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Behavior of Stress-Laminated Parallel-Chord Timber Bridge Decks

Experimental and Analytical Studies

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Abstract

The use of stress lamination for constructing timber bridges may provide a solution to the urgent need for rehabilitating and replacing U.S. highway bridges. This report describes the development, construction, testing, and analysis of a new type of stress-laminated timber bridge: the parallel-chord bridge. A full-scale laboratory test was conducted on a stress-laminated parallel-chord bridge made with Vierendeel trusses. A similar set of shorter trusses was built for a prototype bridge on the Hiawatha National Forest in Michigan. Test results showed that both of these bridges have greater stiffness and can span longer distances than stress-laminated solid-sawn timber bridges. The stress-laminated parallel-chord bridge system effectively transfers applied loads to a wide portion of the deck trusses. Anchorage configurations have little effect on load resisting behavior as long as the anchorages transfer prestressing force into both chords and webs. Good correlation was found between analytical and experimental results. The stress-laminated parallel-chord bridge is easy to build, but the cost of the superstructure may be limiting.

Keywords: Timber, wood bridges, stress laminating, parallel-chord truss

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Contents

Page

Introduction 1

Background 1

Problem 1

Objective and Scope 2

Methodology 2

Member Feasibility 3

Individual Truss Analysis 3

Evaluation 4

Results and Discussion 12

Behavior of Individual Trusses 13

Simulated Truck Loading 13

Panel Joints 14

Prestress Distribution 15

Cyclic Loading 16

Analytical and Experimental Results 16

Conclusions and Recommendations 16

Resistance to Truck Loading 16

Ease of Construction 17

Prestress Rod Anchorage System 17

Repeated Load Resistance 17

Composite Action of Parallel-Chord Truss . . . 17

Analytical Modeling 17

Future Use of Parallel-Chord Stress-Laminated Bridges 18

References 18

Appendix 19

Behavior of Stress-Laminated Parallel-Chord Timber Bridge Decks

Experimental and Analytical Studies

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Introduction

The size of the highway system in the United States and the continued need for maintenance and repair have created tremendous pressure on local, state, and federal governments to seek new, efficient, and economic means of insuring safety. Bridges, critical links in the transportation system, have begun to receive long-overdue attention. The USDA Forest Service, which is directly responsible for thousands of miles in the highway system, has recently taken a lead role in assessing new technological means of addressing highway bridge deficiencies through innovative uses of timber in bridge construction. As part of this role, direct funding has been allocated toward basic and applied timber bridge research through the Forest Products Laboratory. Efforts are also underway to bring the necessary expertise to design engineers. This report describes the development of a new type of timber bridge, which may prove to be an efficient and economical system for replacing many bridges that are beyond repair.

Background

Stress laminating, a new technique for efficiently and rapidly building new bridges or replacement decks, was introduced in Ontario, Canada, in 1976. In this bridge system, a solid timber plate, which acts as a deck bridge, is constructed by vertically laminating individual pieces of lumber using compressive prestress.

The prestress causes compressive stress between the faces of the individual laminae. This compression creates friction, which allows shear forces to be transmitted between the laminae. In addition, when transverse bending occurs, the laminae are held together by the compressive force, which is designed to be high enough to counteract the flexural tension stress likely to be induced by applied bending loads. A bridge deck constructed of stress-laminated solid timber is shown in Figure 1. The entire set of laminae is squeezed together tightly so that the laminae act as a single, solid unit.

Problem

Although stress-laminated solid-lumber decks appear to be a very attractive solution for certain bridge applications, the stiffness of a solid deck is limited. The span of a stress-laminated solid-timber bridge deck should be limited to <40 ft for HS 20-44 truck loading if 16 in. laminae are used and deflections are to be controlled at a reasonable level (Oliva and Dimakis 1988). (See Table 1 for conversion factors for SI units of measurement.) Elimination of cracking or deterioration in an asphalt wearing surface may require a shorter span.

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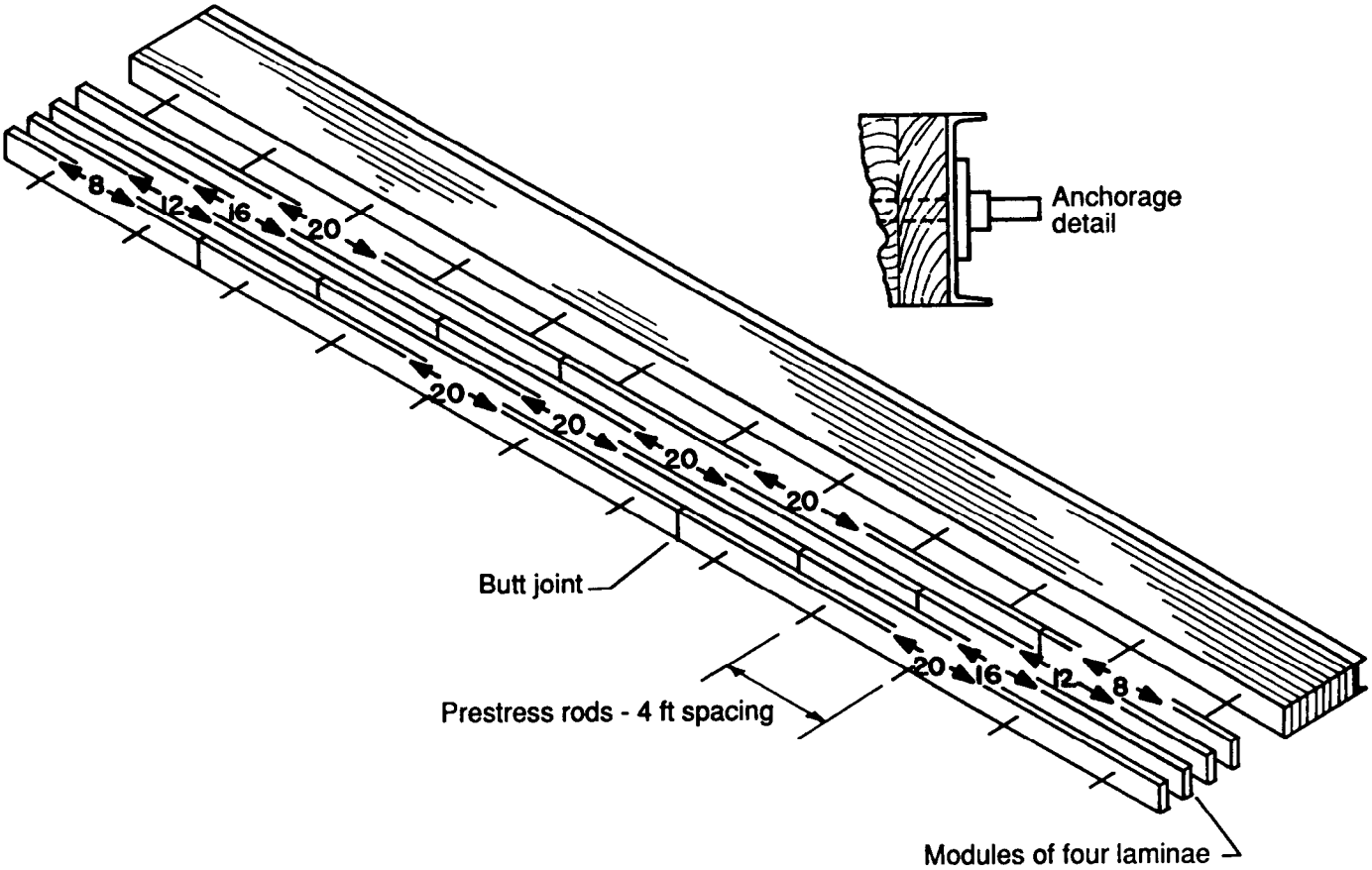

Figure 1—Bridge deck constructed of stress-laminated solid timber. Dimensions are in feet.

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)
board foot (fbm)	0.0024	cubic meters (m^3)
lb/in^2 (stress)	6.89	kilopascal (kPa)

The stress-laminating technique could be used for longer spans if a method could be developed for increasing the deck depth, thereby providing the required stiffness. Attaining a greater depth while maintaining the stress at efficient levels can only be accomplished by changing the shape of the laminae. Thus, the problem in developing longer span stress-laminated bridges is in identifying or developing fabricated laminae with a deeper cross-sectional configuration, which provides improved efficiency in both strength and stiffness.

Objective and Scope

The objective of this study was to develop and evaluate bridge systems consisting of stress-laminated parallel-chord trusses. The scope of the work included an evaluation of previous work on individual members, development of individual trusses, and construction and evaluation of a full-scale laboratory bridge and a prototype field structure.

Methodology

To meet the study objectives, we conducted an analytical investigation of the behavior of individual fabricated members with various cross-sectional configurationss, followed by experimental evaluation to determine whether bridges could be assembled by stress laminating the members together. The feasibility of various potential member sections and preliminary evaluation of individual components were based on previous work. Within this study, methodology involved a preliminary analytical analysis of bridge systems composed of stress-laminated members followed by the experimental and analytical evaluation of two full-scale bridge structures.

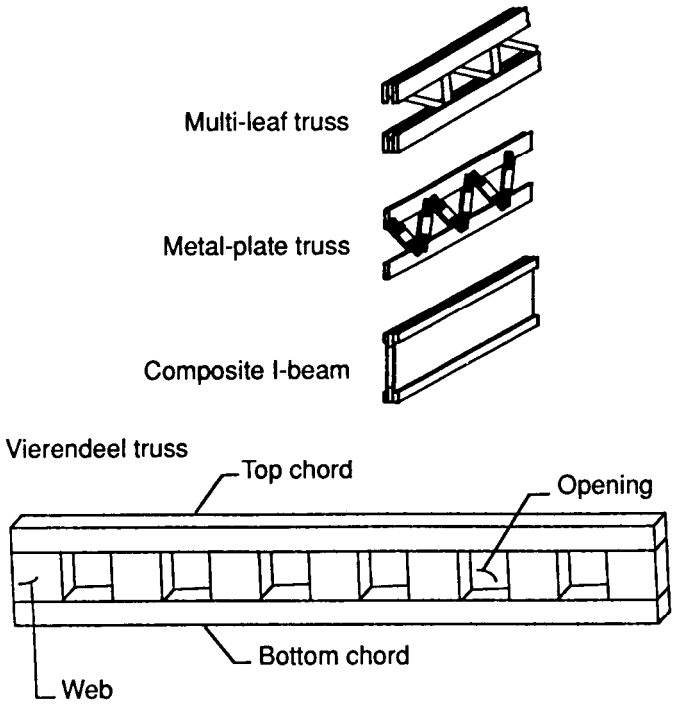

Figure 2—Possible configurations for parallel-chord members initially identified as likely components for a stress-laminated parallel-chord bridge deck.

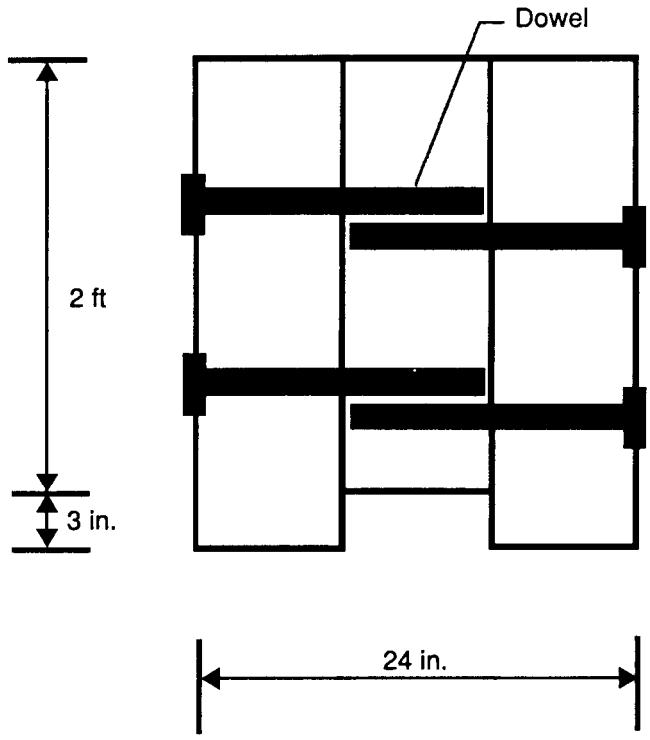

Figure 3—Chord-to-web joint test specimen. Shear stiffness of connection was determined by applying downward load on center wood block.

Member Feasibility

Four types of member configurations were initially identified by Oliva, Tuomi, and Dimakis (1986) as potentially feasible for stress-laminated bridge applications (Fig. 2). Each member was subjected to computational analysis to determine necessary element sizes and connection strengths between elements. On the basis of those initial studies, the composite I-beam members and the multi-leaf truss were discarded. The composite I-beam exhibited a weakness within its plywood webs. Considerable horizontal shear occurs in the webs of laminae from bridge truck loadings, and several layers of plywood are apparently necessary to meet strength requirements. The multi-leaf truss section could be provided with sufficient individual element strength. However, economic fabrication of multi-leaf trusses is limited by the connections between web and chord members and their strength requirements. The Vierendeel-type truss and the metal-plate truss appeared to be feasible and were examined in further detail.

Individual Truss Analysis

Oliva and Lyang (1987) examined Vierendeel parallel-chord trusses in detail. This study entailed testing of the connections and trusses, and analytic simulation of their behavior. Shear tests on individual specimens simulating chord-to-web joints were used to determine truss shear stiffness. Variables considered in the study included the size of individual members in the joint, and the size, number, and length of connecting fasteners (steel dowels). A typical test specimen is shown in Figure 3. Each specimen was tested to failure (that is, large slip with no increase in load) by applying a downward load on the center wood block while the outer pieces were supported.

In addition to shear tests, three types of 25-ft-long Vierendeel trusses (Fig. 4) were tested to determine individual stiffness and to allow correlation between analytically predicted behavior and measured response. Two trusses were 24 in. deep with 4- by 8-in. chords and webs. The third truss was 30 in. deep with 4- by 10-in. chords and webs. Each web block was attached to the chords by two steel dowels. The trusses were initially built without butt joints in the chords. Midway in the testing program, the chords were sawn through to simulate butt joints.

In the study by Oliva and Lyang (1987), Vierendeel trusses were tested individually and in groups to identify overall section stiffness. The known web shear stiffness for the Vierendeel truss was then used in an analytical model to predict the behavior of a complete

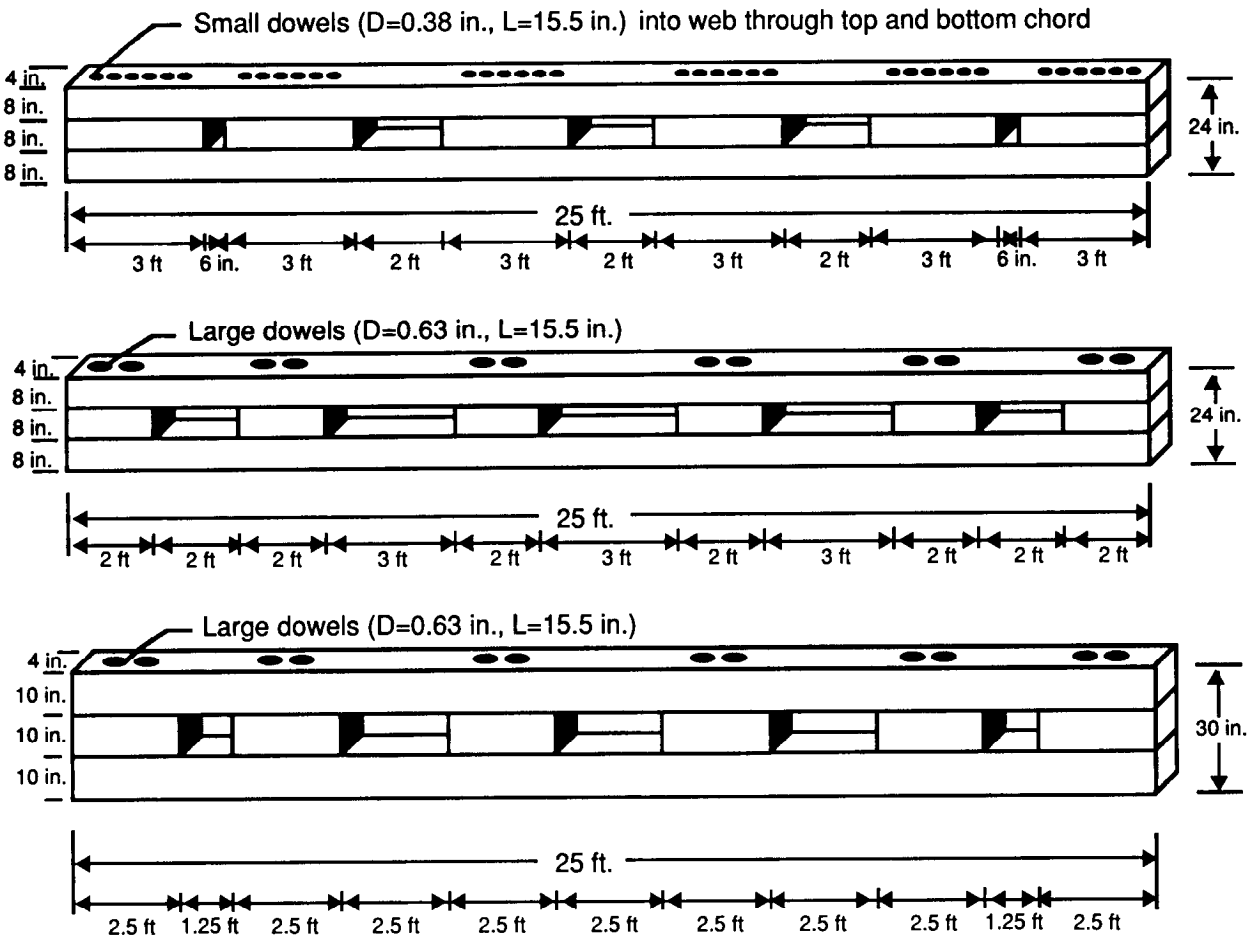

Figure 4—Configurations of three types of Vierendeel trusses subjected to individual loading.

truss. This information was compared with test results to verify the analytical modeling techniques and to determine whether the behavior of the truss could be predicted. Verified techniques were used to analyze a series of trusses with different configurations to identify a truss design with sufficient stiffness and strength for bridge applications. Because of the superior performance and simplicity of fabrication, the Vierendeel truss was selected for the full bridge and became the focus of this study.

The analytical model developed by Oliva and Lyang was used to develop Vierendeel truss configurations for evaluating stress-laminated bridges. Initially, the model was used to simulate a truss with continuous top and bottom chords and semirigid web-chord connections. The shear stiffness obtained from previous tests was used at the chord-to-web joints. The load deformation response of a 52-ft Vierendeel truss was then predicted using the verified analytical model. Because discontinuous top and bottom chords would be required for bridge evaluations, the model was modified to account for three types of butt joints. Analyses were conducted for 51-, 41.5-, and 32-ft spans, measured center-to-center of supports.

Evaluation

Based on the analysis of individual trusses and the analytic evaluation of a stress-laminated truss bridge, two parallel-chord stress-laminated bridges were designed and built for this study. The first bridge was assembled in the laboratory and used to obtain performance data, which were correlated with the analytically predicted bridge response. Based on the knowledge gained from the analytical studies and laboratory testing, a second experimental bridge was designed and constructed over Mormon Creek on the Hiawatha National Forest. The laboratory bridge and the Mormon Creek Bridge are shown schematically in Figure 5.

Laboratory Bridge Tests

The laboratory tests were designed to meet the following objectives: (1) to test the capability of the parallel-chord system to resist truck loading, (2) to determine whether the type of prestress rod anchorage affects resistance to truck loads, (3) to test the stiffness across the proposed joint between prefabricated panels, (4) to appraise the stiffness changes and accumulated deflection after one-half million cycles of simulated truck loading, (5) to determine how the force from a

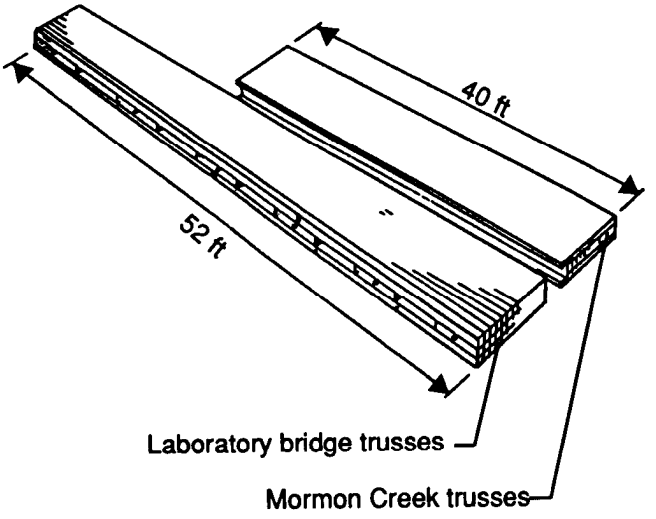

Figure 5—Schematic view of the laboratory and Mormon Creek parallel-chord bridge decks.

prestress rod anchorage flows into the wood and how the stress varies within the wood, and (6) to provide sufficient data on the response of the system to validate an analytical model for predicting response.

Bridge Configuration —A laboratory test bridge was constructed of stress-laminated trusses. The 52-ft-long by 9-ft-wide bridge was made with the following configuration:

- 27 Vierendeel-type trusses, 3.88 in. wide by 24 in. deep with rough-sawn 4- by 6-in. chords and 4- by 12-in. webs planed to 3.88 in. Material was visually graded No. 1 or better Douglas Fir, pressure treated with creosote.
- Butt joints on top and bottom chords, only at web-to-chord connections. Joints were placed in a precise sequence:

  (1) Only one in four trusses was allowed to have a butt joint, at top or bottom chord, within any 8-ft length parallel to the bridge span.

  (2) Butt joints in adjacent trusses were separated by at least 8 ft in the direction of the span.
- Chord member connected to web block by three 0.63-in.-diameter steel dowels.
- Camber of approximately 2 in. provided in each truss to offset dead load deflection.

The individual parallel-chord trusses were very flexible as a result of the butt joints in the chords. Temporary supports had to be provided at midspan locations to preserve the camber in the trusses until it was locked in by the stress-laminating process. After stressing, the intermediate support was removed. The laboratory bridge system is illustrated in Figures 6 to 8.

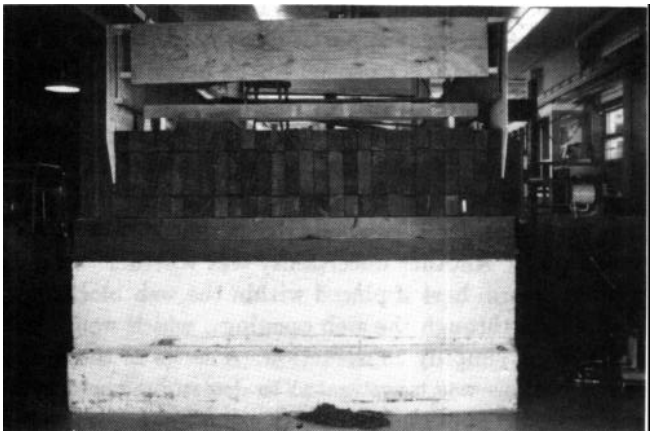

Figure 6—End view of laboratory parallel-chord bridge deck. Deck rests on 12- by 12-in. timber on concrete abutment. Instrumentation reference frame is mounted above deck.

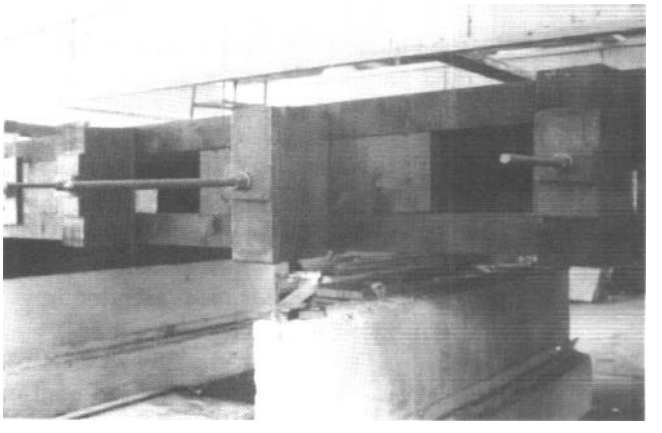

Figure 7—Side view of laboratory bridge deck. Posts and steel plates provide anchorage for transverse prestressing rods.

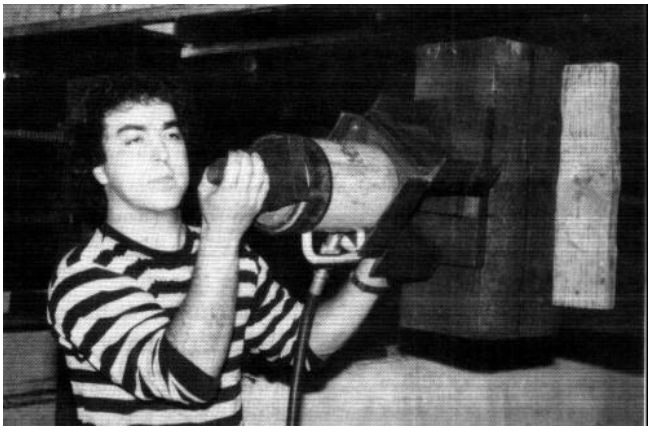

Figure 8—Hydraulic jack used to apply tension force on prestress rod of laboratory parallel-chord bridge.

Rod Anchorage Configurations — Adequate transfer of transverse prestressing from the tensioning rods into the wood of the trusses, particularly those trusses adjacent to the edge, is essential to the performance of a bridge system. Because stress laminating had not been used previously on parallel-chord members, we did not know if prestressing rods were needed in both top and bottom chords, as would be required if the system were considered two solid decks spaced a short distance apart. Another uncertainty was whether the rods would perform best if placed within the web blocks or if threaded through the web openings, which would facilitate and simplify construction. A series of these different options was investigated to determine the actual prestress distribution from the tensioning rods as well as the capability of the anchorage system to effectively laminate the individual trusses. When the rods were placed within the web openings, a separate bulkhead member was required at the rod anchorage to transfer the compression equally into both top and bottom chords.

Coincidentally, the bulkhead member appeared to be able to serve a dual purpose. If a vertical post were used as the bulkhead, it could be extended upward above the deck and possibly serve as a rail post (pending further studies to determine vehicle impact effects on system behavior). With this dual use in mind, the anchorages were designed to be composed of a set of steel plates bearing on the post, which transferred the force to the top and bottom chords, its illustrated in Figure 9. The specification for the posts required a 24FV4, 8.75- by 12-in. Douglas Fir glulam member; the actual posts delivered to the laboratory and used in the tests were solid-sawn 8- by 10-in. members, which performed poorly, as described later in this report.

The rods and anchorages used in the laboratory bridge were constructed in four configurations:

> *Configuration 1:* One 1-in.-diameter rod in each web opening and two 1-in.-diameter rods in each web block. Because the goal was to obtain a uniform compressive stress between every portion of the trusses, more force (and hence more rods) was required in the portions of the trusses where the web blocks were placed. The rod in the web opening was assumed to cause prestress in the chords in its vicinity. The rods in the web block were assumed to prestress the blocks together as well as the portions of the chords in the vicinity of the rods. The specific placement of the rods and the size of steel anchorage plates for this configuration are shown in Figure 10.
>
> *Configuration 2*: Similar to configuration 1, but effect of joint between panels simulated by adding 6- by 4-in. members on one side of deck (Fig. 11).
>
> *Configuration 3*: Similar to configuration 2 in respect to rods within web openings. Two 1-in.-diameter rods in each web block replaced by single 1.25 in.-diameter rod (Fig. 12).
>
> *Configuration 4*: Four 1-in.-diameter rods inserted only through web openings (Fig. 13).

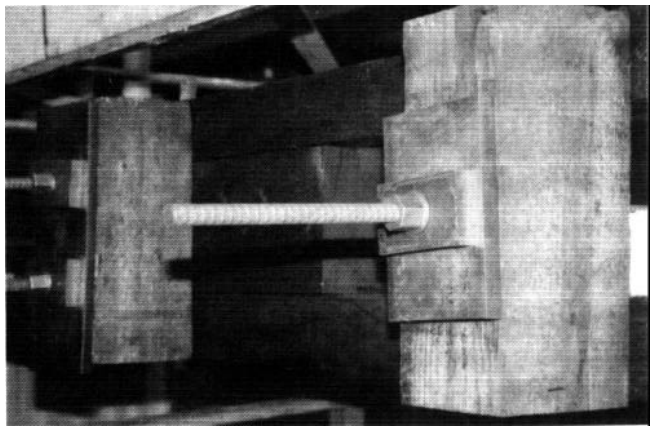

Figure 9—Posts used to distribute prestressing force into top and bottom chords.

In configurations 1, 2, and 3, insertion of the rods through holes in the web blocks required tight tolerances and drilling of adequately sized holes to make assembly possible. This could be a difficult or impossible task, particularly with bridges consisting of more than one traffic lane. Configuration 4 is practical for bridge construction under such conditions. However, trusses without butt joints would probably be required unless temporary midspan support could be provided to maintain camber until prestress was applied.

Simulated Truck Loading—The stressed bridge was loaded with simulated double-wheel truck loading applied at midspan for each anchorage configuration at internal prestress levels of 10, 20, 40, 50, and 70 lb/in^2. This "point" load was actually applied over an area of 20 in. transversely and 15 in. longitudinally to simulate the tire contact area of a truck. The standard test consisted of a point load at the center of the span and the

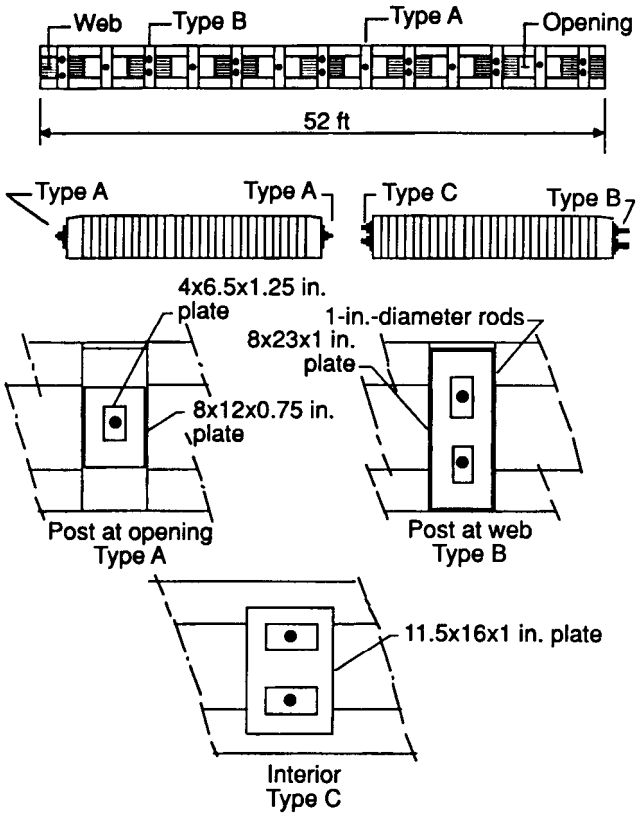

Figure 10—Anchorage configuration 1. Two rods were placed through each web block and one rod through each web opening. Three types of rod anchorages were used, as shown.

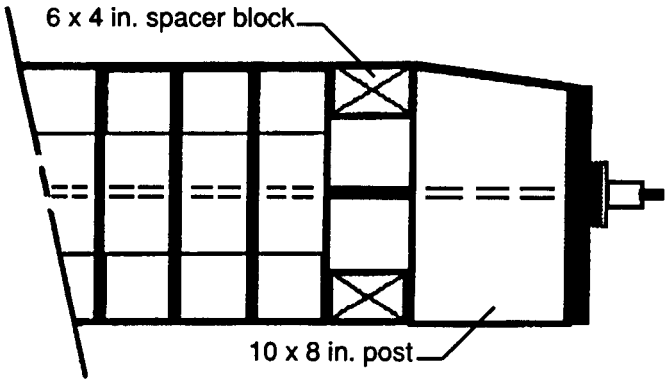

Figure 11—Anchorage configuration 2—detail of one side. Spacer blocks were used to simulate transfer of stress into deck at joint between two panels.

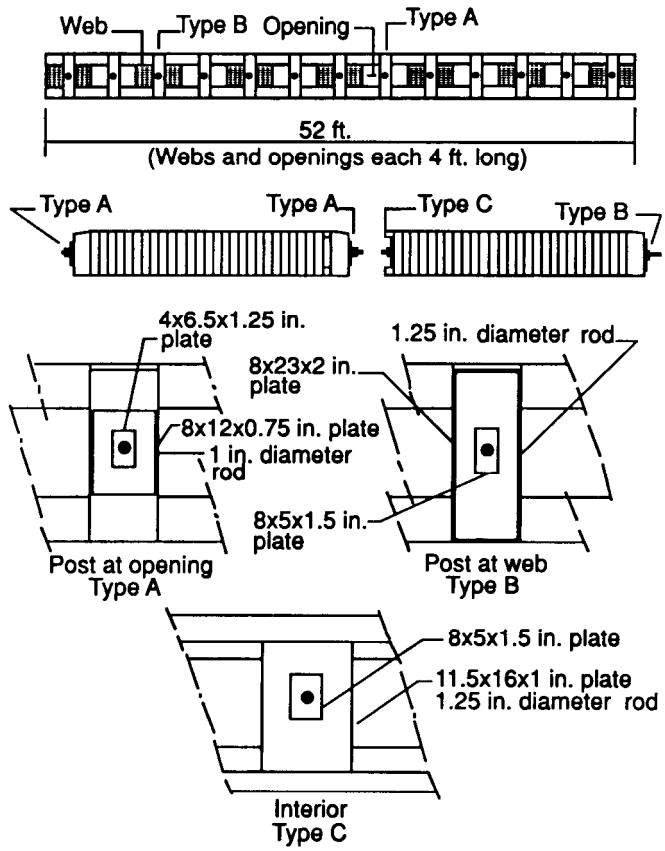

Figure 12—Anchorage configuration 3. One rod was placed through each web block and one rod through each web opening.

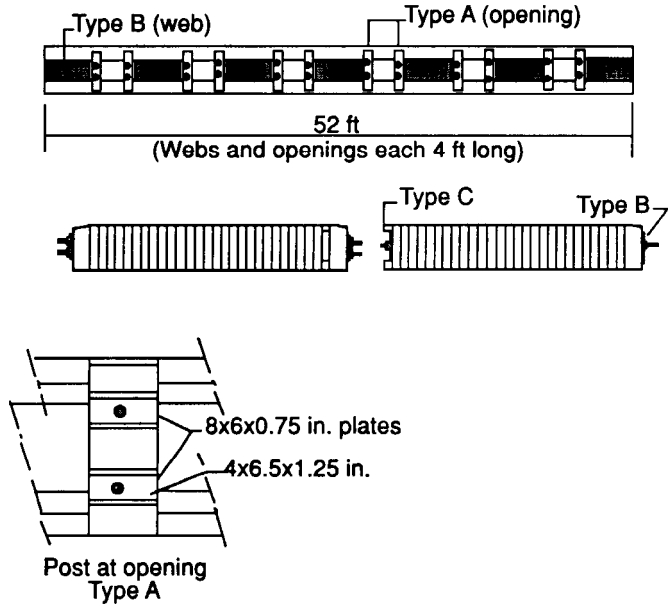

Figure 13—Anchorage configuration 4. Four rods were placed in each web opening. Two posts were used with two rods through each post with anchorages as shown. The anchorage on one side used spacer blocks, as shown in Figure 11.

center of the lane. Other tests included (1) two point-loads spaced in the transverse direction and applied 1.5 ft from each edge of the deck and (2) two point-loads spaced in the longitudinal direction and applied 4 ft to either side of the midspan (directly above web openings). In each case, the loads were applied until the maximum deflection reached approximately 1 in. These load tests were conducted with clear spans of 50, 39.5, and 31 ft by changing the location of the bridge supports. The 39.5-ft clear span was tested under longitudinally asymmetric loading. Table 2 lists the 50 types of simulated truck load tests used to determine bridge capacity and effects of various prestress levels and anchorage types.

Panel Joints-Continuity across the joint in a multi-panel prefabricated bridge is essential to transverse load distribution and wearing surface longevity. In most construction situations, field labor time and cost can be reduced by assembling trusses into panels and prestressing together before they are shipped to the erection site. This approach requires a joint between panels, which may create a problem that stress laminating specifically tries to circumvent: if the panels are not prestressed together, a weak joint remains in the bridge system where differential movement may occur under truck loading.

Previous work on solid stressed decks showed that adequate friction is provided between the laminae to avoid slip even under very low levels of prestress (Oliva and others 1990). A 16-in.-deep solid stressed deck, with a coefficient of friction of 0.35, requires a theoretical level of prestress of 10.4 lb/in^2 to prevent interlaminar slip. Because the effective area for shear transfer across a joint in a parallel-chord truss is potentially less than that provided by a 16-in.-deep stressed deck, a level of prestress >10.4 lb/in^2 would be necessary to prevent interlaminar slip between the panels, assuming a similar coefficient of friction.

To investigate the integrity of the joint subjected to vehicular truck loading, two panels were constructed of 12 and 13 trusses, respectively. A continuous 6-in. by 4-in. spacer block was placed between the panels, attached to the top and bottom chords of an interior truss at the joint, to secure interior anchorage of the individual stressed panels (Fig. 14). The two panels were connected by pressure provided by only the 1-in.-diameter rods. The rods were inserted through the web openings and stressed to the required levels of prestress, resulting in the "design level" prestress between the trusses in each separate panel, but only about one-third the design prestress between the trusses at the joint.

The behavior of the joint was investigated under three levels of prestress in the deck—20 lb/in^2 (less than

Table 2—Truck load tests on laboratory bridge[a]

Anchorage configuration[b]	Load location	Clear span (ft)	Prestress level (lb/in^2)[b]
1	Centerpoint load	50	10
			20
			40
			50
			70
1	Two transverse loads	50	10
			20
			40
			50
			70
2	Centerpoint load	50	50
			70
2	Two transverse loads	50	10
			20
			50
			70
2	Two longitudinal loads	50	10
			20
			50
			70
3	Centerpoint load	50, 39.5, 31	10
			20
			50
3	Two transverse loads	50, 39.5, 31	10
			20
			50
4	Centerpoint load	50, 39.5, 31	50
4	Two transverse loads	50, 39.5, 31	20
			50
5	Point load near center	50	20
			50
			70

[a] Loads were applied to a magnitude required to produce approximately 1 in. of deflection.
[b] Configuration
1—one rod in web opening, two rods in web block
2—same as configuration 1 with special edge bearing
3—one rod in web block
4—all rods in web opening
5—panels jointed

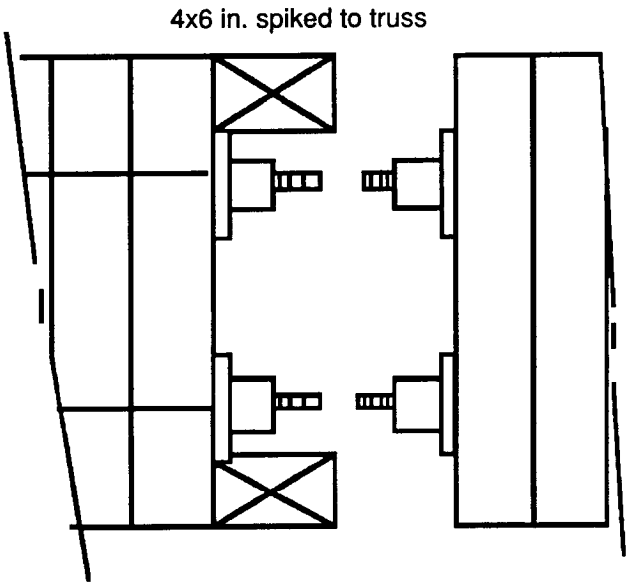

Figure 14—Detail of joint between panels. The two sets of stressing rods, which would appear to strike one another when the panels are pulled together, are actually offset from the drawing. Spacer blocks provided clearance between panels for anchorage plates.

normally acceptable in design), 50 lb/in^2, and 70 lb/in^2, resulting in net pressure across the joint of 6, 14, and 20 lb/in^2, respectively. Centerpoint loads were applied to the side of the joint until a center displacement of approximately 1 in. was achieved, causing maximum shear across the joint.

Prestress Distribution—Transverse compressive deformation, induced from a tensioning rod, is an indicator of the magnitude of compressive stress at any location in the deck. The anchorage system used to transfer the prestressing force into the bridge dictates how concentrated the compressive stresses are near the edge of the deck. With small concentrated anchorage areas, it is impossible to have uniform compression near the anchorage. Uniform compression, or at least a minimum compression above the amount associated with a lower deck stiffness, is desirable within the area of the deck where vehicular wheel loading is possible. The actual compressive stress in a parallel-chord bridge is expected to vary not only with distance from the anchorage in plan but also in the vertical direction.

Compressive deformations in the transverse direction were measured within a region of the deck adjacent to a single anchorage point as the prestress in the rod or rods at the anchorage was varied. The amount of transverse compressive displacement in the wood at any measured location was taken to be proportional to the compressive prestress level at that location. Three transverse stress distribution tests were conducted. Anchorage configuration 1 was used in two of these tests and configuration 3 in the third test (Table 3).

Table 3—Transverse stress distribution tests

Test	Anchorage configuration	Post location	Figure
1	1	Opening	10
2	1	Web	10
3	3	Web	12

Three rows of linear variable differential transformers (LVDTs) were installed on the deck surface to measure the variation in transverse compressive deformation as the distance from the anchorage point increased. The recorded displacements were normalized to units of average strain and then plotted. Generally, three increments of force were applied to a single prestressing rod, and the associated compressive deformations in the timber were recorded. These tests allowed us to determine the effective level of compressive prestress within the deck and to estimate the average distance from the anchorage point at which the compressive stress became approximately uniform.

Cyclic Loading—Cyclic loading was imposed on the jointed deck to determine whether cumulative slip would occur at the joint and whether repeated load would reduce elastic stiffness. The deck was tested at the lowest level of average prestress expected under service conditions, 50 lb/in^2 within the deck panels and 14 lb/in^2 between the two panels. The load was applied at midspan, adjacent to the construction joint. The loads were applied in a cyclic sinusoidal variation.

The first 100,000 cycles of load were applied at a rate of two cycles per second with a cyclic displacement of 0.1 in. about a mean of 0.5 in. An additional 400,000 cycles of load were then applied with a mean level of displacement at 1.1 in. and peak amplitudes of 0.6 and 1.6 in., and the rate of loading was decreased to 1 cycle per second. The peak displacement amplitude of 1.6 in. was calculated to be consistent with the expected deflection caused by an HS 20-44 design truck on the 50-ft span.

Before the cyclic loading test began, a static stiffness test was conducted to determine the initial stiffness of the system. Static tests were also conducted after 100,000, 400,000, and 500,000 cycles to determine reductions in structural stiffness.

Data Measurement-Data measuring devices used in this study included load cells (with strain gauges in a full-scale bridge arrangement), and LVDTs and dial gauges to measure displacements. Load cells were used to measure the variation in load in a prestressing rod. Twenty full-scale bridge load cells were fabricated and

calibrated to a maximum load of 60,000 lb. In this load range, the relationship of load to strain was linear. Transverse compression deformations in the wood, caused by the force from a prestressing rod, were measured by LVDTs. The LVDTs were installed in a special arrangement to study the variation of compression deformations in the transverse direction.

Taking advantage of symmetry, deflections were measured over only one-quarter of the deck. For the truck load tests using anchorage configuration 1, deflections were measured at six "stations" between the support and the center of the bridge in the longitudinal direction. For all other anchorage configurations, displacements were only measured at station 1, located near the center of the span. At each station, seven deflections were measured in the transverse direction between the centerline of the bridge and the deck edge. This information was used to verify the analytical model and to compare the elastic stiffness of the deck with different anchorage configurations, at comparable levels of prestress. Deflections were measured with LVDTs, with a minimum linear range of ± 1 in. All LVDTs were calibrated before testing. The LVDT linear range was well beyond the 1-in. limit.

A computer-aided data-acquisition system was used to record the output from the differential transformers. The output from the load cells (load in the tensioning rods) was read by a strain indicator and two lo-channel switch- and balance-devices. Certain residual deformations occurring as a result of the dynamic truck loading tests were monitored by displacement dial gauges, which were read manually.

Analytical Modeling—The development of a closed form mathematical model to predict the behavior of open web systems is a difficult task. Orthotropic plate theory was used in the past to analyze stiffened plates of constant thickness (Huffington 1956). Bridges constructed with longitudinal open sections were investigated successfully using orthotropic and articulated plate theories (Bakht and Jaeger, 1985), but the literature does not provide any information on methods of analysis of open web systems with web openings located transversely.

The analysis of a system idealized as an orthotropic plate requires the determination of the effective plate elastic rigidity constants D_x, D_x, and D_{xy}, and a suitable solution to the orthotropic plate equation satisfying the existing boundary conditions. This latter problem has been addressed in the literature for various sets of boundary conditions, and solutions exist for classical cases.

The flexural and twisting rigidity constants D_x, D_y, and D_{xy} may be conceived of as applying to a homogeneous orthotropic plate of constant thickness, which is equivalent to the actual parallel-chord open-web system. The term equivalent requires careful definition because the orthotropic plate obviously cannot be equivalent to the actual system in every respect. The deflection of the actual bridge and equivalent orthotropic plate might need to be approximately the same when both have the same loading and boundary conditions.

The flexural and twisting rigidity constants of an orthotropic plate do not depend on the boundary conditions of the plate nor on the distribution of the transverse load. This principle, coupled with an equivalence criterion, provides a basis for determining the effective rigidity constants. If the deflection of both the actual bridge and equivalent orthotropic plate can be found for some applied loading and a particular set of boundary conditions, the application of the equivalence criterion will provide an equation in which the only unknown quantities are the desired rigidity constants.

Care must be exercised in using the results from the analysis of such an "equivalent" plate. For a given bridge, the transverse variation of all response quantities, that is, deflection, moments, and shears, is not the same. The variation pattern becomes more concentrated (higher intensity at the position of the load) as the order of the deflection derivative increases. Thus, a distribution width derived from a transverse deflection distribution pattern is larger than that obtained from a transverse moment distribution pattern. Distribution width design charts are usually based upon transverse profiles of longitudinal moments.

A common commercial finite element analysis program was used to analyze an equivalent plate, which was intended to have the same response as the actual stress-laminated parallel-chord bridge deck. The required rigidities for the orthotropic plate analysis were obtained by first studying the stiffness of an individual truss. An equivalent orthotropic plate thickness was calculated from the average truss stiffness by setting the modulus of elasticity equal to the tabulated value for visually graded No. 1 Douglas Fir (1.8×10^6 lb/in^2) and equating the longitudinal rigidity of the plate with the stiffness of the truss. The transverse plate rigidity and shear rigidity were estimated from relations determined previously for solid-sawn stress-laminated decks (Oliva and others 1990). The rigidity relations were taken as

$$E_t = 149\,f + 15{,}360$$

$$G = 134\,f + 16{,}600$$

Table 4—Properties of equivalent orthotropic plate used in correlation studies

Deck clear span (ft)	Truss[a] ($\times 10^3$ lb/in)	D_x ($\times 10^3$ 1b–in^2/in)	Equivalent thickness (in.)
31	1.97	580,977	15.7
39.5	1.24	715,838	16.8
50	0.72	861,966	17.9

[a] Mean values.

where

f is average prestress in the plate (lb/in^2),

E_t transverse modulus (lb/in^2), and

G shear modulus (lb/in^2).

These properties were used as initial assumptions because there was no basis to expect that the rigidities in the laminated truss system would have similar relations to those in the solid wood deck. The properties of the equivalent orthotropic plates used in the correlation studies are listed in Table 4.

The general analytic ability of the equivalent plate model was verified by comparison with measured results of simulated truck loading in the laboratory tests over various spans and with various load locations. The finite element model of the equivalent solid orthotropic plate, which was used to predict the deformation response of the actual parallel-chord system, is displayed in Figure 15.

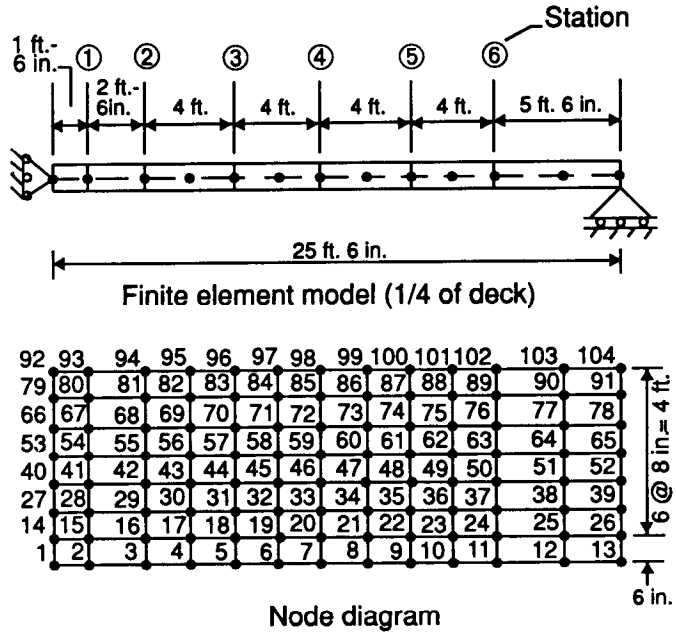

Figure 15—Finite element grid used to model one-quarter of bridge deck for 51-ft span. Symmetry of deformation and support conditions were used to allow modeling of portion of deck. Loads were applied on element near node 1.

Field Bridge Tests

The Mormon Creek Bridge was installed with the cooperation of Hiawatha National Forest personnel. Since construction, the bridge has been field tested under truck loading, and it is being continuously monitored for stress loss and creep deflection. Results of the field performance of this bridge have been reported (McCutcheon 1992).

The Mormon Creek Bridge is generally similar to the laboratory test bridge but has a shorter span. The construction details are as follows:

- 40-ft-long, 38-ft clear span, 16.5-ft-wide, single-lane bridge.
- 48 Vierendeel-type trusses, 3-7/8 in. wide by 24 in. deep. Material visually graded No. 1 Douglas Fir, pressure treated with creosote.
- Chords made from rough-sawn 4- by 6-in. members; webs made from rough-sawn 4- by 12-in.-deep blocks.
- Butt joints in top and bottom chords the same as joints in laboratory test bridge.
- Chord members and web blocks connected by three 5/8-in.-diameter steel dowels.
- Prestressing rods spaced at 4-ft intervals.
- 150 x 10^3 lb/in^2 steel rods in two diameters: 1 in. and 1.25 in.
- Trusses assembled and stressed together in two separate panels by stress rods in solid web blocks for shipping to bridge site. No rods were placed in web openings of panels until the bridge was finally assembled.

The Mormon Creek prototype bridge was designed as a part of this research project and fabrication bids were sought from private contractors. The bridge superstructure was prefabricated in two panels composed of 24 truss laminations each and prestressed by the fabricator before shipping to the job site. The prefabricated panels were shipped to the site and lifted into place on the timber pile caps of the substructure. The bridge was then assembled by laminating the individual panels together through prestressing rods placed in the large web openings. Timber curbs and a plank wearing surface were later added to the deck. Load cells were placed on specific prestressing rods to monitor long-term prestress loss resulting from creep, relaxation, and moisture content variation within the timber.

Figure 16—Placement of prefabricated bridge panel for Mormon Creek parallel-chord bridge. Panels were set in place on prefabricated abutments and then stressed together. (M87 0281-14)

Deflection measuring devices 'were permanently attached to the bottom of the superstructure to monitor creep deflection over time. Figures 16 and 17 show the placement of the prefabricated panels and the final state of the bridge. Itemized costs of the superstructure are listed in the Appendix.

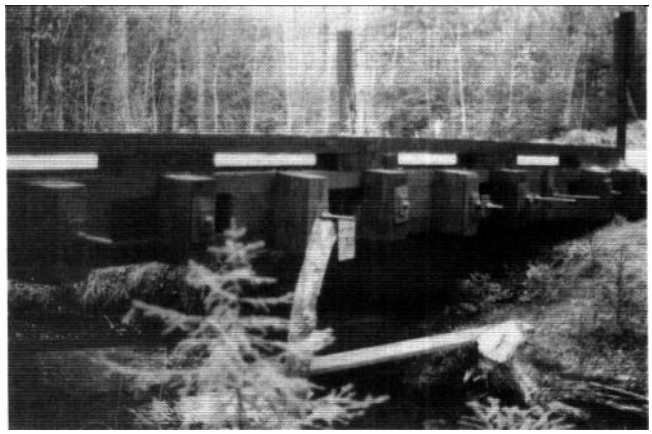

Figure 17—Side view of assembled Mormon Creek parallel-chord stress-laminated bridge. (M87 0324-11)

Results and Discussion

The results of the laboratory tests on both the individual trusses and the stress-laminated bridge deck are summarized in this section. Included are laboratory test results involving simulated truck loading, response of a bridge with individual panels connected by a weak

joint, determination of compressive prestress within the wood, response of the bridge to cyclic load tests, and comparison of analytical and experimental results.

Behavior of Individual Trusses

From research reported by Oliva and Lyang (1987), a plot of load and associated shear slip for a typical chord-to-web-block joint is shown in Figure 18. For a complete individual truss, the average stiffness for a point load at center span, taken as the load divided by the center displacement, was 3.98 x 10^3 lb/in for a truss without butt joints in either chord. Three of the 25-ft trusses were stress-laminated together. Figure 19 shows the load resisting behavior of the trusses; average stiffness per truss was 6.73 x 10^3 lb/in. The increase in stiffness after the trusses were laminated together was due to the stiffer web-to-chord connection created by the prestress rod anchorage system. When a butt joint was placed in the top and bottom chords of the middle truss, the stiffness was initially reduced, but the initial stiffness was regained with a second load repetition.

Three sets of 50-ft trusses from the laboratory bridge were individually loaded after all the truck load tests were completed. Stiffness values obtained from three individual trusses with different butt-joint layouts were 0.69, 0.33, and 0.55 x 10^3 lb/in with a centerpoint load. An additional test on a small group of four trusses, which were not stress-laminated together but were forced to undergo equal simultaneous displacements, resulted in an average stiffness of 0.62 x 10^3 lb/in. Because the trusses had been severely loaded during the simulated truck tests, the results of the individual truss tests probably reflect decreased stiffness.

Simulated Truck Loading

A typical transverse displacement profile for simulated truck loading at center span, obtained while testing the deck under anchorage configuration 1, is shown in Figure 20. The system behaved linear-elastically, with an average stiffness of 20.4 x 10^3 lb/in at a level of prestress representing service conditions; that is, 50 lb/in^2. Stiffness is defined as the load at center divided by the midspan deflection. The displaced shape of the deck when subjected to two loads applied at a transverse section, each 1.5 ft from the deck edge, is shown in Figure 21.

The simulated truck wheel load was applied in five 0.2-in. increasing displacement increments and three decreasing increments. Thus, the maximum displacement imposed on the deck in this stage of the testing was approximately 1 in. This displacement is below that expected from an equivalent HS 20-44 truck load. The maximum load applied on the deck was 20.5 x 10^3 lb. That load gives approximately 80 percent of the longitudinal moment at the center of the deck expected from one line of wheels of an HS 20-44 truck positioned for

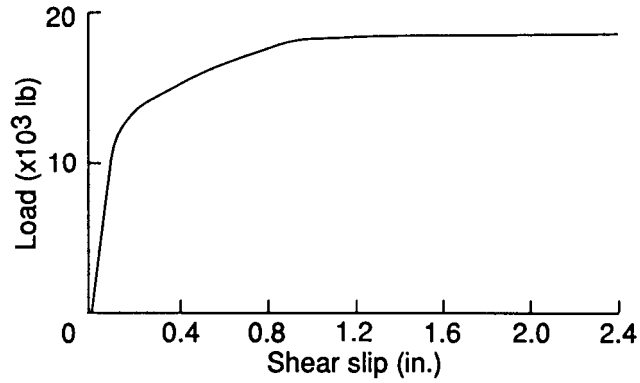

Figure 18—Shear slip as a function of load for individual joint between chord and web in Vierendeel truss.

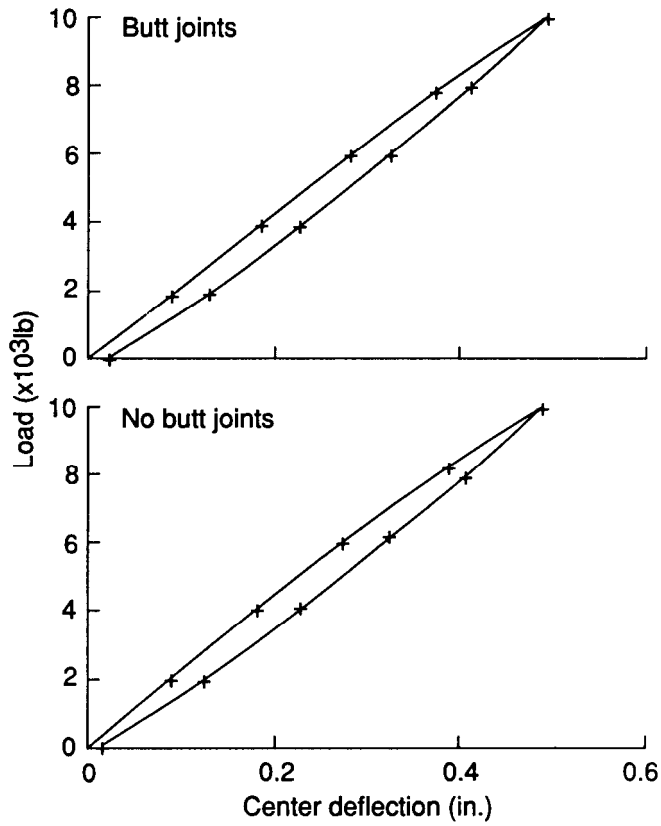

Figure 19—Load resistance of three trusses, with and without butt joints.

maximum longitudinal moment. Thus, the actual deflection under an HS 20-44 truck wheel line would be approximately 1.15 in.

Figure 22 shows the variation in stiffness of the bridge with anchorage configuration 1 as a result of changes in the "uniform" level of prestress in the deck. At a prestress <20 lb/in^2, serious reduction in stiffness occurred, similar to that experienced with solid stress-laminated decks (Oliva and others 1990). A visual

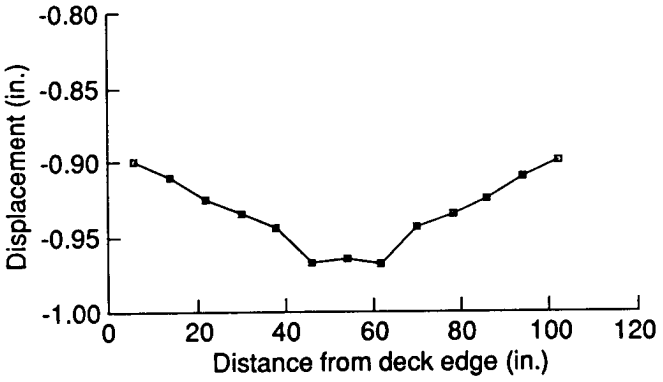

Figure 20—Vertical displacements along transverse cross-section near center of deck resulting from centerpoint load (anchorage configuration 1).

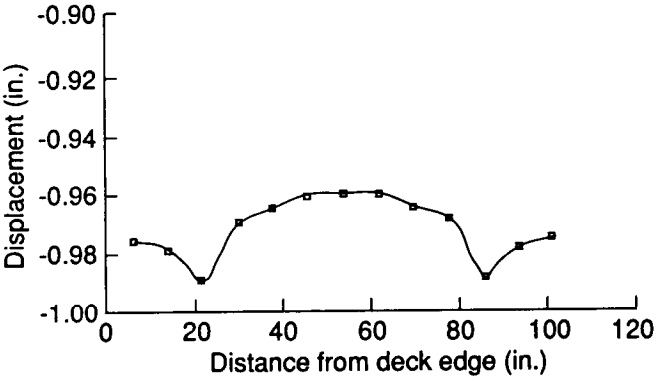

Figure 21—Vertical displacements along transverse cross-section near center of deck resulting from two centerpoint loads (anchorage configuration 1).

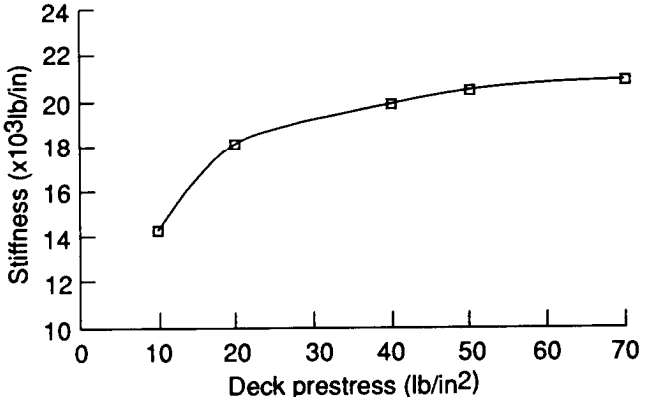

Figure 22—Deck vertical load-resisting stiffness with change in prestress level for anchorage configuration 1 with centerpoint load. (Stiffness is centerpoint load (× 10^3 lb/in) per inch of deflection at center.)

inspection of the deck after loading at a prestress level of 10 lb/in^2 did not indicate any significant slip between the laminae. Thus, we surmise that the loss in stiffness at stress levels <20 lb/in^2, obvious in Figure 22, was a result of interlaminar opening between trusses at the bottom of the deck caused by the transverse bending moments, as noted in the solid deck research. An attempt to investigate the system under prestress >70 lb/in^2 was unsuccessful. The posts acting as the bulkhead system deformed extensively and, in many cases, failed under prestressing forces necessary to achieve average compression >70 lb/in^2 in the timber bridge. Post failure occurred in one of two fashions: (1) the post began bulging in the direction perpendicular to the compressive load and finally split along a vertical plane containing the stressing rod or (2) the post compressed more on one side of the rod than the other and post crushing was initiated. These failures were induced by the varying stiffness of the wood depending on the ring structure of the solid-sawn posts. Because high initial levels of prestress are necessary in actual field installations, glulam posts are recommended to more effectively transfer the compressive stress into the deck.

Simulated truck loading tests were conducted under all anchorage configurations, as described previously. Results for the different systems are plotted together in Figure 23. Note that at a prestress level in the range of 10 to 70 lb/in^2, the structural performance of the system, as measured by stiffness, was approximately the same regardless of the anchorage configuration. Thus, all the anchorage systems are equally adequate from a performance viewpoint, and selection of a particular type of detail should be based on economy and ease of construction. Further study of Figure 23 indicates that at a low prestress level of 20 lb/in^2, configuration 4, which had four rods in each web opening, exhibited slightly higher stiffness than the other configurations. This was expected because all rods are positioned close to the top and bottom chords of the deck in configuration 4, thus enhancing the transverse stiffness capacity of the deck.

Panel Joints

Figure 24 shows plots of transverse displacements at a section near the center of the deck, where the load was applied, for various prestress levels across the joint (6, 14, and 20 lb/in^2) and in the deck (20, 50, and 70 lb/in^2). The measured stiffness of the deck (load at midspan/displacement at midspan) was 14.8, 17.7, and 18.5 × 10^3 lb/in., respectively. At a low level of prestress (20 lb/in^2 uniform prestress in deck, 6 lb/in^2 across joint), slip occurred at the joint, as indicated by the discontinuous displacement profile. At higher prestress levels (70 lb/in^2 uniform prestress in deck, 20 lb/in^2 across joint), the displacement profile was nearly continuous, indicating the existence of adequate moment transfer across the joint. The stiffness at 20 lb/in^2 across joint and 70 lb/in^2 in the panels

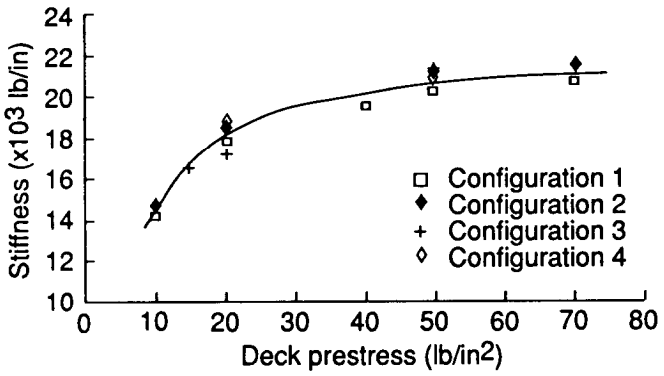

Figure 23—Deck vertical load-resisting stiffness with change in prestress level for all anchorage configurations.

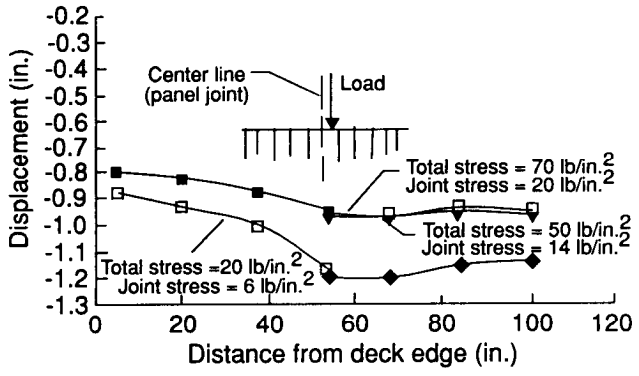

Figure 24—Vertical displacements along transverse cross-section near center of deck, showing the effect of joint between panels at different prestress levels.

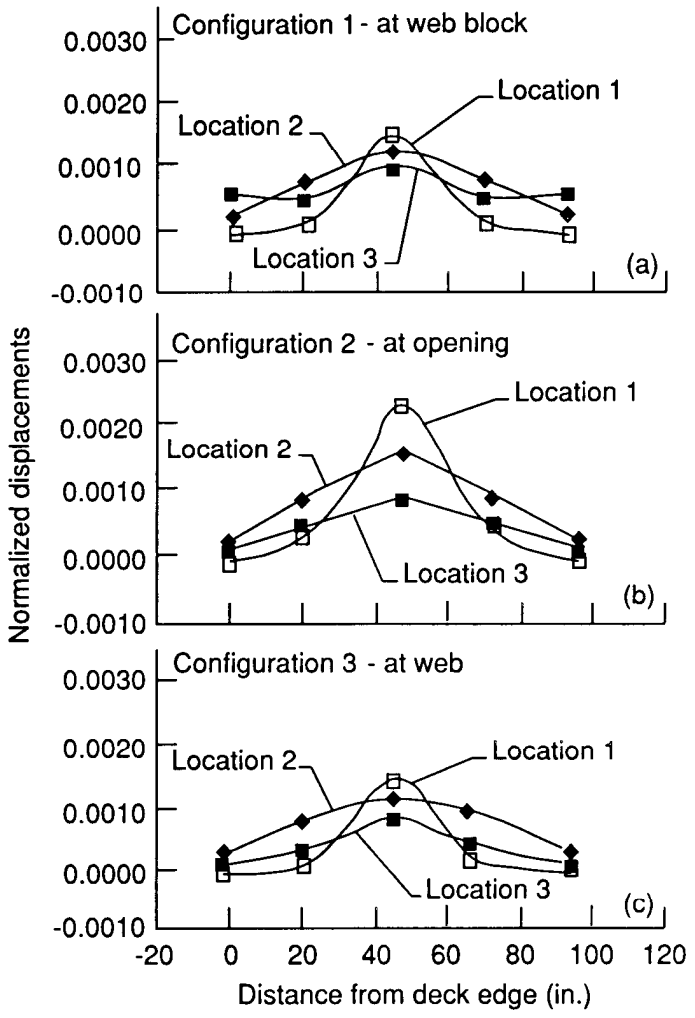

Figure 25—Transverse compression of deck with increase in stress for three anchorage configurations. Adjacent rods located at 0 and 96 in. from deck edge. Location 1—compression in next 18 in. nearest deck edge. Location 2-compression in next 18-in. strip from edge. Location 3—compression in strip 3 ft from edge. Configuration 1—at web with two rods. Configuration 1—at opening with one rod. Configuration 3—at web with one rod.

(18.5×10^3 lb/ in) was near the level shown in Figure 23 for 20 lb/in^2. It is obvious from Figure 24 that the low prestress across the joint did not seriously affect the shear capacity but did offset the transverse flexural stiffness (slope discontinuity).

Prestress Distribution

Figure 25 shows how the transverse compressive deformations, normalized to units of average strain on the deck surface, varied in the vicinity of an anchorage. Each curve in the figure shows deformation at a location successfully further from the edge of the deck; location 1 was within 1.5 ft of the edge and location 3 was >3 ft from the edge. A close examination of the transverse deformation plots suggests the following:

1. Deformation is higher in the immediate region of the applied load. This occurs regardless of rod arrangement (one or two rods) or bulkhead system and indicates that the prestress is transferred over a narrow width near the outside of the deck.

2. At some distance from the edge (that is, near location 3), the deformation approaches a "uniform" state.

3. The two deformation plots representing displacements at webs are qualitatively identical but differ from the plot that represents deformation data from the web opening location. The reason for the higher concentrated displacement at location 1 for the web opening configuation is reasonable, because all prestress is being transferred into the two chords, while in the web configuration, there is variation over the depth of the truss.

Table 5—Stiffness of deck during cyclic loading

Time of test	Stiffness[a] ($\times 10^3$ lb/in)	Loss in stiffness compared to initial stiffness (percent)
Initial	18.7	—
After 100,000 cycles	18.1	-3.2
After 400,000 cycles	17.7	-5.3
After 500,000 cycles	17.2	- 8

[a] Stiffness measured as centerpoint load divided by center deflection ($\times 10^3$ lb/in).

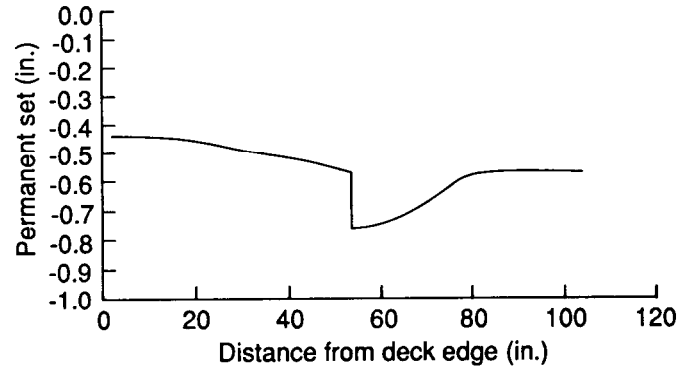

Figure 26—Vertical residual displacements across transverse profile near center of deck after dynamic load testing with one-half million load cycles. Displacement offset occurred at panel joint, where very little prestress existed.

Cyclic Loading

Table 5 lists the initial stiffness value of the two-panel bridge deck along with stiffness values measured later in the cyclic loading test sequence. After one-half million cycles, stiffness was reduced a total of approximately 8 percent. This loss of stiffness was not totally due to deterioration caused by the repetitive action of the applied load. The duration of the cyclic test was approximately 1 week, including some idle time while the test sequence was changed and time spent conducting the various stiffness tests. Relaxation tests conducted on solid stressed decks indicated that the rods lose a portion of their tension load primarily as a result of creep in the wood (Oliva and others 1990). At the end of the dynamic tests, the average stress loss in all rods recorded was 35 percent (deck was initially stressed to 50 lb/in^2). The compressive stress between panels was reduced from 14 to 10 lb/in^2. We can logically assume that this accounted for part, if not all, of the stiffness reduction. A permanent slip did develop at the joint between panels. The slip was confined to within a distance of 2 ft, in the direction of the span, on either side of the applied load. Maximum slip measured was 0.188 in. at the joint and was undoubtedly a result of the unacceptably low level of prestress across the joint. The measured permanent set of the deck across a transverse section directly under the load, after one-half million cycles, is displayed in Figure 26.

Analytical and Experimental Results

The general analytic ability of the equivalent plate model was verified by comparison with measured results of simulated truck loading in the laboratory tests over various spans and with various load locations. Analyses were conducted for three different spans. Correlation plots, between measured and predicted displacements, are shown in Figure 27 for the deck with 50 lb/in^2 of prestress. Each plot shows the deflections that occurred across a transverse cross-section at the bridge midspan. Considering the complexity of the laboratory test model and the assumed relation between the plate rigidities, the correlation between theory and experiment is adequate; the maximum difference was less than 10 percent.

Conclusions and Recommendations

The stiffness of the parallel-chord bridges described in this report was improved compared to that of previously studied stress-laminated solid-sawn timber bridge decks with spans >35 ft. The parallel-chord system appears to be a feasible means of improving the overall performance and efficiency of timber bridges. However, the cost for the superstructure of the prototype bridge must be considered ($33.45/ft^2 for materials only; see Appendix).

Resistance to Truck Loading

In this study, the parallel-chord stress-laminated bridges could provide the desired stiffness; the stiffness can easily be modified by increasing the overall depth of the trusses. Stress laminating allowed the bridge system to effectively transfer applied loads to a wide portion of the deck trusses. This lateral transfer of load depends on the depth of the trusses and results in a wide effective distribution width or effective resisting width of the bridge deck if the deck is assumed to act as a simple beam. In addition to developing a wide transverse load distribution, the system exhibited no measurable slip or rotation between laminae when sufficient prestress was maintained.

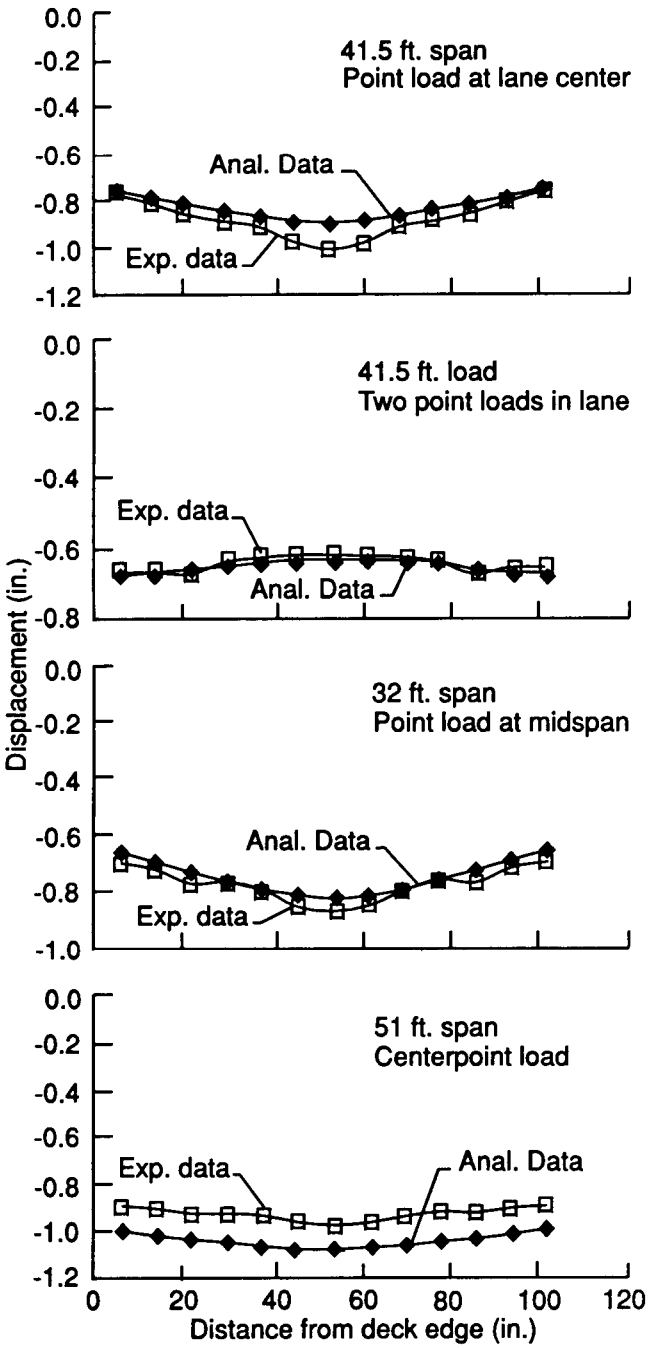

Figure 27—Vertical deflections across transverse profile near center span: experimental and analytical results for various spans and load points.

Ease of Construction

The parallel-chord system lends itself to easier prefabrication of panels and connection at the bridge site compared to solid-sawn stress-laminated panelized decks. Prefabricated laminated panels may be erected in place and subsequently stress-laminated together to form a complete bridge by placing rods through the openings in the truss webs and stressing.

Prestress Rod Anchorage System

Variations in configuration of anchorage systems for prestressing rods in the deck have little effect on load resisting behavior as long as the anchorages are reasonably designed to transfer prestressing force into both the chords and webs. Anchorage plates must be designed to avoid crushing of wood under the plates and posts. If posts are used as part of the anchorage system, we recommend that they be made of glued laminated timber or laminated veneer lumber for improved resistance to splitting under load.

Repeated Load Resistance

Over one-half million cycles of loading, at a level inducing deflections caused by an HS 20-44 truck, failed to create any significant deterioration of the bridge stiffness. The joint between panels is likely to be the one region of the bridge system that may be susceptible to damage by repeated loading if the prestress across the joint is not maintained at a sufficiently high level.

Composite Action of Parallel-Chord Truss

The efficiency of any composite section depends on the transfer of forces between materials in the cross section. The parallel-chord Vierendeel trusses used in this study relied upon the action of steel dowels to transfer shear forces between the chords and webs. Some deformation developed in the dowels, which allowed a small slip between the chords and web elements. The trusses also had higher vertical shear deformation than normally occurs in a truss with continuous diagonal web elements because of vertical deformation in the chords between the discrete web blocks. Tests results indicated that the effective cross-section moment of inertia was 40 percent of the moment of inertia that would occur if the chords were perfectly connected and shear deformation was neglected. This reduction in stiffness was due to horizontal shear-slip in the doweled web connection and vertical shear deformation at openings between the web blocks. The relative size of these two deformation mechanisms depends on dowel size, number of horizontal shear interfaces in chord-to-web connection, and independent moment of inertia of chord members. The size of deformation mechanisms may also be subsequently affected by moisture content variations in the wood.

Analytical Modeling

Analytical modeling of a parallel-chord bridge system would be unacceptably complex if approached in an exact manner. The use of an "equivalent" orthotropic plate analytic model provided predictions of the bridge

deflection response that were within 10 percent of actual measured values.

Future Use of Parallel-Chord Stress-Laminated Bridges

The parallel-chord bridge has been shown to be an attractive alternative to other methods of bridge construction. Nevertheless, this type of bridge is in the experimental stage. Before this system can be used for production bridges, the following issues must be developed or clarified:

1. The stress-laminating technique is completely dependent on maintaining minimum levels of compressive prestress between the laminae. The loss of prestress resulting from creep of the wood, relaxation of the steel, and volumetric changes in the wood caused by moisture variation must be thoroughly understood before this method can be used on any application other than experimental bridges.
2. Methods must be developed for designing individual trusses for the parallel-chord bridge. The design process depends on the shear stiffness of the web-chord connection. Values for shear stiffness of doweled or metal plate connections are not currently well-documented or readily available. Improved parallel-chord systems may be developed that do not rely upon any mechanical fastenings between the members in any individual truss, resulting in a simplification of the design process.
3. The ultimate capacities of individual trusses, at a failure limit state, as well as the capacities of stress-laminated subassemblies need to be documented to provide a design engineer with a measure of the actual safety factor against failure in the system. Load sharing will probably reduce system strength variability and may allow design with an increased capacity over that expected using normal wood stress design tables.
4. Proven techniques for protecting prestressing rods or strands are necessary. Steel is highly susceptible to corrosion when it is in a state of elevated stress. The prestressing elements are more exposed in timber bridges than in prestressed concrete bridges, where grouting or greased sheathing is commonly used in addition to embedment in concrete.

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Appendix

Materials and costs for superstructure of Mormon Creek Bridge.[a]

Bridge component	Item[b]	Number of pieces	Quantity	cost ($)
Trusses	4-in. x 6-in. x 8-ft Douglas Fir	50	800 fbm	880
	4-in. x 6-in. x 16-ft Douglas Fir	50	1,600 fbm	1,760
	4-in. x 6-in. x 24-ft Douglas Fir	50	2,400 fbm	2,640
	4-in. x 6-in. x 32-ft Douglas Fir	50	3,200 fbm	3,520
	4-in. x 12-in. x 3-ft Douglas Fir	100	1,600 fbm	1,320
	4-in. x 12-in. x 4-ft Douglas Fir	200	3,200 fbm	3,520
	Total			13,640
Curbs and scupper blocks	4-in. x 6-in. x 10-ft Douglas Fir	16	320 fbm	352
	6-in. x 10-in. x 22-ft Douglas Fir	4	440 fbm	484
	6-in. x 10-in. x 5-ft Douglas Fir	4	100 fbm	110
	6-in. x 10-in. x 4-ft Douglas Fir	6	120 fbm	132
	Total			1,078
Miscellaneous supplies	10.75-in. x 12-in. x 2-ft posts	22		578
	0.75- x 34-in. dowels		30.8 lb	46
	0.75- x 16-in. dome-head bolts	18	52.0 lb	78
	0.75- x 16-in. lag screws	8	16.3 lb	24
	0.75-in. plate washers	18	15.3 lb	23
	0.75in. cut washers	8	1.2 lb	2
	0.38 x 11 in.-ring-shank spikes	—	20.0 lb	20
	60d (6-in.) ring-shank spikes		9.0 lb	9
	0.6- x 16-in. drift pins	1,602	2,450.0 lb	3,677
	Total			4,457
Anchorages	1-in.-diameter prestress rod		256 ft	397
	1.25-in.-diameter prestress rod	—	78 ft	179
	1-in.-diameter nuts	34	—	177
	1.25-in.-diameter nuts	12		85
	Plates for l-in. rod anchor	34	—	269
	Plates for 1.25-in. rod anchor	12	—	139
	Bearing plates	6	—	334
	Bearing plates	6	—	174
	Bearing plates	6	—	167
	Bearing plates	6	—	131
	Bearing plates	10	—	182
	Total			2,234
Superstructure[c]	Total	—		21,409

[a] See Table 1 in text for metric conversion factors.
[b] Width and thickness of wood are nominal dimensions (inches). Wood was treated with creosote.
[c] Total cost = $33.45/ft^2.