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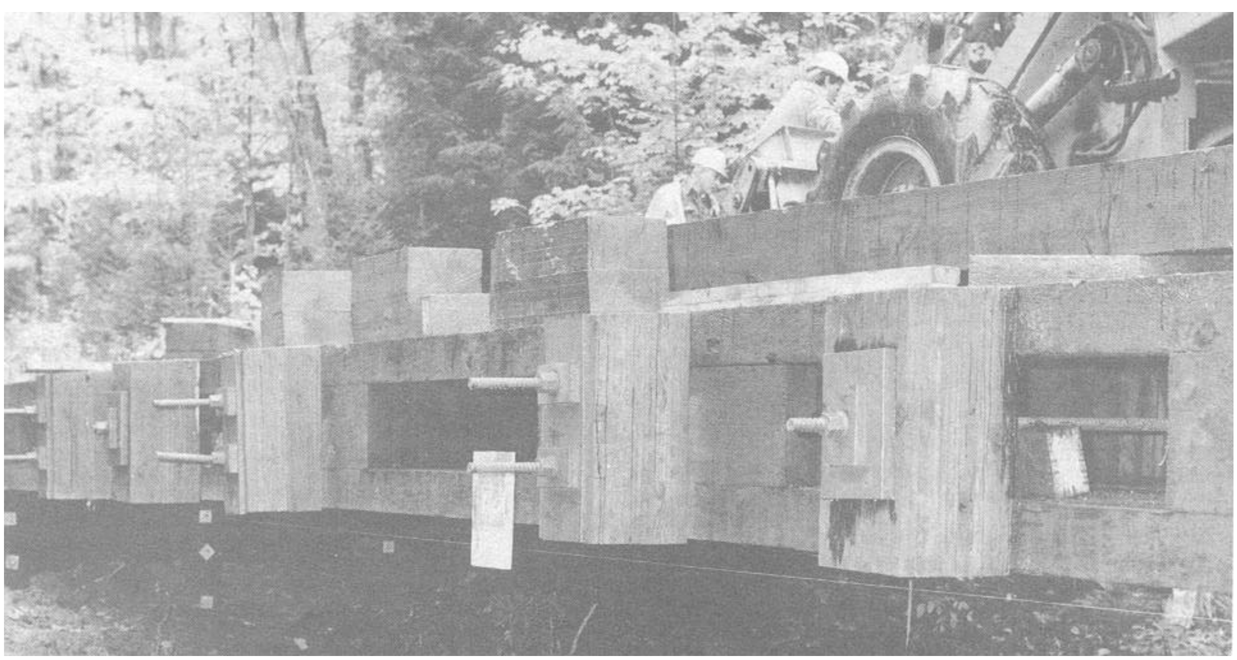
The Mormon Creek Bridge

Performance After Three Years

William J. McCutcheon



Abstract

The Mormon Creek Bridge is an experimental parallel-chord, stress-laminated deck design. It is the first of its kind and has been in service for more than 3 years. The structural performance and experimental features of the bridge have been monitored continuously during this time. Overall, the performance has been excellent. The only problem has been with crushing of the original softwood stress blocks; this was corrected by replacing the blocks with hardwood blocks. The parallel-chord, stress-laminated bridge appears to be a viable alternative to more conventional designs.

Keywords: Timber bridge, stress-laminated, parallel-chord, deflections, stiffness, load test, monitoring, moisture

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Acknowledgments

The Mormon Creek Bridge was designed by Professor Michael Oliva of the University of Wisconsin-Madison. The bridge trusses and the rest of the superstructure were fabricated by Wheeler Consolidated, St. Louis Park, Minnesota. The bridge was constructed by Hiawatha National Forest personnel.

Earl Geske, Forest Products Laboratory, designed, constructed, calibrated, and installed the load cells used to monitor the rod forces. Geske and James Wacker have assisted in many other aspects of the monitoring and load testing. Douglas Crawford has assisted with field work.

The following personnel from the Hiawatha National Forest have been responsible for taking the periodic load cell, camber, and moisture content readings, for ensuring the accuracy of the data, and for transmitting the data to the Forest Products Laboratory: Howard Haselschwardt, Tom Vanleberghe, Kerry Doyle, Geri Rivers, Emily Johnson, Justin Swee, Christine Burgess, and Howard Guindon. Raino Maki and his construction crew have assisted with the stressing operations and structural modifications.

The Mormon Creek Bridge Performance After Three Years

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Introduction

The state of the art in timber bridge design has advanced considerably in the past 40 years. Most notably, traditional nail-laminated transverse decks have been superseded by glued-laminated (glulam) panel decks. Compared to the nail-lam system, the glulam deck provides better structural performance, by supplying a higher degree of plate action, and better protection for the substructure, by providing a more watertight "roof." Design criteria have been promulgated for the glulam deck.

Stress-Lam System

A new concept, the stress-laminated deck, has been developed in Ontario, Canada (Taylor and Csagoly 1979). Originally conceived as a method for rehabilitating existing nail-laminated decks, the method has also been applied to new construction. The basic stress-lam deck consists of solid-sawn laminations with steel stressing rods installed through the laminations; tension in the rods compresses the deck transversely and thereby provides continuity among the individual laminations and imparts a large degree of plate action. However, because of the solid deck, the span capabilities are limited by the depth of material available.

The parallel-chord system is a variation of the stress-lam system that provides greater spans than are possible with the solid deck. Instead of solid-sawn laminations, parallel-chord trusses are built using dimension lumber for the chords and web members. Thus, greater depths and longer spans are possible. The parallel-chord system also uses high-strength steel rods to ensure transverse continuity and plate action.

The stress-lam system, in all its variations, offers great promise for providing efficient and economical longitudinal timber decks. However, questions remain about long-term performance and appropriate design procedures. We need accurate performance data upon which to develop design criteria.

This report describes the construction and performance of a single-lane, experimental, parallel-chord, stress-laminated bridge, the Mormon Creek Bridge. The development, analysis, testing, and construction of stress-laminated, parallel-chord bridges are further described in another Forest Products Laboratory report, Behavior of Stress-Laminated Parallel-Chord Timber Bridge Decks: Experimental and Analytical Studies (Dimakis, Oliva, and Ritter, in press).

Construction of Mormon Creek Bridge

In fall 1987, the Mormon Creek Bridge was constructed over the Mormon Creek on the Hiawatha National Forest in the Upper Michigan Peninsula (Fig. 1). The bridge is 40 ft long (38 ft clear span) and 16-1/2 ft wide. (See Table 1 for metric conversion factors.) It is the first stress-laminated deck bridge of parallel-chord design. The bridge is constructed of creosote-treated No. 1 Douglas Fir, 3-7/8 in. thick. It consists of 50 trusses, with 6-in.-deep top and bottom chords and 12-in.-deep webs. The chords are not full-length; they contain butt joints at some webs. The trusses are constructed entirely with mechanical fasteners; three 5/8- by 16-in. drive spikes connect each chord member to each web block.

The bridge was fabricated in two 8-ft-wide panels with 6-in. spacer blocks between the panels (Figs. 2 to 5). In one panel, six pairs of l-in. steel rods stress the trusses together; in the other panel, six single 1-1/4-in. rods are used. Five single, full-width, l-in. rods join the two panels. The bridge has no guard rails, only curbs. The wearing surface is rough-sawn, untreated,

Figure 1—Mormon Creek Bridge.

Figure 2—Bridge during construction-first panel in place. (M87 0280-18)

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)
pound (lb) (weight)	0.454	kilogram (kg)
lb/in^2 (stress)	6.89	kilopascal (kPa)
kip (1,000 lbf)	4.448	kilonewton (kN)

nominal 2- by 10-in. (standard 38 by 235-mm) planks, covering the entire top deck. The stressing rods transmit their forces into the trusses through full-height (24-in.) glulam blocks, 12 in. deep. Originally, these blocks were 10-3/4 in. wide and made of two pieces of Douglas Fir nailed together; later, one-piece Red Oak blocks, 8-3/4 in. wide, were installed.

The bridge is situated over the Mormon Creek and connects to a gravel road. The traffic mainly consists of logging vehicles, which operate intermittently. The logging vehicles typically are 65 ft long, consist of a truck and pup (trailer), and have a maximum gross weight of 154,000 lb, distributed to 11 axles. In 3-1/2 years, such trucks have crossed the bridge 61 times, empty in one direction, full in the other. Besides the logging traffic, occasional administrative and recreational traffic crosses the bridge.

The bridge has been monitored since it was constructed. Instrumentation was installed on the bridge during construction and shortly thereafter to monitor several aspects of its structural behavior. In addition, the bridge was load tested shortly after construction—in November 1987—and again 1 year later.

Figure 3—Bridge during construction-both panels in place. (M87 0288-4)

Objective

The objective of this study is to test and evaluate the long-term performance of the Mormon Creek Bridge, an experimental parallel-chord stress-laminated design. Engineering assessments of load test results and performance data are needed by code writers and designers who are considering future applications of the parallel-chord system.

Monitoring Program

The monitoring program for the bridge is consistent with general procedures we have established for stress-lam bridges (Ritter and others 1991). The program monitors general bridge condition, rod forces, deck moisture content, camber, and deflection and load distribution.

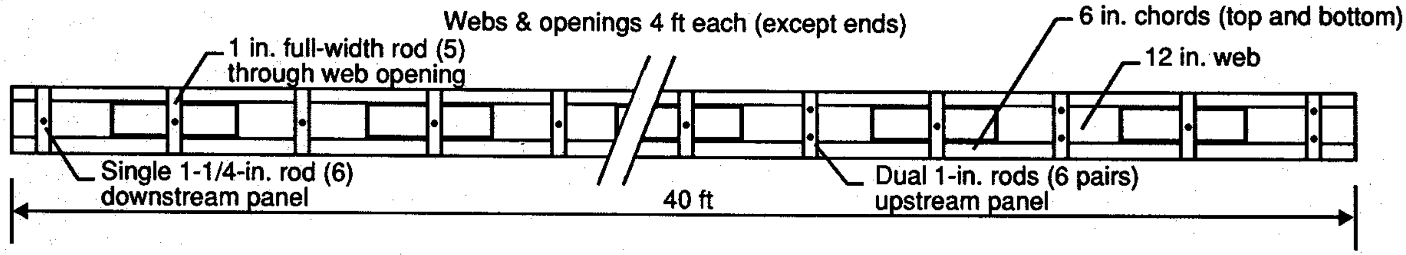

Figure 4—Elevation view of bridge, showing 6-in. chords, 12-in. webs, and layout of stressing rods

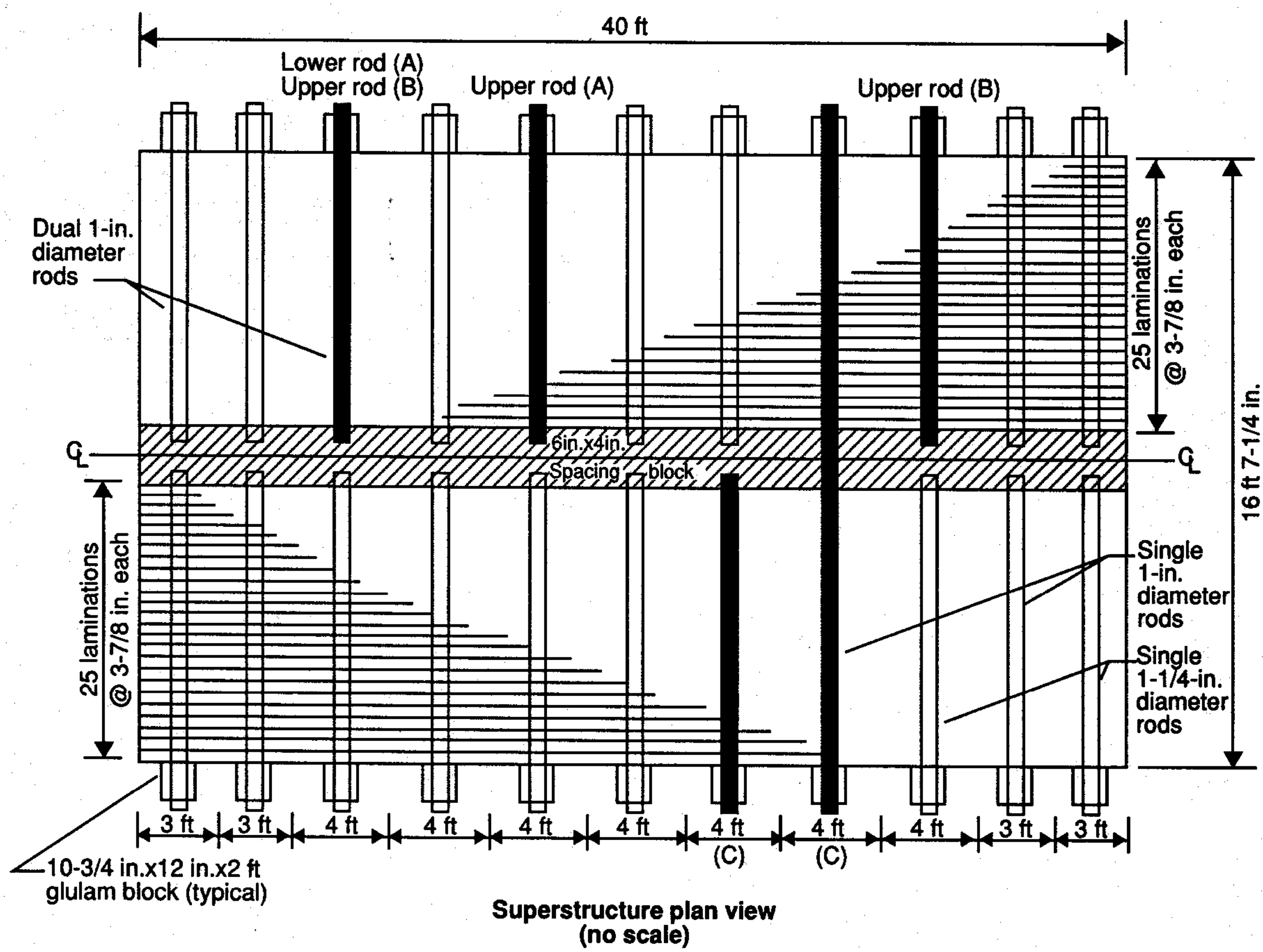

Figure 5—Plan view of bridge. Rods with load cells are darkened. Letters indicate monitoring dates: (A) Sept. 1987-Oct. 1989, (B) Oct. 1989-present, (C) Nov. 1987-present.

General Condition

The bridge is inspected visually to determine the condition of the major components and the wearing surface. Any signs of delamination, distress, checking, water penetration, or deterioration of the wearing surface are noted. These inspections are conducted at least annually.

Rod Forces

The forces in the high-strength rods are critical in the performance of all stress-lam bridges. However, the rods usually lose force over time as a result of drying of the wood, crushing, and creep. A minimum level of force must be maintained if the deck is to maintain its ability to distribute load transversely and behave as an orthotropic plate. Therefore, hollow-cylinder load cells, designed and constructed at the Forest Products Laboratory, were installed on a representative sampling of the rods to monitor rod forces. These load cells were read weekly for the first 2 months after the rods were stressed and monthly thereafter. On the Mormon Creek Bridge, two short 1-in. rods, one short 1-1/4-in. rod, and one long l-in. rod (total of four rods) were monitored (Fig. 5). The nominal design force is 60 kips for l-in. rods, and 120 kips for 1-1/4-in. rods. The deck design is baaed upon the assumption that over the long term, the rods will retain at least 40 percent of their original force.

Moisture Content

Changing moisture content can affect the forces in the rods. Moisture content was measured at six locations on the underside of the deck and six locations on the sides. Measurements were taken at a depth of 1 in. with an electrical resistance meter; monthly for the first year and quarterly thereafter.

Camber

Timber bridges can lose camber over time as a result of two phenomena: long-term creep of the wood members and movement of mechanical fasteners. Because the Mormon Creek Bridge was assembled using multiple mechanical fasteners to connect the webs and chords, and because these fasteners are subject to slip, the movement of fasteners is of special concern. To monitor this aspect of bridge performance, three stringlines and scales were installed under the centerline and edges of the bridge. Readings are taken from these lines and scales at the same time as are readings of rod forces.

Load Distribution

The bridge was test-loaded to determine stiffness and load distribution and to provide data for verifying a mathematical model of deck performance. The initial test was conducted 6 weeks after construction; the second test was conducted 1 year later.

Monitoring Results

General Condition

The most notable characteristic of the bridge was extreme crushing of the original two-piece, spiked, Douglas Fir stress blocks (Fig. 6). Some bearing plates were completely embedded in the blocks. Because of this crushing, minimum force levels could not be maintained in the rods. In July 1989, most of the original Douglas Fir blocks were removed and replaced with temporary softwood blocks, and the entire bridge was restressed.

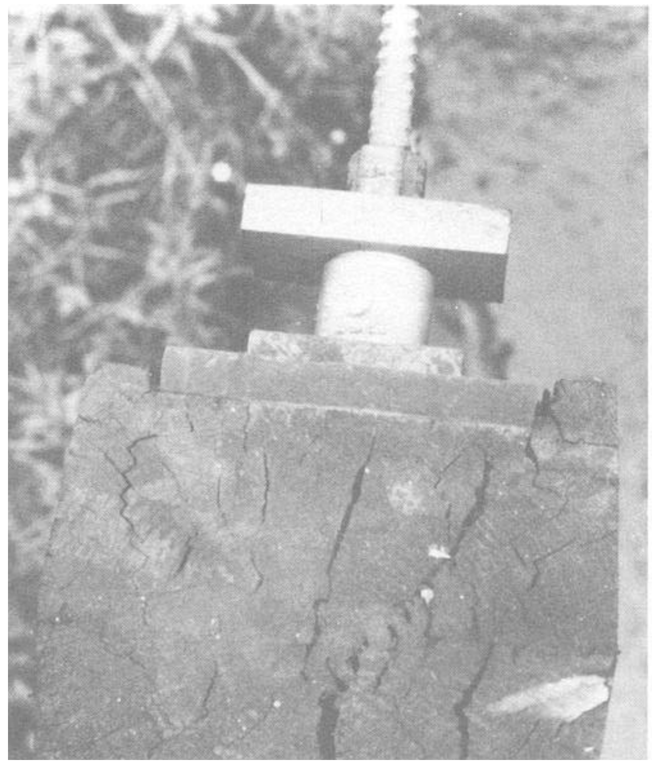

Figure 6—Crushing of Douglas Fir block. Force in these rods is being monitored by a load cell.

One-piece glulam Red Oak blocks were fabricated as permanent replacements and installed in October 1989. Again, the entire bridge was restressed. The Red Oak blocks are performing significantly better than the original Douglas Fir blocks. There is little evidence of crushing beneath the bearing plates. However, some local crushing has occurred beneath two of the blocks, in the sides of the trusses.

The rods were inspected, and full-length rods were repainted when the blocks were replaced. The rods showed no signs of rust; the only “blemishes” were caused by creosote dripping on the rods and blistering the rust-resistant paint in a few spots.

The wearing surface is holding up well. There are no signs of deterioration or excessive wear.

Rod Forces

During the first 2 years, through July 1989, three of the four rods being monitored fell below the target minimum of 40 percent of their initial force values, and the fourth rod was barely above that minimum (Fig. 7). Since installation of the Red Oak blocks and

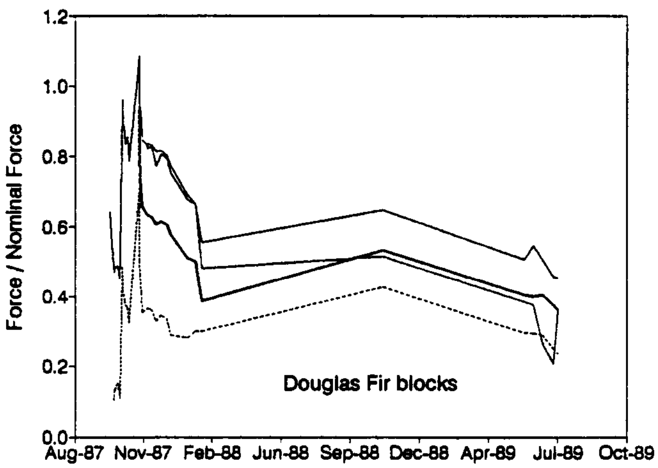

Figure 7—Rod forces with Douglas Fir blocks. Two 1-in. half-width rods (light solid lines), one 1-in. full-width rod (dashed line), and one 1-1/4-in. half-width rod (heavy solid line) were monitored.

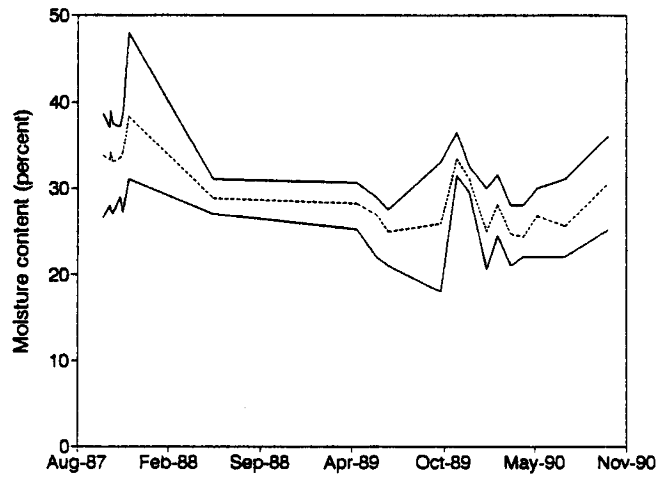

Figure 9—Moisture content of bridge. Solid lines indicate extremes; dashed line, average.

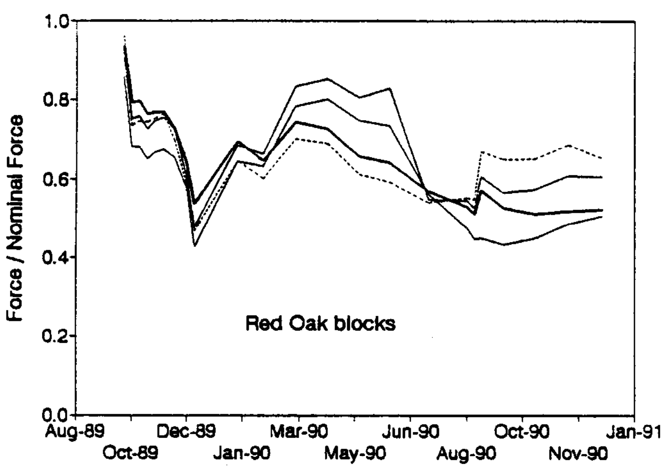

Figure 8—Rod forces with Red Oak blocks. See legend to Figure 7 for description of data.

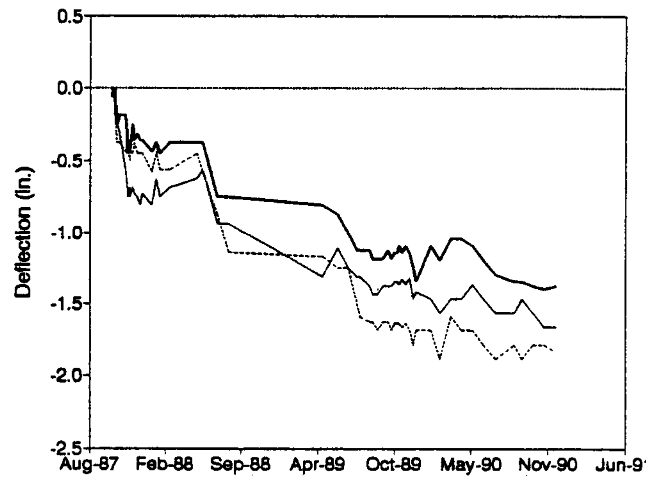

Figure 10—Loss of camber under bridge centerline (heavy solid line), downstream edge (light solid line), and upstream edge (dashed line).

restressing in October 1989, the force of the 1-in. rod being monitored has dropped from 60 kips to 28, 36, and 40 kips (47, 60, and 67 percent of nominal values). The force of the monitored 1-1/4-in. rod has dropped from 120 to 60 kips (50 percent) (Fig. 8).

Moisture Content

Overall, drying of the bridge has been minimal (Fig. 9). At bridge installation, the moisture content readings were between 27 and 39 percent, with an average of 34 percent; by the end of 1990, the range was 25 to 36 percent, with an average of 31 percent. However, this last reading reflects a winter rise in moisture content, which has occurred in three of the four winters since the bridge was constructed.

Camber

Most bridge deflection occurred in the first year after construction (Fig. 10). In September 1988, the loss of camber at midspan was 0.7 in. at the centerline and 1.0 and 1.1 in. at the edges. Additional deflection has occurred in the past 2 years, but at a slower rate. The stringlines registered 1.4, 1.5, and 1.8 in. at the end of 1990. The initial camber in the bridge was 2.3 in., so some positive camber still exists.

Load Distribution

Procedures

The bridge has been load-tested twice: in November 1987 (6 weeks after construction) and 1 year later. In both tests, a lo-wheel gravel truck was used as the load vehicle. The truck was weighed prior to testing; the

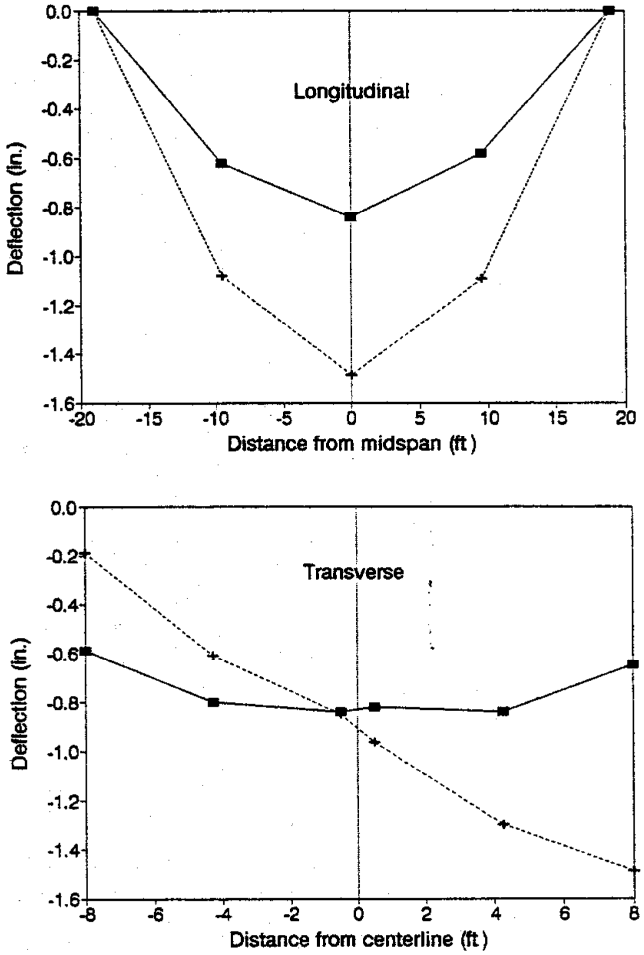

Figure 11—Longitudinal and transverse deflection profiles of 1987 load test. Longitudinal profile shows centerline deflection with truck centered (solid line) and edge deflection with truck at edge (dashed line). Transverse profile shows midspan deflection with truck centered (solid line) and at edge (dashed line). Gross weight of truck was 84,760 lb.

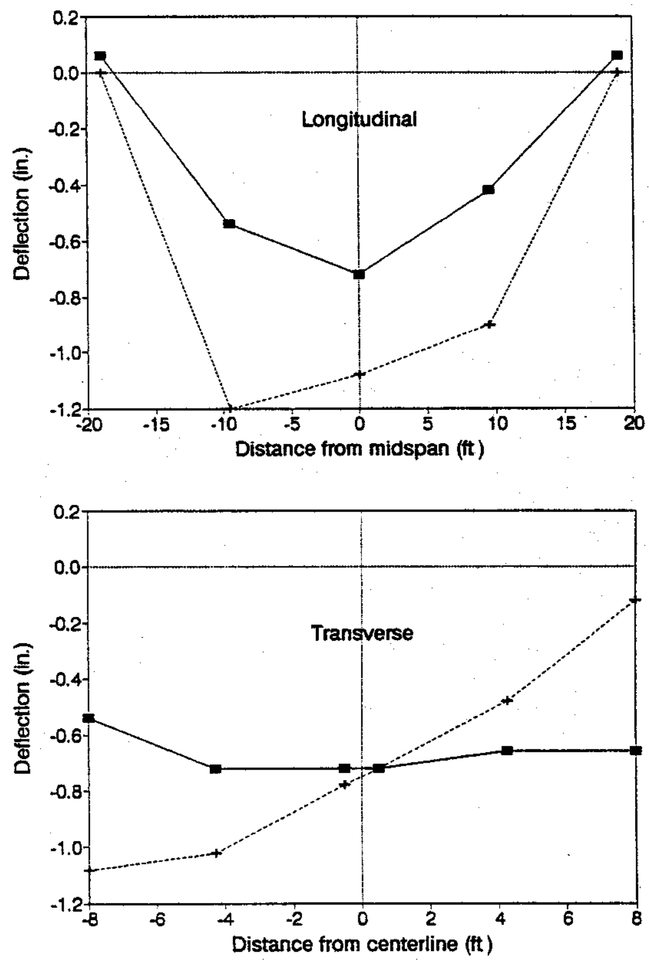

Figure 12—Longitudinal and transverse deflection profiles of 1988 load test. See legend to Figure 11 for description of data. Gross weight of truck was 82,240 lb.

front and rear (tandem) axles were weighed separately. Surveying equipment (level and rods) was used to measure deflections on the underside of the bridge deck. The truck was positioned with its centroid at midspan so as to produce maximum deflection, first with the truck along the centerline of the bridge and again with the truck positioned close to one curb. In 1987, the rear tires were against the downstream curb; in 1988, they were about 1 ft away from the upstream curb.

Results

In the 1987 load test, the gross vehicle weight was 84,760 lb, distributed to three axles. The deflection curves from that test are presented in Figure 11. The maximum deflection with the truck centered was 0.8 in. This compares to a theoretical deflection of 1.1 in. (Oliva and Dimakis 1988). With eccentric loading, the maximum deflection was 1.5 in. at the edge of the bridge.

The 1988 truck weighed 82,240 lb, only slightly less than the 1987 truck. The maximum deflection with the truck on the centerline was 0.7 in.; with eccentric loading, the maximum deflection was 1.2 in. (Fig. 12).

Discussion

General Condition

The bridge is performing well. Except for the severe crushing of the original blocks and the local crushing in two trusses, noted previously, there have been no signs of structural problems. The rods that were removed and inspected (one half-width rod and all five full-width rods) showed no signs of corrosion.

Rod Forces

The loss of stressing force was excessive with the original Douglas Fir glulam blocks. The blocks, which distribute the forces to the bridge trusses, were not adequate for their task. Crushing was severe directly under the bearing plates; blocks were sometimes crushed by more than 1 in. (Fig. 6). Since the Red Oak blocks have replaced the Douglas Fir blocks, the retention of force has been much better. All four monitored rods are still above their minimum target force values.

The allowable compressive stresses perpendicular to grain for dry and wet use are as follows (NFPA 1988):

	Stress (lb/in^2)	
Block type	Dry	Wet (2/3 dry)
Douglas Fir	625	420
Red Oak	820	550

The steel bearing plates that transmit the 120,000-lb forces are 23 by 8 in.; those that transmit 60,000 lb are 12 by 8 in. These dimensions result in the following stresses behind the plates:

	Stress (lb/in^2)	
Nominal force (kip)	Short term (100% force)	Long term (40% force)
120	652	261
60	625	250

Thus, the original compressive stresses in the Douglas Fir blocks were at the full allowable dry-use value and 50 percent above the wet-use value. Given that the moisture content of the bridge is well above 20 percent, the wet-use stresses are applicable. Thus, the poor performance of the Douglas Fir blocks is not surprising; the compressive stresses were not acceptable until the rod forces dropped to two-thirds of the nominal forces.

With the Red Oak blocks, the compressive stresses at full nominal rod forces were slightly above the allowable value for wet use, but were acceptable once the stress dropped to about 85 percent of the nominal force.

When the oak blocks were installed, moderate crushing occurred in two trusses, where the blocks bear upon the trusses. This can be attributed to the fact that the oak blocks are narrower than the Douglas Fir blocks (8-3/4 in. compared to 10-3/4 in.). This raised the compressive stress in the trusses from 465 lb/in^2 at full nominal load to 570 lb/in^2, about 35 percent above the wet-use allowable stress of 420 lb/in^2. As monitoring continues, these locations will be watched for additional crushing. However, the rod forces are now low enough that no additional crushing is expected.

Moisture Content

Loss of moisture has been very gradual in the bridge. There has been a steady downward trend, with some seasonal rises and falls. The large sizes of the members and the fact that only their edges are exposed to the air undoubtedly account for this slow drying.

The equilibrium moisture content for the location of Mormon Creek is about 18 percent (Ritter 1980). Therefore, we can expect considerable additional, albeit very slow, drying. This probably will cause more loss of stressing force in the future.

Camber

Most deflection occurred during the first year after construction. Since then, the deflection curves have become almost level. There is still some positive camber in the bridge. The magnitude of this long-term deflection is about the same as that of the elastic deflection under design loading.

Load Distribution

The load test results (Figs. 11, 12) demonstrate that the bridge deck is behaving as a plate. In particular, the transverse profiles illustrate lateral distribution of the wheel loads. The results of the load tests are summarized in Table 2. The 1987 deflections of 0.8 and 1.5 in. correspond to span/ deflection ratios of approximately 590 and 310, respectively. Considering the weight and configuration of the test vehicle, the deflections should be about 27 percent greater than would occur with a standard American Association of State Highway and Transportation Officials (AASHTO) HS-20 vehicle, which has a gross weight of 72,000 lb. This translates to HS-20 deflections of 0.63 and 1.18 in. and span/deflection ratios of 740 and 400, respectively.

The 1988 deflections of 0.7 and 1.2 in. give span/ deflection ratios of 670 and 390, respectively. The deflections in this case are about 23 percent greater than would be obtained with an HS-20 vehicle. The corresponding HS-20 deflections are 0.57 and 0.98 in. and span/deflection ratios are 820 and 480.

Table 2—Results of load tests

Test	Vehicle location	Test vehicle Deflection (in.)	Test vehicle Span/deflection ratio	HS-20 vehicle Deflection (in.)	HS-20 vehicle Span/deflection ratio
1987	Center	0.8	590	0.63	740
	Edge	1.5	310	1.18	400
1988	Center	0.7	670	0.57	820
	Edge	1.2	390	0.98	480

Current, design recommendations for timber bridges (Ritter 1980) target span/deflections of 360 to 500 under full design load. Thus, the Mormon Creek Bridge has adequate stiffness under live load.

The apparently large difference in the edge deflections between the two tests, despite nearly equal loads, and the opposite slopes of the transverse profiles are attributable to the fact that in 1987, the rear tires of the test vehicle were against the downstream curb; in 1988, the rear tires were about 1 ft away from the upstream curb.

Conclusions

The parallel-chord, stress-laminated Mormon Creek Bridge has been monitored for more than 3 years and load-tested twice. The original softwood blocks of the bridge did not have sufficient crushing strength to withstand the compressive stresses perpendicular to grain. New hardwood blocks are performing much better, although some slight crushing is now evident in the outer trusses of the bridge.

All other aspects of the bridge's performance have been satisfactory. The stressing rods are retaining force adequate to maintain plate action in the deck, as demonstrated by load tests. The load tests have also shown that the bridge is adequately stiff under design loads. The bridge has lost much of its camber but at a decreasing rate, and positive camber still exists. So far, there has been no sign of corrosion in the steel stressing rods or any other sign of physical deterioration.

The performance data and load test results indicate that the parallel-chord system is a viable variation of the stress-lam concept.

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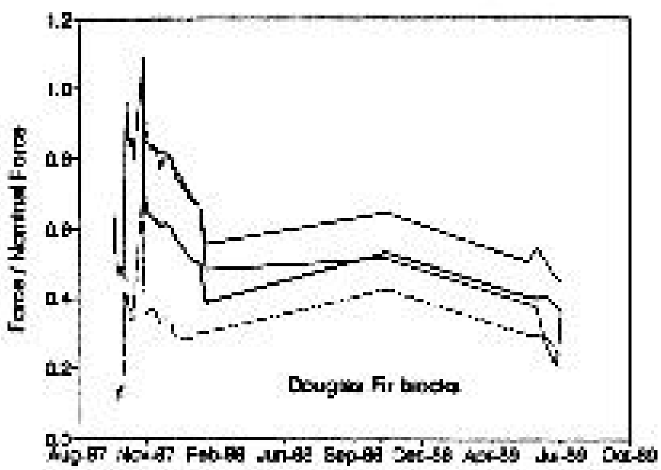
Force / Nominal Force
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1.0
0.8
0.6
0.4
0.2
0.0
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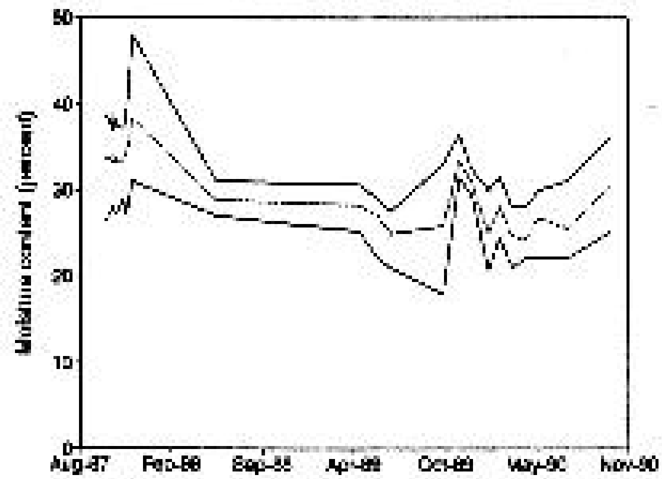
Nov-87
Feb-88
Jun-88
Sep-88
Dec-88
Apr-89
Jul-89
Oct-89


Moisture content (percent)
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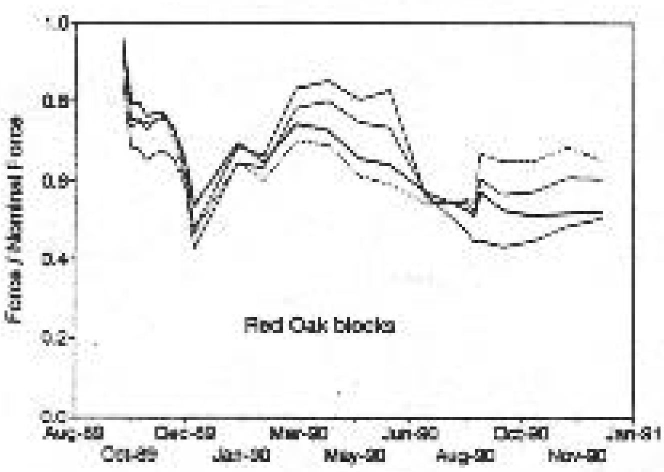
30
20
10
0
Aug-87
Feb-88
Sep-88
Apr-89
Oct-89
May-90
Nov-90


Force / Nominal Force
Red Oak blocks
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0.6
0.4
0.2
0.0
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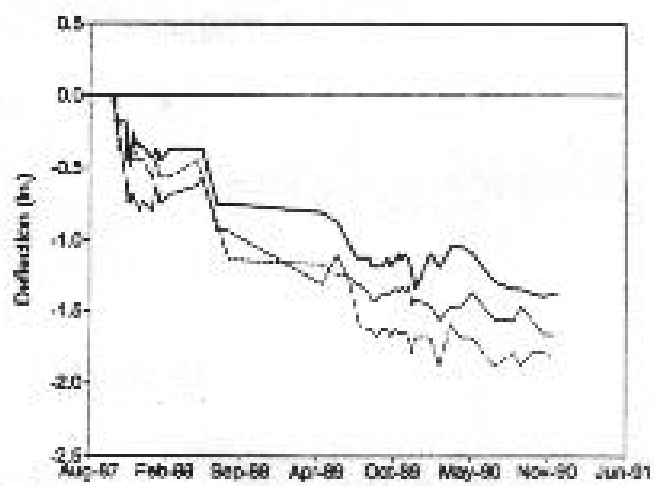
Oct-89
Dec-89
Jan-90
Mar-90
May-90
Jun-90
Aug-90
Oct-90
Nov-90
Jan-91


Deflection (in.)
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-1.0
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-2.5
Aug-87
Feb-88
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Oct-89
May-90
Nov-90
Jun-91