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Field Performance of Timber Bridges

7. Connell Lake Stress-Laminated Deck Bridge

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Abstract

The Connell Lake bridge was constructed in early 1991 on the Tongass National Forest, Alaska, as a demonstration bridge under the Timber Bridge Initiative. The bridge is a stress-laminated deck structure with an approximate 36-ft length and 18-ft width and is the first known stress-laminated timber bridge constructed in Alaska. Performance of the bridge was monitored for 2-1/2 years, beginning at bridge construction. Performance monitoring involved gathering and evaluating data relative to the moisture content of the wood deck, the force level of the stressing bars, and the deflection under static load. In addition, comprehensive visual inspections were conducted to assess the overall condition of the structure. Based on 2-1/2 years of field evaluations, the deck is performing well with no structural deficiencies. However, a slight sag has developed at midspan and several stressing bar bearing plates have crushed into the outside deck laminations.

Keywords: Timber bridge, wood, stress laminated, glued-laminated timber

Acknowledgments

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Introduction

In 1988, the U.S. Congress passed legislation known as the Timber Bridge Initiative (TBI). One objective of this legislation was to provide for the construction of demonstration timber bridges to encourage innovation through the use of new or previously underutilized wood products, bridge designs, and design applications. In so doing, bridge designers and users become more aware of the attributes of wood as a bridge material, and new, economical, structurally efficient timber bridge systems should result. As a national wood utilization research laboratory within the USDA Forest Service, the Forest Products Laboratory (FPL) has taken a lead role in assisting bridge owners in evaluating the field performance of demonstration bridges, many of which use design innovations that have not been previously evaluated. This has involved the development and implementation of a comprehensive national bridge monitoring program.

This report is seventh in a series that documents the field performance of timber bridges built as a part of the TBI. This report describes the design, construction, and field performance of the Connell Lake bridge located in the Ketchikan Area, Tongass National Forest, near Ketchikan, Alaska. The bridge, built in 1991, consists of a single-lane, single-span, stress-laminated deck with a length of 36.1 ft and a width of 17.9 ft. (See Table 1 for metric conversion factors.) The bridge design is unique in that it is the first known U.S. application of a stress-laminated deck in Alaska. An information sheet on the Connell Lake bridge is provided in the Appendix.

Background

The Connell Lake bridge is located approximately 5 miles north of Ketchikan, Alaska, and is on an 18-ft-wide gravel road that crosses a tributary of Ward Creek (Fig. 1). This road provides access primarily for recreational and Forest Service administrative traffic. The original structure was a timber stringer and plank bridge, which inspections showed as deteriorated and in need of replacement. A decision was made by the Ketchikan Area personnel to replace the bridge with a stress-laminated deck. The project was proposed and accepted as a demonstration bridge under the TBI. Subsequently, the FPL and Ketchikan Area entered into an agreement to monitor the performance of the bridge, thus obtaining information on the new, experimental bridge design and the effects of the Alaskan environment on the experimental bridge design.

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

English unit	Conversion factor	SI unit
foot (ft)	0.3048	meter (m)
inch (in.)	25.4	millimeter (mm)
mile	1,609	meter (m)
pound (lb)	4.448	newton (N)
lb/in^2 (stress)	6,894	pascal (Pa)
lb/ft^2	4.88	kilogram/meter^2 (kg/m^2)

Objective and Scope

The objective of this project was to evaluate the field performance of the Connell Lake bridge for a minimum of 2 years, beginning when construction was completed. The project scope included data collection and analysis related to the wood moisture content, stressing bar force, deflection under static load, and general structural performance. The results of this project, in conjunction with the results from monitoring other bridges, will be used to formulate recommendations for the design and construction of similar bridges in the future.

Design and Construction

Design of the Connell Lake bridge was completed by West Virginia University (WVU) personnel and approved by the Forest Service Alaska Regional Office. Construction was completed by a local contractor. This paper presents an overview of the design and construction of the bridge superstructure.

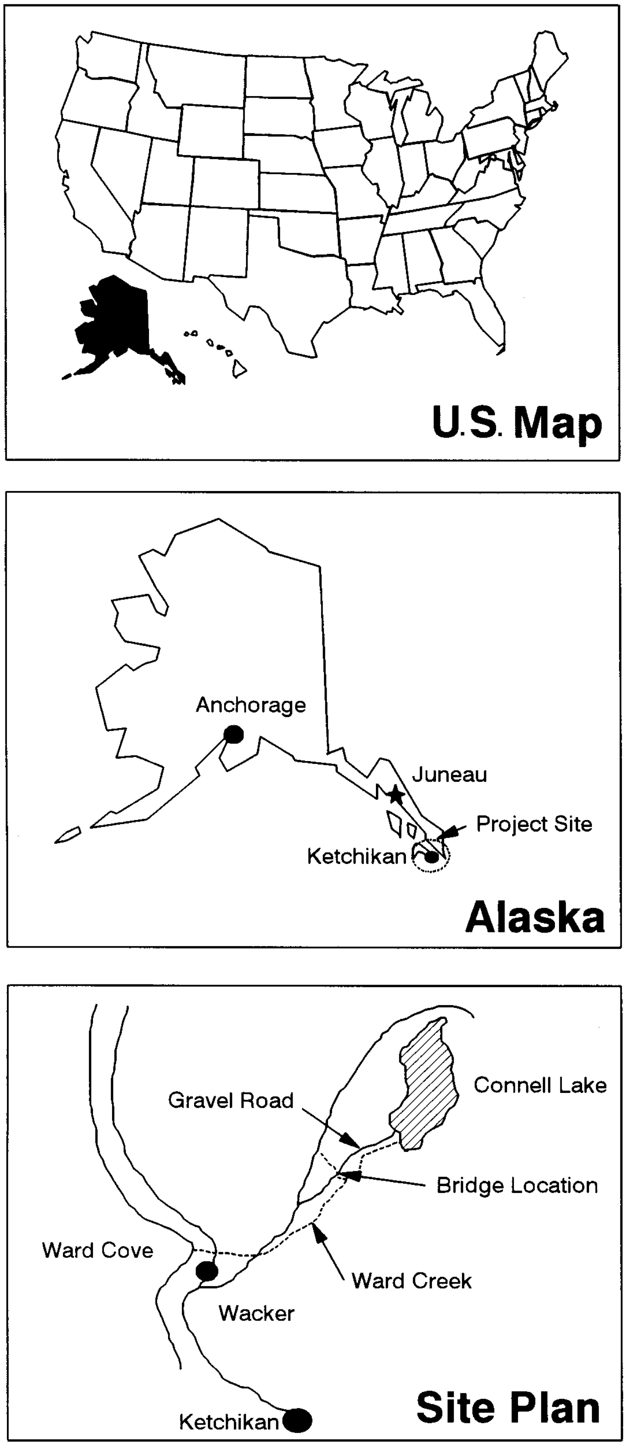

Figure 1—Location of the Connell Lake bridge.

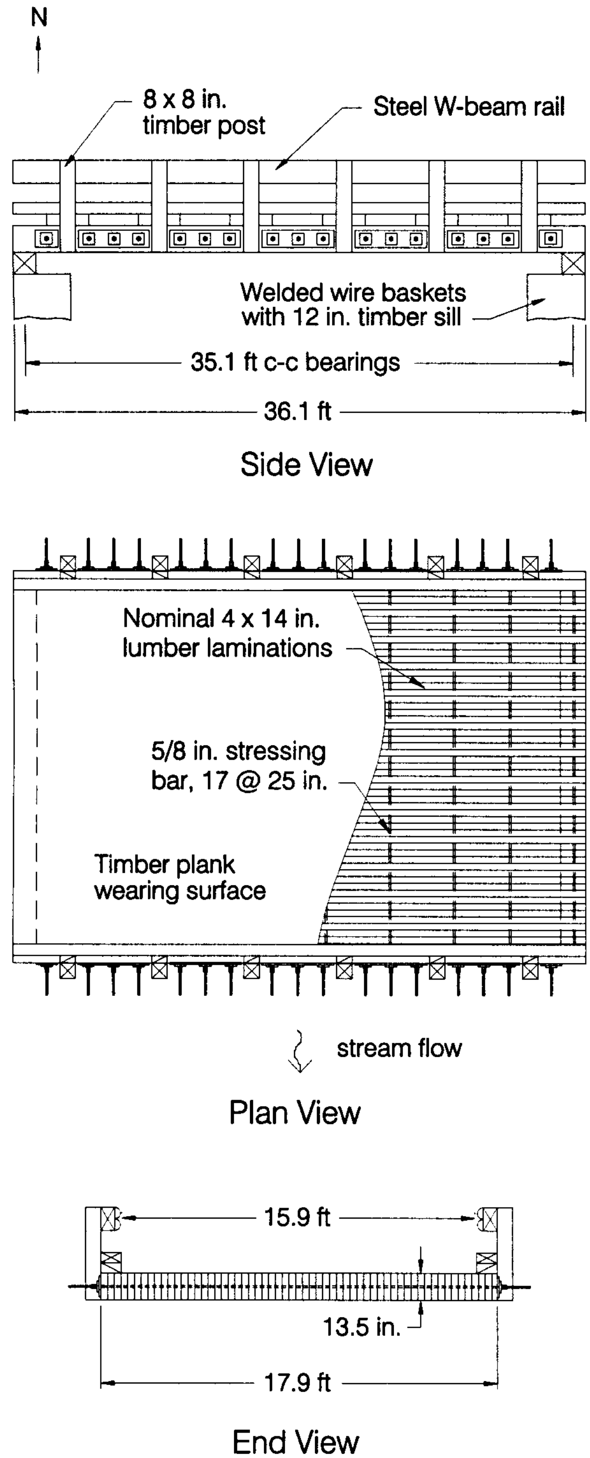

Figure 2—Design configuration of the Connell Lake bridge.

Design

The Connell Lake bridge deck was designed in accordance with the WVU method that is based on criteria developed at WVU (Davalos and others 1993). The bridge was designed for the HS 20-44 vehicle as recommended by the American Association of State Highway and Transportation Officials (AASHTO) *Standard Specifications for Highway Bridges* (AASHTO 1989). The bridge configuration consisted of a span length of 35.1-ft center–center of bearings, a 17.9-ft width out–out, and a nominal 14-in. thickness (Fig. 2). Douglas Fir–Larch, visually graded No. 1 Dense, nominal

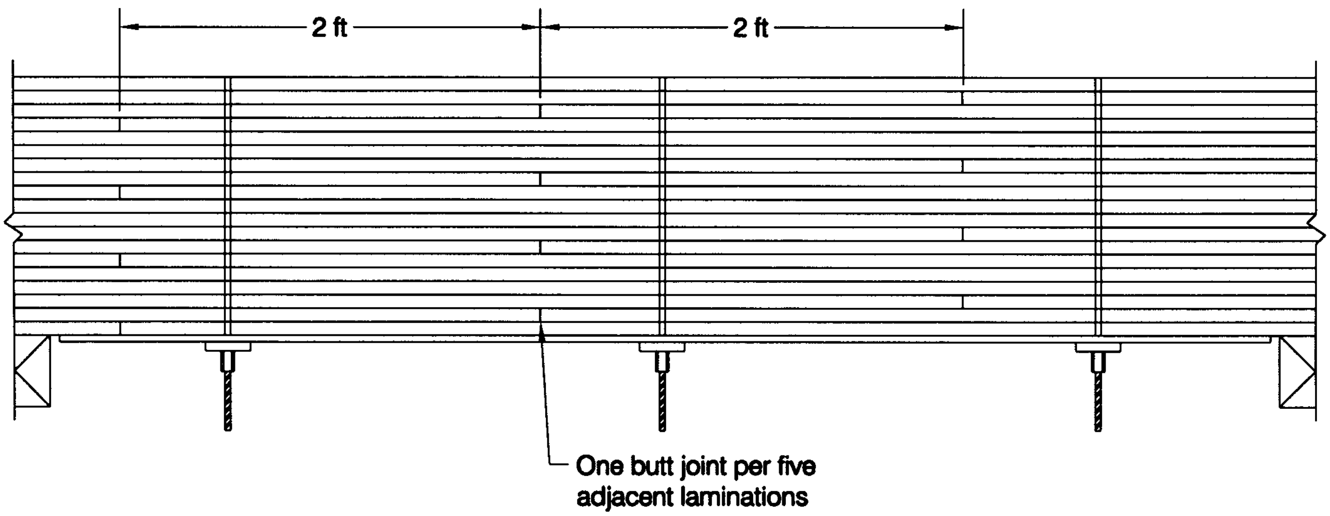

Figure 3—Butt-joint configuration used for the Connell Lake bridge. A butt joint was placed transverse to the bridge span in every fifth lamination. Longitudinally, butt joints in adjacent laminations were separated by 2 ft.

4- by 14-in. laminations were selected as the deck material. The allowable live-load deflection was limited to 1/360 of the bridge span measured center–center of bearings, or 1.17 in.

The design included a butt-joint pattern developed at WVU. Butt joints were located in every fifth lamination transversely, with a 2-ft longitudinal spacing between butt joints in adjacent laminations (Fig. 3). Similar bridges are designed with butt-joint configurations based on butt joints in every fourth lamination, with a 4-ft longitudinal spacing between butt joints in adjacent laminations (Ontario Ministry of Transportation and Communication 1983).

Design of the stressing system was based on the WVU method and provided an initial interlaminar compression of approximately 83 lb/in^2. The design interlaminar compression was provided by 0.625-in.-diameter, high strength stressing bars, which comply with the requirements of ASTM A 722 (ASTM 1988) and provide a minimum ultimate tensile strength of 1.5×10^5 lb/in^2. Bars were spaced 25 in. on-center, beginning 16 in. from the bridge ends. To achieve the 83-lb/in^2 interlaminar compression, a bar tension force of 28,000 lb was specified.

The stressing bar anchorage system was based on a relatively new design developed by WVU. In the past, the bar anchorage for most stress-laminated bridges has been either the steel channel or the discrete plate system (Ritter 1990). For the Connell Lake bridge, a continuous 6- by 60- by 0.5-in. bearing plate that extends over three stressing bars was used for the interior bars, and a 6- by 10- by 0.5-in. discrete plate was used for the two bars at the bridge ends (Fig. 4). In both cases, a 3- by 3- by 0.75- in. anchor plate was placed between the bearing plate and the flat hex nut.

The deck was provided with a 12-in. curb and a bridge rail system, meeting 50 percent of AASHTO static-load design requirements, which is typical for Forest Service, single-lane bridges (AASHTO 1989). The wearing surface consisted of diagonally oriented 4- by 12-in. timber planks connected to the deck with 8-in. double spiral spikes. Following fabrication, all wood components, with the exception of the wearing surface, were specified to be pressure treated with creosote-petroleum oil solution in accordance with American Wood Preservers' Association (AWPA) standard C1 and C2 (AWPA 1991). To provide protection from deterioration, all steel components were galvanized, including stressing bars, hardware, bar anchorage plates, and nuts.

Construction

Bridge construction began February 12, 1991, with removal of the existing abutments and placement of new, rock-filled, welded wire basket abutments with 12-in. timber sills (Fig. 5). In conjunction with the abutment construction, scaffolding and falsework were erected. Banded bundles of deck laminations were delivered to the bridge site and, upon completion of the abutments, individual laminations were placed on the falsework and the bars were inserted through holes predrilled through the center of the laminations. This construction technique was chosen as an alternative to installing prefabricated sections, because the weight of the crane to place prefabricated sections exceeded the access road weight restrictions. Individually placing laminations was manageable due to the relatively small number of

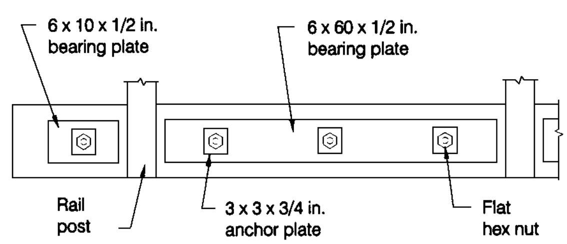

Figure 4—Details of the continuous bearing plate bar anchorage system.

Figure 5—Welded wire basket abutments and timber sills.

Figure 6—Hydraulic jack with built-in ratchet used for tensioning bars.

laminations necessary for the narrow deck width. After all laminations were placed, the bars were tensioned to the 28,000-lb design force using a single hydraulic jack with a built-in ratchet (Fig. 6).

Following the initial bar tensioning, the timber curb and rail system and the timber plank wearing surface were installed. Approximately 9 days after the initial tensioning, the bars were retensioned to compensate for anticipated losses in the bar force noted in previous research (Ritter 1990). Approximately 1 month after the second bar tensioning, the final bar tensioning was completed. At that time, two stressing bars were replaced because their length was inadequate for installation of the monitoring load cells. Construction of the Connell Lake bridge was completed in early 1991 (Fig. 7).

Evaluation Methodology

To evaluate the structural and serviceability performance of the Connell Lake bridge, the Forest Service Alaska Regional Office contacted FPL for assistance. Through mutual agreement, a monitoring plan was developed by FPL based on cooperation with the Ketchikan Area, Tongass National Forest. The plan called for monitoring several performance indicators, including the deck moisture content, bar force, deflection under static load, and condition of the structure for a minimum of 2 years after the final stressing. At the initiation of monitoring, FPL representatives visited the bridge site to install instrumentation and train Ketchikan Area personnel in moisture content and bar force data collection procedures. Load tests and condition assessments were conducted by FPL personnel with the assistance of Ketchikan Area personnel. The evaluation methodology utilized procedures and equipment previously developed (Ritter and others 1991) and is discussed in the following sections.

Moisture Content

Moisture content measurements were taken with an electrical resistance moisture meter with two insulated 3-in. pins in accordance with ASTM D4444-84 procedures (ASTM 1990). Measurements were taken at probe penetrations of 1 to 3 in., at five locations along the deck's underside, and were assumed to be representative of the global bridge moisture content (Fig. 8). The five measurements were averaged and, when necessary, adjusted for temperature to determine the actual moisture content value (Forintek 1984). Measurements were obtained at installation and periodically thereafter.

Bar Force

Measurements of stressing bar force were obtained to monitor changes and ensure that adequate bar force was maintained during the monitoring period. Bar force was measured using two calibrated load cells developed at FPL. The load cells were installed between the bearing and anchor plates on the third and ninth bars from the west abutment (Fig. 9). Load cell measurements were collected with a portable strain indicator by Ketchikan Area personnel. Strain units were converted to bar force based on laboratory calibrations, and the two bar forces were averaged to plot bar force change. Readings were taken biweekly for the first 100 days and periodically thereafter. Because equipment was not available for removal of the load cells, recalibration for zero shift was not performed.

Behavior Under Static Load

A static-load test was conducted at the conclusion of the monitoring, 2-1/2 years after bridge installation, to determine the bridge response to vehicle loading. The load test consisted of positioning a fully loaded, three-axle dump truck on the bridge and measuring the resulting deflections at a series of locations along a transverse cross section at midspan. Deflection measurements were obtained prior to testing (unloaded), after each placement of the test vehicle (loaded),

Figure 7—Completed Connell Lake bridge.

Figure 8—Electrical resistance moisture meter used for measurements.

Figure 9—Load cell installed for measuring bar tension force.

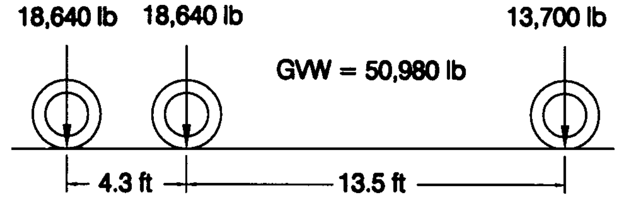

Figure 10—Load test vehicle configuration and axle loads.

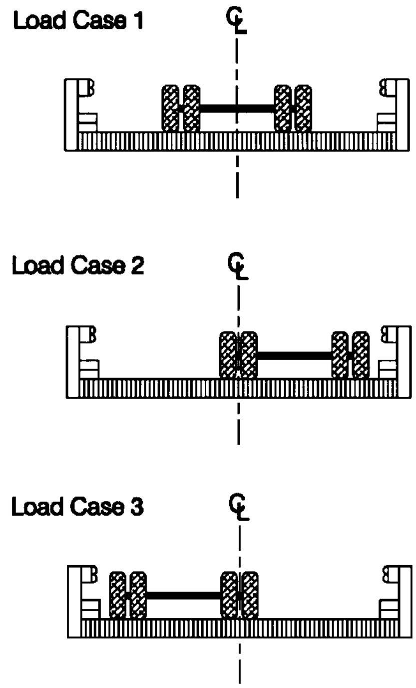

Figure 11—Transverse vehicle load cases as viewed from the east, looking west. For all load cases, the vehicle was facing east with the longitudinal vehicle centroid placed over the bridge midspan. The transverse vehicle track width was 6 ft, measured center–center of the rear tires.

and at the conclusion of testing (unloaded). Vehicle axle and gross weights were measured on a local truck scale. In addition, an analytical assessment was conducted using the theoretical bridge response developed by computer modeling.

Static-Load Test

The static-load test was performed August 25, 1993, using a test vehicle (Fig. 10). At the time of the test, the bar force was approximately 24,250 lb, which is equivalent to an interlaminar compression of approximately 72 lb/in^2. The vehicle was positioned longitudinally on the bridge facing east, with the centroid of the vehicle at midspan. Transversely, three load cases were used (Fig. 11). A surveyor's level was used to read deflection values to the nearest 0.04 in. from calibrated rules suspended from the bridge underside of the bridge. The accuracy of this method for repetitive readings is estimated to be ±0.02 in.

Analytical Assessment

Previous research has shown that stress-laminated decks can be accurately modeled as orthotropic plates (Oliva and others 1990). An orthotropic plate computer model developed at FPL was used to analyze the load test results. Measured and theoretical curves were compared considering the actual bridge and test vehicle characteristics and the representative analytical parameters of the computer model. The average modulus of elasticity of the individual laminations was assumed to be 1,900,000 lb/in^2. Using the same analytical parameters, the theoretical maximum live-load deflection for AASHTO HS 20-44 vehicle loading was derived and compared with the design live-load deflection for the span length center–center of bearings.

Condition Assessment

The general bridge condition was assessed on two occasions during the monitoring period. The first assessment occurred during construction when monitoring instrumentation was installed. The second assessment occurred at the time of the

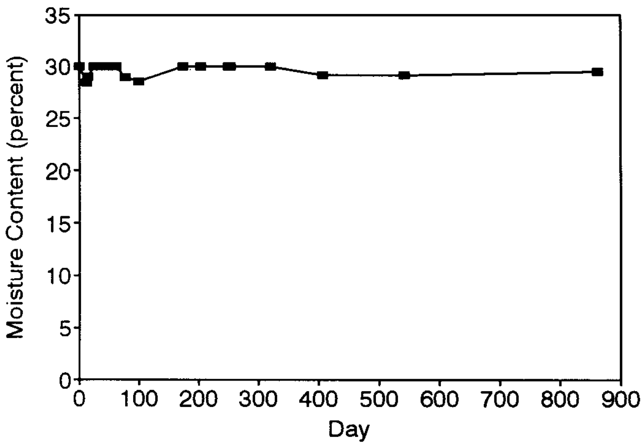

Figure 12—Average moisture content changes in the deck of the bridge.

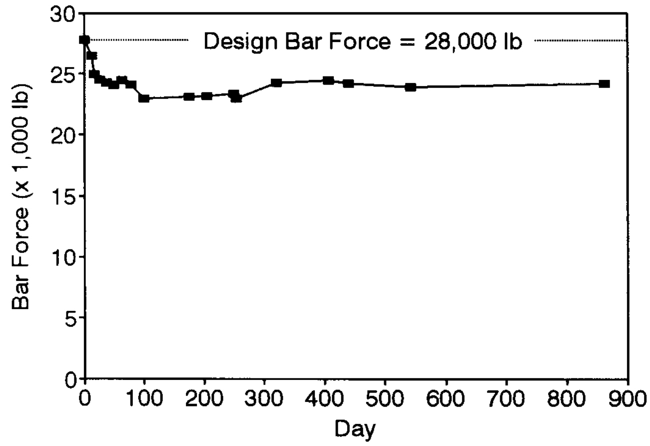

Figure 13—Average bar tension force obtained from load cells installed on two stressing bars.

static-load test at the conclusion of the monitoring period. These assessments involved visual inspections, measurements, and photographic documentation of the bridge's condition. Items of specific interest included condition of the wood components, wearing surface, and stressing bar anchorage system. In addition, changes in deck camber were determined from relative elevation measurements at the deck ends and midspan using a surveyor's rod and level.

Results and Discussion

The performance monitoring of the Connell Lake bridge extended from April 1991 through August 1993. The following presents results of the performance data.

Moisture Content

The trend in the average measured moisture content of the Connell Lake bridge is shown in Figure 12. As shown, the laminations were initially installed at a moisture content greater than 30 percent and remained at approximately the same level for the entire monitoring period. Thus, the moisture content was above the fiber saturation point (FSP), which is generally accepted as 30 percent but can vary from 25 to 30 percent (FPL 1987). Above the FSP, changes in moisture content do not cause dimensional changes in wood. However, dimensional changes do occur when the moisture content drops below the FSP. For stress-laminated bridges, changes in moisture content below the FSP are important because moisture loss causes wood shrinkage and subsequent bar force loss. Typically, the moisture content of a stress-laminated deck installed above the FSP gradually decreases towards an equilibrium moisture content (EMC) for the site.

The EMC depends on several factors, including site exposure, temperature, and relative humidity, and EMC has typically varied from 16 to 20 percent for other stress-laminated bridges (Ritter and others 1995). The Connell Lake bridge has remained above the FSP, and changes in moisture content during the monitoring period have been insignificant. We expect that the eventual EMC of the bridge may be as high as 25 to 28 percent and that changes from the present moisture content toward the EMC will be very slow. The basis for this conclusion centers on the site environmental conditions and the bridge wearing surface. The average rainfall and relative humidity are very high in the Ketchikan area, and periods conducive to wood drying are infrequent and of short duration. The bridge site is sheltered by large trees and frequent cloud cover that retards evaporation caused by warm temperatures. In addition, the wood plank surfacing on the bridge deck is not waterproof, which leads to trapped and retained moisture.

Bar Force

Figure 13 shows the average bar force beginning at the time of the final bar tensioning. Following the final bar tensioning, approximately 18 percent of the bar force was lost during the first 100 days. After this, the force remained relatively stable for the remainder of the monitoring period, with a slight increase in force noted between day 250 and 350. At the conclusion of the monitoring period, the average bar force was 24,250 lb, which represents an interlaminar compression of approximately 72 lb/in^2.

The bar force loss is mainly attributable to stress relaxation in the wood laminations, which is a slow deformation of the wood cells caused by the compressive force exerted by the stressing bars. Research has shown that the effects of stress relaxation increase with moisture content (Youngs 1957). Another possible contributing factor to bar force loss is crushing of the bearing plates into the exterior laminations; however, most of the crushing is believed to have occurred prior to the final bar tensioning.

Based on observations during the monitoring, we do not believe that retensioning of the bars will be required in the foreseeable future. However, retensioning may be required at a later date if stress relaxation and/or plate crushing continues or if the lamination moisture content drops below the FSP and the laminations begin to shrink.

Behavior Under Static Load

Results for the static-load test and analytical assessment of the bridge are presented. Transverse deflection measurements are given at the midspan from the east end looking west. No permanent residual deformation was measured at the conclusion of the test. In addition, no movement at either of the abutments was detected during the test.

Static-Load Test

Load test transverse deflection at midspan is shown in Figure 14, with maximum measured deflection locations and magnitudes denoted by solid boxes. For each load case, the deflection curves are typical of the linear elastic orthotropic plate behavior of stress-laminated bridges (Ritter and others 1990). For all load cases, the deflection curves are relatively smooth, with small differences between adjacent points in the regions of maximum deflection. For load cases 1 and 3, a single point of maximum deflection is not apparent as would be expected. In these cases, it is likely that differences between the maximum deflection and adjacent points are within the accuracy of the measurement method. Thus, the exact location of maximum deflection cannot be conclusively determined from these data.

We compared load cases 2 and 3 using a mirror image of load case 2 (Fig. 15). Assuming uniform material properties, symmetric loading, and accurate deflection measurements, the deflection should be the same. As shown, there are differences up to 0.15 in. that are most pronounced at the bridge edges. Given the locations of the deflection differences and the relative orientation of the plots, it is likely that the truck loading was slightly eccentric, which resulted in a greater loading for one wheel line compared with the other.

Given the 2-ft longitudinal butt-joint spacing, the deflection magnitude for the Connell Lake bridge was less than expected compared with similar bridges with a 4-ft longitudinal butt-joint spacing. The reduced deflection is most likely due to the diagonal timber plank wearing surface. We anticipate that partial composite action was present between the wearing surface and the bridge deck, resulting in a slight increase in the effective deck moment of inertia. This would decrease the deflection magnitude in both the longitudinal and transverse directions.

Analytical Assessment

The measured load test deflections compared with theoretical deflections are shown in Figure 16. The measured deflection is generally less than the theoretical deflection, even though the analysis considered approximately 50 percent of the wearing surface in the deck thickness as a result of partial composite action. Research has not been performed to assess the effects of a diagonal timber plank wearing surface that covers an entire stress-laminated deck. Although it is likely that the moment of inertia is affected, the effect on other analytical parameters is unknown.

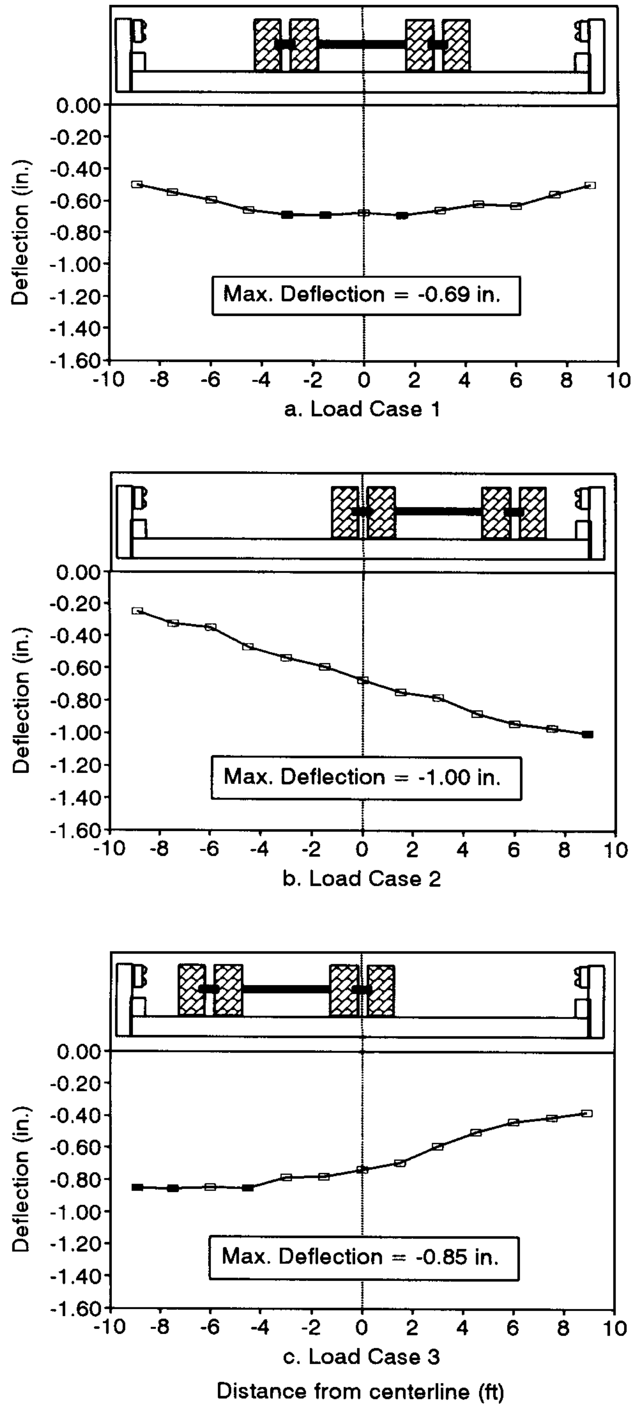

Figure 14—Transverse deflection for load test, measured at midspan. Bridge cross sections and vehicle positions are shown to aid interpretation and are not to scale.

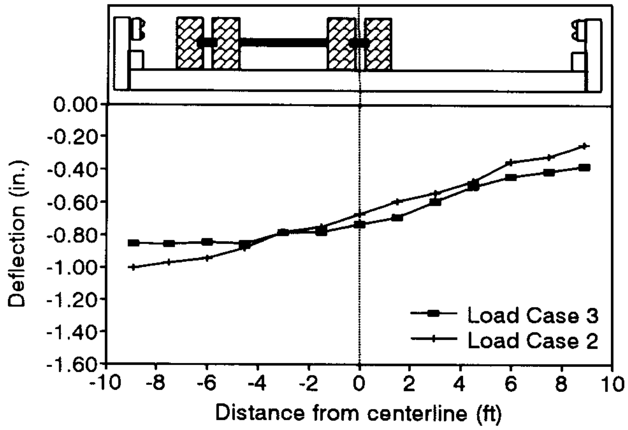

Figure 15—Comparison of load cases 2 and 3.

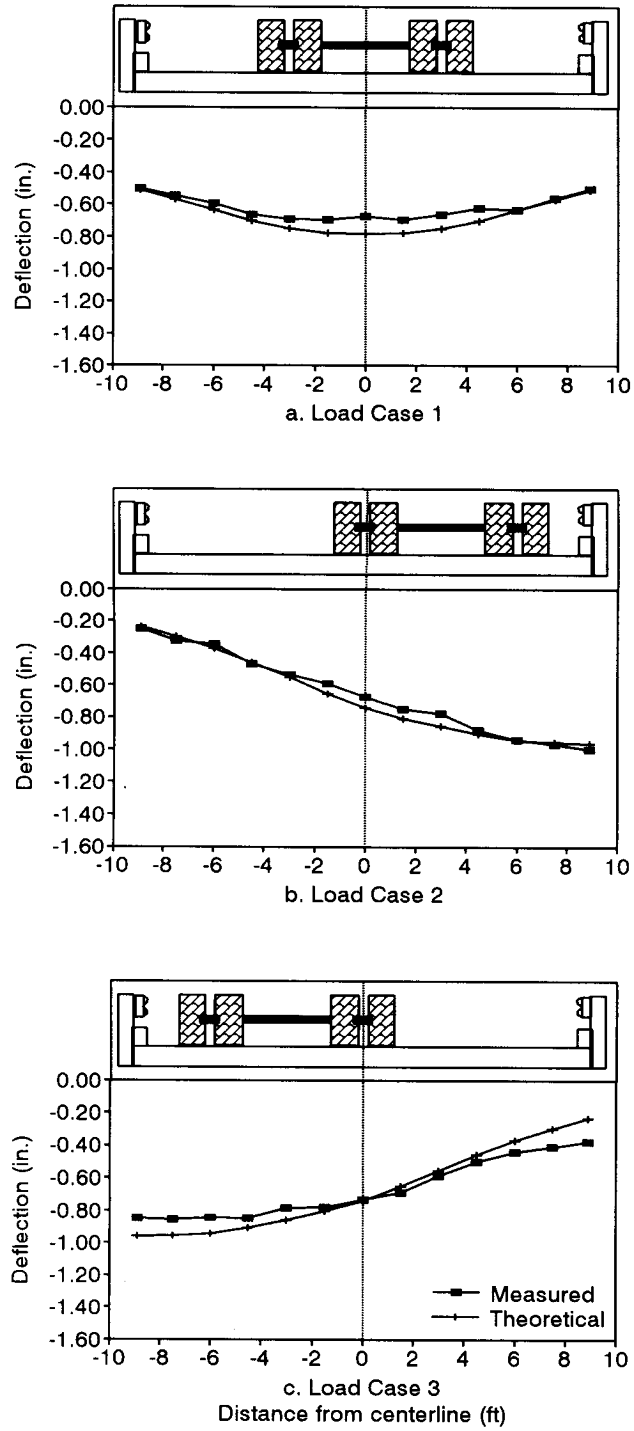

Figure 16—Theoretical deflection compared with measured deflection under the actual applied load using an orthotropic plate analysis.

Using the same analytical input parameters that were used for the load test analysis, the theoretical bridge deflections under AASHTO HS 20-44 truck loading are shown in Figure 17. The maximum theoretical HS 20-44 deflection is 0.96 in. or 1/439 of the bridge center–center of bearings span length for load case 1 and 1.19 in. or 1/354 for load cases 2 and 3. Both are within or very close to the maximum design live-load deflection of 1/360 but do not meet the current AASHTO recommendation of 1/500 (AASHTO 1989). It is likely that the live-load deflection will increase over time as the bar force declines and the partial composite action of the wearing surface decreases as a result of traffic wear and the deterioration of the attachment between the deck and the timber planks.

Condition Assessment

Condition assessments of the Connell Lake bridge performed at the conclusion of monitoring indicated that structural and serviceability performance were acceptable, although several serviceability deficiencies were noted. Inspection results for specific items follow.

Deck Camber

At installation, the deck was approximately straight between abutments. At the conclusion of monitoring, 2.5 in. of sag was measured, mainly as a result of the butt-joint spacing and partially as a result of creep. Sag has been more pronounced on stress-laminated decks designed with a 2-ft longitudinal butt-joint spacing, regardless of the lamination moisture content. Whereas, little or no sag has been observed in stress-laminated decks with a 4-ft butt-joint spacing when the moisture content of the laminations remained close to the FSP. Previous research has shown that creep causes a loss of camber when the moisture content level is high (Ritter and others 1990).

Wood Components

Inspection of the wood components of the bridge showed no signs of deterioration, although minor checking was evident on curb members. The checks did not appear to penetrate the preservative treatment envelope of the members. Inspection showed evidence of wood preservative loss with preservative accumulations on the wood surface as well as on the stream bottom (Fig. 18).

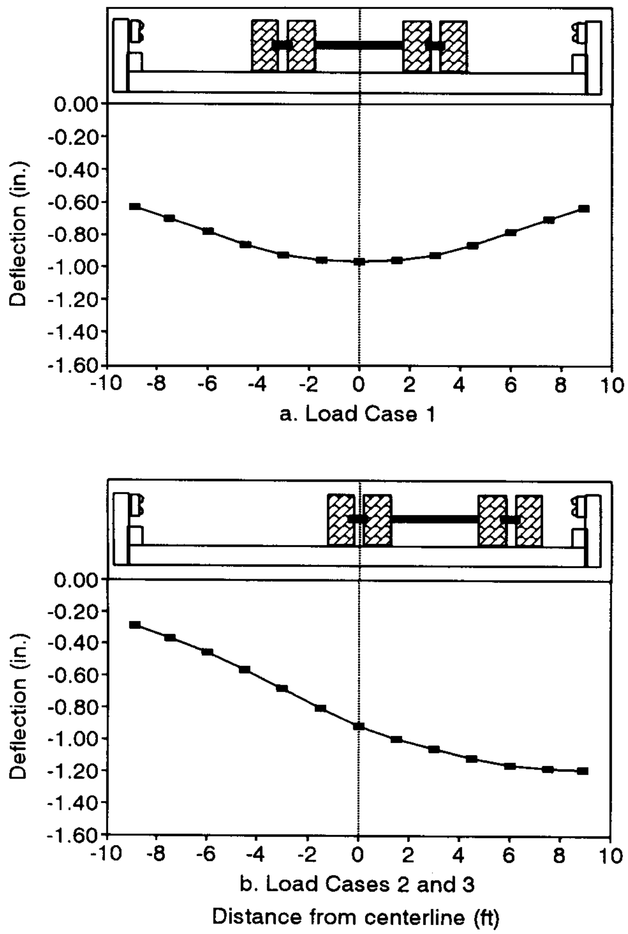

Figure 17—Theoretical maximum deflection under HS 20-44 load.

Figure 18—Excessive preservative leaching from laminations onto streambed.

Wearing Surface

Inspection of the timber plank wearing surface indicated minor plank wear and checking. Aside from these items, the wood planks were in good condition and showed no other signs of distress. A 2-in. layer of soil has been tracked onto the bridge deck and potholes have developed in the unpaved approach roadway at both bridge ends (Fig. 19). The soil on the bridge deck can trap moisture and inhibit drainage, which may lead to premature deterioration. The potholes increase vehicle impact to the bridge that can cause damage to the preservative deck treatment and timber components.

Figure 19—Soil on deck surface should be removed, and potholes along end of bridge should be filled.

Anchorage System

The exposed steel stressing bars and hardware showed no visible signs of corrosion, except at the ends of some stressing bars. At bar ends, minor corrosion appeared where the galvanized coating had been stripped from the bar, exposing uncoated steel, as previously seen (Wacker and Ritter 1992). The continuous bearing plate bar anchorage system is not performing as designed. It is anticipated that the continuous plate system was developed to alleviate the wood crushing problem observed in some other bridges by providing a larger bearing area than the discrete bearing plate system, but

Figure 20—Crushing of outside laminations, causing a wave distortion of the bearing plates.

at a lower cost than the channel bulkhead system (Ritter 1990). Inspection indicated that the portion of the bearing plates near the bars were crushing approximately 0.5 in. into the outside laminations, but no crushing was observed between the bars. This has caused a wave distortion of the bearing plates (Fig. 20). Crushing may have occurred as a result of insufficient bearing plate area for the strength of the exterior lamination, insufficient plate thickness for uniform force distribution, or reduced outer lamination strength because of the high moisture content level. It is probable that the crushing occurred after the initial bar tensioning and is not expected to continue unless the bars are retensioned.

Conclusions

After 2-1/2 years in service, the Connell Lake bridge is exhibiting good performance and has met the expectations of the bridge's owner. Construction of the bridge has successfully demonstrated that stress-laminated timber decks are a viable alternative for short-span structures in Alaska. In general, the bridge is performing very well and only minor serviceability deficiencies were noted during the monitoring period. These deficiencies should not adversely affect structural performance and can be easily remedied in future bridges. Although the bridge has been in place for only a short time, we anticipate that the bridge will have a long, useful life with minimal maintenance.

Based on the extensive monitoring conducted since bridge fabrication, we note the following observations and recommendations:

- Moisture content level has remained above the FSP. It is anticipated to remain high as a result of the high level of rainfall and relative humidity, cool temperatures, and lack of a waterproof surfacing. For this reason, minimal dimensional changes are expected. For future bridge designs, we recommend that the moisture content of the laminations be approximately 19 percent at installation.

- Stressing bar force has remained at an acceptable level during the 2-1/2 years of monitoring, with less than an 18-percent average decrease in bar force. The loss is due to stress relaxation of the wood and some crushing of the exterior laminations. We anticipate that restressing may need to be performed during the life of the structure. This is due to an expected interlaminar compression below the recommended 40 lb/in^2 from continued stress relaxation at the high moisture content level.

- Under static-load conditions, the measured deflection curves represented linear elastic orthotropic plate behavior, which is typical for stress-laminated decks. The maximum deflection was less than expected as a result of an increase in the effective deck moment of inertia from the partial composite action of the timber plank wearing surface. The theoretical maximum deflection of 0.96 in. for the HS 20-44 vehicle is within the maximum design live-load deflection of 1/360 of the center–center of bearings span length but is not within the current AASHTO recommendation of 1/500.

- A sag exists in the bridge's deck and is likely due to the butt-joint pattern and creep. This highlights the importance of a deck design that uses an alternative to the West Virginia University butt-joint pattern, such as one butt joint in every fourth transversely with 4-ft spacing between butt joints in adjacent laminations. In addition, a deck design should include camber to counteract the effect of moisture content resulting in creep.

- Wood checking is evident on the curb system and timber plank surfacing. Both are structurally sound with no penetration of the preservative treatment in the curb system.

- We anticipate that the 2-in. accumulation of soil on the deck will result in deterioration because of trapped moisture and lack of drainage, and the potholes along the deck ends will result in deck edge wear and impact loading. The soil should be removed from the deck surface and the potholes should be filled. Paving the approach roadways would prevent pothole creation and their adverse effects.

- The use of the continuous bearing plate bar anchorage system did not prevent crushing of the outside laminations. Further crushing may occur if the bars are retensioned.

- The ends of some stressing bars show signs of minor corrosion at locations where the galvanizing was removed during construction. This would not have occurred if the nuts had been oversized to compensate for the thickness of the galvanized coating or cold galvanizing was applied after construction.

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Appendix—Information Sheet

General

Name: Connell Lake Bridge

Location: Ketchikan, Alaska

Date of Construction: April 1991

Owner: USDA Forest Service, Tongass National Forest, Ketchikan Area

Design Configuration

Structure Type: Stress-laminated deck

Butt-joint Frequency: 1 in 5; 2 ft longitudinal

Total Length (out–out): 36.1 ft

Skew: None

Number of Spans: 1

Span Length (center–center bearings): 35.1 ft (as-built)

Width (out–out): 17.9 ft (as-built)

Width (curb–curb): 15.9 ft (as-built)

Number of Traffic Lanes: 1

Design Loading: AASHTO HS 20–44

Wearing Surface Type: 4- by 12-in. timber planks

Material and Configuration

Timber:

Species: Douglas Fir–Larch

Size (nominal): 14 by 4 in.

Moisture Condition: Greater than 30 percent at installation

Preservative Treatment: Creosote petroleum oil solution

Stressing Bars:

Diameter: 5/8 in.

Number: 17

Design Force: 28,000 lb; 83 lb/in^2 interlaminar compression

Spacing: 25 in.

Type: Galvanized, high strength, steel threaded bar with course right-hand thread, conforming to ASTM A 722

Anchorage Type and Configuration:

Steel Plates: 1/2 by 6 by 60 in. bearing, galvanized
1/2 by 6 by 10 in. bearing at bridge ends, galvanized
3/4 by 3 by 3 in. anchor, galvanized