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# Field Performance of Timber Bridges

## 15. Pueblo County, Colorado, Stress-Laminated Deck Bridge

Lola E. Hislop



## Abstract

The Pueblo County 204B bridge was constructed in March 1990 in Pueblo, Colorado, as a demonstration bridge under the USDA Forest Service Timber Bridge Initiative. The stress-laminated deck superstructure is approximately 10 m long, 9 m wide, and 406 mm deep, with a skew of 10 degrees. Performance monitoring was conducted for 3 years, beginning at installation, and involved gathering data on the moisture content of the wood deck, the force level of the stressing bars, the behavior of the bridge under static load conditions, and the overall condition of the structure. In addition, long-term performance data were gathered on the force level of the stressing bars 6 years after installation. Based on monitoring evaluations, the bridge is performing well, with some crushing of the bearing plates into the outside laminations but no other structural or serviceability deficiencies.

Keywords: Timber bridge, stress laminated, wood, performance, Colorado

## Acknowledgments

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# Field Performance of Timber Bridges

## 15. Pueblo County, Colorado, Stress-Laminated Deck Bridge

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### Introduction

In 1988, the U.S. Congress passed legislation known as the Timber Bridge Initiative (TBI). One purpose of the legislation was to provide partial funding towards the construction of demonstration timber bridges to encourage innovation through the use of new or previously underutilized wood products, bridge designs, and design applications. In so doing, bridge designers and users become more aware of the attributes of wood as a bridge material, and new, economical, structurally efficient timber bridge systems should result. Responsibility for the development, implementation, and administration of the TBI program was assigned to the USDA Forest Service. As a national wood utilization research laboratory within the USDA Forest Service, the Forest Products Laboratory (FPL) has taken a lead role in assisting local governments in evaluating the field performance of demonstration bridges, many of which employ design innovations that have not been previously evaluated. This report, fifteenth in a series documenting field performance of timber bridges, describes the development, design, construction, and field performance of the Pueblo county bridge, which was built as a part of the TBI.

The Pueblo County 204B bridge, built in 1990 and located in Pueblo, Colorado, is a two-lane, single-span, stress-laminated deck with a length of 9.72 m, a width of 9.17 m, a nominal depth of 406 mm, and a skew of 10 degrees (see Table 1 for English conversion factors). An information sheet on the Pueblo County 204B bridge is provided in the Appendix.

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

| SI unit                        | Conversion factor | English unit                   |
|--------------------------------|-------------------|--------------------------------|
| millimeter (mm)                | 0.0394            | inch (in.)                     |
| meter (m)                      | 3.281             | foot (ft)                      |
| square meter (m <sup>2</sup> ) | 11.11             | square foot (ft <sup>2</sup> ) |
| newton (N)                     | 0.2248            | pound (lb)                     |
| pascal (Pa)                    | 0.000145          | lb/in <sup>2</sup> (stress)    |
| meter (m)                      | 0.000622          | mile                           |

### Background

The Pueblo County 204B bridge is located on McCarthy Boulevard in Pueblo, Colorado (Fig. 1). It crosses the Bessemer irrigation ditch and is part of a two-lane, paved, county road that provides residential access. Inspections of the previous bridge indicated that the components were in poor condition. In addition, the substandard guardrail system and the narrow bridge width had become safety issues. Based on inspection, Pueblo County officials determined that the bridge should be replaced. A proposal was submitted to the USDA Forest Service, requesting partial funding of the bridge as a demonstration project under the TBI demonstration timber bridge program. The design was to incorporate Ponderosa Pine lumber into a skewed, stress-laminated deck configuration. In 1989, the project was approved and partial funding was provided through the TBI.

### Objective and Scope

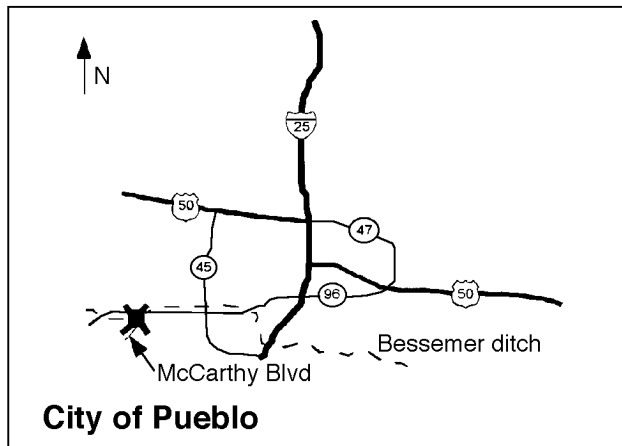
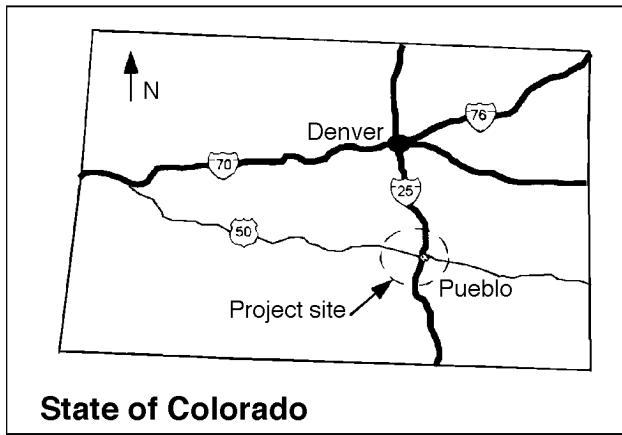
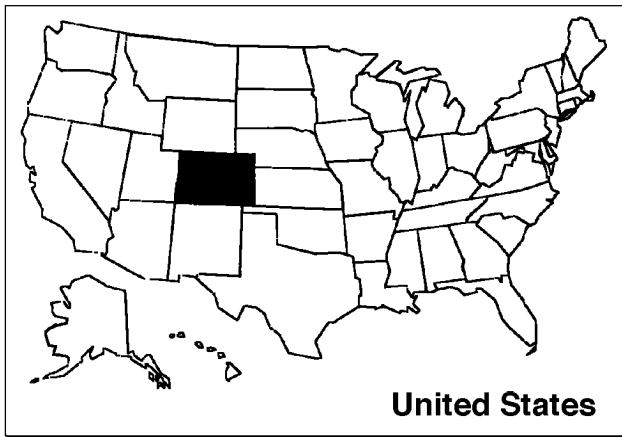
The objective of this project was to evaluate the field performance of the Pueblo County 204B bridge for a minimum of 2 years, beginning after bridge installation. The project scope included data collection and analysis of the moisture content of the wood, stressing bar tension force, bridge behavior under static truck loading, and general structural performance. The results of this project will be used to formulate recommendations for the design and construction of similar bridges in the future.

### Design and Construction

The design, materials, and construction oversight of the Pueblo County 204B bridge were provided by the contractor supplying the timber bridge. Construction labor was provided by the Pueblo County Department of Public Works, Road, and Bridge Division. An overview of the design and construction of the bridge are presented.

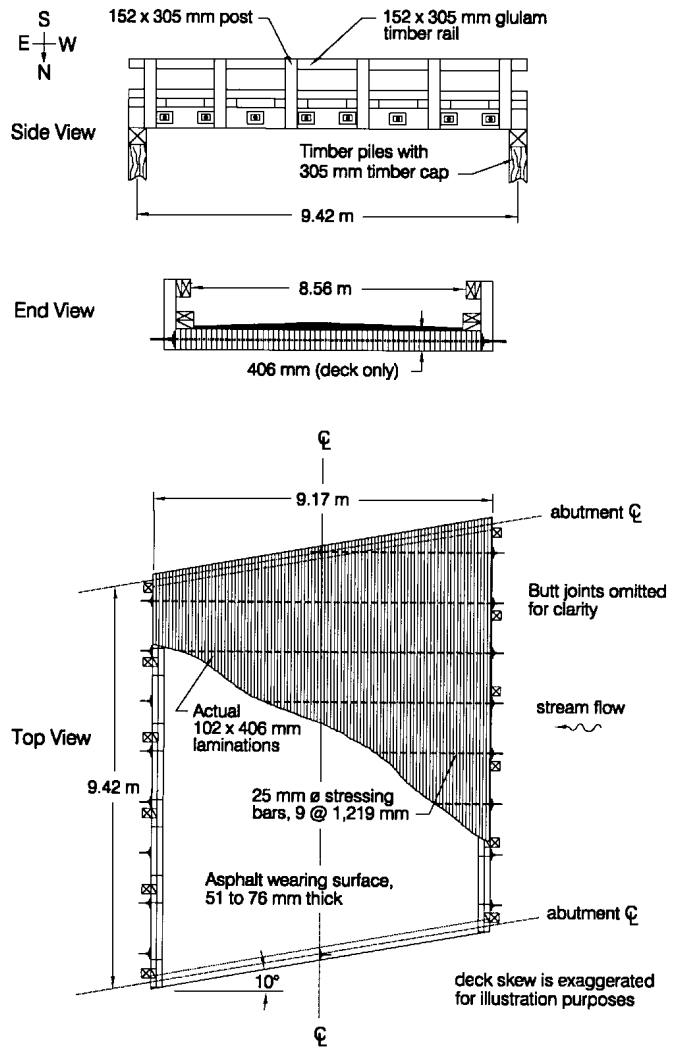
### Design

Although design information was not received from the contractor, it is likely that the bridge design was based on the *Ontario Highway Bridge Design Code* (OMTC 1983). The bridge was designed for a HS 20–44 truck live load, as



**Figure 1—Location of Pueblo County 204B bridge.**

recommended by the American Association of State Highway and Transportation Officials (AASHTO) (AASHTO 1989). The design geometry of the deck provided for a total 9.72-m out-out length, a 9.17-m out-out width, and a skew of 10 degrees (Fig. 2). According to the proposal submitted to the TBI, visually graded No. 1 Ponderosa Pine, 102- by 406-mm (actual size) solid-sawn laminations were specified for the deck material. It was later discovered that visually graded No. 1 & Better Douglas Fir-Larch laminations were used.



**Figure 2—Design geometry of the Pueblo County 204B bridge.**

The stress-laminated deck was designed with butt joints. The butt joint configuration consisted of one butt joint in every fourth lamination in the transverse direction, with a 1.22 m longitudinal separation between butt joints in adjacent laminations. To provide the design interlaminar compression of 717 kPa, 25-mm-diameter, high strength stressing bars were spaced 1,219 mm on center, beginning 610 mm from the end and tensioned to 355.84 kN. The bars were specified to comply with the requirements of the American Society for Testing and Materials (ASTM) A 722 (ASTM 1988) and provide a minimum ultimate tensile strength of 1.03 GPa. The bar anchorage system was designed using discrete steel plates, which consisted of 25- by 305- by 406-mm bearing plates with 25- by 102- by 152-mm anchor plates and flat hex nuts (Fig. 3). As a result of the skew of the bridge, the length of the first bar at each end was only half the width of the bridge with one bar end anchored in the interior of the deck (Figs. 2, 4).

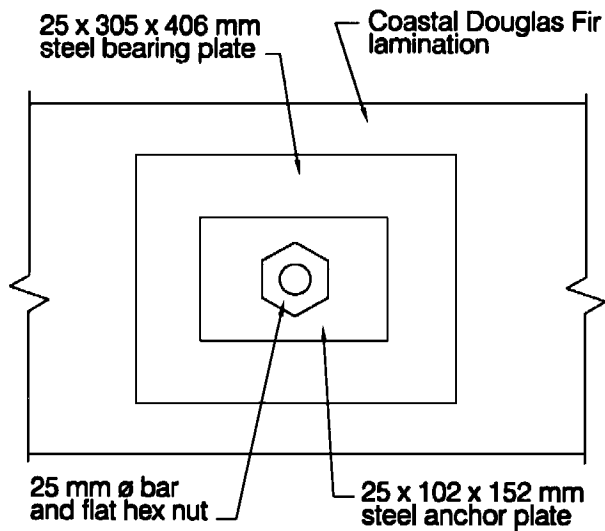


Figure 3—Details of the discrete plate bar anchorage system.



Figure 4—Bar anchorage for interior bar ends as a result of skew.

The deck was provided with a curb and guardrail system, consisting of solid-sawn lumber curbs and posts and glued-laminated (glulam) timber railing and approach rails. The railing design was very similar to a design that was subsequently crash tested to the requirements for performance level 1 as defined by AASHTO. A 51- to 76-mm asphalt wearing surface was specified.

Following fabrication, all wood components were specified to be pressure treated with a creosote-coal tar solution in accordance with American Wood Preservers' Association (AWPA) standard C1 and C2 (AWPA 1991). To provide protection from deterioration, all steel components were galvanized, including hardware, bars, plates, and nuts.

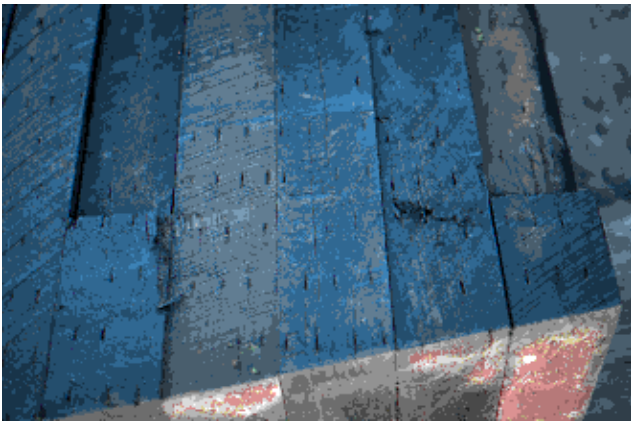
## Construction

Construction of the Pueblo County 204B bridge was completed in March 1990. The superstructure was delivered in four preassembled sections of laminations, interconnected with steel spikes and banded with steel straps (Fig. 5). The delivered sections were placed on the abutments, one at a time. Looking at the ends of each section, it became apparent that many laminations were not cut to length for the skew. Longitudinal alignment of each deck section was accomplished by a few laminations correctly cut to length (Fig. 6a) or nailing very short pieces to the ends of the lamination (Fig. 6b). It is unknown if this affected the bearing area between the deck and the abutments.

After placement of a deck section, the bars were pushed through the entire section and the steel straps were removed prior to placing the next section. This proved to be a difficult method of construction. The free ends of the full-length bars needed to be suspended; otherwise, the bars deformed and became difficult to push through the deck. In addition, it was difficult to line up the holes in the laminations between sections. Near the end of construction, a sledge hammer was used to drive the bars through, with care taken not to damage the ends of the bar (Fig. 7). Stressing began after all sections were in place and was performed using an electric pump and



Figure 5—Bridge superstructure delivered in four preconstructed sections banded with steel straps.



**Figure 6—Two options for obtaining longitudinal alignment: (top) a few laminations correctly cut to length or (bottom) nail-connected short pieces to ends of laminations.**

hydraulic jack with a built-in ratchet (Fig. 8). All bars were tensioned to the full design force, sequentially, beginning at one end of the deck. This sequential bar tensioning was repeated once. The recommended number of repetitions during each deck stressing is 4 to 6 to ensure that the full design force is retained in all the bars. Figure 9 shows the bridge after the initial deck stressing.



**Figure 7—Stressing bars were hammered through the final deck section.**



**Figure 8—Hydraulic jack with built-in ratchet (top) and electric pump (bottom).**

Following the initial stressing, the timber curb, guardrail, and approach rail were constructed. On April 2, April 19, and May 14, the deck of the bridge was restressed to compensate for losses in bar tension force (Ritter 1990), which completed the construction of the deck. The asphalt wearing surface was applied between the second and third stressings. The completed bridge is shown in Figure 10.



**Figure 9—Completed deck of the bridge after initial stressing.**

## Evaluation Methodology

As a result of the unique characteristics of the bridge being skewed and the original plan for the bridge to be constructed from Ponderosa Pine, Pueblo County representatives contacted FPL for assistance in evaluating the structural performance of the Pueblo County 204B bridge. Through mutual agreement, a bridge monitoring plan was developed by FPL and implemented as a cooperative research effort with Pueblo County. The plan called for performance monitoring of the moisture content, stressing bar force, static load behavior, and condition assessments of the structure for a minimum of 2 years following installation. At the initiation of field monitoring, FPL representatives visited the bridge site to install instrumentation and train Pueblo County personnel in moisture content and bar force data collection procedures. Load tests and condition assessments were conducted by FPL personnel with assistance from Pueblo County personnel. The evaluation methodology utilized procedures and equipment that were developed by Ritter and others (1991) and is discussed in the following sections.

### Moisture Content

Moisture content can significantly influence the performance of stress-laminated decks. Changes in moisture content between the fiber saturation point (FSP) of 25% to 30% and the equilibrium moisture content of approximately 18% cause dimensional changes in wood. A decrease in moisture content causes shrinkage and a subsequent loss in bar force. An increase in moisture content causes swelling and a subsequent increase in bar force.

Moisture content was measured using an electrical-resistance moisture meter with insulated 76-mm probe pins in accordance with ASTM D4444–84 (ASTM 1990). Measurements were taken at five locations on the underside of the deck at probe penetrations of 25 to 76 mm. Measurements were taken once a week for the first 3 months, monthly for the next 6 months, once every 2 months for the next year, and once at the conclusion of the 3-year monitoring. Adjustments

for temperature and species were applied to meter values to determine actual moisture content values (FORINTEK 1984).

### Bar Force

Adequate live load distribution of the stress-laminated deck design is dependent on interlaminar compression. Research has shown that the interlaminar compression can decrease by 60% over the life of the structure as a result of stress relaxation of the wood. For this reason, design was based on an initial interlaminar compression of 689.4 kPa, which is anticipated to decrease to 275.8 kPa over the life of the structure. Factors other than stress relaxation, such as decreases in moisture content and subsequent shrinkage of the laminations, can cause the interlaminar compression to decrease below 275.8 kPa. Research has shown that vertical slip between the laminations occurs below an interlaminar compression of approximately 172.4 kPa. Slip leads to a reduction in live load distribution. The Pueblo County bridge was constructed with laminations having a high moisture content; therefore, monitoring of the interlaminar compression was necessary to measure losses greater than 60% of design and the subsequent need to schedule maintenance.

Interlaminar compression is directly related to bar force. To measure bar force, load cells developed by FPL were placed on the second and fifth bars from the south end and read with a portable strain gage indicator (Fig. 11). Load cell measurements were collected in conjunction with moisture content measurements. Measurements were converted from strain units to bar force based on laboratory calibrations. The accuracy of the load cells was validated with a hydraulic jack during load tests and laboratory recalibrations after removal at the end of the 3-year monitoring. Approximately 3 years later, the bar force was measured again using a hydraulic jack to obtain long-term performance information.

### Behavior Under Static Load

Deflection of the stress-laminated deck under live load conditions is an easily measurable indicator of structural performance and design adequacy. Two static load tests were conducted on the same day—at the conclusion of the monitoring period, approximately 35 months after installation. Each load test was performed at a different bar force level to determine the response of the bridge under static loading conditions for varying interlaminar compression levels. The load tests consisted of positioning fully loaded, three-axle dump trucks on the bridge and measuring the resulting deflections at a series of locations along a transverse cross section at mid-span. For each load test, deflection measurements were obtained prior to testing (unloaded), after placement of the test vehicle (loaded), halfway through the load testing (unloaded), and at the conclusion of testing (unloaded). In addition, an analytical assessment using computer modeling was completed to predict the response of the bridge.



Figure 10—Completed Pueblo County 204B bridge.





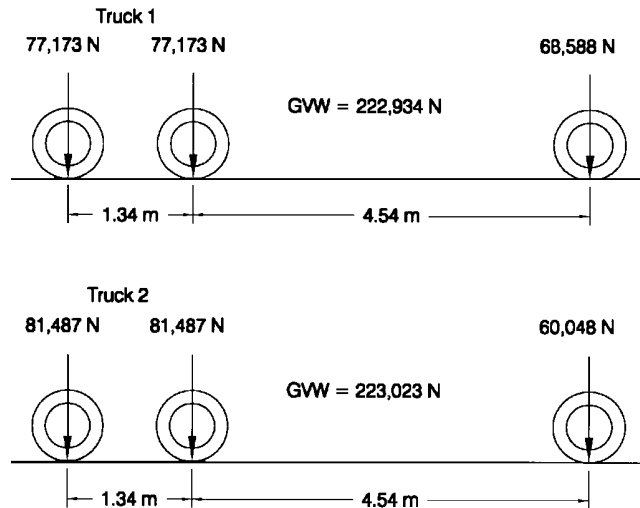
**Figure 11—Measuring bar force data with a portable strain gage indicator and load cell.**

### Static Load Testing

Two load tests were performed on February 17, 1993, with two trucks (Fig. 12). For load test 1, the average bar force was 117.87 kN, which is equivalent to an interlaminar compression of 241.3 kPa. For load test 2, the average bar force was 359.40 kN, which is equivalent to an interlaminar compression of 723.9 kPa. For each load test, six load cases were completed by placing the trucks in different positions transversely on the bridge (Fig. 13). Longitudinally, the rear truck axles were centered over the skewed midspan cross section. Figures 14 and 15 show load cases 3 and 6, respectively, from the north end facing south. For all load cases, the front axles were off the bridge and the vehicles faced in the proper direction of travel. A surveyor's level was used to read deflection values from calibrated rules suspended from the underside of the bridge to the nearest 2 mm (Fig. 16). Accuracy of the measurement method was estimated to be  $\pm 1$  mm.

### Analytical Assessment

Research has shown that stress-laminated decks can be accurately modeled as orthotropic plates (Oliva and others 1990).



**Figure 12—Load configuration axle loads of the test trucks. The width of the transverse truck track was 1.83 m, measured center-center of the rear tires.**

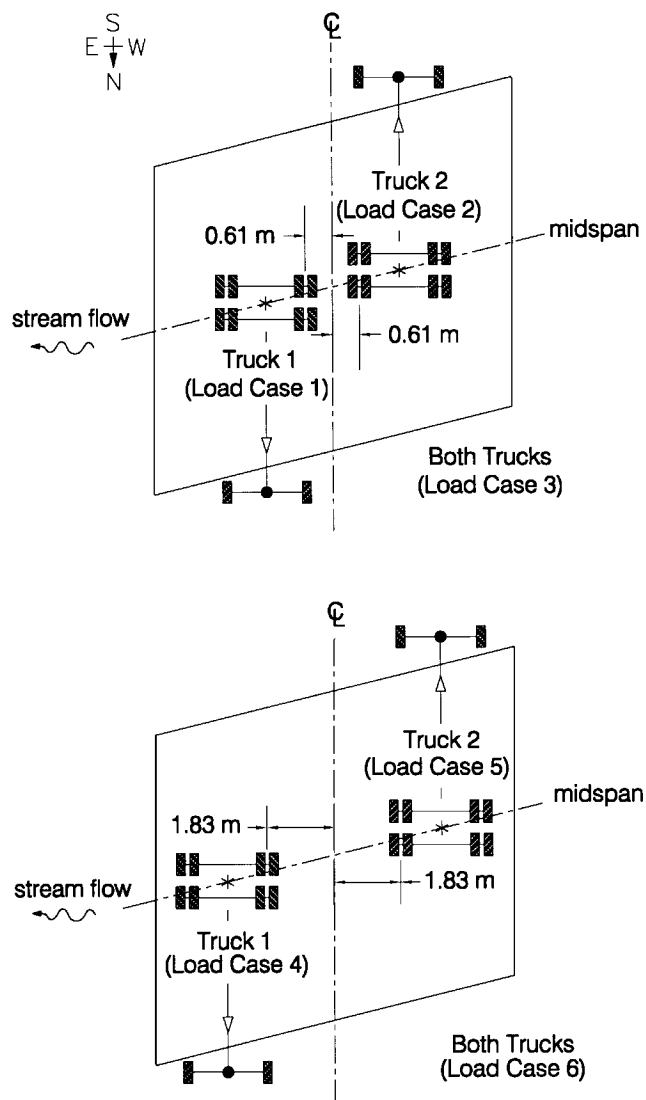
At the completion of the load tests, the measured midspan deflection curves were compared with theoretical deflection curves for the actual truck loading, bridge configuration, and materials. Theoretical curves were determined using an orthotropic plate computer model developed at FPL and a modulus of elasticity of 12.4 GPa. Providing that the measured and theoretical deflection curves were similar and the analytical parameters in the computer model were reasonable, the actual trucks were replaced with AASHTO HS 20-44 trucks to compute theoretical curves for the AASHTO design live load. This resulted in a predicted maximum live load deflection for the deck, which was then compared with design.

### Condition Assessment

The general condition of the bridge was assessed on three occasions during the monitoring period. The first occurred during installation, the second occurred approximately 11 months after installation, and the third occurred at the time of the load tests. These assessments involved visual inspections, measurements, and photographic documentation of the condition of the bridge. Items of specific interest included deck camber and the condition of the wood components, wearing surface, and stressing bar anchorage system. Deck camber was measured using a stringline connected to the timber caps at centerline and a calibrated rule suspended at midspan.

### Results and Discussion

The performance monitoring of the Pueblo County 204B bridge extended from March 1990 through April 1993. In addition, a final check of the bar force was performed in 1996. Results of the performance monitoring data follow.



**Figure 13—Transverse load cases used for load testing. For load cases 1 to 3, trucks were positioned to obtain maximum deflection near the centerline of the deck. For load cases 4 to 6, trucks were positioned to obtain maximum deflection near the edges of the deck.**

## Moisture Content

The average moisture content in the deck of the bridge during the monitoring period is shown in Figure 17. The laminations were installed initially at an average moisture content of 28%, near the FSP. After construction, moisture content fluctuated and, in general, slowly decreased to approximately 25% during the first 2 years. Then, the moisture content rapidly decreased to 16% during the next year. Typically, moisture content fluctuates in the outer 25 to 76 mm of the deck with a very slow downward trend.

For the Pueblo County 204B bridge, the reduction in moisture content was more rapid because the bridge is located in an arid region of Colorado, which experiences very hot

summer temperatures. The asphalt protects the topside of the deck from moisture penetration. The bridge is unprotected by trees or buildings, thus it is fully exposed to a climate (frequent sun, high wind, low relative humidity) that is conducive to rapid moisture evaporation from the surface areas of the deck. It is likely that the moisture content remained high during the first 2 years as a result of the migration of moisture from the interior to the underside of the deck. The remaining rapid moisture reduction occurred as a result of a reduction in the interior moisture content and a subsequent slowing of moisture migration.

Based on the arid location, the equilibrium moisture content of the deck is likely to be 13% to 15%. Although the outer 25 to 76 mm are near equilibrium, the interior portion is probably greater. Globally within the deck, the moisture content will continue to slowly decrease. Small fluctuations will likely occur in the outer 25 to 76 mm throughout the life of the structure with climatic changes. This will also occur at the ends of the bridge as a result of the cracks in the asphalt surface at the ends of the span, which allows moisture migration from the topside.

## Bar Force

The stressing bar force for the 3-year monitoring is shown in Figure 18. In general, the bar force behaved similar to that of other stress-laminated bridges (Ritter and others 1995). Each restress was to a final level slightly less than the 355.84 kN design force. During the first 4 months after the final bar tensioning, the force rapidly decreased approximately 50% to 177.92 kN, where it remained level for 20 months. During the next year, the force decreased to 117.87 kN.

The main causes of bar force loss were stress relaxation and decreasing moisture content. Stress relaxation is the slow compression of the wood cells. It is more pronounced as moisture content increases above 20%. Because the moisture content remained near the FSP for the first 2 years, stress relaxation effected bar force changes whereas shrinkage caused by loss of moisture had little effect. During the final year, shrinkage had the greater effect with the decrease in moisture content and subsequent dimensional changes in the laminations.

During the 3-year monitoring, the deck did not warrant restressing until just prior to the end of the monitoring period. Because the moisture content was anticipated to be near equilibrium, it was assumed that restressing would not be necessary for several years. After 3 more years, the bar force had reduced by 50% on average (Table 2). This loss occurred more quickly than anticipated and was probably due to the global moisture content of the deck continuing to decrease. This emphasizes the long-term effect of installing laminations at a high moisture content level, especially for decks installed in areas conducive to rapid drying.



Figure 14—Truck positions for load case 3.



Figure 15—Truck positions for load case 6.

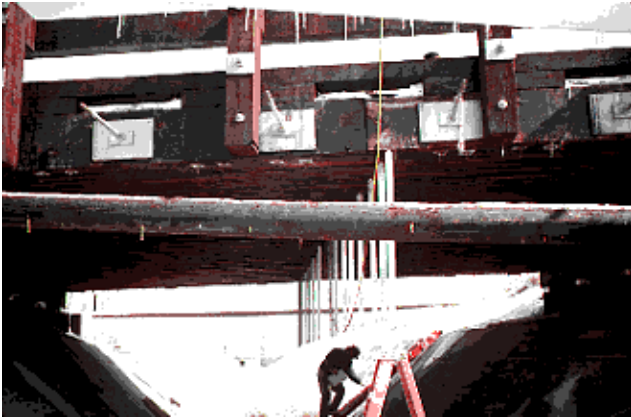


Figure 16—Calibrated scales suspended from the underside of the deck to measure load deflection.

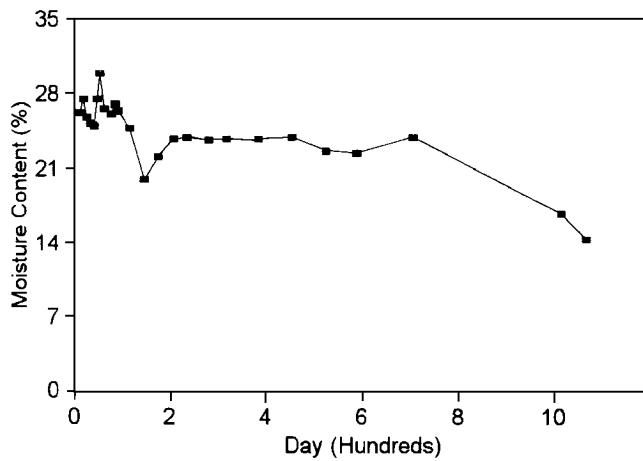


Figure 17—Average moisture content levels in the deck.

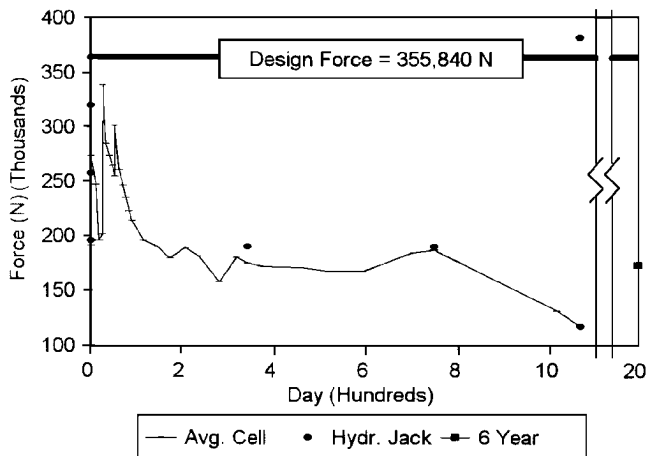


Figure 18—Average bar force obtained from load cells installed on two stressing bars.

## Behavior Under Static Load

Results of the two static load tests and analytical assessments are presented. For each load case, transverse deflection measurements are given at the skewed midspan, viewed from the north end looking south. No permanent residual deformation was measured at the conclusion of the testing. In addition, no movement was measured at either of the abutments.

### Load Test 1

Transverse deflections for each load case are shown in Figure 19. For load cases 4 and 6, there was no apparent single point of maximum deflection. There may have been differences between the points of maximum deflection within each case, but the difference was indistinguishable as a result of the measurement accuracy. In all cases, the deflection was typical of the linear elastic orthotropic plate behavior of stress-laminated bridges, and the location of the maximum measured deflection corresponded to the location observed in similar bridges (Ritter and others 1995).

Assuming that the vehicle positions were symmetrical, the loading along the wheel lines of both trucks were identical, and the material properties of the deck were uniform, several comparisons can be made to validate the accuracy of the measurements. The following comparisons concentrate on the measured location and magnitude of the maximum deflection. Small differences in location and magnitude were due to slight differences in the initial assumptions and the accuracy of the measurement method.

Load cases 1 and 2 (Figs. 19a,b) and load cases 4 and 5 (Figs. 19d,e) should be mirror images of each other. For load cases 1 and 2, the maximum deflection location was centered under the truck and the magnitude was  $-10.3$  and  $-9.5$  mm, respectively. For load cases 4 and 5, the general location was under the outside wheel line and the magnitude was  $-9.5$  and  $-10.7$  mm, respectively. The sum of the deflections under the single truck load cases should be similar to the deflections under the two-truck load case. Comparisons of the deflection curves of load cases 1 plus 2 to load case 3 and load cases 4 plus 5 to load case 6 are shown in Figures 20 and 21. For load cases 1 plus 2 and 3, the maximum deflection location was slightly to one side of the centerline,

Table 2—Bar force 6 years after bridge installation

| Bar number | Bar force (N) | Loss (%) <sup>a</sup> |
|------------|---------------|-----------------------|
| 2          | 227,916       | 36                    |
| 3          | 188,373       | 47                    |
| 4          | 177,920       | 50                    |
| 6          | 148,955       | 58                    |
| 7          | 142,336       | 60                    |
| 8          | 154,532       | 56                    |

<sup>a</sup>Design force of 355,840 N.

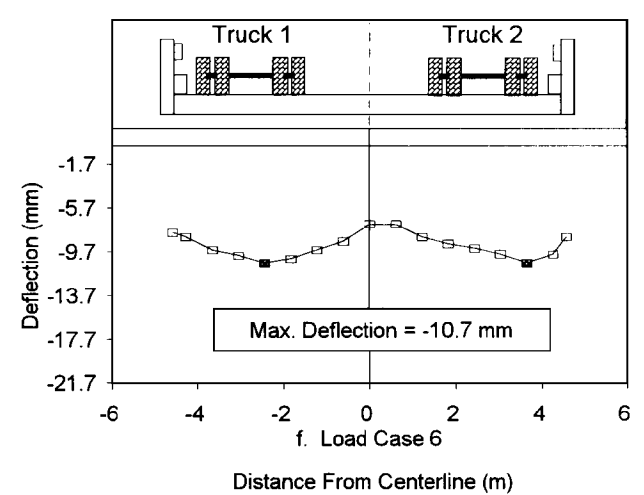
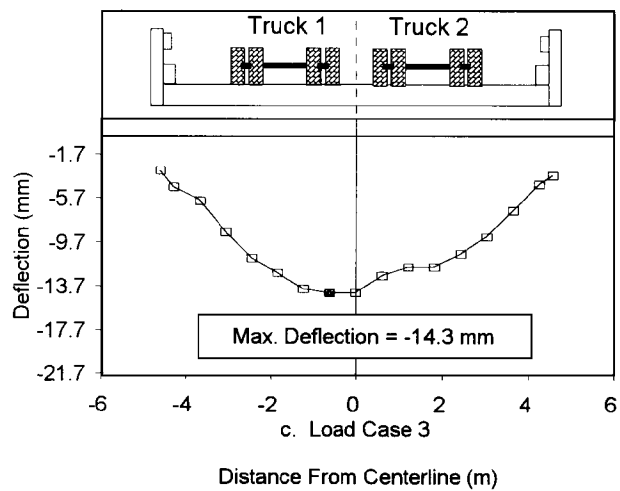
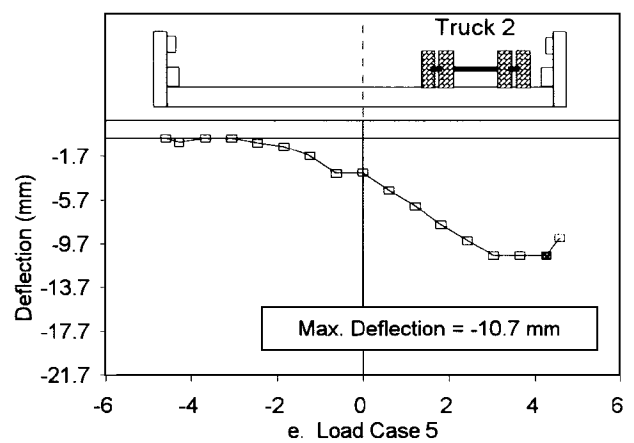
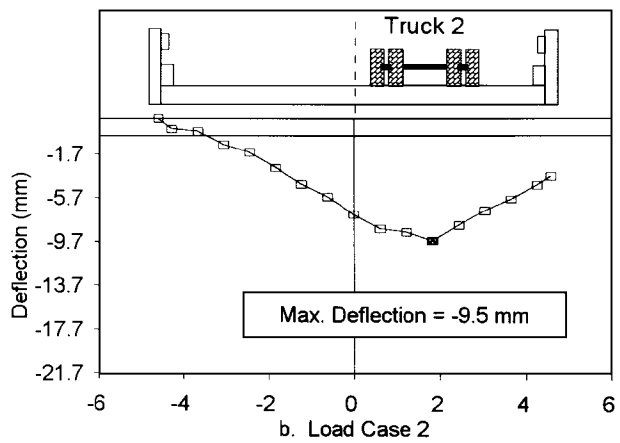
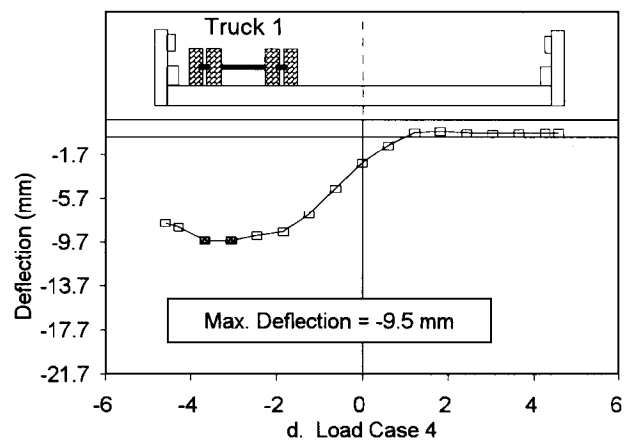
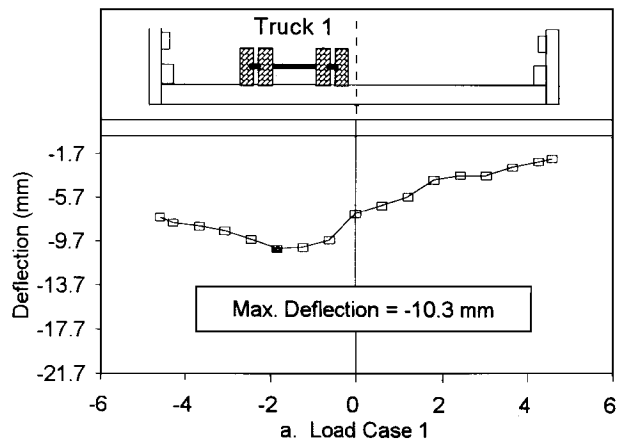


Figure 19—Transverse deflection for load test 1, measured at midspan. Solid box denotes maximum deflection. Bridge cross sections and truck positions are shown to aid interpretation and are not to scale.

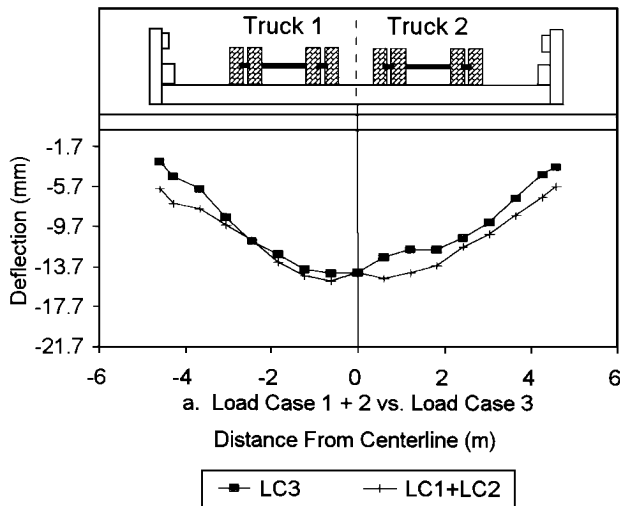


Figure 20—Load test 1: measured deflection of load cases 1 plus 2 compared with load case 3.

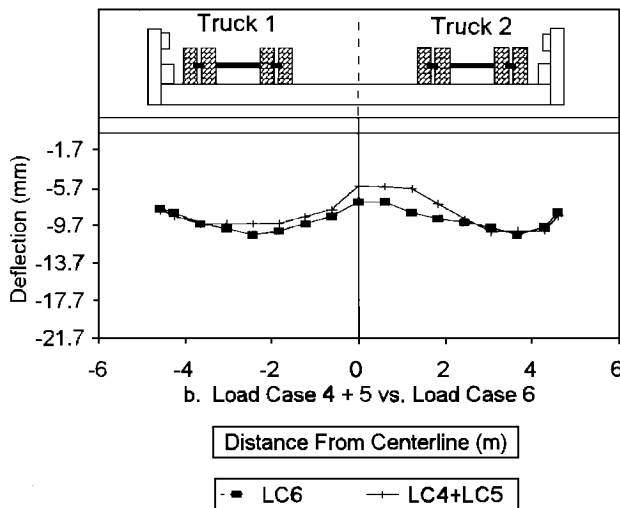


Figure 21—Load test 1: measured deflection of load cases 4 plus 5 compared with load case 6.

with a magnitude of  $-15.1$  and  $-14.3$  mm, respectively. For load cases 4 plus 5 and 6, the location was under the outside wheel line, with a magnitude of  $-10.3$  and  $-10.7$  mm, respectively.

### Load Test 2

Transverse deflections for each load case are shown in Figure 22. As stated for load test 1, for some load cases, there was no apparent single point of maximum deflection and other points deflected the same magnitude. In all cases, the deflection was typical of the linear elastic orthotropic plate behavior of stress-laminated bridges and the location of maximum measured deflection corresponded to the location observed in similar bridges (Ritter and others 1995).

The same deflection comparisons that were made for load test 1 were made for load test 2. For load cases 1 and 2 (Fig. 22a,b), the maximum deflection location was under the truck and the magnitude was  $-7.1$  and  $-8.7$  mm, respectively. For load cases 4 and 5 (Fig. 22d,e), the location was under the outside wheel line and the magnitude was  $-9.0$  and  $-10.3$  mm, respectively. Load cases 1 plus 2 compared with load case 3 and load cases 4 plus 5 compared with load case 6 are shown in Figures 23 and 24. For load cases 1 plus 2 and 3, the maximum deflection location was at the centerline with a magnitude of  $-13.3$  and  $-12.7$  mm, respectively. For load cases 4 plus 5 and 6, the location was under the outside wheel line with a magnitude of  $-10.4$  and  $-9.9$  mm, respectively.

### Load Tests 1 and 2 Comparison

A comparison of the maximum deflection magnitudes of the load tests showed a decrease in the magnitude with an increase in interlaminar compression. The expected change in load distribution with increased interlaminar compression should be most pronounced for the truck positions of load cases 1 to 3. The interlaminar compression was increased 67% from load test 1 (241.3 kPa) to load test 2 (723.9 kPa). For load test 1, load cases 1 to 3 (Figs. 19a,c), the magnitudes were  $-10.3$ ,  $-9.5$ , and  $-14.3$  mm, respectively. For load test 2, load cases 1 to 3 (Figs. 22a-c), the magnitudes were  $-7.1$ ,  $-8.7$ , and  $-12.7$  mm, respectively. The decrease in deflection between load tests 1 and 2 for load cases 1 to 3 were 31%, 8%, and 11%, respectively, with an average 17% decrease in maximum deflection for the 67% increase in interlaminar compression.

### Analytical Assessment

Measured deflections were compared with theoretical deflections for all load cases in each load test (Figs. 25, 26). The theoretical deflections were similar to the measured deflections, especially for load test 2. In general, the measured deflections were slightly less than the theoretical. For both load tests, the guardrail appeared to stiffen the edge significantly, as seen in load cases 4 to 6.

Figure 27 shows the theoretical deflections for HS 20-44 loading for load cases 3 and 6 for each load test. For load test 1, the predicted maximum deflection was  $-21.6$  mm or  $1/435$  of the span length center-center of bearing and  $-18.1$  mm or  $1/522$  for load cases 3 and 6, respectively. For load test 2, the predicted maximum deflection was  $-19$  mm or  $1/497$  and  $-16.1$  mm or  $1/585$  for load cases 3 and 6, respectively. For both load tests, the deflection was well within the original design limit of  $1/360$ . However, for both load tests, load case 3 slightly exceeded the allowable live load deflection of  $1/500$  recently established by AASHTO (1991). Comparing load tests 1 and 2, the decrease in the theoretical maximum deflection with the increase in interlaminar compression was 12% for load case 3.

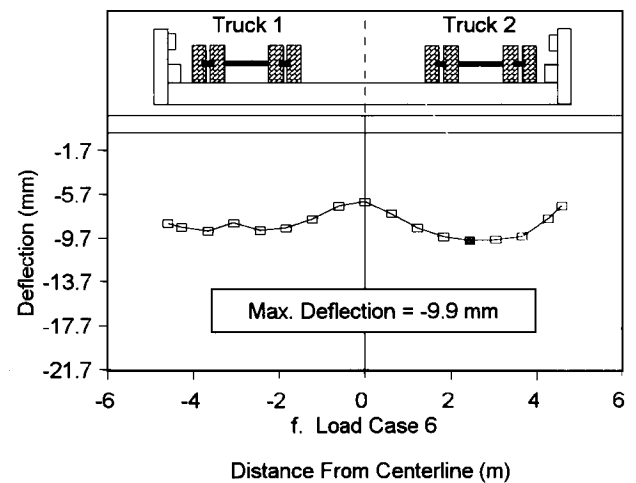
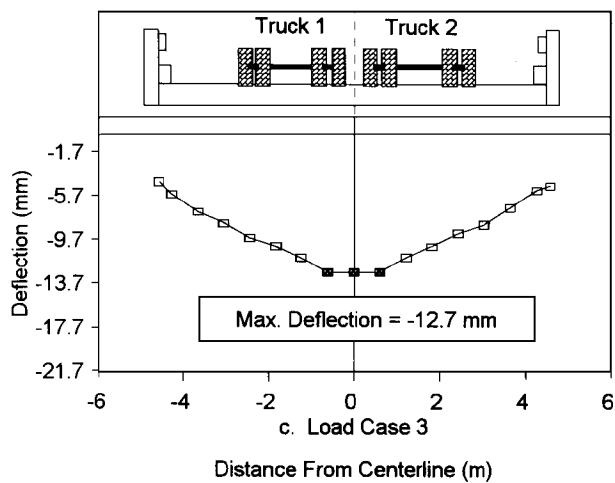
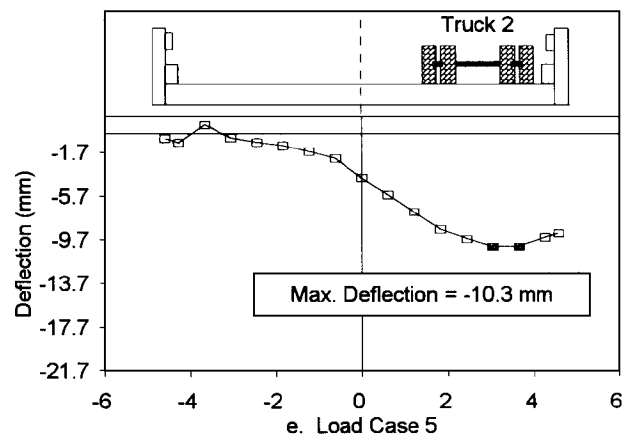
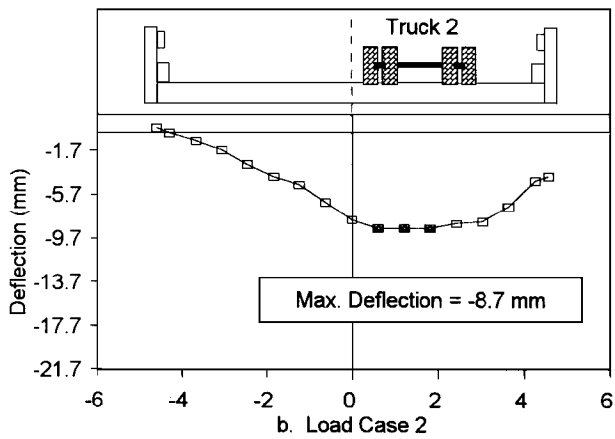
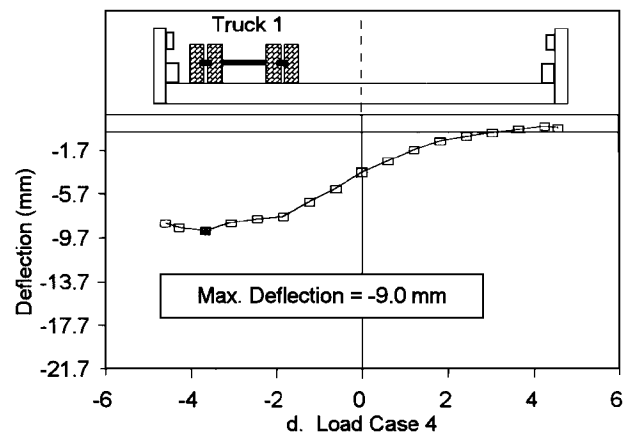
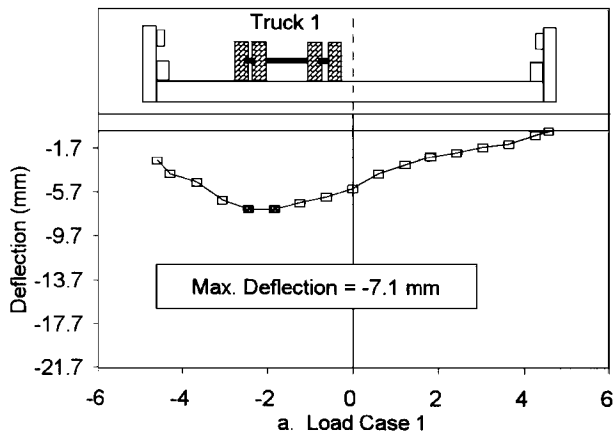


Figure 22—Transverse deflection for load test 2, measured at the midspan. Solid box denotes maximum deflection.

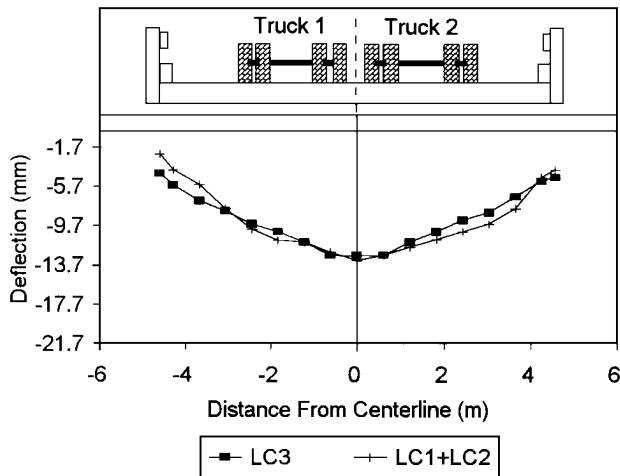


Figure 23—Load test 2: measured deflection of load cases 1 plus 2 compared with load case 3.

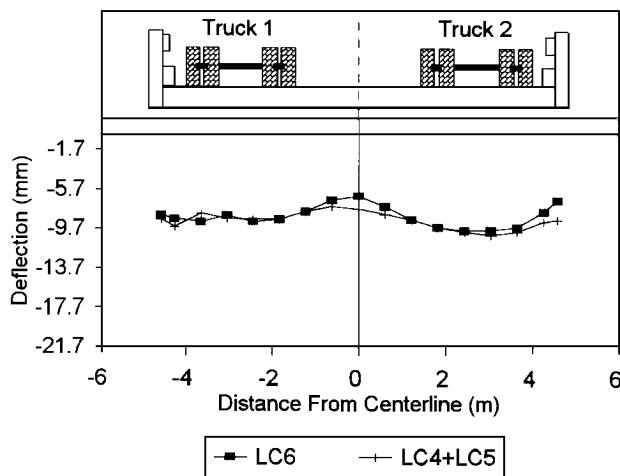


Figure 24—Load test 2: measured deflection of load cases 4 plus 5 compared with load case 6.

## Condition Assessment

Condition assessments of the Pueblo County 204B bridge indicated that structural and serviceability performance were good. Inspection results for specific items follow.

### Deck Camber

No camber was built into the deck during installation. After placement of the guardrail system and asphalt wearing surface, measurements indicated a sag of 12.7 mm as a result of the dead load. Measurements in vertical alignment at the end of the monitoring period indicated no change. This is typical of other stress-laminated decks that have shown little if any loss in camber as a result of creep, even at higher moisture content levels (Ritter and others 1995).

## Wood Components

The outside deck laminations are generally in good condition. There has been some (up to 9.5 mm) plate crushing into the outside lamination on the west side (Fig. 28). This is probably due to undersized bearing plates, especially considering the high moisture content of the laminations. Checking was evident on the top surfaces of rail members exposed to wet-dry cycles. Checking was most pronounced in the end grain of the timber rail posts. In addition, the top of the bridge rail showed minor checking, but the depth of the checks did not appear to penetrate the preservative treatment envelope of the member. Inspection showed an excessive amount of preservative leaching on some exposed surfaces.

## Wearing Surface

Inspection of the asphalt wearing surface indicated 12.7-mm-wide transverse cracks at the ends of the span (Fig. 29). Otherwise, the wearing surface was in good condition with no other signs of distress, such as cracking, rutting, or shoving.

## Bar Anchorage System

All bearing and anchor plates appeared to be in good condition, with no signs of bending or corrosion. The exposed steel bars and hardware showed no visible signs of corrosion, except at the ends of the bars where minor corrosion appeared as a result of the galvanized coating having been stripped from the stressing bars, thus exposing uncoated steel. This occurred on only some of the bars, because the nuts were not adequately oversized to compensate for galvanizing and were forced into the bars during construction.

## Conclusions

After the initial 3 years of FPL monitoring and a check of the bar force after 6 years, the Pueblo County 204B bridge is exhibiting good performance. Several minor deficiencies include the crushing of the bearing plates into the outer laminations and the slight sag of the deck. In addition, the deflection is slightly greater than that allowed by current AASHTO guidelines, although it is within the original design allowable. This bridge was designed prior to the current AASHTO guidelines being available. Design and construction in accordance with the current guidelines will alleviate these minor problems. Most concerns from the condition assessment are minor and can be addressed as part of routine maintenance of the bridge. For future bridges, the most critical item is the moisture content of the timber laminations. The moisture content at installation should be near the anticipated equilibrium moisture content for the area. Otherwise, a maintenance schedule of restressing two to three times during the first 6 to 10 years will be necessary.



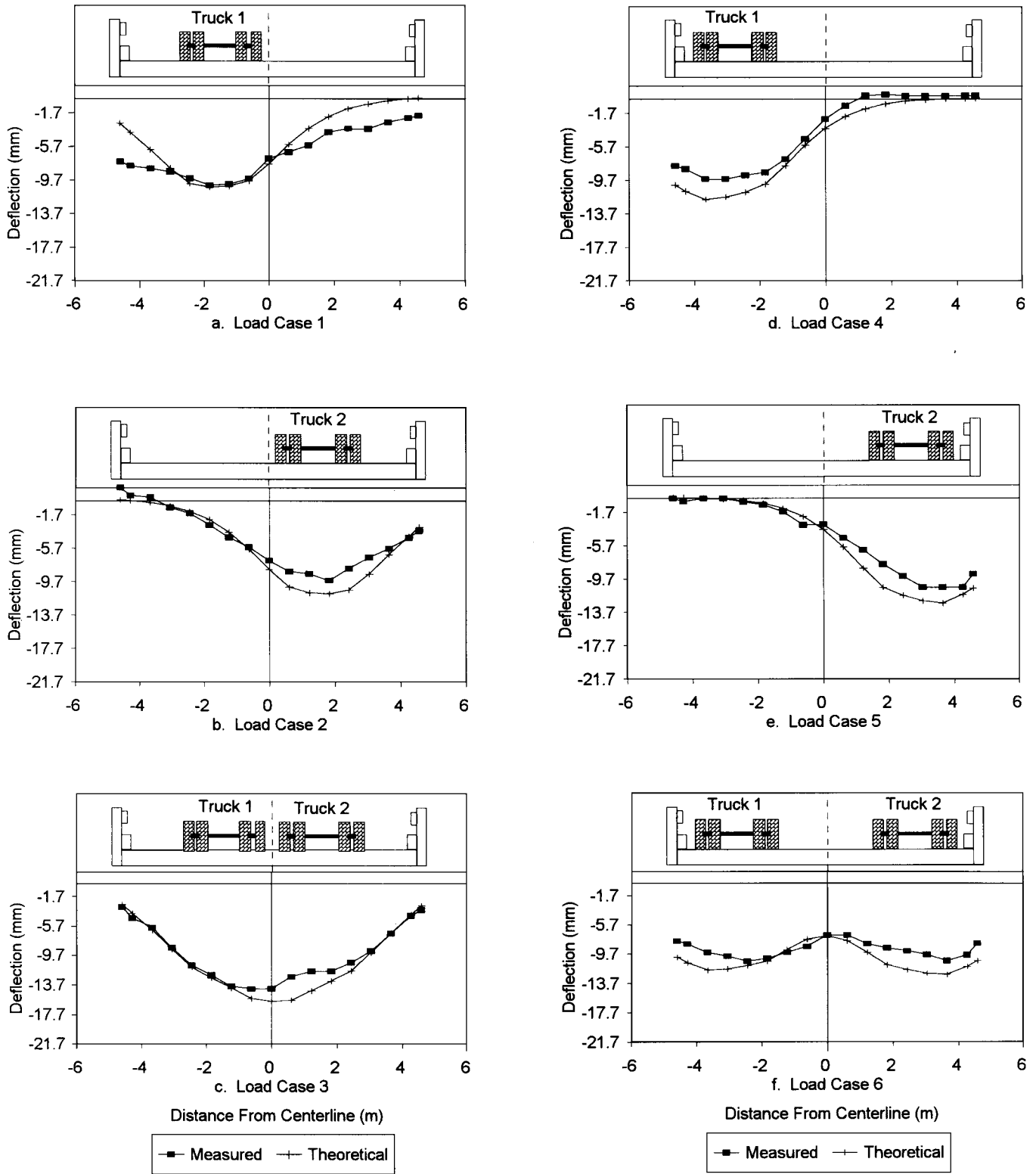
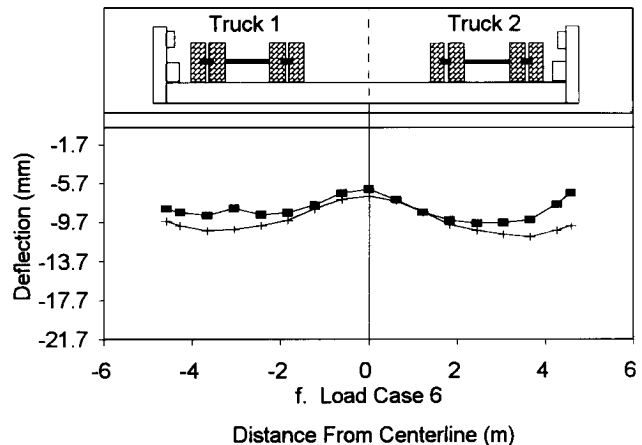
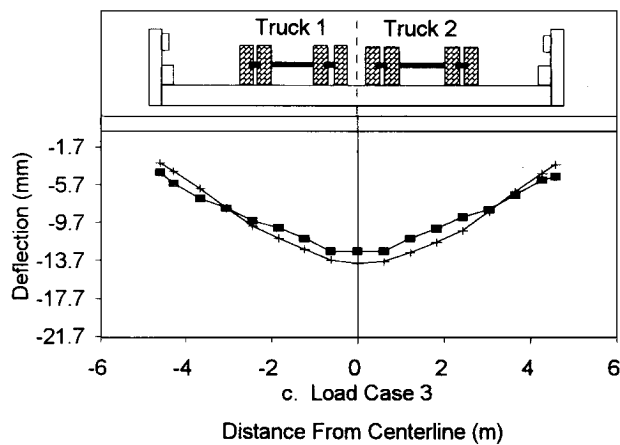
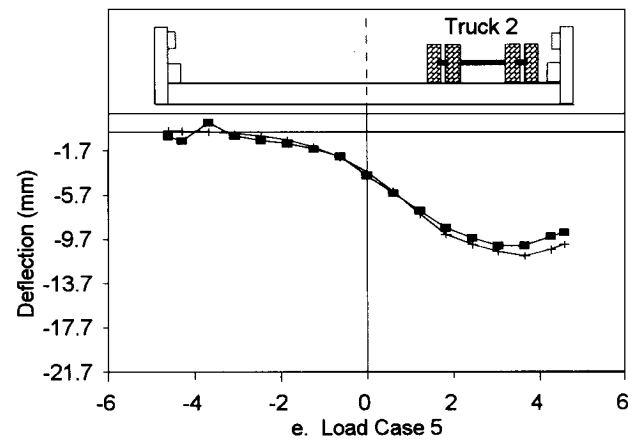
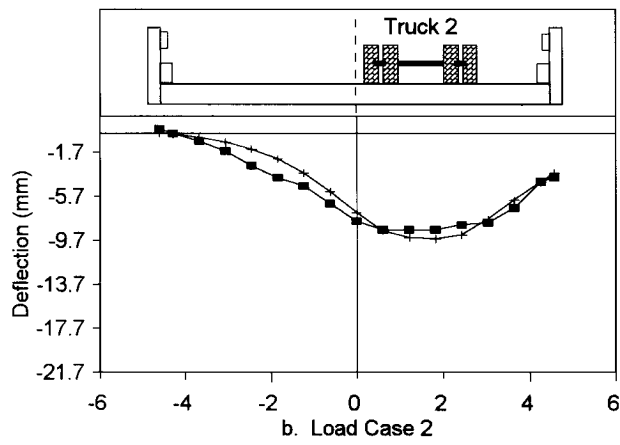
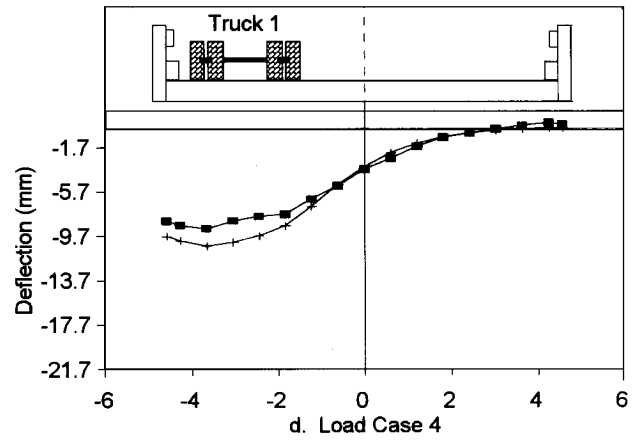
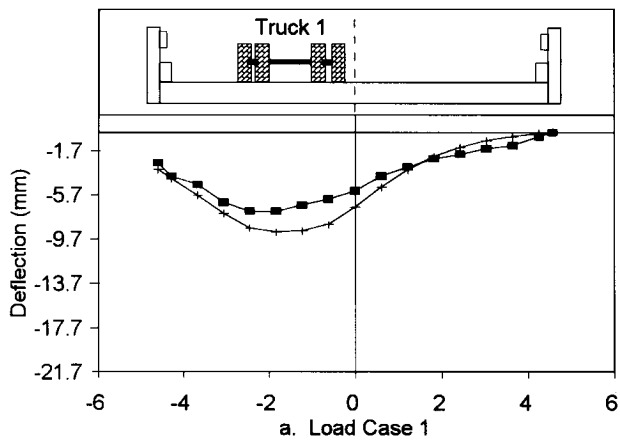


Figure 25—Comparison of theoretical and measured deflections at the midspan of the bridge for the test trucks, load test 1, load cases 1 through 6.



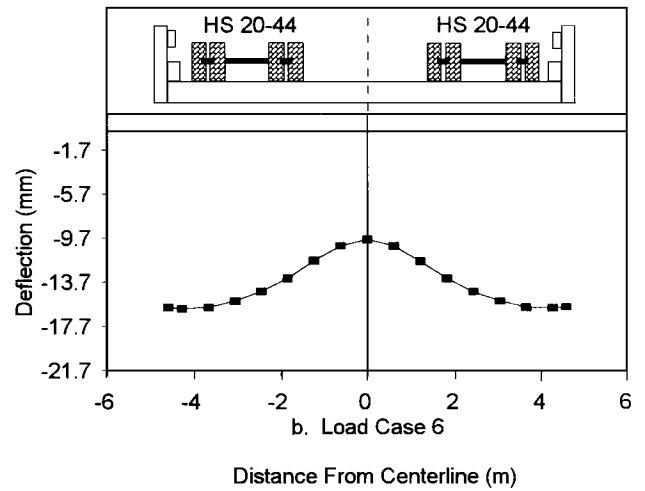
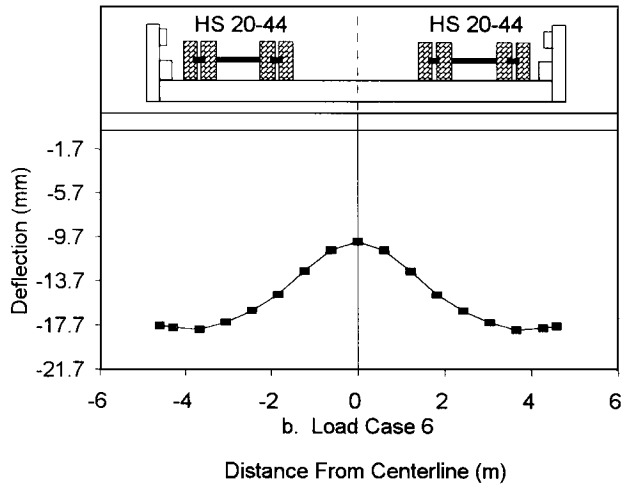
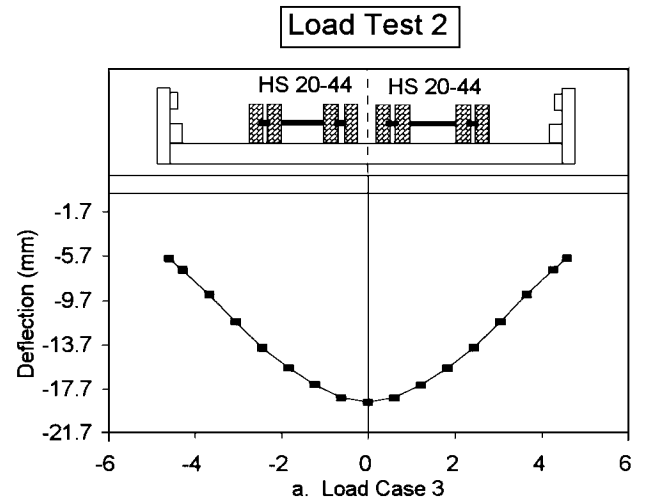
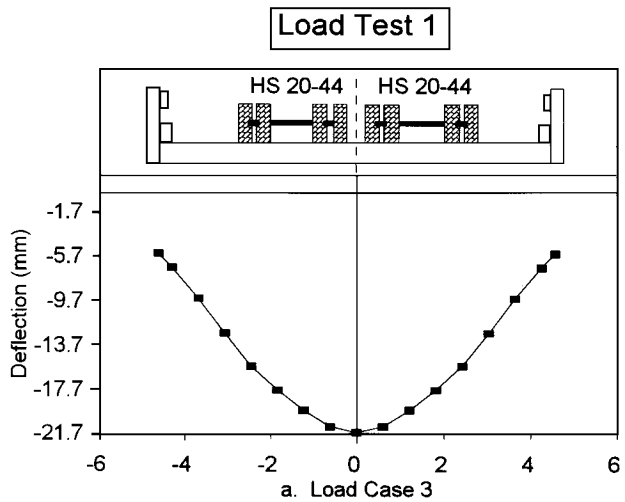
Distance From Centerline (m)

■ Measured    + Theoretical

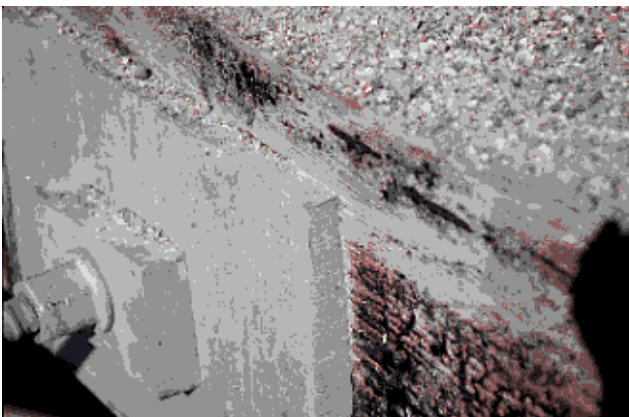
Distance From Centerline (m)

■ Measured    + Theoretical

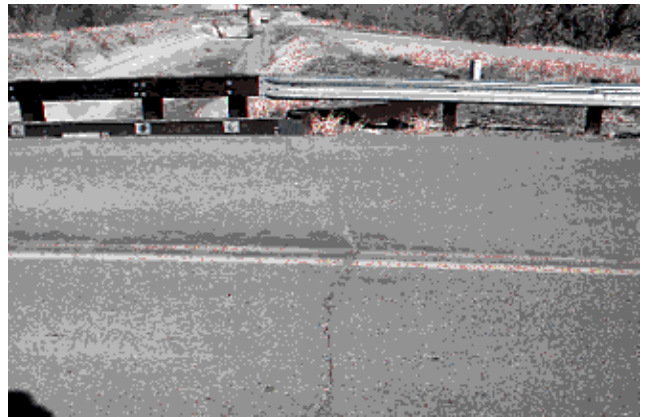
Figure 26—Comparison of theoretical and measured deflections at the midspan of the bridge for the test trucks, load test 2, load cases 1 through 6.



**Figure 27—Theoretical deflections at the midspan of the bridge for two HS 20–44 trucks, load test 1, load cases 3 and 6 and load test 2, load cases 3 and 6.**



**Figure 28—Crushing of the bearing plates into the outside lamination, up to 9.5 mm.**



**Figure 29—Transverse crack (12.7 mm wide) in asphalt at both span ends.**

Based on the extensive monitoring conducted since bridge construction, the following observations and recommendations are given:

- The average trend in moisture content of the deck indicates that the outer 25 to 76 mm have decreased to near the anticipated equilibrium moisture content of approximately 13% to 15%. It is likely that the global moisture content of the deck is continuing to slowly decrease. Cyclic seasonal variations in moisture content are occurring and will continue to occur in the outer 25 to 76 mm of the deck.
- Bar tension force decreased by 67% during the 3-year FPL monitoring. This is attributable primarily to the large decrease in moisture content as well as wood stress relaxation. At the end of the monitoring, the bars were retensioned and during the 3 years since that time, the force has decreased by 50%. This is a slightly less loss compared with the initial 3 years, showing a decrease in the rate of moisture content loss and stress relaxation. Restressing of the bridge should be performed within the next year and will probably not be necessary again for several years.
- Load tests and analysis indicate that the bridge is performing as an orthotropic plate when subjected to static loading. The predicted maximum deflection caused by two lanes of AASHTO HS 20–44 loading was estimated to be 21.6 mm or 1/435 of the center–center of bearings length at 40% of the design bar force and 19 mm or 1/497 at 100% of the design bar force level. Both are within the design allowable of 1/360 but do not meet current AASHTO recommendations of 1/500.
- There is a 12.7-mm sag in the deck as a result of dead load. No camber was built into the deck to offset the effects of dead load.
- The bearing plates have crushed as much as 9.5 mm into the outside laminations. This is probably due to the plates not being adequately sized for the applied force and the reduced strength of the wood as a result of the high moisture content. The crushed areas of the outside laminations are susceptible to deterioration, and preservative should be applied. Wood checking is evident in the exposed end grain of the rail posts and other components. This probably would not have occurred if a bituminous sealer had been applied to the end grain at the time of construction.
- The asphalt wearing surface is in good condition except for transverse cracks at the ends of the bridge. These cracks should be filled to prevent moisture migration to the ends of the deck laminations and timber abutments.
- The ends of some stressing bars show signs of minor corrosion at locations where the galvanizing was removed during stressing nut placement. This would not have occurred if the nuts had been oversized to compensate for the thickness of the galvanized coating or a cold galvanizing compound had been applied to the stressing bars to replace the removed coating.

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# Appendix—Information Sheet

## General

Name: Pueblo County 204B Bridge  
Location: Pueblo County, Colorado  
Date of Construction: March 22, 1990  
Owner: Pueblo County Highway Department

## Design Configuration

Structure Type: Stress-laminated deck  
Butt Joint Frequency: 1 in every 4 laminations  
transversely, 1.22-m separation  
in adjacent laminations  
longitudinally  
Total Length (out-out): 9.72 m  
Skew: 10 degrees  
Number of Spans: 1  
Span Length (center-center bearings): 9.42 m  
Width (out-out): 9.17 m perpendicular  
Width (curb-curb): 8.56 m perpendicular  
Number of Traffic Lanes: 2  
Design Loading: HS 20-44  
Wearing Surface Type: Asphalt pavement;  
51 to 76 mm thick

## Material and Configuration

### Timber:

Species: Coastal Douglas Fir-Larch  
Size (actual): 102 by 406 mm  
Grade: No. 1 & Better visually graded  
Moisture Condition: 22% to 28% at installation  
Preservative Treatment: Creosote-coal tar solution

### Stressing Rods:

Type: High strength steel thread bar with course  
right-hand thread, conforming to  
ASTM A722.  
Diameter: 25 mm  
Number: 9  
Design Force: 355,840 N, interlaminar  
compression of 717 kPa  
Spacing: 1,219 mm

### Anchorage Type and Configuration:

Discrete Steel Plates: 25 by 305 by 406 mm bearing  
25 by 102 by 152 mm anchor  
Railing and Curb: 152- by 305-mm glulam rail  
152- by 305-mm solid-sawn posts  
305- by 305-mm solid-sawn curb

