Pole Creek Metal-Plate-Connected Truss Bridge

Michael H. Triche, Civil and Environmental Engineering, University of Alabama
Michael A. Ritter, Forest Products Laboratory, USDA Forest Service

Abstract

This paper summarizes the performance of the Pole Creek metal-plate-connected wood truss bridge constructed in the Fall of 1992 in rural Tuscaloosa County, Alabama. This two-lane bridge consists of two simple spans. Span 1 is a bolt-laminated transverse deck supported by multitruss girders; span 2 is a stress-laminated truss system. A monitoring program on the Pole Creek bridge, initiated shortly after construction, has provided information on seasonal variations in lumber moisture content, stressing bar force, static-load test behavior, and overall condition, After 3 years, the monitoring program indicates that the Pole Creek bridge is performing adequately with no structural deficiencies.

Keywords: Metal-plate-connected trusses, stress laminated, bridge testing

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 that provided effective and efficient utilization of wood as a structural material for highway bridges. The USDA Forest Service was assigned responsibility for the development, implementation, and administration of the TBI. The three primary program areas established by the Forest Service are demonstration bridges, technology transfer, and research.

In 1992, as part of the demonstration bridge program, a metal-plate-connected (MPC) wood truss bridge was designed and constructed on Old Fayette Road in Tuscaloosa County, Alabama. The bridge crosses Pole Creek, approximately 16 km (10 miles) north of the city of Tuscaloosa. Traffic consists of passenger vehicles and heavy trucks with an estimated average daily traffic of approximately 100 vehicles.

Background

In many areas, the local timber supply is limited to relatively small-diameter trees that can produce structural lumber of limited dimensions. The wide lumber required for solid-sawn longitudinal decks is often available only at a premium price. Thus, design options that allow for smaller lumber sizes can potentially improve bridge economy and utilization of the forest resource.

The Pole Creek bridge is an innovative use of MPC wood trusses, and we believe this bridge is the first application of such components in roadway bridges. The bridge is 12.2 m (40 ft) long with a 8.5-m (28-ft) curb-to-curb width. It has two 6.1-m (20-ft) spans, and structural elements are MPC wood trusses fabricated from nominal 50- by 100-mm (2- by 4-in.) and 50- by 150-mm (2- by 6-in.) Southern Pine lumber treated

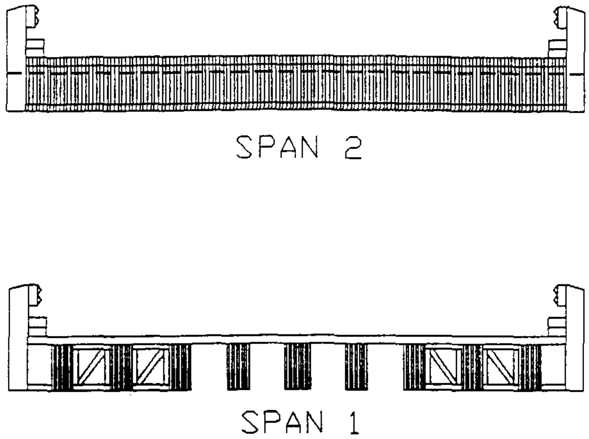


Figure 1—Cross-sections of bridge spans.

with chromated copper arsenate (CCA) wood preservative. Span 1 utilizes a bolt-laminated transverse timber deck over multitruss girders; span 2 consists of stress-laminated trusses that form the deck and transfer loads to the substructure (Fig. 1).

The Pole Creek bridge was designed for standard HS20-44 truck loading, based on load provisions of the American Association of State Highway and Transportation Officials (AASHTO), *Standard Specifications for Highway Bridges* (AASHTO 1992). Because bridge design using MPC trusses is not specifically addressed by AASHTO, conservative assumptions were made regarding the distribution of wheel loads. For the girder span, wheel loads were assumed to be distributed according to the provisions for nail-laminated decks on timber stringers. The stress-laminated trusses were designed assuming that the wheel load was not distributed beyond the tire width.

The individual trusses were designed using provisions of the Truss Plate Institute Design Specification (TPI 1985). A series of designs was completed, with the appropriate fraction of the wheel loads located at different locations along the bridge, in order to find the maximum wood stresses and joint forces. The final truss design used the largest plates and lumber sizes required for wheel loads positioned at any location on the bridge.

The design for the Pole Creek bridge is experimental, and the information gained from monitoring this bridge and similar bridges (Dagher and others 1995) should lead to improvements in future bridges of this type. This will, in turn, lead to the development of design specifications for consideration by AASHTO.

This paper summarizes the field performance of the Pole Creek bridge after 4 years of service. Compete details on the development, design, construction, and cost of this bridge will be presented in a document published by the USDA Forest Service, Forest Products Laboratory (FPL).

Research Methods

A 3-year monitoring plan was established to evaluate the field performance of the Pole Creek bridge. In December 1992, approximately 2 months after the bridge was constructed, it was load tested and load cells were installed on three of the seven stressing bars. Since that time, visits to the bridge have been made at least once a month to measure stressing bar force, obtain moisture content readings, and perform visual inspections. The monitoring program concluded with a second load test that was conducted in June 1996.

Bar Force

For the stress-laminated span, calibrated load cells were installed between the anchorage and bearing plates on three of the seven stressing bars (Ritter and others 1991). The stressing bars are 15.88 mm (5/8 in.) in diameter and have high strength steel bars with an allowable load of 129.0 kN (29,000 lb). As part of the construction process, the bars were tensioned four times during the 2 months prior to the installation of the load cells. Load cell readings were taken monthly with a portable strain indicator.

Moisture Content

Moisture content readings were taken using *an* electrical resistance meter at 10 locations. These locations were distributed throughout the bridge and included the trusses on either edge of the bridge, the bottom chord of the trusses beneath the bridge, and the bridge deck of the girder span.

Load Testing

The first static-load test of the Pole Creek bridge was completed in December 1992, and is reported elsewhere (Triche and others 1994, Triche and Ritter 1996). The second static-load test was conducted June 10, 1996, approximately 3.5 years after bridge construction. The testing consisted of positioning fully loaded trucks on each of the spans and measuring the resulting deflections at a series of transverse locations at midspan. Measurements of bridge deflections were taken prior to testing (unloaded), for each load case (loaded), and at the conclusion of testing (unloaded). At the time of load testing, the average bar force was approximately 53.4 kN (12,000 lb). This bar force produces 275.8 kPa (40 lb/in^2) of transverse compression, which is the

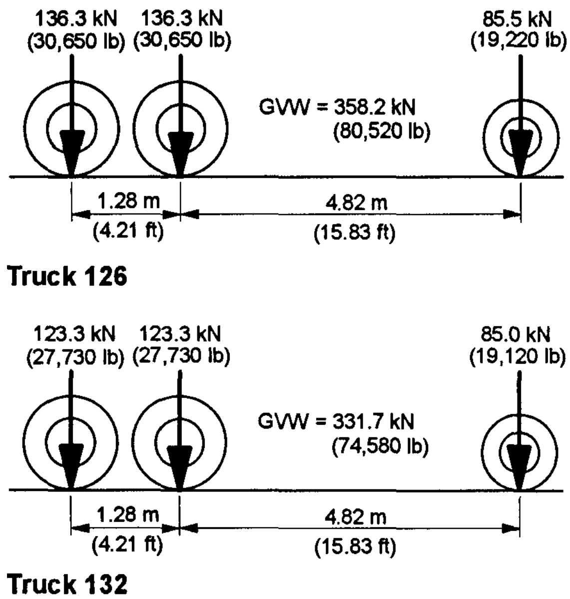


Figure 2—Load test truck configurations and axle loads. The track width of each truck was 1.83 m (6 ft), measured center-to-center of rear tires.

minimum level of prestress recommended for stress-laminated deck bridges in service (Ritter 1990).

Load test vehicles consisted of two fully loaded dump trucks: truck 126 with a gross vehicle weight (GVW) of 358.2 kN (80,520 lb) and truck 132 with a GVW of 331.7 kN (74,580 lb) (Fig. 2). The vehicles were positioned longitudinally on each span so that the two rear axles were centered at midspan and the front axles were off the bridge. For the stress-laminated and girder spans, the vehicles faced west and east, respectively. Transversely, the vehicles were placed for six load cases (Fig. 3). For load cases 1, 2, and 3, the vehicles were placed at the center of the bridge width, with the inside wheel lines 610 mm (2 ft) from centerline. For load cases 4, 5, and 6, the vehicles were placed at the edges of the bridge, with the inside wheel lines 1.83 m (6 ft) from centerline. Deflection measurements from an unloaded to loaded condition were obtained by placing calibrated rules on the underside of the deck and reading values with a surveyor's level to the nearest 0.2 mm (0.01 in.).

Results and Discussion

This paper presents selected results of the monitoring program. These include bar force, moisture content, condition assessment, and load test behavior. Comprehensive analyses of the performance and load testing of the Pole Creek bridge will be published at a later date.

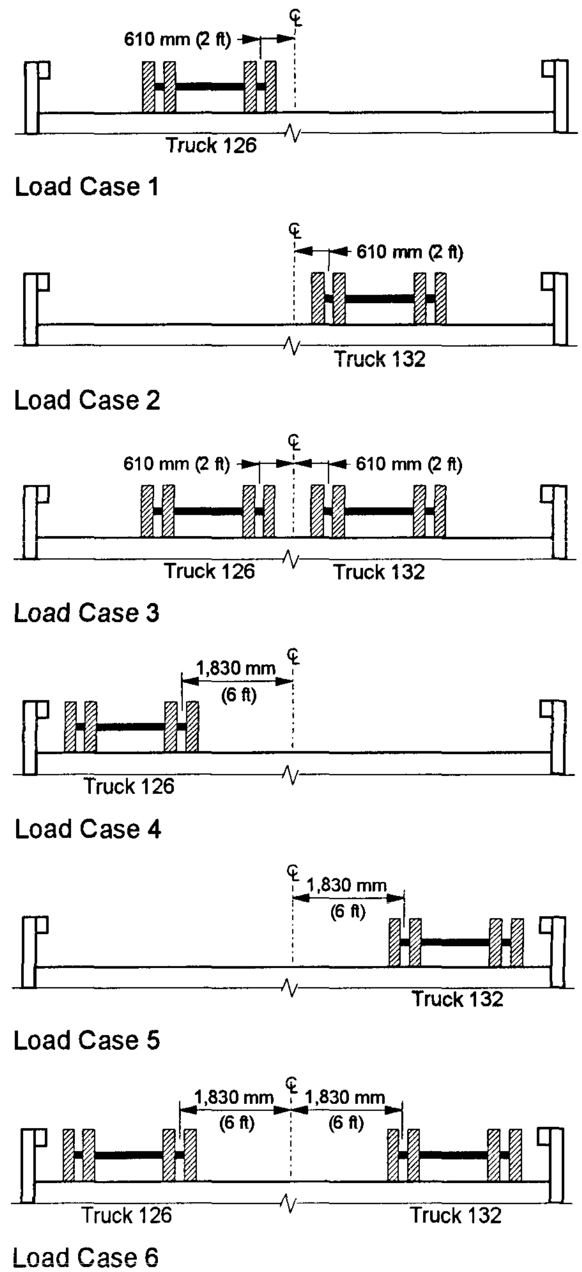


Figure 3—Load test transverse positions (looking east). For all load cases, the rear truck axles were centered at midspan and the front axles were off the bridge.

Bar Force

Figure 4 shows the bar force compared with time after the fourth bar tensioning. Cell 136 measures the force in the bar located at midspan, and cells 135 and 137 are located on the bars adjacent to the midspan bar. Approximately 5 months after the load cells were installed, the stress was removed from the bars, a new load cell zero balance was obtained, and the bars were

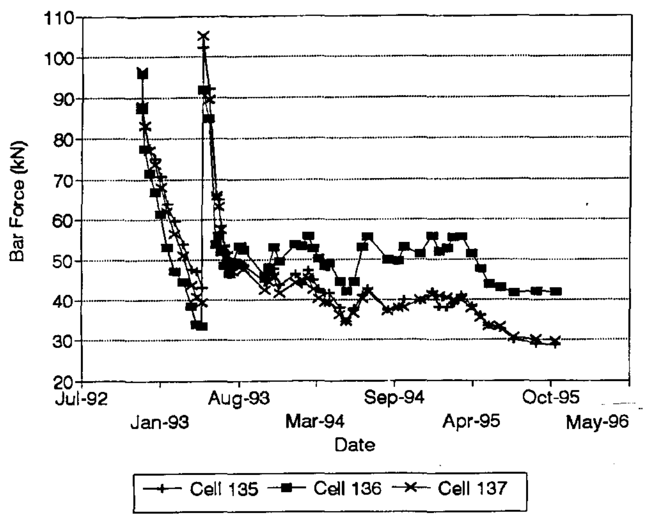


Figure 4—Average trend in bar tension force.

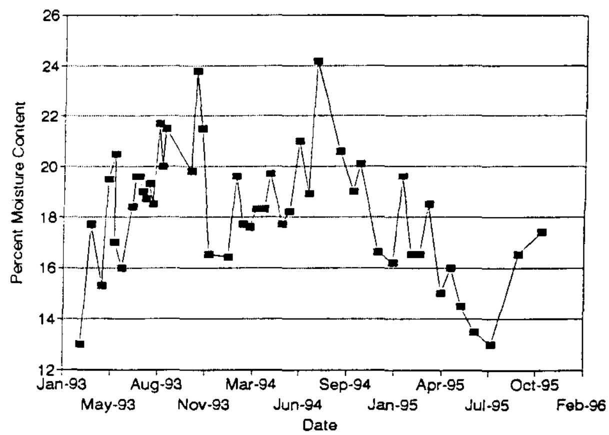


Figure 5—Seasonal variations in moisture content from measurements taken on the underside of the stress-laminated span.

retensioned. This was the fifth bar tensioning and is the reason for the sharp increase in bar force between January and August 1993. Since that retensioning, the bar force has continued to decrease but at a much slower rate. As a result, a sixth bar retensioning (not shown in Fig. 4) was performed in June 1996, shortly before the second load test.

The significant and continued loss in bar force is not typical of stress-laminated deck bridges. The loss in bar force for these stress-laminated trusses is primarily the result of two factors: (1) gaps between the trusses caused by nails used to fasten the individual trusses into bundles for handling purposes and (2) small gaps under individual metal-connector plates that were gradually reduced under the transverse stress.

Moisture Content

During the monitoring period, unadjusted moisture content readings ranged from 16%-27%. These values converted to an approximate range of 13%–24% when adjustments for temperature and CCA treatment effects were considered. Moisture content readings were calibrated to ovendry samples and, according to these data, meter readings averaged about 3% greater than those determined by the ovendry method.

Figure 5 shows the variation in moisture content over time for a single location on the underside of the stress-laminated span. The initial moisture content of the lumber was approximately 12% and has gradually increased to an average of approximately 18%. In 1995, Alabama experienced an unusually hot and dry spring and early summer. During this period, the moisture content decreased to approximately 13%. Since that time, the moisture content has increased and is currently near the 18% average. Referring to Figure 4, there was also an increase in bar force loss during the hot and dry period; bar force began to stabilize for the last two readings as moisture content increased.

Load Test Behavior

Results for the second load test of the stress-laminated and girder spans follow. In each case, transverse deflection plots are shown at the bridge midspan as viewed from the west side (looking east). For each load test, no permanent residual deformation was measured at the conclusion of the testing. In additional, there was no detectable movement at the supports.

Stress-Laminated Span— Transverse deflections for the stress-laminated span are shown in Figure 6. As shown, the deflection profiles are approximately symmetric for corresponding load cases. For load cases 1 and 2, the maximum measured deflections occurred under the outside wheel lines and measured 2.0 mm (0.08 in.) for load case 1 and 2.2 mm (0.09 in.) for load case 2. For load case 3, with both vehicles on the span in the same relative positions, the maximum measured deflection of 2.5 mm (0.10 in.) occurred under the inside wheel line of truck 126. Maximum deflections for load cases 3 and 4 were measured under or adjacent to the outside wheel line and were 3.6 mm (0.14 in.) for load case 4 and 2.7 mm (0.11 in.) for load case 5. For load case 6, the maximum deflection was the same as load case 4 and measured 3.6 mm (0.14 in.) under the outside wheel line of truck 126.

Assuming linear elastic behavior, uniform material properties, and accurate load test methodology, the sum of the bridge deflections for the individual vehicles

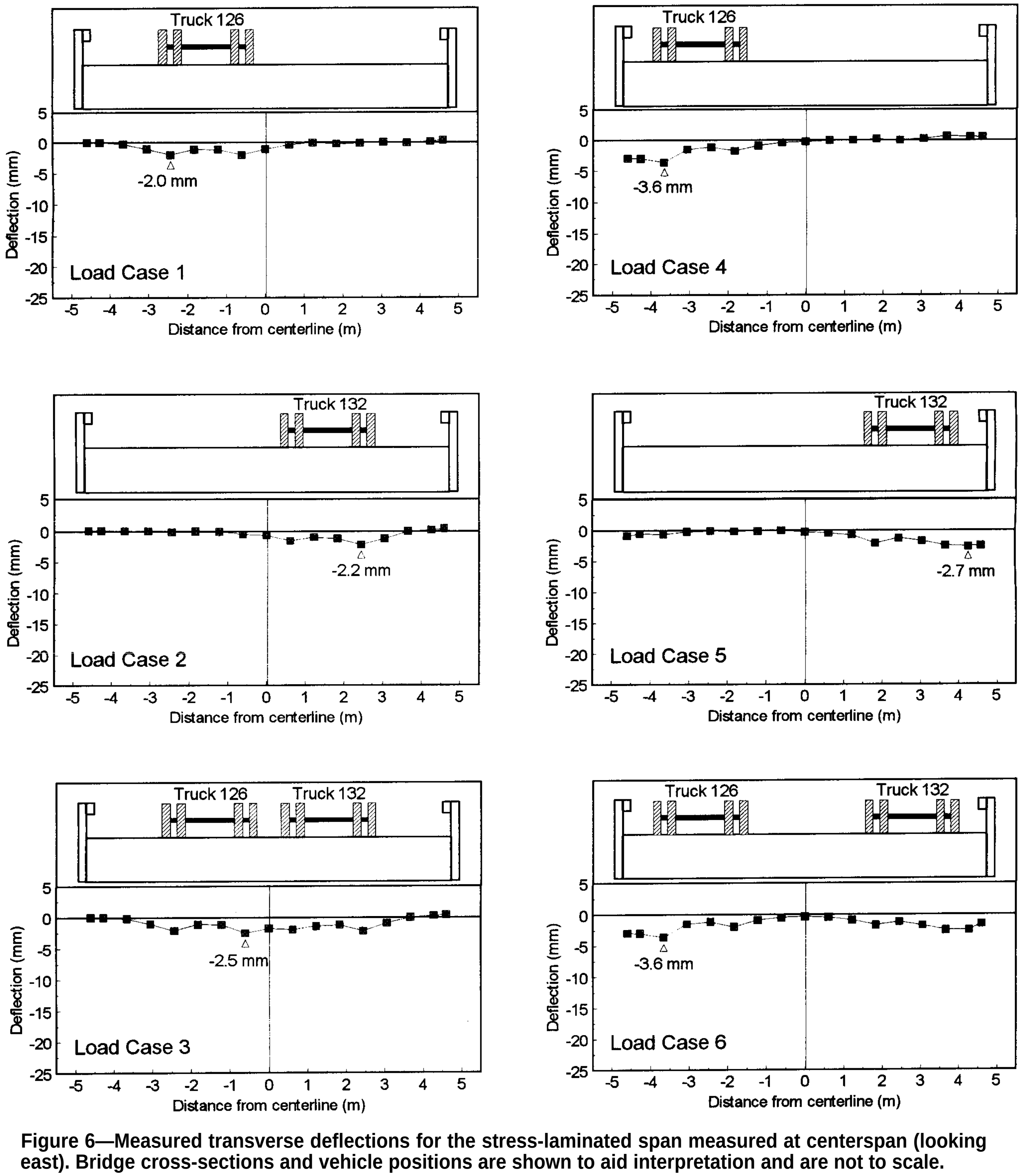


Figure 6—Measured transverse deflections for the stress-laminated span measured at centerspan (looking east). Bridge cross-sections and vehicle positions are shown to aid interpretation and are not to scale.

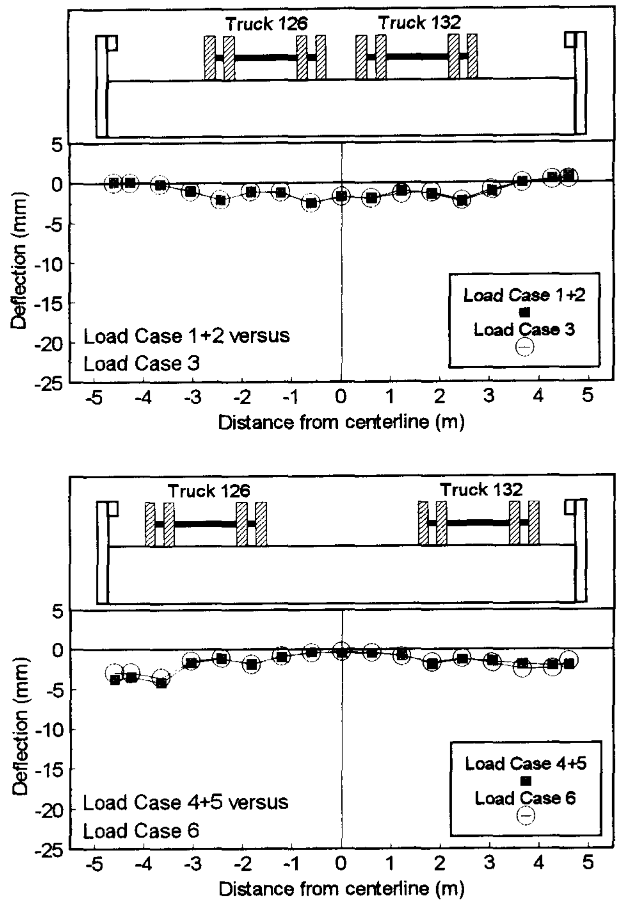


Figure 7—Comparison of results for the stress-laminated span: load cases 1 and 2 compared with load case 3 (top); load cases 4 and 5 compared load case 6 (bottom).

should be the same as the deflections for both vehicles placed simultaneously. Using superposition, the measured deflections for load cases 1 and 2 compared with load case 3 and load cases 4 and 5 compared with load case 6 are given in Figure 7. As shown, the deflections are virtually identical, with only slight differences along the edges of the bridge for load case 6.

Truss Girder Span— Transverse deflections for the truss girder are shown in Figure 8. As with the stress-laminated span, the deflection profiles are approximately symmetric for corresponding load cases. For load cases 1 and 2, the maximum measured deflections occurred in the girder near the outside wheel line and measured 4.7 mm (0.19 in.) for load case 1 and 5.0 mm (0.20 in.) for load case 2. For load case 3, with both vehicles on the span, the maximum measured deflection of 5.6 mm (0.22 in.) occurred near the inside wheel line of truck 126. Maximum deflections for load cases 3 and 4 were measured at the outside girder and measured 8.4 mm (0.33 in.) for load case 4 and 6.0 mm (0.26 in.) for load case 5. For load case 6, the maximum deflection measured 8.6 mm (0.34 in.) at the outside girder near truck 126.

Measured deflections for load cases 1 and 2 compared with load case 3 and load cases 4 and 5 compared with load case 6 are shown in Figure 9. As with the stress-laminated span, the deflections are nearly identical.

Load Test Comparison

In comparing the load test results for the stress-laminated span and the truss girder span, it is evident that the maximum deflections for the stress-laminated span are approximately half those measured on the truss girder span. This response was expected because there are substantially more trusses in the stress-laminated span and the longitudinal stiffness is greater. The plots also indicate that the deflections are much more localized for the stress-laminated span and there is a significant increase in deflection along the edges of the bridge for the truss girder span. Again, this was expected due to the difference in the relative transverse stiffness of the two spans. For the stress-laminated span, transverse stiffness results from the effects of stress laminating. For the truss girder span, transverse stiffness is a function of the stiffness of the nominal 150-mm (6-in.) bolt-laminated deck, and the transverse stiffness is considerably less than that of the stress-laminated span.

Condition Assessment

The general condition of the Pole Creek bridge was assessed at the time of installation and approximately every 3 months thereafter. These assessments involved visual inspections, bridge and component measurements, and photographic documentation, specifically the condition of the asphalt wearing surface, stressing bars and anchorage systems, and the metal-connector plates. Visual inspections revealed only a few minor deficiencies.

Cracks in the asphalt wearing surface were common on the girder span. This span utilized a bolt-laminated deck, and movement, both within and between panels as a result of vehicle loading, caused reflective cracking. The wearing surface for the stress-laminated span, which experienced the same vehicle loading, showed no cracking.

Regarding suitability of MPC trusses for components of a stress-laminated bridge system, several problems are apparent. Most of these are due to the use of a crowned pile cap. When the bridge was placed on the crowned abutment, the bottom of the trusses followed

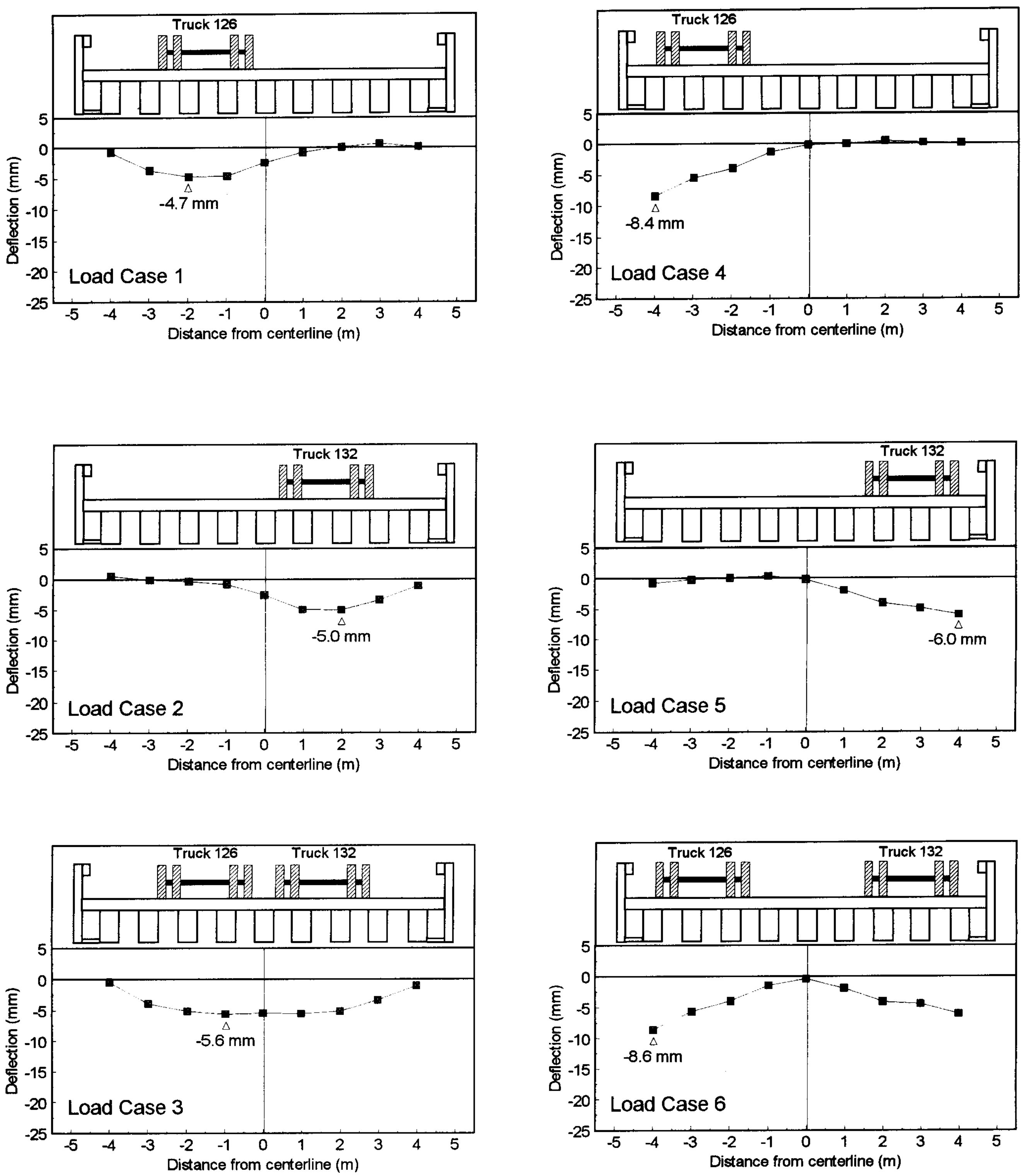
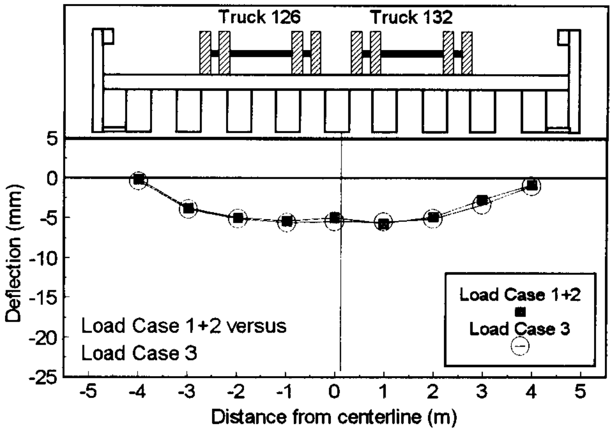


Figure 8—Measured transverse deflections for the girder span measured at centerspan (looking east). Bridge cross-sections and vehicle positions are shown to aid interpretation and are not to scale.



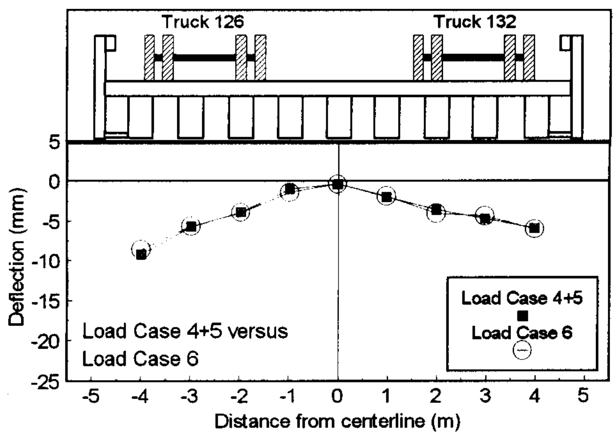


Figure 9—Comparison of results for the girder span: load cases 1 and 2 compared with load case 3 (top); load cases 4 and 5 compared with load case 6 (bottom).

the slope of the abutment cap. When the bars were tensioned, the trusses became horizontal and the trusses along the edge of the bridge lifted above the cap. Modified design details, including the use of a flat pile cap and modified stressing bar positioning, could eliminate these problems in future bridges. There are also some instances of local crushing of the lumber under the bearing plates at the stressing bar anchorages.

No instance of corrosion on the metal-connector plates was detected. Corrosion was addressed in the design by using a galvanized coating and providing an impermeable membrane above the trusses. To date, this design decision seems to be adequate in preventing corrosion in this environment.

Concluding Remarks

The innovative MPC Pole Creek wood truss bridge has been in service for approximately 4 years. Bar force loss on the stress-laminated span was substantial at the beginning of the monitoring period but has remained relatively stable for the past 2 years. The bolt-laminated deck used for the girder span resulted in extensive cracking on the wearing surface. Results of this study indicate that this type of deck is not suitable for bridge decks intended to have asphalt wearing surfaces. The use of a glulam panelized deck would greatly reduce the cracking problem on the wearing surface.

Aside from the relatively minor deficiencies previously noted, both spans are performing well. However, the simplicity of the girder span makes it an attractive alternative. The girder span system eliminates the need for stressing bars and the associated maintenance required to restress the bars periodically. It more closely resembles traditional bridge systems, and the girder span is easier and less costly to design and construct. With an alternative deck design, this system has immediate potential for widespread usage.

Further investigation of the stress-laminated span is needed to determine the required positioning of stressing bars to prevent unequal compression between the top and bottom chords of the truss that cause out-of-plane bending. The requirement for periodic bar tensioning is a disadvantage of this system.

References

AASHTO. 1992. Standard specifications for highway bridges, 15th ed. Washington, DC: American Association of State Highway and Transportation Officials.

Dagher, H; Caccese, V.; Altimore, F. [and others]. 1995. Using lightweight MPC wood trusses in bridges. In: Sanayei, Masoud, ed. Restructuring: America and beyond. Proceedings, structures congress 13; 1995 April 2–5; Boston, MA. New York: American Society of Civil Engineers: 9–12. Vol. 1.

Ritter, M.A. 1990. Timber bridges: design, construction, inspection, and maintenance. EM 7700-8. Washington, DC. U.S. Department of Agriculture. 944 p.

Ritter, M.A.; Geske, E.A.; McCutcheon, W.J. [and others]. 1991. Methods for assessing the field performance of stress-laminated timber bridges. In: Proceedings, 1991 International timber engineering conference; 1991 September; London, England.

TPI. 1985. Design specification for metal plate connected wood trusses. TPI–85. Madison, WI: Truss Plate Institute. 67 p.

Triche, M.H.; Ritter, M.A.; Lewis, S.L.; Wolfe, R.W. 1994. Design and field performance of a metal-plate connected wood truss bridge. In: Baker, N.C; Goodno, B.J., eds. Proceedings of structures congress XII: 1994; April 24–28; Atlanta, GA. New York: American Society of Civil Engineers: 1310–1315. Vol. 2.

Triche, M.H.; Ritter, M.A. 1996. Three-year evaluation of a metal-plate connected wood truss bridge. In: Ghosh, S.H.; Mohammadi, Jamshid, eds. Building and international community of structural engineers: Proceedings of structures congress XIV; 1996 April 15–18; Chicago, IL. New York: American Society of Civil Engineers: 276–281. Vol. 1.

Acknowledgments

We thank Stuart Lewis and William McAlpine of Alpine Engineered Products for their technical support and donation of materials for this bridge. Thanks also to Terry Wipf, Steven Taylor, M.G. Draft, and S. Bullitt for their assistance in conducting the load tests. Lastly, we acknowledge the many Tuscaloosa County personnel involved in the construction and testing of the bridge.

In: Ritter, M.A.; Duwadi, S.R.; Lee, P.D.H., ed(s). National conference on wood transportation structures; 1996 October 23-25; Madison, WI. Gen. Tech. Rep. FPL- GTR-94. Madison, WI: U.S. Department of Agriculture, Forest Service, Forest Products Laboratory.