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Field Performance of Timber Bridges

13. Mohawk Canal Stress-Laminated Bridge

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Abstract

The Mohawk Canal bridge was constructed in August 1994, just outside Roll, Arizona. It is a simple-span, double-lane, stress-laminated deck superstructure, approximately 6.4 m (21 ft) long and 10.4 m (34 ft) wide and constructed with Combination 16F-V3 Douglas Fir glued-laminated timber beam laminations. The performance of the bridge was monitored continuously for 2 years, beginning shortly after installation. Performance monitoring involved gathering and evaluating data relative to the moisture content of the wood deck, the force level of the steel stressing bars, the vertical creep of the deck, and the behavior of the bridge under static load conditions. Furthermore, comprehensive visual inspections were conducted to assess the overall condition of the structure. Based on field evaluations, the bridge is performing properly with no structural deficiencies.

Keywords: Stress laminated, timber, bridge, wood, performance, glued laminated

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Contents

Page

Field Performance of Timber Bridges

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Introduction

In 1988, the U.S. Congress passed legislation known as the Timber Bridge Initiative (TBI). The objective of this legislation was to establish a national timber bridge program to encourage the effective and efficient use of wood as a structural material for highway bridges. Responsibility for developing, implementing, and administrating the TBI was delegated to the USDA Forest Service. The program included three emphasis areas: 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, manages 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 new or previously underutilized wood products, bridge designs, or design applications (S&PF 1995).

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 employ 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. This provides a basis for validating or revising design criteria and further improving efficiency and economy in bridge design, fabrication, and construction.

In addition to the TBI, the U.S. 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. Because many aspects of the FHWA research program paralleled those underway at FPL, a joint effort was initiated to combine the respective research of the two agencies into a central research program. As a result, the FPL and FHWA merged resources to jointly develop and administer a national timber bridge research program.

This report describes the development, design, construction, and field performance of the Mohawk Canal bridge in Yuma County, Arizona. The bridge constructed in August 1994, is a double-lane, single-span, stress-laminated deck approximately 6.4 m (21 ft) long and 10.4 m (34 ft) wide. The laminations are Combination 16F-V3 Douglas Fir, glued-laminated timber (glulam) beams. Characteristics of the bridge are summarized in the Appendix.

Background

The bridge site is located approximately 56 km (35 miles) east of Yuma, Arizona, in the Wellton–Mohawk Valley District, near Roll, Arizona (Fig. 1). It is on County 5th Street, a double-lane, dirt roadway that provides access to local residences and agricultural areas and crosses over the Mohawk Canal. The average traffic is estimated to be 200 vehicles per day and consists of passenger vehicles, school busses, and farm vehicles and machinery.

The existing structure, constructed in 1955, was a timber stringer bridge with a timber plank deck, supported by concrete abutments (Fig. 2). The original bridge was approximately 4.6 m (15 ft) long and 6.7 m (22 ft) wide. Inspection of the bridge in 1988 indicated that the deck was in poor condition. Several planks were broken or deteriorated, and the bridge was restricted to a load limit of 89 kN (10 tons). In addition, the railing system on the south side was broken. It was apparent that major rehabilitation or replacement of the structure would be required. After evaluating the structural deficiencies, a decision was made by Yuma County officials to replace the bridge.

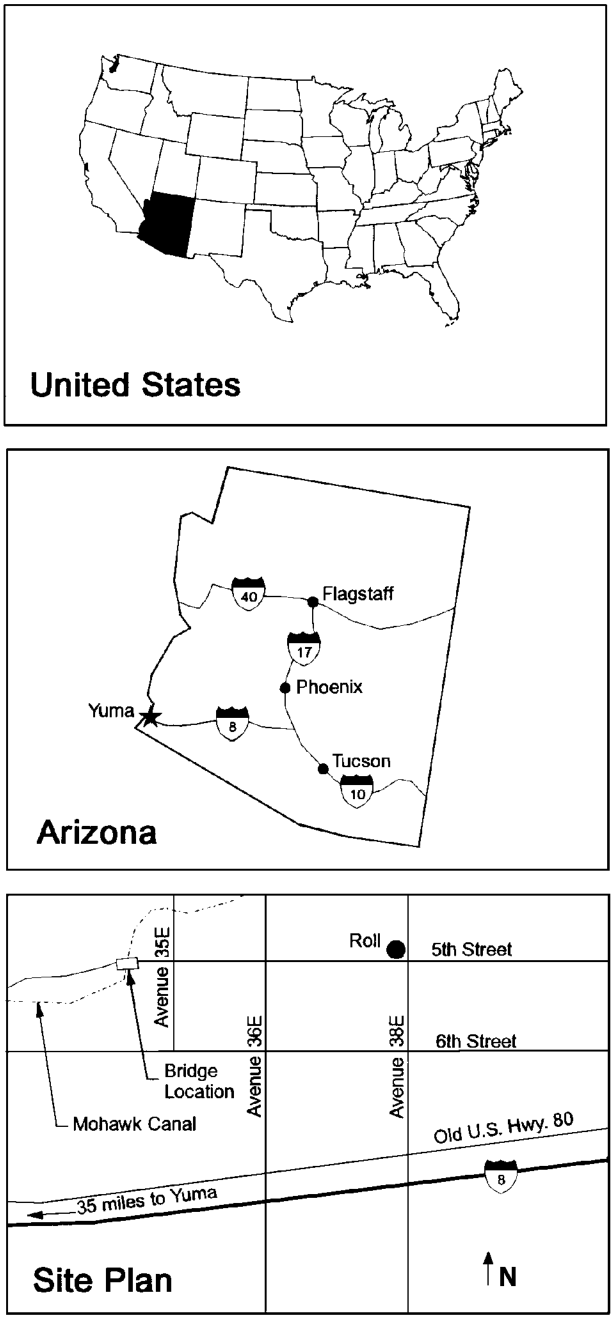

Figure 1—Location of the Mohawk Canal bridge.

Motivated by the TBI, Yuma County staff submitted a proposal for a stress-laminated demonstration bridge using laminations composed of Combination 16F-V3 Douglas Fir glulam beams. The project was accepted and partially funded as a NWITIC demonstration bridge in 1991. Because the stress-laminated deck was a newly developed system, it was determined that the field performance of the bridge should be monitored after installation to provide assurance of acceptable behavior. Subsequently, an agreement was developed to

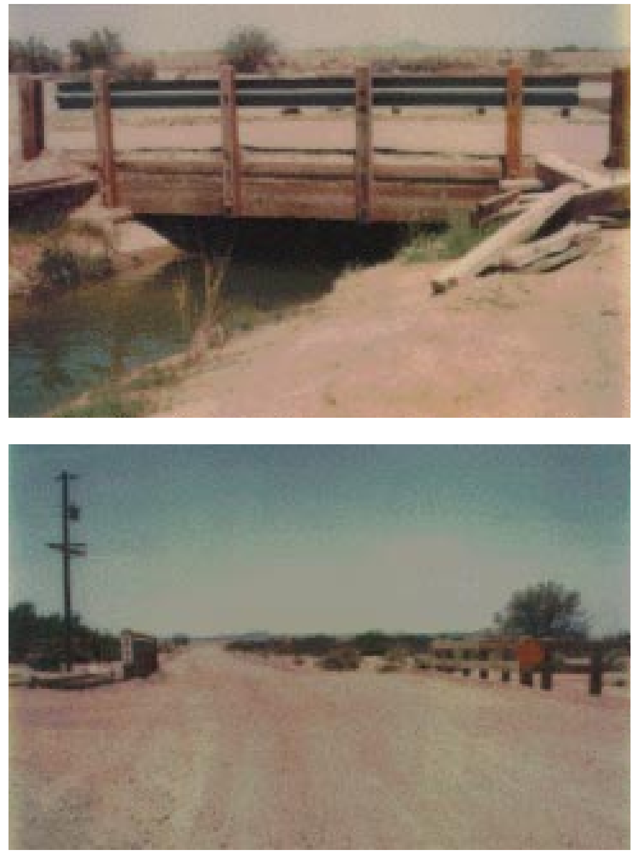

Figure 2—Original Mohawk Canal bridge: side view (top), end view (bottom).

include the Mohawk Canal bridge in the FPL/FHWA bridge monitoring program.

Objective and Scope

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

Design and Construction

Design and fabrication of the Mohawk Canal bridge were completed by contract. Construction was directed by the Yuma County Department of Development Services staff and completed by a county construction crew. An overview of the design and construction process follows.

Design

The Mohawk Canal bridge was designed and fabricated by contract. Design criteria for the bridge aspects relating directly to stress laminating were based on *Guide Specifications for the Design of Stress-laminated Wood Decks*, published by the American Association of State Highway and Transportation Officials (AASHTO) (AASHTO 1991). All other aspects of the superstructure design were based on the *Standard Specifications for Highway Bridges*, also published by AASHTO (1989).

The bridge was designed for AASHTO HS20–44 loading (AASHTO 1989), a span length of 6.1 m (20 ft) center-to-center of bearings, a width of 10.3 m (33.9 ft), and a skew of 15 degrees (Fig. 3). Combination 16F-V3 Douglas Fir glulam beams were selected as the deck laminations. The design was based on 130- by 305-mm- (5.13- by 12-in.-) wide laminations, pressure treated with pentachlorophenol and heavy oil. Full-length laminations with 10 mm (0.38 in.) of camber were specified. The stressing system was designed for 25-mm- (1-in.-) diameter high strength, threaded steel bars, conforming to the requirements of ASTM A722 (ASTM 1988).

Because of the skew, only three bars were continuous across the entire deck. The remaining bars (two on each side) extended only partially across the deck, with one end embedded in the deck. The average bar spacing was 1.4 m (54 in.) on–center, beginning approximately 457 mm (18 in.) from the ends of the bridge. The design bar tension force was 289 kN (65,000 lb), resulting in approximately 706 kPa (102 lb/in^2) of interlaminar compression; a discrete plate anchorage system was used (Fig. 4). The railing of the bridge consisted of a glulam timber rail without curb. In 1995, this rail configuration was approved by FHWA as a crash-tested railing for AASHTO performance level 1 criteria (Ritter and others 1995a). An asphalt wearing surface, consisting of compacted asphalt cold mix, was specified.

Construction

Construction of the Mohawk Canal bridge began with removal of the existing superstructure and construction of new concrete abutments (Fig. 5). As illustrated in Figure 5, the Douglas Fir glulam beams were delivered and stored on site several days before the superstructure assembly began. In late August 1994, after work on the approach roadway and abutments was completed, construction of the superstructure commenced. The laminations were lifted and positioned above the abutments by a crane (Fig. 6a). As the laminations were lowered onto the abutments, the county construction crew guided their placement (Fig. 6b). Because of the bridge skew angle, lamination order and placement were critical for proper hole alignment. An additional lamination placement

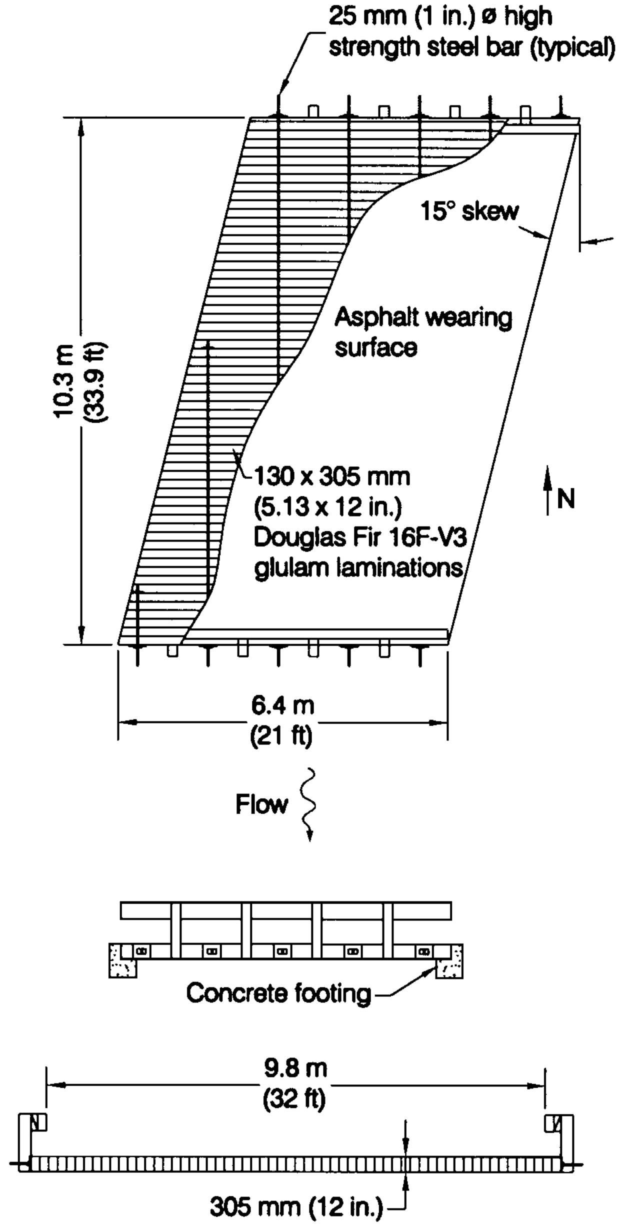

Figure 3—Design configuration of the Mohawk Canal bridge.

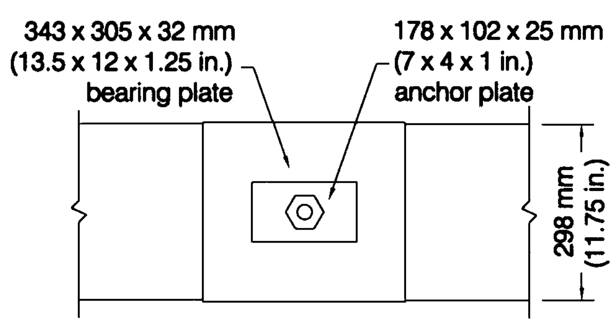

Figure 4—Detail of the discrete plate bar anchorage configuration.

Figure 5—Construction of east concrete abutment and spread footing. The Douglas Fir glulam beams were stored on site for several days.

Figure 7—After deck assembly, steel post supports were attached to the deck and railposts were inserted.

Figure 6—The Mohawk Canal bridge was assembled by a county crew: (a) Laminations were lifted by crane from the on-site storage location and positioned above the abutments. (b) Workers guided placement of laminations as they were lowered onto the abutments.

requirement was the deck-to-substructure anchorage, which required an anchor to pass through a lamination every 781 mm (30.75 in.) (Fig. 6b). Because perfect alignment of deck-to-substructure connection could not be obtained, additional holes were field drilled to align the connection. These holes penetrated the preservative envelope; therefore, they were treated with a wood preservative in the field.

As lamination placement progressed, embedded stressing bars were inserted through the deck and the interior anchorages were installed. After all deck laminations were in place, the three stressing bars that traverse the entire deck were inserted, steel bearing and anchorage plates were placed on the bars, and the anchor nuts were hand tightened. The steel post supports were attached to the sides of the deck and the railposts were installed (Fig. 7). It took several days to complete the installation of the superstructure.

Because stressing equipment was not available, the bars were tensioned approximately 1 month following assembly of the superstructure. During this time, traffic was not permitted on the bridge. A single hydraulic jack was used to tension the bars to the required 289-kN (65,000-lb) design force. Approximately 2 weeks following the first tensioning, the bars were retensioned to the design force. The bars were tensioned a third time, 10 weeks after the initial tensioning, to approximately 321 kN (72,200 lb). The railing was installed, and the asphalt wearing surface was applied shortly before the second bar tensioning. The as-built configuration of the Mohawk Canal bridge varied slightly from the design configuration in Figure 3. The measured deck thickness was 298 mm (11.75 in.,) which varied slightly from the design value of 305 mm (12 in.). The completed bridge is shown in Figure 8.

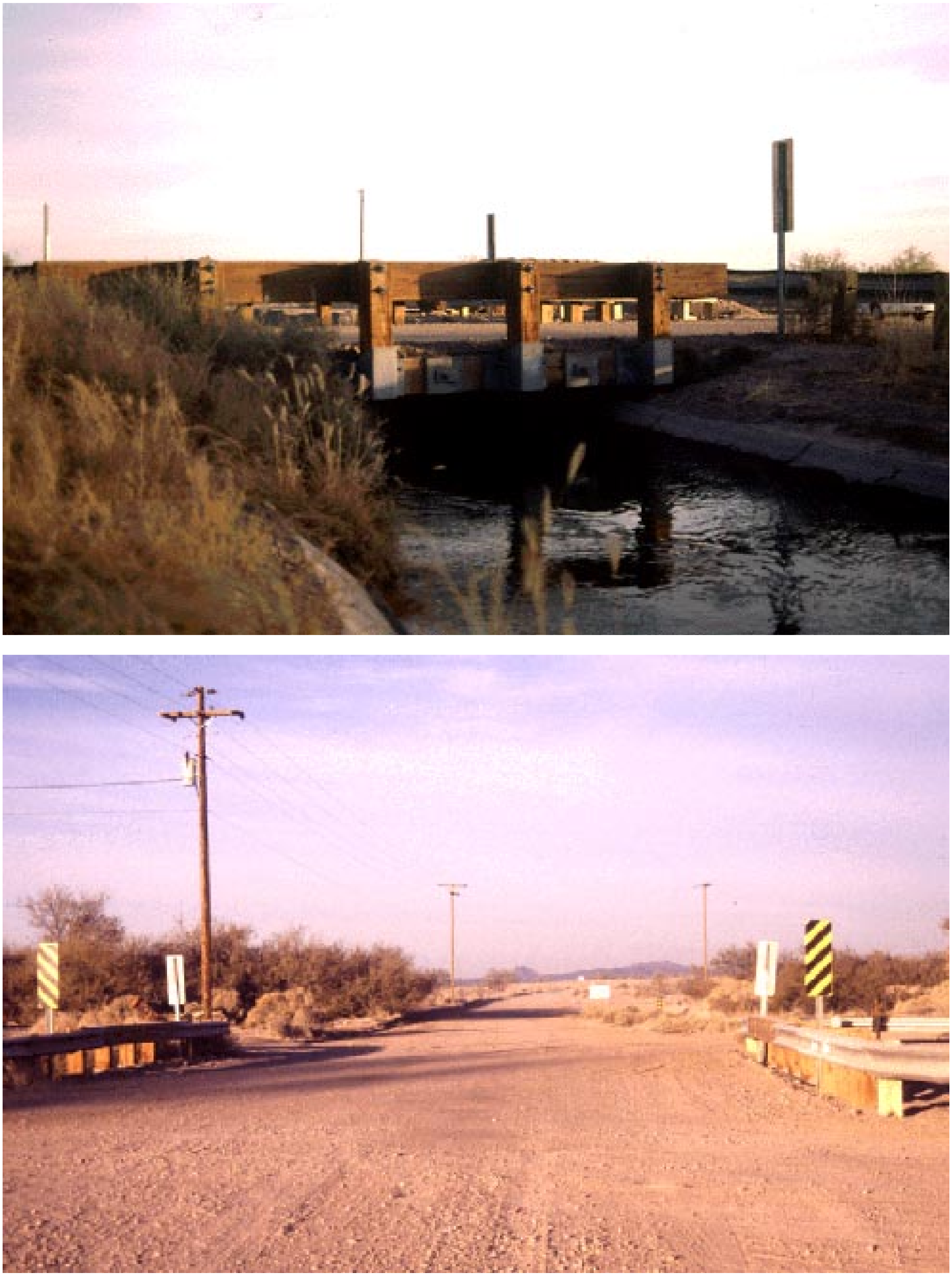

Figure 8—Completed Mohawk Canal bridge: side view (top), end view (bottom).

Evaluation Methodology

To evaluate the structural performance of the Mohawk Canal bridge, Yuma County representatives contacted FPL for assistance and requested to be included in the FPL/FHWA bridge monitoring program. Through mutual agreement, a monitoring plan was developed and implemented as a cooperative effort with Yuma County. The plan called for the performance monitoring of the moisture content of the deck, the stressing bar force, vertical creep, load test behavior, and condition assessment of the structure. The evaluation methodology utilized procedures and equipment previously developed by FPL (Ritter and others 1991).

Moisture Content

The moisture content of the bridge was measured in accordance with ASTM D4444-84 procedures (ASTM 1990) using an electrical-resistance moisture meter. Measurements were obtained by driving the insulated probe pins into the underside of the deck at a 25- to 51-mm (1- to 2-in.) depth, recording the moisture content from the unit, and adjusting the values for temperature and wood species (FORINTEK 1984). Moisture content measurements were taken at the beginning, midpoint, and conclusion of the monitoring period.

Bar Force

Bar force was measured using load cells developed by FPL. The load cells were installed between the bearing and anchor plate on the second, third, and fourth stressing bars from the east abutment along the upstream edge of the bridge. Load cell measurements were obtained using a portable strain indicator by Yuma County Department of Development Services personnel on a weekly basis for 3 months following load cell installation and approximately bi-weekly thereafter. Strain measurements were converted to units of bar tensile force by applying a laboratory conversion factor.

Vertical Creep

Vertical creep was measured at the beginning and end of the monitoring period. Measurements were obtained by attaching a stringline to the bearings to create a horizontal benchmark and at midspan, measuring the elevation of the deck with respect to the benchmark using a calibrated rule.

Load Test Behavior

Static load testing was conducted at the beginning and end of the monitoring period. Each test consisted of positioning fully loaded trucks on the bridge and measuring the resulting deflections at a series of transverse locations at midspan. Deflection measurements were taken prior to testing (unloaded), for the first three load positions (loaded), halfway through testing (unloaded), for the last three load positions (loaded), and at the conclusion of testing (unloaded). Measurements 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. 9). Accuracy of the measurements is estimated to be ±0.1 mm (0.004 in.).

Load Test 1

The first load test was completed November 28, 1994, 3 months following installation of the bridge. The bridge interlaminar compression at the time of the test was approximately 785 kPa (114 lb/in^2). The test vehicles, trucks A and B, were three-axle dump trucks with gross vehicle weights of 209.3 and 202.3 kN (47,050 and 45,470 lb), respectively (Fig. 10). The vehicles were positioned longitudinally on the bridge so that the rear axles were centered about the skewed midspan, with the front axles off the span (Fig. 11). Transversely, six load positions were used (Fig. 12).

Load Test 2

The second load test was completed November 13, 1996. At the time of the test, the interlaminar compression level was approximately 547 kPa (79 lb/in^2). The test vehicles consisted of fully loaded, five-axle, and two-axle trucks: trucks A and B with gross vehicle weights of 335.8 and 145.2 kN (75,500 and 32,650 lb), respectively (Fig. 13). The vehicles were positioned on the bridge longitudinally, with the rear axle(s) centered about the skewed midspan and transversely in the same six load positions used for load test 1 (Figs. 11, 12, 14).

Figure 9—Load test deflection measurements were obtained by reading values with a surveyor's level from calibrated rules suspended from the underside of the deck.

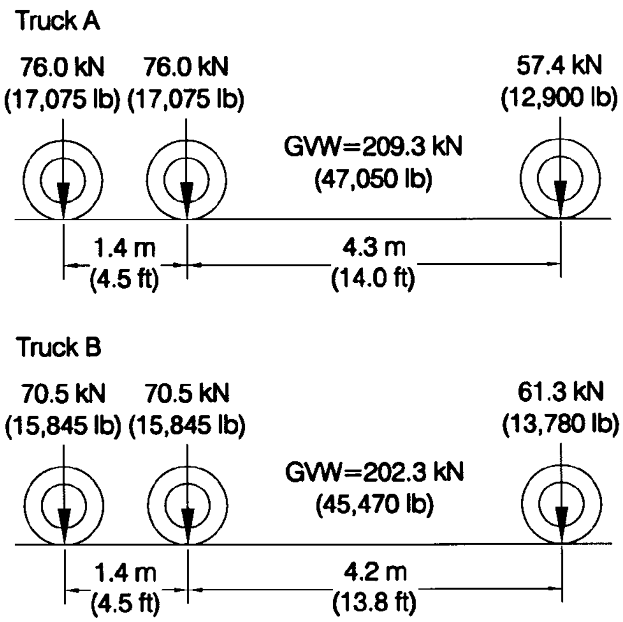

Figure 10—Load test 1 truck configurations and axle loads. The transverse vehicle track width, measured center-to-center of the rear tires, was 1.8 m (6 ft).

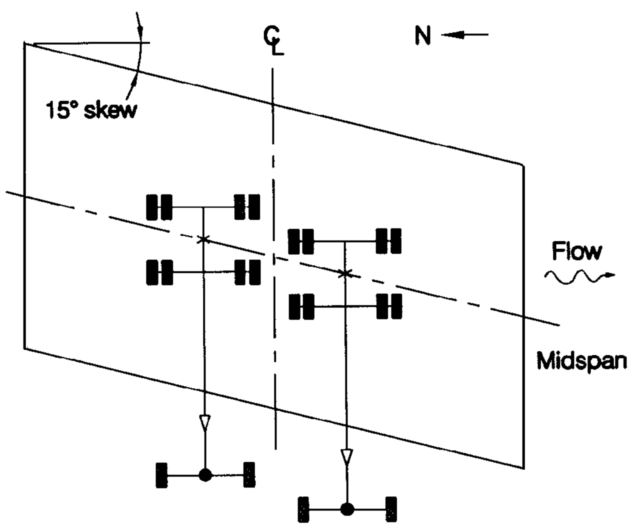

Figure 11—For each load position, the vehicles were positioned longitudinally on the bridge with the rear axles centered about the skewed midspan.

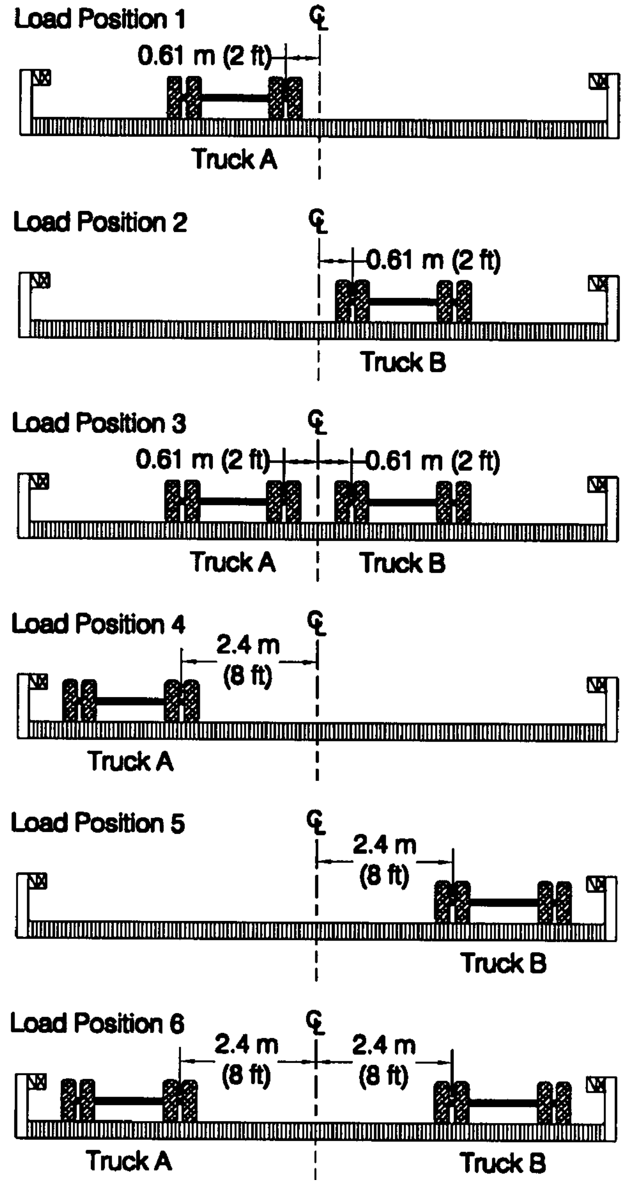

Figure 12—Transverse load positions (looking east). For all load positions, the rear axles were centered about the skewed midspan.

Analytical Evaluation

Following completion of the load tests, analytical assessments were conducted to determine the theoretical bridge response. Previous research has shown that stress-laminated decks can be accurately modeled as orthotropic plates (Ritter and others 1995b). To further investigate the theoretical behavior of the Mohawk Canal bridge, an orthotropic plate computer model currently being developed at FPL was employed to analyze the load test results and determine the theoretical bridge deflection for AASHTO HS20–44 truck loading. A modulus of elasticity (MOE) value of 13,790 MPa (2,000,000 lb/in^2) was used for modeling. Although this value is 3,447 MPa (500,000 lb/in^2) greater than the established design value for Douglas Fir glulam Combination 16F-V3, it accurately models the bridge behavior (NFPA 1991). In addition, the larger MOE value is expected to be correct because for the lower bending strength glulam combinations, the beams are often manufactured from wood greater in quality than that required.

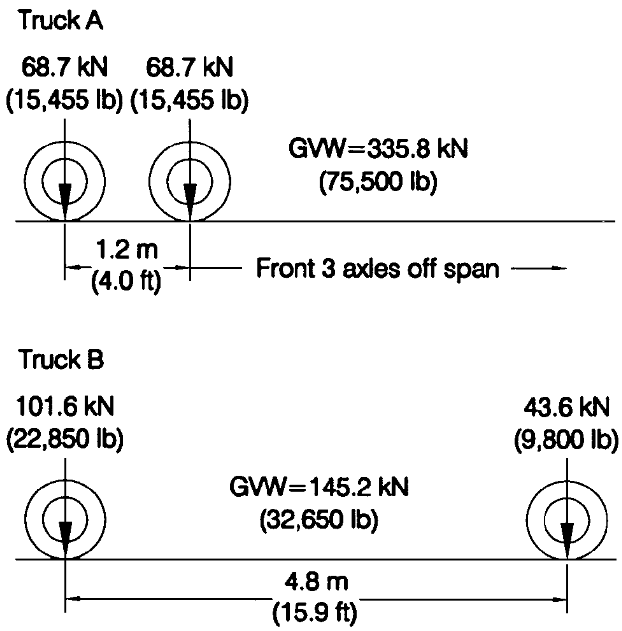

Figure 13—Load test 2 truck configurations and axle loads. The transverse vehicle track width, measured center-to-center of the rear tires, was 1.8 m (6 ft)

Figure 14—Transverse load positions 3 (top) and 5 (bottom) used for load test 2.

Condition Assessment

The overall condition of the bridge was assessed at the beginning and end of the monitoring period. The evaluations entailed visual inspections, measurements, and photographic documentation. Items of specific interest included the bridge geometry, condition of the timber deck and rail system, and condition of the stressing bars and anchorage system.

Results and Discussion

Performance monitoring of the Mohawk Canal bridge extended from November 28, 1994, through November 13, 1996. Results and discussion of the performance data follow.

Moisture Content

The average lamination moisture content of the bridge was 8% in November 1994, 3 months after superstructure installation. The average lamination moisture content 1 and 2 years later was 8% and 7.3%, respectively. The measurements indicated little change during the 2-year monitoring. The glulam beam laminations were installed at a very low moisture content, and because the local climate is extremely dry, the moisture content of the deck underwent little change. The moisture content may fluctuate with seasonal climatic changes but it is anticipated that the level will remain stable as a result of the warm, dry desert environment. Dimensional instability as a result of moisture content fluctuation is not expected.

Bar Force

The average bar force for the Mohawk Canal bridge is shown in Figure 15. It also indicates the design force and the force level required to maintain the minimum recommended interlaminar compression level of 275.8 kPa (40 lb/in^2) (Ritter 1990). Following the third bar tensioning on November 28, 1994, the average bar force was approximately 321.2 kN (72,200 lb), or 111% of the design force. For the next 13 months, the bar force declined at an approximately linear rate. In January 1996, the bar force had decreased to approximately 219.3 kN (49,300 lb), which corresponds to an interlaminar compression level of 536 kPa (78 lb/in^2). At this time, the average bar force began to increase for an approximate half year, then resumed its overall decline. This slight increase may be attributed to fluctuations in the moisture content of the wood deck. However, as a result of the frequency of the moisture content readings, this cannot be

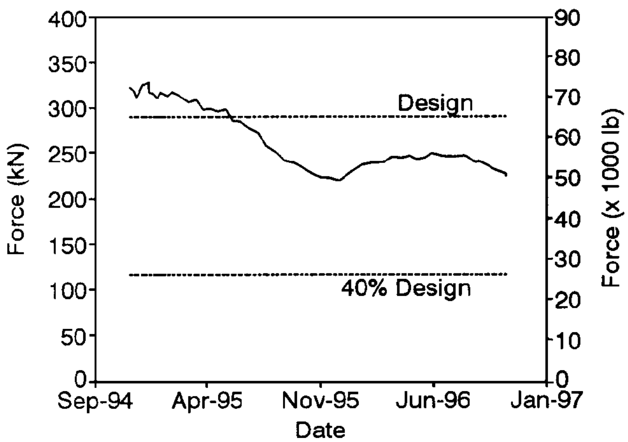

Figure 15—Average bar force.

determined with certainty. The top surface of the deck may absorb moisture from rainfall because there is no waterproof membrane beneath the asphalt and the bottom surface may absorb moisture from evaporation of the canal. At the conclusion of the monitoring period, the average bar force was approximately 224.2 kN (50,400 lb), or 78% of the design force, which corresponds to an average interlaminar compression of approximately 547 kPa (79 lb/in^2).

Vertical Creep

At the time of the first load test, approximately 13 mm (0.5 in.) of positive camber was measured with a string line and vertical rule along the edges of the bridge. At the conclusion of the monitoring period, measurements indicated that no vertical creep of the bridge had occurred.

Load Test Behavior

Static load test results and the theoretical bridge response under load test and AASHTO HS20–44 loading are presented in this section. All transverse deflections with the locations and magnitudes of the maximum measured deflections are shown at the midspan of the bridge, as viewed from the west end (looking east). For each load test, no permanent residual deformation was measured at the conclusion of the testing, and no movement was detected at either of the abutments. The interlaminar compression was approximately 785 and 547 kPa (114 and 79 lb/in^2) for load test 1 and 2, respectively. Measured deflections from each load test are typical of orthotropic plate behavior of stress-laminated bridges (Ritter and others 1990).

Load Test 1

Transverse deflections for load test 1 are shown in Figure 16. As indicated, the maximum deflections for load positions 1 and 2 occurred between the wheel lines, 610 mm (24 in.) south of the upstream (left) wheel line. For load positions 4 and 5, the maximum deflections were measured beneath the outside wheel line of the test vehicle. An absolute maximum deflection of 7.5 mm (0.30 in.) was measured beneath the downstream wheel line of truck A for load position 3 and beneath the upstream edge of the bridge for load position 6.

Assuming linear elastic behavior, uniform material properties, proper vehicle placement, and accurate deflection measurements, the summation of the deflections resulting from two individual truck loads applied separately should equal the deflection resulting from both trucks applied simultaneously. This is illustrated in Figure 17, where the sum of load positions 1 and 2 and load positions 4 and 5 are compared with load positions 3 and 6, respectively. The deflections are similar, with minor variations that are within the accuracy of the measurement methods, indicating that the bridge is behaving in a linear elastic manner under the applied loads.

Load Test 2

Transverse deflections for load test 2 are shown in Figure 18. As with load test 1, the deflections are typical of orthotropic plate behavior. The maximum measured deflection for load position 1 occurred between the wheel lines. For load position 2, the maximum deflection occurred beneath the upstream (left) wheel line. As with load test 1, the maximum deflections for load position 4 and 5 occurred beneath the outside wheel line of the test vehicle. For load position 6, the maximum deflection occurred between the wheel lines of truck A. The absolute maximum deflection of 6.7 mm (0.26 in.) occurred during load position 3, beneath the downstream (right) wheel line of truck A.

As illustrated with the measured deflections of load test 1, the summation of deflections resulting from two separately applied loads should equal the deflection of both loads applied together, if uniform material properties, proper vehicle placement, and accurate deflection measurements are assumed. This principal of superposition is illustrated in Figure 19, where the sum of load positions 1 and 2 and load positions 4 and 5 are compared with load positions 3 and 6, respectively. As with load test 1, the deflections are virtually the same, with slight variations within the accuracy of the measurement methods, demonstrating 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 are shown in Figures 20 and 21 for load test 1 and 2, respectively. As illustrated, the theoretical bridge deflection is very close to that measured, with the greatest differences occurring at the edges of the bridge. Employing the same analytical parameters used to determine the theoretical bridge response, the theoretical deflection for

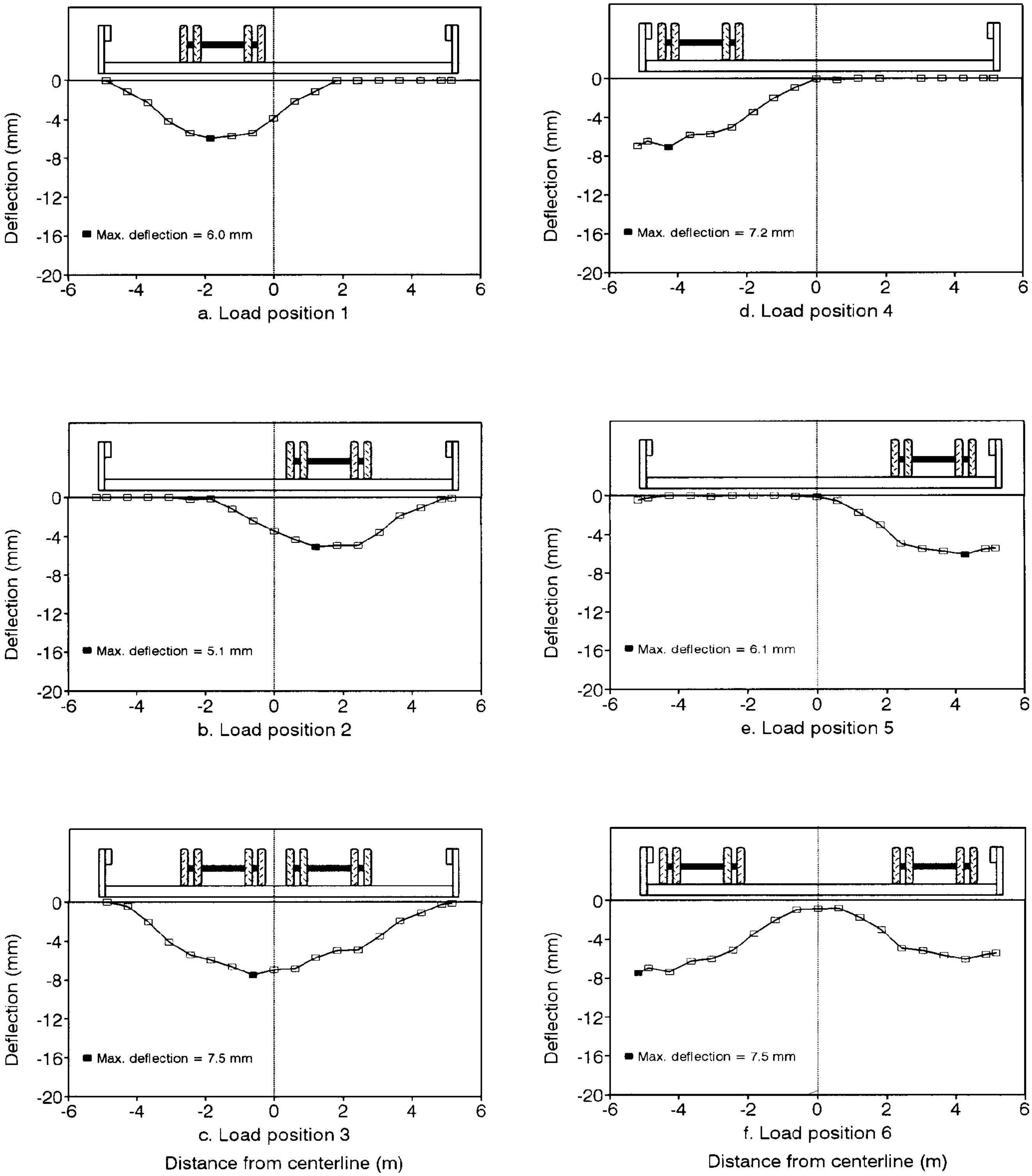

Figure 16—Transverse deflections for load test 1, measured at the bridge midspan (looking east). Bridge cross-sections and vehicle positions are shown to aid interpretation and are not to scale.

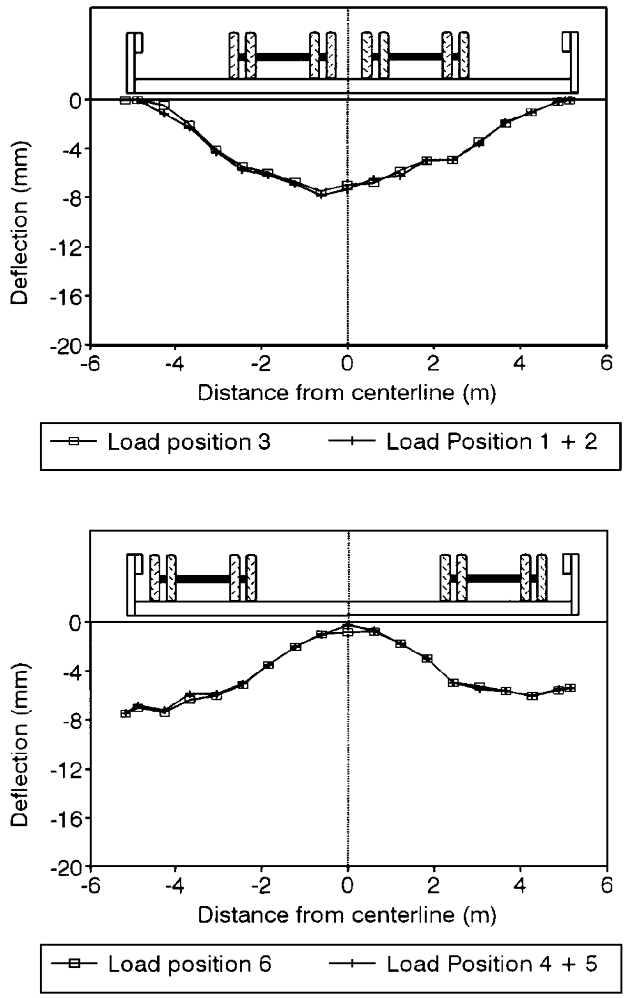

Figure 17—Transverse deflections for load test 1, comparing the sum of measured deflections from load positions 1 and 2 and load positions 4 and 5 with load positions 3 and 6, respectively. Bridge cross-sections and vehicle positions are shown to aid interpretation and are not to scale.

AASHTO HS20–44 truck loading is shown in Figure 22. Based on this analysis, the maximum AASHTO HS20–44 static deflection occurred at centerline when two HS20–44 vehicles were placed centrically on the bridge. The resulting theoretical maximum deflection is approximately 7.6 mm (0.30 in.) or 1/804 of the bridge span for load test 1, and approximately 7.3 mm (0.29 in.) or 1/831 of the bridge span for load test 2. The results indicate that the longitudinal bridge stiffness was approximately the same for both load tests.

Condition Assessment

Condition assessments of the Mohawk Canal bridge indicate that structural and serviceability performance are acceptable. Inspection results for specific items follow.

Bridge Geometry

Measurements taken during the monitoring period indicate that the width of the bridge remained relatively stable, narrowing approximately 20 mm (0.84 in.). The change in width is most likely the result of stress relaxation in the laminations, resulting from the applied compression. Additional reductions in the width of the bridge are not anticipated because the laminations are extremely dry.

Wood Condition

Inspection of the wood components of the bridge revealed that the glulam beams appeared to be in good condition. Checks were not present in the exterior deck laminations. No evidence existed to indicate preservative loss, and preservative or solvent accumulations were not present on the wood surface. However, inspection of the underside of the deck revealed that small wood splinters with cross sections of approximately 85 mm^2 (0.13 in^2) were separating or splitting longitudinally from the bottom lamination of the glulam beams. This phenomenon did not occur in all glulam beams, and typically only one separation or splinter occurred per beam.

The glulam rail was in excellent condition, exhibiting no checks or delaminations. Severe checks and splits were observed in the railposts and spacer blocks (Fig. 23). A review of photographs taken during construction revealed that the posts were severely checked when installed. Such a condition is not advisable because the splits penetrate the preservative envelope and provide avenues for moisture penetration, accumulation, and potential decay as well as reduce the strength characteristics of the member. At this time, the posts and spacer blocks exhibit no visible signs of decay. Such deterioration is not anticipated in the near future because of the dry local environment; however, these areas should be closely monitored in future inspections of the bridge.

Wearing Surface

Inspection of the wearing surface at the conclusion of monitoring revealed a slight deterioration of the asphalt. The top surface had experienced some erosion, causing the wearing surface to appear to be constructed of compacted gravel. In addition, cracks were noted in the wearing surface at the end of the bridge and approach road interface (Fig. 24). Both the erosion and interface cracks are attributed to a deficiency in the asphalt mix or application procedures. Aside from this erosion, the surface of the bridge appeared to be in good condition and exhibited no additional signs of cracking, rutting, or other deterioration.

Bar Anchorage System

The galvanized stressing bar anchorage system is performing as designed and shows no signs of distress. There is no

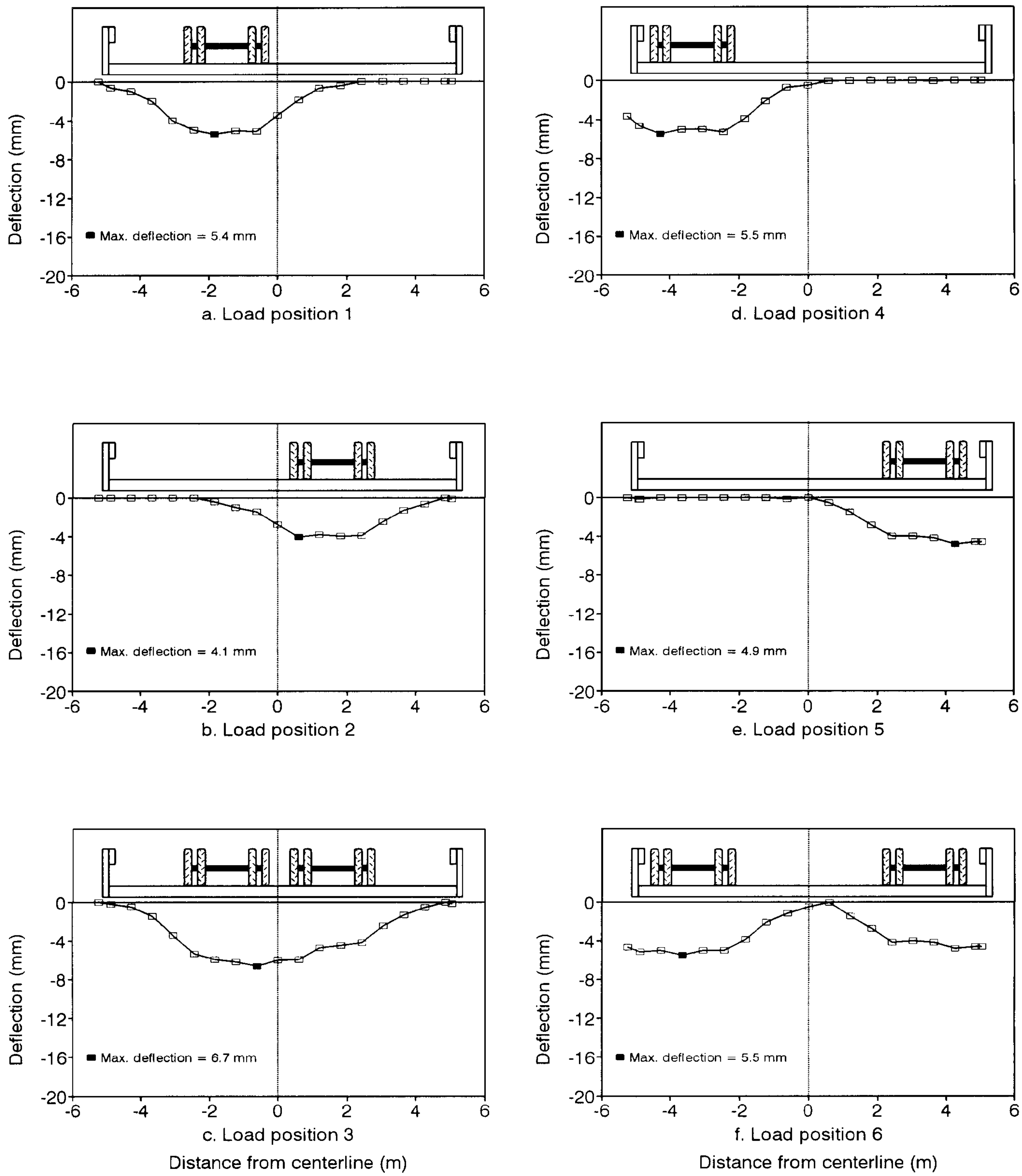

Figure 18—Transverse deflections for load test 2, measured at the bridge midspan (looking east). Bridge cross-sections and vehicle positions are shown to aid interpretation and are not to scale.

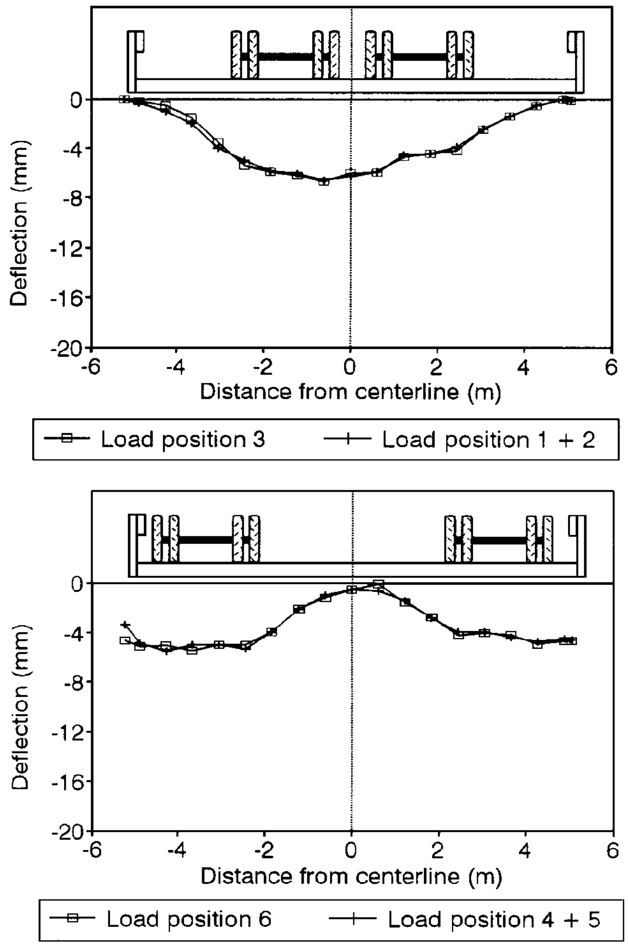

Figure 19—Transverse deflections for load test 2, comparing the sum of measured deflections from load positions 1 and 2 and load positions 4 and 5 with load positions 3 and 6, respectively. Bridge cross-sections and vehicle positions are shown to aid interpretation and are not to scale.

indication of the discrete plate anchorage crushing into the exterior laminations and no measurable distortion of the bearing plate. The steel plates and exposed steel stressing bars and hardware exhibited no visible signs of corrosion or other distress.

Conclusions

After approximately 2 years in service, the Mohawk Canal bridge is exhibiting good performance, although minor serviceability deficiencies were noted. Based on monitoring conducted since construction of the bridge, the following observations and recommendations are made:

- It is feasible to construct stress-laminated decks using Douglas Fir glulam beams.
- On-site assembly of the deck is a viable method of bridge construction, although some traffic disruption does occur.
- The average moisture content in the outer 25 to 51 mm (1 to 2 in.) of the laminations of the Mohawk Canal bridge remained relatively stable throughout the monitoring period at approximately 8%. Because the lamination moisture content was low at the time of installation and the local climate is extremely dry, the moisture content of the deck underwent little change. The bridge is located in a desert environment and the moisture content will likely remain stable; therefore, dimensional stability problems as a result of moisture content fluctuations and subsequent bar force retention problems are not expected.
- During the monitoring period, the average bar force for the Mohawk Canal bridge decreased from 321.2 to 224.2 kN (72,200 to 50,400 lb), which corresponds to a decrease in interlaminar compression from 785 to 547 kPa (114 to 79 lb/in^2). Overall, the bars are maintaining a high level of interlaminar compression. Slight changes in bar force may be attributed to local changes in moisture content caused by rainfall or evaporation of the canal.
- Measurable vertical creep did not occur during the monitoring period.
- Load testing and analysis indicate that the Mohawk Canal bridge exhibits linear elastic orthotropic plate behavior when subjected to static truck loading. Comparisons of the analytical load test results at different levels of interlaminar compression indicate that the longitudinal bridge stiffness was approximately the same for both load tests. The maximum predicted deflection as a result of AASHTO HS20–44 static truck loading is estimated to be 7.6 mm (0.30 in.) (L/804) for 785 kPa (114 lb/in^2) interlaminar compression, and 7.3 mm (0.29 in.) (L/831) for 547-kPa (79-lb/in^2) interlaminar compression.
- A minor reduction of 20 mm (0.84 in.) in bridge width was noted at the conclusion of the monitoring period and is probably the result of stress relaxation in the laminations. Additional reductions are not anticipated because the laminations are extremely dry.
- Visual inspections of the bridge indicate that the performance of most wood components is satisfactory, although severe checks and splits were evident in the railposts and spacer blocks. Photographs taken during construction indicate that the railposts were severely split when installed. Members exhibiting such severe conditions should not be used in railing construction.
- The asphalt wearing surface exhibited deterioration, resulting in a gravel-like appearance. Cracks were visible at the end of the bridge and approach road interface. These conditions are attributed to a deficiency in the asphalt mix or application procedures.
- The exposed steel stressing bars and hardware show no visible signs of corrosion or other distress. The discrete plate bar anchorage is not distorted or crushing into the lumber laminations.

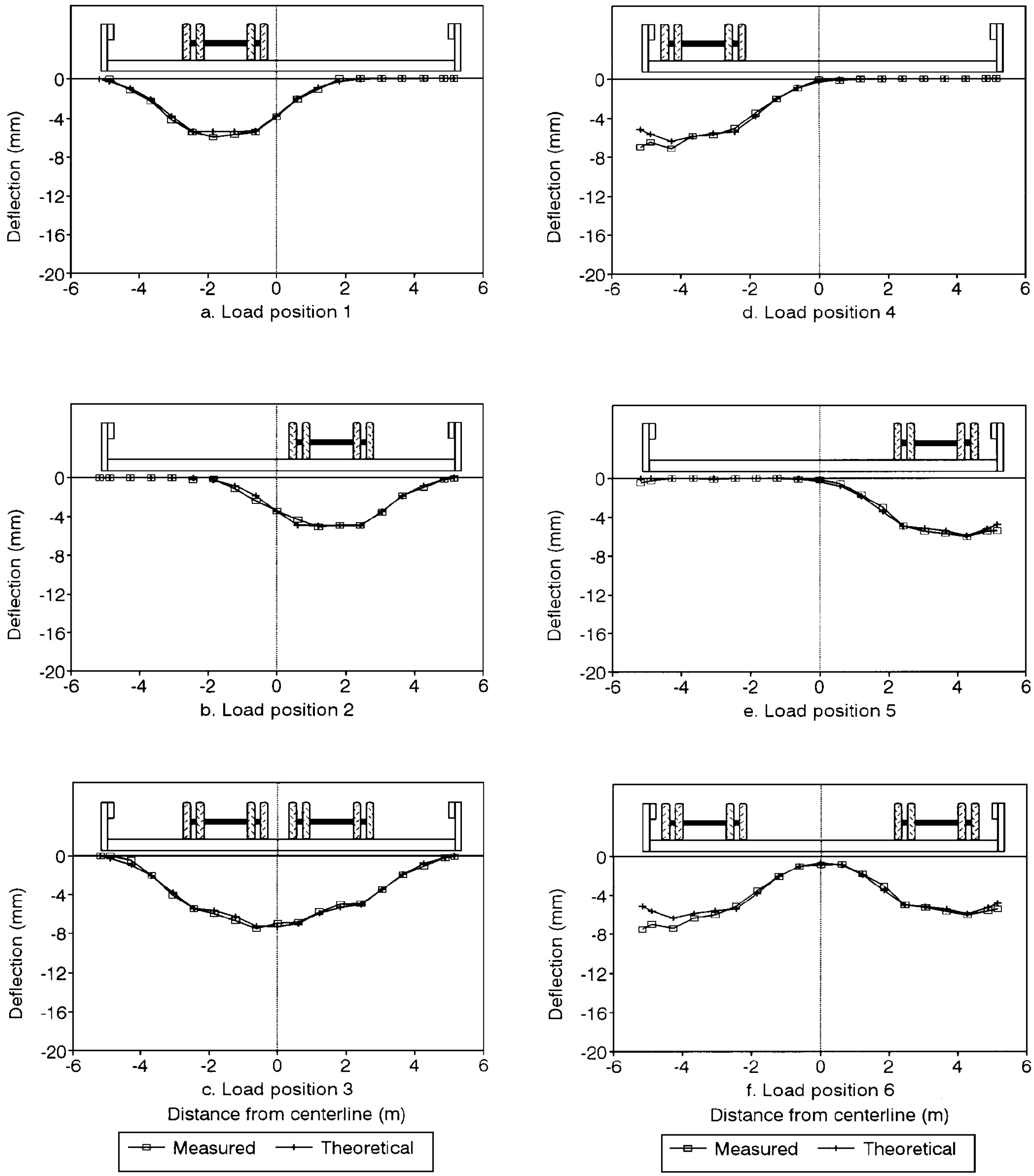

Figure 20—Comparison of the measured deflections for load test 1 with the theoretical deflections using orthotropic plate analysis (looking east). Bridge cross-sections and vehicle positions are shown to aid interpretation and are not to scale.

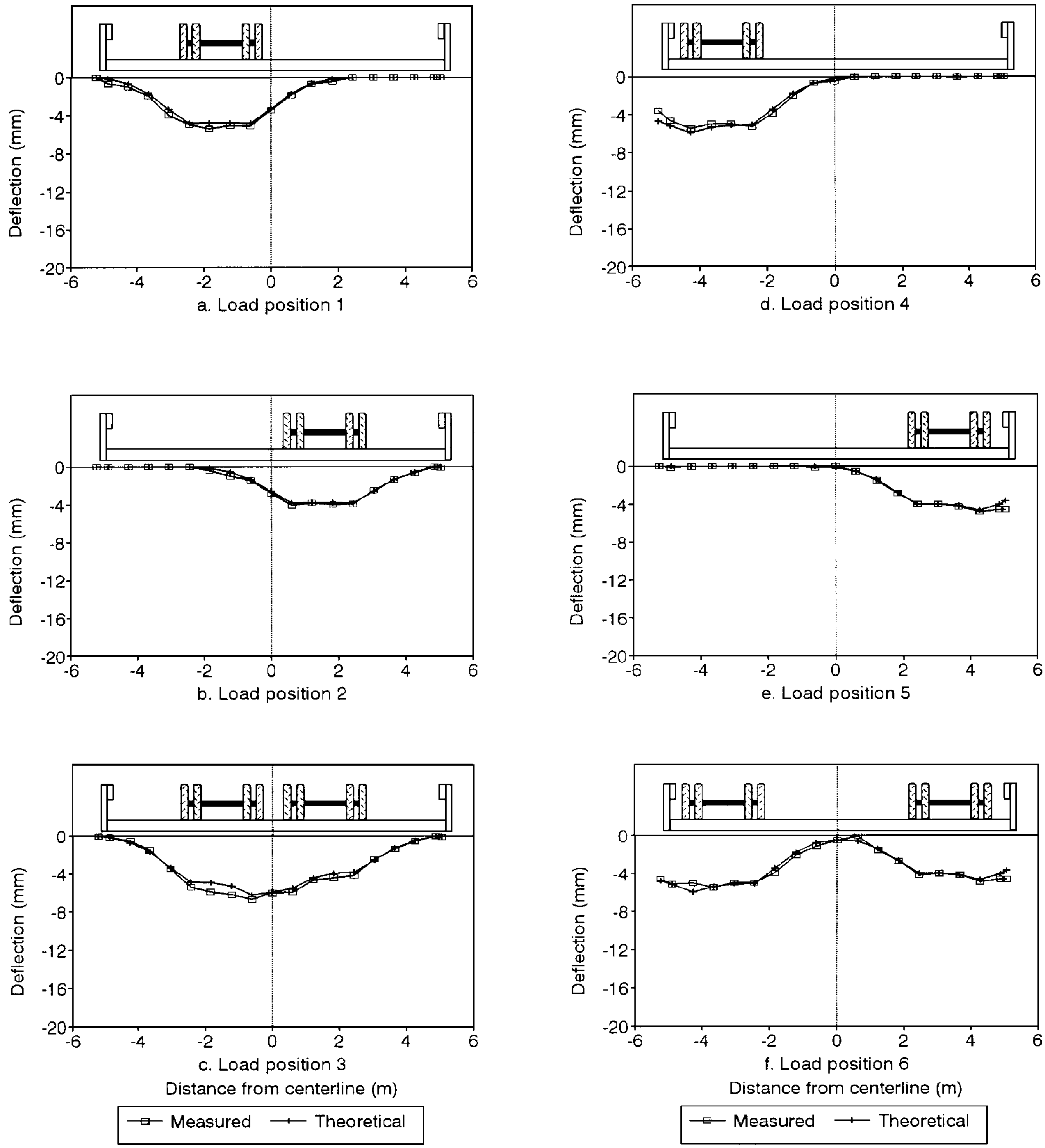

Figure 21—Comparison of the measured deflections for load test 2 with the theoretical deflections using orthotropic plate analysis (looking east). Bridge cross-sections and vehicle positions are shown to aid interpretation and are not to scale.

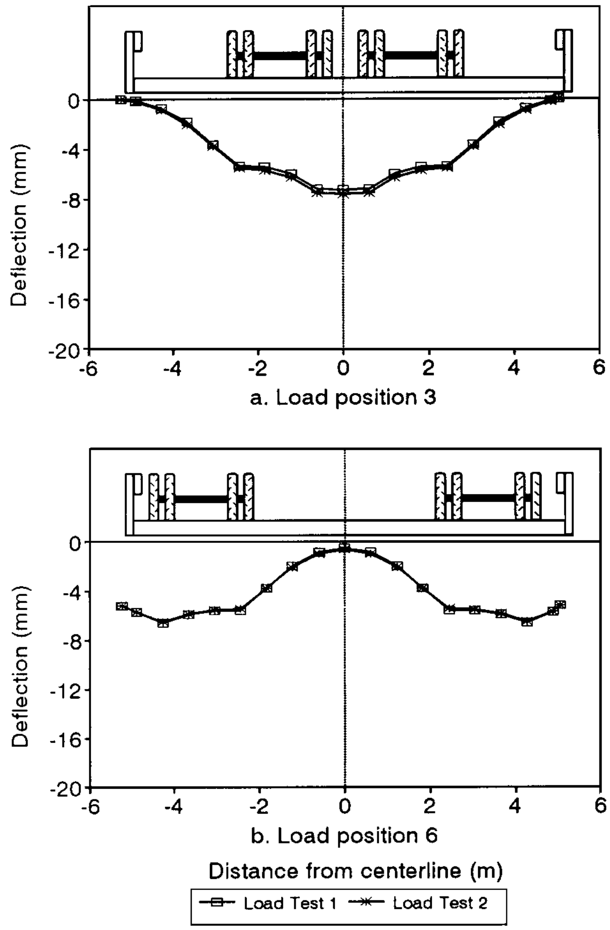

Figure 22—Maximum predicted deflection profiles at the bridge midspan for AASHTO HS20–44 truck loading for load positions 3 and 6 (looking east). Bridge cross-sections and vehicle positions are shown to aid interpretation and are not to scale.

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Figure 23—Severe checks and splits in railposts and spacer blocks noted during condition assessment.

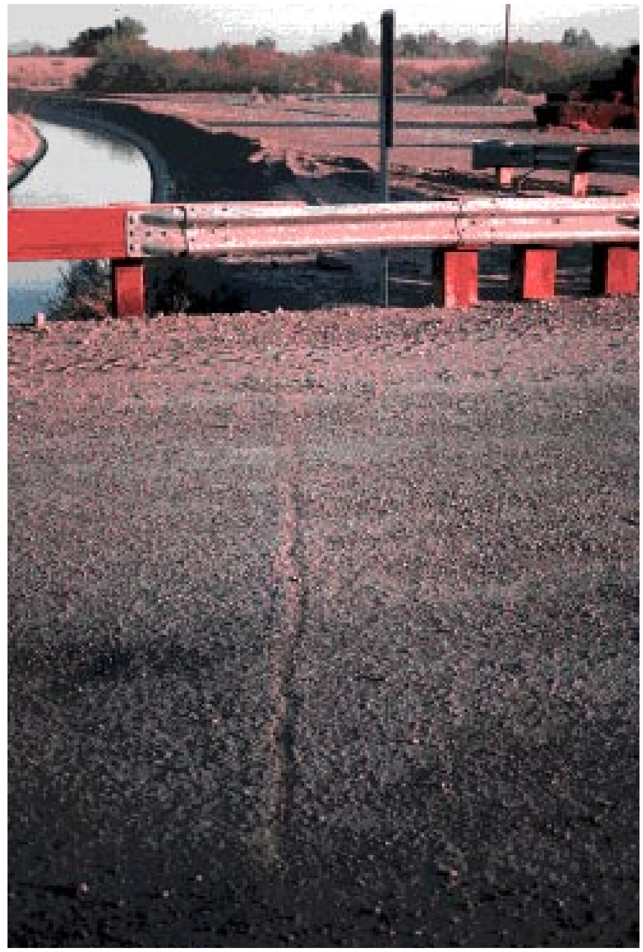

Figure 24—Crack in asphalt at end of bridge and approach road interface observed during condition assessment.

Appendix—Information Sheet

General

Name: Mohawk Canal bridge

Location: County 5th Street, west of Roll, Arizona

Date of Construction: August 1994

Owner: Yuma County

Design Configuration

Structure Type: Stress-laminated deck

Butt Joint Frequency: None

Total Length (out–out): 6.4 m (21 ft)

Skew: 15 degrees

Number of Spans: 1

Span Length (center-to-center bearings): 6.1 m (20 ft)

Width (out–out): 10.2 m (33.9 ft)

Width (curb–curb): 9.6 m (31.9 ft)

Number of Traffic Lanes: 2

Design Loading: AASHTO HS20–44

Wearing Surface Type: Asphalt

Material and Configuration

Timber:

Species: Douglas-fir

Size (actual): 130 mm wide (5.13 in.)
298 mm deep (11.75 in.)

Grade: Glulam Combination 16F-V3

Moisture Content: 8% average 3 months after installation at 25- to 51-mm (1- to 2-in.) depth

Preservative Treatment: Pentachlorophenol with heavy oil

Stressing Bars:

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

Diameter: 25 mm (1 in.)

Number: 7 (3 transverse the full bridge width)

Design Force: 289 kN (65,000 lb)

Spacing: 1.4 m (54 in.) average center-to-center

Anchorage Type and Configuration:

Steel Plates: 343 by 305 by 32 mm (13-1/2 by 12 by 1-1/4 in.) bearing
178 by 102 by 25 mm (7 by 4 by 1 in.) anchor