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Field Performance of Timber Bridges

11. Spearfish Creek Stress-Laminated Box-Beam Bridge

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Abstract

The Spearfish Creek bridge was constructed in 1992 in Spearfish, South Dakota. It is a single-span, stress-laminated, box-beam superstructure. Performance of the bridge is being monitored for 5 years, beginning at installation. This report summarizes results for the first 3-1/2 years of monitoring and includes information on the design, construction, and field evaluations of the wood moisture content, force level in the stressing bars, behavior under static loading, and overall structure condition. Based on field evaluations, the bridge is performing satisfactorily with no structural or serviceability deficiencies. However, two bridge restressings were performed due to excessive bar force loss.

Keywords: Timber, wood, bridge, performance, stress laminated, box beam

Acknowledgments

This project was a cooperative effort between the South Dakota Department of Transportation; the City of Spearfish, South Dakota; the USDA Forest Service, Forest Products Laboratory (FPL); and the Federal Highway Administration.

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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 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 Timber Bridge Information Resource Center (TBIRC) in Morgantown, West Virginia. Under the demonstration program, the TBIRC provides matching funds to local governments to construct demonstration timber bridges, which 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 employ design innovations or materials that have not been previously evaluated. Through such assistance, FPL is able to collect, analyze, and distribute information on the field performance of timber bridges, 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 included provisions for a timber bridge program aimed at improving the utilization of wood transportation structures. Responsibility for the development, implementation, and administration of the ISTEA timber bridge program was assigned to the Federal Highway Administration (FHWA) and included demonstration timber bridge, technology transfer, and research programs. Because many aspects of the FHWA research program paralleled those underway at FPL, a joint effort was initiated to combine the respective research of the two agencies into a central research program. As a result, the FPL and FHWA merged resources to jointly develop and administer a national timber bridge research program.

This report, eleventh in a series, documents the field performance of the Spearfish Creek bridge located in Spearfish, South Dakota. It summarizes the design, construction, cost, and field evaluation for the first 3-1/2 years of a 5-year FPL/FHWA monitoring project of the Spearfish Creek bridge. This bridge is a two-lane, single-span, stress-laminated box-beam superstructure that is 65 ft long, 39 ft wide, and 31.5 in. deep. (See Table 1 for metric conversion factors.) Built in 1992, the Spearfish Creek bridge was constructed as a TBI demonstration bridge with funds provided by the South Dakota Department of Transportation (DOT), USDA Forest Service, and the City of Spearfish. An information sheet of specific bridge characteristics is provided in the Appendix.

Table 1—Factors for converting English units of measurement to SI units

English 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)

Objective and Scope

The objective of this project was to evaluate the field performance of the Spearfish Creek bridge for a minimum of 5 years, beginning shortly after installation of the bridge superstructure. The scope of the project includes data collection and analysis related to wood moisture content, stressing bar force, behavior under static truck loading, and general structure performance. Results of this project will be considered with similar monitoring projects in an effort to improve design and construction methods for future stress-laminated timber bridges.

Background

The Spearfish Creek bridge site is located in Spearfish, South Dakota (Fig. 1). The bridge is adjacent to the Historic D.C. Booth Fish Hatchery, where Canyon Street crosses over Spearfish Creek. The roadway over the bridge is a two-lane, paved road that provides access to the fish hatchery and a nearby city campground. The average daily traffic across the bridge varies because of the summer tourism but is estimated at 800 vehicles per day.

Before replacement in 1992, the Spearfish Creek bridge was a 71-ft long, single-lane, steel pony truss superstructure on a timber substructure (Fig. 2). Initially constructed in the early 1950s, the steel trusses were badly corroded and the bridge could no longer safely carry standard highway loads. The single-lane bridge also posed a hazard to the two-way traffic flow on Canyon Street. After considering several replacement options, it was determined that a timber bridge replacement would be most appropriate for the site. The city council supported this replacement option in order to accent the natural setting of the campground and the Historic D.C. Booth Fish Hatchery.

Through a cooperative effort between the South Dakota DOT and the Spearfish Department of Public Works, a proposal was submitted to the USDA Forest Service for partial funding of the replacement structure. The project proposed a stress-laminated box-beam configuration, utilizing Ponderosa Pine lumber and Southern Pine glued-laminated (glulam) timber beams. In 1991, the project received funding and plans for the design and construction of the Spearfish Creek bridge were finalized. Subsequently, FPL/FHWA provided assistance in developing and implementing a field evaluation program to monitor bridge performance.

Design, Construction, and Cost

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

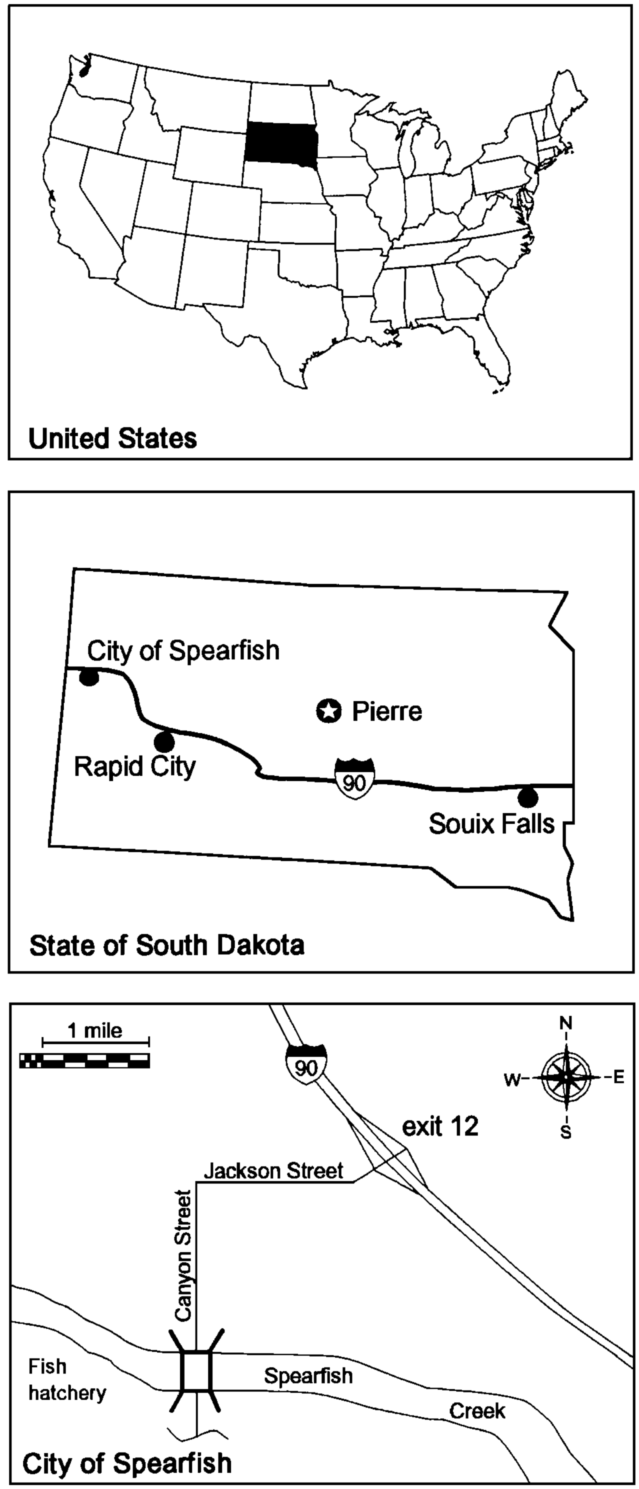

Figure 1—Location maps for the Spearfish Creek bridge.

Figure 2—Original Spearfish Creek bridge constructed in the early 1950s.

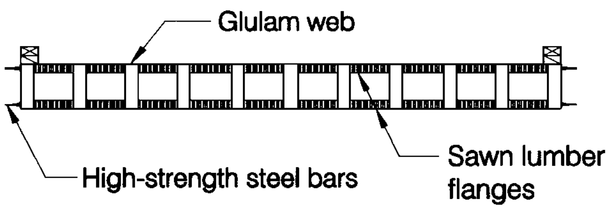

Figure 3—Cross-section of a typical stress-laminated box-beam bridge.

Design

Design of the Spearfish Creek bridge was completed by the South Dakota DOT 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, it is assumed that the components act together as a single unit.

With the exception of those features related to the stress-laminated box-beam, design of the Spearfish 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 stress-laminated box-beams 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 65 ft long, 39 ft wide, and 31.5 in. deep, with a pedestrian walkway along the upstream side (Fig. 4). Design calculations were based on a 64-ft span length (center–center of bearings) and a 36-ft clear roadway width. Design of the glulam webs and 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 65 ft long and 31.5 in. deep. To allow for prefabrication into modular bridge units, glulam web widths of 5.125 and 8.75 in. were specified. Sawn lumber flange laminations were specified as nominal 2- by 6-in., visually graded No. 2 Ponderosa Pine, pressure treated with pentachlorophenal in heavy oil. 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 limited to no more than one in every four adjacent laminations. Longitudinally, rows of butt joints were spaced at 3-ft intervals.

For stress laminating, the design specified 0.625-in. diameter, ASTM A722 high strength, threaded steel bars with an ultimate strength of 150,000 lb/in^2 (ASTM 1988). The bars were spaced at 34-in. intervals, beginning 38 in. from the bridge ends. The design bar tension force of 19,600 lb provides an interlaminar compressive stress of 100 lb/in^2. Bar anchorage was with a discrete-plate anchorage system, consisting of 5.75- by 11- by 0.50-in. steel bearing plates, 2- by 5- by 1-in. steel anchor plates, and hexagonal nuts. All components of the stressing system and other 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 to be full-span, glulam, measuring 6 by 10.75 in. Rail posts were designated as visually graded Dense Select Structural Douglas Fir sawn lumber, measuring 8 by 12 in. (nominal), and were spaced 68 in. on-center. The curbs were visually graded No. 2 Douglas Fir sawn lumber, measuring 6 by 12 in. (nominal).

To compensate for dead load and creep deflection, a 6-in. camber was specified for the glulam web members. To protect the bridge from deterioration, a 2- to 3-in.-thick asphalt wearing surface with a waterproof geotextile membrane was specified.

Construction

Construction of the Spearfish Creek bridge by a local contractor began in June 1992. During construction, a temporary crossing consisting of corrugated steel culverts was installed just upstream of the bridge site. The existing bridge was then removed and salvaged for use as pedestrian crossings. Following removal of the existing bridge, new reinforced concrete abutments and wingwalls were constructed (Fig. 6).

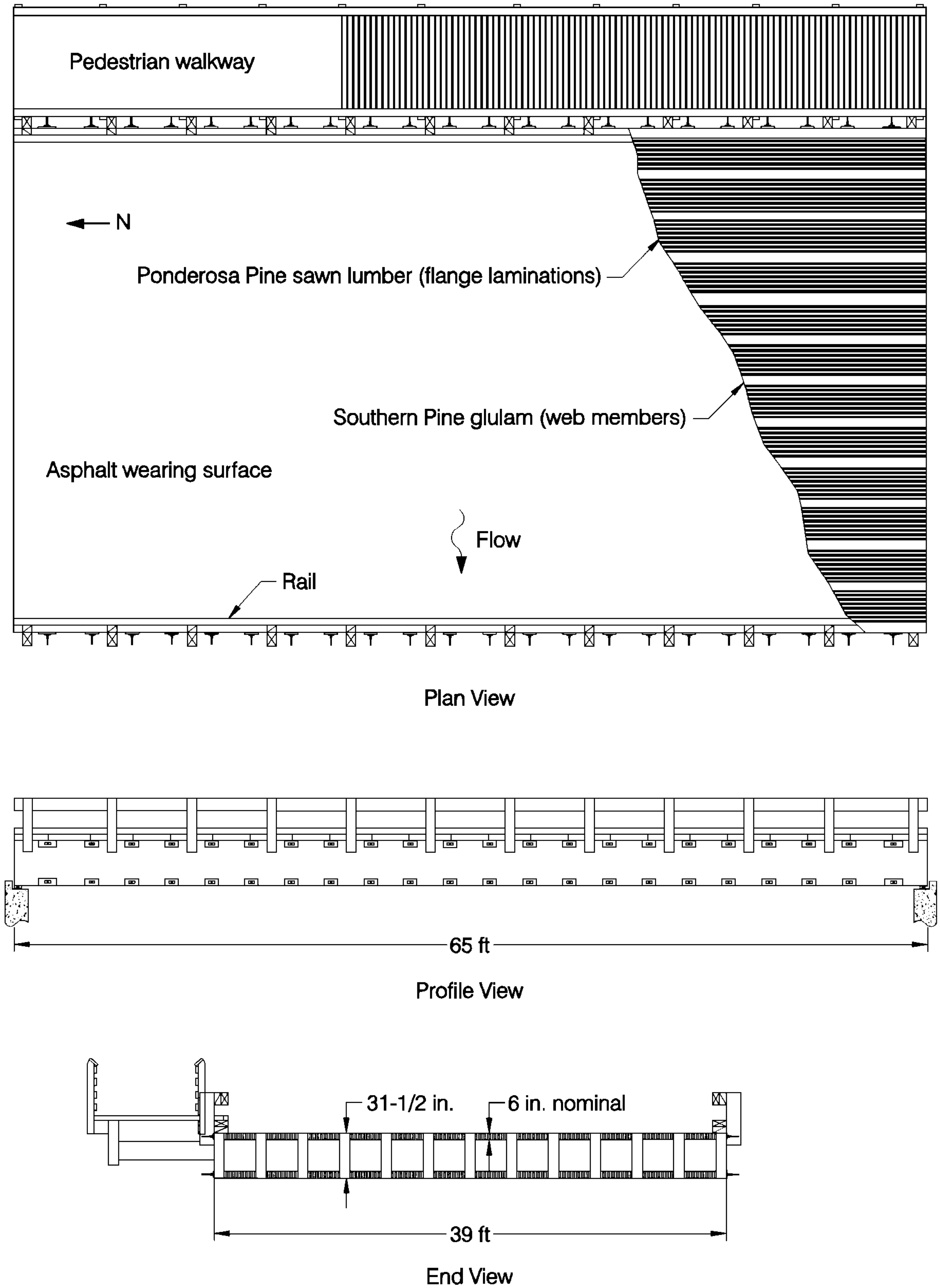

Figure 4—Design geometry of the Spearfish Creek bridge.

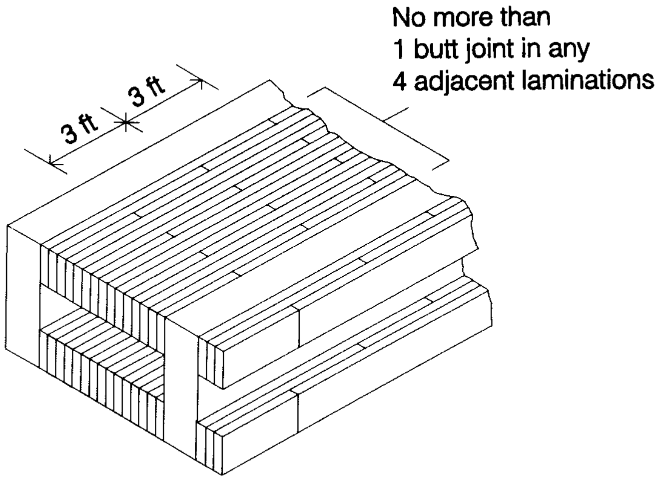

Figure 5—Repetitive butt-joint pattern for sawn lumber flange laminations.

Figure 7—Bridge components at the treating plant, after the pressurized treatment process.

Figure 6—Construction of reinforced concrete abutments and wingwalls.

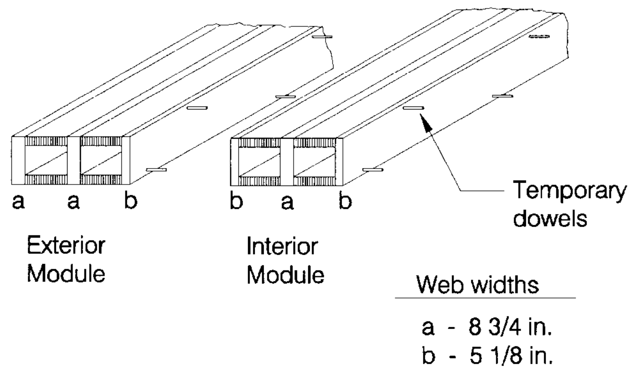

Figure 8—The bridge was preassembled into a series of four interior and two exterior bridge modules.

During construction of the substructure, prefabrication and preassembly of the superstructure were completed at a nearby lumber fabrication plant. After prefabrication, the bridge components were pressure treated with preservative and preassembly was completed (Fig. 7). To facilitate transportation and installation, the design specified assembly of the bridge into a series of six bridge modules (two exterior and four interior), to be placed side by side to form the bridge width (Fig. 8). Using this approach, the width of the glulam webs at the module interface was 5.125 in., which provided for a total web width of 10.25 in. when the modules were joined. At other locations, a web width of 8.75 in. was used. The bridge modules were assembled using nails and temporary dowels (Fig. 9), then transported to the bridge site on flatbed trailers.

The bridge superstructure was installed July 27, 1992. At the bridge site, the modules were lifted and placed on the abutments by a large overhead crane (Fig. 10). Temporary dowels were removed and full-width, steel stressing bars were inserted by hand through prebored holes in the top and bottom flanges (Fig. 11). Steel anchorage plates and nuts were attached to each bar end, and the initial stressing of the bridge commenced.

The bridge was stressed initially at installation and 3 and 7 weeks after installation. At the initial stressing, the bars were individually tensioned to the full design level in a sequential manner, beginning at one end of the bridge. Subsequent stressings at 3 and 7 weeks after installation were performed to the full design force using the same procedures. At the conclusion of the stressing, it was noted that the bridge width measured 38 ft, which was 1 ft greater than specified in the design. The width of stress-laminated decks is typically increased during fabrication to compensate for anticipated losses as a result of high compressive forces during the stress-laminating process. The increased bridge width was probably due to overestimating the amount of compression in the box-beam superstructure.

Figure 9—Preassembly of the bridge components into modules at the fabrication plant. Temporary supports were used to align members (top); temporary dowels and nails were placed to fasten members (bottom).

Figure 10—Lifting preassembled bridge modules onto concrete abutments with an overhead crane.

After the initial stressing and prior to constructing the abutment backwalls, the superstructure was anchored to the concrete abutments. A 6- by 12-in. timber sleeper block and the lower flange of the superstructure were attached to the abutment sill with bolts. The asphalt wearing surface was

Figure 11—Placement of full-width, steel stressing bars through prebored holes in the top and bottom flanges.

Figure 12—The completed rail system and pedestrian sidewalk.

installed approximately 6 weeks after bridge installation, and the railing and walkway were constructed approximately 3 months after bridge installation (Fig. 12). The completed Spearfish Creek bridge is shown in Figure 13.

Cost

Material and labor costs for the design, fabrication, and construction of the Spearfish Creek bridge superstructure totaled \$161,500. Based on a total deck surface area of 2,539 ft^2, the unit cost for the superstructure was approximately \$64/ft^2.

Evaluation Methodology

To evaluate the structural performance of the Spearfish Creek bridge, the South Dakota DOT contacted FPL for assistance. Through mutual agreement, a 5-year monitoring plan was developed by the FPL and implemented through a Cooperative Research and Development Agreement with the South Dakota DOT. The plan called for performance monitoring of the moisture content of the deck, stressing bar

Figure 13—The completed Spearfish Creek bridge: side view (top) and end view (bottom), looking north.

force, static-load behavior, and general bridge condition. The plan evaluation methodology utilized procedures and equipment previously developed and used on similar structures (Ritter and others 1991). The monitoring period was initiated at the final bridge stressing, approximately 7 weeks after installation.

Moisture Content

Wood moisture content was measured with an electrical-resistance moisture meter with 3-in. insulated probe pins in accordance with ASTM D4444–84 (ASTM 1990). Measurements were obtained from sawn lumber and glulam members at several locations on the underside of the superstructure by driving the probe pins to a depth of approximately 2 in. Measurements were obtained by City of Spearfish personnel on approximately a quarterly basis throughout the monitoring period. Meter readings were adjusted with the appropriate temperature and wood species corrections.

Bar Force

Stressing bar force was measured with calibrated, steel load cells and a portable strain indicator. At the initiation of monitoring, four load cells were installed at interior bar locations where load cells were placed on bars through the top and bottom flanges. Measurements were obtained by City of Spearfish personnel on approximately a biweekly basis throughout the monitoring period. Load cell readings were converted from units of strain to force based on laboratory calibrations. In addition, accuracy of the load cells was validated with hydraulic equipment at subsequent bridge stressings.

Behavior Under Static Load

Static-load testing of the Spearfish Creek bridge was conducted at 2 and 25 months after installation to determine the response of the bridge to highway truck loads. In addition, predicted deflections were determined for each load test based on static analysis for actual and 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 bridge centerspan and abutment cross sections. A surveyor's level was used to read deflection values from calibrated rules suspended from the underside of the bridge. 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).

Load Test 1

Load test 1 was conducted September 18, 1992, and utilized six load cases and two fully loaded trucks: truck A with a gross vehicle weight of 42,680 lb and truck B with a gross vehicle weight of 44,320 lb (Fig. 14). For load cases 1 to 3, the trucks were positioned transversely 2 ft from the roadway centerline. For load cases 4 to 6, the trucks were positioned transversely 10 ft from the roadway centerline. For all load cases, the truck center of gravity was positioned at midspan, and deflections were measured to within 0.03 in.

Load Test 2

Load test 2 was conducted August 22, 1994, and utilized six load cases and two fully loaded trucks: truck A with a gross vehicle weight of 46,700 lb and truck B with a gross vehicle weight of 47,400 lb (Fig. 15). To align the truck wheel lines directly over web members, the trucks were slightly offset in the transverse direction from the load test 1 positions. For load cases 1 to 3, the trucks were positioned 3.1 ft from the roadway centerline. For load cases 4 to 6, the trucks were positioned transversely 9.8 ft from the roadway centerline. The truck center of gravity was positioned at midspan for all load cases, and deflections were measured to within 0.04 in. Load cases 3 and 6 are shown in Figure 16.

Predicted Deflection Analysis

At the conclusion of load testing, predicted deflections were calculated for AASHTO HS20–44 loading. Because design procedures and analytical models for stress-laminated box beam bridges are currently under development, a simplified procedure using measured load test deflections and a ratio of deflection magnitudes was used. The procedure was based on a deflection coefficient (DC) determined through computer analysis (Murphy 1994) and the following relationship:

$$\Delta_{\mathrm{HS2O}} = \Delta_{\mathrm{Load\ test}}\left(\frac{\mathrm{DC}_{\mathrm{HS2O}}}{\mathrm{DC}_{\mathrm{Load\ test}}}\right)$$

where

Δ_{HS2O} = HS20 predicted deflection (in.);

$\Delta_{\mathrm{Load\ test}}$ = Maximum measured load test deflection (in.);

$\mathrm{DC}_{\mathrm{HS2O}}$ = HS20 deflection coefficient ($\mathrm{lb \cdot in^4}$); and

$\mathrm{DC}_{\mathrm{Load\ test}}$ = Load test vehicle deflection coefficient ($\mathrm{lb \cdot in^4}$).

Condition Assessment

The general condition of the Spearfish Creek bridge was assessed at the initiation of monitoring and at the time of the second load test. These assessments involved visual inspections, measurements, and photographic documentation. Items of specific interest included bridge geometry, deck camber, wood components, wearing surface, stressing bar anchorages, and steel hardware.

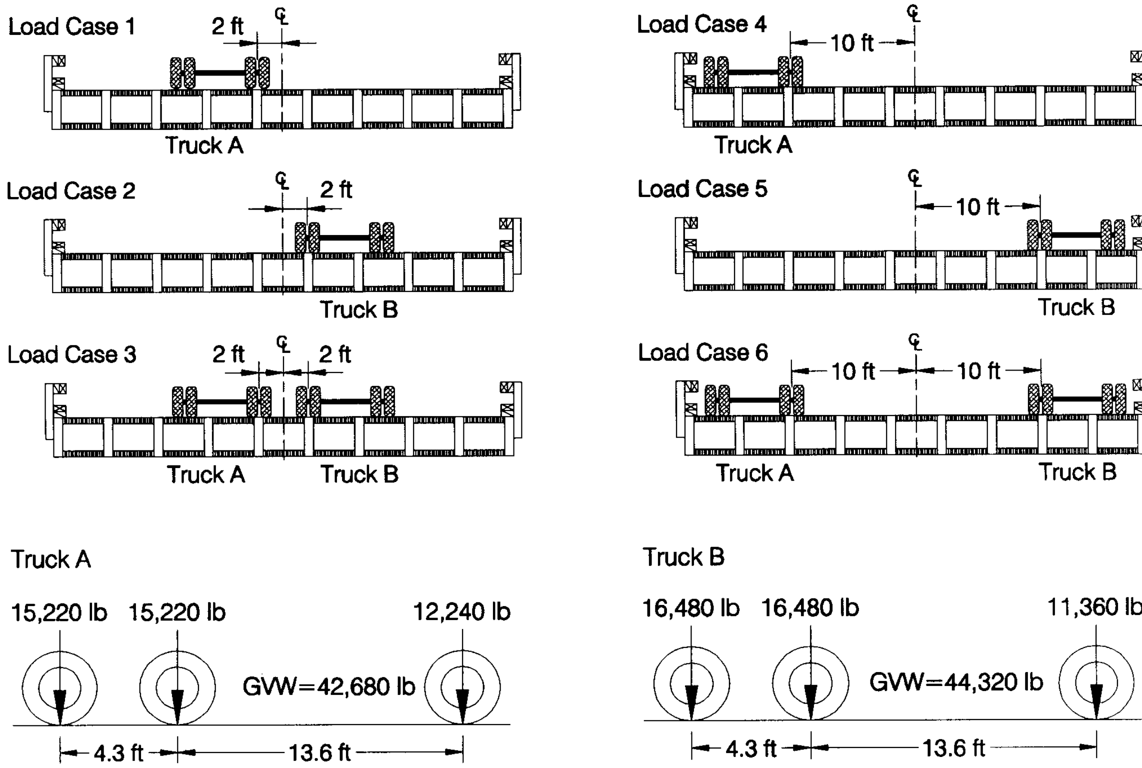

Figure 14—Load test 1 truck weights, axle spacings, and transverse load test positions.

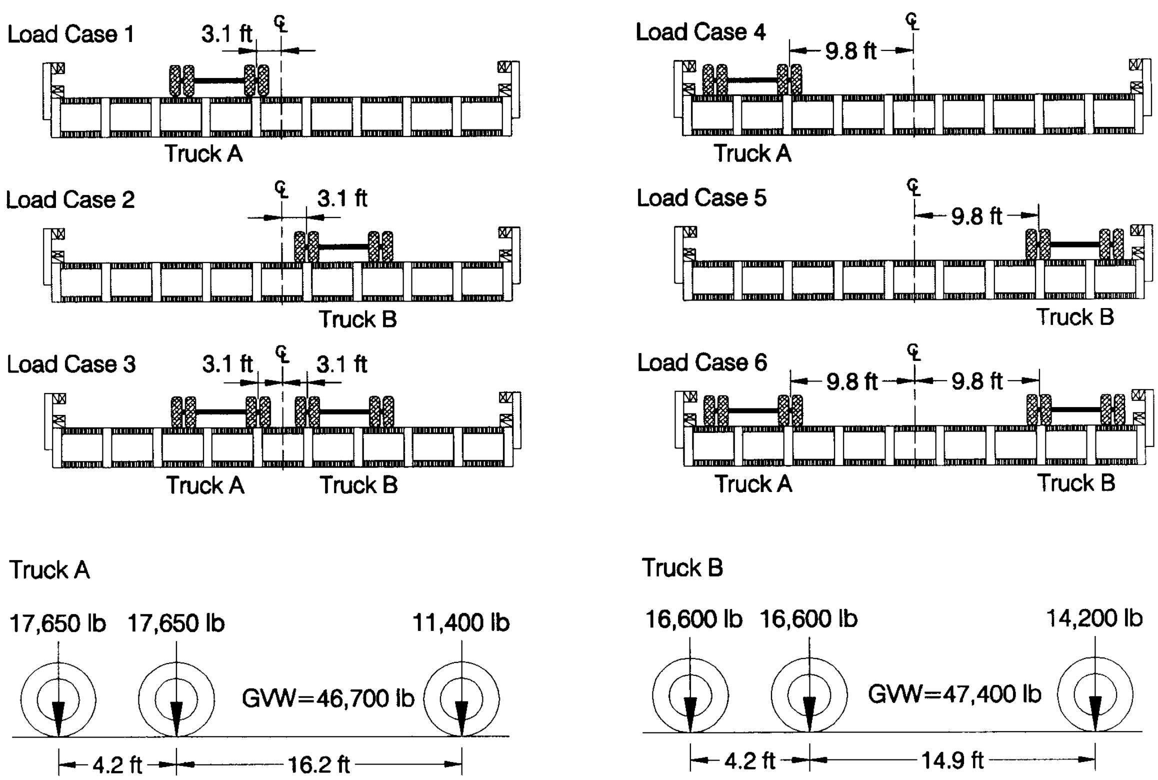

Figure 15—Load test 2 truck weights, axle spacings, and transverse load test positions.

Figure 16—Load test 2 truck positions for load cases 3 and 6 (looking south).

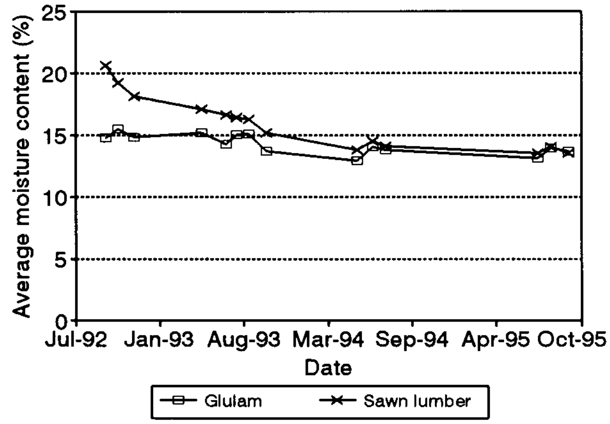

Figure 17—The average trend in moisture content.

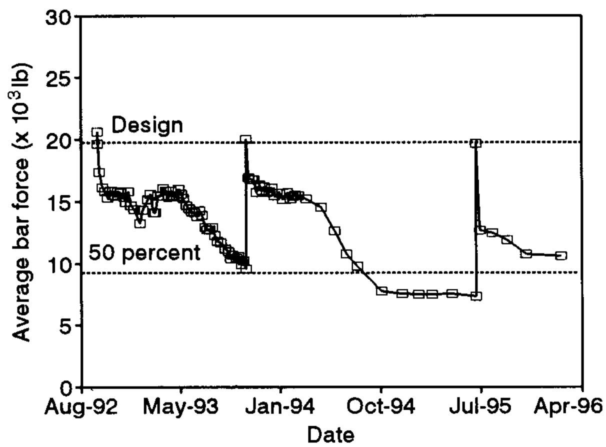

Figure 18—The average trend in stressing bar force.

Results and Discussion

Performance of the Spearfish Creek bridge has been continuously monitored since September 1992. The following results are presented based on data collected through February 1996, or the first 3-1/2 years of a planned 5-year monitoring program.

Moisture Content

The average trend in wood moisture content is presented in Figure 17. At the initiation of monitoring, the average moisture content was approximately 20 percent for the sawn lumber and 15 percent for the glulam timber. For 2 years, beginning in September 1992, the average moisture content of the sawn lumber gradually decreased to approximately 15 percent. During the same 2 years, the average moisture content of the glulam timber also decreased slightly to 14 percent. Readings between June 1994 and October 1995 indicated that both the sawn lumber and the glulam timber were equilibrating to the surrounding environment and stabilizing at an average equilibrium moisture content of approximately 14 percent.

Bar Force

The average trend in stressing bar force is shown in Figure 18. From the final construction stressing in September 1992 through October 1993, the average bar force decreased 50 percent to approximately 10,000 lb, or 50 lb/in^2 interlaminar compression. For stress-laminated box-beam bridges, the potential for interlaminar slip between the web and flange increases below 50 lb/in^2 interlaminar compression. Therefore, FPL advised the South Dakota DOT to retension all stressing bars to the full design level. After bar retensioning was completed in October 1993, the average bar force decreased during 1 year by 60 percent to approximately 8,000 lb, or 40 lb/in^2 interlaminar compression. Because the average bar force decreased below the 50 lb/in^2 interlaminar compression level for a second time, bar retensioning was recommended but was not performed until the following summer. Data collected since the last bar retensioning in July 1995 showed that the average bar force decreased 40 percent to approximately 12,000 lb, or 60 lb/in^2 interlaminar compression level. Future bar retensionings may be necessary if the average bar force decreases below 10,000 lb, or 50 lb/in^2 interlaminar compression.

The observed bar force loss is attributable to the combined effects of decreasing moisture content and stress relaxation in the sawn lumber flanges. The 6-percent decrease in the sawn lumber moisture content caused flange shrinkage and was probably most significant during the first half of the monitoring period, when the greatest moisture content loss occurred. Stress relaxation in the sawn lumber laminations, a phenomenon previously discussed by Ritter (Ritter and others 1991), has been observed to cause bar force loss in numerous other stress-laminated bridges. It is probable that stress relaxation was most responsible for the rapid loss in bar force after retensioning.

Behavior Under Static Load

Results of static-load testing and the predicted response are presented. For each load case, transverse deflection measurements 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 measurable 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 100 and 50 lb/in^2, respectively.

Load Test 1

Transverse deflections for load test 1 are presented in Figure 19. The maximum deflections for load cases 1 and 2 occurred under the inside truck wheel line and measured 0.45 and 0.5 in., respectively (Fig. 19a,b). For load case 3, the maximum deflection of 0.76 in. was at the roadway centerline and represented the largest measured deflection of all load cases (Fig. 19c). Maximum deflections for load cases 4 and 5 measured 0.67 and 0.63 in., respectively, and occurred near the outside truck wheel line (Fig. 19d,e). The maximum deflection for load case 6 measured 0.69 in. and was under the outside wheel line of truck A (Fig. 19f). For all load cases, the deflected shape of the centerspan cross-section 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 20 shows the load test 1 comparison between individual and simultaneous truck loading. As shown in Figure 20, 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 deflections for load test 2 are shown in Figure 21. The maximum deflection of 0.51-in. for load case 1 occurred under the outside truck wheel line (Fig. 21a), and the maximum deflection for load case 2 was 0.54 in. under the inside truck wheel line (Fig. 21b). For load case 3, the maximum deflection of 0.79 in. was measured under the inside wheel line of truck A, adjacent to the roadway centerline, again representing the largest deflection of all load cases (Fig. 21c). Maximum deflections for load cases 4 and 5 occurred under the outside truck wheel line and measured 0.74 and 0.70 in., respectively (Fig. 21d,e). The maximum deflection for load case 6 measured 0.76 in. and was under the outside wheel line of truck A (Fig. 21f).

Figure 22 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 23 for load cases 3 and 6. The plots are similar in shape, but the deflections for load test 2 are greater at numerous data point locations. Several factors may have contributed to these deflection differences. The load test 2 trucks were approximately 8 percent heavier than those for load test 1, which would increase load test 2 deflections. Transverse truck positions were also slightly different for the two tests, which could result in variations in the shape and magnitude of the deflections. Another contributing factor was the 50-percent reduction in interlaminar compression for load test 2, which tends to reduce the transverse bridge stiffness resulting in a larger deflection and a slight change in the transverse deflection profile.

Predicted Response

Table 2 summarizes the maximum measured deflections for both load tests 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 HS20–44 span/deflection ratio as a function of the bridge span L, measured center–center of bearings. It also shows the load test deflection as a percentage of HS20–44 deflection.

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 1.04 in., which resulted in a span/deflection ratio of L/738. For load test 2, predicted HS20–44 deflection was 1.05 in. or L/731. Despite the 50-percent decrease in interlaminar compression at load test 2, the predicted HS20–44 deflections for the two tests were approximately equal. This indicates that the significant change in interlaminar compression had little effect on the longitudinal bridge stiffness. It is likely that the full-span glulam webs tend to minimize the impact of prestress effect on the longitudinal bridge stiffness.

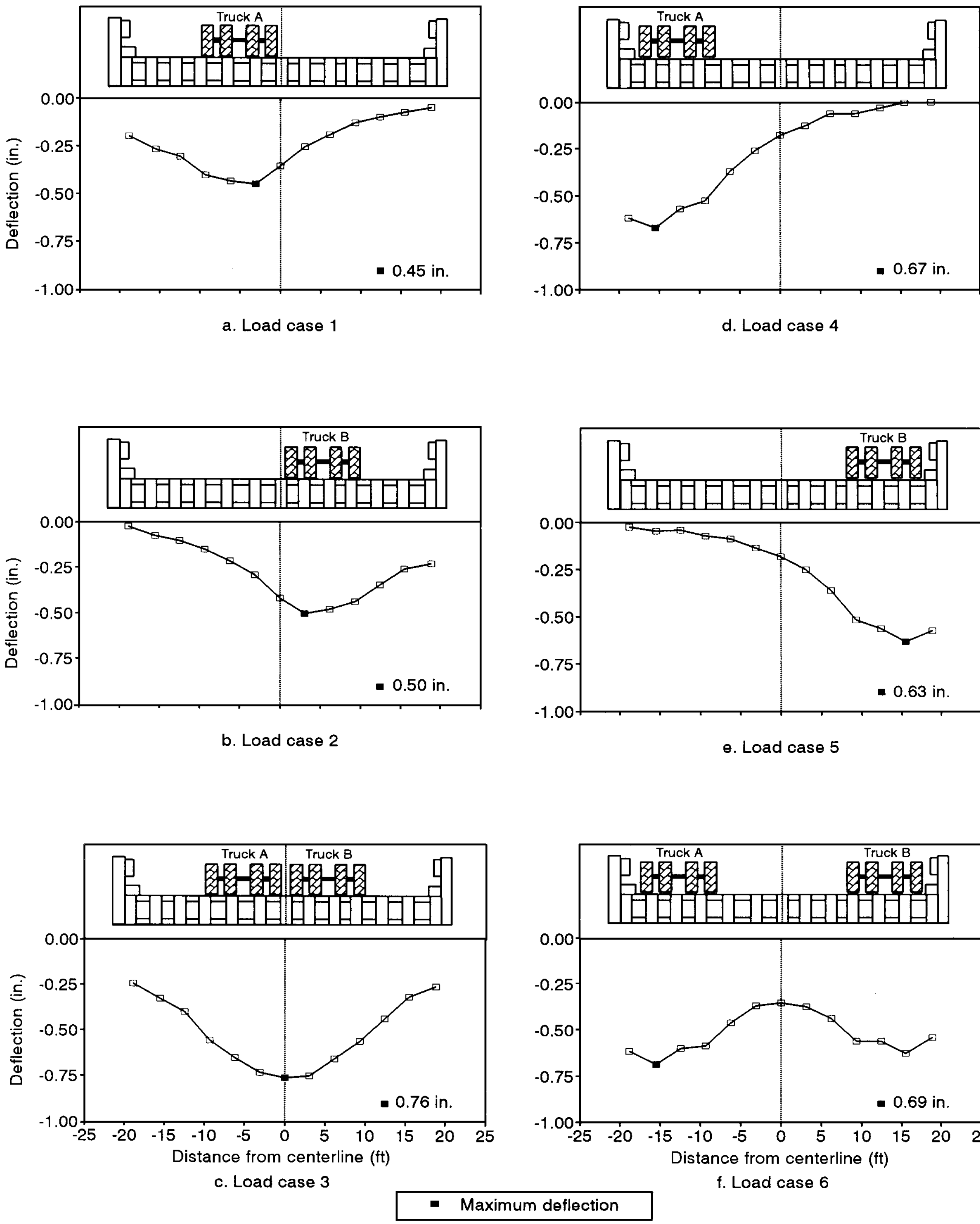

Figure 19—Load test 1 transverse deflection measured at the bridge centerspan (looking north). Bridge cross-sections and vehicle positions are presented to aid interpretation only and are not to scale.

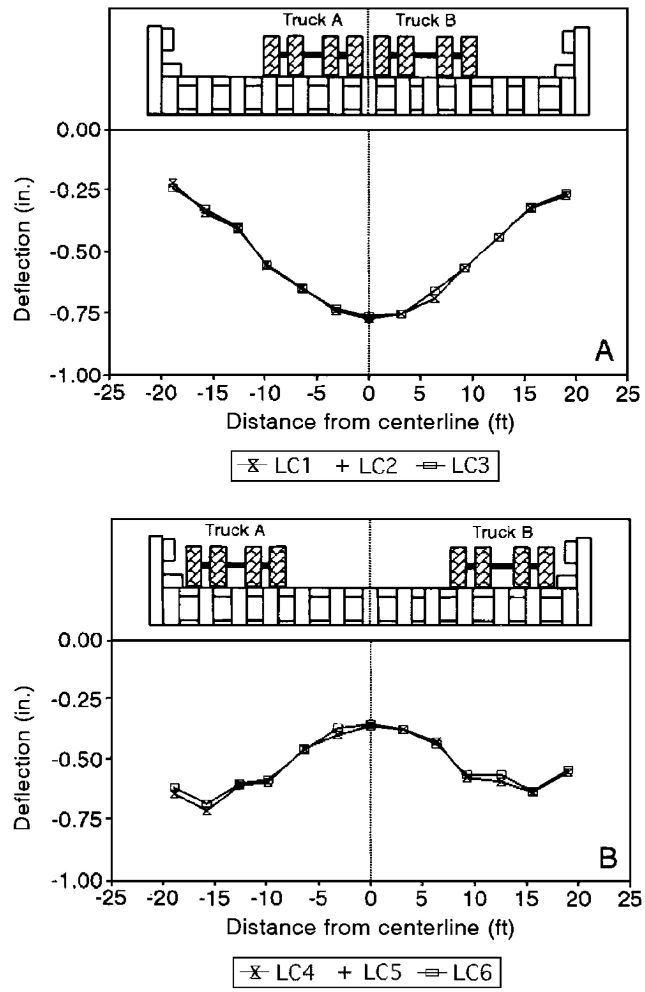

Figure 20—Load test 1 comparisons: (A) sum of the measured deflections from load cases 1 and 2 to the measured deflections for load case 3, (B) sum of the measured deflections from load cases 4 and to the measured deflections for load case 6.

Condition Assessment

General condition assessments indicated that the structural and serviceability aspects of the Spearfish Creek bridge were satisfactory. Results of the specific areas inspected follow.

Bridge Geometry

Width measurements taken at the initiation of monitoring indicated that the box beam was 2.5 in. narrower at the midspan than at the abutments. This was likely due to the sequential bar tensioning with a single jack. The slight distortion has not increased in magnitude and should not affect overall bridge performance.

Deck Camber

Measurements taken prior to load test 2 indicated positive camber of 4 in. at midspan, which is approximately 66 percent of the specified design camber. With a substantial percentage of the original camber remaining, vertical creep was minimal through August 1995.

Wood Components

Visual inspection of the wood components of the bridge indicated no signs of deterioration or damage. All bolted connections remained tight, with no signs of wood crushing beneath the connectors. The guardrail components had accumulations of creosote surface residue at the initiation of monitoring, but no bleeding or dripping was noted.

Wearing Surface

The asphalt wearing surface is in good condition, with only minor transverse reflective cracking visible over the bridge supports. This is typical of simple-span bridges and was expected. No longitudinal asphalt rutting or cracking was evident.

Anchorage System

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

Conclusions

Based on data collected during the initial 3-1/2 years of a planned 5-year monitoring program, performance of the Spearfish Creek bridge is satisfactory. There are no structural or serviceability deficiencies that would prevent the bridge from providing satisfactory performance in the future. However, two bridge restressings were performed due to excessive bar force loss. Because of inclusion in a monitoring program, the excessive bar force loss was detected and remedied before structural problems occurred. Based on the monitoring results, we present the following conclusions and observations:

- The sawn lumber and glulam components have stabilized at an equilibrium moisture content of approximately 14 percent. Since installation, the moisture content of the sawn lumber has decreased gradually from approximately 20 percent, and the glulam has remained essentially unchanged at approximately 14-percent moisture content.

- Bar force loss warranted two unplanned bridge restressings through February 1996. In February 1996, approximately 8 months since the last bar retensioning, bar forces have decreased to approximately 12,000 lb, or 60 percent of the original design force. Bar forces should be checked on an annual basis until they stabilize above the 50 lb/in^2 interlaminar compression level.

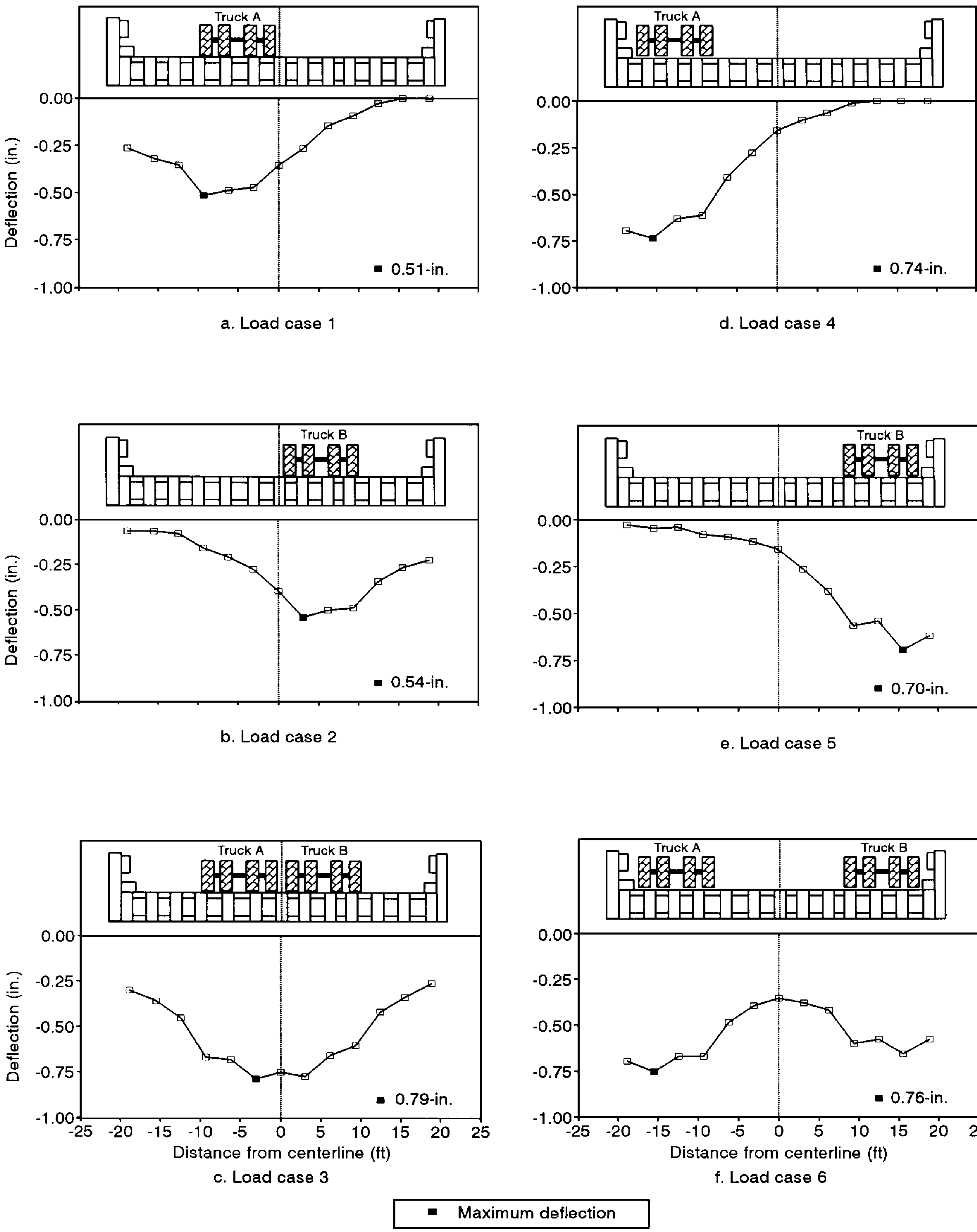

Figure 21—Load test 2 transverse deflection measured at the bridge centerspan (looking north). Bridge cross-sections and vehicle positions are presented to aid interpretation only and are not to scale.

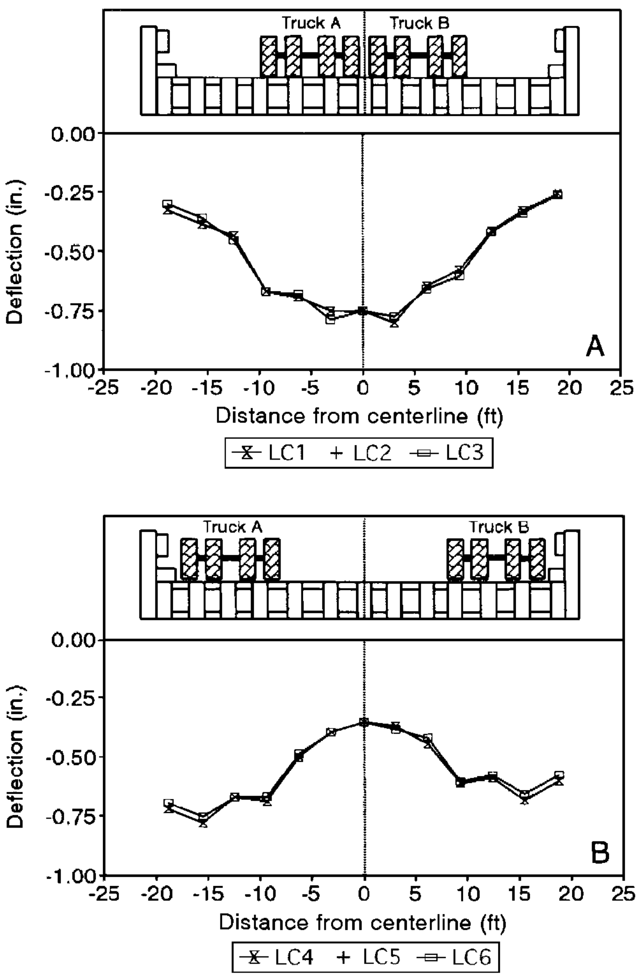

Figure 22—Load test 2 comparisons: (A) sum of the measured deflections from load cases 1 and 2 to the measured deflections for load case 3, (B) sum of the measured deflections from load cases 4 and 5 to the measured deflections for load case 6.

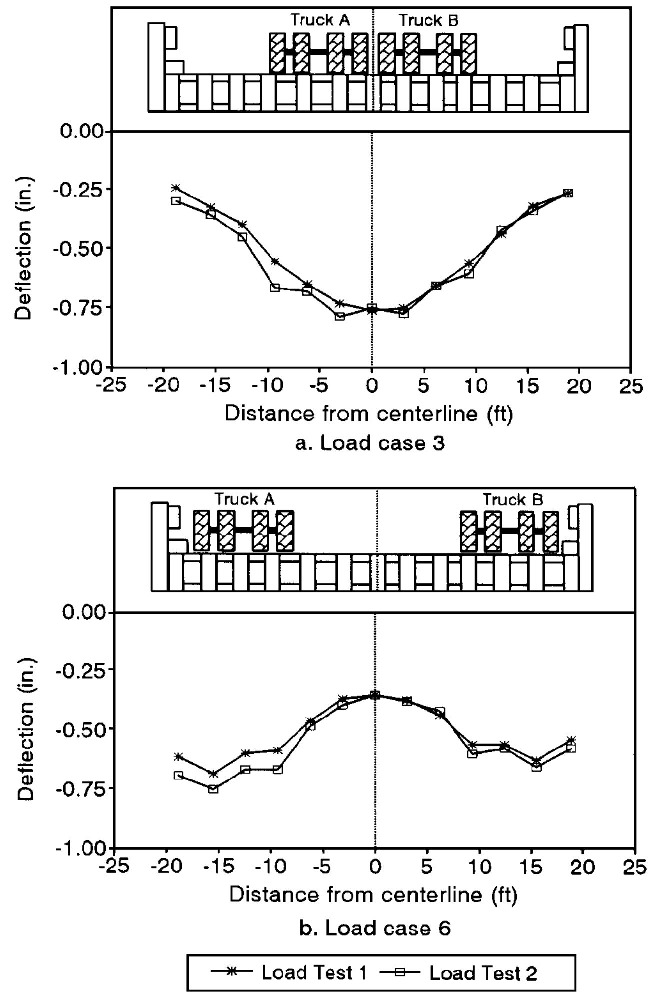

Figure 23—Comparison of load tests 1 and 2 deflections.

Table 2—Summary of maximum load test and predicted HS20–44 midspan deflections

	Load test vehicle		AASHTO HS20–44 vehicle	
Load test	Maximum measured deflection (in.)	Percentage of equipment HS20–44 deflection	Maximum predicted deflection (in.)	Span/ deflection ratio
1	0.76	73	1.04	L/738
2	0.79	75	1.05	L/731

- Vertical creep was minimal through August 1995, with an upward camber of 4 in. remaining in the bridge superstructure.
- Static-load testing and analysis indicate that the Spearfish Creek bridge is performing in a linear-elastic manner. When subjected to truck loading at interlaminar compression levels of 100 and 50 lb/in^2, the bridge had a decrease in transverse stiffness. However, the change in interlaminar compression did not significantly affect the longitudinal bridge stiffness.
- The predicted maximum deflection for two lanes of AASHTO HS20–44 loading was below the design limit of L/500, where L is the span length measured center–center of bearings. For load test 1 at 100 lb/in^2 interlaminar compression, the maximum HS20–44 deflection is estimated to be 1.04 in., or 1/738 of the span length. For load test 2 at 50 lb/in^2 interlaminar compression, the maximum HS20–44 deflection is estimated to be 1.05 in., or 1/731 of the span length.
- Visual inspections indicate no signs of deterioration of the wood or steel components.

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Appendix—Information Sheet

General

Location: Spearfish, South Dakota

Date of Construction: July 1992

Owner: South Dakota Department of Transportation

Configuration

Structure Type: Stress-laminated box-beam

Butt Joint Frequency: 1 in 4 laminations transversely and separated 3 ft longitudinally

Total Length (out–out): 65 ft

Skew: 0 degrees

Number of Spans: 1

Span Length (center–center of bearings): 64 ft

Width (out–out): 38 ft, 39 ft (as-built)

Width (curb–curb): 36 ft, 37 ft (as-built)

Number of Traffic Lanes: 2

Design Loading: AASHTO HS20–44

Camber: 6 in.

Wearing Surface Type: Asphalt pavement, 2- to 3-in. thickness

Material and Configuration

Flange Laminations:

- Species: Ponderosa Pine
- Size: 2 by 6 in. nominal
- Grade: No. 2
- Moisture Condition: Approximately 20 percent at the initiation of monitoring
- Preservative Treatment: Pentachlorophenal in heavy oil

Webs:

- Species: Southern Pine
- Size (actual): 5.125 by 31.50 in. and 8.75 by 31.50 in.
- Beam Designation: 24F–V3
- Moisture Condition: Approximately 15 percent at the initiation of monitoring
- Preservative Treatment: Creosote

Stressing Bars:

- Diameter: 0.625 in.
- Number: 22 sets (through top and bottom flange)
- Design Force: 19,600 lb (interior), 27,900 lb (exterior)
- Spacing (center–center): 34 in. (interior), 29 in. (2 exterior), beginning 38 in. from bridge ends
- Type: High strength, steel thread bar with course right-hand thread, conforming to ASTM A722

Rail and Curb System:

- Design: Crash-tested at AASHTO Performance Level 1 on a longitudinal spike-laminated deck
- Species: Douglas Fir
- Member Sizes:
  - Rails: 6 by 10.75 in., glulam
  - Posts: 8 by 12 in. nominal, Dense Select Structural grade, sawn lumber
  - Curbs: 6 by 12 in. nominal, No. 1 grade, sawn lumber
- Preservative Treatment: Creosote

Bar Anchorage Type: Discrete steel plates:
5.75 by 11 by 0.50 in. bearing (with flat hex nut)
2 by 5 by 1 in. anchor