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T-Section Glulam Timber Bridge Modules: Modeling and Performance

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Summary:

This paper describes the design, modeling, and testing of two portable timber bridges, each consisting of two non-interconnected longitudinal glued-laminated timber (glulam) deck panels 1.8 m (6 ft) wide. One bridge is 12.2 m (40 ft) long while the other bridge is 10.7 m (35 ft) long. The deck panels are fabricated in a unique double-tee cross section. The bridges exhibited linear elastic behavior in static bending tests. When actual lumber property data were used in transformed section and finite element models, the models overpredicted the stiffness of the bridge panels. The transformed section results provided the best agreement with test results. The bridges were cost effective and although some damaged occurred to the bridge modules during their removal, they have functioned well overall.

Keywords:

Portable bridge, glued-laminated timber, glulam, bridge deck, tee-section, forest road

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ABSTRACT

This paper describes the design, modeling, and testing of two portable timber bridges, each consisting of two non-interconnected longitudinal glued-laminated timber (glulam) deck panel modules 1.8 m (6 ft) wide. One bridge is 12.2 m (40 ft) long while the other bridge is 10.7 m (35 ft) long. The modules are fabricated in a double-tee cross section. The bridge modules exhibited linear elastic behavior in static bending tests. When actual lumber property data were used in transformed section and finite element models, the models overpredicted the stiffness of the bridge panels. The transformed section results provided the best agreement with test results. The bridges were cost effective and although some damage occurred to the bridge modules during their removal, they have functioned well overall.

Keywords: Portable bridge, glued-laminated timber, glulam, bridge deck, tee-section, forest road.

INTRODUCTION

Portable or temporary bridges have been used traditionally 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 period. Also, portable bridges are 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. This interest is primarily the result of efforts to reduce environmental impacts from forest road construction. Rothwell (1983) and Swift (1985), in separate studies on forest roads, found that stream crossings were the most frequent sources of erosion and sediment introduction into streams. Bridges, culverts, and fords are the common stream crossing structures on forest roads. Of these different stream crossing structures, several studies have shown that proper installation of a portable bridge could significantly reduce levels of sediment introduced into the stream compared to other crossings such as fords and culverts (Taylor et al., 1999). 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.

The objective of this paper is to discuss the design, modeling, and testing of a new type of portable longitudinal glued-laminated timber (glulam) deck bridge system. The bridge system uses two non-interconnected modules that are fabricated in a unique double-tee cross section. The paper will present a comparison of stiffness results from field tests of the bridge modules to stiffnesses predicted by transformed section and finite element models. Bridge performance also will be discussed.

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 a1.,1995). Log stringer bridges and non-engineered timber mats or “dragline mats” have been used for many years. However, the recent advances in timber bridge technology include several engineered designs that can be easily adapted for use as portable bridges. Probably the most

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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 was 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 spread footing, 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 to 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 8 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 by 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 cost of $8,000 was slightly higher than the steel skidder bridge described by Weatherford (1996), which cost approximately $7,200. This cost difference may discourage some users from purchasing this type of bridge over the non-engineered designs frequently used for off-highway vehicles.

Taylor and Ritter (1996) and Taylor et al. (1997) described initial results of using the T-section glulam bridges described in this paper. Further modeling and performance results are provided next.

LONGITUDINAL T-SECTION GLULAM BRIDGE MODULES DESIGN, INSTALLATION, AND COST

Design

The portable longitudinal deck timber bridge designs discussed previously have been limited to spans of approximately 9 m (30 ft) due to 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 m (50 ft). Therefore, two longitudinal glulam deck bridges have been 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. Taylor et al. (1996), Taylor and Ritter (1996), and Taylor et al. (1997) described initial results of using the T-section bridges discussed here. This paper will provide further details on modeling and performance of the bridge modules.

The first bridge was purchased by Georgia Pacific Corporation and was designed to be used as a portable bridge carrying log trucks and other forestry equipment in company timber harvesting operations. The bridge consists of two longitudinal panels 12 m (40 ft) long and 1.8 m (6 ft) wide giving a total bridge width of approximately 3.6 m (12 ft). The second bridge was purchased by the Morgan County, Alabama Forestry Planning Committee and was also designed for traffic similar to the first bridge. The bridge was intended to be used in demonstrations for timber harvesting Best Management Practices. The second bridge was identical in width to the first bridge; however it was 10.7 m (35 ft) long. Both bridges were manufactured by Structural Wood Systems, Inc. of Greenville, Alabama. Figure 1 is a sketch of the bridges.

The design vehicle for both bridges 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 spread footing, which can be placed directly on the stream banks. Each panel was constructed in a double-tee cross section with dimensions given in Figure 2. Vertically-laminated flanges were 171 mm (6.75 in.) thick and 1.816 m (71.5 in.) wide and were fabricated using No. 1 Southern Pine nominal 50 by 203 mm (2 by 8 in.) 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 50 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 for 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 spread footing.

To manufacture the bridge panels, the personnel at the laminating plant began by fabricating the flanges as one piece. Then, the flanges were surfaced and prepared for attachment of the webs. To prepare for clamping the web laminations to the flanges after gluing, vertical holes were predrilled through the flanges and webs. The holes were 25 mm (one in.) in diameter and were spaced approximately 305 mm (12 in.) apart along the length of the webs. At this point, adhesive was applied to each lamination and the laminations were placed on the flanges. Threaded steel rods were placed through the holes, then clamping blocks and nuts were placed on the ends of the rods. The nuts were then tightened and the adhesive was allowed to cure. After the gluing operation, no further surfacing of the flanges or webs was conducted.

Interior diaphragms measuring 286 mm (11.25 in.) wide and 210 mm (8.25 in.) thick were placed 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 (1 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 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. The curb rail was intended only for delineation purposes and was 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 eye 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 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 preservative 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. Also, the steel hardware did not affect preservative penetration or retention in the wood. 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 owners.

Installation

Georgia Pacific Bridge

The bridge owned by Georgia Pacific Corporation was installed for the first time on March 14, 1996 near Newnan, Georgia. Since that time, it has been installed two more times near Reynolds, Georgia. Installations were completed by personnel from Georgia Pacific Corp. and a local construction contractor that was hired to install the bridge.

Before construction began, spread footings were prefabricated by personnel from Georgia Pacific Corp. The footings consisted of sills that 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 preservative treated with Chromated Copper Arsenate (CCA).

A typical installation begins by clearing the road approach to one side of the stream crossing with a crawler tractor. The contractor then uses an excavator or backhoe to unload the bridge panels from a truck and place them in a staging area near the stream crossing. The backhoe is used to level each stream bank and then reach across the stream to place the first sill on the far side of the stream. At this point, the backhoe carries the first bridge panel from the staging area to the stream and places it on the sill. A chain or cable is 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. The backhoe places the second panel in a similar fashion. After the second panel has been placed, the second sill can be pushed under the bridge panel ends on the near side of the creek. At most sites, it is not necessary to operate any equipment in the stream during the installation. Therefore, since the stream channel is not disturbed, there will be minimal water quality impacts during the installation. At the site near Reynolds, GA, the distance between the top edges of the stream banks was slightly wider than the length of the bridge. Therefore, supplementary footings were constructed by placing rip rap on the sides of the stream banks. The timber sills were then placed on the rip rap. On these wider streams, the backhoe can be placed in the center of the stream channel and then used to lift the bridge panels into place.

Clearing the stream banks and placing the bridge panels has been completed in an average time of 2 hours. After the panels are in place, wire ropes are secured to the chain loops at each of the bridge comers and to nearby trees to prevent movement of the bridge during flood events. This securing of the bridge requires an additional. hour. Additional time is then required to complete the final road approaches to the bridge. Removal of the bridge is accomplished in a manner similar to the installation and has required an average time of two hours.

Morgan County Bridge

The Morgan County bridge was installed on August 26, 1996 near Moulton, Alabama on a demonstration forest site owned by Champion International Corporation. It has not been moved since the original installation. Installation was completed by personnel from a road construction contractor and Champion International. Sills to be used as spread footings similar to those described for the Georgia Pacific bridge were constructed for the Morgan County bridge.

The bridge panels were unloaded at a staging area approximately 0.8 km (0.5 miles) from the stream site before the road was constructed to the stream crossing. Once the road was cleared, the contractor pulled the bridge panels to the site using a crawler tractor. The first panel was pulled as close as possible to the edge of the stream bank before the tractor was unhooked from the panel. Then the tractor was positioned behind the panel and small logs were placed on the ground in front of the panel (perpendicular to the direction of travel). Also, a small log approximately 5 m (15 ft) long was placed on the opposite stream bank (parallel to the direction of travel) with its base in the stream channel and its top near the top of the stream bank. The tractor then pushed the panel toward the stream channel. The logs laying perpendicular to the panel were used to help the bridge panel roll toward the stream crossing and then once the forward end of the panel tipped downward into the stream channel, the panel slid up the log laying parallel to the panel. After the panel was in place, a chain was attached to the log that was laying against the stream channel; then the chain was attached to the tractor and the tractor pulled the log out from under the bridge panel. The second panel was pushed into place using the same procedure. At this point, workers attached a chain to the ends of each panel and to the blade on the crawler tractor. Then the tractor picked up the panels and made final adjustments in their position to achieve the correct alignment of the panels.

After the panels were in place, the sills were pushed under the panel ends by the crawler tractor. Then, wire ropes were secured to the chain loops at each bridge comer and then to nearby trees. The total time to install the bridge was three hours. It is anticipated that removal can be accomplished in a similar manner.

Cost

Georgia Pacific Bridge

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 sills was $600. The average cost for labor and equipment to install and remove the bridge was $1,000. This cost includes $540 for the excavator cost, $300 for trucking costs, and $160 for additional labor. Therefore, the total cost to install this bridge one time was $18,600. The projected total cost to install and remove the bridge at 10 different sites is $10,000. When this is added to the initial cost of the bridge and mud sill, the estimated total cost of the bridge system, distributed over 10 sites, is $27,600 or $2,760 per site. For the larger size streams where this bridge is being used, this cost will be less than the cost of installing traditional fords or culverts.

Morgan County Bridge

Cost for the materials, fabrication, treating, and shipping of the glulam bridge was $14,000. Based on a deck area of 39.0 m^2 (420 ft^2), the cost was approximately $359/m^2 ($33/ft^2). The cost for the sills, cable and associated hardware was $825. The cost for labor and equipment to install and remove the bridge was $1095. Therefore, the total cost to install and remove this bridge the first time was $15,920. The projected total cost to install and remove the bridge at 10 different sites is approximately $10,950. When this is added to the initial cost of the bridge and the sills, the estimated total cost of the bridge system, distributed over 10 sites, is $25,775 or $2,578 per site. This cost per site is very competitive with the cost of installing permanent culverts or fords on the larger streams where this bridge will be used.

BRIDGE EVALUATION AND MODELING METHODOLOGY

The monitoring plans for the bridges 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. Modeling procedures involved comparing stiffness results predicted by transformed section and finite element models to stiffness results from static tests of the bridge modules. These evaluation and modeling procedures are discussed in the following sections.

Lamination and Finished Bridge Deck Panel Stiffness Tests

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 was complete, static bending tests were conducted to determine the apparent MOEs 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 were intended to be installed in the field; i.e., the bearings were placed under the flange overhangs at the end of the bridge immediately adjacent to the end of the webs. This resulted in test spans (from center of bearing to center of bearing) of 11.28 m (37 ft) and 9.8 m (32.25 ft) for the Georgia Pacific and Morgan County bridges, respectively.

During testing, deflection readings were taken with dial gages and LVDT's 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 panels using a hydraulic cylinder and was measured by a load cell placed between the hydraulic cylinder and the deck panels. During the tests, the force was steadily increased to approximately 66.72 kN (15,000 lbs) 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% and 77% of the design moments for the Georgia Pacific and Morgan County bridges, respectively. 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 MOEs and stiffnesses of the deck panels.

Analytical Assessment of Bridge Panel Stiffness

At the conclusion of the stiffness testing of the lumber, a transformed section analysis was used to develop a target "E-rated" layup for the flanges and webs. This lay-up, which is shown in Figure 3, consisted of 5 different lumber groups. The MOE values shown in Figure 3 represent the target mean MOE of the lumber used in the flanges or in 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.

After fabrication of the bridge panels, the lumber stiffness and location data were then used as input for two different analytical models: 1) a transformed section analysis computer program developed at the USDA Forest Products Laboratory, and 2) a finite element model developed using the commercially available software package Algor®. The models were used to predict the stiffnesses of the finished deck panels for comparison with static bending test results.

In the transformed section analysis, the lumber properties data were input for consecutive 610 mm (24 in.) long segments along the length of the panels. Actual lumber stiffness data were input for each lamination in the webs. For the flange properties, the mean MOE of the lumber used in each flange was used in the analyses. The analyses were conducted for a single point load applied at midspan (to correspond to static bending tests described earlier). The model calculated various section properties, stiffnesses and MOE values, and bending stress levels as a function of applied load.

In addition to the transformed section analysis, finite element models were created for each bridge panel. These models were based on the same actual lumber data used in the transformed section analysis. Since the MOE of the lumber varied throughout the cross section, the models may be referred to as "non-homogeneous." The finite element models consisted of a mesh of three-dimensional brick elements arranged to mimic the actual bridge panels. The finite element models were used to model bridge deflection as well as to determine the apparent stiffness and MOE of each bridge panel. For bridge deflection, the models for each bridge panel were loaded with the same center point load used during each static bending test and the bearing conditions for each test were imposed upon the models. For stiffness determination, a theoretical center point load was imposed upon the bridge panel models and also on similar "homogeneous" models of each bridge panel. The homogeneous model was constructed with a uniform MOE over the entire cross section. The MOE of the homogeneous model was adjusted until the deflections of both the non-homogeneous model and the homogeneous models matched. At that point, the value of MOE in the homogeneous model was considered equal to the apparent modulus of elasticity of the bridge panel predicted by the non-homogeneous finite element model.

Field Load Test Behavior

Georgia Pacific Bridge

A static field test of the Georgia Pacific bridge was conducted on June 4, 1996, approximately 3 months after first installation of the bridge. 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 as close as possible to the footings. Deflection measurements were taken prior to testing (unloaded), for each load case (loaded), and at the conclusion of testing (unloaded). Measurements of bridge deflection from an unloaded to loaded condition were obtained by placing calibrated rules on the deck underside and at the bridge footings and reading values with a surveyors level to the nearest 0.2 mm (0.01 in.).

The load test vehicle consisted of a fully loaded tandem-axle dump truck with a gross vehicle weight of 161.l kN (36,220 lb) and a track width at the rear axles of 1830 mm (72 in.) (Figure 4). 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 5. 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.

Morgan County Bridge

A static field test of the Morgan County bridge was conducted on September 10, 1998, which was nearly two years after installation. Although dynamic tests were also conducted on this bridge, their results will be presented in a separate paper. The static test consisted of positioning a loaded truck on the bridge deck and measuring the resulting deflections at a series of transverse locations at midspan and as close as possible to the footings. Deflection measurements were taken prior to testing (unloaded), for each load case (loaded), and at the conclusion of testing (unloaded). Measurements of bridge deflection from an unloaded to loaded condition were obtained by placing Celesco Model PT101 direct current displacement transducers (DCDT's) on the deck undersides at midspan and at the footings. Data from the DCDT's were recorded through an electronic data acquisition module, which was in turn connected to a laptop computer. Due to a limited number of DCDT's, only one bridge module at a time was instrumented with DCDT's. Therefore, each panel was tested separately.

The load test vehicle consisted of a loaded tandem-axle flatbed truck with a gross vehicle weight of 172.2 kN (38,720 lb) and a track width at the rear axles of 1830 mm (72 in.) (Figure 4). 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 52% of the design moment. Transversely, the vehicle was placed for three load cases on each panel as shown in Figure 6. For Load Cases 1 and 3, the vehicle wheel line was positioned directly over the panel outside and inside webs, respectively. For Load Case 2, the vehicle was positioned with the truck wheel line over the flange centerline at the center of the panel width. Although these tests were conducted in a slightly different manner than those of the Georgia-Pacific bridge, the net effects on the bridge modules are essentially the same.

Condition Assessment

The general condition of the Georgia Pacific bridge was assessed at the time of the first load test on June 4, 1996, and on November 15, 1996, April 2, 1997, and July 18, 1997. The condition of the Morgan County bridge was assessed on July 31, 1997 and November 13, 1998. These assessments involved visual inspection of the bridge components, measurement of moisture content of the wood members with a resistance-type moisture meter, and photographic documentation of bridge condition. During the inspection of the Morgan County bridge conducted in 1998, core samples were taken for subsequent laboratory determination of wood moisture content. 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 bridges is nearly complete. Results and discussion of the modeling and performance data follow.

Lamination and Bridge Panel Stiffness

Test results for the lumber are summarized in Table 1. 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 the lumber used in the Georgia Pacific bridge panel flanges prior to gluing indicated that it had a mean flatwise MOE of 18,126 MPa (2.629 million psi) with a coefficient of variation (CV) 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 17,493 MPa (2.537 million psi). Results of tests on the lumber used in the Morgan County bridge flanges indicated that it had a mean flatwise MOE of 17,885 MPa (2.594 million psi) with a CV of 17.4 %. The corresponding mean edgewise MOE was 17,258 MPa (2.503 million psi).

The lumber used to fabricate the webs of the bridge panels 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 the lumber used in the Georgia Pacific bridge webs indicated that it had a mean flatwise MOE of 16,955 MPa (2.459 million psi) with a CV of 12.5%. The mean flatwise MOE of the lumber used in the Morgan County bridge webs was 16,104 MPa (2.336 million psi) with a CV of 17.0%.

Bending test data were used to calculate the stiffnesses (MOE x I) and MOE of the finished bridge deck panels. These data are shown in Table 2. For the Georgia Pacific bridge Panels 1 and 2 respectively, the stiffness results were 1.46 x 10^{11} kN-mm^2 (51.0 million kip-in^2) and 1.42 x 10^{11} kN-mm^2 (49.5 million kip-in^2). When the full cross sections of the bridge panels were used to calculate the moment of inertia, the corresponding MOE values were 16,341 MPa (2.37 million psi) and 15,858 MPa (2.30 million psi). For the Morgan County bridge Panels 1 and 2, the stiffness results were 1.34 x 10^{11} kN-mm^2 (46.7 million kip-in^2) and 1.43 x 10^{11} kN-mm^2 (50.0 million kip-in^2). When the full cross sections of the bridge panels were used to calculate the moment of inertia, the corresponding MOE values were 14,962 MPa (2.17 million psi) and 15,996 MPa (2.32 million psi). Based on the force-deflection plots from these tests, all of the deck panels appeared to exhibit linear elastic behavior up to the maximum loads used in the tests.

Analytical Assessment of Bridge Panel MOE

Transformed Section Analysis

Data for the location of each board and its corresponding MOE were used as input to the transformed section program. The transformed section analysis used lumber data input for consecutive 610 mm (24-in.) long segments along the length of each lamination. 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 across the entire span. The latter assumption was not entirely accurate since the webs were tapered near their ends and the flanges extended past the end of the webs.
Predicted deflection results for the bridge modules are shown in Tables 3 and 4. The deflection results in Table 3 are based on a single 44.5 kN (10,000 lb) load applied at midspan (as in the static bending tests conducted at the laminating plant). The data show that the transformed section model predicted maximum panel deflections with good accuracy. The predicted midspan deflections were within 2% of the test values for all panels except Georgia Pacific Panel 2. Table 4 contains deflection results corresponding to the static field tests of the bridge panels. These values are based on the actual loads reported earlier for the truck axles. Again, the model predicted maximum panel deflections with good accuracy. Differences between predicted and actual deflection values ranged from 0.5% to 7%.

Stiffness results for the analytical assessment are summarized in Table 5. Based on the transformed section analysis of the original target E-rated layup, the theoretical predicted stiffness and MOE for all the deck panels were 1.60 x 10^{11} kN-mm^2 (55.8 million kip-in^2) and 17,858 MPa (2.59 million psi), respectively. When the full cross section is used, the moment of inertia of the panels was (21,532 in^4) When the actual lumber MOE data were used in the transformed section analysis, the predicted MOE for Georgia Pacific Panel 1 was 17,651 MPa (2.56 million psi) versus an actual MOE of 16,341 MPa (2.37 million psi). For Georgia Pacific Panel 2, the predicted MOE was 17,238 MPa (2.50 million psi) versus an actual MOE of 15,858 MPa (2.30 million psi). For both bridge panels, the transformed section model overpredicted the actual MOEs by approximately 8.5%.

The predicted MOE for Morgan County Panel 1 was 16,892 MPa (2.45 million psi) versus an actual MOE of 14,962 MPa (2.17 million psi). For Morgan County Panel 2, the predicted MOE was 17,927 MPa (2.60 million psi) versus an actual MOE of 15,996 MPa (2.32 million psi). For both bridge panels, the transformed section model over-predicted the actual MOEs by approximately 12.5%.

Finite Element Analysis

As in the transformed section analysis, the actual lumber data were used as input for the finite element models. Results from the finite element deflection modeling are summarized in Tables 2 and 3. When predicting deflection behavior under the conditions of the static bending tests, the finite element model predicted maximum deflections with greater accuracy than the transformed section model (within 1% on average). Also, when modeling deflection performance during the field load tests, the finite element model results were within 2% of the actual deflections, on average.

Stiffness results from the finite element modeling are summarized in Table 6. When the panels were supported on the flanges (as in the static bending tests), the predicted MOE for Georgia Pacific Panel 1 was 18,203 MPa (2.64 million psi) versus an actual MOE of 16,341 MPa (2.37 million psi). For Georgia Pacific Panel 2, the predicted MOE was 18,203 MPa (2.64 million psi) versus an actual MOE of 15,858 MPa (2.30 million psi). For both bridge panels, the finite element model overpredicted the actual MOEs by approximately 13%.

The predicted MOE for Morgan County Panel 1 was 17,858 MPa (2.59 million psi) versus an actual MOE of 14,962 MPa (2.17 million psi). For Morgan County Panel 2, the predicted MOE was 18,548 MPa (2.69 million psi) versus an actual MOE of 15,996 MPa (2.32 million psi). For both bridge panels, the finite element model overpredicted the actual MOEs by approximately 18%.

Discussion

Both the transformed section and finite element models overpredicted the actual MOEs of the panels with the finite element model predicting the highest MOE values. It appears that the relatively simple transformed section model predicts the stiffness of the panels with sufficient accuracy. The differences in predicted and actual panel MOEs may have been due, at least partially, to test conditions where the overhanging flange supported the bridge deck panel. This test setup probably resulted in a loss in apparent stiffness of the deck panel due to shear lag. Another factor affecting the agreement between values predicted by the transformed section model and the actual values is the assumption, in the transformed section analyses, of a uniform cross section for the entire length of the bridge panel. As discussed earlier, the actual webs are tapered near their ends, which results in different section properties for portions of the bridge. The finite element model, however, was developed to account for the tapered webs and the overhanging flanges. Another factor contributing to the differences between predicted and actual results is the reduction in actual cross section due to the vertical holes drilled through the webs and flanges. Both the transformed section and finite element models assumed the cross-section to be uniform throughout the bridge span. These localized reductions in the moment of inertia would have resulted in some reduction in the modulus of elasticity of the bridge modules.

Load Test Behavior

Transverse load test deflection plots for the Georgia Pacific and Morgan County bridges are shown in Figures 7 and 8, respectively, as viewed from the south end (looking north). For each load test, no permanent residual deformation was measured at the conclusion of the testing. Additionally, there was no detectable movement at either of the footings. However, when testing the Morgan County bridge, it appeared that the webs of both panels were resting on the stream banks. Therefore, the spread footings were probably not supporting much of the bridge load.

Georgia Pacific Bridge

Figure 7 shows that for Load Case 1 and Load Case 3, the symmetry of loading resulted in deflection profiles that are approximately mirror images of one another. Deflection differences of corresponding data points for the two positions were 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 were expected since the truck was loading the flange in its weak axis.

For Load Case 2 and Load Case 4, deflections were 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 resulted in similar deflections 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 was likely due to minor differences in the truck transverse position. The maximum deflection recorded for Load Case 4 corresponded to a deflection value of approximately L/975, at 55% of design bending moment.

Morgan County Bridge

Figure 8 shows that for Load Case 1 and Load Case 3, the deflection profiles are very similar to those of the Georgia Pacific bridge. The deflection profiles are almost mirror images of one another with the exception of the deflection reading from the outside flange of Panel 2. Deflection differences of corresponding data points for the two positions were generally within 0.75 mm (0.03 in.) except for the outside flange of Panel 2. Maximum deflections for these load cases occurred in Panel 1 and measured 5.8 mm (0.23 in.) at the outside panel edge for Load Case 1 and 4.6 mm (0.18 in.) at the exterior web of Panel 2 for Load Case 3.

For Load Case 2, deflections recorded for Panel 2 were as expected. However, results for Panel 1 reveal that the exterior web and flange deflected more than the interior web and flange locations. Since the truck tires were positioned in the center of the panel, this was not due to an error in loading. One possible explanation for this inconsistency may be because the bridge panel webs were resting on the stream bank (i.e., the timber sill was not fully supporting the flange of the panel as the design intended). In this case, the soil may have settled under the load imposed by the truck and the bridge panel may have rotated during the test. With the wheel line centered on Panel 1 for Load Case 2, the maximum deflection recorded was 3 mm (0.12 in.). These deflections are much smaller than what would be expected under the design load because the actual span was less than the full design value. Using the estimated span, the load exerted by the truck resulted in a bending moment approximately 52% of the design moment. The maximum deflections under Load Case 2 corresponded to a deflection value of approximately L/2400.

Condition Assessment

Georgia Pacific Bridge

Inspection 1. At the time of the first load test, which was conducted near Newnan, GA, very limited traffic had used 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 of the webs; 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.

Inspection 2. The second inspection occurred after the bridge had been installed near Reynolds, GA and used by logging traffic approximately one month. The primary damage to the bridge panels consisted of failures in both of the curbs. While the curbs were still intact, the curb on Panel 1 displayed bending failures near each end of the bridge panel. The curb on Panel 2 showed a similar failure near one of the bridge ends. These failures were apparently caused when the logging crew drove a dual-wheel equipped skidder across the bridge. The outside wheels of the skidder were too wide to fit on the bridge deck and therefore ran along the curb. This resulted in the curb rails actually supporting the entire weight of the skidder. Since the curbs were designed for delineation purposes only, it is not surprising that the curbs failed under the skidder loads.

Other features noted during this visit included several checks on the bottom faces of two of the webs. The checks appeared to propagate from holes drilled through the flanges and webs during fabrication at the laminating plant. The clamping hardware was placed through the holes while the glue cured. The holes are 25 mm (1 in.) in diameter and are spaced approximately 305 mm (12 in.) apart along the length of the webs. The checks occurred in one web of Panel 1 and one web of Panel 2 and were located in regions centered about the bridge midpsan approximately 1.8 m (6 ft) long. At the time of this visit, the largest check in Panel 1 was approximately 300 mm (11.8 in.) long and 30 mm (1.2 in.) deep. The largest check in Panel 2 was approximately 300 mm (11.8 in.) long and 15 mm (0.6 in.) deep.

Another item recorded during the inspection was a loose center diaphragm on Panel 1. Also, slight damage to the surfaces of two of the webs had apparently occurred during the previous bridge installation.

Inspection 3. Between the second and third inspection visits, the site had experienced severe flooding with water depths as high as approximately 3 m (10 ft) above the original elevation of the bridge deck. Although the bridge was held in place by cables, the flood waters had moved the panels and when the water receded, Panel 1 was left laying on its side with one end partially submerged. Although the bridge panels floated during the flood period, most of the panels remained below the surface of the water with only the flanges and curbs visible. Personnel from Georgia Pacific estimated that the bridge was in this condition for approximately 30 days.

During this visit, no significant damage was noted beyond what was observed at the second visit. The checks noted in the second visit were reexamined and no significant change was noted in their length or depth.

Moisture contents were taken with a resistance type moisture meter with 25 mm (1 in.) long pins at several locations on the webs and flanges of the panels. The flange moisture content readings (on a dry-basis) ranged from 16% to 41% with most readings between 20% and 30%. Web moisture content readings ranged from 16% to 29%.

Inspection 4. After the third inspection visit, the bridge was repositioned at the site and logging restarted and continued for a one month period until the site was flooded again. This flooding period lasted for approximately 45 days and floodwater depths similar to the previous events were experienced. After the floodwater receded, the bridge was repositioned and logging activities were completed. At this point, the logging contractor removed the bridge by using a grapple skidder to skid the panels approximately 1.6 km (1 mile) to a staging area. The fourth inspection occurred after the bridge panels were brought to this staging area.

Several areas of damage were noted during this inspection. The most noticeable items apparently resulted from rough handling by the grapple skidder. It should be pointed out that using a skidder to remove the panels and skid them for such distances was not the intended removal method for the bridge. Most of the curb rail for Panel 1 was missing, with only short pieces near the panel ends remaining. It appeared that the skidder had dragged the panel against a tree and pulled the curb off of the flange. The lag screws that held the two steel curb brackets to the flange were pulled out of the flange; however, it appeared that the flange was not significantly damaged. One end of the curb on Panel 2 was also broken and pulled away from the steel bracket at the panel end. This apparently resulted from taking the grapple of the skidder and clamping the end of the bridge to drag the bridge to the staging area. At the same location where the curb was broken, the edges of the flange were also apparently damaged slightly by the skidder grapple. Also, one of the steel angles attached to the end of Panel 2 was pulled loose from the flange.

Another feature damaged during this removal was the steel protector plates fastened to the ends of the webs. On the ends of the panels that were dragged on the ground, the steel plates were peeled back away from the wood. This was the result of dragging the panels such a long distance over a rough gravel road. Because the panels were sitting on the ground during the inspection, there was no way to determine if the webs were damaged.

Since the bottom surface of the webs could not be inspected, no information could be obtained on the condition of the checks in the webs. However, at this point, several checks on the top surface of the flanges were observed. The majority of the checks were found on Panel 2. The checks ran longitudinally along the flange surface with the largest checks near the holes drilled in the flanges. The largest checks were 24 mm (0.95 in.) deep and 5 mm (0.19 in.) wide. It is possible that these checks were present during the earlier visits, but were not observed due to the mud and gravel that was on the surface of the bridge. It was only during this visit that we were able to clean the bridge surface without interfering with traffic using the bridge. The checks also could have been the result of the extended time that the bridge was partially submerged in the water.

Moisture contents of the flanges were checked in a manner similar to that described on the previous visit. The flange moisture readings ranged from 15.7% to 27.5%. Web readings were not taken because of limited access to the webs.

One other feature was noted when looking at the ends of the panels. Both panels appeared to have a very slight positive camber across the width of the flanges, with the highest points near the panel centerline. This may be due to increased moisture contents on the top surface of the flanges relative to the lower sides of the flanges. Although several items were damaged after the last removal of the bridge, none of the damage appeared to have been significant enough to affect the structural adequacy of the bridge.

Morgan County Bridge

Inspection 1. The first detailed inspection occurred after the bridge was in service for 11 months. The bridge had received only light vehicular traffic during this time. No damage was observed during the installation or during this period of use.

Minor surface checking was observed on the surface of Panel 2. The largest check on the surface of the flange of Panel 2 was 10 mm (0.38 in.) deep, 2 mm (0.10 in.) wide, and 1.7 m (66 in.) long. These checks did not appear to be associated with the holes drilled through the flanges. No checks were observed on the surface of the flange of Panel 1. Several large checks, similar to those on the Georgia Pacific bridge, were observed on the bottom surfaces of the webs of both panels. Again, the checks appear to propagate from the holes drilled through the webs. The largest check on the webs of Panel 1 was 33 mm (1.31 in.) deep, 3 mm (0.13 in.) wide, and 518 mm (20.4 in.) long. The largest check on the webs of Panel 2 was 35 mm (1.38 in.) deep, 4 mm (0.16 in.) wide, and 305 mm (12 in.) long. At this point, the checks do not appear to have affected the structural adequacy of the bridge. As with the Georgia Pacific bridge, both panels appeared to have a very slight positive camber across the width of the flanges, with the highest points near the panel centerline. Moisture contents in the webs ranged from 16.5% to 21.5%. Aside from the checks in the webs, the bridge appeared to be in very good condition overall.

Inspection 2. The second detailed inspection occurred after the bridge had been installed approximately two years. The bridge had still only received light traffic over the two-year period. No damage was observed since the previous inspection visit and the bridge still appeared to be in very good condition overall.

Surface checks on the flange of Panel 2, which were observed in the first visit, did not appear to have changed in length or width since the first visit, although the maximum depth of the check was measured at 12 mm (0.5 in.). As in the first visit, no checks were observed on the surface of Panel 1. The checks observed on the webs of Panel 1 do not appear to have increased in their depth; however, the maximum width has increased to 10 mm (0.38 in.). Checks observed on the webs of Panel 2 do not appear to have changed in length or depth; however, the maximum width has increased to 8 mm (0.31 in.). Moisture contents in the webs ranged generally from 17.3% to 28.7% with one outlier value of 41.9%. The slight camber observed in the flange was still evident during this second inspection.

SUMMARY AND CONCLUSIONS

Based on a two-year program of modeling and testing, the longitudinal T-section glulam deck bridges have performed well and should continue to provide acceptable service as portable logging bridges. The following specific conclusions can be made at this time:

1. It is feasible and practical to construct a longitudinal glulam deck panel in a double-tee cross section.

2. The total time to install the bridges was less than 3 hours. Installations were easily accomplished using common construction equipment. In the typical installations, there was no disturbance of the stream channels, and therefore there were no water quality impacts during construction activities.

3. The costs of the Georgia Pacific and Morgan County bridge superstructures were \$381/m^2 (\$35/ft^2) and \$359/m^2 (\$33/ft^2), respectively. These costs are competitive with other timber bridge superstructure systems. The estimated costs for installation and removal of the bridges at 10 different sites were \$2,760 and \$2,578 per site, for the Georgia Pacific and Morgan County bridges, respectively. These costs compared very favorably with the costs of installing other traditional stream crossing structures on similar size streams.

4. Static bending test results indicated that the T-section glulam decks exhibited linear elastic behavior when subjected to loads approaching their design loads.

5. Both transformed section and finite element models overpredicted the stiffnesses and MOEs of the panels. The stiffness and MOE values predicted by simple transformed section analyses were 8.5% and 12.5% higher than those measured in static bending tests of the Georgia Pacific and Morgan County bridges, respectively. Stiffness and MOE values predicted by the finite element models were approximately 13% and 18% higher than test values for

the Georgia Pacific and Morgan County bridges, respectively. Although the finite element model is more powerful, it appears that the transformed section model predicted overall panel stiffnesses that agreed more closely with the actual static bending test results.

6. Results from static load tests of both bridges indicated that the T-section glulam deck exhibited acceptable levels of deflection. The maximum midspan deflections recorded when the truck wheel lines were positioned near the center of the panel were equivalent to L/975 at 55% of design bending moment for the Georgia Pacific bridge and L/2400 at 52% of design bending moment for the Morgan County bridge.

6. When handled properly, the bridges have performed well with minimal damage. Rough handling by a grapple skidder resulted in damage at several locations on the Georgia Pacific bridge.

7. Large checks have developed at several locations on the webs of both bridges and on the top surface of the flange on one of the Georgia Pacific bridge panels. These checks appear to be related to holes that were drilled through the flanges and webs during fabrication of the panels. The checks appear to have increased in size during the monitoring period; however, they do not appear to have significantly affected the structural adequacy of the bridges. Alternative fabrication techniques would probably eliminate this problem for future bridges.

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Table 1. Results of MOE tests on individual lumber specimens for the Georgia Pacific bridge and the Morgan County bridge. Both web and flange lumber data are flatwise MOEs.

	Web Lumber Modulus of Elasticity		Flange Lumber Modulus of Elasticity	
	Georgia Pacific Bridge	Morgan County Bridge	Georgia Pacific Bridge	Morgan County Bridge
Mean	16,955 MPa 2.459 million psi	16,104 MPa 2.336 million psi	18,126 MPa 2.629 million psi	17,885 MPa 2.594 million psi
Coefficient of Variation	12.5%	17.0%	17.3%	17.4%
Number of Boards	108	108	297	300

Table 2. Results of static bending tests of bridge panels.

	Actual MOE from Bending Test	Actual Stiffness (MOE*I) from Bending Test
Georgia Pacific Panel 1	16,341 MPa 2.370 million psi	1.464×10^{11} N-mm^2 51.02 million kip-in^2
Georgia Pacific Panel 2	15,845 MPa 2.298 million psi	1.420×10^{11} N-mm^2 49.48 million kip-in^2
Morgan County Panel 1	14,966 MPa 2.171 million psi	1.341×10^{11} N-mm^2 46.74 million kip-in^2
Morgan County Panel 2	15,999 MPa 2.320 million psi	1.434×10^{11} N-mm^2 49.96 million kip-in^2

Table 3. Deflection results from static bending tests and analytical models. Deflections listed are those measured or predicted based on a concentrated 44.5 kN (10,000 lb) load at midspan.

	Actual Deflection at Midspan from Static Bending Test Using Center Point Load	Predicted Deflection at Midspan from Transformed Section Model	Predicted Deflection at Midspan from Finite Element Model
Georgia Pacific Panel 1	9.169 mm 0.3610 in.	9.334 mm 0.3675 in.	8.834 mm 0.3478 in.
Georgia Pacific Panel 2	8.202 mm 0.3229 in.	9.507 mm 0.3743 in.	8.964 mm 0.3529 in.
Morgan County Panel 1	6.093 mm 0.2399 in.	6.223 mm 0.2450 in.	6.124 mm 0.2411 in.
Morgan County Panel 2	5.972 mm 0.2351 in.	5.862 mm 0.2308 in.	5.870 mm 0.2311 in.

Table 4. Deflection results from field load tests and analytical models. Deflections listed are those measured or predicted based on the actual loads shown in Figure 4.

	Actual Deflection at Midspan from Field Load Test	Predicted Deflection at Midspan from Transformed Section Model	Predicted Deflection at Midspan from Finite Element Model
Georgia Pacific Panel 1	10.874 mm 0.4281 in.	11.621 mm 0.4575 in.	10.894 mm 0.4289 in.
Georgia Pacific Panel 2	10.749 mm 0.4232 in.	10.696 mm 0.4211 in.	9.995 mm 0.3935 in.
Morgan County Panel 1	3.683 mm 0.1450 in.	3.480 mm 0.1370 in.	3.670 mm 0.1445 in.
Morgan County Panel 2	3.048 mm 0.1200 in.	3.269 mm 0.1287 in.	3.518 mm 0.1385 in.

Table 5. Comparison of stiffness results from static bending tests and predictions by the transformed section analysis.

	Stiffness from Static Bending Test	MOE from Static Bending Test	Stiffness from Transformed Section Analysis	MOE from Transformed Section Analysis	Per Cent Difference Between Experimental and Predicted
Georgia Pacific Panel 1	1.464×10^{11} N-mm^2 51.02×10^6 kip-in^2	16,338 Mpa 2.37×10^6 psi	1.585×10^{11} N-mm^2 55.23×10^6 kip-in^2	17,686 Mpa 2.56×10^6 psi	8.2%
Georgia Pacific Panel 2	1.420×10^{11} N-mm^2 49.48×10^6 kip-in^2	15,844 Mpa 2.30×10^6 psi	1.546×10^{11} N-mm^2 53.87×10^6 kip-in^2	17,251 Mpa 2.50×10^6 psi	8.9%
Morgan County Panel 1	1.341×10^{11} N-mm^2 46.74×10^6 kip-in^2	14,967 Mpa 2.17×10^6 psi	1.511×10^{11} N-mm^2 52.67×10^6 kip-in^2	16,865 Mpa 2.45×10^6 psi	12.7%
Morgan County Panel 2	1.434×10^{11} N-mm^2 49.96×10^6 kip-in^2	15,999 Mpa 2.32×10^6 psi	1.610×10^{11} N-mm^2 56.09×10^6 kip-in^2	17,961 Mpa 2.60×10^6 psi	12.3%

Table 6. Comparison of stiffness results from static bending tests and predictions by the finite element analysis.

	Stiffness from Static Bending Test	MOE from Static Bending Test	Stiffness from Finite Element Analysis	MOE from Finite Element Analysis	Per Cent Difference Between Experimental and Predicted
Georgia Pacific Panel 1	1.464×10^{11} N-mm^2 51.02×10^6 kip-in^2	16,338 Mpa 2.37×10^6 psi	1.631×10^{11} N-mm^2 55.77×10^6 kip-in^2	18,203 Mpa 2.64×10^6 psi	11.4%
Georgia Pacific Panel 2	1.420×10^{11} N-mm^2 49.48×10^6 kip-in^2	15,844 Mpa 2.30×10^6 psi	1.631×10^{11} N-mm^2 57.92×10^6 kip-in^2	18,203 Mpa 2.64×10^6 psi	14.9%
Morgan County Panel 1	1.341×10^{11} N-mm^2 46.74×10^6 kip-in^2	14,967 Mpa 2.17×10^6 psi	1.600×10^{11} N-mm^2 56.84×10^6 kip-in^2	17,858 Mpa 2.59×10^6 psi	19.3%
Morgan County Panel 2	1.434×10^{11} N-mm^2 49.96×10^6 kip-in^2	15,999 Mpa 2.32×10^6 psi	1.662×10^{11} N-mm^2 56.84×10^6 kip-in^2	18,548 Mpa 2.69×10^6 psi	15.9%

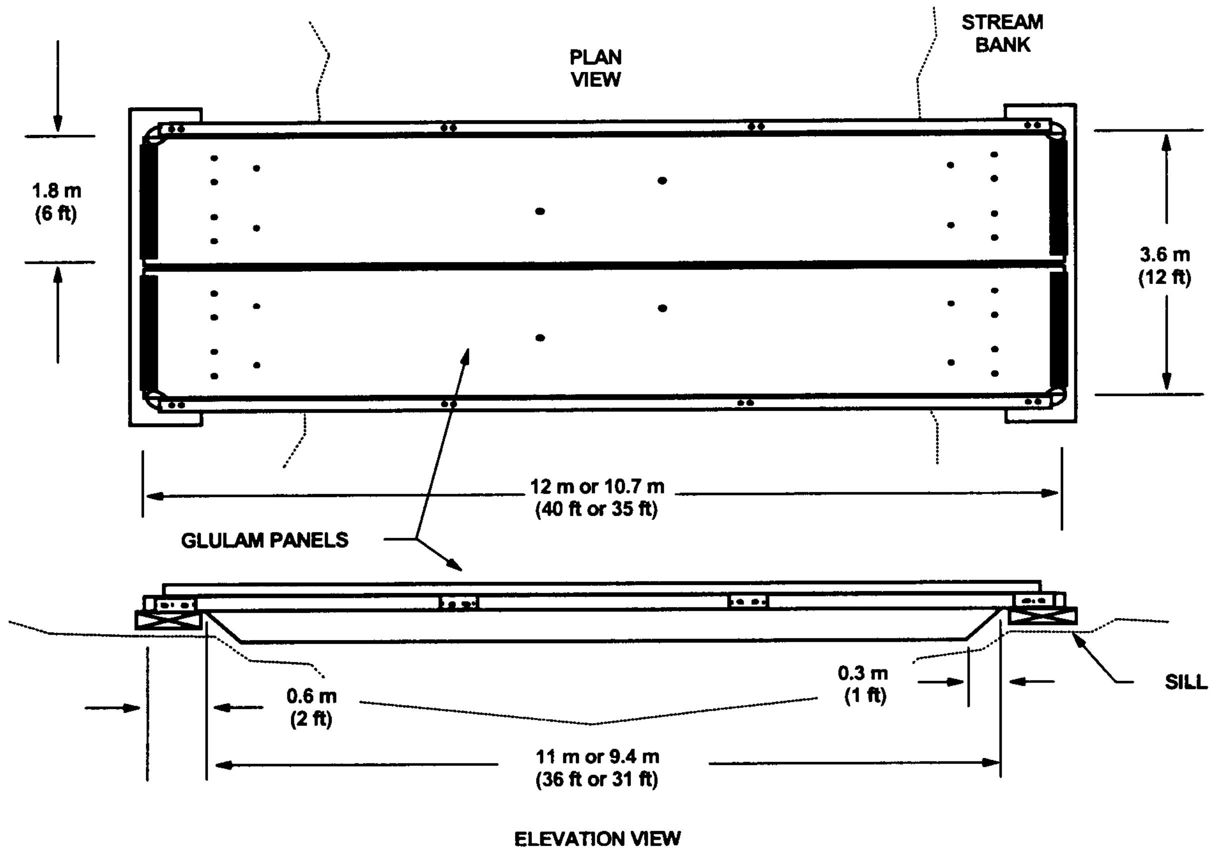

Figure 1. Sketch of the bridge installations showing overall dimensions of the portable longitudinal T-section glulam bridges.

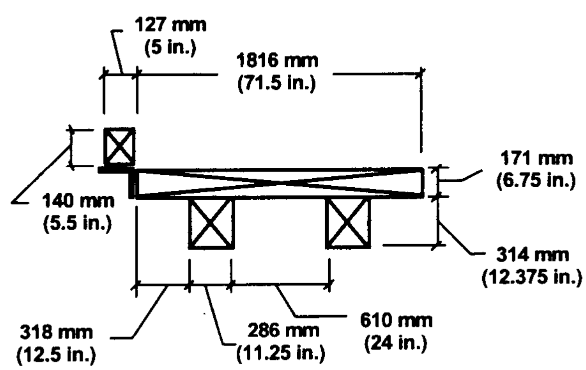

Figure 2. Cross section view of the longitudinal T-section glulam deck panels with the curb rail attached. Diaphragms and connectors are omitted for clarity.

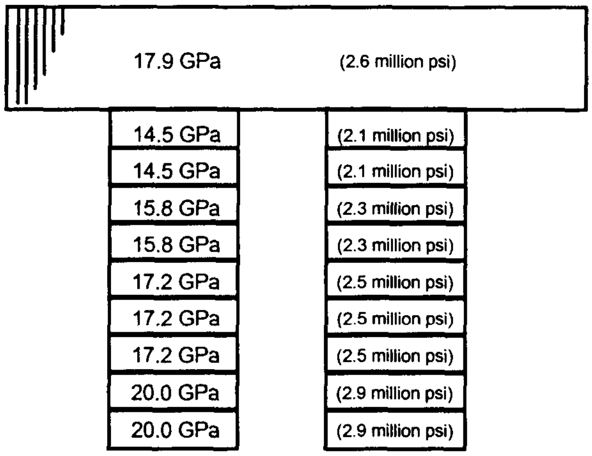

Figure 3. Theoretical distribution of lumber MOE classes in the T-section glulam deck panels.

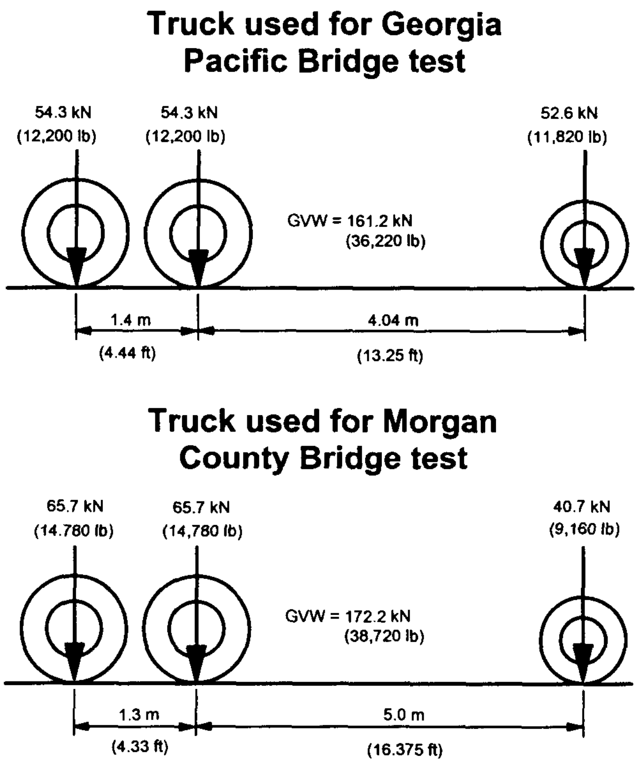

Figure 4. Load test truck configurations and axle loads. The transverse vehicle track width, measured center-to-center of the rear tires, was 1.8 m (6 ft) for the truck used in the Georgia Pacific test and 1830 mm (72 in.) for the truck used in the Morgan County test.

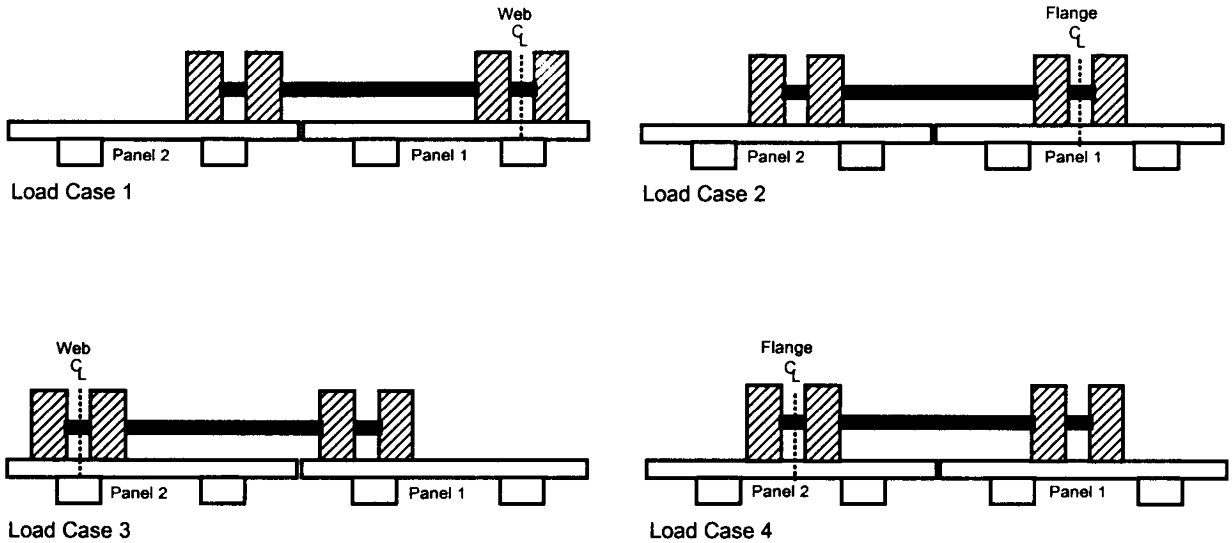

Figure 5. Transverse load positions (looking north) and deck panel numbers for the load test of the Georgia Pacific bridge. For all load cases, the two rear axles were centered over the bridge centerspan.

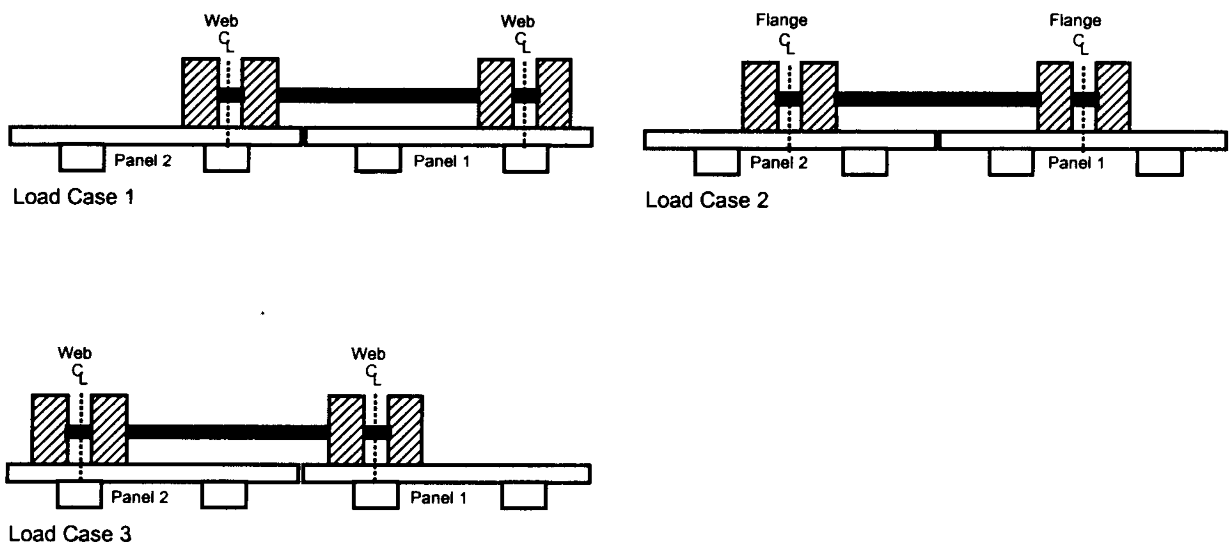

Figure 6. Transverse load positions (looking north) and deck panel numbers for the load test of the Morgan County bridge. For all load cases, the two rear axles were centered over the bridge centerspan.

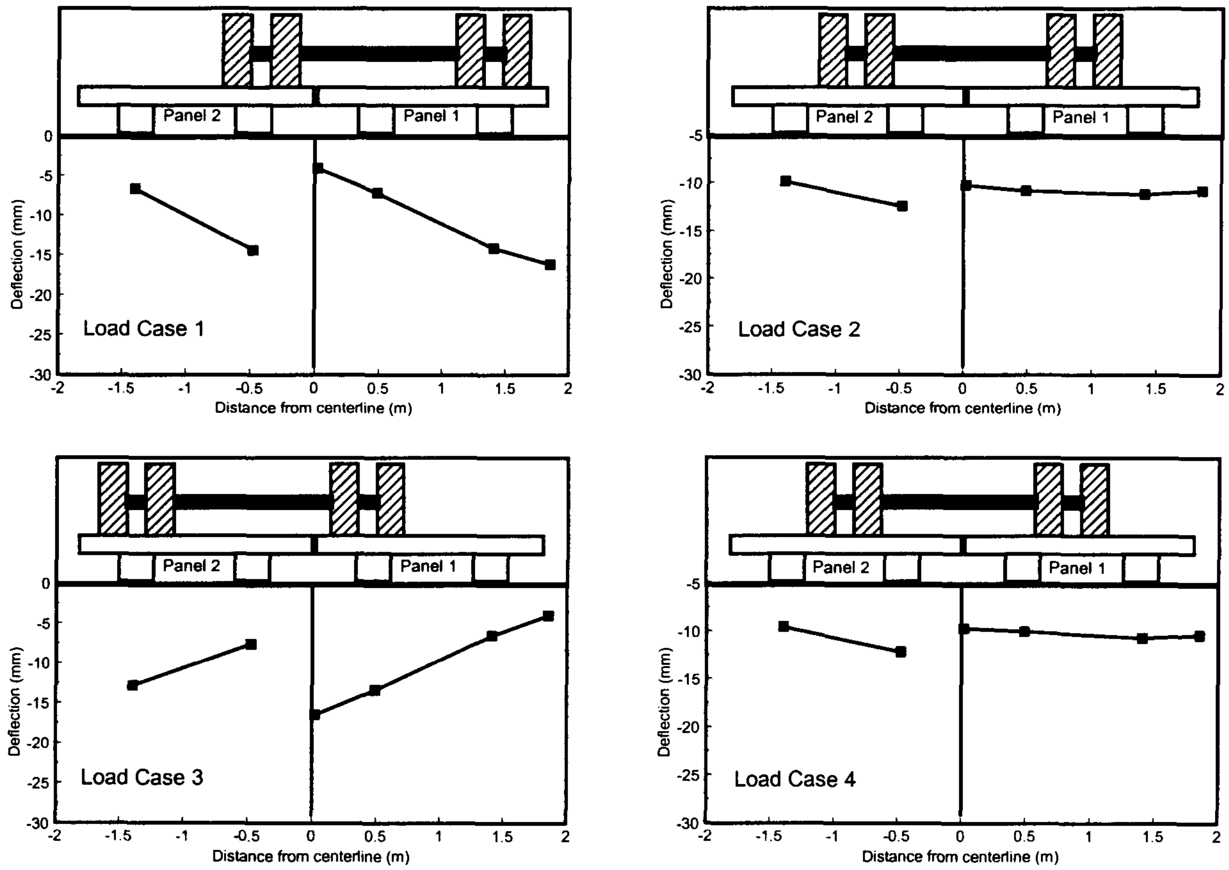

Figure 7. Transverse deflection for the load test of the Georgia Pacific bridge measured at the bridge centerspan (looking north). Bridge cross-section and vehicle positions are shown to aid interpretation and are not to scale.

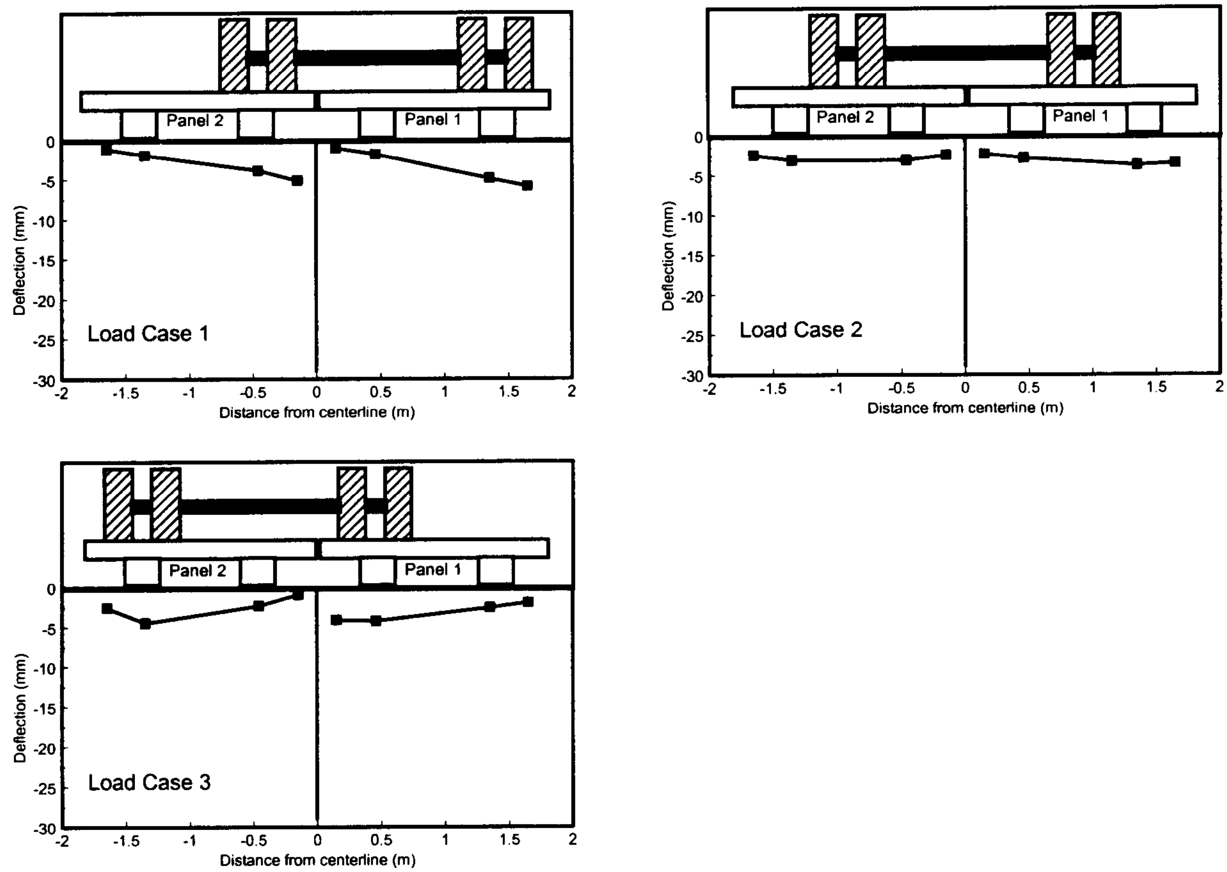

Figure 8. Transverse deflection for the load test of the Georgia Pacific bridge measured at the bridge centerspan (looking north). Bridge cross-section and vehicle positions are shown to aid interpretation and are not to scale.