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Field Performance of Timber Bridges

20. Gray Stress-Laminated Deck Bridge

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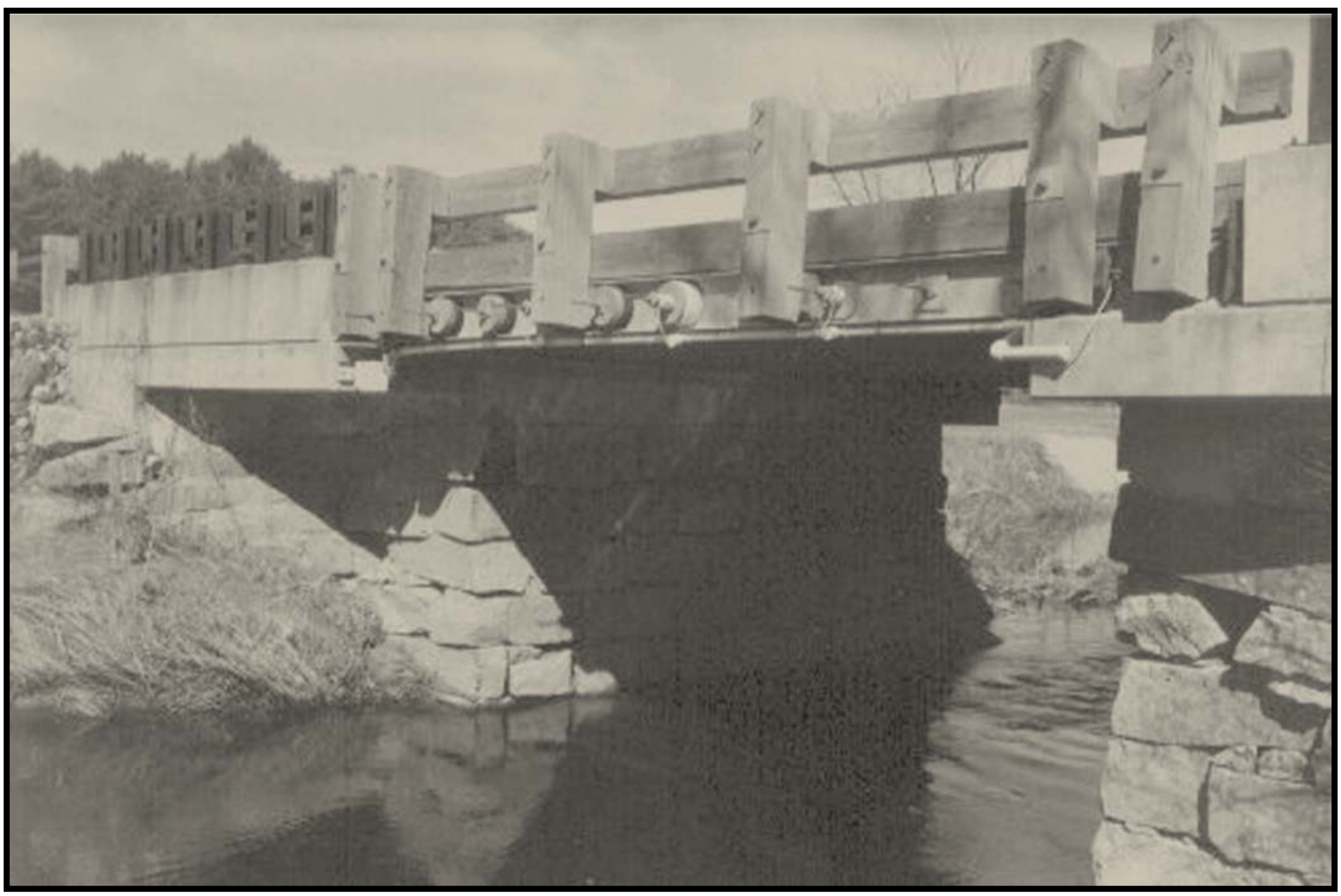

Abstract

The Gray bridge was constructed in the fall of 1991 in Gray, Maine. The bridge is a single-span, two-lane, stress-laminated deck structure that is approximately 24 ft long and 23 ft wide. It was constructed from chromated-copper-arsenate- (CCA-) treated eastern hemlock grown in Maine. This report presents information on the design, construction, and field performance of this bridge. The field performance of the bridge was monitored for 6½ years, beginning shortly after construction. During the field monitoring program, data were collected related to wood moisture content, force level of stressing bars, behavior under static truck loading, and overall structural condition. With the exception of having to be retensioned approximately every 3 years, the bridge is performing well, with no structural or serviceability deficiencies.

Keywords: Timber, bridge, wood, stress-laminated

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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 (USDA 1995). Responsibility for the development, implementation, and administration of the TBI was assigned to the USDA Forest Service. To implement the program, the Forest Service established three primary emphasis areas: demonstration bridges, technology transfer, and research. Responsibility for technology transfer and demonstration bridge programs was assigned to the National Wood in Transportation Information Center (NWITIC), formerly the Timber Bridge Information Research Center, in Morgantown, West Virginia. Under the demonstration program, the NWITIC provides matching funds to local governments to construct demonstration timber bridges, which encourages innovation through the use of new or previously underutilized wood products, bridge designs, and/or 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, thus providing 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, Congress passed the Intermodal Surface Transportation Efficiency Act (ISTEA) in 1991, which includes 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). 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 develop and administer a national timber bridge research program.

This paper is 20th in a series that documents the field performance of timber bridges included in the FPL timber bridge monitoring program. It addresses the field performance of a chromated-copper-arsenate- (CCA-) treated stress-laminated truss bridge located in Gray, Maine. This report summarizes results from a 6½-year field-monitoring program, which was initiated when the bridge was constructed in December 1991. Data were collected on wood moisture content, force level of stressing bars, behavior under static truck loading, and overall structural condition.

The Gray bridge is a single-span, two-lane, stress-laminated structure that is approximately 24 ft long, 23 ft wide, and 14 in. deep. (See Table 1 for metric conversion factors.) The deck was constructed from CCA-treated eastern hemlock grown in Maine.

Table 1—Factors for converting inch–pound units of measurement to SI units

Inch–pound	Conversion factor	SI unit
inch (in.)	25.4	millimeter (mm)
foot (ft)	0.3048	meter (m)
square foot ft^2	0.09	square meter (m^2)
pound (lb)	4.448	Newton (N)
lb/in^2 (stress)	6,894	Pascal (Pa)
lb-in	0.1129	Newton meter (N-m)
lb/ft^3	16.01	kilogram per cubic meter (kg/m^3)

Background

The Gray bridge is located near Gray, Maine (Fig. 1). The bridge is on Merrill Road, a two-lane paved road that crosses Collyer brook and provides access to a local dairy farm and a residential area. The average daily traffic is unknown but is estimated to be relatively low.

The original Gray bridge consisted of a single-lane precast concrete bridge supported by granite abutments. Although the original bridge had undergone substantial deterioration, the primary reason for the replacement was to widen the structure to allow two-way traffic.

Through a cooperative effort involving the University of Maine and the Maine Department of Transportation, a proposal was submitted to the USDA Forest Service to partially fund the Gray bridge replacement with funds from the Timber Bridge Initiative. The proposal specified a stress-laminated timber deck bridge constructed from CCA-treated eastern hemlock grown in Maine.

Objective and Scope

The objective of this project was to evaluate the field performance of the Gray bridge for 6½ years. The scope included field monitoring of wood moisture content, force level of stressing bars, behavior under static truck loading, and overall structural condition of the bridge. In an effort to improve future design and construction methods, results of this project will be considered with similar monitoring projects.

Design and Construction

The design and construction of the Gray bridge involved mutual efforts of several agencies and individuals. An overview of the design and construction of the bridge follows.

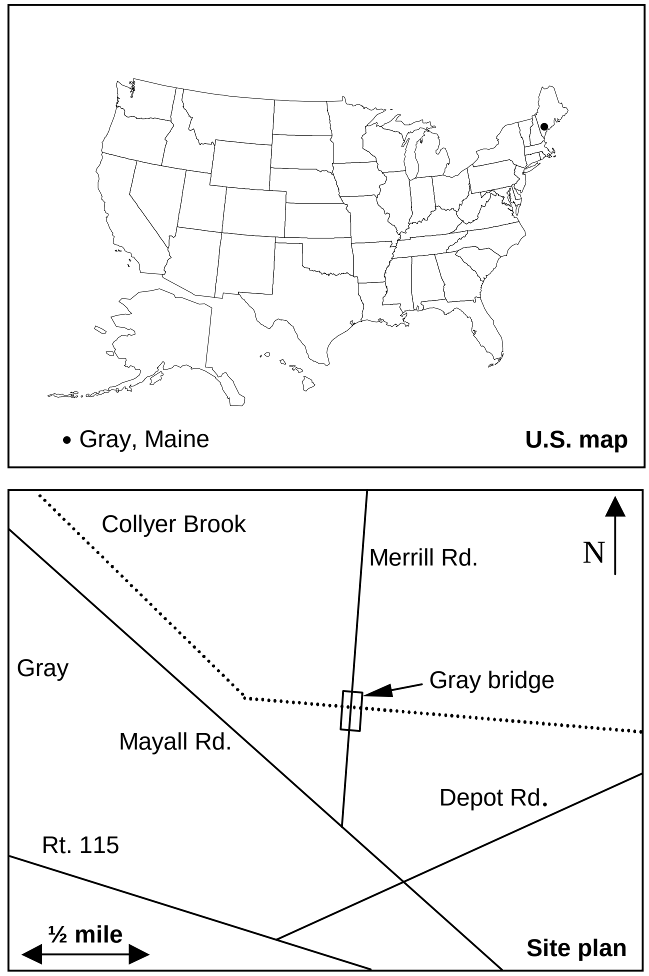

Figure 1—Location of Gray bridge.

Design

Design of the Gray bridge was completed by a team of engineers at the University of Maine, in cooperation with the Main Department of Transportation and with assistance from the FPL. The design featured a stress-laminated deck structure using CCA-treated No. 2 and better eastern hemlock laminations. For this bridge configuration, the laminations were placed side by side across the span. High strength steel bars were inserted through prebored holes in the laminations. The bars were tightened to provide sufficient friction to develop load transfer between the individual laminations (Figs. 2 and 3). Thus, the components were assumed to act together as a single unit.

With the exception of those features related specifically to stress laminating, design of the Gray bridge conformed to the American Association of State Highway and Transportation Officials (AASHTO 1989*) Standard Specifications for Highway Bridges* for two lanes of HS20–44 loading and the National Forest Products Association (NFPA 1991a,b) *National Design Specification for Wood Construction*.

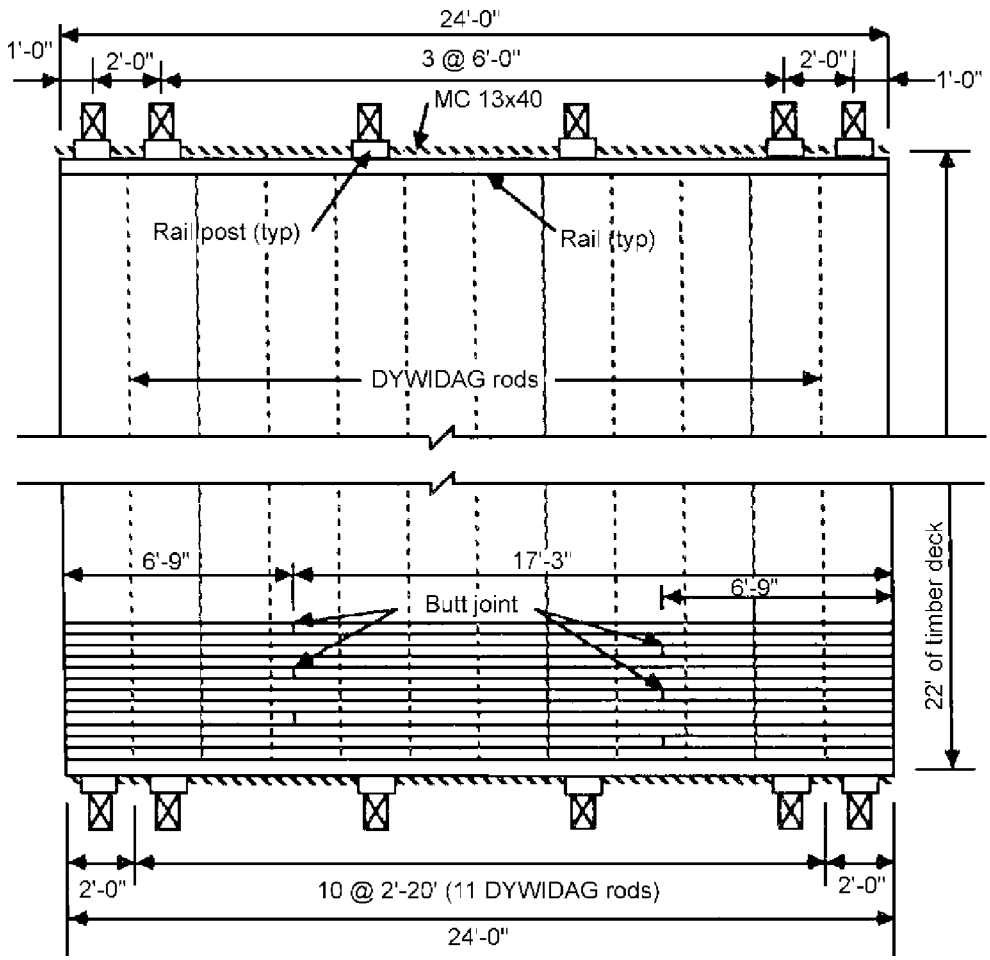

Figure 2—Design plan for Gray bridge.

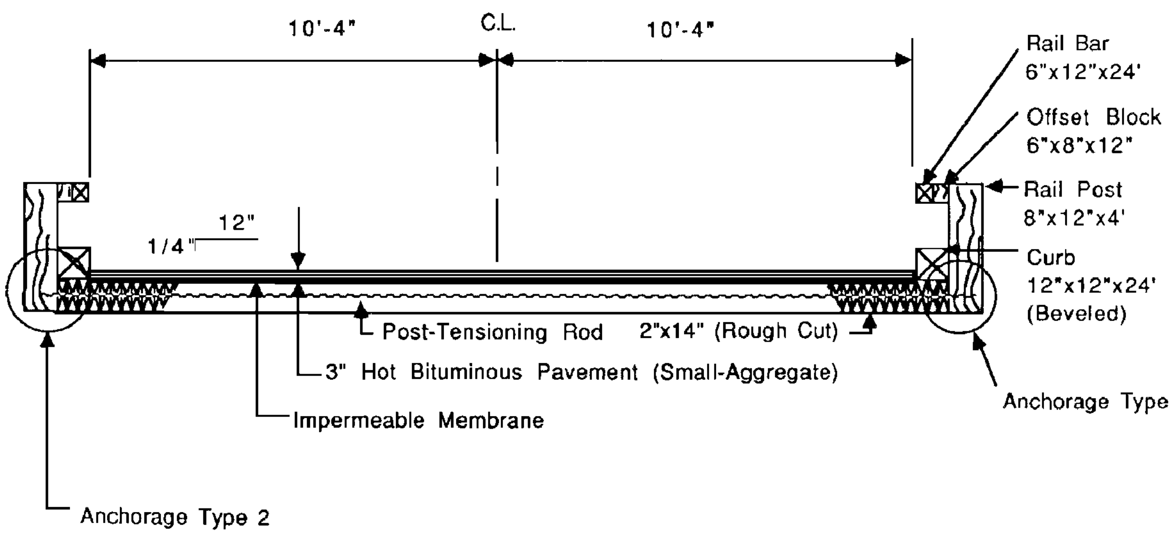

Figure 3—Design section of Gray bridge.

The Gray bridge was designed before the 1991 AASHTO guide specifications for stress-laminated bridge decks were published (AASHTO 1991). The design procedures were based on the 1983 Ontario Highway Bridge Design Code (OHBDC 1983) and research conducted at the University of Wisconsin (Dimakis and Oliva 1987, 1988).

The design procedure assumes that the wheel load from an HS20–44 wheel line is distributed over a 60-in. width of the bridge. Based on this distribution width, the stresses and deflections were calculated using the basic principles of mechanics.

The design geometry provided for a single-span superstructure 24 ft long, 23 ft wide, and 14 in. deep at a 0° skew. The depth of the bridge was limited to 14 in. because of clearance constraints at the site. The design specified No. 2 and better eastern hemlock laminations. Prior to fabrication, all wood members were cut and drilled, then pressure treated with a CCA–type III preservative to a minimum retention of 0.60 lb/ft^3. Not enough full-length laminations to complete the bridge were available at the time of construction; therefore, butt joints were specified in some laminations. The design specified one butt joint in every fourth lamination separated by 10.5 ft.

For stress laminating, the design specified 5/8-in.-diameter, epoxy-coated, high strength steel threaded bars with an ultimate strength of 150,000 lb/in^2. The design bar force of 26,000 lb provides an interlaminar compressive stress of 77 lb/in^2. The stressing system was designed for eleven 5/8-in.-diameter, high strength steel threaded bars. The spacing of the bars was 24 in. on center, beginning 24 in. from the bridge ends. The bar anchorage system used an MC 13 by 40 bulkhead channel with two types of bar anchorage. Five of the 11 bars were anchored using square 8- by 8- by ½-in. galvanized steel bearing plates (Fig. 4). The remaining 6 bars were anchored with 10-in.-diameter Belleville springs with a 7/8-in.-thick, 10-in.-diameter bearing plate (Fig. 5). The springs consist of four sets of two Key Belleville K1000–U–312 springs grouped to form stacks on 6 of the 11 bars. The springs were installed to help reduce bar force loss from stress relaxation and reduction in wood moisture content. All components of the stressing system were provided with corrosion protection. The stressing bars and nuts were epoxy coated, and the steel bearing channels and bearing plates were galvanized.

Design of the curb and rail system 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 curb and rail were specified to be roughsawn lumber, measuring 12- by 12-in. and 6- by 12-in., respectively, and pressure treated with a CCA–type III preservative to a minimum retention of 0.60 lb/ft^3. Rail posts were designated to be CCA-treated roughsawn lumber measuring 8- by 12-in. and were spaced 48 in. on center.

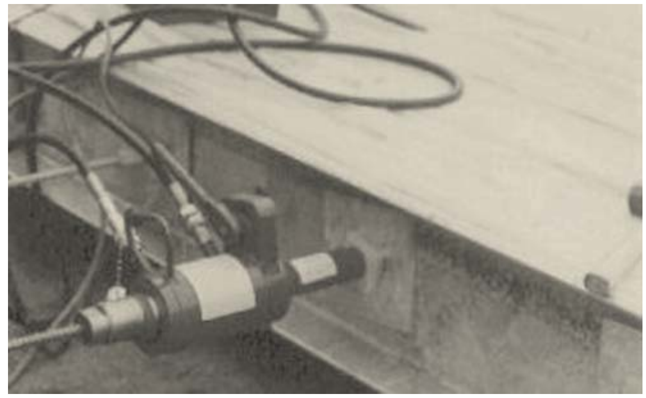

Figure 4—Steel channel and discrete plate bar anchorage configuration.

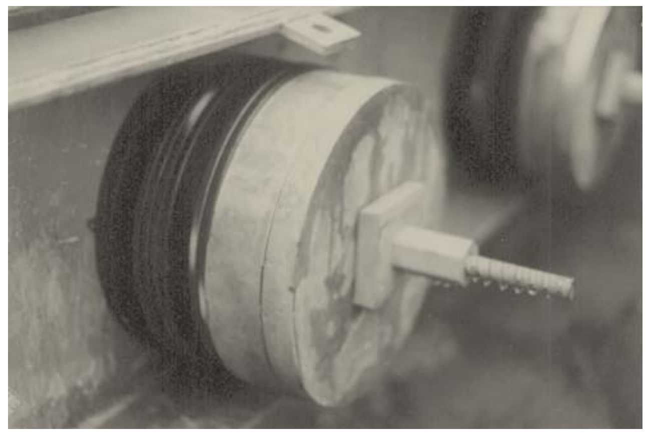

Figure 5—Steel channel and Belleville spring bar anchorage configuration.

To protect the bridge from moisture, one coat of adhesive primer was specified to be painted directly onto the wood deck, followed by the installation of two layers of waterproof membrane. The pavement was specified to consist of a 3-in.-thick hot bituminous wearing surface. An information sheet on specific bridge characteristics and material specifications is provided in the Appendix.

Construction

Construction of the Gray bridge was completed by a local contractor in the fall of 1991. Following work on the approach alignment and rehabilitation of the bridge abutments, construction of the bridge superstructure began October 31, 1991, and was completed December 3. The bridge was preassembled at the site on temporary timber supports placed adjacent to the east bridge abutment (Fig. 6). To construct the deck, laminations were placed edgewise across the supports. Steel prestressing rods were inserted through the

Figure 6—Preassembly of bridge at site on temporary timber supports placed adjacent to east bridge abutment.

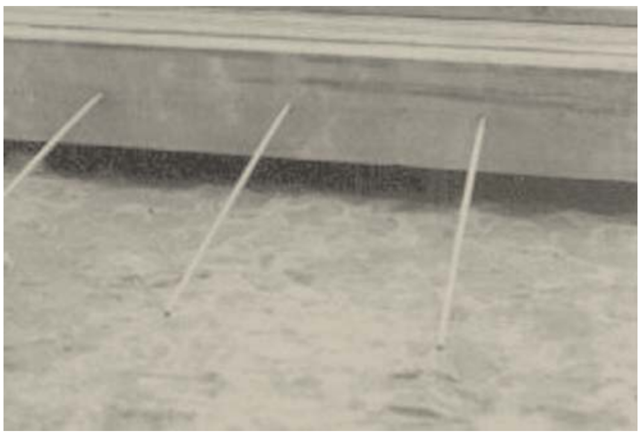

Figure 7—Insertion of prestressing rods through predrilled holes in laminations.

Figure 8—After the stressing system was installed, the bridge was stressed to the full design value.

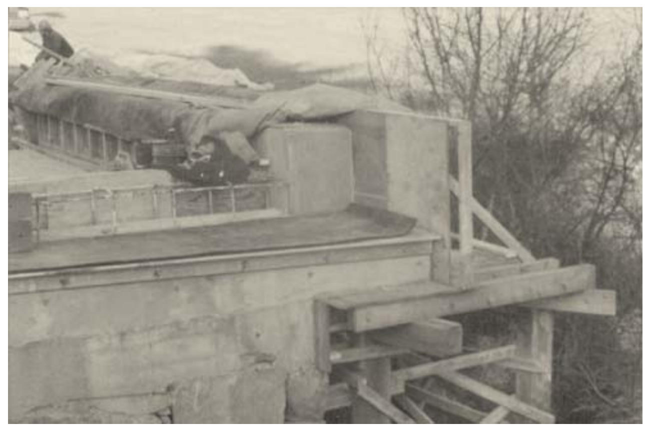

Figure 9—Prior to placing the deck, galvanized steel tie-down angles were bolted to the sides of the concrete cap of each abutment.

predrilled holes in the laminations. As the prestressing rods were advanced, timbers were placed over the ends of the rods. This process of advancing the rods and adding timber was continued until the entire deck was assembled (Fig. 7). Steel bearing channels were installed on the bridge edge, followed by the Belleville springs, anchorage plates, and nuts.

Following installation of the stressing system, the bridge was stressed to the full design value of 77 lb/in^2 (Fig. 8). During the initial stressing procedure, the bars were sequentially tightened by a hydraulic jack to the desired stress value. The same process was used to stress the bridge at 1 week, 6 weeks, 4 years, and 6½ years after installation. After the deck was stressed, a 12- by 12-in. CCA-treated eastern hemlock curb was bolted to each edge of the deck. Prior to placing the deck, galvanized steel tie-down angles were bolted to the sides of the concrete cap of each abutment and an elastometric bearing pad was placed on top of each concrete abutment cap (Fig. 9). The entire deck was lifted from the temporary supports with a large overhead crane (Fig. 10). The deck was moved over the span (Fig. 11) and carefully placed on the abutments (Fig. 12).

The superstructure was attached by bolting the bottom of the bridge deck to the steel tie-down angles attached to the abutment sides. Following attachment of the deck, the timber post and rails were installed (Fig. 13).

To protect the bridge from moisture, one coat of adhesive primer was painted directly onto the wood deck, followed by the installation of two layers of waterproof membrane. The waterproofing membrane was wrapped over the backwalls to completely seal the top surface of the structure from moisture. The bridge was paved with a 3-in.-thick hot bituminous wearing surface. The completed Gray bridge is shown in Figure 14.

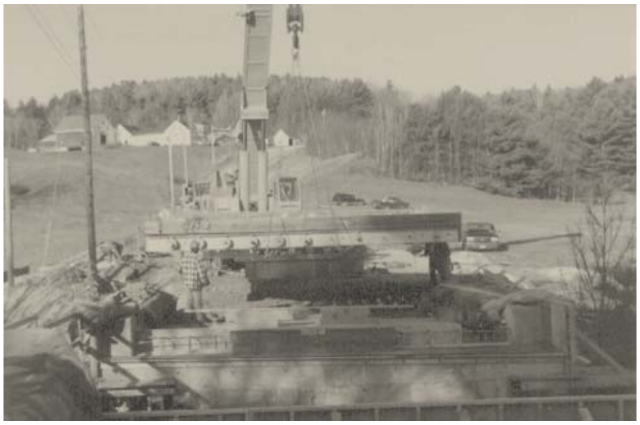

Figure 10—The entire deck was lifted from the temporary supports with a large overhead crane.

Figure 11—The deck was moved by crane over the span.

Figure 12—The deck was placed by crane onto the abutments.

Figure 13—Gray bridge following installation of the post and rail system.

Evaluation Methodology

To evaluate the field performance of the Gray bridge, a plan was developed by the University of Maine in cooperation with the FPL. The plan called for two static load tests of the completed structure and monitoring of the moisture content of the deck, stressing bar force, and general bridge condition. The evaluation methodology utilized procedures and equipment previously developed and used on similar structures (Caccese and others 1991, 1993; Dagher and others 1991, Ritter and others 1991).

Moisture Content

The moisture content of the Gray bridge was measured using an electrical-resistance moisture meter with 2-in. probe pins in accordance with ASTM D4444–84 (ASTM 1990). Measurements were obtained from several locations on the bridge superstructure by driving the probe pins into the wood approximately 1 in., recording the moisture content values, and adjusting the values for temperature and wood species. Moisture content readings were taken at the time of bridge installation and during the condition assessments.

Bar Force

Stressing bar force was measured with calibrated steel load cells developed at the University of Maine (Fig. 15) and with a hydraulic jack during the scheduled retensionings. The load cells were installed on five bars prior to the initial construction tensioning. Load cell measurements were obtained using a computer-controlled data acquisition system. Strain measurements were converted to units of bar tensile force by applying a calibration factor to the strain reading. Bar force measurements were also obtained from five bars prior to each retensioning by noting the jack pressure required to move the stressing nut away from the anchorage plate of each bar. The jack pressure was converted to bar force by applying a laboratory calibration factor to the pressure value.

Figure 14—Completed Gray bridge.

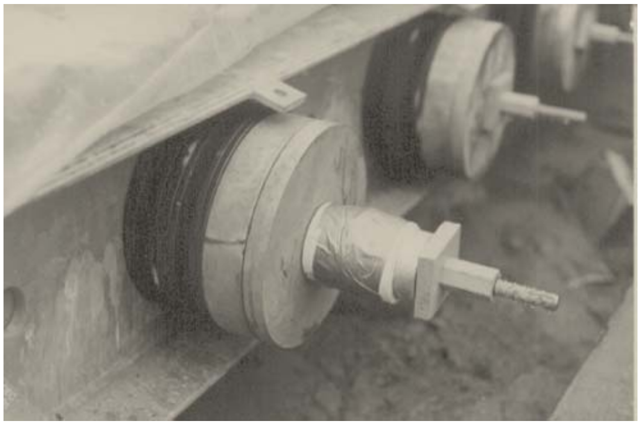

Figure 15—One of five load cells installed on stressing bars to monitor changes in bar tension.

Figure 16—Load test 1. Bridge deflections were measured using a surveyor's level and rulers attached to stiff boards mounted to the underside of the bridge.

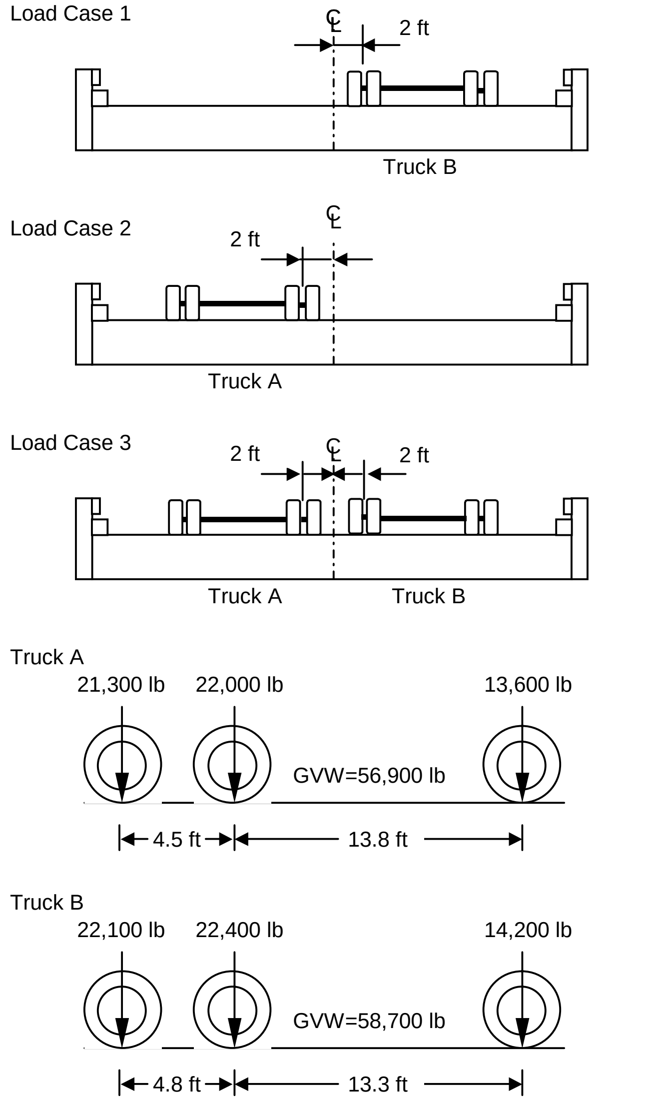

Figure 17—Truck weights, axle spacings, and transverse load test positions for load test 1.

Behavior Under Static Load

To determine the response of the Gray bridge to highway truck loads, static-load testing was conducted immediately before the bridge was opened to traffic and approximately 4 years afterward. In addition, the maximum predicted deflection was determined on the basis of static analysis for HS20–44 loading. Load testing involved positioning one or two fully loaded dump trucks on the bridge and measuring the resulting deflections at a series of locations along the bridge centerspan and abutment cross sections.

Deflection measurements for load test 1 were obtained using a surveyor's level and rulers attached to stiff boards mounted to the underside of the bridge (Fig. 16). Deflection measurements for load test 2 were obtained using displacement transducers mounted to a temporary support erected under the centerspan of the bridge. The transducer measurements were read with a voltmeter and converted to units of displacement by applying a laboratory calibration factor. Deflection measurements were obtained prior to each loading (unloaded bridge) and after placement of the test trucks (loaded bridge) for each load case. Each load case was carried out twice, and the results were averaged. Deflection measurements were also obtained at the conclusion of the load testing (unloaded bridge).

Load Test 1

Load test 1, conducted December 3, 1991, used three load cases and two fully loaded, three-axle dump trucks: truck A, with a gross vehicle weight of 56,900 lb, and truck B, with a gross vehicle weight of 58,700 lb (Fig. 17). For load cases 1, 2, and 3, the trucks were positioned transversely 2 ft from the centerline of the bridge. For all load cases, the truck center of gravity was positioned at midspan and deflections were

Figure 18—One of the two trucks used for load test 1.

measured to within 0.06 in. One of the two trucks used for load test 1 is shown in Figure 18.

Load Test 2

Load test 2, conducted November 7, 1995, used three load cases and two fully loaded, three-axle dump trucks: truck A, with a gross vehicle weight of 61,100 lb, and truck B, with a gross vehicle weight of 60,650 lb (Fig. 19). For load cases 1, 2, and 3, trucks were positioned transversely 2 ft from the centerline of the bridge. For all load cases, the truck center of gravity was positioned at midspan and deflections were measured to within 0.01 in.

Condition Assessment

The general condition of the Gray bridge was assessed on three occasions during the monitoring period. The first assessment was December 3, 1991, during the first load test. The second assessment took place November 7, 1995, during the second load test after approximately 4 years of service. The third assessment occurred April 7, 1998, after approximately 6½ years of service. The condition assessments involved visual inspections, measurements, and photographic documentation. Items of specific interest included the bridge geometry, wood components, wearing surface, and stressing bar system.

Results and Discussion

The following results are based on data collected during the 6½-year monitoring program for the Gray bridge.

Moisture Content

The average trend in wood moisture content is presented in Figure 20. At the initiation of the monitoring, the average moisture content was approximately 17%. After 4 years of service, the moisture content decreased to 13%. After

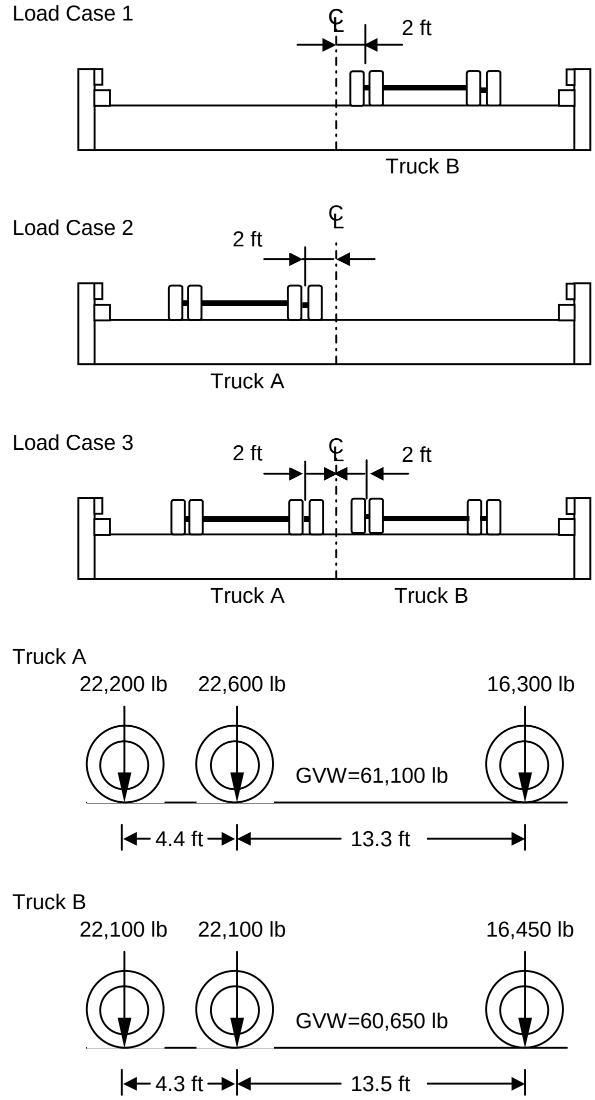

Figure 19—Truck weights, axle spacings, and transverse load test positions for load test 2.

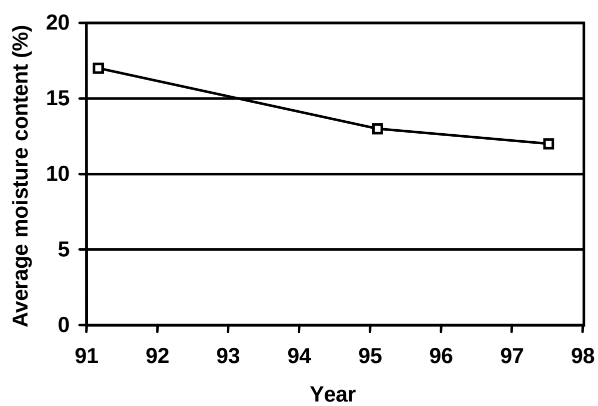

Figure 20—Average trend in moisture content, October 1991 to 1998.

6½ years, the moisture content seemed to have stabilized at approximately 12%. Moisture content measurements and visual inspections of the wood indicated that the waterproof membrane and pavement crown have been effective in protecting the bridge from water.

Bar Force

The average trend in bar force is shown in Figure 21. For stress-laminated structures to perform efficiently, adequate bar force must be maintained to prevent interlaminar slip. The bar force was expected to decrease after construction; therefore, the bridge was retensioned to the full design value of 26,000 lb or 77 lb/in^2 interlaminar compression after 1 week, 6 weeks, 4 years, and 6½ years of service.

Data collected during the first retensioning indicated that the average bar force had decreased 35% to approximately 50 lb/in^2 interlaminar compression during 1 week. Data collected during the second retensioning indicated that the average bar force had decreased approximately 40% to 45 lb/in^2 interlaminar compression during 5 weeks.

Measurements taken 46 months after the second retensioning indicated that the bar force had decreased 60%, to approximately 30 lb/in^2 interlaminar compression. Subsequently, the bars were retensioned. Bar force measurements taken approximately 30 months after the third retensioning indicated that the bar force had again decreased approximately 60%, to 30 lb/in^2 interlaminar compression. Therefore, the bars were tensioned again.

Bar force was expected to decrease as a result of the combined effects of a decrease in wood moisture content and stress relaxation. The 5% decrease in moisture content caused wood shrinkage, which was probably most significant during the first half of the monitoring period when the greatest moisture content loss occurred. Stress relaxation in the laminations has been observed to cause bar force loss in numerous other stress-laminated bridges (Ritter and others 1991).

The data indicated that the bar force decreased approximately 60%, to 30 lb/in^2, prior to the last two retensionings. The AASHTO guide specifications for stress-laminated bridges require a bar force of 37 lb/in^2 for this type of bridge. It is probable that the bar force will continue to decrease, and future bar retensionings will be necessary to maintain an adequate level of bar force in the Gray bridge.

Behavior Under Static Load

Results of static load testing and predicted response are presented in the following text. For each load case, transverse deflections are given at the centerspan of the bridge as viewed from the south end (looking north). No permanent residual deflection was measured between load cases or at the conclusion of load testing. In addition, no significant movement was detected at the bridge supports during testing. At the time of load tests 1 and 2, the average bridge interlaminar compressive stress was 77 and 30 lb/in^2, respectively.

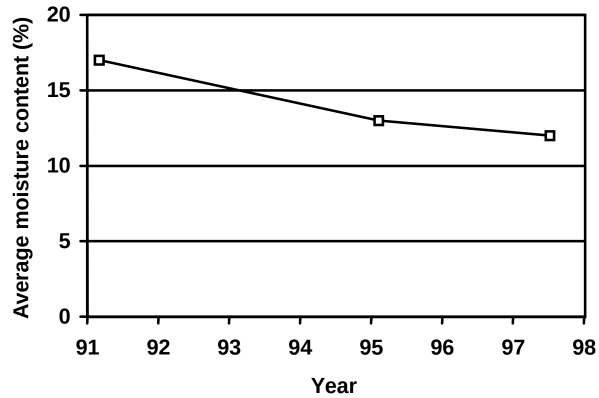

Figure 21—Average trend in prestress level, October 1991 to 1998.

Load Test 1

Transverse deflections from load test 1 (which was conducted December 3, 1991) at a prestress level of 77 lb/in^2 are shown in Figure 22. The maximum deflection for load case 1 occurred under the outside truck wheel line and measured 0.21 in. (Fig. 22a). The maximum deflection for load case 2 occurred between the truck wheel lines and measured 0.34 in. (Fig. 22b). The maximum deflection for load case 3 of 0.41 in. occurred between the truck B wheel lines and represented the largest deflection for all load cases (Fig. 22c).

Assuming accurate load test results and linear elastic behavior, the sum of the deflections resulting from individual truckloads should equal the deflections from both trucks applied simultaneously. Figure 23 shows the load test 1 comparison of individual and simultaneous truck loading. As shown in this figure, the two plots are similar, with variations within the accuracy of the measurements. From this information, it is concluded that the bridge behavior is within the linear elastic range.

Load Test 2

Transverse deflections from load test 2, which was conducted November 7, 1995, are shown in Figure 24. The maximum deflection for load case 1 occurred between the truck wheel lines and measured 0.37 in. (Fig. 24a). The maximum deflection for load case 2 also occurred between the truck wheel lines and measured 0.32 in. (Fig. 24b). The maximum deflection for load case 3 of 0.49 in. occurred under the inside wheel line of truck A and represented the largest deflection of all load cases (Fig. 24c).

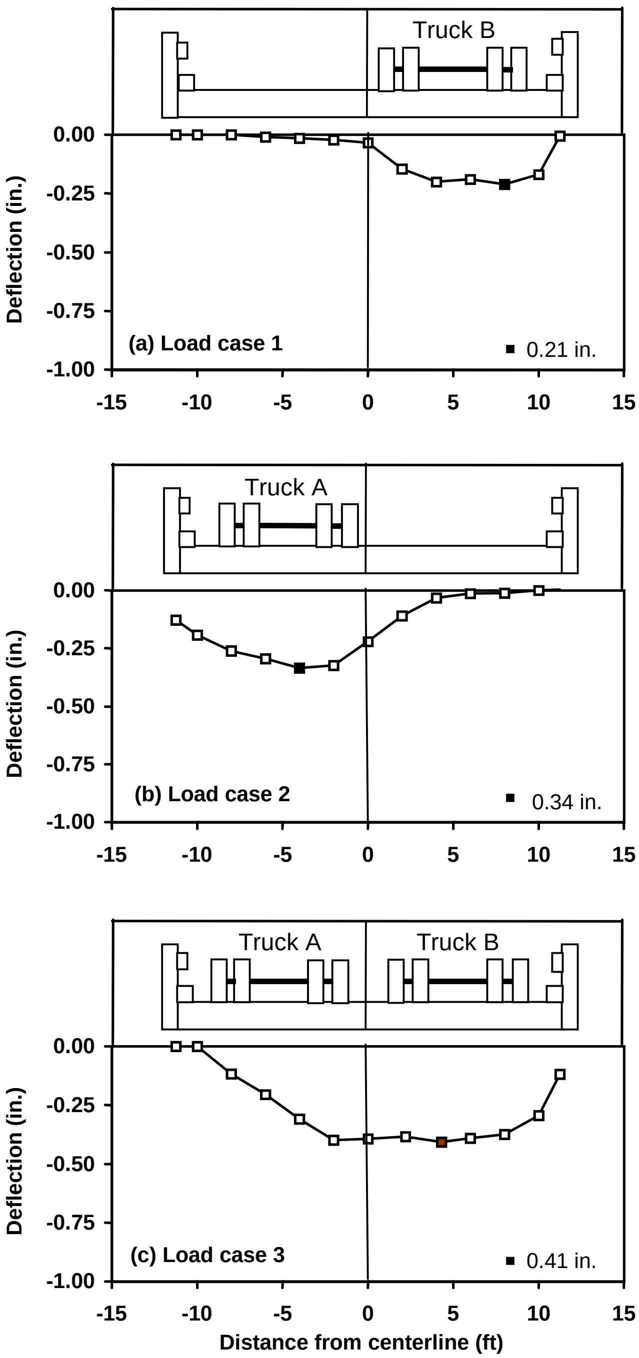

Figure 22—Tranverse deflections for load test 1, measured at bridge centerspan (looking north). Bridge cross-sections and vehicle positions are presented for the purpose of interpretation only and are not drawn to scale.

Figure 25 shows the load test 2 comparison of individual and simultaneous truck loading. The two plots are nearly identical, again indicating that the bridge behavior is within the linear elastic range.

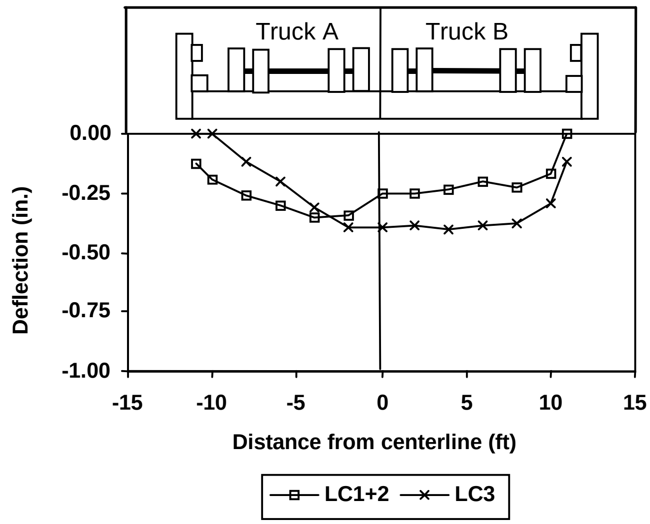

Figure 23—Load test 1 comparison: sum of measured deflections from load cases 1 plus 2 compared with deflections from load case 3.

Load Test Comparison

A comparison of measured deflections for both load tests is presented in Figure 26 for load case 3. The plots are similar in shape, but the deflections measured in load test 2 are greater at all but a few data points than the deflections for load test 1. The maximum measured deflection for load test 1 of 0.41 in. was 16% less than the maximum measured deflection of 0.49 in. for load test 2. Several factors, including differences in the truck size and weight and in the prestress level, may have contributed to the differences in deflection. The trucks used for load test 2 were approximately 5% heavier than the trucks used for load test 1, which would increase the deflections in load test 2. In addition, slight differences in the test truck configurations could have resulted in slight variations in the deflections. The 60% reduction in interlaminar compression for load test 2, which tends to reduce the bridge transverse stiffness and therefore increase deflections, contributed to the increased deflections for load test 2. The comparison of load test 1 and 2 showed that after adjusting for the 5% difference in truck weights, the maximum deflection increased only 12% when the prestress level was reduced from 77 to 30 lb/in^2.

Predicted Response

Table 2 summarizes the maximum measured deflections for both load tests, e allowable deflection for design purposes, and the predicted deflection for AASHTO HS20–44 truck loading. The measured values were obtained from load case 3, where the maximum load test deflection occurred. The allowable deflection for design purposes was L/400 or

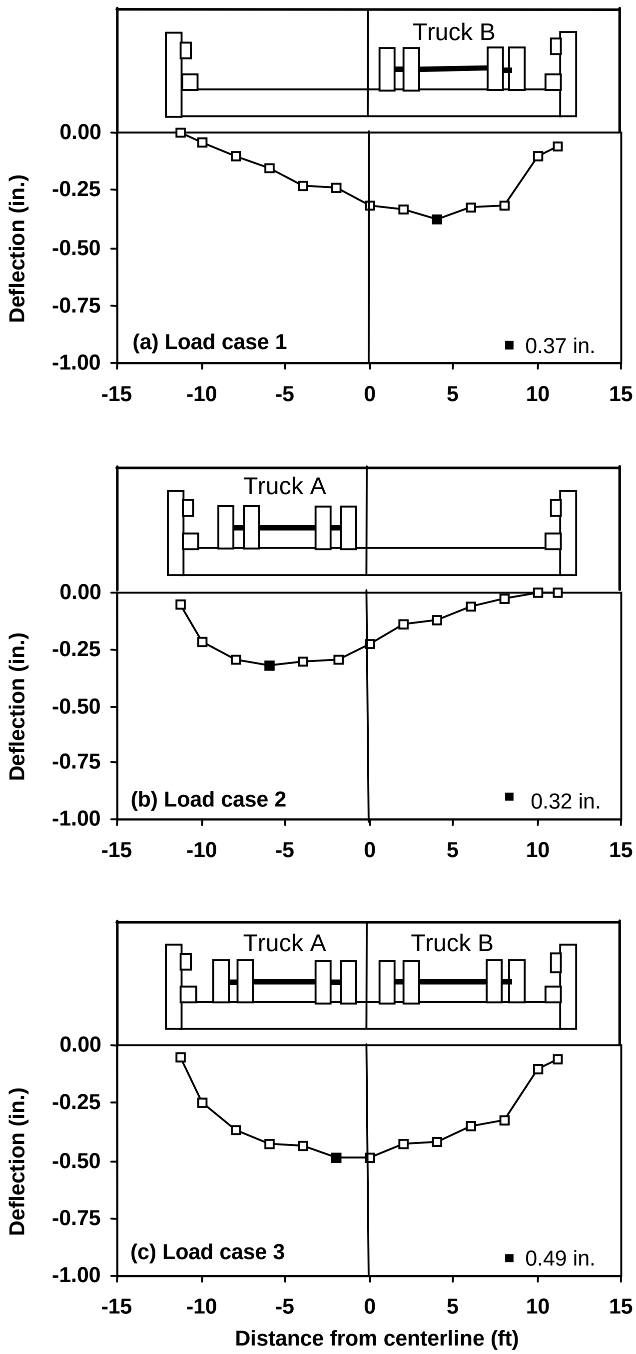

Figure 24—Transverse deflections for load test 2, measured at bridge centerspan (looking north). Bridge cross-sections and vehicle positions are presented for the purpose of interpretation only and are not drawn to scale.

0.69 in., where L is the bridge span, measured center–center of bearings. The maximum predicted deflection of 0.49 in. was obtained by assuming that one wheel line from an HS20–44 truck was carried by a 60-in. width of the bridge.

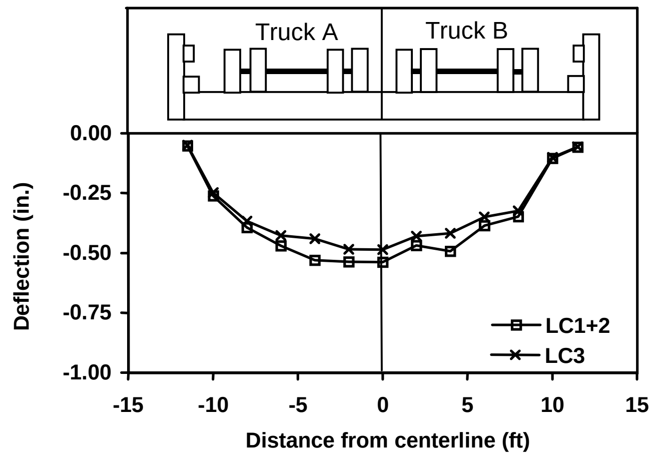

Figure 25—Load test 2 comparison: sum of deflections from load cases 1 plus 2 compared with deflections from load case 3.

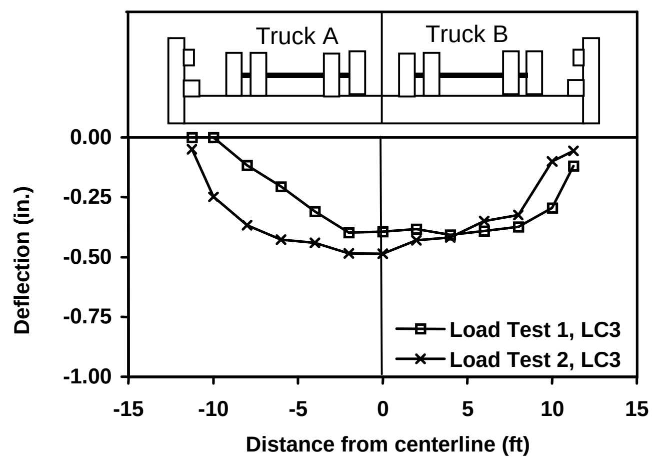

Figure 26—Comparison of load test 1 and 2 deflections.

Table 2 shows that the maximum measured and predicted deflections did not exceed the design limit of L/400 or 0.69 in. The maximum predicted HS20–44 deflection of 0.49 in. resulted in a span deflection ratio of L/540, which was lower than the design limit of L/400.

Table 2—Summary of load test, allowable, and predicted deflections

Load test	Maximum measured load test deflection (in.)	Maximum allowable deflection (in.)	Maximum predicted HS20–44 deflection (in.)
1	0.41	0.69	0.49
2	0.49	0.69	0.49

Condition Assessment

General condition assessments indicated that the structural and serviceability aspects of the Gray bridge are satisfactory. The areas subjected to inspection were bridge geometry, wood components, wearing surface, and anchorage system.

Bridge Geometry

Width measurements taken at the initiation of monitoring indicated that the stress-laminated structure was 1 in. narrower at the east abutment than at the west abutment. This was probably due to the sequential bar tightening with a single jack. In addition, the bridge was approximately 2 in. narrower at midspan than at the abutments. This slight distortion should not affect overall bridge performance.

Wood Components

Visual inspection of the wood components of the bridge indicated no signs of deterioration. However, minor damage to the curb, probably from a snowplow, was noted during the third condition assessment.

Wearing Surface

The asphalt-wearing surface is in good condition, with minor transverse reflective cracking visible over the bridge abutments. This is typical for single-span bridges and was expected. Longitudinal asphalt rutting or cracking was not evident.

Anchorage System

The continuous steel channel anchorage system is performing satisfactorily. No signs of wood crushing were visible beneath the channels. Surface rust was visible on some steel components in areas where the epoxy coating had chipped off. It is recommended that these areas be brush coated with an approved epoxy paint.

Conclusions

Based on the results of this research, we present the following conclusions and recommendations:

- Data collected during this research program indicate that the performance of the Gray bridge is satisfactory. With the exception of having to be retensioned approximately every 3 years, the bridge is performing well with no structural or serviceability deficiencies evident.

- During the 6½-year field monitoring, wood moisture content decreased gradually from approximately 17% to 12%. Based on moisture content readings and visual inspections, it is concluded that the waterproof membrane has been effective in protecting the bridge from moisture.

- The bridge was retensioned to the full design value of 77 lb/in^2 four times during the 6½-year field monitoring, The bar force had decreased to 30 lb/in^2 prior to each of the last two retensionings.

- Static load testing indicated that the Gray bridge is performing in the linear elastic range when subjected to two 61,000-lb trucks positioned with their center of gravity at midspan.

- The predicted deflection of 0.49 in. for HS20–44 loading was below the design limit of 0.69 in. or L/500, where L is the span length measured center–center of bearings.

- For load test 1, which was conducted at a bar force level of 77 lb/in^2, the maximum measured deflection from two 58,000-lb trucks positioned with their center of gravity at midspan was 0.41 in. This is below the design limit of 0.69 in. or L/400.

- For load test 2, which was conducted at a bar force level of 30 lb/in^2, the maximum measured deflection from two 61,000-lb trucks positioned with their center of gravity at midpan was 0.49 in. This is also below the design limit of 0.69 in. or L/400.

- The load test comparison of load tests 1 and 2 shows that, after adjusting for the slight difference in the truck weights, the maximum deflection increased only 12% when the bar force was reduced from 77 to 30 lb/in^2.

- Visual inspections indicated no signs of deterioration of the wood components. Surface rust is visible on some of the stressing system hardware in the vicinity of the anchorage nuts.

- There was no measurable difference in performance between the stressing bars equipped with Belleville springs and the stressing bars that were not equipped with Belleville springs.

- Assessment of wood moisture content, bar force, and general condition of bridge components should be performed on an annual basis.

- During annual assessments, the bars should be retensioned to the full design value of 26,000 lb if the bar force is less than 13,000 lb.

- Areas of the stressing system hardware where the epoxy coating has chipped off should be brush coated with an approved epoxy-based paint.

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Appendix—Information Sheet

General

Location: Gray, Maine

Date of construction: December 1991

Owner: Maine Department of Transportation

Configuration

Structure type: Stress-laminated solid timber deck

Butt joint frequency: 1 in 4 laminations transversely, separated 10.5 ft longitudinally

Total length (out–out): 24 ft

Skew: 0°

Number of spans: 1

Span length (center–center of bearings): 23 ft

Width (curb–curb): 23.7 ft (as built)

Number of traffic lanes: 2

Design loading: AASHTO HS20–44

Camber: 0 in.

Wearing surface: asphalt pavement, 3 in. thick

Material and Configuration

Truss Laminations:

Species: Eastern hemlock

Size: 2- by 14-in. roughsawn

Grade: Number 2 or better

Moisture condition: Approximately 17% at initiation of monitoring

Preservative treatment: CCA–type III, minimum retention of 0.60 lb/ft^3

Stressing Bars:

Diameter: 5/8 in.

Number: 11

Design force: 26,000 lb

Spacing (center–center): 24 in.

Type: High strength steel threaded bar
(Dywidag Systems International, Lincoln Park, NJ)

Bar Anchorage Type:

Continuous MC 13 × 40 steel channels with two types of bearings: Type 1: 8- by 8- by 3/4-in. steel anchorage plates
Type 2: 10-in.-diameter steel Belleville springs

Rail and Curb System:

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

Species: Eastern hemlock

Member sizes: Rails: 6- by 12-in. sawn timbers
Posts: 8- by 12-in. sawn timbers
Curbs: 12- by 12-in. sawn timbers

Preservative treatment: CCA–type III, minimum retention of 0.60 lb/ft^3

Waterproof Membrane System:

Membrane waterproofing Type 10A
Membrane waterproofing Type 108 ARN