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Field Performance of Timber Bridges

18. Byron Stress-Laminated Truss Bridge

Habib J. Dagher
Frank M. Altimore
Vincent Caccese
Michael A. Ritter



Abstract

The Byron bridge was constructed in the fall of 1993 in Byron, Maine. The bridge is a single-span, two-lane, stress-laminated truss structure approximately 46 ft long and 32 ft wide. The truss laminations were produced using chromated-copper-arsenate- (CCA-) treated Southern Pine connected with metal plate connectors. This report includes information on the design, construction, and field performance of the bridge. Field performance was monitored for approximately 5 years, beginning shortly after bridge construction. Performance monitoring involved collecting and evaluating data relative to wood moisture content, force level of stressing bars, behavior under static truck loading, and overall structural condition. The field evaluations showed that the Byron bridge is performing well, with no structural or serviceability deficiencies.

Keywords: timber, bridge, wood, stress laminated, truss

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Field Performance of Timber Bridges

18. Byron Stress-Laminated Truss Bridge

Habib J. Dagher, Director of Advanced Engineered Wood Composites Center and Professor of Civil Engineering
University of Maine, Orono, Maine

Frank M. Altimore, Engineering Research Consultant
Greenville, Maine

Vincent Caccese, Associate Professor of Mechanical Engineering
University of Maine, Orono, Maine

Michael A. Ritter, Research Engineer
USDA Forest Service, Forest Products Laboratory, Madison, Wisconsin

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 bridges 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 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 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 for 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 research 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, FPL and FHWA merged resources to develop and administer a national timber bridge research program.

This paper, 18th in a series, documents the field performance of timber bridges included in the FPL timber bridge monitoring program. It addresses the field performance of a stress-laminated truss bridge treated with chromated copper arsenate (CCA). This report summarizes the results from a 5-year field monitoring program, which was initiated when the bridge was constructed near Byron, Maine, in November 1993. 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.

The Byron bridge is a single-span, two-lane structure that is approximately 46 ft long, 32 ft wide, and 36 in. deep. (See Table 1 for metric conversion factors.) The truss laminations were produced from CCA-treated Southern Pine and metal plate connectors. This bridge is the second known stress-laminated structure to be constructed from metal plate connector truss laminations. The first known bridge of this type was constructed in Tuscaloosa County, Alabama, in 1992 (Triche and others 1994).

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

Inch-pound unit	Conversion factor	SI unit
inch (in.)	25.4	millimeter (mm)
foot (ft)	0.3048	meter (m)
square foot (ft ²)	0.09	square meter (m ²)
pound (lb)	4.448	newton (N)
lb/in ² (stress)	6,894	pascal (Pa)
lb-in	0.1129	newton meter (N-m)
lb/ft ³	16.01	kilogram/cubic meter (kg/m ³)
mile (mi)	1.609	kilometer (km)

Background

The Byron bridge is located near Byron, Maine, on State Highway 17, a two-lane, paved road that crosses the west branch of the Swift River, providing access to several popular recreation areas and serving as a primary route for logging traffic to area mills (Fig. 1). Average daily traffic varies seasonally; it was estimated to be 505 vehicles in 1989, and will probably increase to 810 vehicles by 2009.

The original Byron bridge, which was constructed in 1937, consisted of steel stringers with a nail-laminated timber deck supported by log crib abutments. The bridge was 40 ft long and 22 ft wide, and it included a rail system constructed of steel angles and channels. In 1969, the abutments were replaced with ones identical to those used in 1937. The timber deck was replaced with a noncomposite, reinforced concrete deck, and the existing structural steel and rail system was re-used. In 1991, inspection of the bridge indicated that the substructure was failing, and the bridge was rated structurally deficient.

Through a cooperative effort involving the Maine Department of Transportation, FPL, and the University of Maine, a proposal was submitted to FHWA to partially fund the Byron bridge replacement. The proposal specified a stress-laminated timber truss.

Objective and Scope

The objective of this project was to evaluate the field performance of the Byron bridge for 5 years. The scope included the development and verification of a transverse load distribution analysis procedure and field monitoring of wood moisture content, force level of stressing bars, behavior under static truck loading, and overall structural condition. The results of this project will be compared with those from similar monitoring projects in an effort to improve future design and construction methods.

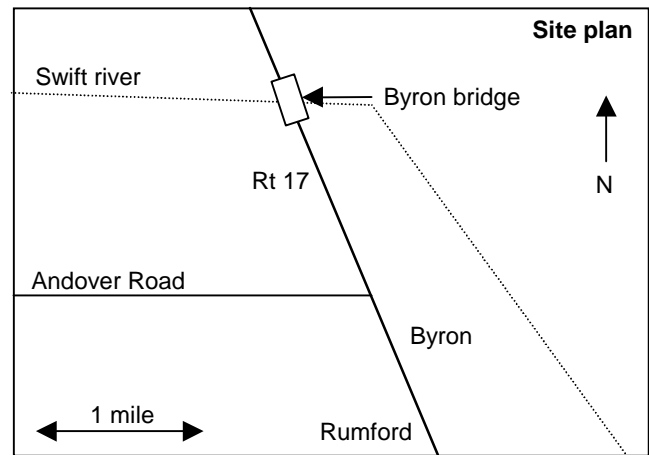
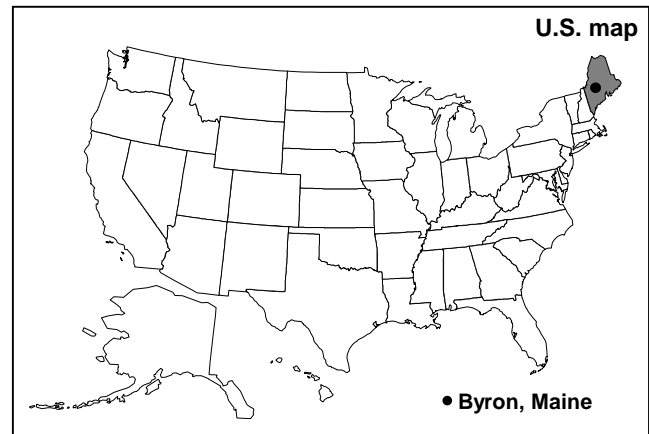


Figure 1—Location of Byron bridge.

Design and Construction

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

Design

The design was completed by a team of engineers at the University of Maine in cooperation with the Maine Department of Transportation, with assistance from the FPL. The design features a stress-laminated timber truss structure (Figs. 2 and 3) with dimension lumber chords and webs connected with metal plate connectors. For this bridge configuration, the trusses were placed side by side across the span. High strength steel bars were inserted through the web openings and pre-bored holes in the top and bottom chords. The bars were tightened to provide sufficient friction to develop load transfer between the individual truss laminations. Thus, the components were assumed to act together as a single unit.

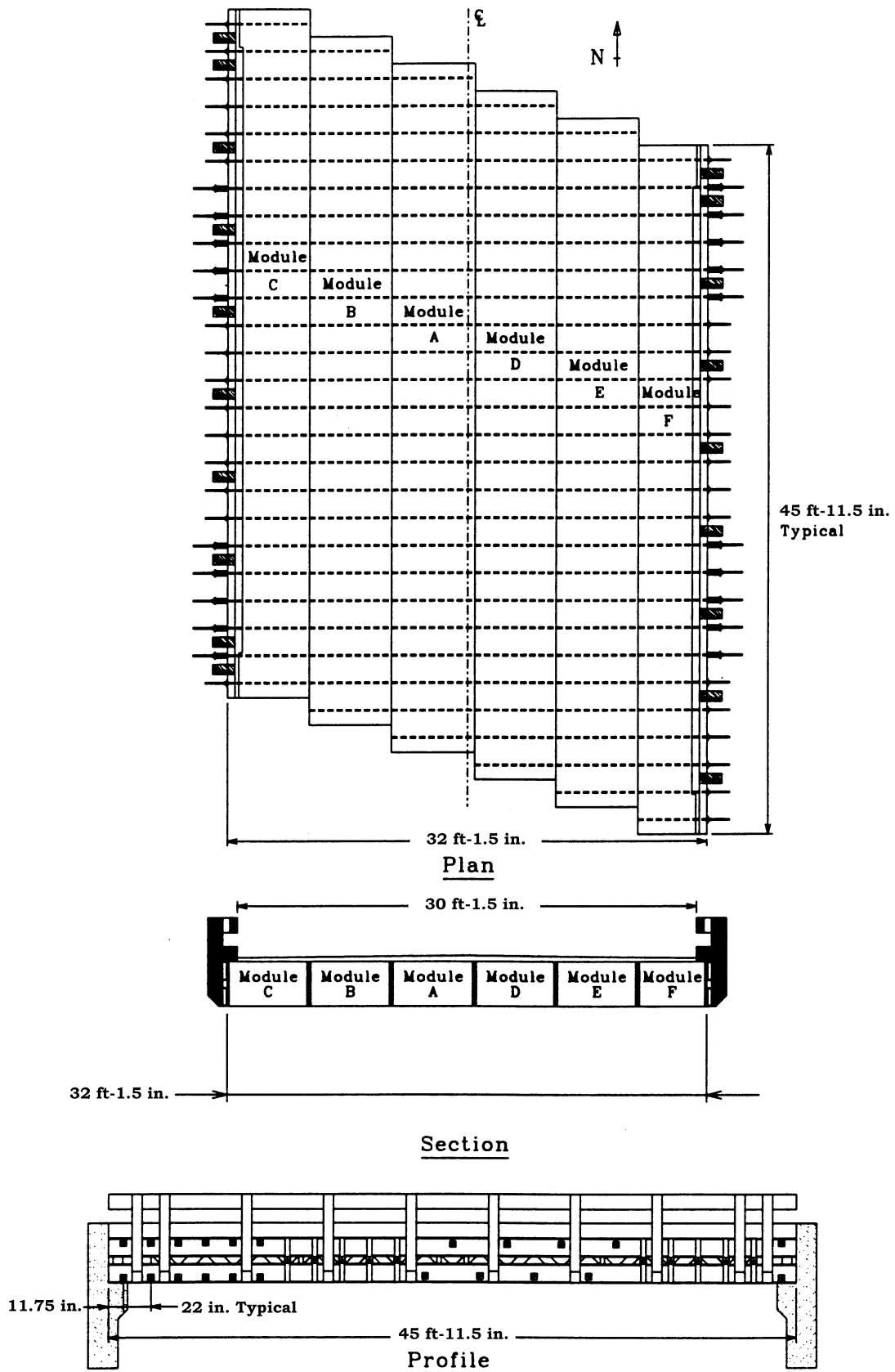
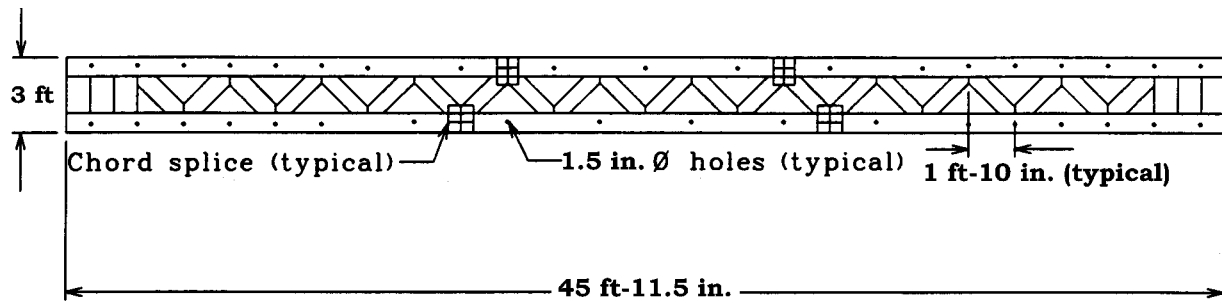
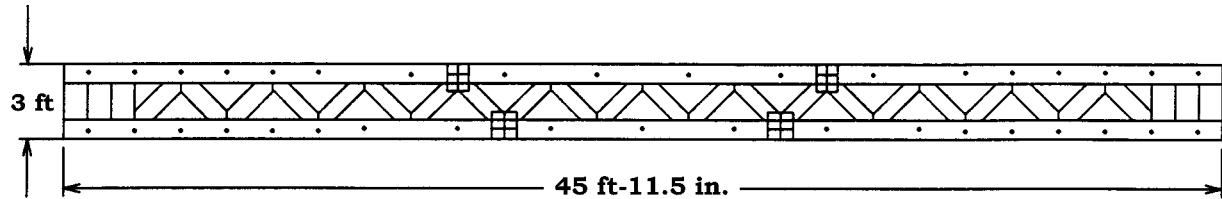


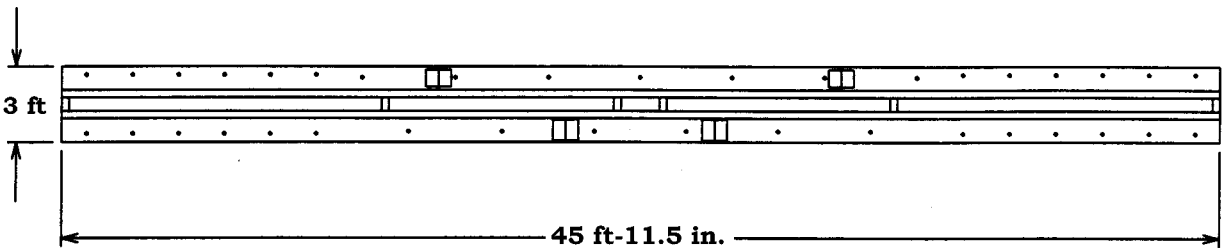
Figure 2—Design configuration of Byron bridge.



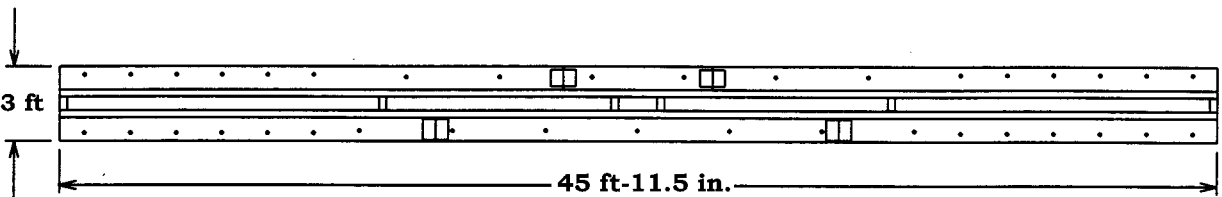
Structural Truss Type I



Structural Truss Type II



Spacer Truss Type I



Spacer Truss Type II

Figure 3—Profile of structural and spacer trusses.

With the exception of those features related specifically to the stress-laminated truss, design of the Byron bridge conforms to the American Association of State Highway and Transportation Officials (AASHTO 1989) *Standard Specifications for Highway Bridges* for two lanes of HS25-44 loading and the American Forest & Paper Association *National Design Specification for Wood Construction* (NFPA 1991a,b). The Byron bridge was designed in the absence of a recognized design procedure for stress-laminated truss bridges. The primary design concerns were as follows:

- Corrosion and stress-corrosion cracking of metal plate connectors

- Fatigue of metal plate connectors
- Transverse load distribution in a solid stress-laminated truss deck

To reduce the rate of metal plate connector corrosion, the moisture content of a wooden bridge should be kept below 19% and the metal plate connectors should be protected with an appropriate coating (Bruno and Weaver 1989). To address the fatigue and load distribution concerns, two separate research programs were conducted at the University of Maine.

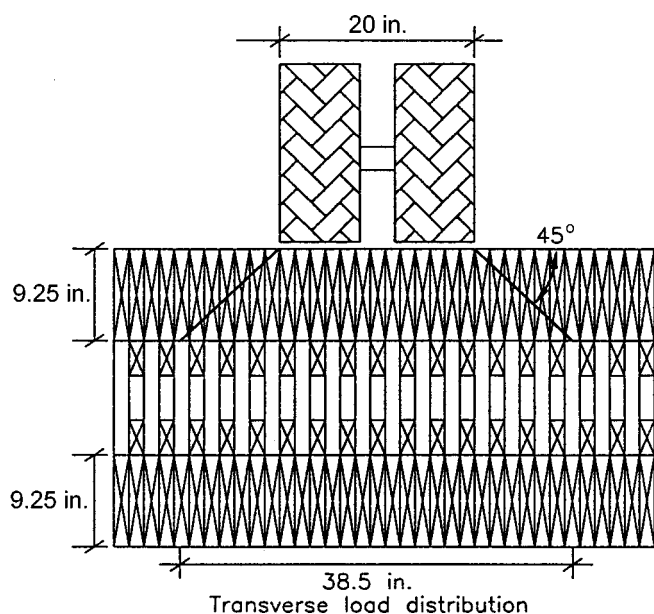


Figure 4—Simplified depiction of load distribution.

The first research program examined the fatigue behavior of metal plate connector trusses in low-volume bridges (Dagher and others 1992, 1994; Dagher and West 1998). Nearly 300 individual metal plate connector joints and 35 full-scale trusses were tested under high-cycle fatigue loading, resulting in fatigue design recommendations for metal plate connector joints.

The second research program developed and verified a simplified transverse load analysis procedure for stress-laminated truss decks (Altimore 1995). The analysis procedure assumes that the AASHTO design truck wheel load is distributed transversely at a 45° angle through the thickness of the top chord of the bridge (Fig. 4). Therefore, normal to the direction of the span, the wheel load is distributed over the width of the tire plus twice the thickness of the top chord of the deck. For deflection calculations, the distribution width was increased by 15%, paralleling the AASHTO recommendation for solid stress-laminated decks (AASHTO 1991). A two-dimensional model was used to analyze the trusses, with top and bottom chords modeled as continuous beam elements and the webs as truss elements.

Prior to construction of the bridge, the transverse load distribution assumptions were verified to be conservative through laboratory testing of an 8-ft-wide by 46-ft-long model of the Byron bridge. Results indicated that at a bar force of 125 lb/in², the transverse load distribution analysis overpredicted the maximum stress by 39% and the maximum deflection by 28%.

The design geometry of the Byron bridge consisted of a single-span superstructure 46 ft long, 32 ft wide, and 36 in. deep at an 18° skew. The depth of the trusses was limited to

Table 2—Metric conversion of dimensional lumber sizes

Nominal (in.)	Standard (mm)
2 by 4	38 by 89
2 by 6	38 by 140
2 by 8	38 by 184
2 by 10	38 by 235
2 by 12	38 by 286

36 in. because of clearance constraints at the site. The design-specified trusses were constructed from machine-stress-rated (MSR) Southern Pine and fabricated with 20-gauge metal plate connectors. Grades for the various sizes of lumber used were as follows: 2 by 12 lumber, MSR 1950f-1.7E; 2 by 10 and 2 by 8, MSR 2250f-1.9E; and 2 by 6 and 2 by 4, MSR 2400f-2.0E. (See Table 2 for metric conversion of nominal dimensions.) Prior to fabrication, all wood members were cut and drilled, then pressure-treated with CCA-type III to a minimum retention level of 0.60 lb/ft³.

Two truss configurations were used: structural and spacer. Each structural truss had 22 diagonal webs that connected the top and bottom chords. The structural trusses were placed next to the spacer trusses. The spacer trusses had deeper top and bottom chords compared with those of the structural trusses. Each spacer truss was connected with only enough metal plate connector joints and vertical webs to allow it to be handled as a single unit. The deeper chords of the spacer trusses covered the metal connector plates of the structural trusses. This detail prevented the structural truss metal plate connectors from withdrawing as result of cyclic loading and moisture and temperature changes.

The bridge was constructed from six rectangular modules. Each module was staggered by 22 in. to allow for the 18° skew. The skew required that two types of structural trusses be developed to maintain continuous web openings across the bridge for the prestressing rods. These structural trusses differed in the origination of the diagonal webs. For Type I trusses, the diagonal webs started from the top chord, and for Type II trusses, the webs started from the bottom chord. These two truss types were alternated from module to module.

Two types of spacer trusses were also developed. Chord butt joints of Type I spacer trusses were positioned so that they did not coincide with chord butt joints of Type I structural trusses. Similarly, the position of chord butt joints of Type II spacer trusses did not coincide with that of chord butt joints of Type II structural trusses. Consequently, one butt joint occurred in every two laminations. The butt joints in the top and bottom chords of all trusses were connected with metal plate connectors.

For stress laminating, the design specified 1-in.-diameter, epoxy-coated, high strength, steel threaded bars with an ultimate strength of 150 lb/in². The design bar force of 50,000 lb provided an initial interlaminar compressive stress of 125 lb/in².

Two types of anchorage systems were used. Because of the 18° skew, the bars in the ends of the bridge did not pass completely through the bridge. In this region and in part of the central region of the bridge, the bars passed through holes drilled in the top and bottom chords, and they were anchored by a discrete plate anchorage system. The bar ends were sustained on continuous C 15 by 50 steel channels through 5- by 5- by 1.25-in. epoxy-coated anchorage plates (Fig. 5a). In the remaining portion of the bridge, the rods passed through the openings in the webs and were anchored by a vertical tube anchorage system. The bar ends were sustained on continuous C 15 by 50 steel channels through 10- by 4- by 0.50-in. structural tubes and 5- by 5- by 1.25-in. anchorage plates (Fig. 5b). All components of the stressing system were protected from corrosion. The stressing bars, nuts, and anchorage plates were coated with epoxy. The structural steel

tubes and continuous steel channels were specified to be Grade 50 all-weather steel.

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, rail, and rail post were specified to be constructed of glulam, which was treated after gluing with a CCA-type III preservative to a minimum retention of 0.60 lb/ft³. The curb and rail measured 12- by 12-in. and 6- by 12-in., respectively. Rail posts measured 8- by 12-in. and were spaced 68 in. on center.

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 a self-sealing waterproof membrane. The pavement was specified to consist of a 1-in. leveling course and an asphalt wearing surface that measured 3.5 in. at the crown and 1.5 in. at the curb. An information sheet on specific bridge characteristics and material specifications is provided in the Appendix.

Construction

After the bridge approach was aligned and the bridge abutments were rehabilitated, construction of the Byron bridge began October 15, 1993; it was completed November 17, 1993.

The CCA-treated trusses were fabricated in Biddeford, Maine, and transported to the bridge site on a flatbed trailer in bundles banded together with light metal straps. Before the bridge was assembled, the galvanized metal plate connectors of each truss were brush painted at the site with an epoxy-based paint for additional protection from corrosion (Fig. 6). After the paint dried, the trusses were placed by crane onto temporary supports to form mini-modules (Fig. 7).

The mini-modules consisted of 10 to 12 trusses and were formed by nailing structural trusses to spacer trusses. Figure 8 shows the assembly of a mini-module at the bridge site. After the trusses were nailed together, the mini-modules were again banded together with light metal straps. The mini-modules were lifted by a crane and placed side by side to form the bridge deck. They were positioned to form six rectangular modules to account for the 18° skew. To hold the trusses in position, several steel bars were inserted through holes in the chords as the modules were assembled. After all the modules were assembled, the remaining steel stressing bars were inserted through predrilled holes in the truss chords and through some web openings. Steel bearing channels were installed on the edge of the bridge, followed by the structural tubes, anchor plates, and nuts (Fig. 9).

After installation of the stressing system, the bridge was initially stressed to half the design value (63 lb/in²). During stressing, each bar was individually tightened to the desired stress value, using a hydraulic jack; the bars were tightened

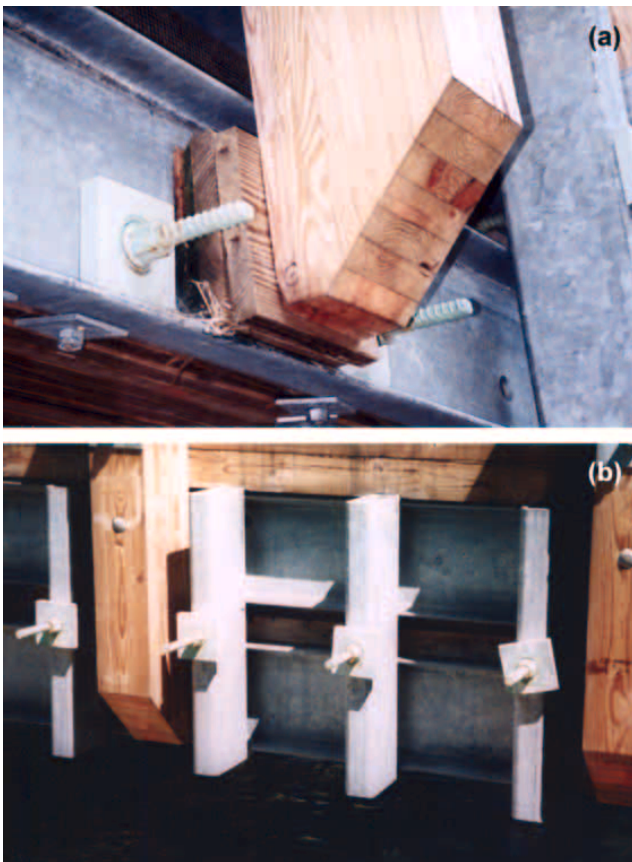


Figure 5—Anchorage system for Byron bridge: (a) continuous steel channel and discrete plate bar configuration; (b) continuous steel channel and vertical structural tube bar configuration.



Figure 6—Brush painting of galvanized metal plate connectors of each truss at site for additional protection from corrosion.



Figure 9—Installation of stressing system.



Figure 7—Placement of painted trusses onto temporary supports for formation of mini-modules.



Figure 10—Tightening of stressing bars during stressing.



Figure 8—Preassembly of laminations into mini-modules at bridge site.

sequentially, beginning at one end of the bridge (Fig. 10). This procedure was repeated until the bars held the desired stress of 63 lb/in². The bars were retensioned at 1 week, 6 weeks, 9 months, 22 months, and 54 months after installation to the full design stress of 125 lb/in² using the same procedure.

After initial stressing was completed, we noted that the width of the bridge measured 31.6 ft, 5.5 in. less than specified in the design. As a result, four additional trusses were added to increase the width of the bridge. The width of a stress-laminated deck is typically increased during fabrication to compensate for anticipated losses caused by high compressive stresses during the stress-laminating process. The reduction of bridge width was probably due to underestimating the amount of compression within the gaps created from the metal plate connectors. The superstructure was attached by bolting 5- by 5- by 0.5-in. steel angles with bolts spaced

every 24 in. to the side of the substructure abutments, then bolting the bridge to the steel angles (Fig. 11).

After the superstructure was attached, construction of the concrete backwalls was completed. The glulam curb and rails were installed shortly after the backwalls were poured (Fig. 12). To protect the bridge from moisture, one coat of adhesive primer was painted directly onto the deck, followed by the installation of two layers of self-sealing waterproof membrane. The waterproof membrane was wrapped over the backwalls to completely seal the top surface of the structure from moisture (Fig. 13). The bridge was paved with a 1-in. leveling course and a variable thickness bituminous pavement, measuring 3.5 in. at the crown and 1.5 in. at the curb. The completed Byron bridge is shown in Figure 14.

Evaluation Methodology

To evaluate the field performance of the Byron bridge, a 5-year monitoring 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 truss moisture content, bar force, and general condition. Procedures and equipment for the evaluation were developed previously and had been used on similar structures (Caccese and others 1991, 1993; Dagher and others 1991; Ritter and others 1991).

Moisture Content

The moisture content of the Byron bridge trusses 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 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 when the bridge was installed, during the second load test, and at the conclusion of the monitoring period.

Bar Force

During the scheduled retensionings, bar force was measured with calibrated steel load cells developed at the University of Maine (Fig. 15) and with a hydraulic jack. The load cells were installed on six bars prior to the initial construction stressing. 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 anchorage 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 11—Superstructure was attached by bolting steel angles to sides of abutments, then bolting bridge to steel angles.



Figure 12—Bridge after construction of backwalls and installation of rail post and curb.



Figure 13—Installation of waterproof membrane.



Figure 14—Completed Byron bridge: (a) profile of completed bridge (looking east); (b) completed bridge just after paving (looking south).



Figure 15—Load cells were installed on stressing bars to monitor changes in bar tension.



Figure 16—Bridge deflections of load tests 1 and 2 were measured using displacement transducers attached to a wood beam supported by scaffolding.

Behavior Under Static Load

Static load testing was conducted immediately before the bridge was opened to traffic and 21 months later to determine the response of the bridge to highway truck loads. In addition, the maximum predicted deflection was determined for each load test based on static analysis for actual and HS25–44 loading. Load testing involved positioning one or two fully loaded dump trucks, weighing between 54,000 and 65,000 lb, on the bridge and measuring the resulting deflections at a series of locations along the bridge centerspan and abutment cross sections. Deflection measurements were obtained under the centerspan of the bridge using displacement transducers mounted to a temporary support consisting of two 2 by 8's nailed together and supported every 6 ft (Fig. 16). 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 (November 25, 1993) consisted of four load cases and used one fully loaded three-axle dump truck with a gross vehicle weight of 64,750 lb (Fig. 17). For load cases 1 and 2, the truck was positioned transversely 2 ft from the centerline of the bridge. For load cases 3 and 4, the truck was positioned transversely 6.5 ft from the roadway centerline. For all load cases, the truck center of gravity was positioned at midspan and deflections were measured to within 0.01 in.

Load Test 2

Load test 2 (August 7, 1995) consisted of six load cases and used two fully loaded three-axle dump trucks: truck A with a

gross vehicle weight of 54,300 lb, and truck B with a gross vehicle weight of 54,100 lb (Fig. 18). For load cases 1, 2, and 3, the trucks were positioned transversely 2 ft from the centerline of the bridge. For load cases 4, 5, and 6, the trucks were positioned transversely 6.5 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. Load cases 3, 4, and 6 are shown in Figure 19.

Condition Assessment

The general condition of the Byron bridge was assessed three times: (1) during the first load test (November 25, 1993), (2) during the second load test, after 21 months of service (August 7, 1995), and (3) after approximately 4½ years of service (April 6, 1998). The condition assessments involved visual inspections, measurements, and photographic documentation. Items of specific interest included the bridge geometry, wood components, wearing surface, prestressing system, and metal plate connectors.

Results and Discussion

The following results are based on data collected during the 5-year monitoring program for the Byron bridge.

Moisture Content

The average trend in wood moisture content is presented in Figure 20; measurements were taken in September. When monitoring was initiated, average wood moisture content was approximately 22%. After 20 months of service, moisture content decreased to 16%, and after 53 months, it stabilized at approximately 14%. Moisture content measurements and visual inspections of the wood indicated that the waterproof membrane and pavement crown were effective in protecting the bridge from water.

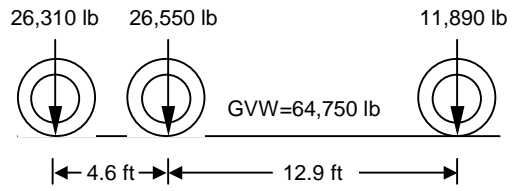
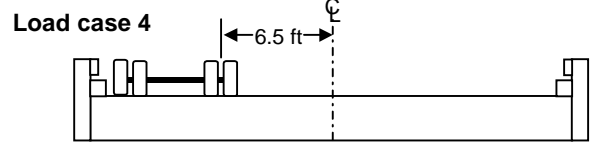
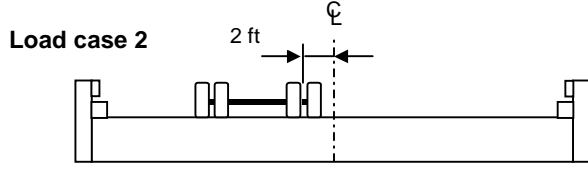
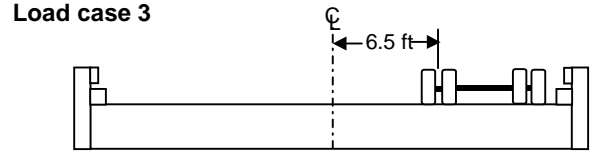
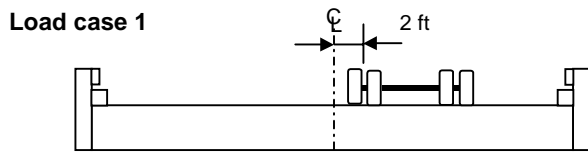


Figure 17—Truck weights, axle spacings, and transverse load test positions for load test 1.

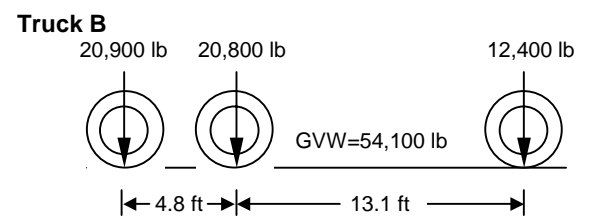
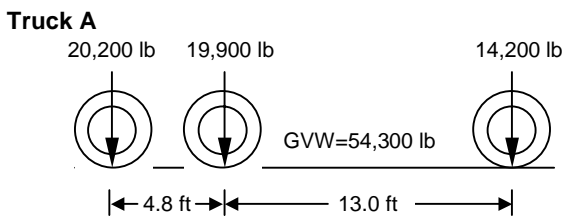
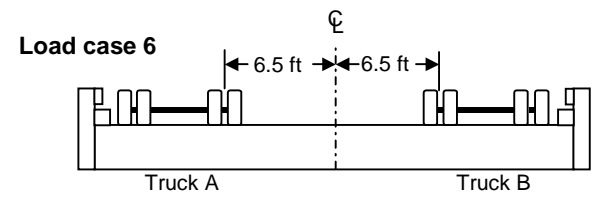
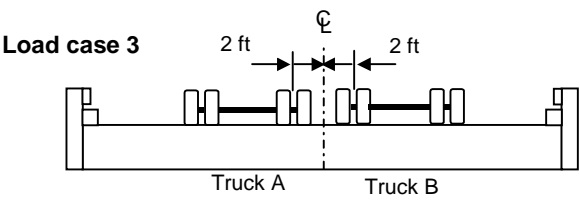
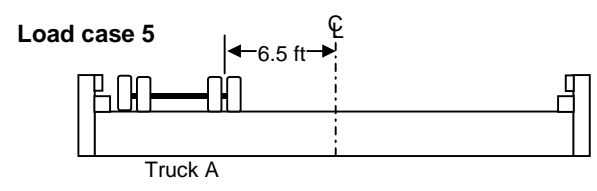
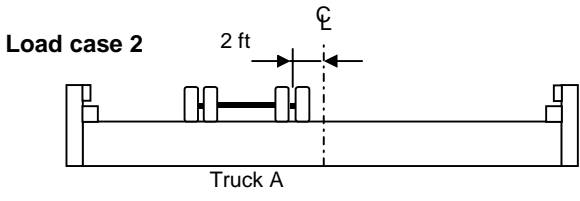
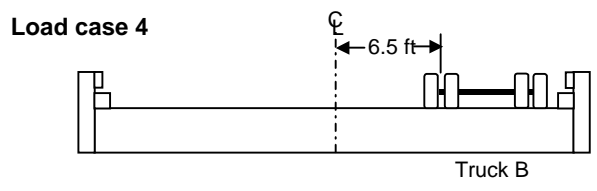
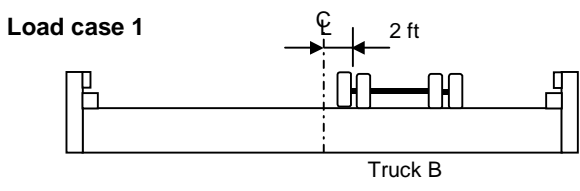


Figure 18—Truck weights, axle spacings, and transverse load test positions for load test 2.



Figure 19—Truck positions for load test 2 for three load cases (looking north): (a) load case 3, (b) load case 4, (c) load case 6.

Bar Force

The average trend in bar force is shown in Figure 21; measurements were taken in September. 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 as a result of the combined effects of a decrease in wood moisture content, stress relaxation, and seating of the metal plate connectors. Therefore, the bridge was retensioned to the full design value

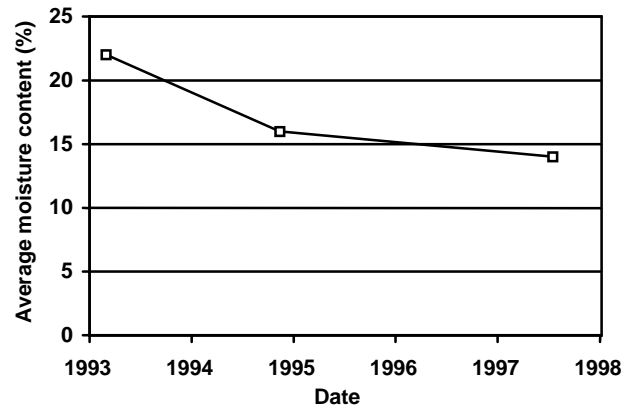


Figure 20—Average trend in wood moisture content.

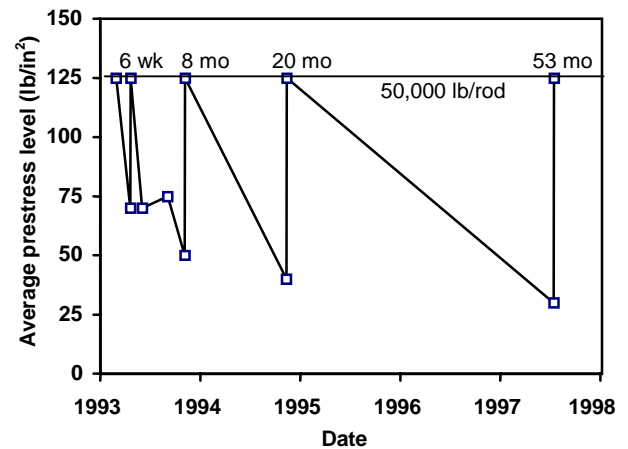


Figure 21—Average trend in prestress level of bar force.

of 50,000 lb or 125 lb/in² interlaminar compression after 6 weeks, 8 months, 20 months, and 53 months of service.

Data collected during the first retensioning indicated that the average bar force had decreased 45% to approximately 70 lb/in² interlaminar compression during the first 6 weeks. Data collected during the second retensioning 6½ months later indicated that the average bar force had decreased 60% to approximately 50 lb/in² interlaminar compression.

Measurements taken 12 months after the second retensioning indicated that the bar force had decreased 68% to approximately 40 lb/in² interlaminar compression. The bars were retensioned again. Bar force measurements taken 33 months after this retensioning indicated that the bar force had decreased 75%, to approximately 30 lb/in² interlaminar compression. Therefore, the bars were tensioned again.

The 8% decrease in moisture content caused wood shrinkage; shrinkage was probably most significant during the first half of the monitoring period, when the greatest moisture content loss occurred. Stress relaxation in wood laminations has been observed to cause bar force loss in many other stress-laminated bridges (Ritter and others 1991). Shrinkage and

stress relaxation are primary sources of bar force loss in solid stress-laminated decks. However, additional bar force loss was expected for the Byron bridge structure because of the seating of the metal plate connectors into the wood of adjacent trusses.

At the time of the design and construction of the bridge, the magnitude and duration of the bar force loss were unknown. However, data collected during the 53 months of monitoring showed that the bar force has not stabilized. Although the rate of prestress loss decreased with each retensioning, bar force still decreased approximately 75% to 30 lb/in² in the 33 months prior to the last retensioning. Because of the ongoing bar force loss, it is recommended that the bar force be assessed annually.

The AASHTO guide specifications for stress-laminated bridges apply only to solid-sawn wood slab decks and not to trusses; therefore, an AASHTO minimum bar force could not be directly obtained for this bridge. However, by assuming that the sum of the thickness of the top and bottom chords is equivalent to the thickness of the bridge deck, the AASHTO

guide specifications require a minimum bar force of approximately 45 lb/in² for this bridge. Therefore, it is recommended that the bars be retensioned to the design value of 125 lb/in² whenever the bar force is below 45 lb/in².

Behavior Under Static Load

For each load case, transverse deflections are given at the bridge centerspan 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 both load tests, the average bridge interlaminar compressive stress was 125 lb/in². The allowable deflection for design purposes was L/500 or 1.08 in.

Load Test 1

Transverse deflections for load test 1, which utilized one fully loaded dump truck and four load cases, are presented in Figure 22. The maximum deflection for load cases 1 and 2

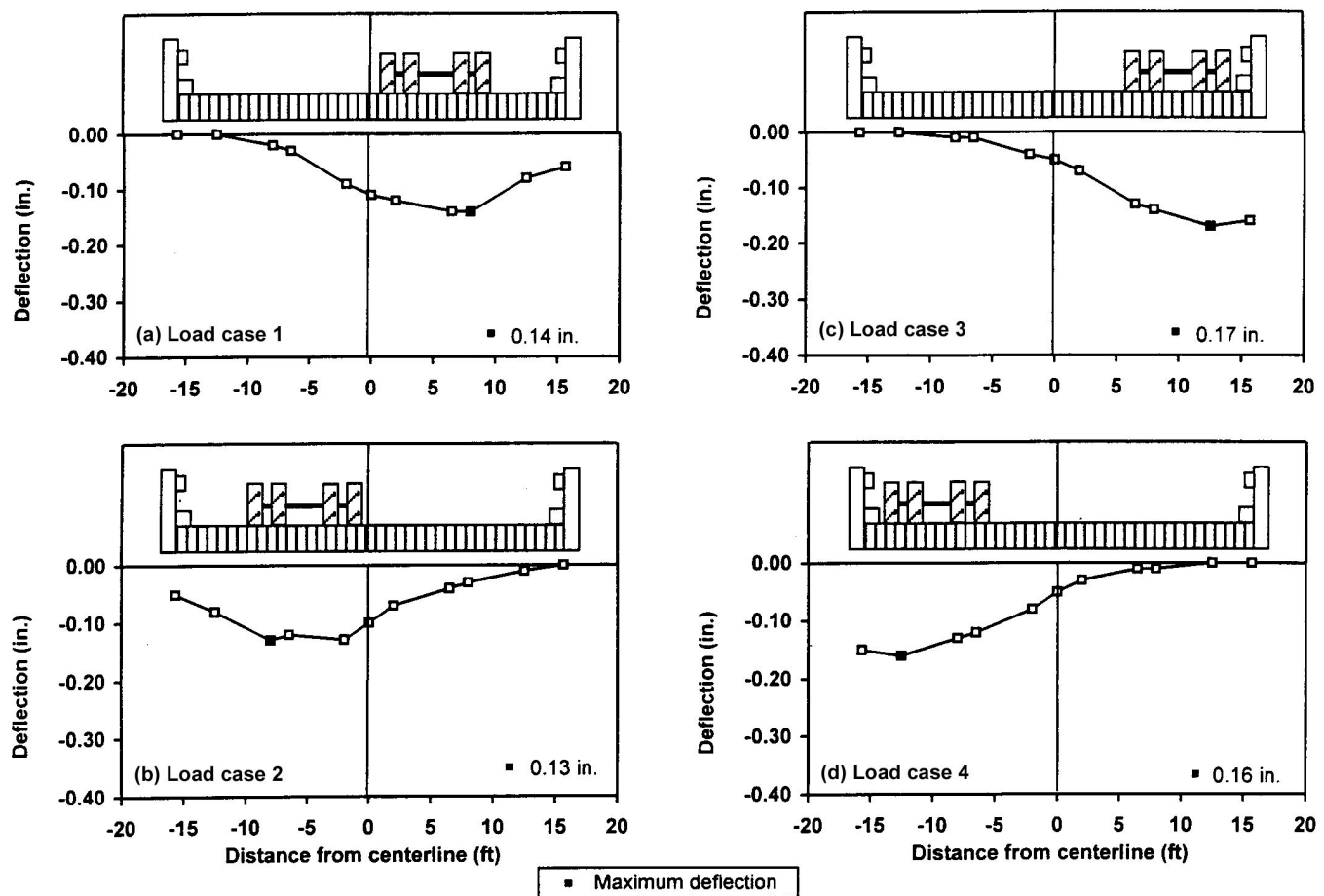


Figure 22—Transverse deflections for load test 1, measured at bridge centerspan (looking north). Bridge cross section and vehicle positions are for the purpose of interpretation only and are not drawn to scale.

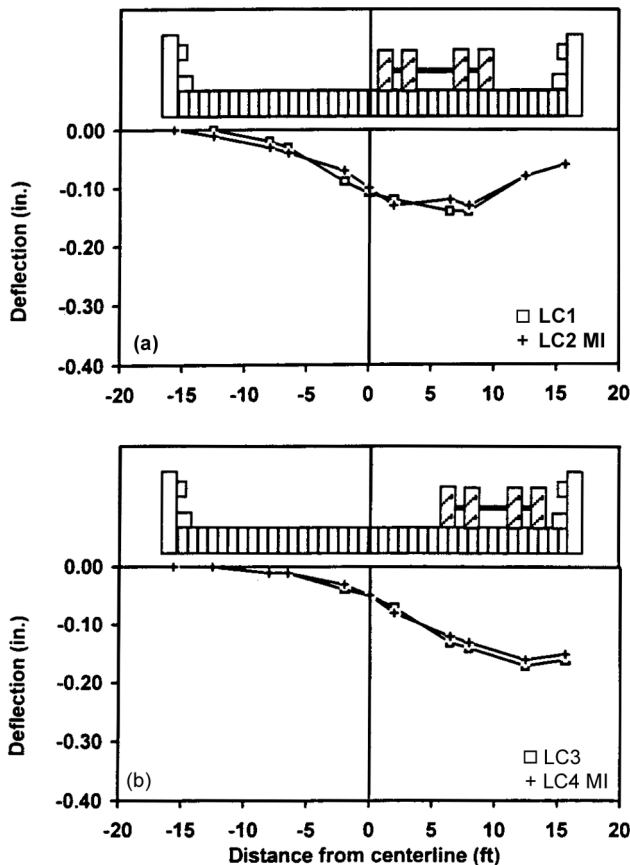


Figure 23—Measured deflections for load test 1: (a) actual deflection for load case 1 (LC1) and mirror image of load case 2 (LC2 MI); (b) LC3 actual deflection and LC4 MI.

occurred under the outside truck wheel line and measured 0.14 and 0.13 in., respectively (Fig. 22a,b). Maximum deflections for load cases 3 and 4 occurred under the outside truck wheel line and measured 0.17 and 0.16 in., respectively (Fig. 22c,d)

For these four load cases, the deflected shape of the center-span cross section followed the symmetrical truck positions, with maximum measured deflection occurring at the same relative positions.

Only one truck was available for load test 1; therefore, experimental data are not directly available for two trucks on the bridge. However, assuming accurate load test results and linear elastic behavior, the sum of the deflections resulting from individual truck loads should equal the deflection from two trucks applied simultaneously. To verify the accuracy of the test results, the deflections from load case 1 were compared with the mirror image of the deflections from load case 2, and the deflections from load case 3 were compared with the mirror image of the deflections from load case 4.

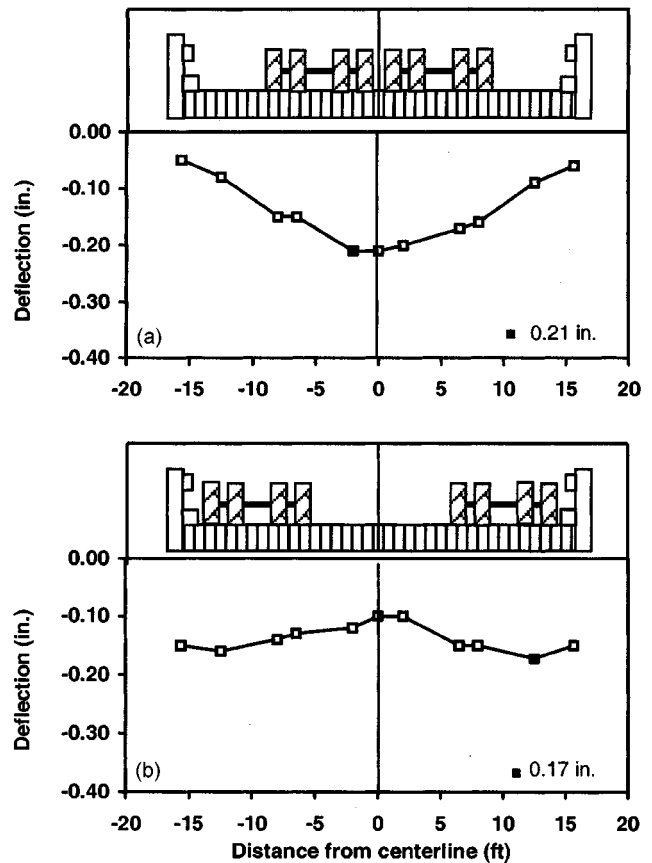


Figure 24—Sum of deflections for load test 1: (a) load cases 1 plus 2, (b) load cases 3 plus 4.

As shown in Figure 23, the plots are nearly identical, verifying the accuracy of the test results. The linear elastic behavior of the bridge under truck loading was verified in load test 2, when two trucks were available for the load test.

Figure 24 shows the deflections resulting from the sum of load cases 1 plus 2 and the deflections resulting from the sum of load cases 3 plus 4. For load cases 1 plus 2, maximum deflection (0.21 in.) occurred under the inside wheel line of the southbound lane. For load cases 3 plus 4, maximum deflection (0.17 in.) occurred under the outside wheel line of the northbound lane.

Load Test 2

Transverse deflections for load test 2, which utilized two fully loaded dump trucks and six load cases, are shown in Figure 25. The maximum deflection of 0.11 in. for load case 1 occurred under the outside truck wheel line (Fig. 25a), and the maximum deflection of 0.13 in. for load case 2 occurred under the outside truck wheel line (Fig. 25b).

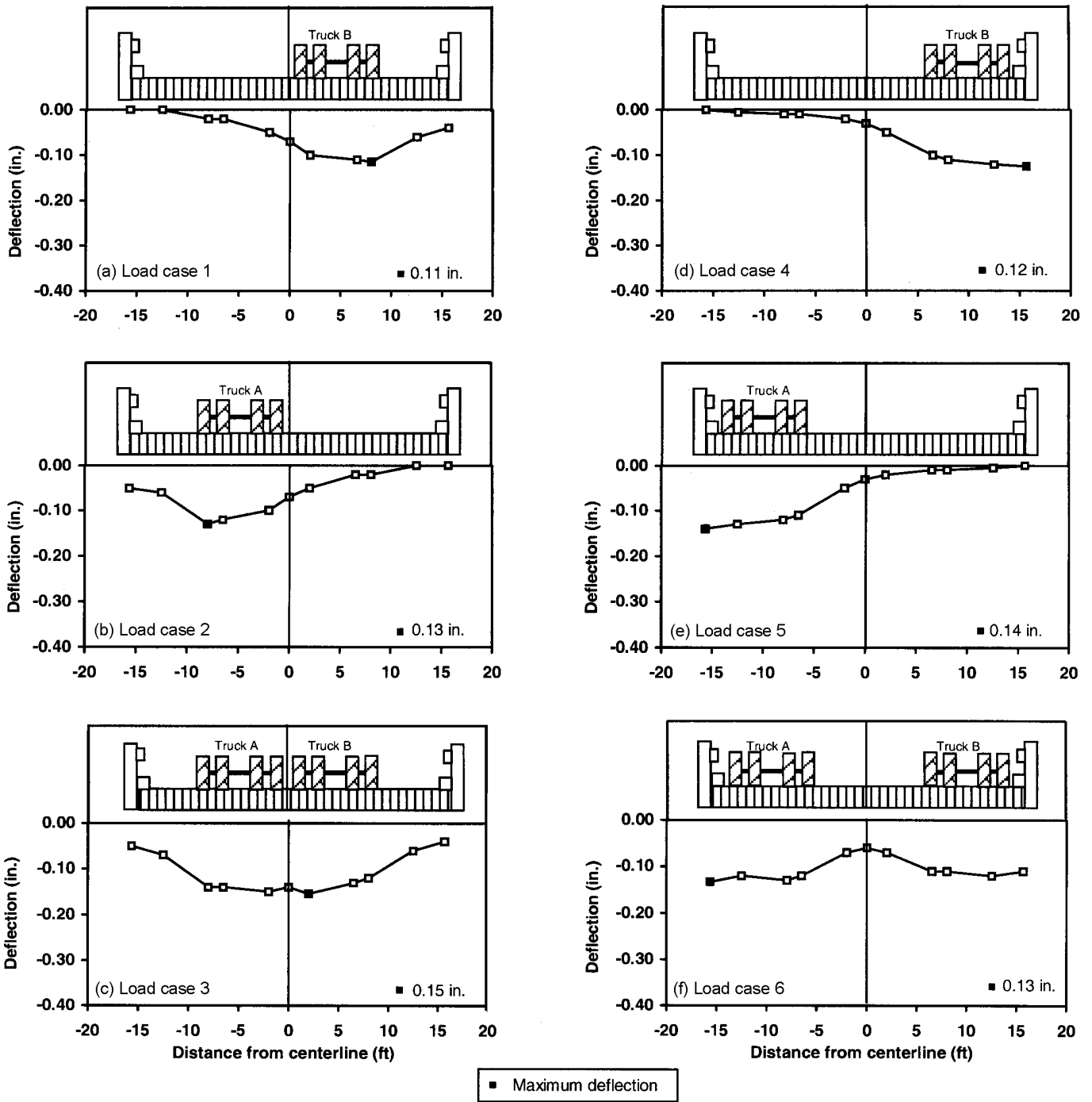


Figure 25—Transverse deflections for load test 2, measured at bridge centerspan (looking north). Bridge cross section and vehicle positions are for the purpose of interpretation only and are not drawn to scale.

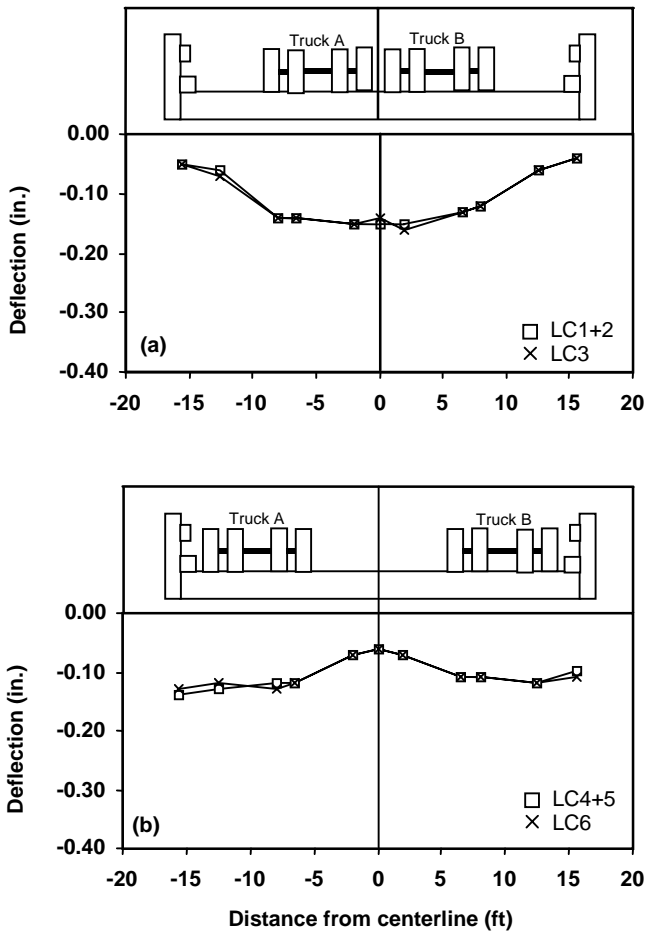


Figure 26—Measured deflections for load test 2: (a) sum of measured deflections for LC1 and LC2 compared with measured deflections for LC3; (b) sum of measured deflections for LC4 and LC5 compared with measured deflections for LC6.

The maximum deflection of 0.15 in. for load case 3 occurred under the outside truck wheel line. For load case 3, maximum deflections occurred under the inside truck wheel line of truck B, adjacent to the centerline of the bridge, and represented the largest deflection for all load cases (Fig. 25c). Maximum deflections for load cases 4 and 5 occurred at the bridge edge adjacent to the outside truck wheel line and measured 0.12 and 0.14 in., respectively (Fig. 25d,e). The maximum deflection of 0.13 in. for load case 6 occurred at the bridge edge, adjacent to the outside wheel line of truck A (Fig. 25f).

For load cases 1 through 4, the deflected shape of the center-span cross section followed the symmetrical truck positions, with maximum measured deflections occurring at the same relative positions.

Assuming accurate load test results and linear elastic behavior, the sum of the deflections resulting from individual truck loads should equal the deflections from both trucks applied

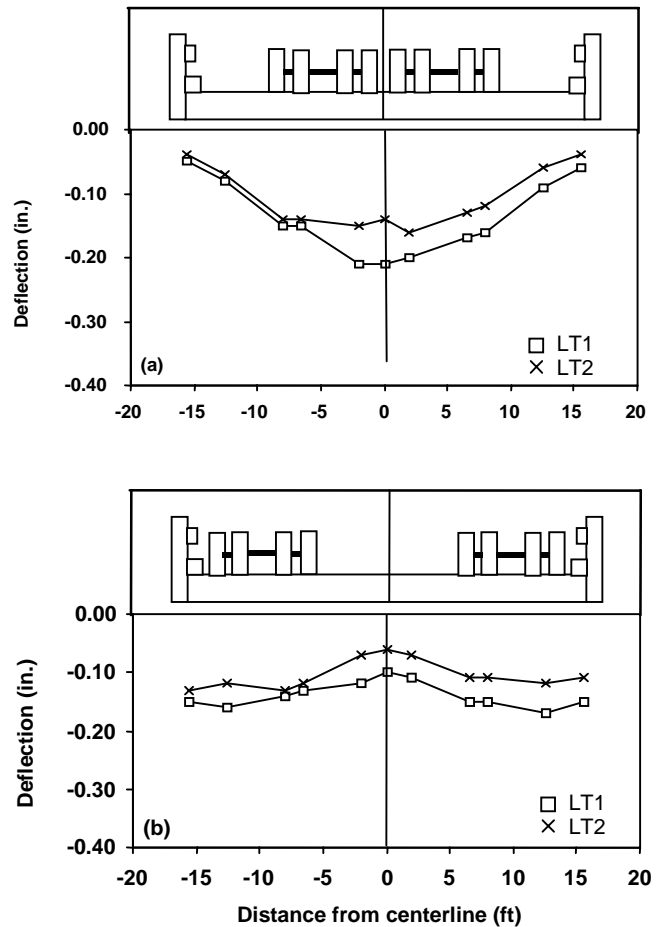


Figure 27—Comparison of deflections for load tests: (a) Load test 1 (LT1), LC1+3 compared with LT2, LC3; (b) LT1, LC3+4 compared with LT2, LC6.

simultaneously. Figure 26 shows the load test 1 comparison of individual and simultaneous truck loading. As shown in Figure 26, the two plots are nearly identical with only minor variations, which are within the accuracy of the measurements. From this information, we conclude that the bridge behavior is within the linear elastic range.

Load Test Comparison

Figure 27 compares measured deflections for both load tests for two trucks on the bridge. The plots are similar in shape, but the deflections measured in load test 1 are greater than those measured in load test 2. At the time of testing, average bridge interlaminar compressive stress was 125 lb/in². The difference in the deflections is probably due to the greater weight of the trucks in load test 1, approximately 20% more than that of the trucks in load test 2. Another contributing factor was a slight difference in the type of trucks used for the tests, which could have resulted in a slight variation in the deflections.

Table 3—Summary of load test and predicted HS25–44 midspan deflections

Load test	Maximum measured load test deflection (in.)	Maximum predicted load test deflection (in.)	Maximum predicted HS25–44 deflection (in.)	Span/deflection ratio for predicted HS25–44 deflection
1	0.21	0.31	0.43	L/1200
2	0.15	0.26	0.43	L/1200

Predicted Response

Table 3 summarizes the maximum measured and predicted deflections for both load tests and the predicted maximum deflections for AASHTO HS25–44 truck loading.

The maximum measured deflections were expected to be less than the predicted deflections because of (a) the conservatism of the design procedure and (b) the performance of the load tests when the bridge was stressed to the full design value of 125 lb/in². The maximum measured deflection for load test 1, 0.21 in., was 0.10 in. less than the 0.31-in. deflection predicted by the analysis model for loading. The maximum measured deflection for load test 2, 0.15 in., was 0.11 in. less than the 0.26-in. deflection predicted by the analysis model for loading. The maximum predicted deflection of the Byron bridge under HS25–44 truck loading, 0.43 in., resulted in a span/depth ratio of approximately L/1200, which is within the design limit of L/500 or 1.08 in.

Condition Assessment

Assessment of the general condition of the Byron bridge indicated that its structure and serviceability are satisfactory. The areas subjected to inspection were bridge geometry, wood components, wearing surface, anchorage system, and metal plate connectors.

Bridge Geometry

Width measurements taken at the initiation of monitoring indicated that the stress-laminated truss structure was 2 in. narrower at the south abutment than at the north abutment. This was probably due to the sequential tightening of bars with a single jack. The slight distortion was reduced to 1.5 in. during the monitoring period and should not affect overall bridge performance.

Wood Components

Visual inspection of the wood components of the bridge indicated no signs of deterioration. However, checking was noted in the guardrail post during the second condition assessment, and each post was subsequently covered with a thin metal cap. In addition, 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-based paint.

Metal Plate Connectors

The metal plate connectors are performing well. They showed no signs of deterioration or rust on inspection.

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 Byron bridge is satisfactory. With the exception of having to be retensioned periodically, there are no structural or serviceability deficiencies evident in the bridge.
- The moisture content decreased gradually from approximately 22% to 14% during the 5-year field monitoring. Based on moisture content readings and visual inspections, it is concluded that the waterproof membrane and pavement crown have been effective in protecting the bridge from moisture.
- During performance monitoring, the bridge was retensioned to the full design value of 125 lb/in² four times. After 53 months of monitoring, the bar force has not stabilized above the 45 lb/in² minimum recommended. This ongoing bar loss for more than 4 years is not typical of stress-laminated decks and is probably caused by a narrowing of the gaps between the truss laminations caused by the thickness of metal connector plates. The rate of bar force loss has decreased with each retensioning; therefore, it is expected that the bar force will eventually stabilize.
- Based on data collected, the transverse load distribution analysis is conservative when used to analyze this bridge. This analysis assumes that normal to the direction of the span, the wheel load is distributed over the width of the tire plus twice the thickness of the top chord of the deck. For deflection calculations, the distribution width was increased by 15%, paralleling the AASHTO recommendation for solid stress-laminated decks (AASHTO 1991).

Results from two load tests indicate that at a bar force of 125 lb/in², the transverse load distribution analysis over predicted the maximum deflection by an average of 37%. The results from laboratory testing indicate that at a bar force of 25 lb/in², the transverse load distribution analysis over predicted the maximum stress by 39% and the maximum deflection by 28%.

- The predicted deflection for AASHTO HS25-44 loading using the transverse load distribution analysis was 0.43 in. or L/1250. This is well below the design limit of L/500 or 1.08 in., where L is the span length measured center-center of bearings.
- For load test 1, the maximum deflection from two 65,000-lb trucks positioned with their center of gravity at midspan was calculated to be 0.21 in. or L/2570 when the bridge was stressed to the full design value of 125 lb/in². Two trucks were not available for load test 1; therefore, this result was obtained by adding the deflection from one truck positioned in the northbound lane to the deflection from the same truck positioned in the southbound lane.
- For load test 2, the maximum deflection from two 54,000-lb trucks positioned with their center of gravity at midspan was measured to be 0.15 in. or L/3600 when the bridge was stressed to the full design value of 125 lb/in².
- Static load testing indicates that the Byron bridge is performing in the linear elastic range when subjected to two 54,000-lb trucks positioned with their center of gravity at midspan when the bridge was stressed to the full design value of 125 lb/in².
- Visual inspections indicate no signs of deterioration of the wood or metal plate connectors. Surface rust is visible on some of the stressing system hardware in the vicinity of the anchorage nuts.
- Assessment of the moisture content, bar force, and general condition of the bridge components, including the metal plate connectors, should be performed on an annual basis.
- During annual assessment, the bars should be retensioned to the full design value of 50,000 lb if the bar force is between 10,000 and 18,000 lb or 25 and 45 lb/in² interlaminar compression. Under no circumstances should the bar force be permitted to decrease below 10,000 lb or 25 lb/in² interlaminar compression.
- 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

Name: Byron bridge
Location: Byron, Maine
Date of construction: November 1993
Owner: Maine Department of Transportation

Design Configuration

Structure type: Stress-laminated trusses with metal plate connectors
Butt-joint frequency: 1 in 2 laminations transversely separated 22 in. longitudinally
Total length (out–out): 45.9 ft
Skew: 18°
Number of spans: 1
Span length (center–center of bearings): 44 ft
Width (out–out): 31.6 ft (as built)
Number of traffic lanes: 2
Design loading: AASHTO HS25–44
Camber: 0 in.
Wearing surface: asphalt pavement, 1.5 to 3.5 in. thick

Material and Configuration

Truss Laminations:
Species: machine-stress-rated (MSR) Southern Pine
Size and grade: 2 by 12 in. MSR 1950f–1.7E
2 by 10 in. MSR 2250f–1.9E
2 by 8 in. MSR 2250f–1.9E
2 by 6 in. MSR 2400f–2.0E
Moisture condition: approximately 22% at time of construction
Preservative treatment: CCA–type III
Metal plate connectors: galvanized, 20-gauge plates (MiTek Industries, St. Louis, MO), brush painted with Series 27 FC epoxy (Tnemec Company, Inc., Kansas City, MO)

Stressing Bars:

Diameter: 1.0 in.
Number: 28 partial-width bars, 18 full-width bars
Design force: partial-width bars, 25,000 lb;
full-width bars, 50,000 lb
Spacing (center–center): partial-width bars, 2 bars every 22 in.; full-width bars, 1 bar every 22 in.
Type: High strength steel-threaded bar, epoxy coated (Dywidag Systems International, Lincoln Park, NJ)

Bar Anchorage Type:

Continuous C 12 × 30 grade-50 all-weather steel channels on top and bottom chords with two types of bearings:
Type 1: 6- by 6- by 1.25-in. steel anchorage plates
Type 2: 10- by 4- by 0.50-in. grade-50 all-weather steel tubes and 6- by 6- by 1.25-in. steel anchorage plates

Rail and Curb System:

Design: crash-tested at AASHTO Performance Level 1 on longitudinal spike-laminated deck
Species: Southern Pine
Member sizes: Rails: 6- by 12-in. glulam
Posts: 8- by 12-in. glulam
Curbs: 12- by 12-in. sawn lumber

Preservative treatment: CCA–type III

Waterproof Membrane System:

80A adhesive primer
M400A self-sealing waterproof membrane