Field Performance of Timber Bridges

12. Christian Hollow Stress-Laminated Box-Beam Bridge

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Abstract

In January 1992, the Christian Hollow bridge was constructed in Steuben County, New York. The bridge is a single-span, stress-laminated box-beam superstructure that is 9.1 m long, 9.8 m wide, and 502 mm deep (30 ft long, 32 ft wide, and 19-3/4 in. deep). The performance of the bridge was continuously monitored for 28 months, beginning shortly after installation. Performance monitoring involved gathering and analyzing data relative to the wood moisture content, force level in the stressing bars, vertical bridge creep, and behavior under static load conditions. In addition, comprehensive visual inspections were conducted to assess the condition of the overall structure. Based on field evaluations, the bridge is performing well with no structural or serviceability deficiencies.

Keywords: bridge, stress laminated, box beam, field performance

Acknowledgments

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Introduction

In 1988, the U.S. Congress passed legislation known as the Timber Bridge Initiative (TBI). The objective of 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 the TBI was assigned to the USDA Forest Service. To implement a program, the Forest Service established three primary emphasis areas: demonstration bridges, technology transfer, and research. Responsibility for the technology transfer and demonstration bridge programs was assigned to the Forest Service National Wood in Transportation Information Center (NWITIC) (formerly the Timber Bridge Information Resource Center) in Morgantown, West Virginia. Under the demonstration program, the NWITIC provides matching funds to local governments for constructing demonstration timber bridges that encourage innovation through the use of new or previously underutilized wood products, bridge designs, and design applications.

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. As part of this broad research program, FPL assumed a lead role in assisting local governments in evaluating the field performance of demonstration timber bridges, many of which use design innovations or materials that have not been previously evaluated. Through such assistance, FPL is able to collect, analyze, and distribute information on the field performance of timber bridges. This provides a basis for validating or revising design criteria and further improves the efficiency and economy in bridge design, fabrication, and construction.

In addition to the TBI, Congress passed the Intermodal Surface Transportation Efficiency Act (ISTEA) in 1991, which included provisions for a timber bridge 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) and included a demonstration timber bridge, technology transfer, and research program. Because many aspects of the FHWA 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, twelfth in a series documenting the field performance of timber bridges, describes the design, construction, cost, and field performance of a 28-month monitoring project of the Christian Hollow bridge, which is located in Steuben County, New York. The bridge is a two-lane, single-span, stress-laminated box-beam superstructure 9.1 m long, 9.8 m wide, and 502 mm deep (30 ft long, 32 ft wide, and 19-3/4 in. deep). Built in 1992, the Christian Hollow bridge was constructed with matching funds through the Forest Service NWITIC and monitored as a part of the FPL/FHWA National bridge monitoring program. An information sheet on the specific characteristics of the Christian Hollow bridge is given in the Appendix.

Background and Development

The Christian Hollow bridge is located in the town of Greenwood, Steuben County, New York (Fig. 1). The bridge is on Christian Hollow–West Union road (County Highway 60) where it crosses Christian Hollow Creek, approximately 1.2 km (0.75 miles) west of State Highway 248. The roadway over the bridge is a two-lane paved road; traffic is mostly passenger vehicles at an average rate of 500 per day.
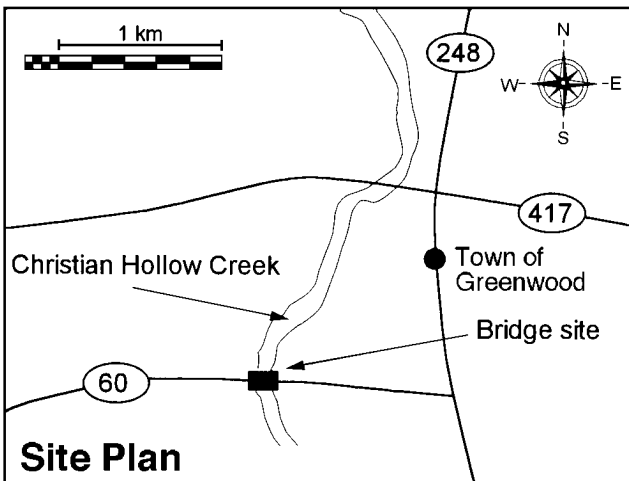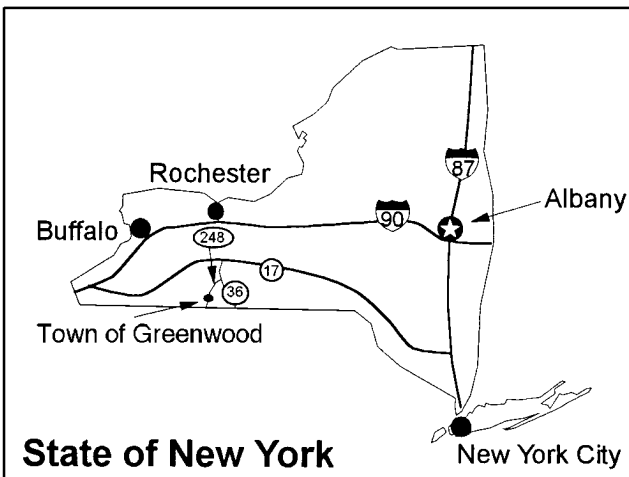
Figure 1—Location maps for the Christian Hollow bridge.

Figure 2—Original bridge constructed in the 1950s.

The original Christian Hollow bridge, constructed in the 1950s, consisted of steel stringers supporting a concrete slab deck with tubular steel guardrails (Fig. 2). The girders and deck were in poor condition, and the steel tubing guardrails were inadequate and posed significant safety hazards to motorists. Because of an interest by the Steuben County staff to use innovative timber bridge designs, a decision was made to replace the existing structure with the newly developed stress-laminated box-beam timber superstructure.

Through a cooperative effort between the Sullivan Trail Resource, Conservation, and Development Council and the Steuben County Department of Public Works, a proposal was submitted to the USDA Forest Service for partial funding of the replacement structure as a demonstration bridge under the TBI (USDA 1990). The project proposed a stress-laminated box-beam configuration utilizing local mixed hardwood lumber and Southern Pine glued-laminated (glulam) timber beams. In 1990, the project received funding through the NWITIC, and plans for the design and construction of the Christian Hollow bridge were finalized. Subsequently, FPL was contacted to provide assistance in developing and implementing a field evaluation plan to monitor bridge performance.

Objective and Scope

The objective of this project was to evaluate the field performance of the Christian Hollow bridge for 28 months after installation. The scope of the project included data collection and analysis related to wood moisture content, stressing bar force, vertical bridge creep, static truck loading, and general structure performance. The results of this project will be considered with similar monitoring projects in an effort to improve design and construction methods for future stress-laminated box-beam timber bridges.

Design, Construction, and Cost

Design and construction of the Christian Hollow bridge involved mutual efforts from several agencies and individuals. An overview of the design, construction, and cost of the project follows.

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Figure 3—Cross section of a typical stress-laminated box-beam bridge.

Design

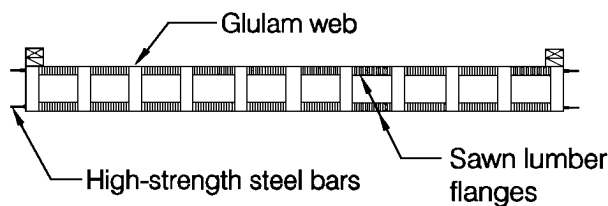Design of the Christian Hollow bridge was completed by the Steuben County staff with assistance from an engineering consultant. The design features a stress-laminated box-beam superstructure, a relatively new type of timber bridge, with continuous glulam webs and sawn lumber flanges (Fig. 3). For this bridge configuration, high strength steel bars are inserted through prebored holes in the webs and flanges and tensioned to provide sufficient friction between the individual components to develop load transfer. Thus, the design is predicated upon the components acting together as a single unit.

With the exception of those features related to stress laminating, design of the Christian Hollow bridge conformed to the American Association of State Highway and Transportation Officials (AASHTO) *Standard Specifications for Highway Bridges* (AASHTO 1989) for two lanes of HS20-44 loading. At the time of design, an AASHTO-accepted design procedure for stress-laminated box-beam bridges was not available. Therefore, specific design requirements for the stress-laminated box-beam were based on standard guidelines developed from research conducted at West Virginia University (Lopez–Anido and GangaRao 1993).

The design geometry provided for a single-span superstructure, 9.1 m long, 9.8 m wide, and 502 mm deep (30 ft long, 32 ft wide, and 19-3/4 in. deep) (Fig. 4). Design calculations were based on an 8.8-m (29-ft) span length (center-to-center of bearings) and a 9.1-m (30-ft) clear roadway width. Design of the glulam webs and the sawn lumber flanges was based on requirements set by the American Forest and Paper Association (AFPA 1986, 1988) and the American Wood Preservers' Association (AWPA 1989). Web members were creosote-treated combination 24F-V3 Southern Pine glulam beams and were 9.1 m long, 219 mm wide, and 502 mm deep (30 ft long, 8-5/8 in. wide, and 19-3/4 in. deep). Sawn lumber flange laminations were specified as standard 38- by 140- mm (nominal 2- by 6-in.) visually graded No. 2 mixed hardwoods, pressure treated with creosote. Moisture content of both the web and flange members was specified to be 19 percent (maximum) at installation. Because the flange laminations were not available in lengths required to span the entire bridge, the design included lamination butt joints in a repetitive pattern (Fig. 5). Transversely, butt joints were four laminations apart. Longitudinally, butt joints in adjacent laminations were 0.9 m (3 ft) apart.

For stress laminating, the design specified a 16-mm- (5/8-in.-) diameter ASTM A722 high strength threaded steel bars with an ultimate strength of 1,034 kPa (150 lb/in^2) (ASTM 1988). The bars were spaced at 0.9-m (3-ft) intervals along the bridge except for the end bars. A design bar tension force of 96 kN (21,600 lb) provided an interlaminar compressive stress of 689 kPa (100 lb/in^2). Bar anchorage was with a discrete-plate anchorage system consisting of 152- by 305- by 19-mm (6- by 12- by 3/4-in.) steel bearing plates, 51- by 127- by 25-mm (2- by 5- by 1-in.) steel anchor plates, and hexagonal nuts. All components of the stressing system and other steel hardware were galvanized for corrosion protection.

Design of the railing and curb was based on a crash-tested railing developed for longitudinal spike-laminated timber decks in accordance with AASHTO Performance Level 1 criteria (FHWA 1990). The bridge rail was specified as a full-span glulam beam measuring 152 by 273 mm (6 by 10-3/4 in.) Rail posts were visually graded Dense Select Structural Douglas Fir sawn lumber measuring standard 190 by 292 mm (nominal 8 by 12 in.) and were spaced 1.8 m (6 ft) on-center. The curbs were visually graded No. 2 Douglas Fir sawn lumber measuring standard 139 by 292 mm (nominal 6 by 12 in.).

To compensate for dead load and creep deflection, a 25-mm (1-in.) camber was specified for glulam beams. To protect the bridge deck, a 76-mm- (3-in.-) thick asphalt wearing surface and a waterproof geotextile membrane were specified.

Construction

Construction of the Christian Hollow bridge was completed by the Steuben County bridge crew in 2 months, beginning January 1992. During construction, a temporary crossing consisting of corrugated steel culverts was installed just upstream of the bridge site. Following demolition and removal of the existing bridge structure, new reinforced concrete abutments and wingwalls were constructed (Fig. 6). To increase hydraulic flow capacity at the crossing, the span distance between abutments was increased and the abutments were elevated.

It took the Steuben County crew 1 week for in-place assembly of the stress-laminated box-beam superstructure. In-place assembly is not a popular method of construction for stress-laminated box-beam bridges, because it is difficult to assemble the butt-jointed flanges in the field. Stress-laminated box-beam bridges are typically prefabricated into partial width bridge modules that are temporarily nailed together and transported to the bridge site for placement with an overhead crane. However, in-place assembly is advantageous when low-cost labor is readily available.

Bridge components arrived at the bridge site on flatbed trailers (Fig. 7) and were stockpiled adjacent to the crossing. To begin the installation process, the glulam beams were lifted with the backhoe and set to span the abutments at the required center-to-center spacing (Fig. 8 top). After all glulam beams were in place, temporary support boards were attached to the underside of the beams to facilitate the

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16 mm (5/8 in.) ø high-strength steel bar (top and bottom)

Asphalt
wearing
surface

51 by 152 mm
(2 by 6 in.) mixed
hardwood flange
laminations

Southern Pine
glulam web

Plan

203 by 305 mm (8 by 12 in.)
sawn lumber post

152 by 273 mm
(6 x 10-3/4 in.) glulam rail

Concrete abutment

9.1 m (30 ft)

Profile

152 mm (6 in.) nominal flanges

502 mm (19-3/4 in.) glulam webs

9.8 m (32 ft)

Section
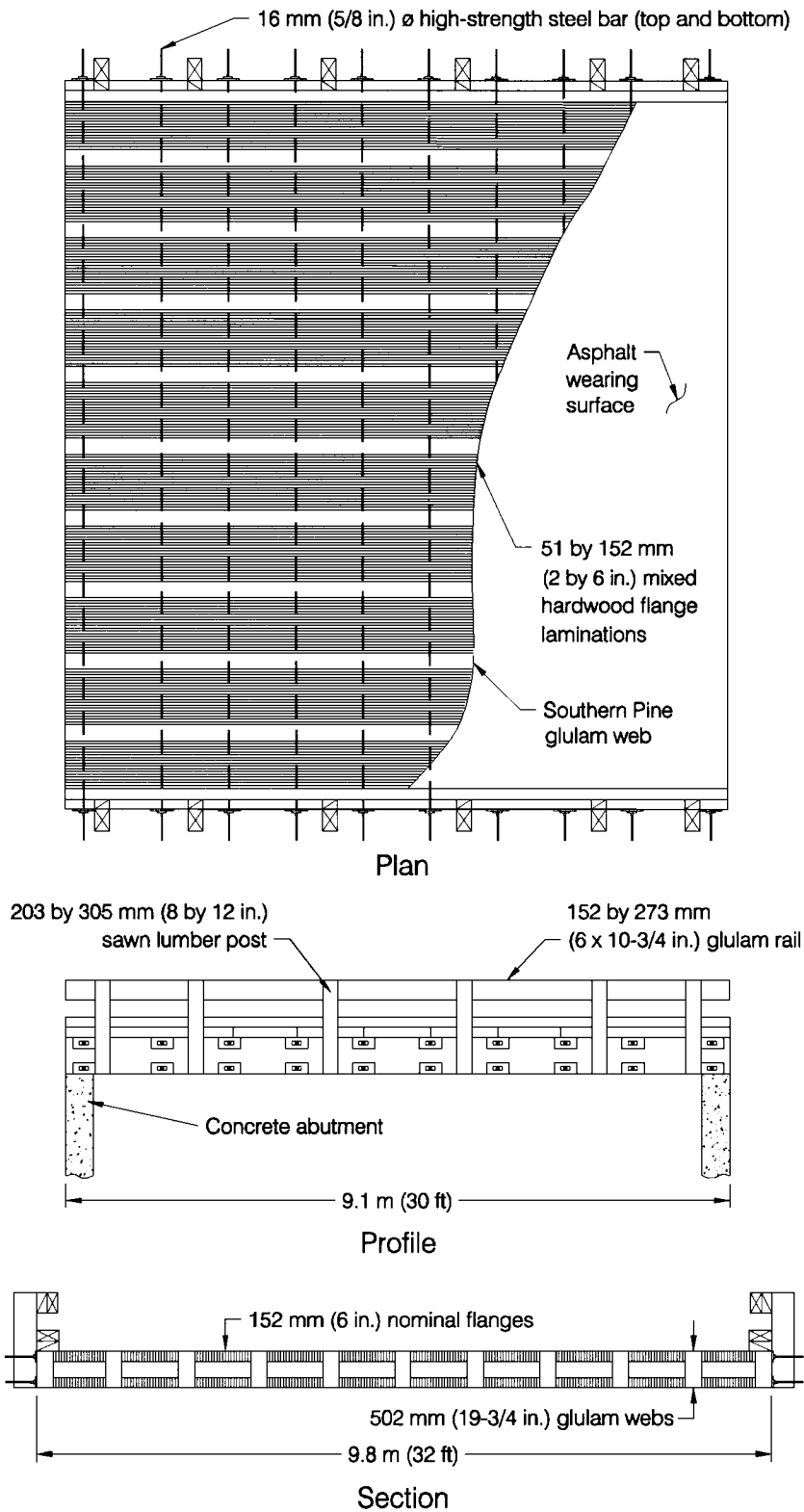
Figure 4—Design configuration of the Christian Hollow bridge.

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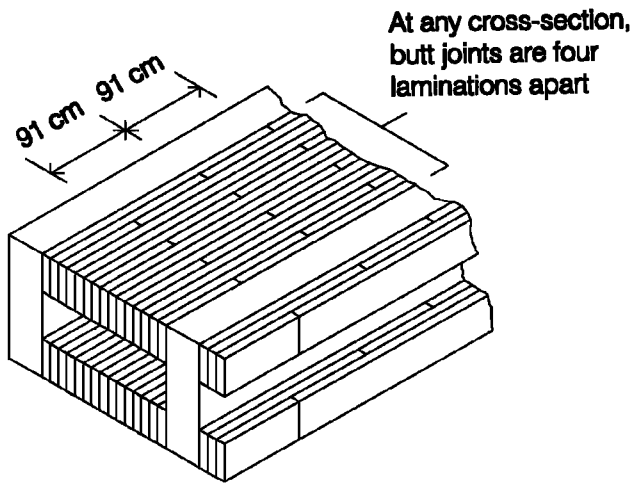
Figure 5—Repetitive butt-joint pattern used for the sawn lumber flange laminations.

Figure 6—Construction of reinforced concrete abutments.

Figure 7—Delivery of timber components to the bridge site with tractor trailers.

Figure 8—On-site assembly of the superstructure consisted of (top) placement of the glulam beam web members with a backhoe and nylon straps, (middle) placement of the lower flange lumber laminations by hand with standard 38- by 89-mm boards providing temporary support at the butt-joint locations, (bottom) insertion of the solid-block diaphragms at the ends of the bridge as the upper and lower flanges were completed.

Figure 9—Full-width stressing bars were inserted into the upper and lower flanges by hand (top). When poor hole alignment prevented insertion by hand, a portable hand drill was used (bottom).

placement of the sawn lumber decking. The use of scaffolding would have improved efficiency. This involves providing temporary support for the bottom flange laminations and creating a work platform. The sawn lumber decking was then hand-placed at the flange locations, beginning at the bottom layer (Fig. 8 middle). After the top and bottom flange layers were placed, solid-block diaphragms were inserted between the glulam beams at the bridge ends (Fig. 8 bottom). Stressing bars were manually inserted through the webs and flanges (Fig. 9). In some cases, the bars did not easily fit through the prebored holes and a power hand-drill was attached to the bars to turn them into place. After all bars were threaded through the bridge, anchorage plates and nuts were attached to the bar ends in preparation for the initial stressing.

The bridge was stressed initially at installation and at 1 and 10 weeks thereafter. At the initial stressing, the bars were individually tensioned, top to bottom, to the full design force level in a sequential manner, beginning at one end of the bridge. Subsequent stressings, completed at 1 and 10 weeks after installation, were performed to the full design force level in the same manner.

The box-beam superstructure was anchored to the concrete abutment with 152- by 152- by 5-mm (6- by 6- by 3/16-in.)

steel angle brackets attached to the underside of the beams and the abutment wall. Because anchor holes were drilled at installation and the superstructure became slightly narrower after bar tensionings, hole alignment problems were encountered and only half the anchor bolts could be installed. These alignment problems could have been avoided if hole drilling had been delayed until after the second stressing, after which little transverse movement occurred.

The curb and rail system was attached after 10 days, and the asphalt wearing surface was applied 2 months after installation. The completed Christian Hollow bridge is shown in Figure 10.

Cost

Cost for the design, fabrication, and construction of the Christian Hollow bridge superstructure totaled $63,846. Based on a total deck area of 89 m² (960 ft²), the unit cost was approximately $717/m² ($67/ft²).

Evaluation Methodology

To evaluate the structural performance of the Christian Hollow bridge, the Steuben County Department of Public Works contacted FPL for assistance. Through mutual agreement, a 28-month bridge monitoring plan was developed by the FPL and implemented through a Cooperative Research and Development Agreement with Steuben County. The plan called for performance monitoring of the deck moisture content, stressing bar force, vertical bridge creep, static load behavior, and general bridge condition. The plan evaluation methodology utilized procedures and equipment previously developed (Ritter and others 1991) and used on similar structures. The monitoring period was initiated at the final design stressing, approximately 12 weeks after bridge installation.

Moisture Content

Wood moisture content was measured with an electrical-resistance moisture meter and 76-mm (3-in.) insulated probe pins in accordance with ASTM D4444-84 (ASTM 1990). Measurements were obtained from both the glulam webs and sawn lumber flange members at several locations on the underside of the superstructure by driving the pins to a depth of 51 mm (2 in.). Moisture content measurements were obtained by Steuben County personnel on approximately a monthly basis. Meter readings were adjusted with appropriate temperature and wood species correction factors (FORINTEK 1984).

Bar Force

Stressing bar force was measured with calibrated steel hollow-core load cells and a portable strain indicator. At the initiation of monitoring, four load cells were installed at two locations along the bridge by placing them on bars through the top and bottom flanges. Load cell measurements were obtained by Steuben County personnel on approximately a

Figure 10—Completed Christian Hollow bridge: (top) side view, looking Northwest; (bottom) end view, looking west.

monthly basis throughout the monitoring period. Load cell measurements were converted from units of strain to force based on laboratory calibrations. In addition, the accuracy of the load cells was validated with laboratory re-calibrations and hydraulic force measurements at the conclusion of monitoring.

Vertical Creep

Vertical bridge creep was monitored by measuring the superstructure camber with a surveying level and calibrated rules. Camber readings were taken at the initiation and conclusion of the monitoring period. Superstructure camber was measured as the elevation difference between the underside of the superstructure at the abutments and at the cross section of the centerspan.

Behavior Under Static Load

To determine the response of the bridge to highway truck loads, static load testing of the Christian Hollow bridge was conducted at the beginning and end of the monitoring period. In addition, predicted deflections were determined for each load test based on static analysis for HS20-44 loading. Load testing involved positioning fully loaded trucks on the bridge span and measuring the resulting deflections at a series of locations along the centerspan of the bridge and cross sections of the abutments. A surveying level was used to read deflection values from calibrated rules that were suspended from the glulam webs. Deflection measurements were obtained prior to testing (unloaded), after placement of the test trucks for each load case (loaded), and at the conclusion of testing (unloaded). Truck positioning for each load case is shown in Figure 11. For load cases 1 through 3, trucks were positioned transversely 0.6 m (2 ft) from the roadway centerline. For load cases 4 through 6, the trucks were positioned 2.1 m (7 ft) from the centerline of the roadway. For all load cases, the center of gravity of the truck was positioned at centerspan to maximize deflections.

Load Test 1

Load test 1 was conducted March 17, 1992, with two dump trucks (Fig. 11): truck A with a gross vehicle weight of 296 kN (66,500 lb) and truck B with a gross vehicle weight of 278 kN (62,400 lb). Deflections were measured with a precision of 1.5 mm (1/16 in.).

Load Test 2

Load test 2 was conducted June 28, 1994, with two dump trucks (Fig. 11): truck A with a gross vehicle weight of 309 kN (69,300 lb) and truck B with a gross vehicle weight of 292 kN (65,600 lb). Deflections were measured with a precision of 1.0 mm (1/25 in.). Load cases 3 and 6 are shown in Figure 12.

Predicted Deflection Analysis

At the conclusion of load testing, predicted deflections were calculated based on actual load test conditions and AASHTO

HS20-44 loading. Design procedures and analytical models for stress-laminated, box-beam bridges are currently under development; therefore, a simplified procedure based on static analysis and a linear extrapolation to AASHTO HS20-44 deflections was used. The procedure utilizes a deflection coefficient (DC), a term used in calculating deflection independent of material properties, in the following relationship:

$$\Delta_{\text{HS20}} = \Delta_{\text{Loadtest}}\left(\frac{DC_{\text{HS20}}}{DC_{\text{Loadtest}}}\right) \qquad (1)$$

where

Δ_{HS20} = HS20 predicted deflection (mm);

Δ_{Loadtest} = maximum measured load test deflection (mm);

DC_{HS20} = HS20 deflection coefficient; and

DC_{HS20} = load test vehicle deflection coefficient.

Condition Assessment

The general condition of the Christian Hollow bridge was assessed at initiation and conclusion of the monitoring period. In addition, a subsequent assessment was performed August 1995, 14 months after the conclusion of monitoring. These assessments involved visual inspections, measurements, and photographic documentation of the condition of the bridge. Items of specific interest included bridge geometry, wood components, wearing surface, stressing bar anchorages, and steel hardware.

Results and Discussion

Performance of the Christian Hollow bridge was continuously monitored for 28 months, beginning March 1992 and ending June 1994. The following results are presented.

Moisture Content

The average trend in moisture content is presented in Figure 13. At the initiation of monitoring, the average moisture content was approximately 20 percent for the glulam webs and 26 percent for the sawn lumber flange laminations. Minor fluctuations within the range of ±3 percent occurred during the monitoring period, but the moisture content remained relatively stable. At the conclusion of monitoring, the moisture content of both the glulam beams and the sawn lumber flange laminations had decreased slightly.

The average trend indicates that no significant changes in moisture content occurred during the monitoring period. The glulam web moisture content is near the anticipated equilibrium level for this site. Although installed slightly greater than the specified maximum of 19 percent, the sawn lumber moisture content did not decrease in service as typically observed for most timber structures installed near fiber saturation. However, it is anticipated that the moisture content of the sawn lumber will gradually decrease and equilibrate with its surrounding environment during the next several years.
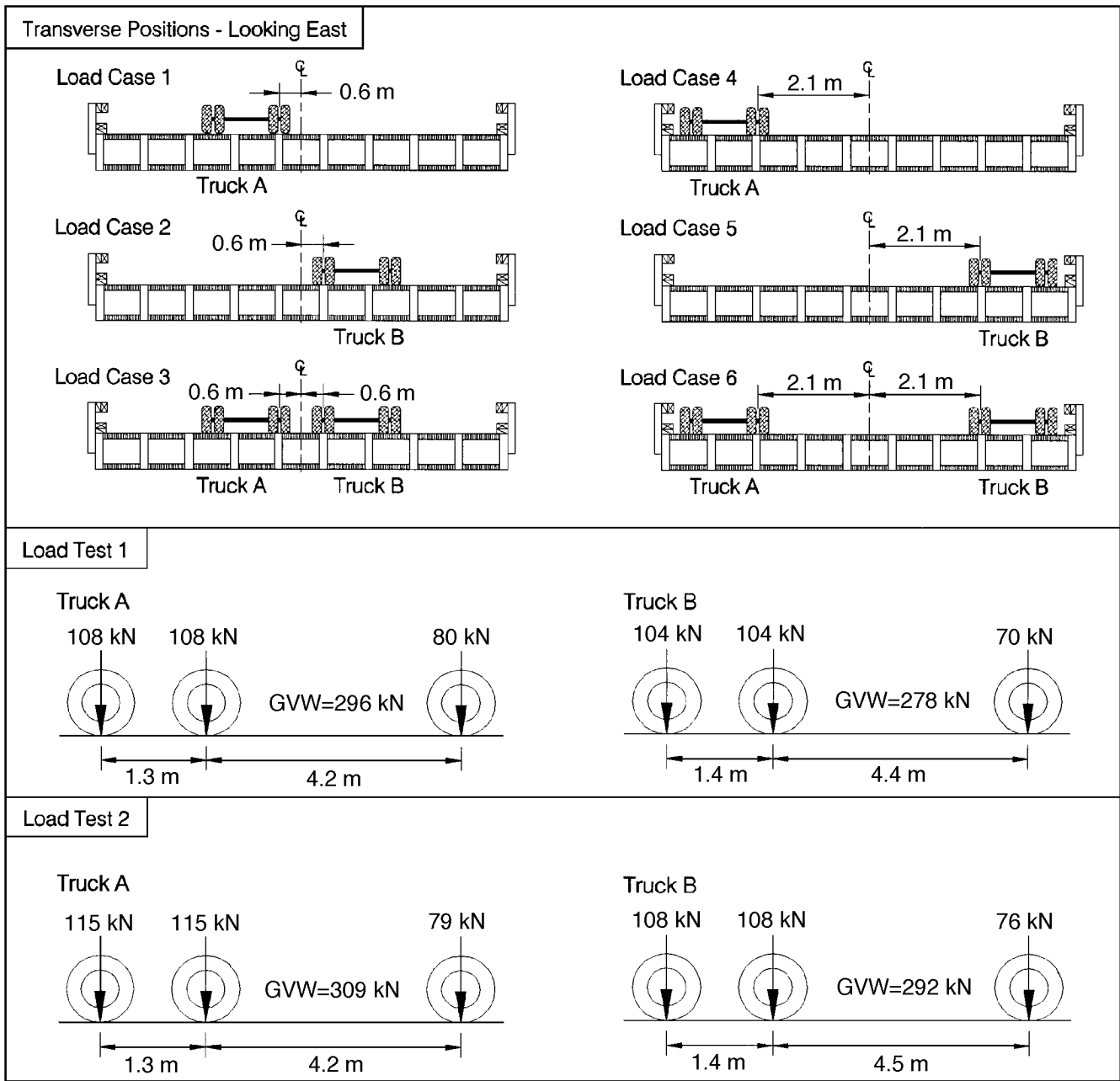
Figure 11—Truck configurations and transverse positioning used for static load testing. For all load cases, the truck center of gravity was placed at the centerspan of the bridge. (GVW is gross vehicle weight.)

Bar Force

The average trend in stressing bar force is presented in Figure 14. From the final design stressing in March 1992 through March 1993, the average bar force decreased from the design level of 96 kN (21,600 lb) to approximately 58 kN (13,000 lb), or approximately 414 kPa (60 lb/in^2) interlaminar compression. During the subsequent 6 months ending October 1993, the average bar force slightly increased but then decreased to 49 kN (11,000 lb), or approximately 345 kPa (50 lb/in^2) interlaminar compression. In October 1993, the bridge was restressed to the full design level of

96 kN (21,600 lb), because the potential for interlaminar slip between the web and flange increases as the interlaminar compression decreases below 345 kPa (50 lb/in^2). Between October 1993 and March 1994, the bar force decreased to 67 kN (15,000 lb), or approximately 483 kPa (70 lb/in^2) interlaminar compression, and became somewhat stabilized for the remainder of the monitoring period. Additional measurements taken in August 1995, 14 months after completion of monitoring, indicated an average bar force of 58 kN (13,000 lb), or approximately 414 kPa (60 lb/in^2) interlaminar compression.

Figure 12—Truck positions for (top) load case 3 and (bottom) load case 6.

Figure 13—Average trend in moisture content.

Figure 14—Average trend in stressing bar force.

Bar forces remained greater than 48 kN (10,800 lb), or 345 kPa (50 lb/in^2) interlaminar compressive stress, throughout the monitoring period, although one restressing was required in October 1993. The moisture content of the web and flange laminations was essentially constant during the monitoring period; therefore, lamination shrinkage resulting from decreasing moisture content did not substantially affect bar force. A large majority of bar force loss can be attributed to stress relaxation in the web and flange laminations. As observed in previous field evaluations (Ritter and others 1995), bar force losses as a result of stress relaxation may have been accelerated by the relatively high moisture content (24 to 26 percent) of the sawn lumber flange laminations. Future inspections of the Christian Hollow bridge should include hydraulic bar force measurements to ensure that the forces remain greater than approximately 48 kN (10,800 lb), or 345 kPa (50 lb/in^2) interlaminar compressive stress.

Vertical Creep

Camber measurements are summarized in Table 1. As previously stated, the box-beam superstructure had a 25-mm (1-in.) camber specified at installation. At the initiation of monitoring, approximately 12 weeks after installation,
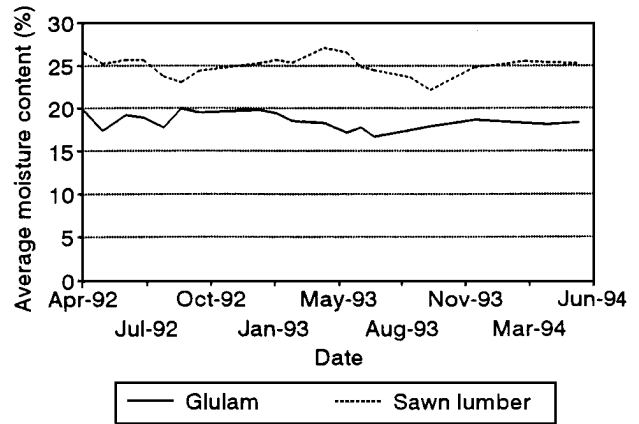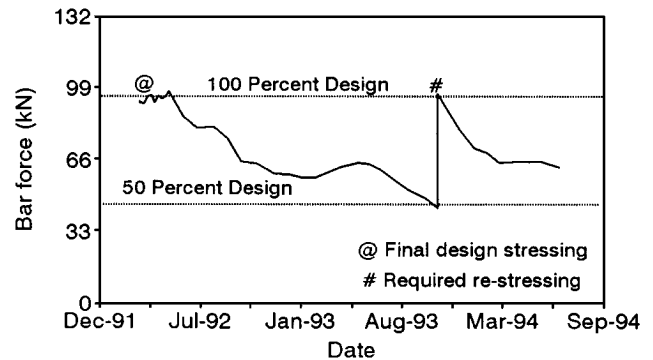
measurements indicated an average camber of 8.9 mm (0.35 in.) in the superstructure. Therefore, an apparent decrease in camber of 16.8 mm (0.66 in.) occurred with the initial dead loading at installation. At the conclusion of monitoring, measurements indicated an average camber of 4.8 mm (0.19 in.) along both bridge edges. Therefore, average camber loss, or vertical creep, of the Christian Hollow bridge totaled 4.1 mm (0.16 in.) during the monitoring period.

Table 1—Summary of camber measurements

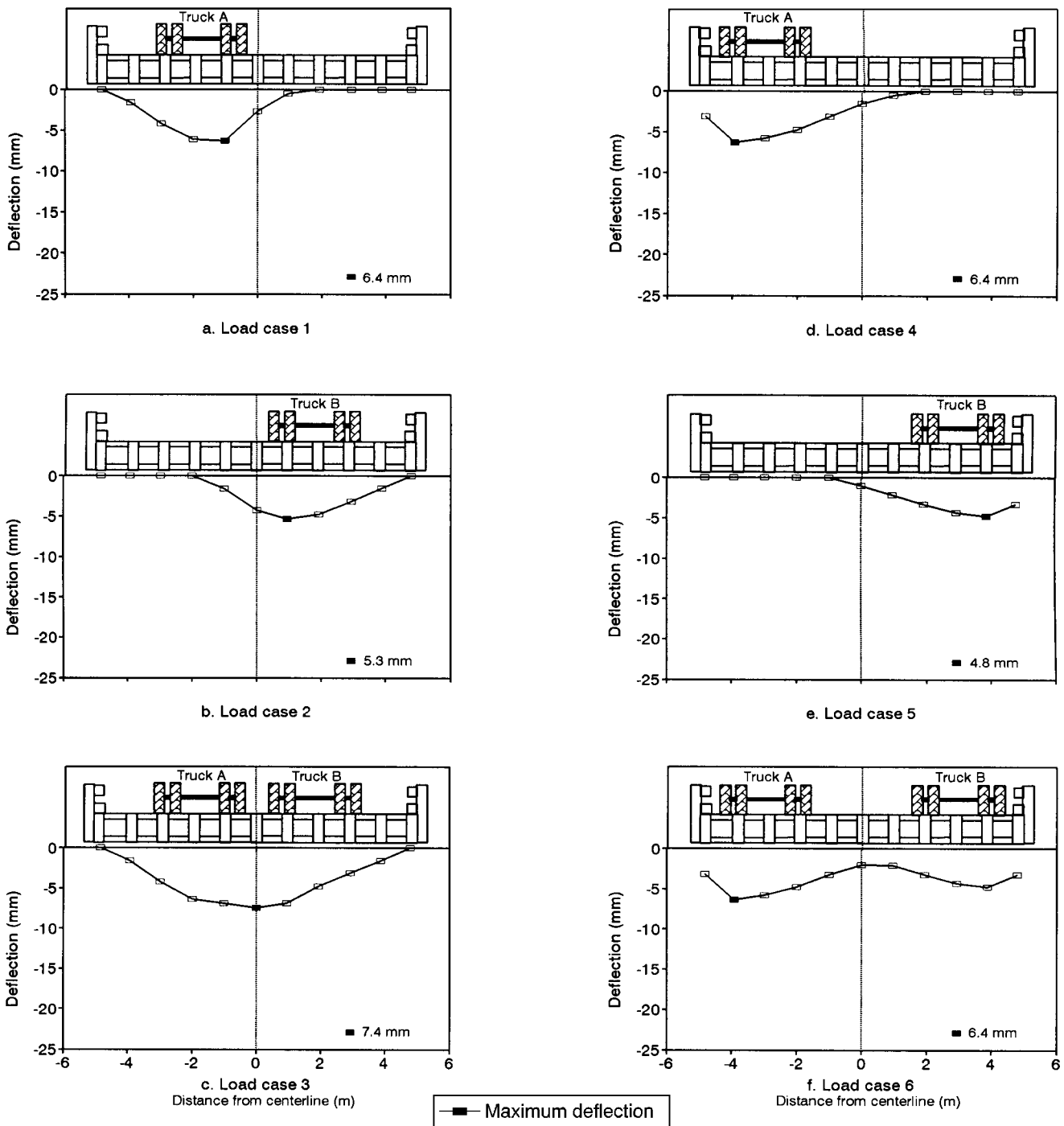|	Longitudinal camber (mm (in.))		
Centerspan location	Initiation of monitoring	Conclusion of monitoring	Vertical creep (mm (in.))
Upstream edge	+9.1 (0.36)	+4.8 (0.19)	4.3 (0.17)
Downstream edge	+8.6 (0.34)	+4.8 (0.19)	3.8 (0.15)
Average	+8.9 (0.35)	+4.8 (0.19)	4.1 (0.16)

Figure 15—Load test 1 transverse deflection measured at the centerspan of the bridge (looking east).
Cross sections of the bridge and vehicle positions are shown to aid interpretation and are not to scale.

Behavior Under Static Load

For each load case, transverse deflection measurements are given at the bridge centerspan, as viewed from the west end of the bridge (looking east). No permanent residual deflection was measured between load cases or at the conclusion of load testing. In addition, no measurable deflection was observed at the bridge supports during load testing. At the time of load tests 1 and 2, the interlaminar compressive stress was at 689 and 483 kPa (100 and 70 lb/in^2), respectively.

Load Test 1

Transverse deflection for load test 1 is presented in Figure 15. The maximum deflections for load cases 1 and 2

occurred under the inside truck wheel line and measured 6.4 and 5.3 mm (0.25 and 0.21 in.), respectively (Fig. 15a,b). For load case 3, the maximum measured deflection of 7.4 mm (0.29 in.) was at the roadway centerline and was the largest measured deflection of all load cases (Fig. 15c). Maximum deflections for load cases 4 and 5 measured 6.4 and 4.8 mm (0.25 and 0.19 in.), respectively, and occurred near the outside truck wheel line (Fig. 15d,e). The maximum deflection for load case 6 measured 6.4 mm (0.25 in.) and was under the outside wheel line of truck A (Fig. 15f). For both the centered and eccentric load cases, the deflected shape of the cross section of the centerspan follows the symmetrical truck positions, with maximum measured deflections for the single truck load cases occurring at the same relative positions for the two truck locations.

Assuming accurate load test results and linear-elastic behavior, the sum of the deflection resulting from individual truck loads should equal the deflection from both trucks applied simultaneously. Figure 16 shows the load test 1 comparison between individual and simultaneous truck loading. As shown in Figure 16, the two plots are nearly identical with only minor variations, which are within the accuracy of the measurements. From this information, it can be concluded that bridge behavior was within the linear-elastic range.

Load Test 2

Transverse deflection for load test 2 is presented in Figure 17. The maximum deflection of 6.4 mm (0.25 in.) for load case 1 occurred near the outside wheel line (Fig. 17a), and the maximum deflection of 5.8 mm (0.23 in.) for load case 2 occurred near the inside truck wheel line (Fig. 17b). For load case 3, the maximum deflection of 8.1 mm (0.32 in.) was measured at the roadway centerline and was again the largest deflection for all load cases (Fig. 17c). Maximum deflections for load cases 4 and 5 occurred near the outside truck wheel line and measured 7.4 and 6.6 mm (0.29 and 0.26 in.), respectively (Fig. 17d,e). The maximum deflection for load case 6 measured 7.4 mm (0.29 in.) and was near the outside wheel line of truck A (Fig. 17f).

Figure 18 shows the load test 2 comparison between individual and simultaneous truck loading. As with load test 1, the two plots are nearly identical, indicating that bridge behavior was within the linear-elastic range.

Load Test Comparison

A comparison of measured deflections for both load tests is presented in Figure 19 for load cases 3 and 6. The plots are similar in shape, but load test 2 deflections are greater at numerous data point locations. Different load testing conditions may have caused greater deflections for load test 2, especially the differences in truck loads and interlaminar compression. Load test 2 trucks were approximately 5 percent heavier than those for load test 1, which would tend to cause slightly greater deflections for load test 2. In addition, interlaminar compression was 207 kPa (30 lb/in^2) less at load test 2 than at load test 1. A lower interlaminar compression of this magnitude has been shown to result in greater relative deflections. The combined effect of these different
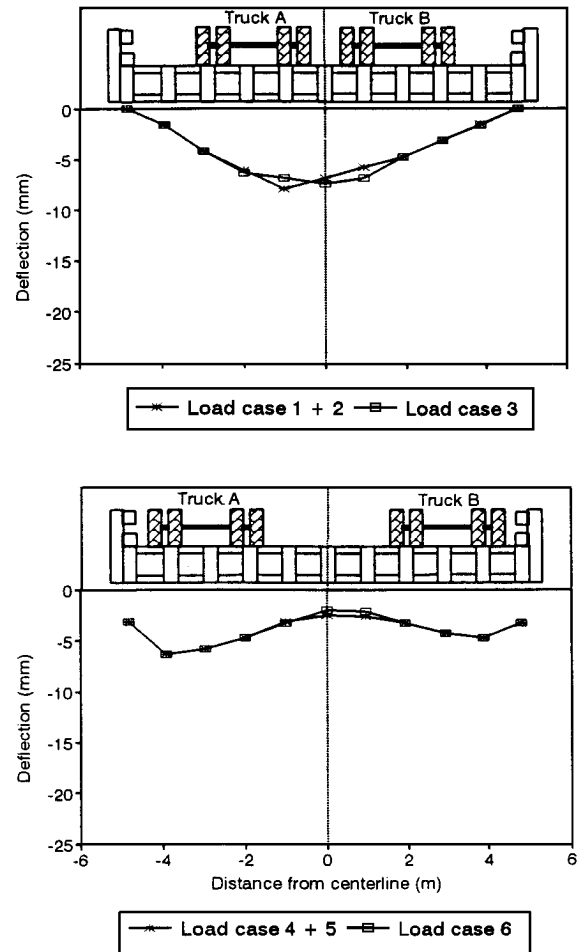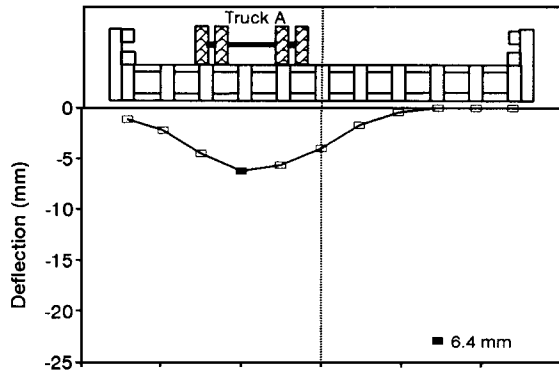

Figure 16—Load test 1 comparisons (looking east): (top) the sum of the measured deflections from load cases 1 and 2 to the measured deflections for load case 3; (bottom) the sum of the measured deflections from load cases 4 and 5 to the measured deflections for load case 6.

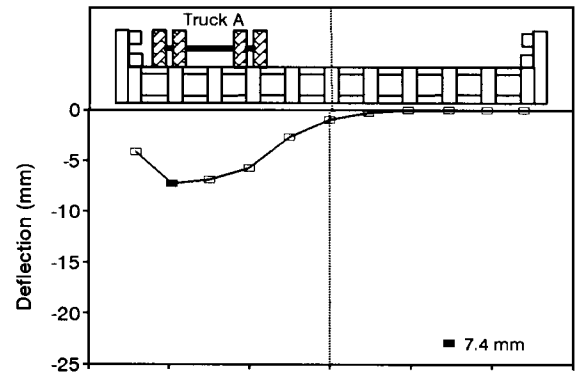load test conditions resulted in the greater deflections observed in load test 2.

Predicted Response

Table 2 summarizes the maximum measured deflections for each load test and the predicted maximum deflections for AASHTO HS20-44 truck loading. In both cases, the values are based on the load case 3 vehicle positions where the maximum load test deflections occurred. In addition to the absolute deflections, Table 2 also presents the span/deflection ratio as a function of the bridge span, L, measured center-to-center of bearings.
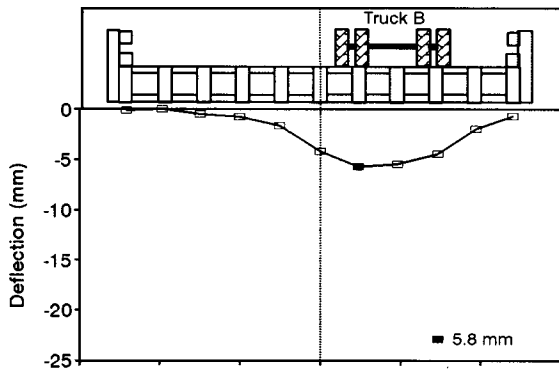
For both load tests, the predicted HS20-44 deflection was within the design limit of L/500. For load test 1, the predicted HS20-44 deflection was 7.1 mm (0.28 in.), which resulted in a span/deflection ratio of L/1,242. For load test 2, the predicted HS20-44 deflection was 7.9 mm (0.31 in.),
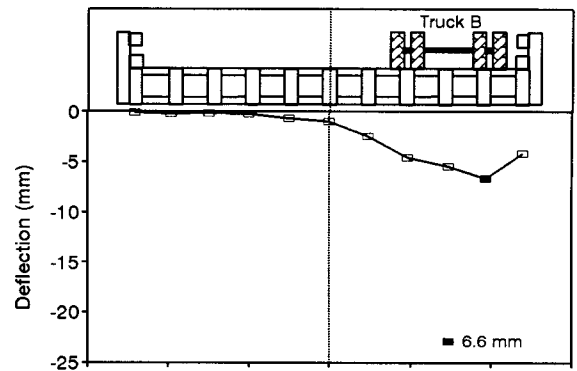
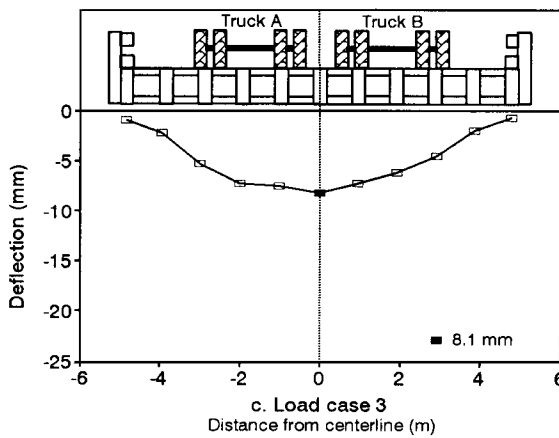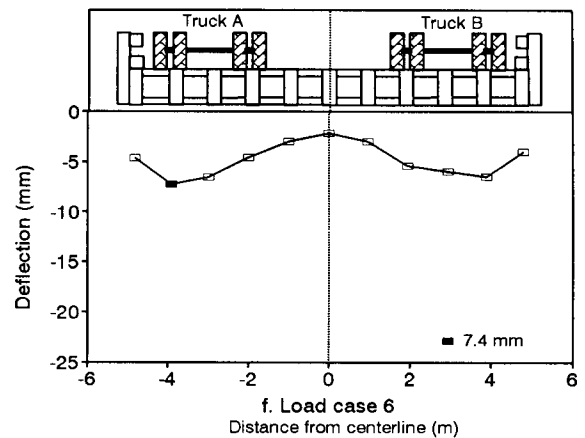**Figure 17—Load test 2 transverse deflection measured at the bridge centerspan (looking east). Cross sections of the bridge and vehicle positions are shown to aid interpretation and are not to scale.**
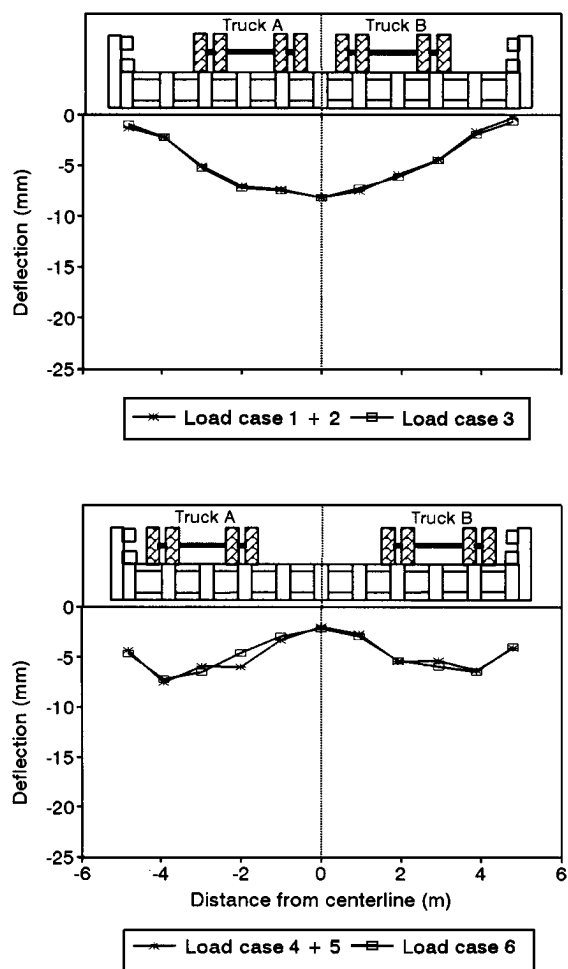
Figure 18—Load test 2 comparisons (looking east): (top) the sum of the measured deflections from load cases 1 and 2 to the measured deflections for load case 3; (bottom); the sum of the measured deflections from load cases 4 and 5 to the measured deflections for load case 6.

Figure 19—Comparison of measured deflections for load tests 1 and 2 (looking east).

which resulted in a span/deflection ratio of L/1,123. A 30-percent lower interlaminar compression for load test 2 compared with load test 1 caused an approximate 10-percent greater predicted HS20-44 deflection. Thus, the stiffness of the bridge may have decreased at the lower interlaminar compression level for load test 2.

Condition Assessment

General condition assessments indicated that the structural and serviceability aspects of the Christian Hollow bridge were satisfactory. Results of specific areas follow.

Bridge Geometry

At the initiation of monitoring, a slight narrowing of the bridge was noticed before applying the asphalt wearing

surface (Fig. 20). Width measurements indicated that the cross section of the centerspan was 102 mm (4 in.) narrower than the ends of the bridge. This was probably due to the sequential bar tensioning with a single hydraulic jack. This slight distortion has not increased in magnitude and should not affect overall bridge performance.

Table 2—Summary of deflections from static load testing

| | Deflection (mm (in.)) | | |
Load test	Measured	Predicted HS20-44	HS20-44 span/ deflection ratio
1	7.4 (0.29)	7.1 (0.28)	L/1,242
2	8.1 (0.32)	7.9 (0.31)	L/1,123

Figure 20—Slight hourglass shape was noticeable soon after initial stressing. Note stringline aligned with web member in middle of photograph.

Figure 21—Cardboard corner strapping protector was not removed at installation and caused a visible separation between lower flange laminations.

Figure 22—Excess creosote preservative was noticeable at several steel bearing plates (top) and along the top surface of the glulam timber rail (bottom).

Wood Components

Visual inspection of the wood components indicated no signs of deterioration or damage. All bolted connections remained tight with no signs of wood member crushing beneath the connectors. A cardboard "corner-protector" was caught between laminations (Fig. 21) and caused a short separation between adjacent laminations at the underside of the bridge that potentially could have caused performance problems. Fortunately, the effect of this small separation between laminations on the performance of the bridge appears to be insignificant.

Excess creosote preservative was bleeding from members with a southern exposure and stained several steel bearing plates (Fig. 22, top). In addition, the side and top surfaces of the rail members had a visible layer of residue (Fig. 22, bottom) that was leaching from the glulam members. Proper treatment procedures, including a post-treatment vacuum or steam process, could have alleviated this problem.

Wearing Surface

The asphalt wearing surface is in good condition, with minor transverse reflective cracking visible near the supports of the bridge. Reflective cracking near supports is typical of simple-span bridges and was expected. Minor longitudinal rutting, approximately 6.4 mm (1/4 in.) deep, was evident at the conclusion of monitoring, but it did not appear to collect standing water because of the roadway grade. No longitudinal asphalt cracking was observed.

Stressing Bar Anchorages

The steel bearing plate anchorage system is performing satisfactorily. There are no signs of wood crushing beneath the anchorage plates or corrosion on any of the steel components, including the exposed portion of the stressing bars.

Conclusions

Based on data collected during the 28 months of monitoring, performance of the Christian Hollow bridge is satisfactory. There are no structural or serviceability deficiencies that would prevent the bridge from providing satisfactory performance in the future. Based on the monitoring results, the following conclusions and observations are presented:

- The average trend in moisture content indicates that both the glulam timber and sawn lumber components behaved similarly with no significant changes in moisture content. At the conclusion of monitoring, the average moisture content was approximately 19 percent for the glulam and 25 percent for the sawn lumber.

- The average trend in stressing bar force indicates that sufficient bar force was maintained throughout the monitoring period. Bar force losses were primarily due to stress relaxation of the glulam web and sawn lumber flange laminations and warranted a bridge restressing after 1-1/2 years. Since that restressing, the bar forces leveled off at 67 kN (15,000 lb), or approximately 483 kPa (70 lb/in^2) interlaminar compressive stress. Subsequent measurements, taken 16 months after completion of the monitoring, indicate that the bar forces were at 59 kN (13,000 lb), or approximately 414 kPa (60 lb/in^2) interlaminar compressive stress. Future inspections of the Christian Hollow bridge should include bar force measurements to ensure that they remain greater than 48 kN (10,800 lb).

- Vertical creep was small and approximately 5-mm (0.19-in.) positive camber remained in the superstructure at the conclusion of monitoring.

- Static load testing and analysis indicate that the Christian Hollow bridge is performing in a linear-elastic fashion. When the interlaminar compression decreased from 689 to 483 kPa (100 to 70 lb/in^2), there was a 10-percent decrease in longitudinal bridge stiffness.

- Predicted maximum deflections for two lanes of AASHTO HS20-44 loading were within the design limit of L/500, where L is the span length measured center-to-center of bearings. For load test 1 at 689 kPa (100 lb/in^2) interlaminar compression, the maximum HS20-44 deflection is estimated to be 7.1 mm (0.28 in.), or approximately 1/1,242 of the span length. For load test 2 at 483 kPa (70 lb/in^2) interlaminar compression, the maximum HS20-44 deflection is estimated to be 7.9 mm (0.31 in.), or approximately 1/1,123 of the span length.

- Visual inspections indicate no signs of deterioration of the wood or steel components. Minor longitudinal rutting and reflective cracking were visible in the asphalt wearing surface at the conclusion of monitoring.

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Appendix—Information Sheet

General

Name: Christian Hollow bridge

Location: Town of Greenwood, Steuben County, New York

Date of Construction: January 1992

Owner: Steuben County, New York

Design Configuration

Structure Type: Stress-laminated box-beam

Butt Joint Frequency: 1 in 4 laminations transversely, 0.9 m (3 ft) longitudinal spacing

Total Length (out-out): 9.14 m (30 ft), 9.02 m (29.6 ft) as-built

Skew: 0 degrees

Number of Spans: 1

Span Length (center–center of bearings): 8.84 m (29 ft), 8.61 m (28.25 ft), as-built

Width (out–out): 9.75 m (32 ft), 9.91 m (32.5 ft), as-built

Width (curb–curb): 9.14 m (30 ft), 9.30 m (30.5 ft), as-built

Number of Traffic Lanes: 2

Design Loading: AASHTO HS20-44

Wearing Surface Type: Asphalt pavement, 51- to 76-mm (2- to 3-in.) thickness

Material and Configuration

Flange Laminations:

Species: Mixed hardwoods (sawn lumber)

Size : standard 38 by 140 mm (nominal 2 by 6 in.)

Grade: No. 2

Moisture Condition: approximately 26 percent at initiation of monitoring

Web Laminations:

Species: Southern Pine (glulam)

Size (actual): 219 by 502 mm (8-5/8 by 19-3/4 in.)

Beam Designation: 24F-V3

Moisture Condition: approximately 20 percent at initiation of monitoring

Preservative Treatment: Creosote

Stressing Bars:

Diameter: 16 mm (5/8 in.)

Number: 10 sets (through top and bottom flange)

Design Force: 96 kN (21,600 lb) interior; 104 and 88 kN (23,400 and 19,800 lb) two exterior bars

Spacing (center–center):   0.91 m (3 ft) interior 1.1 m (3.5 ft) exterior

Type: High strength steel threadbar with course right-hand thread, conforming to ASTM A722

Bar Anchorage Type: Discrete steel plates (with flat hex nut): 152 by 305 by 19 mm (6 by 12 by 3/4 in.) bearing 51 by 127 by 25 mm (2 by 5 by 1 in.) anchor

Rail and Curb System:

Design: Crash tested at Performance Level 1 on a longitudinal spike-laminated deck

Species: Douglas-fir

Member Sizes: Rails: 140 by 235 mm (6 by 10-3/4 in.), 24F-V4, glulam

Posts: 184 by 286 mm (nominal 8 by 12 in.) Dense Select Structural, sawn lumber

Curbs: 140 by 286 mm (nominal 6 by 12 in.) No. 1, sawn lumber