions during the monitoring period. Condition

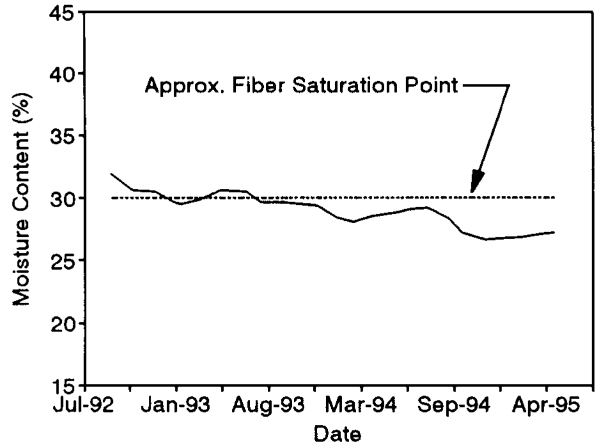

Figure 17—Average trend in electrical resistance moisture content readings.

assessments were performed by FPL personnel at bridge installation and during load testing. The assessments involved visual inspections, measurements, and photographic documentation of the condition of the bridge. Items of specific interest included geometry, camber, wood condition, wearing surface, and stressing system.

Results and Discussion

Performance monitoring of the Big Erick's bridge extended for 35 months, beginning in September 1992. Results and discussion of the performance data follow.

Moisture Content

The average trend in electrical resistance moisture content readings is shown in Figure 17. Readings taken at the beginning of the monitoring period indicate an average moisture content of approximately 33 percent. During the monitoring period, an overall decrease in moisture content was observed, with seasonal changes of ±2 percent. At the end of monitoring, moisture content was approximately 28 percent, a 5-percent decrease. The laminations remained within ±3 percent of the fiber saturation level for Eastern Hemlock, which is generally assumed as 30 percent but can range from 25 to 30 percent (FPL 1987).

Bar Force

The average trend in bar force, starting at the third stressing, is shown in Figure 18. At the third stressing, all bars were tensioned to the design level of 100 lb/in^2, or 26,400 lb for the approach spans and 16,800 lb for the center box. After 2 months, all three spans lost 40 to 50 percent of the design bar force and the bars were retensioned. Eleven months after installation, all three spans lost 50 to 60 percent of the design bar force and were tensioned again. Approximately 2 weeks after this fifth bar tensioning, the bar force stabilized at approximately 70 percent of the design bar force, then decreased slowly for the remainder of the monitoring period. At the conclusion of monitoring, bar force values were 14,800 lb

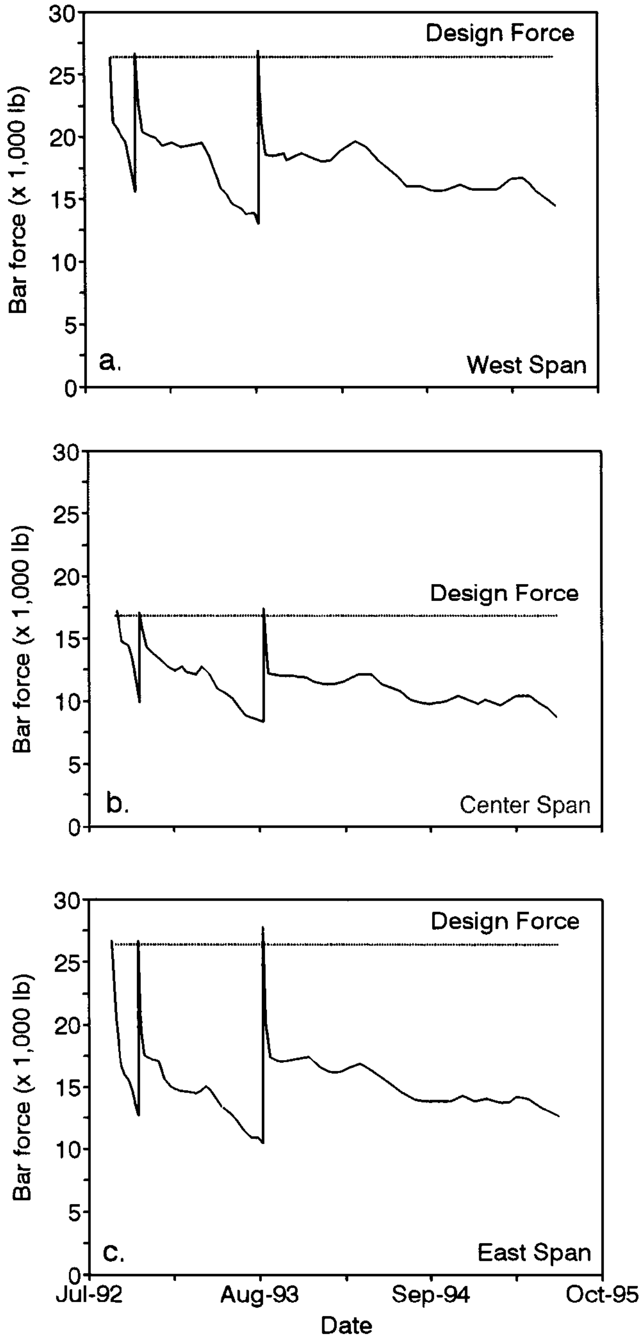

Figure 18—Average trend in bar force, starting at the third bar tensioning for (a) west, (b) center, and (c) east spans.

for the west span, 8,700 lb for the center span, and 12,500 lb for the east span, which is 45 to 55 percent of the design bar force. With each bar tensioning, the rate of bar force loss decreased, although the overall amount of bar force lost between stressings remained relatively equal.

The bar force loss was probably the result of three factors: stress relaxation, prefabrication methodology, and crushing of the timber laminations. The majority of bar force loss was the result of stress relaxation, a time-dependent phenomenon caused by the long-term compressive force of the steel bars acting on the wood microstructure. The rate of stress relaxation was greater on the Big Erick's bridge when compared with other stress-laminated bridges and was probably the result of the high moisture content in the timber laminations. An additional factor affecting bar force loss was the prefabrication methodology of completing two of the recommended bar tensionings on partial-width sections. This methodology diminished the positive effects of repetitive tensionings, because the bars were not full width until the third tensioning. Another plausible cause of bar force loss was crushing of the exterior laminations on the approach spans. Crushing was observed under the bearing plates during construction and the monitoring period. The rate of crushing and the corresponding bar force loss decreased with each subsequent bar tensioning. This is discussed in detail in the Condition Assessment section.

The observed bar force on the center and approach spans behaved similarly, as displayed in Figure 18. The center and approach spans have different bar force design values; therefore, percentage of bar force loss was used for comparison. All spans exhibited similar bar force losses of 40 to 50 percent before the fourth stressing, 50 to 60 percent prior to the fifth stressing, and 45 to 55 percent at the end of monitoring. These similarities between the approach spans and the center span suggest that stress-laminated decks and boxes, built from comparable materials, perform alike when subjected to the same environmental conditions.

Load Test Behavior

Results of the static-load tests and analytical assessment of the Big Erick's bridge are presented. For each load case, transverse deflection measurements are given at the midspan of each span as viewed from the east end (looking west). No permanent residual deformation was measured at the conclusion of the load testing, and there was no detectable movement at bridge supports. At the time of load tests 1A and 1B, the average bridge prestress was approximately 100 lb/in^2. For load test 2, the bridge prestress was approximately 56 lb/in^2 in the west span, 52 lb/in^2 in the center span, and 47 lb/in^2 in the east span.

Load Test 1A and 1B

Transverse deflections for load test 1A and 1B with the locations and magnitudes of the maximum measured deflections are shown in Figure 19. For each of the three load positions on the west and east approach spans, the deflections are typical of the orthotropic plate behavior observed for other stress-laminated decks (Ritter and others 1990). For the west span, the maximum measured deflection was 0.45 in. for load case 1 and 0.49 in. for load cases 2 and 3. For the center span, the maximum measured deflection was 0.31 in. for load case 1, 0.37 in. for load case 2, and 0.35 in. for load case 3. On the east span, the maximum measured deflection was 0.37 in. for load case 1 and 0.43 in. for load cases 2 and 3. As shown in Figure 19, the approach span maximum measured deflections occurred virtually in the same locations for each load case with minor variations, which are within

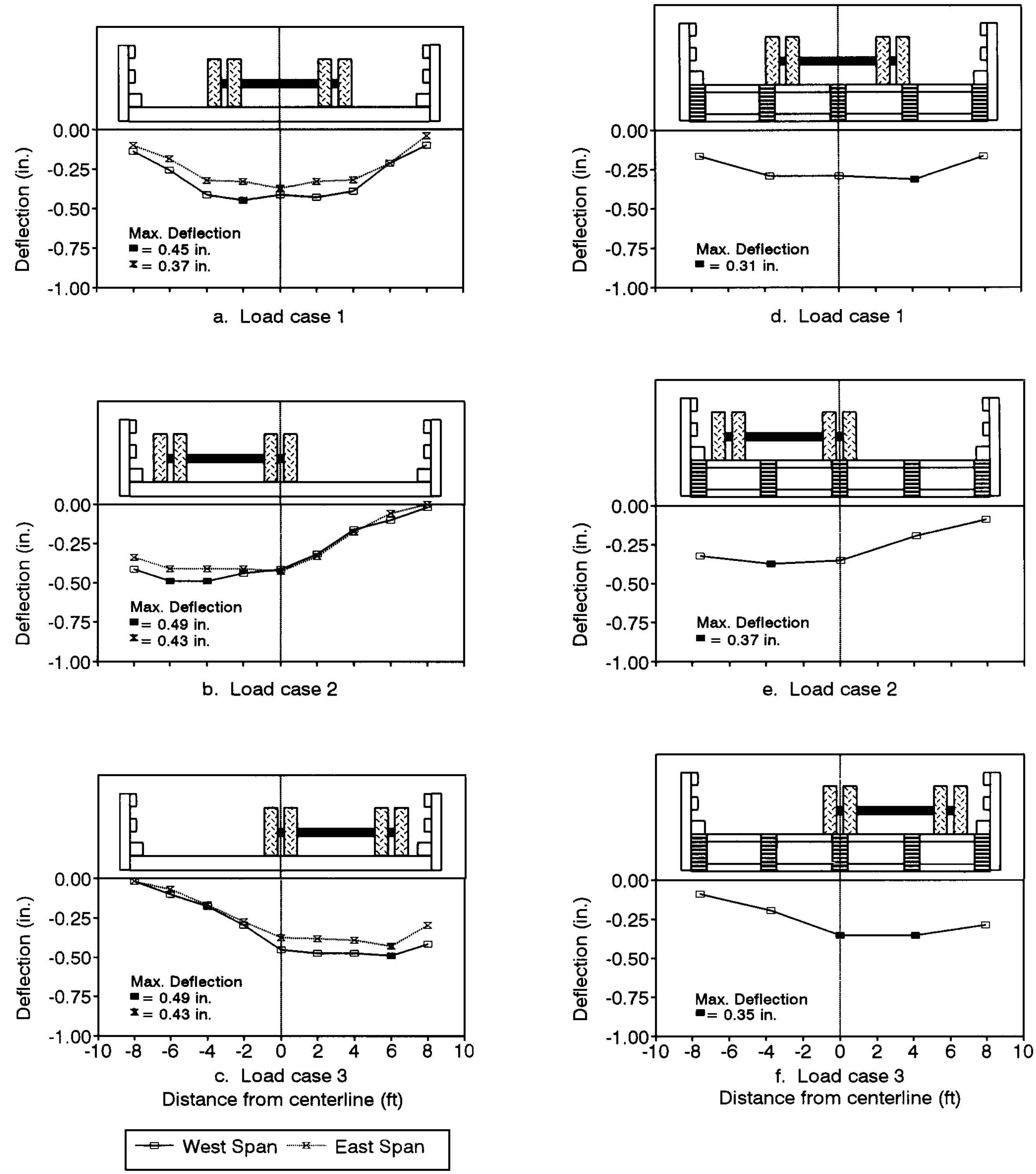

Figure 19—Transverse deflection measured at midspan (looking west) for load tests 1A and 1B. Bridge cross-sections and vehicle cases are presented to aid interpretation only and are not to scale.

the measurement error. However, deflections on the east span were 12 to 18 percent less than deflections on the west span. The difference in measured deflection was most likely the result of differences in the sawn lumber laminations between the two decks, because wood naturally has material property variations, even in the same grade. Deflections for the center span were less than the approach spans as a result of the differences in design and cross section.

Assuming uniform material properties, symmetric loading, and accurate deflection measurements, the bridge deflections for load cases 2 and 3 should be a mirror image. Load tests 1A and 1B of the measured deflections for load case 2 and a mirror image of load case 3 are displayed in Figure 20. As shown, there are slight deflection variations near the longitudinal centerline, but the deflections are essentially the same. The variations may be due to eccentricities in the loading or errors within the accuracy of the measurement methods.

Load Test 2

Load test 2 transverse deflections for the west, center, and east spans are shown in Figure 21. As with load test 1, the deflections are typical of the orthotropic plate behavior observed on stress-laminated decks. For the west span, the maximum measured deflection was 0.47 in. for load case 1 and 0.51 in. for load cases 2 and 3. The maximum measured deflection for the center span was 0.28 in. for load case 1, 0.35 in. for load case 2, and 0.31 in. for load case 3. On the east span, the maximum measured deflection was 0.39 in. for load case 1 and 0.49 in. for load cases 2 and 3. As with the first load test, the approach spans maximum measured deflections are in the same locations and east span deflections for load case 1 are again 12 to 18 percent less than load case 1 deflections on the west span. However, load cases 2 and 3 approach span deflections are virtually overlapping. The variation in measured deflections between load cases and load tests was probably a result of a combination of prestress and material property differences between the decks. The prestress effect describes the decrease of transverse stiffness when the amount interlaminar compression in the deck decreases (see Analytical Assessment). The west and east approach spans were load tested at prestress levels of 56 and 47 lb/in^2, respectively; therefore, the east span had a lower transverse stiffness during load testing. A combination of a stiffness decrease caused by prestress effect and material property differences described in the load test 1 section, yields similar deflection results for load test 2, load cases 2 and 3. The deflection differences observed in load case 1 were probably a result of a smaller prestress effect for the concentric loading.

Measured deflections for load case 2 and a mirror image of load case 3 are given in Figure 22. As with load test 1, deflection curves for each span are similar except for minor variations. These variations were probably the result of measurement errors during load testing and are within the accuracy of the measurement method.

Load Test Comparison

A comparison of the load case deflection for load tests 1 and 2 is shown in Figure 23. For each respective load case,

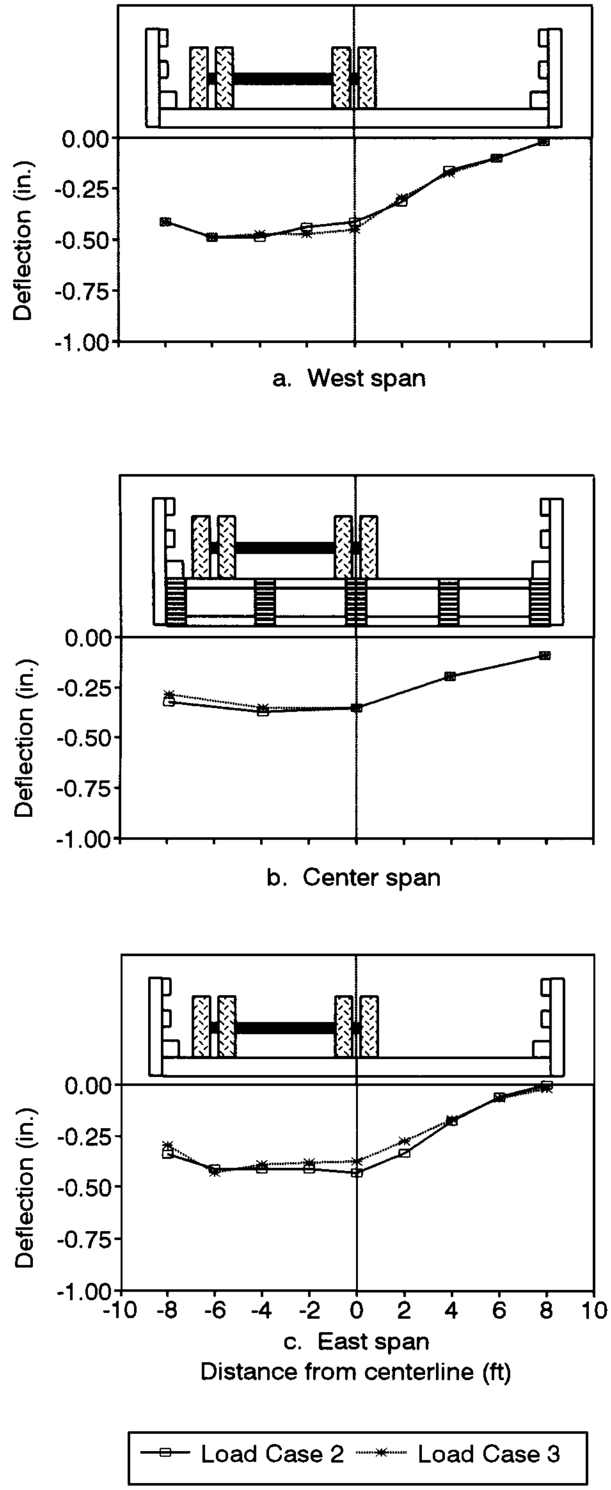

Figure 20—Comparison of approach span measured deflections for load tests 1A and 1B, showing the actual deflection of load case 2 and the mirror image of load case 3 (looking west).

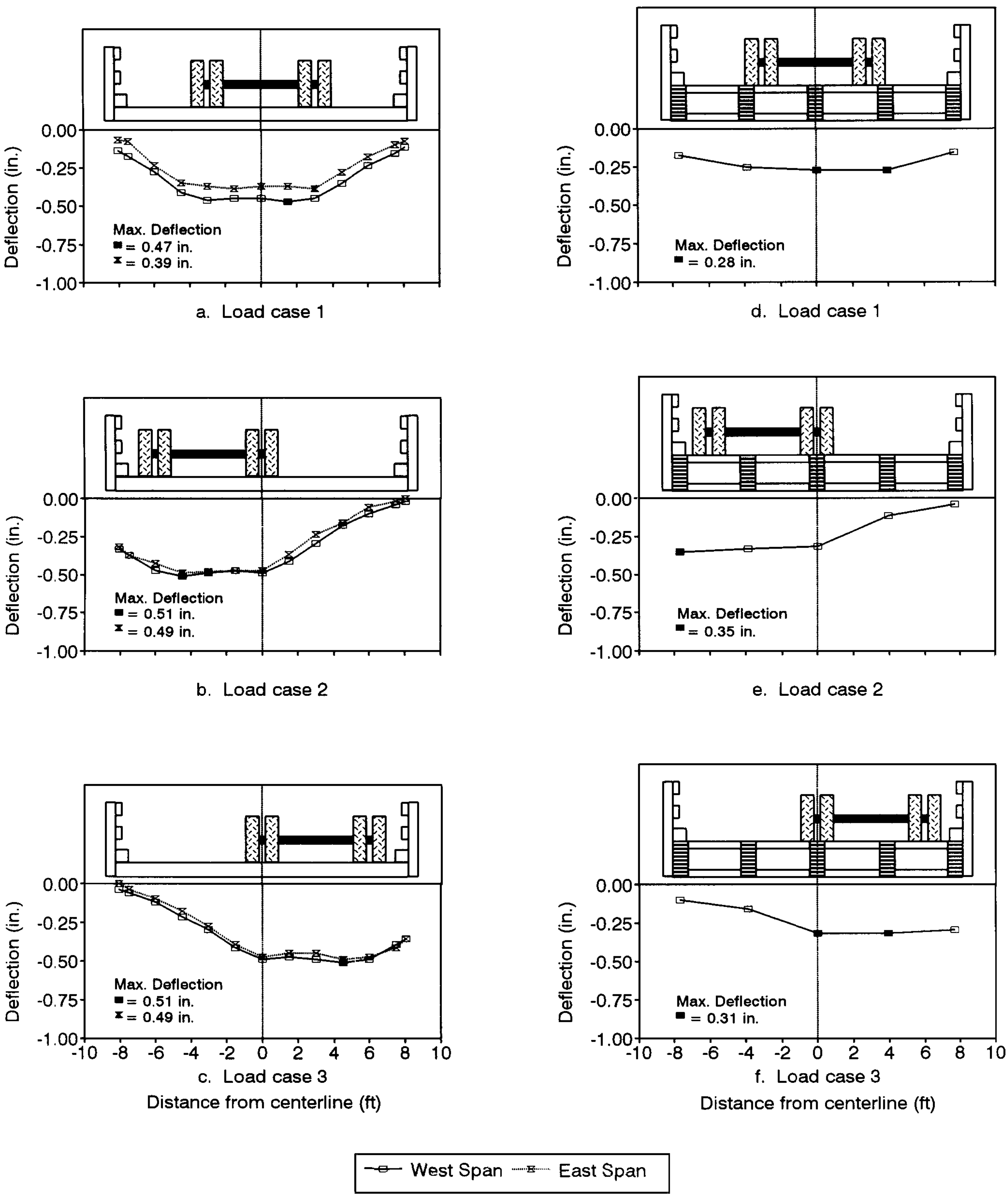

Figure 21—Transverse deflection measured at midspan (looking west) for load test 2. Bridge cross-sections and vehicle cases are presented to aid interpretation only and are not to scale.

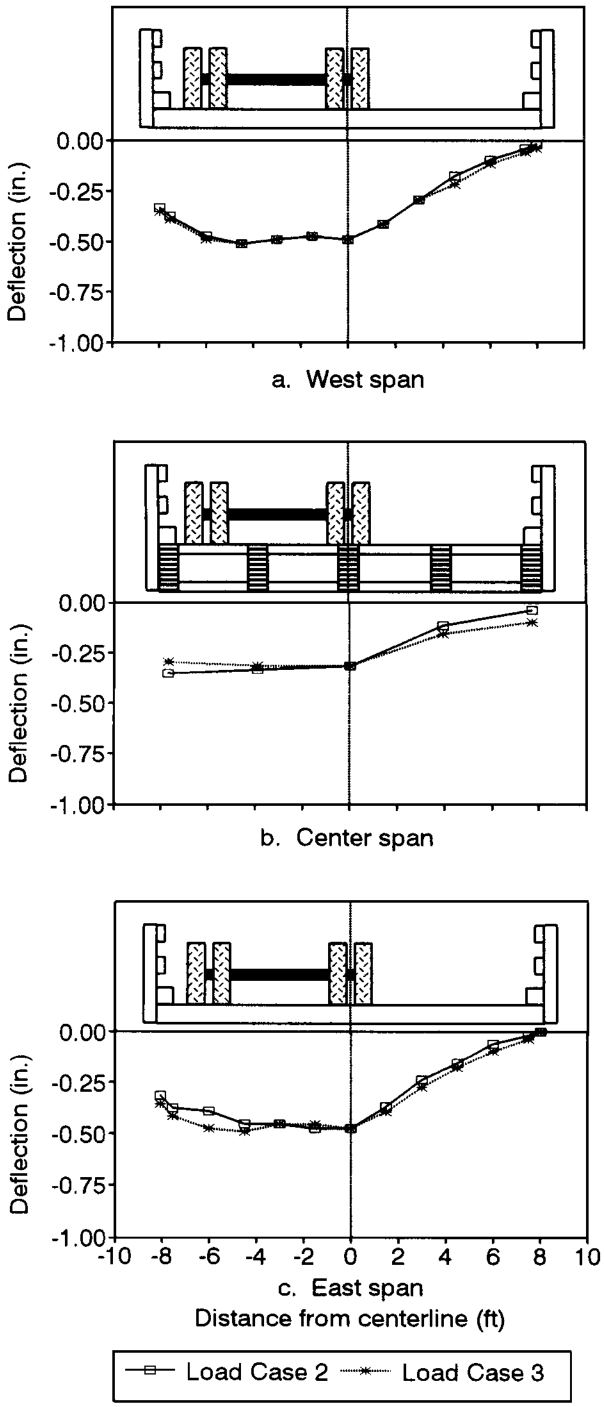

Figure 22—Comparison of approach span measured deflections for load test 2, showing the actual deflection of load case 2 and the mirror image of load case 3 (looking west).

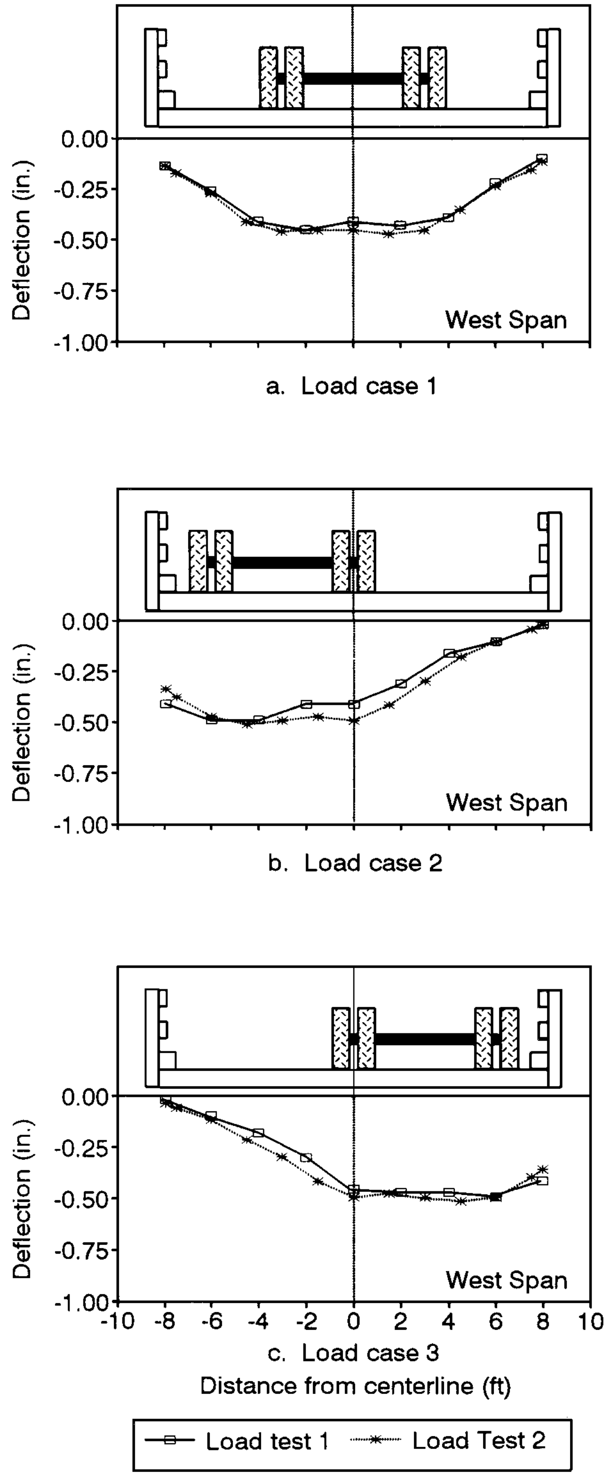

Figure 23—Comparison of the measured deflections for load tests 1 and 2 (looking west).

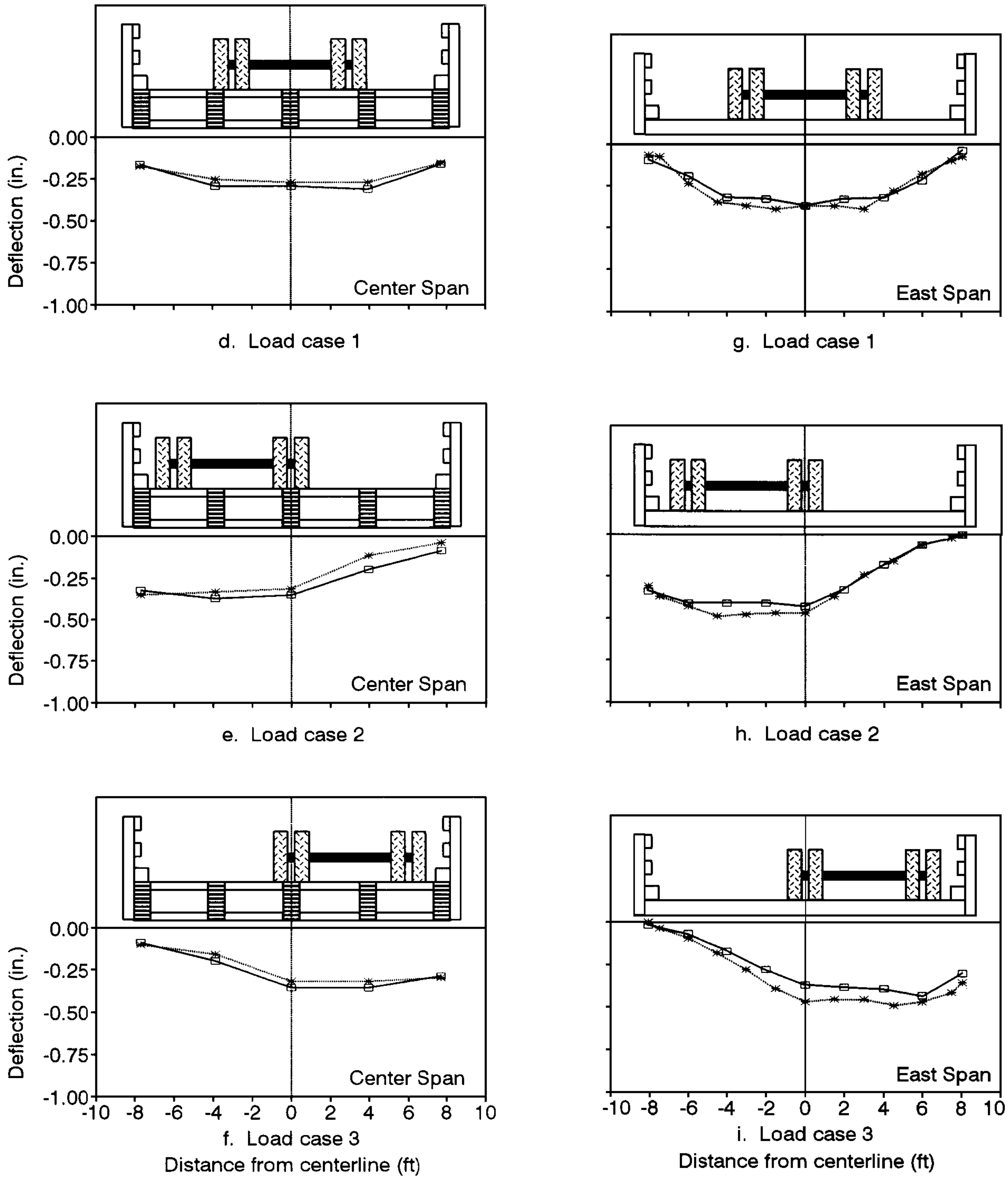

Figure 23—con.

deflections for the two load tests are similar. On the approach spans, the slight increases in deflection observed during load test 2 were attributable to decreases in the prestress (discussed in Analytical Assessment). For the center span, the measured deflections decreased on load test 2 compared with load test 1. The decrease in deflection was probably due to the 3,200-lb lighter load applied at load test 2. In addition, prestress factors probably played a smaller role for the center span because the glulam beam webs were full span.

As expected, the center span stress-laminated box displayed less deflection under similar loading than did the stress-laminated deck approach spans. The difference in measured deflection for the center span compared with the approach spans was 0.20 in. for all load cases. The smaller deflection indicates that the center span configuration could have been extended to a longer span than was built.

Analytical Assessment

Analytical assessment was completed only for the approach spans. Results of the measured deflections compared with the predicted bridge response based on orthotropic plate analysis are shown in Figure 24 for load test 1 and Figure 25 for load test 2. As shown, the predicted response was similar to the measured response with minor variations. The differences at the bridge edges were expected because the curb and railing do provide some stiffness, which was not accounted for in the analysis. Other variations were relatively minor and within the measurement reading error.

For further analysis, the same analytic parameters used for the load tests were used to determine the predicted response of an AASHTO HS 25–44 truck for load cases 1 and 3. (Load case 2 is a mirror image of load case 3.) Figure 26 displays the predicted response at the respective bar force levels for load tests 1A and 2. For load tests 1A and 2, the theoretical maximum deflection obtained for the HS 25–44 truck occurred in load case 3. The theoretical maximum deflections for load test 1A were 0.46 in., or 1/490 for the west span and 0.39 in., or 1/578 for the east span. For load test 2, the maximum theoretical deflection was 0.54 in., or 1/417 for the west span and 0.49 in., or 1/460 for the east span. Deflections computed using AASHTO recommended design procedures for the specified MOE were 0.66 in., or L/340 on the approach spans and 0.61 in., or L/610 on the center span.

Assuming constant bridge properties, the same theoretical bridge deflection would normally be expected for the same AASHTO HS 25–44 loading used for each load test. However, it has been determined that a decrease in interlaminar compression in stress-laminated bridges with butt joints results in a decrease in longitudinal bridge stiffness (Oliva and others 1990). The differences in the theoretical deflections for the two load tests were attributable to an approximate 15- to 20-percent decrease (15 percent west and 20 percent east) in longitudinal bridge stiffness for load test 2 as a result of the decreased level of interlaminar stress from 100 lb/in^2 for load test 1A to 56 lb/in^2 on the west span and 47 lb/in^2 on the east span for load test 2.

Condition Assessment

Condition assessment of the Big Erick's bridge indicated that structural performance was good, but serviceability deficiencies were noted. Inspection results for specific items follow.

Geometry

Width measurements taken on all spans indicate that the center span was rectangular, but each approach span midspan width was narrower than that at the abutments, commonly referred to as an hourglass shape (Ritter and others 1995). Approach span midspan widths were approximately 1.5 in. narrower than at the abutments. The shape was probably attributable to deck restraints at the abutments and piers that did not allow for transverse movement at these locations following the third stressing. Eleven months after installation, the connections were removed to allow free transverse movement. A second contributing factor was the use of a single hydraulic jack during the stressing process. An hourglass shape has appeared on other similar bridges that were stressed with a single hydraulic jack. The hourglass shape does not affect bridge strength, but it does make alignment of rails and curbs difficult and is not aesthetically pleasing.

Camber

Table 3 displays the camber measurements taken 11 months after installation and the end of monitoring period. During 24 months, camber loss of 0.60 in., 0.72 in., and 0.36 in. was measured for the west, center, and east spans, respectively. These losses were most likely a result of creep that was accelerated by the high moisture content of the timber laminations. Although camber does not affect bridge strength, perceptions of bridge safety are affected by noticeable sag that will occur with extensive creep. Initial measurements were not taken after the bridge was installed; therefore, the amount of camber loss that was due to short-term bridge dead load is unknown.

Wood Condition

Inspection of the wood components of the bridge showed no signs of deterioration. Minor checking was evident on the end grain of the rail posts that were exposed to rapid wet–dry cycles. This checking could have been alleviated through the use of a sealer on the post end grain. In several locations on the curb and railing system, vehicle damage was observed (Fig. 27). In some cases, the damage has broken the preservative envelope and could cause premature deterioration. The south-side exterior laminations on the stress-laminated approach spans were also surface checked and distorted as a result of rapid wet–dry cycling. Some railing system bolts and washers were crushed into the timber laminations, probably a result of bolt over tightening at installation. At the conclusion of monitoring, preservative staining and dripping were observed on the concrete abutments and piers and several bar anchorages (Fig. 12). In addition, a light grey coating on exposed bridge members could indicate an excess of preservative treatment that is migrating to the surface and combining with dust from the approach roadway.

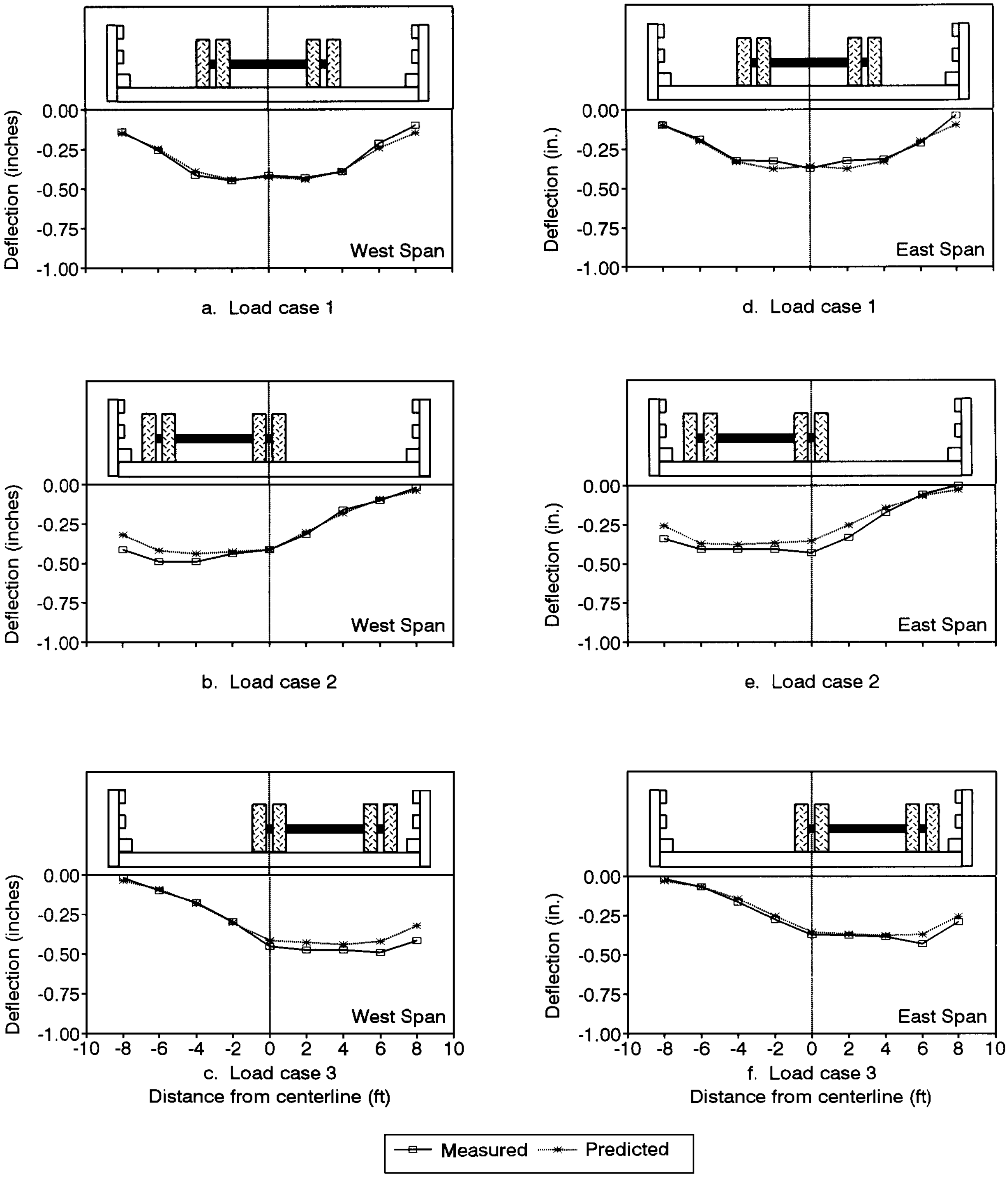

Figure 24—Comparison of the approach span measured deflections for load test 1 and the theoretical deflection based on orthotropic plate analysis (looking west).

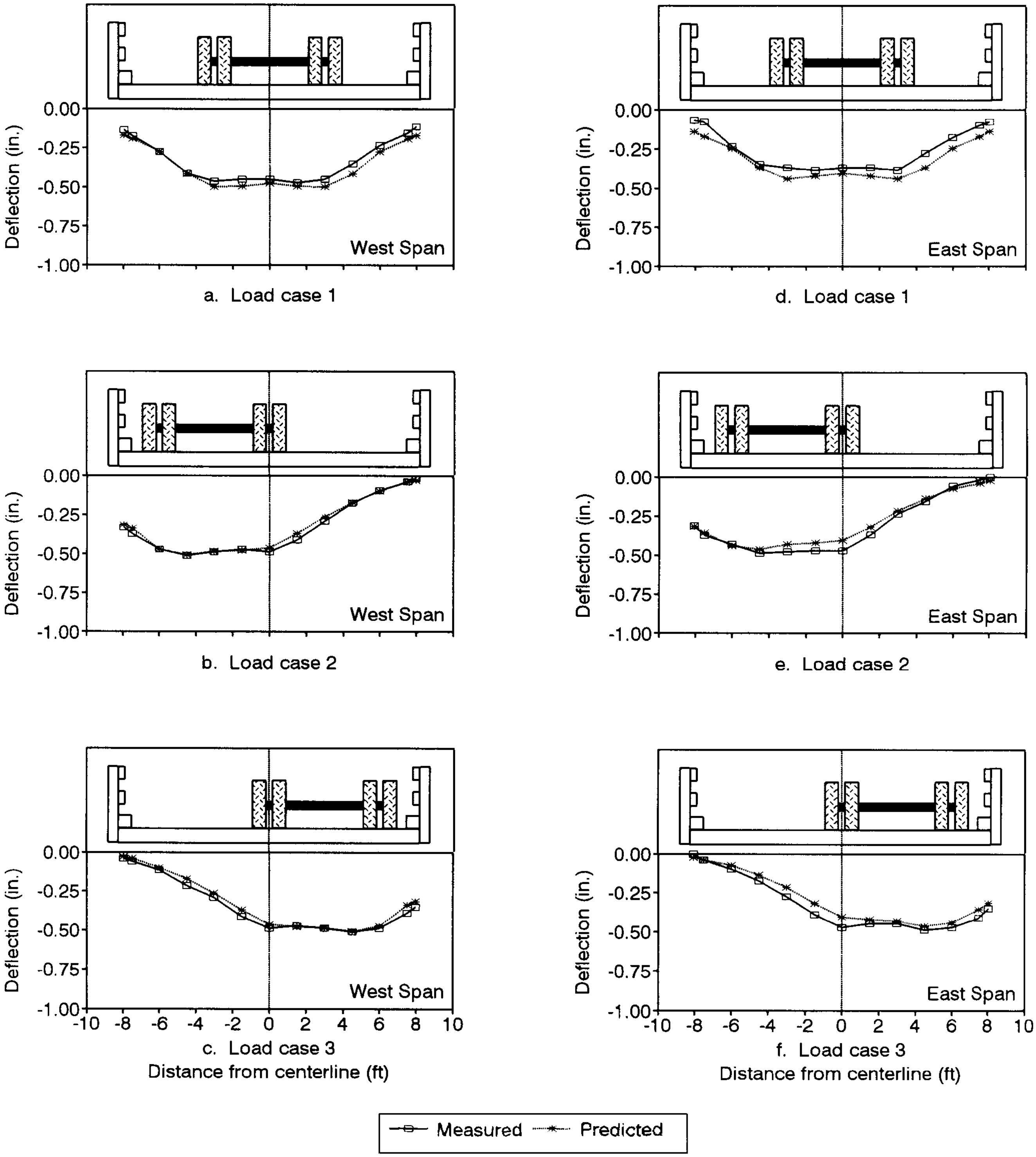

Figure 25—Comparison of the approach span measured deflections for load test 2 and the theoretical deflection based on orthotropic plate analysis (looking west).

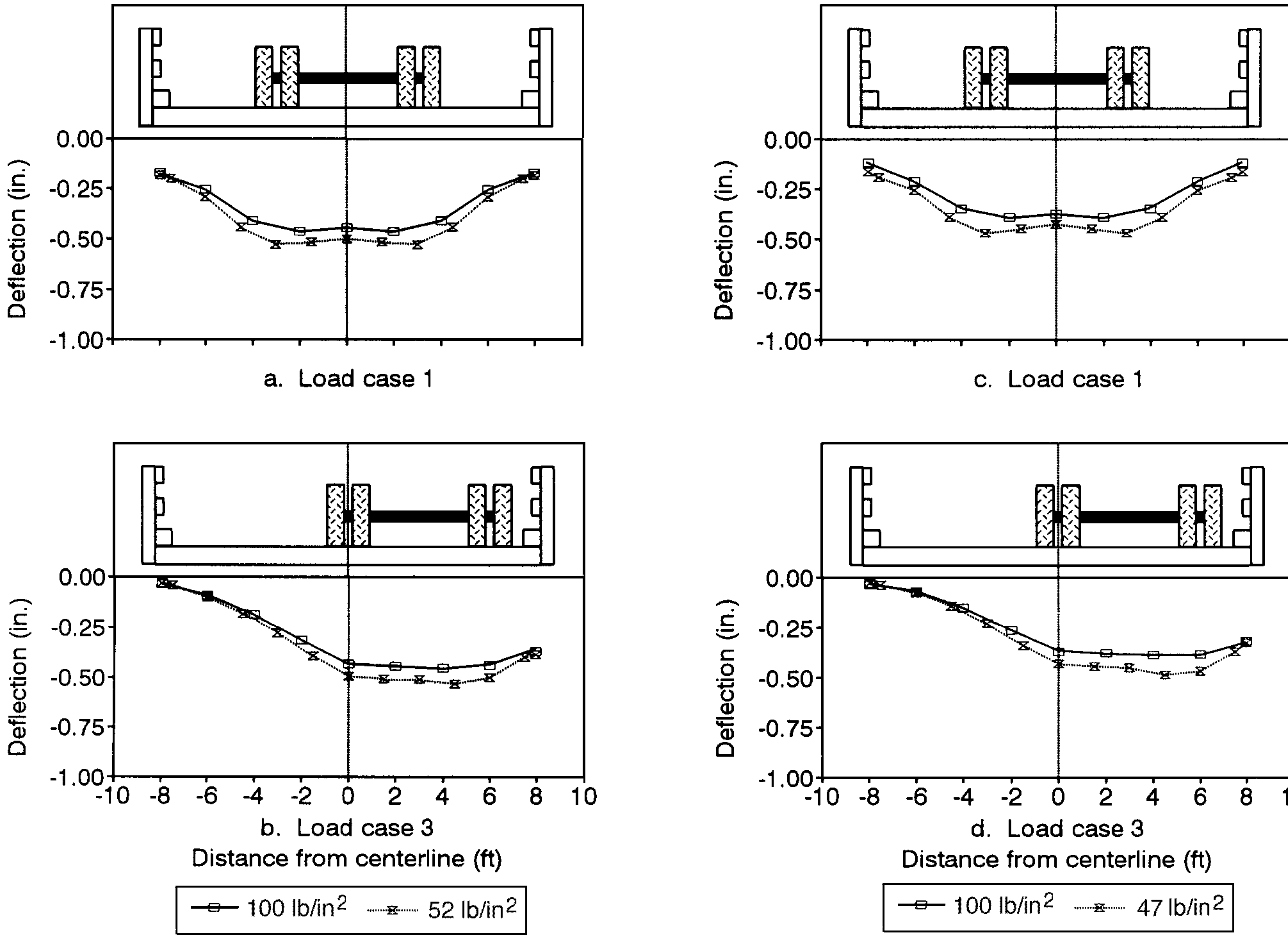

Figure 26—Maximum theoretical deflection at approach span midspan for AASHTO HS 25–44 truck loading at 100 lb/in^2 (both spans), 52 lb/in^2 (west span), and 47 lb/in^2 (east span).

Wearing Surface

The asphalt wearing surface was observed to be in poor condition at the conclusion of the monitoring period. Large transverse cracks were observed at the piers and at the west abutment. At the west pier, transverse cracking has heaved 1 to 2 in. above the roadway grade. Longitudinal rutting was also observed along two vehicular wheel lines in the asphalt on the bridge and the approach roadway, which varied from 0.5 to 1.5 in. in depth. The rutting is most severe at the center span (Fig. 28). Rutting was first observed 11 months after bridge installation and increased during the monitoring period. The heaving and rutting were probably caused by a poor asphalt mix and will cause rain water to remain on the deck for extended periods. This condition also tends to trap a substantial quantity of gravel and other debris on the surface from the unpaved road. The debris accumulations could lead to premature deterioration of the wearing surface.

Stressing System

The stressing bar anchorage system has performed adequately. No measurable distortion in the bearing plates was observed, and the exposed steel stressing bars, hardware, and anchorage plates showed no visible signs of corrosion or other deterioration. There was no apparent crushing of the

Table 3—Camber measurements

	Camber measurements (in.)			
Span	Design	11 months after installation November 1992	End of monitoring July 1995	Creep loss during monitoring
West	0.625	0.60	0.00	0.60
Center	1.00	0.90	0.18	0.72
East	0.625	0.18	–0.18	0.36

bearing plates into the Southern Pine glulam exterior laminations of the center span. However, crushing of the Eastern Hemlock laminations on the approach spans was first observed during bridge construction and continued progressively until the end of monitoring (Fig. 29), and seems to have contributed to short-term bar force loss after bar tensioning. This crushing occurred even though the bearing plates were properly designed, based on published values for compression perpendicular to grain adjusted for wet-use conditions. This indicates that the published design values for compression perpendicular to grain may not be representative of the material for use in this application. The use of a treatable hardwood species, such as oak or maple, on the outer laminations has been shown to perform well without crushing problems (Ritter and others 1995).

Figure 27—Vehicle damage to curb and rail system observed during a site visit near the end of the monitoring period.

Conclusions

After 35 months in service, the Big Erick's bridge is performing satisfactorily and should provide many years of acceptable service. Based on extensive bridge monitoring, we make the following observations and recommendations:

- The sawn lumber components of the Big Erick's bridge were initially installed at 33-percent moisture content. The average trend in moisture content indicates that changes are occurring slowly, with an average decrease of 5 percent during the monitoring period. The high moisture content contributed to bar force and camber loss during the monitoring period. It is expected that the moisture content will gradually decrease and lamination dimensional change should be expected. Future bridges constructed of this type should utilize laminations with moisture content at or below 19 percent.

- Following the final design bar tensioning, two additional bar tensionings were required due to rapid loss of bar force. The decrease in bar force was primarily attributable to transverse stress relaxation in the wood laminations with additional effects from crushing of the approach span exterior laminations near the bar anchorages. It is anticipated that the bridge will require retensioning in the near future. In addition, as the moisture content decreases, the bar force will also decrease until an equilibrium moisture content is reached. Therefore, bar force should be checked annually and retensioned as necessary until it reaches a constant level.

- Load testing and analysis indicate that all three spans of the Big Erick's bridge are performing in a linear elastic manner, and the approach spans exhibit orthotropic plate behavior when subjected to truck loading. HS25–44 truck loading conditions produced maximum deflections of 0.46 and 0.39 in. for the west and east approach spans at 100 lb/in^2 and 0.56 in., and 0.49 for the west and east approach spans at 57 and 47 lb/in^2, respectively. These deflections correspond to values of L/490, L/578, L/417, and L/460, based on the center–center of bearing span length.

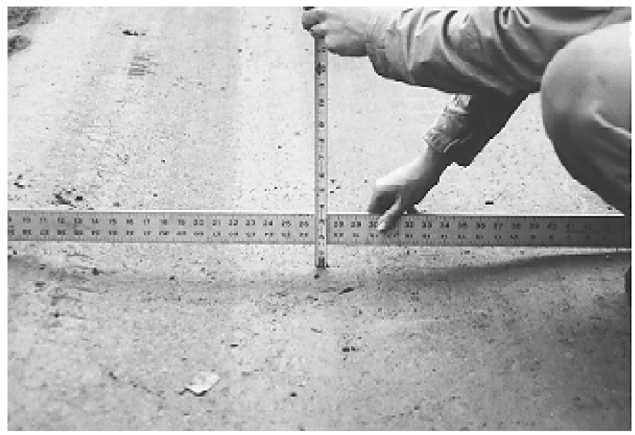

Figure 28—Rutting of asphalt wearing surface measured at the center span during a site visit for load test 2.

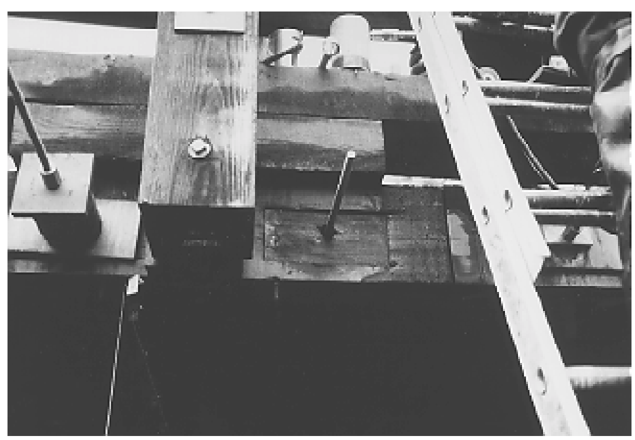

Figure 29—Visible crushing of approach span, Eastern Hemlock lumber lamination, observed after the removal of the bearing plate at the final site visit.

- Width measurements taken on all spans indicate that although the center span is rectangular, each approach span midspan width is narrower than the width at the abutments, which is referred to as an hourglass shape. The hourglass shape is probably the result of the attachment of the span ends after the third bar tensioning, which prevented transverse movement at the abutments and piers during subsequent bar tensionings. For future bridges, attachments to the abutments and/or piers should be made only at the underside of the longitudinal centerline to allow for transverse movement or after the bridge reaches bar force equilibrium.
- Camber measurements of the stress-laminated decks and box indicate that the camber decreased during 24 months of monitoring. At the conclusion of monitoring, the west span is approximately straight between supports, the center span has a vertical camber of approximately 0.2 in., and the east span has a slight sag of approximately 0.2 in.
- Wood checking is evident in the exposed end grain of the bridge rail posts. It is likely this would not have occurred if a sealer had been placed over end grain at the time of construction. In addition, preservative treatment has dripped and stained the concrete abutment and piers.
- The asphalt wearing surface has longitudinal ruts and heaving at the piers, which contribute to increased standing water and debris on the surface of the bridge. These conditions could lead to premature deterioration of the wearing service.
- The use of high moisture content Eastern Hemlock sawn lumber on the outside laminations of the approach decks adversely affected the performance of the bar anchorages. The wood crushing observed was approximately 0.75 in. underneath the bearing plates. The inclusion of hardwood laminations for the two exterior laminations would have greatly enhanced the bar anchorage performance.
- Prefabrication of stress-laminated timber bridges saves on-site construction time by allowing installation of the bridge in sections by an overhead crane. However, bar tensioning prior to installation on the prefabricated half-width panels adversely affects the bar force performance. When using prefabrication, additional bar tensionings should be planned when full-width bars are installed.

References

AASHTO. 1989. Standard specifications for highway bridges 14th ed. Washington, DC: American Association of State Highway and Transportation Officials.

AASHTO. 1990. Standard specifications for transportation materials and methods of sampling and testing. Vol. 1: Specifications. Washington, DC: American Association of State Highway and Transportation Officials. 877 p.

AITC. 1987. Standard specifications for structural glued laminated timber of softwood species. AITC 117–87–Design. Vancouver, WA: American Institute of Timber Construction. 28 p.

AFPA. 1988. Design values for wood construction; a supplement to the 1986 edition of the national design specification. Washington, DC: National Forest Products Association. 40 p.

ASTM. 1988. Standard specification for uncoated, high strength steel bar for prestressing concrete; ASTM A722–86. Philadelphia, PA: American Society for Testing and Materials. 3 p.

ASTM. 1990. Use and calibration of hand-held moisture meters; ASTM D 4444–84. Philadelphia, PA: American Society for Testing and Materials. 6 p.

AWPA. 1990. Standards. Woodstock, MD. American Wood Preservers' Association. 200 p.

FPL. 1987. Wood handbook: Wood as an engineering material. Agric. Handb. 72 (rev). Washington, DC: U.S. Department of Agriculture; 466 p.

NELMA. 1986. Grading rules for northeastern lumber. Cumberland Center, ME: Northeastern Lumber Manufacturing Association.

Oliva, M. G.; Dimakis, A.G.; Ritter, M.A.; Tuomi, R.L. 1990. Stress-laminated wood bridge decks: experimental and analytical evaluations. Res. Pap. FPL–RP–495. Madison, WI: U.S. Department of Agriculture, Forest Service, Forest Products Laboratory. 24 p.

Ritter, Michael A. 1990. Timber bridges: design, construction, inspection and maintenance. EM 7700–8. Washington, DC: U.S. Department of Agriculture, Forest Service, Engineering Staff. 944 p.

Ritter, M.A.; Geske, E.A.; Mason, L. [and others]. 1990. Performance of stress-laminated bridges. Wood Design Focus. 1(3): 12–16.

Ritter, M.A.; Geske, E.A.; McCutcheon, W.J. [and others]. 1991. Methods for assessing the field performance of stress-laminated timber bridges. In: Marcroft, Julian, comp. Proceedings, 1991 international timber engineering conference; 1991 September 2–5; London, United Kingdom. High Wycombe, United Kingdom: Timber Research and Development Association: 319–326. Vol. 3.

Ritter, M.A.; Wacker, J.P.; Duwadi, S.R. 1995. Field performance of stress-laminated timber bridges on low-volume roads. In: Proceedings, 6th international conference on low-volume roads; 1995 June 25–29; Minneapolis, MN. Washington, DC: National Academy Press: 2: 347–357.

USDA. 1995. The timber bridge initiative, fiscal year 1995 status report. Radnor, PA: U.S. Department of Agriculture, Forest Service, State and Private Forestry, Northeastern Area.

Wacker, J.P.; Ritter, M.A. 1992. Field performance of timber bridges: 1. Teal River stress-laminated deck bridge. Res. Pap. FPL–RP–515. Madison, WI: U.S. Department of Agriculture, Forest Service, Forest Products Laboratory. 19 p.

WV–DOH. 1989. West Virginia department of highways standard bridge plans: stressed timber bridge type B. Morgantown, WV: West Virginia Department of Highways.

Appendix—Information Sheet

General

Name: Big Erick's bridge

Location: Baraga County, Michigan

Date of Construction: September 1992

Owner: Michigan Department of Natural Resources

Design Configuration

Number of Spans: 3 (simple)

Structure Type:

Approach Spans: Stress-laminated decks with butt joints

Center Span: Stress-laminated box with glulam webs and sawn lumber butt-jointed flanges

Butt-Joint Configuration:

Approach Spans: 1 in 8 transverse laminations with joints in adjacent deck laminations separated 4 ft longitudinally

Center Span: 1 in 4 transverse laminations with joints in adjacent deck flange laminations separated 4 ft longitudinally

Total Length (out–out): 72 ft

Skew: None

Span Lengths (out–out): 20 ft (approach spans); 32 ft (center span)

Span Lengths (center–center bearings): 18.8 ft (approach spans); 31 ft (center span)

Bearing Lengths: 0.75 by 6 in. on full width neoprene pads; each span overhangs 3 in. at each end

Width (out–out): 16 ft

Width (curb–curb): 14 ft

Number of Traffic Lanes: 1

Design Loading: AASHTO HS25–44

Wearing Surface Type: Asphalt pavement; 2- to 2.5-in. thickness

Material and Configuration

Deck/Flange Laminations:

Species: Eastern Hemlock

Size (actual): 2 by 11 in. (approach spans); 2 by 7 in. (box flanges)

Grade: No. 1 (not visually graded)

Moisture Condition: Approximately 34 percent at installation

Webs:

Species: Southern Pine

Size (actual): 80.75 by 22.5 in.

Beam Designation: 24F–V3

Moisture Condition: Approximately 13 percent average at installation

Rails:

Species: Southern Pine

Size (actual): 5.125 by 6 in., spliced at piers

Beam Designation: 24F–V3

Posts:

Species: Southern Pine

Size (actual): 8.75 by 10.5 in.

Beam Designation: 24F–V3

Curb and Scupper:

Species: Eastern Hemlock

Size (actual): 6 by 12 in., spliced at piers

Grade: No. 1 (not visually graded)

Preservative Treatment: Creosote

Stressing Bars:

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

Diameter: 0.625 in.

Number:

Approach Spans: 18

Center Span: 30

Design Force: 26,400 lb, sawn lumber end spans 16,800 lb, center box span

Spacing: 24 in. center–center, beginning 2 ft from span ends

Anchorage Type and Configuration:

Anchor plate and nut: 2 by 5 by 1 in.

Approach Spans: 6- by 12- by 0.875-in. bearing plates

Center Span: 6- by 10- by 0.875-in. bearing plates