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Field Performance of Timber Bridges

21. Humphrey Stress-Laminated T-Beam Bridge

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Abstract

The Humphrey bridge was constructed during the summer and fall of 1993 in Cattaraugus County, New York. The bridge is a single-span, stress-laminated T-beam structure that measures 14.1 m (48.6 ft) long and 10.2 m (33.5 ft) wide. Performance of the bridge was monitored for 35 months, beginning approximately 8 months after installation. Monitoring involved gathering and evaluating data relative to the moisture content of the wood components, force level of the stressing bars, and behavior of the bridge under static load conditions. In addition, comprehensive visual inspections were conducted to assess the overall condition of the structure. Based on field evaluations, the bridge is performing well, with only a few minor serviceability issues.

Keywords: Timber, bridge, T-beam, creosote

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21. Humphrey Stress-Laminated T-Beam Bridge

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Introduction

In 1988, the U.S. Congress passed legislation known as the Timber Bridge Initiative (TBI), which was enacted to develop a program to expand the use of wood in infrastructure development. The USDA Forest Service was assigned the responsibility for program development. The resulting program was divided into three programs areas: research, demonstration projects, and technology transfer. The National Wood in Transportation Information Center (NWITIC) (formerly the Timber Bridge Information Resource Center, TBIRC) in Morgantown, West Virginia, took on the role of assigning funds for demonstration timber bridges and distributing information on timber bridge performance research projects. The USDA Forest Service, Forest Products Laboratory (FPL) in Madison, Wisconsin, was assigned the research area of the program. As part of the research strategy, FPL developed a monitoring program that examined some of the early demonstration timber bridges that were funded through the NWITIC program. To date, FPL has included more than 100 bridges in its monitoring program.

In addition to the TBI legislation, Congress passed the Intermodal Surface Transportation Efficiency Act (ISTEA) in 1991. ISTEA included provisions for a research 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). The program developed by the FHWA included funding for demonstration timber bridges, technology transfer, and research programs. Many aspects of the FHWA timber bridge research program paralleled those underway at the FPL; therefore, a joint effort was initiated to combine the respective research of the two agencies. The resulting FPL–FHWA monitoring program has examined additional bridges funded through ISTEA.

As part of the FPL–FHWA monitoring program, many bridges constructed as demonstration structures have been monitored for several years after construction. This report is the 21st in a series of reports documenting the field performance of timber bridges. It describes the development, design, construction, and field performance of the Humphrey bridge in Cattaraugus County, New York. The bridge is a 14.8-m- (48.6-ft-) long, double lane, single-span stress-laminated T-beam structure. Built in 1993, the Humphrey bridge was funded through a competitive grant from the FHWA, with 20% of the funds supplied by Cattaraugus County, New York. An information sheet on the characteristics of the Humphrey bridge is provided in the Appendix.

Background

The Humphrey bridge is located in Cattaraugus County, approximately 8 km (5 miles) north of Olean, New York, on Cattaraugus County Route 19 (Fig. 1). The bridge is owned by Cattaraugus County and is part of a vital link from smaller rural communities to Olean. According to Cattaraugus County officials, in 1993 the average daily traffic over this section of the road was approximately 1,000 vehicles. The majority of this traffic consisted of passenger vehicles and light commercial trucks.

The original Humphrey bridge was 7.3 m (24 ft) wide and 9.8 m (32 ft) long and consisted of a steel grate deck on steel stringers with concrete abutments (Fig. 2). When periodic (annual) monitoring revealed that the steel components were deteriorating and hydraulic flow under the bridge was insufficient during high water, replacement of the bridge was deemed necessary.

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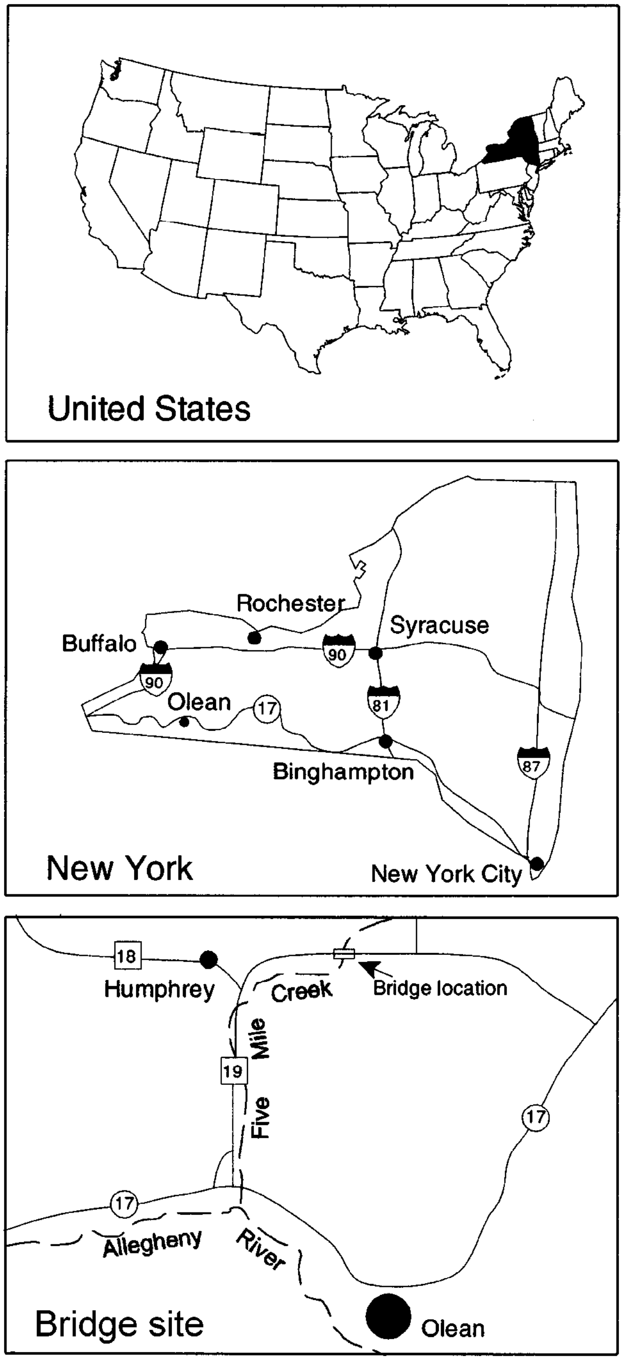

Figure 1—Location maps for Humphrey bridge.

Cattaraugus County decided to pursue replacement with a timber bridge because of funding available under ISTEA. The Cattaraugus County Department of Public Works submitted a proposal to ISTEA for a demonstration timber bridge. As part of this proposal, a preliminary bridge design was developed that consisted of a stress-laminated T-beam bridge using Southern Pine glued-laminated (glulam) webs and hardwood lumber for the flanges. After approval by

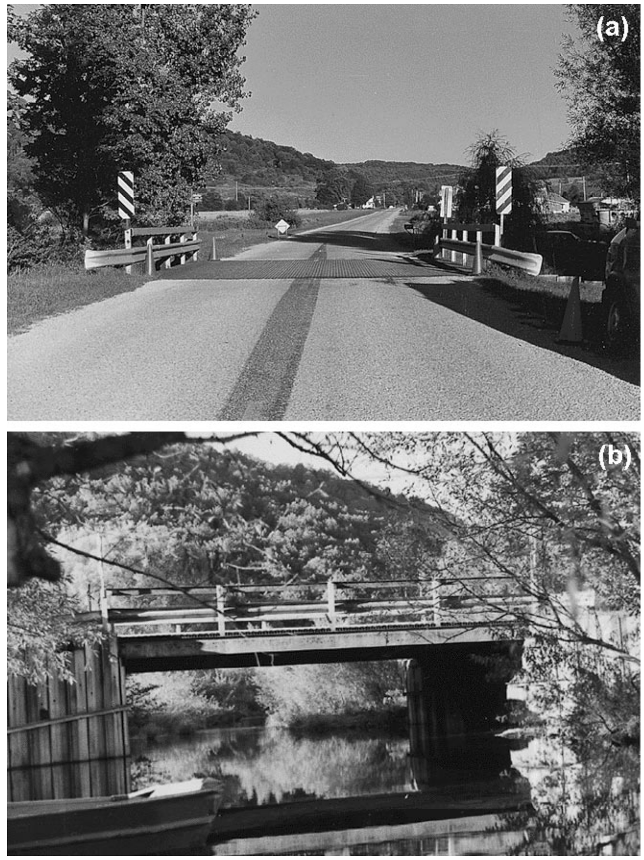

Figure 2—Original bridge: (a) end view, (b) side view.

the FHWA, funds were awarded and final design of the replacement bridge was initiated. The bridge was constructed in the summer and fall of 1993. Approximately 3 months after construction, FPL was contacted to monitor the new bridge. The FPL and Cattaraugus County Department of Public Works developed a monitoring plan that was initiated approximately 8 months after bridge installation.

Objective and Scope

The objective of this project was to evaluate the field performance of the Humphrey bridge for 35 months. The project scope included data collection and analysis related to wood moisture content, 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 T-beam bridges.

Design and Construction

Design and construction of the Humphrey bridge were completed by several agencies and individuals. An overview of the design and construction of the bridge superstructure is presented.

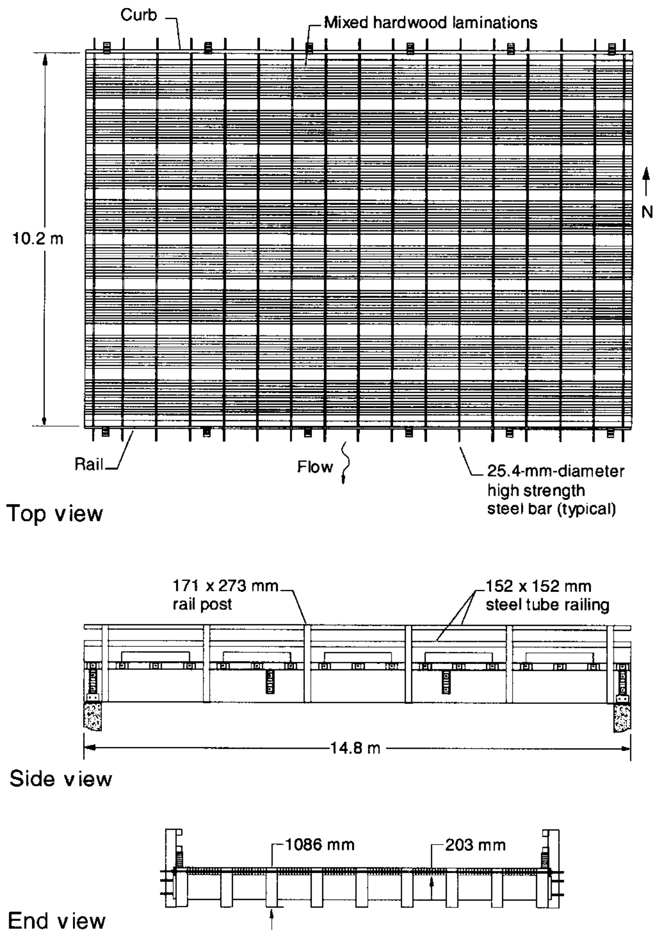

Figure 3—Design configuration of Humphrey bridge.

Design

Design of the Humphrey bridge was completed by a consulting engineering firm retained by Cattaraugus County. The design geometry provided for a single-span, simply supported structure 14.8 m (48.6 ft) long and 10.2 m (33.5 ft) wide (Fig. 3). To allow for additional hydraulic flow, the new bridge was designed to be 5.1 m (16.5 ft) longer than the original bridge.

Aside from those 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 1992) for two lanes of AASHTO HS25-44 truck loading. An AASHTO-approved method for designing the stress-laminated T-beam bridge was not available at the time of design; therefore, design criteria were based on standard guidelines developed from research conducted at West Virginia University (Davalos and Salim 1993).

The Humphrey bridge configuration consisted of Southern Pine glulam lumber webs and mixed hardwood sawn lumber

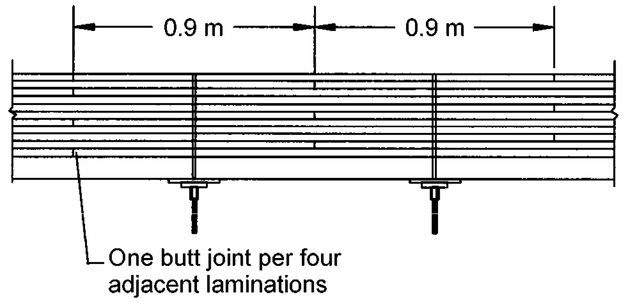

Figure 4—Butt-joint configuration used with sawn lumber flange laminations. A butt joint was placed transverse to the span in every fourth lamination. Longitudinally, butt joints in adjacent laminations were separated by 0.9 m (3 ft).

flanges. The glulam webs were 311.2 mm (12-1/4 in.) wide and 1085.8 mm (42-3/4 in.) deep. Glulam design was based on material properties for combination 24F–V3 SP/SP, as specified by AASHTO (1992). Tabulated design values were 16.5 MPa (2.4×10^3 lb/in²) for bending strength, 12.41 GPa (1.8×10^6 lb/in²) for modulus of elasticity (MOE), 1.4 MPa (200 lb/in²) for shear strength, and 3.9 MPa (560 lb/in²) for compression strength perpendicular to grain. All design values were adjusted by appropriate wet-use factors per AASHTO requirements.

The mixed hardwood sawn lumber flanges were constructed with 51- by 203-mm (2- by 8-in.) roughsawn material. Butt joints were used in the sawn lumber flanges because material was not available in the required full 14.8-m (48.6-ft) length. Butt joints were specified at an interval of one butt joint every four adjacent laminations, spaced 0.9 m (3 ft) longitudinally (Fig. 4). Design values for the sawn lumber flange material were based on the mixed hardwoods species group, graded No. 2 in accordance with Northeastern Lumber Manufacturers' Association (NELMA) rules (NELMA 1991). The specified design values for the mixed hardwood lumber were 9.0 MPa (1.3×10^3 lb/in²) for bending strength, 620.5 kPa (90 lb/in²) for shear strength, and 4.2 MPa (615 lb/in²) 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% moisture content prior to preservative treatment and to bridge installation.

The deck stressing system used seventeen 25.4-mm- (1-in.-) diameter, coarse, left-hand thread, high strength steel bars that complied with the requirements of ASTM A722 (ASTM 1988) and provided a minimum ultimate tensile strength of 1,034 MPa (150×10^3 lb/in²). The bars were inserted through oversized, predrilled holes located at the center of the sawn lumber flange and 102 mm (4 in.) from the top of the glulam webs. Fifteen bars were spaced 914 mm (36 in.) on center, starting 1,013 mm (39.875 in.) from the bridge ends. The two remaining bars were placed 229 mm (9 in.) from the

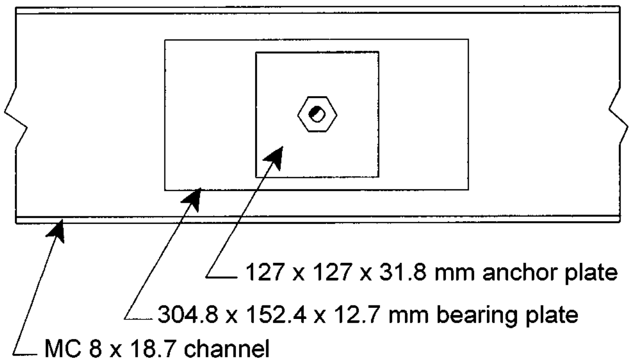

Figure 5—Anchorage configuration consisting of semi-continuous channel, bearing plate, anchor plate, and hexagonal bell nut.

ends. In addition to the deck stressing system, eight additional full-width stressing bars of the same type were used to hold four sets of diaphragms in place at the ends of the bridge and at third points along the length. All bars required a tensile force of 320.3 kN (72,000 lb) to provide 1.72 MPa (250 lb/in^2) compression stress between the laminations. The tension in each bar was transferred into the deck using a steel channel anchorage system consisting of an MC 8 by 18.7 channel, a 305- by 152- by 13-mm (12- by 6- by 0.5-in.) bearing plate, and a 127- by 127- by 32-mm (5- by 5- by 1.25-in.) anchor plate with a hexagonal bell nut (Fig. 5). The steel channel was not continuous over the span because of the location of the railing posts.

Design of the bridge rail and curb system was based on a combination of standard crash-tested systems for New York State (NYDOT 1981) and the Forest Service. The bridge rail and curb consisted of two 152- by 152- by 4.8-mm (6- by 6- by 0.1875-in.) steel tube rails and a 172- by 419-mm (6.75- by 16.5-in.) glulam curb and scupper block combination. The rail and curb were attached to 2,115- by 172- by 267-mm (83.25- by 6.75- by 10.5-in.) sawn lumber posts. The six posts on each side were spaced 3 m (9 ft) apart, starting 0.6 m (1.8 ft) from the end of the bridge.

For protection from deterioration, all steel components, including stressing hardware, stressing bars, and anchorage plates, were galvanized in accordance with AASHTO M232 (AASHTO 1990). All wood components were preservative treated with creosote in accordance with American Wood Preservers' Association (AWPA) standard C14 (AWPA 1990). A 51- to 76-mm (2- to 3-in.) asphalt-wearing surface with membrane was also specified for the bridge.

Construction

Construction of the Humphrey bridge was completed in the fall of 1993. During construction, a detour was established just north of the bridge. To create the detour, the contractor moved the original bridge upstream and set it on temporary steel abutments. The detour limited road closure to only one afternoon while the original bridge was moved.

The concrete abutments that had supported the original bridge were removed from the site, and the site was prepared for the new concrete abutments. The new abutments consisted of a concrete footing and wing walls. A steel plate was set into the top of each abutment to facilitate attachment of the glulam beams by steel bearing shoes.

The glulam beams and the sawn lumber laminations were milled and manufactured at a separate location. Materials were then prefabricated (cut and drilled) to the finished specifications and pressure treated with creosote. After preservative treatment, all components were shipped to the bridge site for installation.

The glulam beams were set on the abutments. Each beam was set in steel angle shoes on the abutments; the shoes were welded into place after the remainder of the bridge was installed. The sawn lumber deck pieces were moved to the contractor's yard where they were assembled into deck flange sections. Each sawn lumber piece was placed in position on temporary supports, and galvanized steel tubes were inserted through the drilled holes (Fig. 6). Each deck assembly was then banded and loaded on a flatbed trailer for delivery to the bridge site. The diaphragm sections were completed in a similar manner (Fig. 7).

When all the deck and diaphragm sections were assembled and shipped to the bridge site, each section was lifted into position between the glulam beams (webs). First, four diaphragm sections were placed between two beams (Fig. 8). Then, the deck section was placed on top of the diaphragm sections and all steel tubing was inserted in the holes in the beams. The beams flanking the diaphragms and deck sections were then moved closer together. This process was completed until all diaphragm and deck sections were installed. After all the wood components of the deck were installed, a temporary steel strand was inserted through the flange at the center of the bridge and stressed to bring all the laminations into contact. During this time, the timber posts and curb were installed on the exterior beams (Fig. 9).

The high strength steel bars were then inserted into the predrilled holes in the deck and diaphragm sections of the bridge. The holes in the glulam webs were found to be too small to allow the bars to move freely through the bridge. Consequently, the holes were drilled again using a larger bit size on a long extension that stretched across the width of the bridge. The enlarged holes did not solve the problem, and the bars were forced through the deck by a jackhammer (Fig. 10). After the bars were positioned, the channel sections, plates, and nuts were installed at the ends of each bar.

The bars were tensioned on August 31, 1993, using a single hydraulic ram and pump (Fig. 11). Starting at one end of the bridge, all bars were tensioned to the design force, including

Figure 6—Fabrication of flange assemblies.

Figure 7—Finished diaphragm assemblies awaiting transport to bridge site.

Figure 8—Installation of diaphragm and flange assemblies at bridge site.

Figure 9—Installation of rail posts, curb, and scupper block combination on exterior beams.

Figure 10—Insertion of tension bars using a jack hammer.

Figure 11—Bar tensioning using a single hydraulic pump and ram.

Figure 12—Completed Humphrey bridge: (a) side view, (b) end view.

the bars that held the diaphragms. This procedure was repeated to ensure uniform tension in all bars. Approximately 7 days after the first tensioning, the bars were tensioned again in the same manner. After the second tensioning (September 6, 1993), the bridge was paved, striped, and opened for traffic. The detour was closed and the original bridge was removed. Twenty days after the second bar tensioning, the steel beam shoes were welded to the top of the abutments. Approximately 7 weeks after the initial bar tensioning (October 22, 1993), the bars were tensioned for the third time. The completed bridge is shown in Figure 12.

Evaluation Methodology

To evaluate the structural and serviceability performance of the Humphrey bridge, Cattaraugus County contacted FPL for assistance. Through mutual agreement, a 3-year bridge-monitoring plan was developed and implemented approximately 8 months after bridge construction. The plan included performance monitoring of deck moisture content, stressing bar force, static load test behavior, and general bridge condition. The evaluation methodology employed procedures and equipment previously developed by FPL and used on similar structures (Ritter and others 1991). In addition, a remote data acquisition system was used to obtain and record bar force measurements.

Moisture Content

An electrical resistance moisture meter was used to characterize changes in moisture content. Moisture measurements were taken on the glulam beams, sawn lumber flanges, and sawn lumber diaphragms. These locations were used to gather information on moisture content on the three elements of the bridge. Measurements were obtained in accordance with ASTM D4444–92 (ASTM 1992) by driving the insulated moisture pins into the bridge underside at depths of 51 to 76 mm (2 to 3 in.), recording the moisture content value from the unit, then adjusting this value for temperature and wood species, if necessary.

Bar Force

To monitor stressing bar force, three calibrated load cells were placed on the stressing bars at the initiation of monitoring. The cells were placed between the bearing and anchorage plates to measure bar force based on the strain variations in the load cell. Load cell readings were obtained and recorded by a remote data acquisition (data logger) system installed on site. The data logger obtained readings on an hourly basis that were stored for future download. A telephone line was also installed at the site that allowed interface with the data logger at any time through modem communications. At monthly intervals, data were downloaded through the modem. At the conclusion of the monitoring period, the load cells were removed, adjusted for zero balance shift, and re-calibrated in the laboratory. In addition to taking load cell readings, hydraulic stressing equipment was used during site visits to verify bar force levels.

Load Test Behavior

Two static load tests were conducted during the monitoring period. The first load test was completed in June 1994, and the second test was completed approximately 3 years later in May 1997.

The static load test consisted of positioning fully loaded trucks on the bridge and measuring the resulting deflections at midspan. For both load tests, the trucks were positioned for six transverse load positions (Fig. 13). For each load position, trucks were positioned with their center of gravity at the bridge midspan. Each truck faced the direction of traffic in their respective lane (Fig. 14). Measurements of bridge deflections were taken prior to testing (unloaded), for each load position (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 bridge and reading values with a surveyor's level. The accuracy of this method for repetitive readings is estimated to be ±1 mm (0.04 in.).

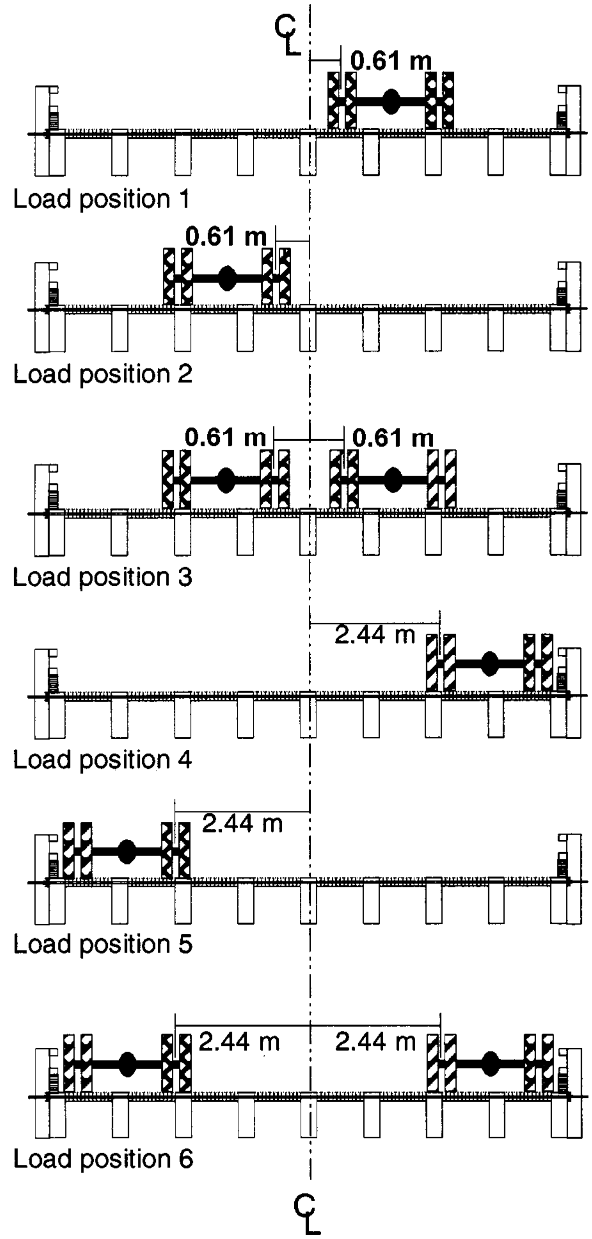

Figure 13—Transverse load test positions used for load tests: load position 1, 0.6 m upstream; load position 2, 0.6 m downstream; load position 3, combination of load positions 1 and 2; load position 4, 2.44 m upstream; load position 5, 2.44 m downstream; load position 6, combination of positions 4 and 5. All measurements were made from the longitudinal centerline.

Figure 14—Load testing of trucks in load position 3. Note the location and direction of each truck.

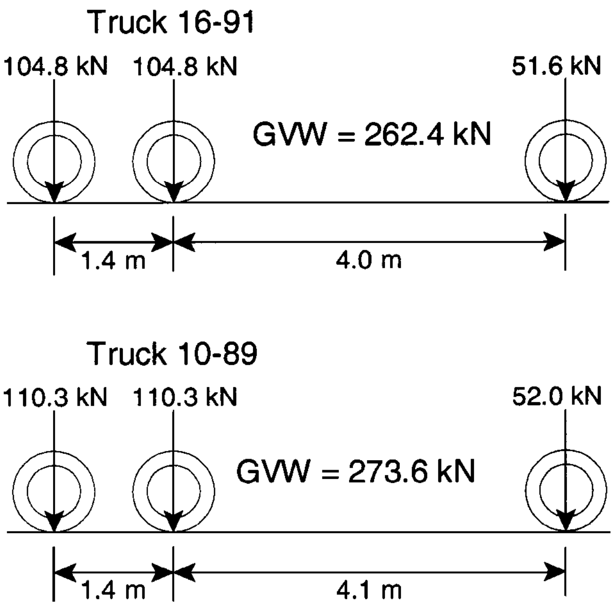

Figure 15—Truck configurations and axle loads for load test 1. For both trucks, vehicle track, measured center-to-center of rear tires, was 1.8 m (6 ft) wide.

Load Test 1

The first load test was conducted on June 27, 1994. The test vehicles were two three-axle dump trucks: truck 16–91 with a gross vehicle weight of 262.4 kN (59,000 lb) and truck 10–89 with a gross vehicle weight of 273.6 kN (61,500 lb) (Fig. 15). Measurement points were positioned transversely along the bridge midspan at the centerline of each glulam web.

Load Test 2

The second load test was conducted on May 1, 1997. As for load test 1, the test vehicles were three-axle dump trucks:

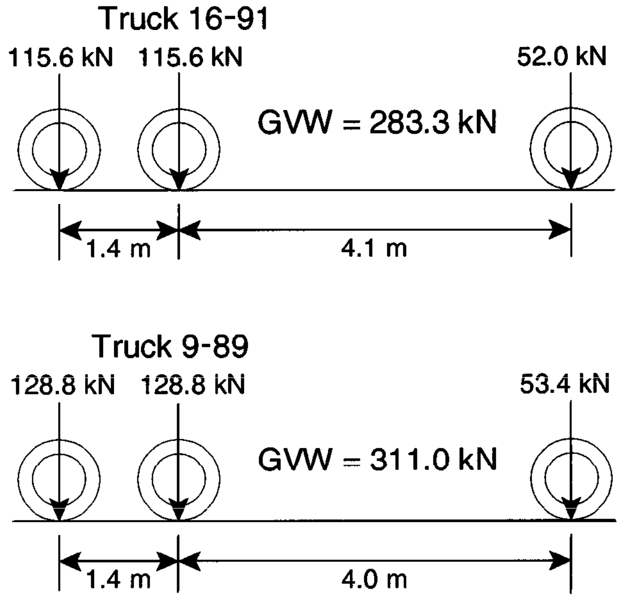

Figure 16—Truck configurations and axle loads for load test 2. For both trucks, vehicle track, measured center-to-center of rear tires, was 1.8 m (6 ft) wide.

truck 16–91 with a gross vehicle weight of 283.3 kN (63,680 lb) and truck 9–89 with a gross vehicle weight of 11.0 kN (69,920 lb) (Fig. 16). Measurement points were the same as those used for load test 1.

Predicted Deflection Analysis

At the conclusion of the load testing, predicted deflections were calculated for AASHTO HS25–44 loading. Design procedures and analytical models for stress-laminated T-beam bridges are currently under development; therefore, a simplified procedure using measured load test deflections and a ratio of deflection coefficients was used. The procedure was based on a deflection coefficient (DC) determined through computer analysis (Murphy 1994) and the following relationship:

$$\Delta_{\mathrm{HS25}} = \Delta_{\mathrm{Load\,test}} \left(\frac{\mathrm{DC}_{\mathrm{HS25}}}{\mathrm{DC}_{\mathrm{Load\,test}}} \right)$$

where

Δ_{HS25} is HS25 predicted deflection (mm),

$\Delta_{\mathrm{Load\ test}}$ maximum measured load test deflection (mm),

$\mathrm{DC}_{\mathrm{HS25}}$ HS25 deflection coefficient (kN·m^4), and

$\mathrm{DC}_{\mathrm{Load\ test}}$ load test vehicle deflection coefficient (kN·m^4).

Condition Assessment

The general condition of the bridge was assessed on four occasions during the monitoring period. Staff from FPL performed condition assessments during site visits. The assessments involved visual inspections, measurements, and photographic documentation of the condition of the bridge. Items of specific interest were geometry, wood condition, wearing surface, and stressing system.

Results and Discussion

The performance of the Humphrey bridge was monitored for 35 months, beginning in June 1994. Results and discussion of the performance data follow.

Moisture Content

Average electrical resistance moisture content readings are shown in Figure 17. As shown, the readings for the glulam webs, sawn lumber flanges, and sawn lumber diaphragms showed slight variations. Moisture content of the glulam webs was consistently higher than that of the diaphragms or flanges. Glulam moisture content was approximately 21% initially, decreased approximately 5% from July 1994 to August 1996, and then increased to approximately 17% at the end of the monitoring period. Moisture content of the sawn lumber components (flanges and diaphragms) behaved in a similar manner to that of the glulam webs. At the start of monitoring, flange moisture content was approximately 15%. Approximately 1 year later, flange moisture content had increased to approximately 16%. At this time, readings were taken of the sawn lumber diaphragms, and moisture content was found to be approximately 14%. As for the glulam webs, moisture content of the sawn lumber components decreased during the monitoring period but increased at the end of the period (to approximately 18%). The small variation in moisture content indicates that the materials were near the equilibrium moisture content for the environmental conditions. The consistency of the readings during the monitoring period reflects the effect of oil preservative treatment on moisture content fluctuations (Kainz and others 1996).

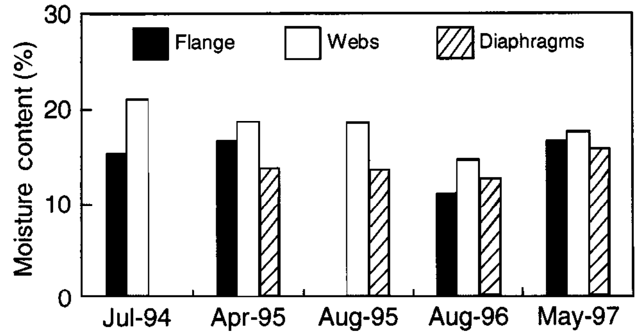

Figure 17—Average trend in electrical resistance moisture content readings.

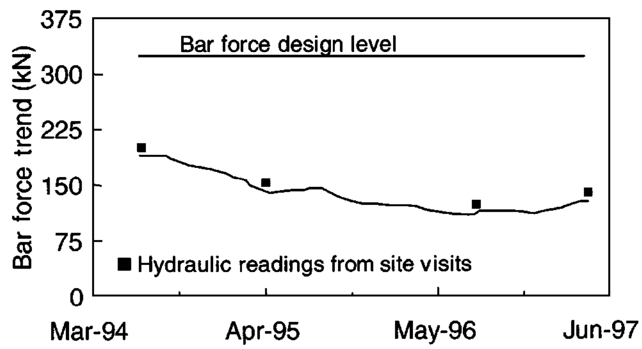

Figure 18—Average bar force trend approximately 8 months after the third bar tensioning.

Bar Force

The average trend for bar force is shown in Figure 18. Bar force data were obtained from the load cells on the bars and verified with hydraulic pump and readings during four site visits. At the start of monitoring (approximately 8 months after bridge installation), tension in all bars was approximately 190 kN (42,700 lb), which corresponds to 1.02 MPa (148 lb/in^2). Bar force decreased gradually during the monitoring period, to a final value of 150 kN (33,700 lb) or 806 kPa (117 lb/in^2).

Figure 18 does not reflect losses from the initial bar force of 320 kN (72,000 lb). Bar force losses throughout the monitoring period were probably the result of stress relaxation. Stress relaxation is a time-dependent phenomenon resulting from constant force on the wood laminations. Even with the losses in bar force, interlaminar compression in the Humphrey bridge is still well above the minimum level of 345 kPa (50 lb/in^2). The large reserve in bar force and interlaminar compression is due to a design bar force that is approximately 2.5 times the recommended value for stress-laminated timber bridge decks (Ritter 1990). The higher design bar force allows a larger amount of bar force to be lost over time while maintaining an acceptable amount of bar force in the bridge. However, additional research is needed to determine the minimum amount of bar force needed in a stress-laminated T-beam bridge to maintain its structural capacity.

Load Test Behavior

In this section, results of the static load tests and analytical assessment of the Humphrey bridge are presented. For each load position, transverse deflection measurements are given at the mid- span as viewed from the west end (looking east). No permanent residual deformation was measured at the conclusion of load testing, and there was no detectable

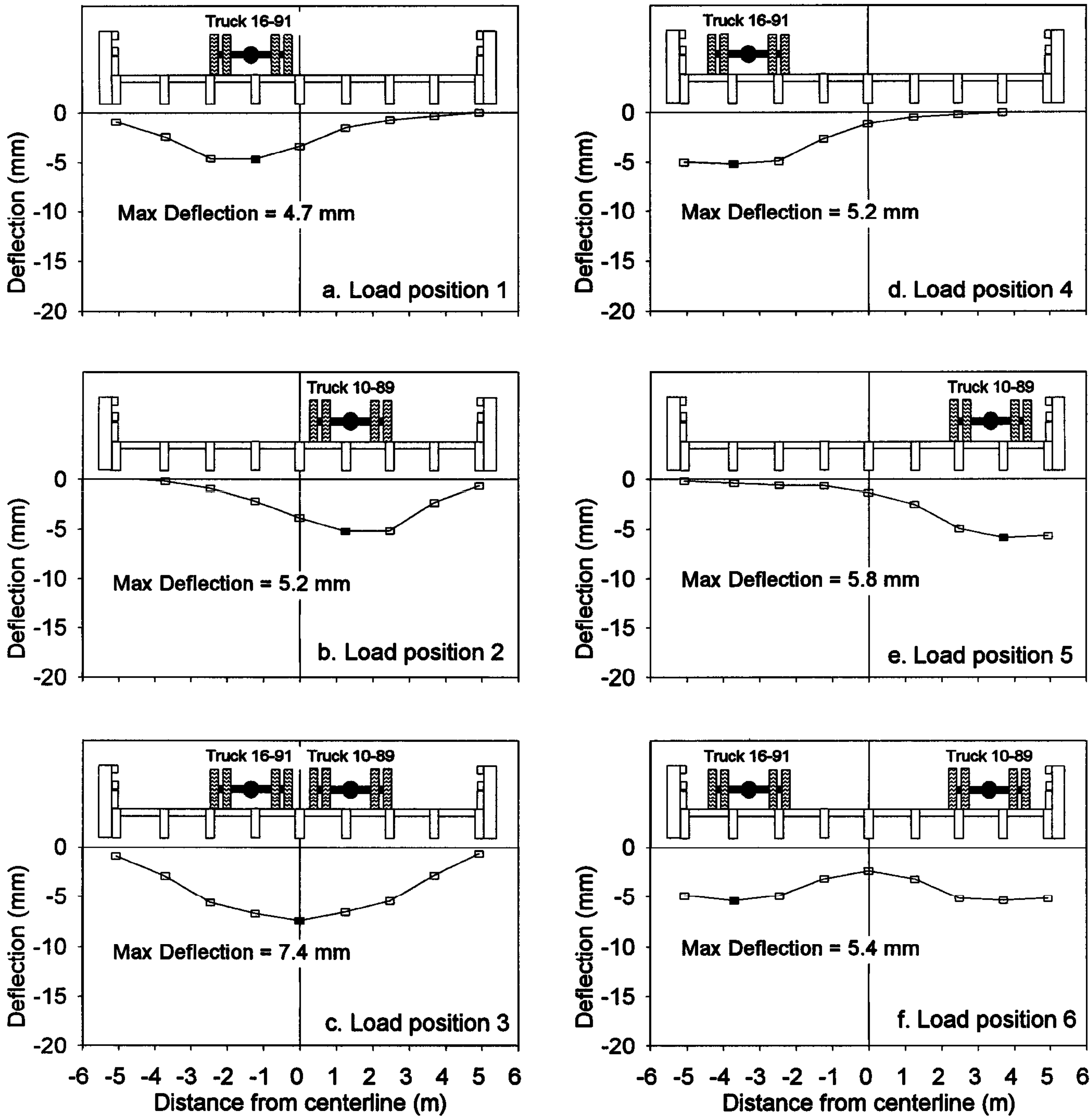

Figure 19—Transverse deflections measured at mid-span (looking east) for load test 1. Bridge cross sections and vehicle positions are presented for the purpose of interpretation only and are not drawn to scale.

movement at bridge supports. At the time of load test 1, the average bridge prestress was approximately 1.02 MPa (148 lb/in²). For load test 2, the bridge prestress was approximately 806 kPa (117 lb/in²).

Load Test 1

The maximum measured transverse deflections for load test are shown in Figure 19. The maximum measured deflections occurred near the wheel lines of the test vehicles.

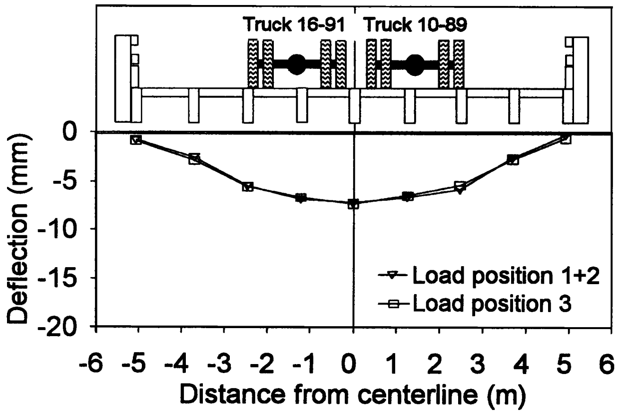

Figure 20—Comparison of measured deflections for load test 1: actual deflection of load position 3 and sum of load positions 1 and 2 (looking east).

These deflections were symmetrical between similar load positions such as load positions 1 and 2 and load positions 4 and 5. For the load positions with two vehicles, the deflection from load position 3 was larger than all other measured deflections; the smaller deflection from load position 6 was similar to that of load positions 1, 2, 4, and 5. This was due to the transverse stiffness of the deck created by interlaminar compression stress.

Assuming uniform material properties, symmetric loading, and accurate deflection measurements, the bridge deflections for load positions 1 and 2, when summed, should equal the deflections from load position 3. Figure 20 displays the summation of load positions 1 and 2 overlaid on load position 3. As shown, deflections are essentially the same.

Load Test 2

The maximum measured transverse deflections for load test 2 (Fig. 21) are shown in Figure 21. Maximum measured deflections, which occurred near the wheel lines of the test vehicles, were symmetrical between similar load positions, such as load positions 1 and 2 and load positions 4 and 5. For the load positions with two vehicles, the deflection from load position 3 was greater than that of all other measured deflections; the smaller deflection from load position 6 was similar to that of load positions 1, 2, 4, and 5. As in load test 1, this was due to the transverse stiffness of the deck created by interlaminar compression stress.

Figure 22 displays the summation of load positions 1 and 2 overlaid on load position 3 for load test 2. Aside from slight deflection variations, the deflections are essentially the same.

Load Test Comparison

Comparison of the results from load tests 1 and 2 revealed similar maximum deflections for each load position (Fig. 23). Because a slightly heavier load was used for load test 2, deflections from this load test were slightly greater than those from load test 1. The effect of load weight was especially evident for load positions 3, 5, and 6 because of the substantially heavier load of the right (upstream) truck (Fig. 23).

On most stress-laminated bridges with butt joints, there is a correlation between the transverse stiffness of the bridge and the level of interlaminar compression stress. Wacker and others (1996) observed that the transverse stiffness of several stress-laminated deck bridges decreased with a decrease in the level of interlaminar compression in the deck. The change in interlaminar compression seemed to have little effect on the transverse stiffness of the Humphrey bridge.

Predicted Deflection Analysis

To compare the Humphrey bridge with other bridges, a theoretical deflection based on a standard HS25–44 truck was determined. Using the method previously described, maximum deflections of 9.9 mm (0.39 in.) for load test 1 and 9.3 mm (0.37 in.) for load test 2 were determined. These deflections correspond to L/1449 for load test 1 and L/1528 for load test 2 and are significantly less than the minimum design deflection criterion of L/500.

Condition Assessment

Condition assessment of the Humphrey bridge indicated good structural and serviceability performance. The inspection included bridge geometry, wood condition, preservative treatment, wearing surface, and anchorage system.

Bridge Geometry

Measurements obtained during site inspections revealed changes in bridge geometry over the monitoring period. One change is that the bridge has become narrower at mid-span than at the abutments. This behavior, commonly called "hour glassing," is caused by compressive deformation (creep) at mid-span and consequent decrease in the size of the laminates. Creep occurs because the attachment of the webs to the abutments makes the bridge resistant to movement. Hour glassing does not affect the structural performance of the bridge but it can have an adverse aesthetic effect, if severe. Hour glassing on the Humphrey bridge is minor and generally not noticeable.

Another change is that the exterior beams are not vertical at mid-span (Fig. 24). The beams seem to be "toeing-in" as a result of the tensioned diaphragm bars pulling the beams together at the one-third points and the deck resisting the movement. The sawn lumber area was less resistant to the bar tension at the diaphragms than at the deck. Therefore, when all bars were tensioned to the same level, the diaphragms shrunk at a greater rate than did the deck, which created the "toe in." The structural performance of the bridge is not directly affected by the toe-in, but the railing system is no longer parallel because the posts are attached to the exterior beams. This detracts from the appearance of the bridge.

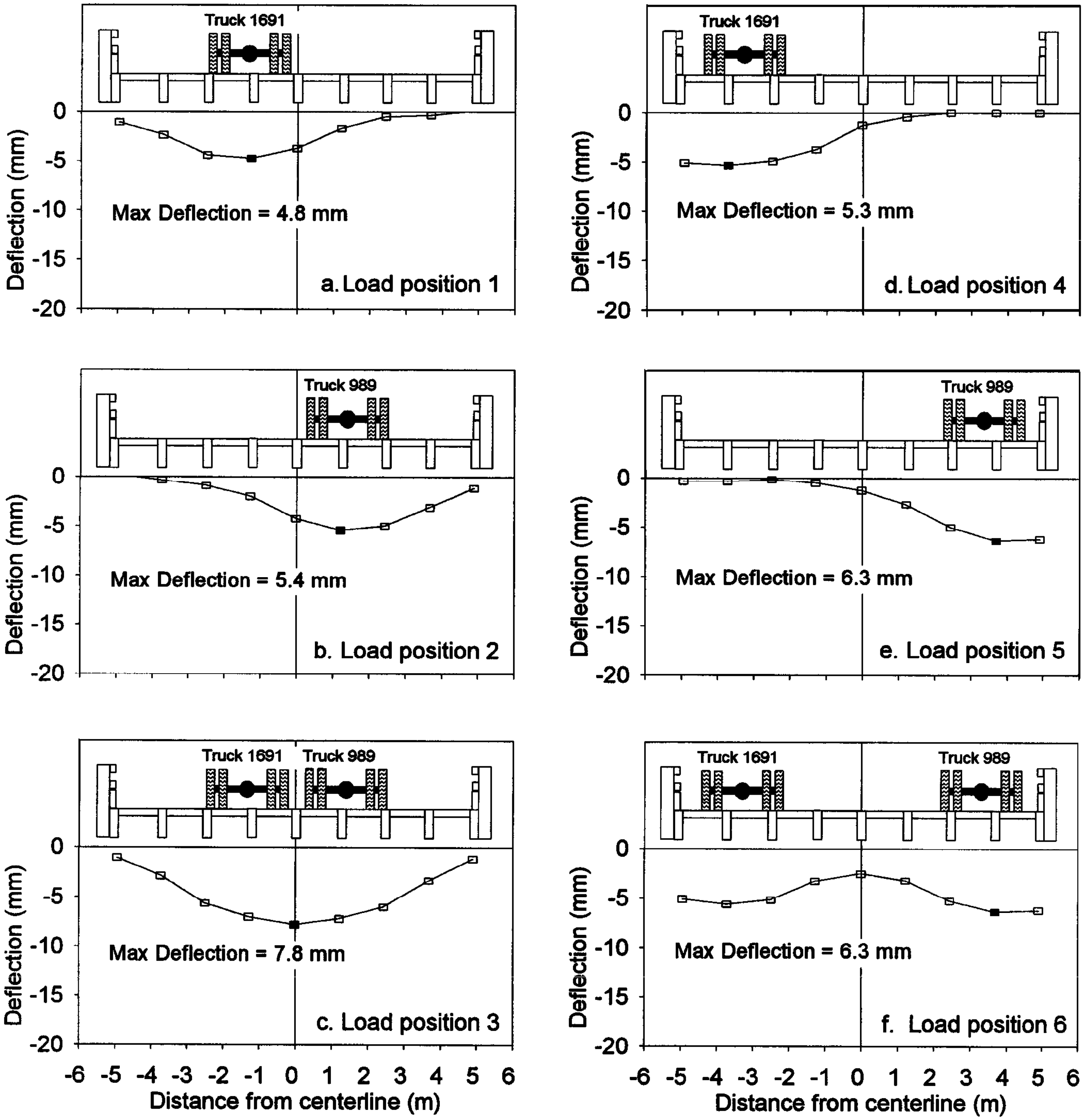

Figure 21—Transverse deflection measured at mid-span (looking east) for load test 2. Bridge cross sections and vehicle positions are presented for the purpose of interpretation only and are not drawn to scale.

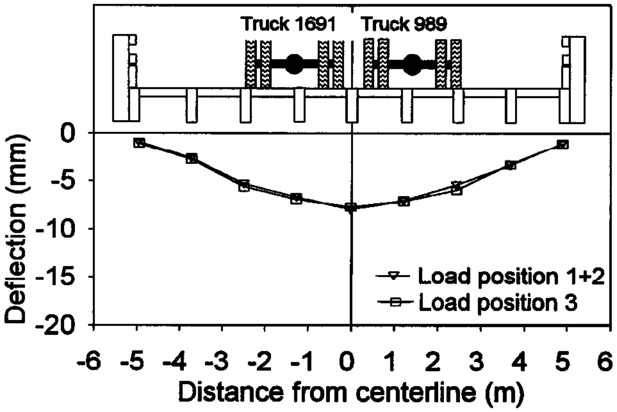

Figure 22—Comparison of measured deflections for load test 2: actual deflection of load position 3 and sum of load positions 1 and 2 (looking east).

Wood Condition

Wood components of the bridge showed no signs of deterioration. However, inspection of the exterior beams revealed cracks just beneath the steel bearing channel (Fig. 25). The cracking is a result of the toe-in of the exterior beams. The bottom of each beam is restrained at the abutment and the top is continuously restrained by the deck; therefore, any change in transverse dimension causes the beam to bend and twist in the Y–Y axis. The strength of a glulam beam is in the X–X axis. Therefore, to alleviate the stresses developed in the beams, cracking formed. If the cracks are allowed to propagate through the member, a substantial reduction in load capacity could result. The cracks also open the exposed exterior beams to potential decay, because the preservative may not have penetrated to the depth of the cracks.

In addition to beam cracks, minor checking was evident on the end grain of the rail posts exposed to rapid wet–dry cycles. This checking could have been alleviated through the use of a sealer on the end grain of the posts.

Preservative Treatment

Inspection indicated that creosote treatment may have been too heavy in some areas. Several areas on the concrete abutments were stained with creosote (Fig. 26), and excess creosote was present on the surfaces of the rail and posts. Leaching of creosote could have been eliminated by proper pressure preservation, including appropriate retention levels, post-treatment drying, and the use of reverse vacuum techniques (AWPA 1990).

Wearing Surface

The wearing surface is in good condition, with only minor reflective cracking at the abutments. Asphalt cracking at the abutments is commonly observed on timber bridges.

Anchorage System

The stressing bar anchorage system has performed adequately. No measurable distortion in the bearing channel 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 bearing channel into the glulam beams.

Conclusions

The Humphrey bridge is performing satisfactorily. Based on extensive bridge monitoring for 35 months, we make the following observations and recommendations:

- The glulam webs and sawn lumber flanges experienced small variations in moisture content during the monitoring period. Glulam and sawn lumber components of the Humphrey bridge were specified to be installed below 19% moisture content. When monitoring began 8 months after bridge installation, moisture content levels of the glulam and sawn lumber were 21% and 16%, respectively.
- Bar force decreased slightly throughout the monitoring period to a final value of approximately 150 kN (33,700 lb). It appears that the deck bar force reacted favorably to a relatively low moisture content level, and the small change in bar force occurred as a result of stress relaxation. Most loss in bar force occurred prior to the monitoring period.
- Load testing and analysis indicate that the Humphrey bridge is performing in a linear elastic manner when subjected to truck loading. The theoretical HS25–44 truck loading conditions produced maximum deflections of 9.9 mm (0.39 in.) for load test 1 and 9.3 mm (0.37 in.) for load test 2. These deflections correspond to L/1449 and L/1528 and are based on the center-to-center bearing lengths of the span. These deflections are smaller than the maximum allowed design deflection of L/500 for this span.
- The exterior beams are “toeing in” and cracking in response to the tensioning of the diaphragm bars. Over time, additional cracking may occur that could ultimately lead to a loss of section on these beams. We recommend that the bar force on the diaphragm bars be reduced or removed. To perform their function, diaphragms do not need to be restrained under compressive stress.
- Wood checking is evident in the exposed end grain of the bridge rail posts. This probably would not have occurred if a sealer had been placed over the end grain at the time of construction.
- There are no indications of corrosion on the galvanized steel stressing bars, hardware, or plates.
- The asphalt wearing surface is performing well, with only reflective cracking over the abutments. No signs of rutting or longitudinal cracking are present.

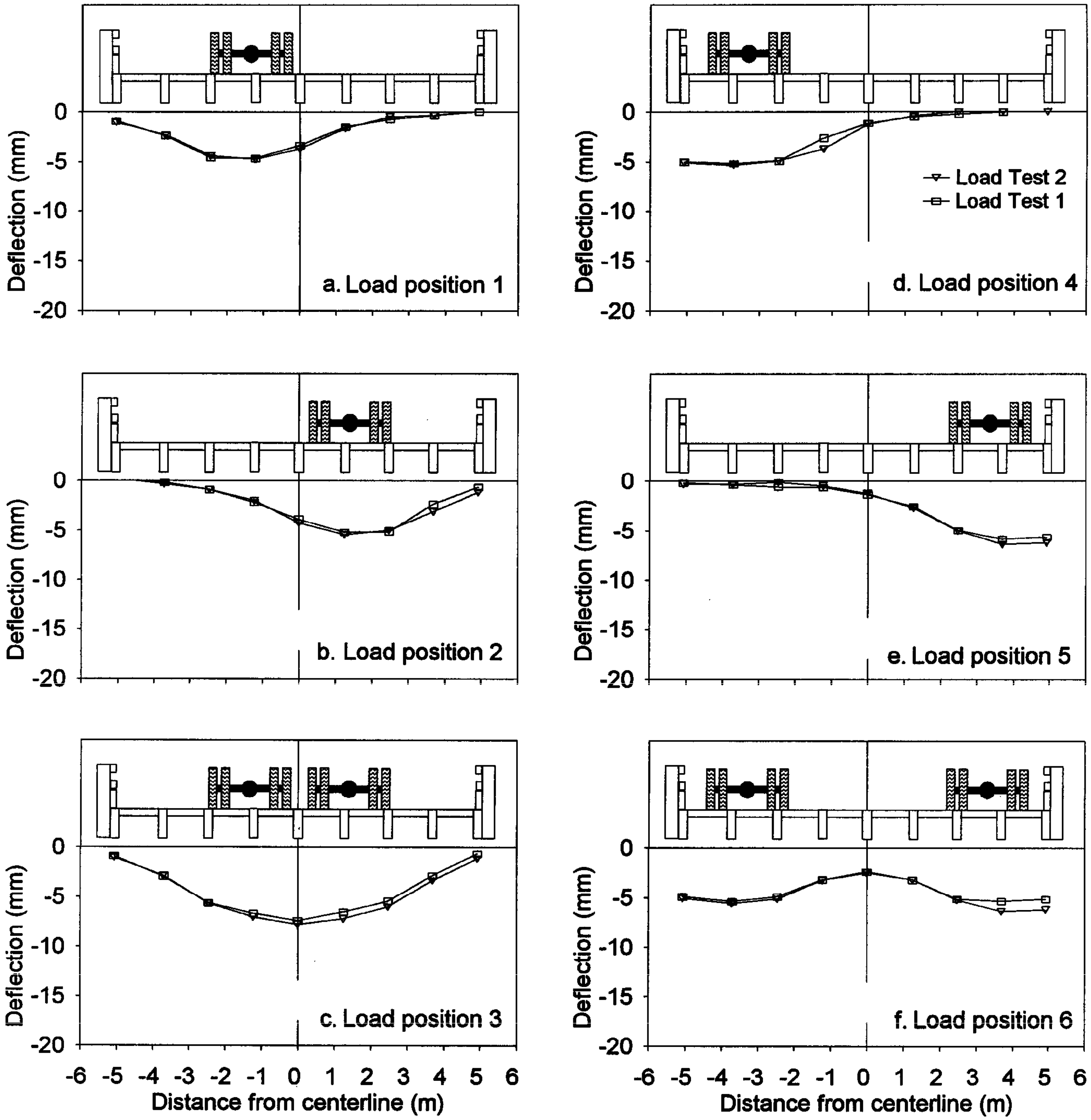

Figure 23—Comparison of load tests 1 and 2 at all load positions.

Figure 24—"Toe in" of exterior beams resulting from tensioned diaphragms.

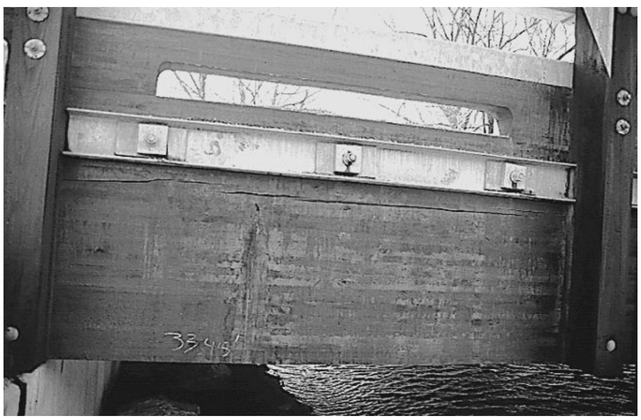

Figure 25—Cracking in exterior beams resulting from beam twisting.

Figure 26—Creosote staining of concrete abutments resulting from preservative leaching.

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Appendix—Information Sheet

General

Name: Humphrey bridge

Location: Cattaraugus County, New York

Date of construction: June 1993

Owner: Cattaraugus County Department of Public Works

Design Configuration

Number of spans: 1

Structure type: Stress-laminated ‘T’ with glulam webs and sawn lumber butt-jointed flanges

Butt-joint configuration: 1 in 4 transverse laminations; joints in adjacent deck flange laminations separated 0.9 m (3 ft) longitudinally

Total length (out–out): 14.8 m (48.6 ft)

Skew: None

Span length (out–out): 14.8 m (48.6 ft)

Span length (center-to-center bearings): 14.4 m (47.1 ft)

Bearing length: 305 mm (12 in.) with overhang

Width (out–out): 10.2 m (33.5 ft)

Width (curb–curb): 9.9 m (32.5 ft)

Number of traffic lanes: 2

Design loading: AASHTO HS25–44

Wearing surface type: Asphalt pavement, 50.8 to 63.5 mm (2 to 2½ in.) thick

Material and Configuration

Flange laminations

Material: Mixed hardwood sawn lumber

Size: Standard 51 by 203 mm (nominal 2 by 8 in.)

Grade: No. 2

Webs

Material: Southern Pine glulam

Size (actual): 311.2 by 1085.8 mm (12.25 by 42.75 in.)

Beam designation: 24F–V3 SP/SP

Rails

Material: Steel tube

Size (actual): 152.4 by 152.4 by 4.8 mm (6 by 6 by 0.1875 in.)

Posts

Material: Mixed hardwood sawn lumber

Size (actual): 171.5 by 266.7 mm (6.75 by 10.5 in.)

Curb and scupper

Material: Southern Pine glulam combination

Size (actual): 171.5 by 419.1 mm (6.75 by 16.5 in.)

Preservative treatment: Creosote

Stressing bars

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

Diameter: 25.4 mm (1 in.)

Number: 17

Design force: 320.3 kN (72,000 lb)

Spacing: 914 mm (36 in.) center-to-center, beginning 1,013 mm (39.875 in.) from span end

Anchorage type and configuration

Steel MC 8 by 18.7 channel

Anchor plate: 127 by 127 by 31.8 mm (5 by 5 by 1.25 in.); hexagonal bell nut

Bearing plate: 304.8 by 152.4 by 12.7 mm (12 by 6 by 0.5 in.)