Portable T-Section Glulam Bridge for Low-Volume Roads

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Abstract

Recent interest in portable bridge systems has increased as a result of a heightened awareness for reducing environmental impacts at road stream crossings. This paper describes the design and initial testing of a portable timber bridge consisting of two non-interconnected longitudinal glued-laminated timber (glulam) deck panels; each panel is 1.8 m (6 ft) wide and 12.2 m (40 ft) long. The deck panels are fabricated in a unique double-tee cross section. The panels exhibited linear elastic behavior and achieved 92% composite action under static bending tests. The complete bridge system appears to be cost effective with a superstructure cost of $\$381/m^2$ ($\$35/ft^2$). If the bridge is reused and installed at ten different sites, the estimated cost per site is $5,120, which is competitive with the cost of traditional stream crossing structures, such as fords or culverts.

Keywords: Portable bridge, Glued-laminated timber, Glulam, Bridge deck, Double-tee.

Introduction

Portable bridges have traditionally been used in military or construction applications. In typical civilian construction applications, portable bridges are used when a permanent highway bridge is being replaced and a temporary bypass is needed during the construction.

Portable bridges are also needed to serve as temporary structures during disaster situations, e.g. when a flood washes out a highway bridge. In addition, there are many situations where temporary access is needed across streams in remote areas for the construction or maintenance of utility structures.

Currently, much interest in portable bridge systems is occurring in the forestry and related natural resource industries. Access to our nation's forest resource requires an extensive roadway network over a wide spectrum of geographical conditions. In general, these roads are designed for low-volume traffic conditions and are often single lane and unpaved. Because forest management activities are both diverse and sporadic, traffic volumes and loads can vary significantly. During resource management periods, traffic volumes are low and consist primarily of light passenger vehicles. However, during forest harvesting operations, roadways may be subjected to higher-volume truck traffic with loads in excess of the maximum legal highway load. In either case, roadway use is commonly limited to short periods over a relatively long forest management cycle. For example, roadway access may be required for only 6 months during a 10-year cycle. As a result, there is a trend to close these roads when they are not needed for management activities.

Forest roads typically require a large number of structures to cross streams and other topographical features.

Rothwell (1983) and Swift (1985), in separate studies on forest roads and skid trails, found that stream crossings were the most frequent sources of erosion and sediment introduction into streams. Fords and corrugated-metal or concrete culverts have been common stream crossing structures on forest roads for many years. Thompson et al. (1996) reported that during the construction of a gravel ford on a stream approximately 1.8 m (6 ft) wide, peak sediment concentration in water samples taken downstream from the ford was nearly 2,810 mg/l greater than that of samples taken upstream from the ford. In addition to the disturbance caused during construction activities, using fords introduces sediment into the stream each time vehicles drive through the ford. Thompson et al. (1996) also showed that on the same ford, when light vehicular traffic drove through the stream, sediment concentration in water samples taken downstream from the ford was as much as 255 mg/l greater than that of the upstream samples.

Although some problems with fords are alleviated by culverts, there can be considerable sediment loads introduced into the stream during the excavation and fill work that accompanies culvert installation. Thompson (1996) reported that during the installation of a typical corrugated metal pipe culvert, sediment concentration in water samples taken downstream of the culvert was more than 950 mg/l greater than that of the upstream samples. Other results reported by Swift (1985) showed that the cumulative amount of soil placed in a stream at the road-stream crossing during construction was more than 10 times greater than the sedimentation during logging operations. In addition, culverts may clog with debris and wash out during heavy runoff periods, thereby introducing additional sediment into the stream. In the case of roads or skid trails that are not permanent, the stream crossing structure may be removed after logging operations or other activities are complete. Removal of a culvert also appears to introduce heavy sediment loads into the stream.

Historically, bridges for low-volume forest roads have been of two types: permanent or temporary. Permanent bridges, which are constructed of wood, steel, or concrete, depending on span requirements and economic considerations, are typically designed for service lives of 40 to 50 years. These permanent bridges are not economically feasible for short use periods and often require expensive maintenance for continued service. In addition, permanent bridges for limited-use, low-volume forest roads are commonly designed to a lower standard than most public access facilities and can be a potential liability to the bridge owner if public access is possible. A common temporary bridge has been the log stringer bridge that is either removed or left to deteriorate at the end of the use period. The use of temporary log stringer bridges has substantially declined over the last decade because of the difficulty in locating logs of the size and quality required for bridge construction. In addition, if the temporary bridge is not installed or removed properly, there may be adverse water quality impacts.

One solution to short-term bridge needs on low-volume forest roads is the concept of portable bridges. If properly designed and constructed, portable bridges can be easily transported, installed, and removed for reuse at multiple sites. This ability to serve multiple installations makes them much more economically feasible than a permanent structure. In addition, if they are installed and removed so that disturbance to the site is minimized, they can alleviate many water quality and other potential environmental problems. Thompson et al. (1995) reported that proper installation of a portable bridge could significantly reduce levels of sediment introduced into the stream compared with other crossings, such as fords and culverts.

Many advantages of timber bridges, which include using locally available materials, having long service lives, being relatively lightweight, and being prefabricated, make them ideal for portable stream crossings. The objective of this paper is to discuss the design and initial evaluation of a portable longitudinal glulam deck bridge. The bridge uses two non-interconnected panels that are fabricated in a unique double-tee cross section. Design, installation, and cost are discussed along with results of tests on the bridge components and the finished bridge.

Background

A variety of portable bridge designs have been constructed from steel, concrete, and timber, with steel and timber bridge designs being the most prevalent types (Mason 1990, Taylor et al. 1995). Although log stringer bridges and non-engineered timber mats or “dragline mats” have been used for many years, the recent advances in timber bridge technology include several engineered designs that can be easily adapted for use as portable bridges. Probably the most promising designs for spans up to 12 m (40 ft) consist of longitudinal glulam or stress-laminated decks that are placed across the stream. These designs can be quickly and easily installed at the stream crossing site using typical forestry equipment, such as hydraulic knuckleboom loaders or skidders. Also, it is possible to install these bridges without operating the equipment in the stream, which minimizes site disturbance and associated erosion and sediment load on the stream.

Hassler et al. (1990) discussed the design and performance of a portable longitudinal stress-laminated deck bridge for truck traffic on logging roads. This bridge was constructed of untreated, green mixed hardwoods. It was 4.8 m (16 ft) wide, 12.2 m (40 ft) long, 254 mm (10 in.) thick and fabricated in two 2.4-m- (8-ft-) wide modules. Taylor and Murphy (1992) presented another design of a portable stress-laminated timber bridge. It consisted of two separate stress-laminated panels 1.4 m (4.5 ft) wide placed adjacent to each other with a 0.6-m (2-ft) space between panels. The overall width of the complete bridge was 3.3 m (11 ft). The panels could be constructed in lengths up to 9.7 m (32 ft).

Taylor et al. (1995) presented the results of using a portable longitudinal glulam deck bridge designed for use by logging trucks and other forestry equipment. It was 4.9 m (16 ft) wide and 9.1 m (30 ft) long. It used four glulam deck panels, 1.2 m (4 ft) wide and 267 mm (10.5 in.) thick. The bridge was designed to be installed on a mud sill with the bridge deck extending 0.6 to 1.5 m (2 to 5 ft) on either side of the stream banks, thereby leaving an effective span of approximately 6.1 to 7.9 m (20 to 26 ft). They concluded that the bridge performance was satisfactory and that if it could be reused at least 10 times, its cost was comparable or less than the cost of installing fords or culverts. It had an initial cost of $15,500 and an estimated cost per site of approximately $2,550.

Keliher et al. (1995) described the use of another longitudinal glulam deck bridge designed specifically for log skidder traffic. This bridge consisted of two glulam panels 1.2 m (4 ft) wide, 216 mm (8.5 in.) thick, and 7.9 m (26 ft) long. The glulam panels were placed directly on the stream banks and were not interconnected. They were placed by using the grapple on skidders or winching into place with a skidder or crawler tractor. This bridge performed well in service and was well received by forest landowners and loggers that used it. However, its relatively high initial cost of approximately $8,000 may discourage some users from purchasing this type of bridge over the non-engineered designs frequently used for off-highway vehicles.

Design, Installation, and Cost

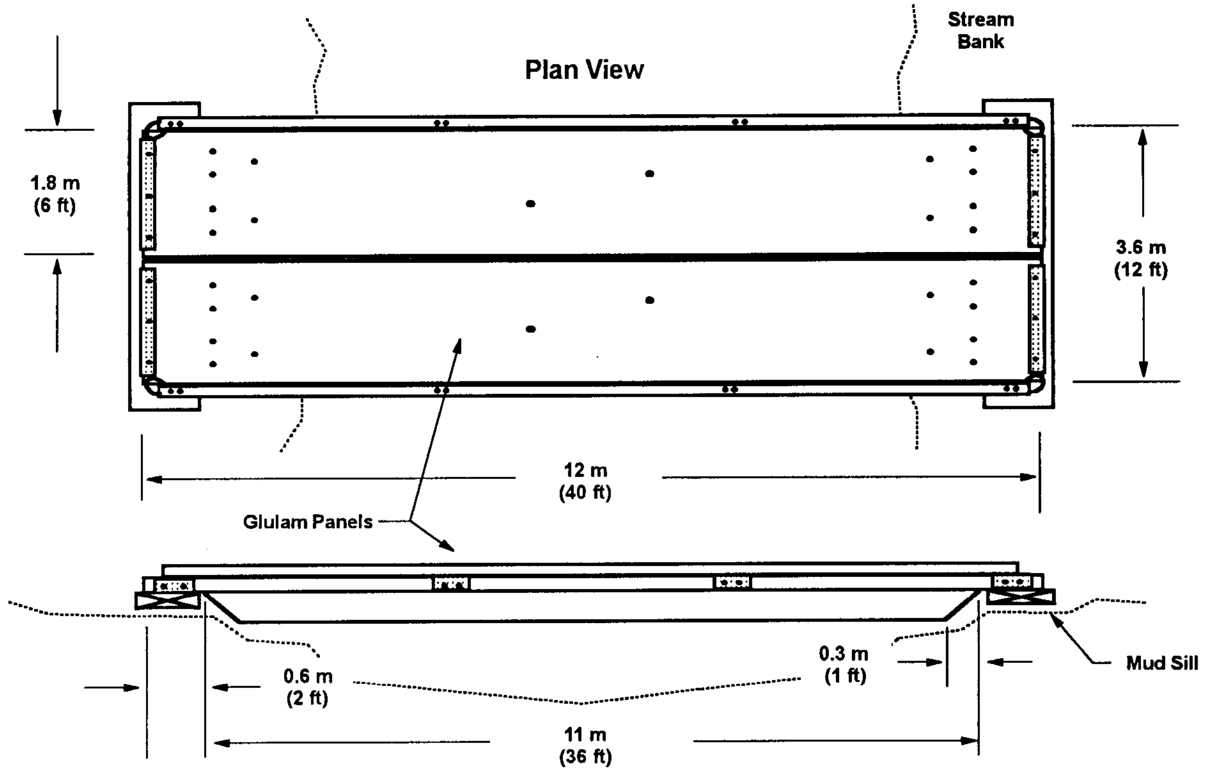
Design

The portable longitudinal deck timber bridge designs discussed previously have been limited to spans of approximately 9.1 m (30 ft) because of practical limitations on the thickness of the deck panels. However, there is a need for more efficient technology to allow the use of portable timber bridges on spans up to 15.2 m (50 ft). Therefore, a longitudinal glulam deck bridge was designed and constructed in a double-tee cross section to test the feasibility of achieving longer spans for portable bridges, while retaining the concept of a longitudinal deck bridge. This bridge was purchased by Georgia Pacific Corporation and designed to be used as a portable bridge carrying log trucks and other forestry equipment. The bridge was manufactured by Structural Wood Systems, Inc. of Greenville, Alabama.

The bridge consists of two longitudinal panels, 12.2 m (40 ft) long and 1.8 m (6 ft) wide, giving a total bridge width of approximately 3.6 m (12 ft) as shown in Figure 1. The design vehicle for the bridge was an American Association of State Highway and Transportation Officials (AASHTO) HS20 truck (AASHTO 1993) with no specified deflection limitation. The panels are not interconnected, therefore, each panel is assumed to carry one wheel line of the design vehicle. The panels were designed to be placed side by side on a mud sill, which can be placed directly on the stream banks. Each panel was constructed in a double-tee cross section (dimensions given in Figure 2). Vertically laminated flanges were 171 mm (6.75 in.) thick and 1.8 m (6 ft) wide and fabricated using No. 1 Southern Pine nominal 51- by 203-mm (2- by 8-m) lumber. Two 286-mm- (11.25-in-) wide and 314-mm- (12.375-in-) thick webs were horizontally laminated to the lower side of the flange. The webs were fabricated using Southern Pine nominal 51- by 305-mm (2- by 12-in.) lumber that met specifications for 302-24 tension laminations (AITC 1993). The designers did not necessarily intend that webs in future bridges of this type be constructed using all 302-24 lumber. However, the laminator had a large supply of lumber in this size and grade and therefore chose to use it in this prototype bridge. At the ends of the bridge panels, the flange extended 0.6 m (2 ft) beyond the end of the webs. This extension of the flange was intended to facilitate the placement of the bridge panel on a mud sill.

Interior diaphragms measuring 286 mm (11.25 in.) wide and 210 mm (8.25 in.) thick were provided between the webs at three locations along the length of the panels: one at each end and one at midspan. In addition, to provide additional strength in the weak axis of the flange, 25-mm- (l-in.-) diameter ASTM Grade 60 steel reinforcing bars were epoxied into the glulam flange and the diaphragms. The reinforcing bars were placed in holes drilled horizontally through the flanges at the panel third points. Additional reinforcing bars were placed horizontally through the diaphragms near the ends of the panels.

At each end of the panels, 19-mm- (0.75-in-) diameter bolts were installed through the horizontal axis of the flange. At the inside edge of the flange, a 152- by 152- by 13-mm (6- by 6- by 0.5-in.) steel plate was attached to the bolts. At the outside edge of the flange, a 305-mm (12-in.) long 152-by 152-by 13-mm (6- by 6- by 0.5-in.) steel angle was attached to the bolts. Chain loops were welded to the square plates and the steel angles to facilitate lifting



Plan View

Elevation View

Figure 1- Sketch of bridge installation showing overall dimensions of the portable longitudinal T-section glulam deck bridge.

of the panel ends and securing the panels at the site. The angles served as supporting brackets for a curb rail that extended the length of the bridge. Additional curb brackets were provided at third points along the outside edge of the flange. The curb rail consisted of a single 140-mm- (5.5-in-) deep, 127-mm- (5-in-) wide, and 11.6-m- (38-ft) long Southern Pine Combination 48 (AITC 1993) glulam beam, running the length of the bridge. For economic considerations, the curb rail was intended only for delineation purposes and not designed as a structural rail.

A wearing surface was not provided on the bridge. However, a 1.8-m (6-ft) long 152- by 102- by 13-mm (6- by 4- by 0.5-in) steel angle was attached with three 19-mm- (0.75-in-) diameter lag screws to the top face of the flange at each end of the bridge to prevent damage as vehicles drive onto the bridge. In addition, to prevent damage during installation of the bridge, a 6-mm- (0.25-in.-) thick steel plate was attached to the end of each web with 19-mm- (0.75-in-) diameter bolts. To facilitate lifting of the bridge panels, lifting eyes were placed 0.9 m (3 ft) from either side of the bridge panel midspan. These eyes consisted of a 51-mm (2-in.) inside diameter steel pipe with a 13-mm- (0.5-in-) thick steel plate flange welded to one end. The eyes were installed in holes drilled through the bridge deck flanges and attached using 19-mm- (0.75-in.-) diameter lag screws. The intent of the lifting eyes was to allow a chain or wire rope to be fed down through one eye and back up through the other eye to form a sling. Then, the ends of the chain or wire rope could be attached to a shackle or hook on a crane, loader, or backhoe. All steel plate, angles, lag screws, and bolts conformed to ASTM A36 or ASTM A307. A primer coat of paint was applied to all steel hardware before

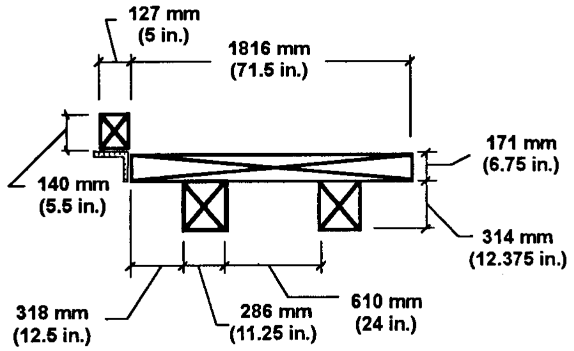


Figure 2--Cross-section view of longitudinal T-section glulam deck panels with curb rail attached. Diaphragms and connectors are omitted for clarity.

installation.

The steel hardware was installed on the finished deck panels before they were shipped from the laminating plant. The deck panels were then shipped to a treating facility where they were preservatively treated with creosote to 194 kg/m^3 (12 lb/ft^3) in accordance with American Wood Preservers' Association (AWPA) Standard C14 (AWPA 1991). The treating process had no detrimental effect on the steel hardware and did not affect preservative penetration or retention in the wood under the steel. The installation of hardware before shipping to the treating facility allowed the finished bridge to be installed with no further fabrication or assembly on the part of the bridge owner.

Installation

The bridge was installed for the first time on March 14, 1996, on land owned by Georgia Pacific Corp. near Newnan, Georgia. Installation was completed by personnel from Georgia Pacific Corp. and a local construction contractor that was hired to build the logging road into the site.

Before construction began, mud sills were prefabricated by personnel from Georgia Pacific Corp. The sills were 762 mm (30 in.) wide and 4.9 m (16 ft) long and were constructed from nominal 152- by 152-mm (6- by 6-in) Southern Pine timbers that were bolted together with 19-mm- (0.75-in-) diameter bolts. The timbers were preservatively treated with chromated copper arsenate (CCA).

Installation began by clearing the road approach to one side of the stream crossing with a crawler tractor. The contractor then used a tracked backhoe to unload the bridge panels from a truck and place them in a clearing approximately 46 m (150 ft) from the stream crossing. The backhoe was used to level each stream bank and then reach across the stream to place the first mudsill on the far side of the stream. At this point, the backhoe was used to carry the first bridge panel from the clearing to the stream and place it on the mud sill. A chain was placed through the lifting eyes on the bridge panel and secured in a hook on the bucket of the backhoe to lift and carry the panel as shown in Figure 3. The backhoe placed the second panel in a similar fashion. After the second panel had been placed, the second mud sill was pushed under the bridge panel ends on the near side of the creek. It was not necessary to operate any equipment in the stream during the installation. Therefore, since the stream channel was not disturbed, water quality was not impacted during the installation. Clearing the stream banks and placing the bridge panels was completed in approximately 2.5 hours. After the panels were in place, wire ropes were secured to the chain loops at each of the bridge corners and to nearby trees. This securing of the bridge required an additional hour. Additional time was also required to complete the final road approaches to the bridge. It is anticipated that removal of the bridge will be accomplished in a manner similar to the installation.

Cost

Cost for the materials, fabrication, treating, and shipping of the glulam bridge was $17,000. Based on a deck area of 44.6 m^2 (480 ft^2), the cost was approximately $381/m^2 ($35/ft^2). The cost for the mud sills was $600. The cost for labor and equipment to install the bridge was $1,680. Therefore, the total cost to install this bridge the first time was $19,280. Assuming removal costs are similar to installation costs, the projected total cost to install and remove the bridge at 10 different sites is approximately $33,600. When this is added to the initial cost of the bridge and mud sill, the total cost to install the bridge at 10 sites is $51,200 or $5,120 per site. This cost per site is competitive with the cost of installing permanent culverts or fords on the larger streams where this bridge will be used.

Bridge Evaluation Methodology

The monitoring plans for the bridge called for stiffness testing of the individual lumber laminations prior to the fabrication of the deck panels and the completed glulam deck panels after fabrication. In addition, static load test behavior and general bridge condition were assessed. These evaluation procedures are discussed in the following sections.

Lamination and Finished Panel MOE

Modulus of elasticity (MOE) tests were performed at the laminating plant prior to fabrication of the deck panels to determine the stiffness of each lumber specimen used in the flanges and webs. These tests were conducted using commercially available transverse vibration equipment. During the tests, an identification number and the MOE was placed on each lumber specimen to facilitate resorting the lumber at a later time.

After fabrication of the deck panels, static bending tests were conducted to determine the apparent MOE values of each panel. These bending tests were conducted using a testing frame at the laminating plant and consisted of applying a single point load at the center of each deck panel. A steel beam was used to distribute the load across the width of the flange. The panels were placed in the test jig as they would be installed in the field, i.e., the bearings

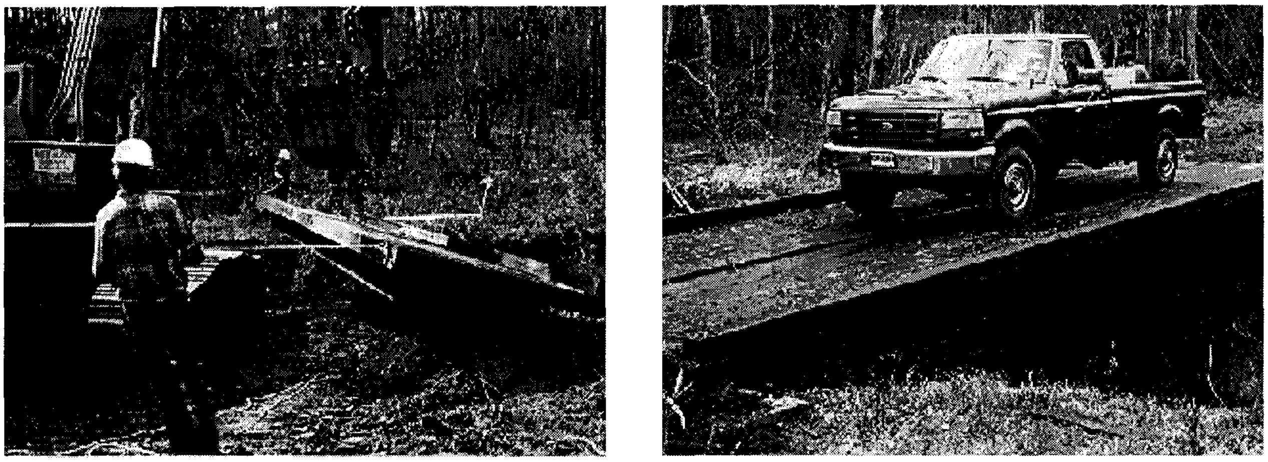


Figure 3-Bridge panels being lifted into place (left) and the completed bridge installation (right).

were placed under the flange overhangs at the end of the bridge immediately adjacent to the end of the webs. This resulted in a test span (from center of bearing to center of bearing) of 11.28 m (37 ft).

During testing, deflection readings were taken with dial gages and linear-variable displacement transducers (LVDTs) at several locations along the length of the panels: at each bearing, approximately 610 mm (24 in.) from each bearing, and at midspan. Force was applied to the panel using a hydraulic cylinder and measured by a load cell placed between the hydraulic cylinder and the deck panel. During the tests, the force was steadily increased to approximately 66.72 kN (15,000 lb) with deflection readings taken at 11.12 kN (2,500 lb) intervals. The maximum force used in the tests resulted in a bending moment approximately 70% of the design moment. Deflection readings were recorded to the nearest 0.025 mm (0.001 in.). These force and deflection data were then used to calculate the apparent static bending MOE of the deck panels.

Analytical Assessment of Bridge Panel MOE

At the conclusion of the stiffness testing of the lumber, a target "E-rated" layup was developed for the flanges and webs. This layup, which is shown in Figure 4, consisted of five different lumber groups. The MOE values shown in Figure 4 represent the target mean MOE of the lumber used in the flanges or the various laminations of the webs. Personnel in the laminating plant were able to sort the lumber into the different MOE classes and place the lumber laminations in the desired panel locations during the manufacturing process. The identification numbers for the boards used in the flanges and the identification numbers and locations of each board used in the webs were recorded during the fabrication process. These data were then used as input for a transformed section analysis computer program developed at FPL, which included an estimate of the effect of shear deflection. Using lumber data, the program was used to predict the MOE of the finished deck panels for comparison with bending test results.

Load Test Behavior

Static load testing was conducted June 4, 1996, approximately 3 months after bridge installation. The test consisted of positioning a fully loaded truck on the bridge deck and measuring the resulting deflections at a series of transverse locations at midspan and the abutments. Deflection measurements were taken prior to testing (unloaded), for each load case (loaded), and at the conclusion of testing (unloaded).

The load test vehicle consisted of a fully loaded tandem-axle dump truck with a gross vehicle weight of 161.1 kN (36,220 lb) and a track width at the rear axles of 1,829 mm (72 in.) (Figure 5). Measurement of wheel line loads indicated that the right side of the rear axles was approximately 4.4 kN (1,000 lb) heavier than the left side. The vehicle was positioned longitudinally on the bridge so that the two rear axles were centered at midspan. This resulted in maximum bending moments approximately 55% of the design moment. Transversely, the vehicle was placed for four load cases as shown in Figure 6. For load cases 1 and 3, the vehicle wheel line was positioned directly over the panel outside web. For load cases 2 and 4, the vehicle was positioned with the truck wheel line over the flange centerline at the center of the panel width. Measurements of bridge deflection from an unloaded to loaded condition were obtained by placing calibrated rules

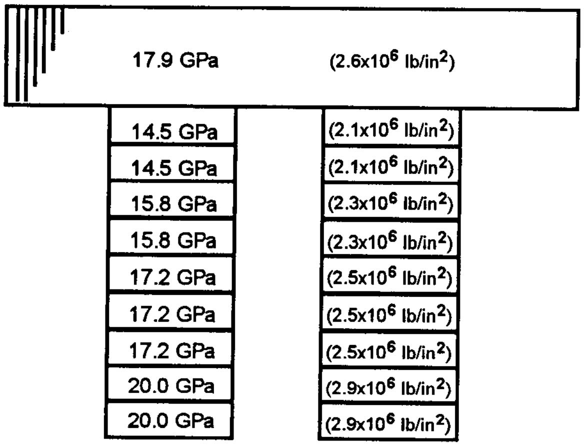


Figure 4-Theoretical distribution of lumber MOE classes in the T-section glulam deck panels.

on the underside of the deck, and at the abutments and reading values with a surveyor's level to the nearest 0.2 mm (0.01 in.).

Condition Assessment

The general condition of the bridge was assessed at the time of the first load test. This assessment involved visual inspection of the bridge components, measuring moisture content of the wood members with a resistance-type moisture meter, and photographic documentation of bridge condition. Items of specific interest included the condition of the top surface of the deck panel flanges, the bottom face of the webs, the curb system, and anchorage systems.

Results and Discussion

The performance monitoring of the bridge is still in its initial stages and will continue for 2 years. Results and discussion of the initial performance data follow.

Lamination and Bridge Panel MOE

The lumber used to fabricate the flanges was nominal 50 by 203-mm (2- by 8-in.) No. 1 Southern Pine. Results of MOE tests on this lumber prior to gluing indicated that it had a mean flatwise MOE of 18,130 MPa (2.63 x 10^6 lb/in^2) with a coefficient of variation of 17.3%. The flatwise MOE can be converted to an edgewise value by applying a flatwise adjustment factor of 0.965 (Williams et al. 1992). This resulted in a mean edgewise MOE of 17,510 MPa (2.54 x 10^6 lb/in^2).

The lumber used to fabricate the webs was nominal 50-by 305-mm (2- by 12-in) Southern Pine graded at the laminating plant to meet the specifications of 302-24 tension laminations (AITC 1993). Results of MOE tests on this lumber prior to gluing indicated that it had a mean flatwise MOE of 16,960 MPa (2.46 x 10^6 lb/in^2) with a coefficient of variation of 12.5%.

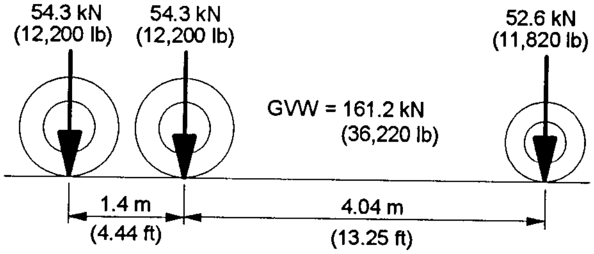


Figure 5-Load test truck configuration and axle loads. The transverse vehicle track width, measured center-to-center of the rear tires, was 1.8 m (6 ft).

Bending test data were used to calculate the MOE of the two finished bridge deck panels. For panel 1, the MOE was 16,340 MPa (2.37 x 10^6 lb/in^2). For panel 2, the MOE was 15,860 MPa (2.30 x 10^6 lb/in^2). Based on the force-deflection plots from these tests, the deck panels appeared to exhibit linear elastic behavior up to the maximum loads used in the tests.

Analytical Assessment of Bridge Panel MOE

Data for the location of each board and its corresponding MOE were used as input to the transformed section program. This analysis assumed that there was complete composite behavior in the double-tee deck panel and that the cross section of the panel was uniform along the entire span. The latter assumption was not entirely accurate since the webs were tapered near their ends. Based on the transformed section analysis of the original target E-rated layup, the theoretical predicted MOE for the deck panels was 17,860 MPa (2.59 x 10^6 lb/in^2). When the actual lumber MOE data were used in the transformed section analysis, the predicted MOE for Panel 1 was 17,720 MPa (2.58 x 10^6 lb/in^2) versus an actual MOE of 16,341 MPa (2.37 x 10^6 lb/in^2). For Panel 2, the predicted MOE was 17,250 MPa (2.50 x 10^6 lb/in^2) versus an actual MOE of 15,860 MPa (2.30 x 10^6 lb/in^2). The difference between the actual MOE and the predicted MOE indicates that the actual beam stiffness was approximately 92% of the theoretical stiffness (which assumes a uniform cross section along the entire span). This is primarily due to the tapered webs at the ends of the actual deck panels. It is also possible that there was a loss in stiffness due to shear lag in the flange. Additional experimental testing and analysis will investigate this further in order to optimize the efficiency of the system.

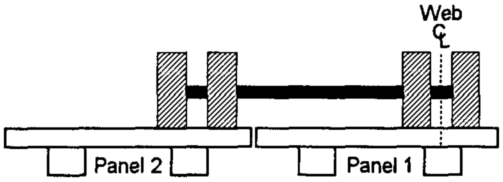
Load Test Behavior

Transverse load test deflections are shown in Figure 7, as viewed from the south end (looking north). For each load test, no permanent residual deformation was measured at the conclusion of the testing. In addition, there was no detectable movement at either of the abutments. For load cases 1 and 3, the symmetry of loading resulted in deflection profiles that are approximately mirror images of one another and deflection differences of corresponding data points for the two positions are within approximately 1 mm (0.04 in.). Maximum deflections for these load cases occurred in panel 1 and measured 16.2 mm (0.64 in.) at the outside panel edge for load case 1 and 16.5 mm (0.65 in.) at the interior panel edge for load case 3. It is probable that the maximum deflection for load case 1 occurred at the interior edge of panel 2; however, deflections at that point were not measured. The greater deflections recorded at the outside panel edges are expected because the truck was loading the flange about the weak axis.

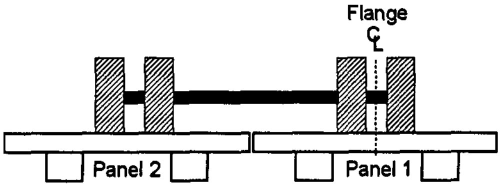
For load cases 2 and 4, deflections are nearly identical and differences at corresponding data points for the two load cases are within 1 mm (0.04 in.). With the wheel line centered on panel 1 for load case 2, the approximately uniform load distribution across the panel width results in similar deflection at each data point. For load case 4, it was anticipated that the panel 2 deflections would also be uniform and approximately equal those for panel 1, load case 2. The approximate 2.5 mm (0.10 in.) difference in the load case 4 web deflections for panel 2 is likely due to minor differences in the truck transverse position. The maximum deflection recorded for load case 4 corresponds to a deflection value of approximately L/975, at 55% of design bending moment.

Condition Assessment

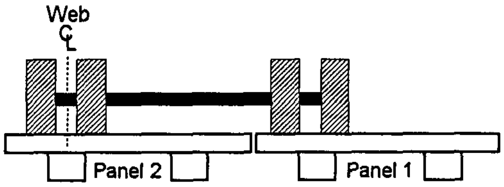
At the time of the first load test, limited traffic ,had crossed the bridge. Therefore, there was little overall change from the bridge's original condition. There was a small amount of damage to the outer face of the tension lamination of one web; however, the damaged area did not appear to significantly reduce the structural adequacy of the bridge. This apparently occurred during preparation for installation when the panel was dragged on the ground. The small amount of overall damage may be attributed to the use of the lifting eyes, which eliminated the need for the construction crew to wrap chains or cables around any exposed wood surfaces. Some surface checking was noticed on the top surface of the flange, but it did not appear to affect the structural adequacy of the flange. There were locations where excess creosote had accumulated on the top surface of the flange.



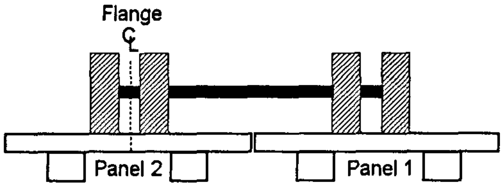
Load Case 1



Load Case 2



Load Case 3



Load Case 4

Figure &Transverse load positions (looking north) and deck panel numbers for the load test. For all load cases, the two rear axles were centered over the bridge centerspan.

Concluding Remarks

Based on initial testing, the longitudinal T-section glulam deck bridge is performing well and should provide acceptable service as a portable logging bridge. The following specific conclusions can be made at this time:

- It appears to be feasible and practical to construct a longitudinal glulam deck panel as a double-tee.
- The total time to install the bridge was less than 3.5 hours. Installation was easily accomplished using common construction equipment, and because the stream channel was not disturbed, water quality was not impacted during construction activities.
- The cost of the bridge superstructure was $\$381/m^2$ ($\$35/ft^2$), which is competitive with other timber bridge superstructure systems. The estimated cost for

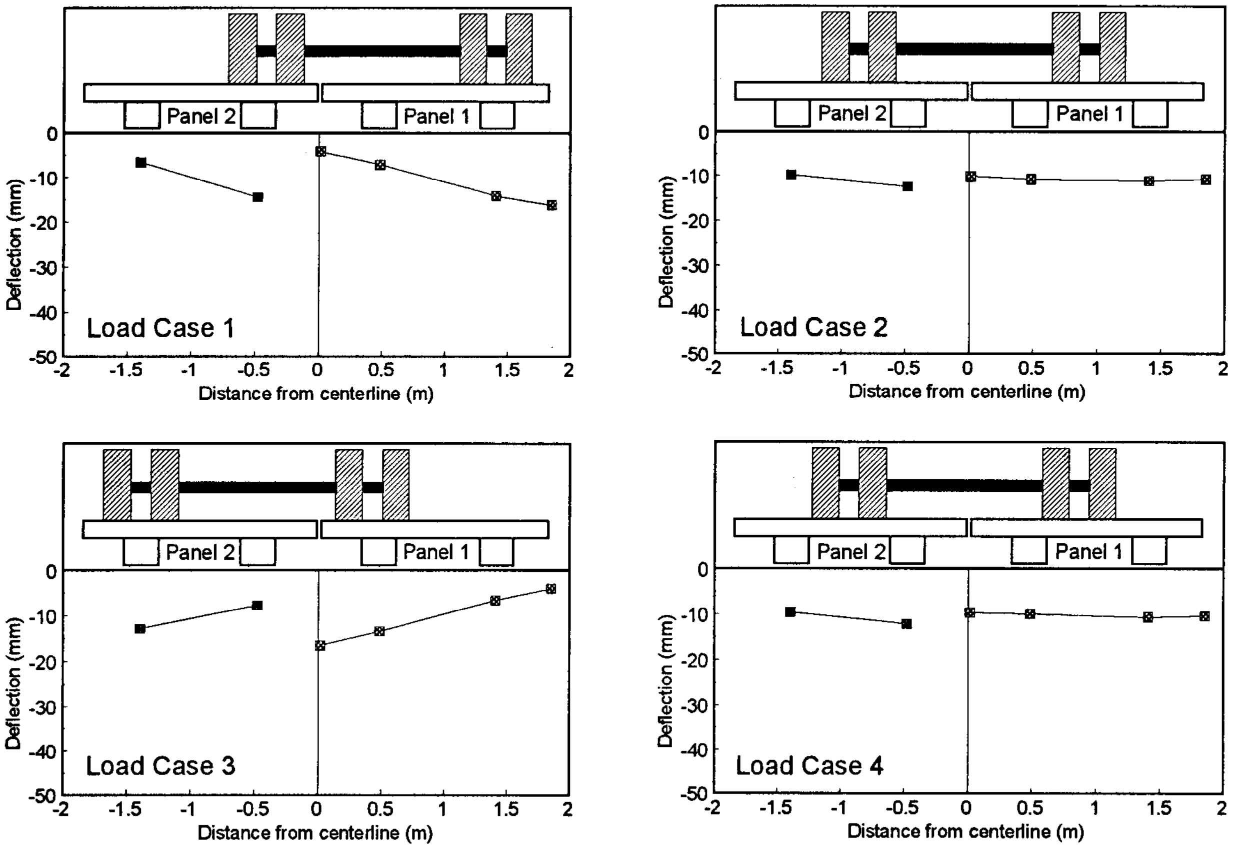


Figure 7-Transverse deflection for the load test of the longitudinal T-section glulam deck bridge measured at the bridge centerspan (looking north). Bridge cross-section and vehicle positions are shown to aid interpretation and are not to scale.

installation and removal of the bridge at 10 different sites was $5,120 per site, which is competitive with other traditional stream crossing structures on similar size streams.

- Static bending test results indicated that the T-section glulam deck exhibited linear elastic behavior when subjected to loads approaching the design load. The stiffness of the deck panels was approximately 92% of the theoretical stiffness, based on results from a transformed section analysis that assumed a uniform cross section along the entire span.
- Results from load tests indicated that the T-section glulam deck exhibited acceptable levels of deflection. The maximum midspan deflection recorded when the truck wheel line was positioned near the center of the panel was equivalent to L/975 at 55% of design bending moment.
- Minor damage to one tension lamination apparently occurred during the installation process. Also, minor checking was observed on the surface of the flange. Long-term observations are needed before determining if these factors will affect the structural adequacy of the deck panels.

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