

Performance of Portable T-Section Glulam Timber Bridges

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Written for Presentation at the
1997 Annual International Meeting
sponsored by
ASAE

Minneapolis Convention Center
Minneapolis, Minnesota
August 10-14, 1997

Summary:

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

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ABSTRACT

The use of portable bridge systems has increased due to a heightened awareness of the need to reduce environmental impacts at road stream crossings, particularly in forestry applications. This paper describes the design and initial 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 and actual bridge panel stiffnesses were 92% and 89% of predicted values for the 12.2-m (40-ft) and 10.7-m (35-ft) bridges, respectively. The complete bridge systems appear to be cost effective with superstructure costs of \$381/m² (\$35/ft²) for the 12.2-m (40-ft) bridge and \$359/m² (\$33/ft²) for the 10.7-m (35-ft) bridge. If the bridges are reused and installed at ten different sites, the estimated costs per site are \$2,760 and \$2,578 for the 12.2-m (40-ft) and 10.7-m (35-ft) bridges, respectively. These bridge costs are competitive with the costs of traditional stream crossing structures such as fords or culverts.

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. The roads that provide access to our forest resources 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 or heavier forestry equipment. In either case, roadway use is commonly limited to short periods over a relatively long forest management period. For example, roadway access may be required for only a six-month period over 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, 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. Thompson et al. (1996) reported that during the construction of a gravel ford on a stream approximately 2 m (6 ft) wide, peak sediment concentration in water samples taken downstream from the ford was nearly 2810 mg/l higher than that of samples taken upstream from the ford. Also, when light vehicular traffic drove

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through the stream, sediment concentration in water samples taken downstream from the ford was as much as 255 mg/l higher than that of the upstream samples.

While some of the 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 corrugated metal pipe culvert, sediment concentration in water samples taken downstream of the culvert was over 950 mg/l higher than that of the upstream samples. Also, culverts may clog with debris and may be washed out during heavy runoff periods, thereby introducing additional sediment into the stream. In the case of roads or 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.

Bridges for low-volume forest roads can be either 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, and are not economically feasible for short use periods and often require expensive maintenance for continued service. Additionally, 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 during recent years 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 for 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 to other crossings such as fords and culverts.

Many of the advantages of timber bridges make them ideal for temporary stream crossings. The objective of this paper is to discuss the design and initial evaluation of a new type of portable longitudinal glulam deck bridge system. The bridge system uses two non-interconnected panels that are fabricated in a unique double-tee cross section. Design, installation and cost will be discussed along with results of tests of two separate prototype bridges.

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, 1989; Taylor et al., 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 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 relatively high initial cost of \$8,000 may discourage some users from purchasing this type of bridge over the non-engineered designs frequently used for off-highway vehicles.

LONGITUDINAL T-SECTION GLULAM BRIDGE 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) and Taylor and Ritter (1996) described initial results of using the first of the T-section bridges discussed here. This paper will provide additional details on performance of the first bridge and discuss test results for the second bridge.

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.

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 preservatively treated with creosote to 194 kg/m³ (12 lb/ft³) 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. 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 preservatively 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 a tracked 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 no 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 mud 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 m² (480 ft²), the cost was approximately \$381/m² (\$35/ft²). 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 trackhoe 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 m² (420 ft²), the cost was approximately \$359/m² (\$33/ft²). 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 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. These evaluation procedures are discussed in the following sections.

Lamination and Finished Bridge Deck 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 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 would 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 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 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. These data were then used as input for a transformed section analysis computer program developed at the USDA Forest Products Laboratory. Using the lumber data, the program was used to predict the MOE of the finished deck panels for comparison with bending test results.

Load Test Behavior

Detailed static and dynamic load tests are planned for both bridges. However, at this time, only an initial static load test has been completed for the Georgia Pacific bridge. This test 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 at 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 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. 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.).

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. 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. 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 still in its initial stages and will continue for a period of one more year. Results and discussion of the initial performance data follow.

Lamination and Bridge Panel MOE

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 MOE of the finished bridge deck panels. For the Georgia Pacific bridge Panels 1 and 2 respectively, the MOE results were 16,341 MPa (2.370 million psi) and 15,845 MPa (2.298 million psi). For the Morgan County bridge Panels 1 and 2, the MOE results were 14,966 MPa (2.171 million psi) and 15,999 MPa (2.320 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

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 across the entire span. The latter assumption was not entirely accurate since the webs were tapered near their ends.

Results for the analytical assessment are summarized in Table 2. Based on the transformed section analysis of the original target E-rated layup, the theoretical predicted MOE for the deck panels was 17,858 MPa (2.59 million psi). When the actual lumber MOE data were used in the transformed section analysis, the predicted MOE for Georgia Pacific Panel 1 was 17,686 MPa (2.565 million psi) versus an actual MOE of 16,341 MPa (2.370 million psi). For Georgia Pacific Panel 2, the predicted MOE was 17,252 MPa (2.502 million psi) versus an actual MOE of 15,845 MPa (2.298 million psi). For both bridge panels, the actual MOEs were approximately 92% of the predicted MOEs.

The predicted MOE for Morgan County Panel 1 was 16,865 MPa (2.446 million psi) versus and actual MOE of 14,966 MPa (2.171 million psi). For Morgan County Panel 2, the predicted MOE was 17,961 MPa (2.605 million psi) versus an actual MOE of 15,999 MPa (2.320 million psi). For both bridge panels, the actual MOEs were approximately 88.9% of the predicted MOEs.

The differences in predicted and actual panel MOEs may be 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. Further tests will help determine how much stiffness is lost due to shear lag at the supports, and in turn will help refine the evaluation of composite behavior. Another factor affecting the agreement between predicted and 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.

Load Test Behavior

Transverse load test deflection plots for the Georgia Pacific bridge are shown in Figure 6 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. 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 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

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 midspan 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 has 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 appears to be in very good condition overall.

SUMMARY AND CONCLUSIONS

Based on initial testing, the longitudinal T-section glulam deck bridges are performing 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 $\$381/\text{m}^2$ ($\$35/\text{ft}^2$) and $\$359/\text{m}^2$ ($\$33/\text{ft}^2$), 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. The actual MOEs of the deck panels were approximately 92% and 89% of the values predicted by simple transformed section analyses for the Georgia Pacific and Morgan County bridges, respectively.
5. Results from load tests of the Georgia Pacific bridge 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.
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. 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. At this point, these checks do not appear to be increasing in size and do not appear to have significantly affected the structural adequacy of the bridges.

Additional research is underway to refine models that predict the stiffness of the glulam T-section panels. In addition, more static and dynamic load tests will be conducted on the bridges. Also, we will continue to monitor the performance and condition of the bridges for one more year. Improvements to design details will be developed at the conclusion of the study.

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ACKNOWLEDGMENTS

The authors wish to thank Toby Anderson, Jack Gazin, Keith Keliher, Dr. Michael Triche, and Dr. Terry Wipf for their assistance during tests of the lumber and the bridge. In addition, the authors wish to thank Structural Wood Systems, Inc. of Greenville, AL, Georgia-Pacific Corporation, Champion International Corporation, and the Morgan County Alabama Forestry Planning Committee for their cooperation during the design, fabrication, and testing of the bridge. This research was supported by the USDA Forest Service Forest Products Laboratory, USDA Forest Service Southern Research Station - Engineering Research Work Unit and the Alabama Agricultural Experiment Station.

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 and transformed section analyses to predict bridge panel MOE.

	Actual MOE from Bending Test	Predicted MOE from Transformed Section Analysis	Ratio of Actual to Predicted MOE
Georgia Pacific Bridge Panel 1	16,341 MPa 2.370 million psi	17,686 MPa 2.565 million psi	92.4%
Georgia Pacific Bridge Panel 2	15,845 MPa 2.298 million psi	17,252 MPa 2.502 million psi	91.8%
Morgan County Bridge Panel 1	14,966 MPa 2.171 million psi	16,865 MPa 2.446 million psi	88.7%
Morgan County Bridge Panel 2	15,999 MPa 2.320 million psi	17,961 MPa 2.605 million psi	89.1%

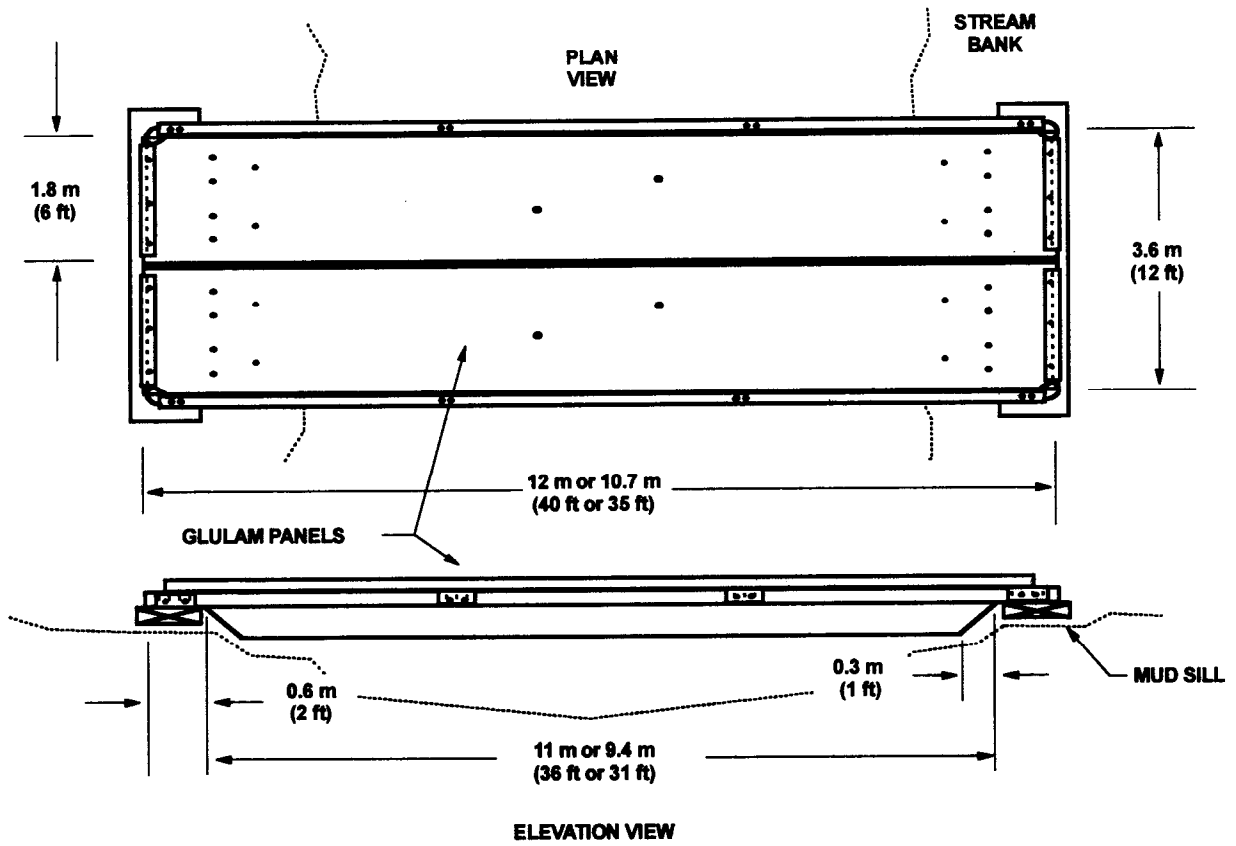


Figure 1. Sketch of the bridge installations showing overall dimensions of the portable longitudinal T-section glulam bridges.

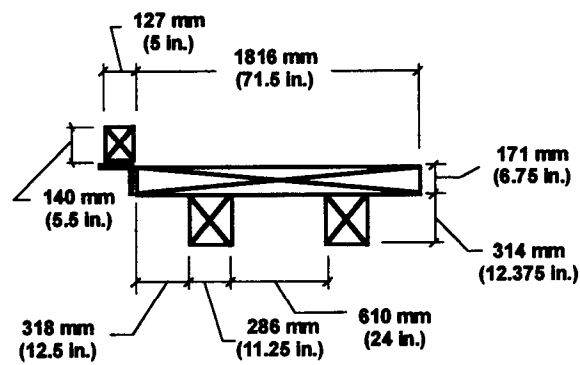


Figure 2. Cross section view of the longitudinal T-section glulam deck panels with the curb rail attached. Diaphragms and connectors are omitted for clarity.

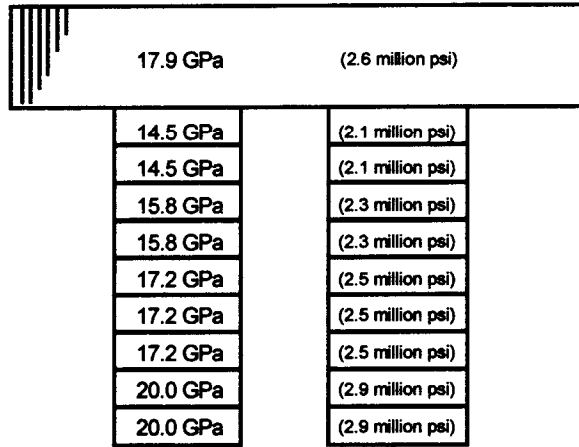


Figure 3. Theoretical distribution of lumber MOE classes in the T-section glulam deck panels.

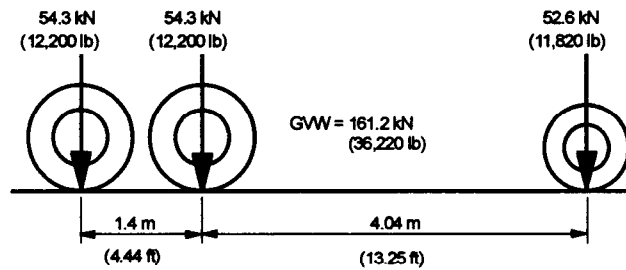


Figure 4. Load test truck configuration and axle loads. The transverse vehicle track width, measured center-to-center of the rear tires, was 1.8 m (6 ft).

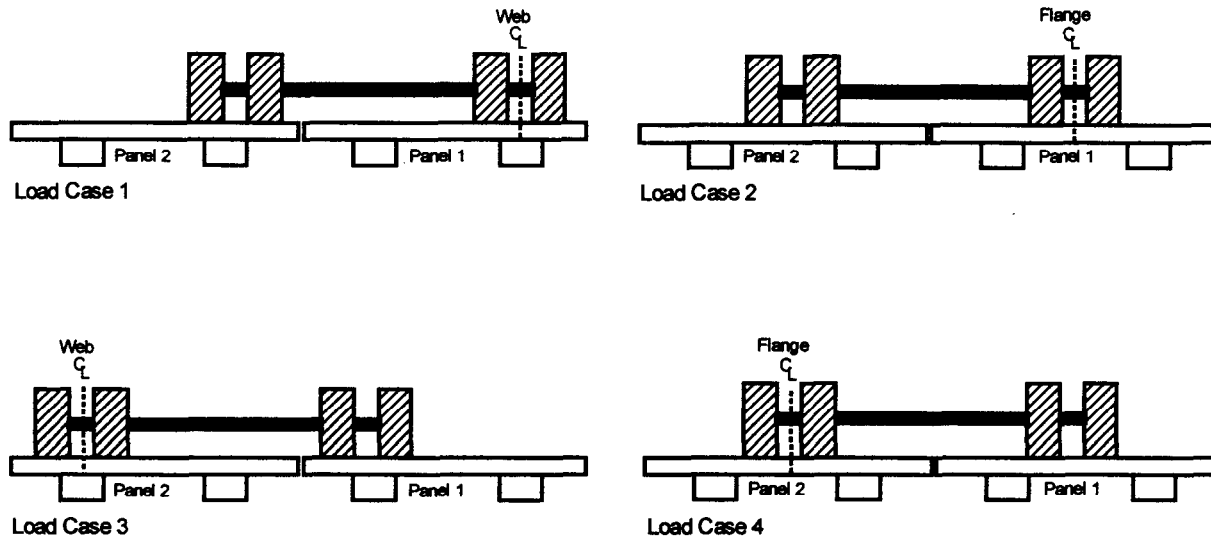


Figure 5. Transverse load positions (looking north) and deck panel numbers for the load test of the Georgia Pacific bridge. For all load cases, the two rear axles were centered over the bridge centerspan.

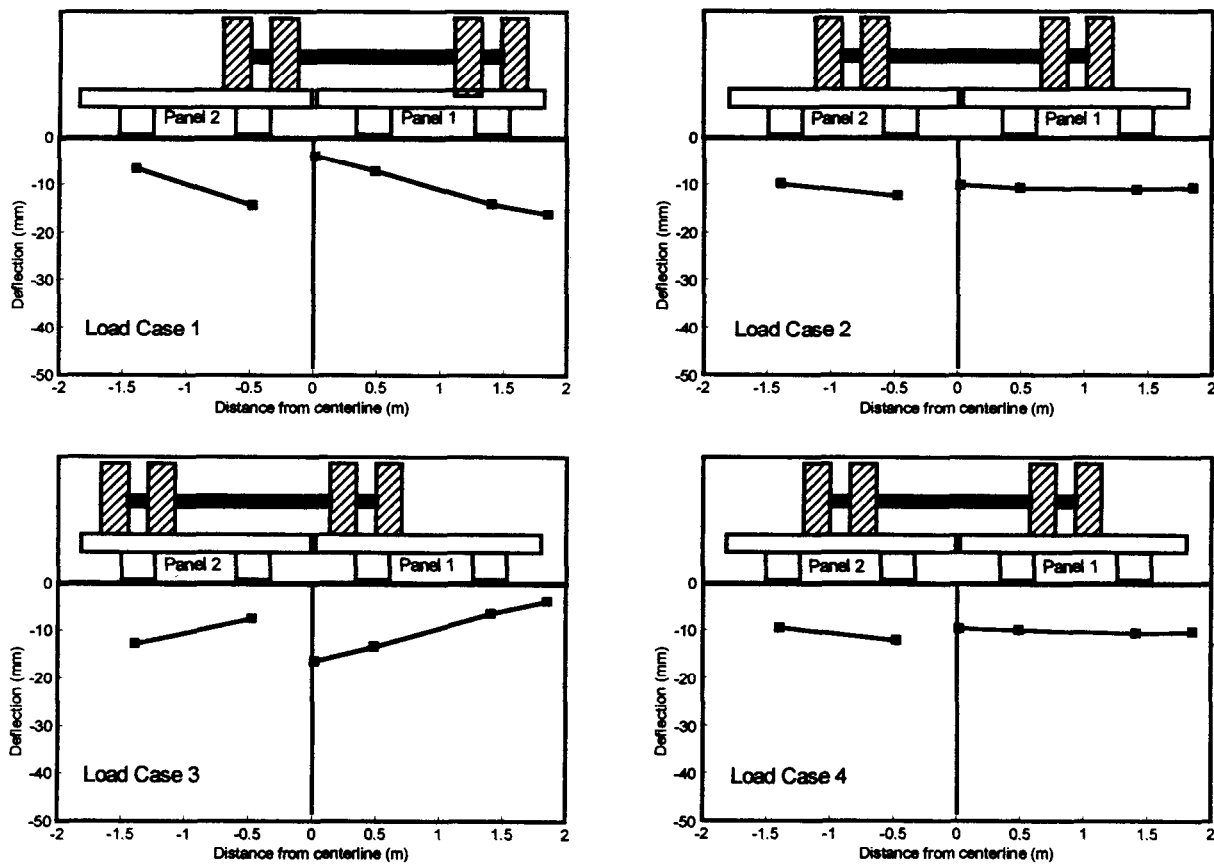


Figure 6. Transverse deflection for the load test of the Georgia Pacific bridge measured at the bridge centerspan (looking north). Bridge cross-section and vehicle positions are shown to aid interpretation and are not to scale.