Portable Timber Bridge Systems for Forest Roads


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PORTABLE TIMBER BRIDGE SYSTEMS FOR FOREST ROADS

ABSTRACT
Interest in portable bridge systems has increased in the U.S. due to heightened awareness for reducing environmental impacts and costs associated with road stream crossings. This paper presents case studies of portable bridges constructed with longitudinal glued-laminated timber decks and it discusses appropriate levels of strength design criteria for portable bridges. Specific emphasis is given to quantifying the effects of dynamic loads on portable bridge design.

INTRODUCTION
There is considerable interest in the U.S. for portable bridge systems designed for forestry and related natural resources industries. Access to our forests and other natural resources requires an extensive roadway network over a wide range of geographical conditions. 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 forest road stream crossings were the most frequent sources of erosion and sediment introduction into streams. Several studies have documented significant increases in sediment levels downstream from stream crossings (Thompson et al., 1995; Thompson et al., 1996; Tornature et al., 1996; Welch et al., 1998; White Water Associates, 1997).

Of the main types of stream crossing structures, fords, or low-water crossings, are often least expensive but they appear to have the greatest impacts on water quality. During ford construction, sediment is introduced into streams. When vehicles drive through streams they introduce additional sediment. Also, without properly designed water diversion devices around fords, sediment-laden runoff flows down road approaches and directly into streams. While some problems with fords are alleviated by culverts, considerable sediment loads can be introduced into streams during excavation and fill work that accompanies culvert installation. Also, culverts may clog with debris and may be washed out during heavy runoff periods, thereby introducing additional sediment into streams. In the case of temporary roads, stream crossing structures may be removed after activities are complete. Removal of culverts or fords also introduces heavy sediment loads into streams.

Bridges for forest roads can be permanent or temporary. Permanent bridges, which are typically designed for service lives of 40 to 50 years, are not economically feasible for short
use periods frequently encountered in forest operations. Also, permanent bridges for low-volume forest roads are commonly designed to lower standards than most public access facilities and can be a potential liability to bridge owners if public access is possible. One solution to short-term bridge needs is the concept of portable bridges. Blinn et al. (1998) and Mason (1990) summarized portable bridges available in the US. If properly designed and constructed, portable bridges can be easily transported, installed, and removed for reuse at multiple sites. The ability to serve multiple installations makes them more economically feasible than permanent structures. In addition, if they are installed and removed so that disturbance to the site is minimized, they alleviate many water quality and other potential environmental problems. Thompson et al. (1995) and Tornature et al. (1996) reported that proper installation of portable bridges could significantly reduce levels of sediment introduced into streams compared to other crossings. In addition to their environmental sensitivity and cost effectiveness, portable bridges provide forest operations planners with additional structures that can be used to bypass existing stream crossings that are structurally or functionally deficient. For example, loggers can temporarily place portable bridges over existing bridges that are unsuitable for carrying heavy truck loads. The portable bridge is used to support log truck traffic while harvesting occurs, then at the completion of the operation, the portable bridge can be removed for use at another site.

Many of the advantages of timber bridges make them ideal for temporary stream crossings. This paper reviews two case studies of portable glued-laminated timber (glulam) bridges and it discusses appropriate levels of strength design criteria for portable bridges. Specific emphasis is given to quantifying the effects of dynamic loads on portable bridge design.

PORTABLE TIMBER BRIDGE CASE STUDIES
To illustrate the use of portable timber bridges in the U.S., two case studies are presented here. The first bridge is a longitudinal glued-laminated deck bridge designed for use by log skidders. The second bridge is a prototype longitudinal glued-laminated deck bridge designed for use by log trucks.

GLULAM BRIDGE FOR SKIDDER TRAFFIC
Design
Taylor et al. (1996) described a longitudinal glulam deck bridge designed for wheeled log skidders used in forest harvesting operations. A photograph of the bridge is shown in Figure 1. The design vehicle was a 15,454 kg (34,000 lb) skidder with a 3 m (10 ft) wheelbase. This bridge consists of two Combination 48 (AITC, 1993) glulam panels 1.2 m (4 ft) wide, 216 mm (8.5 in.) thick, and 8 m (26 ft) long. The bridge panels were not intended to be interconnected; therefore, each panel was designed to carry one wheel line of the vehicle. No curb or rail was used in this design. The panels were preservative treated with creosote to a retention of 194 kg/m³ (12 lb/ft³) in accordance with American Wood Preservers’ Association (AWPA) Standard C14 (AWPA, 1991).
After the deck panels were preservative treated, 6 mm (0.25 in.) thick steel plate was attached to the ends and sides of the panels to prevent damage from skidder grapples. Also, a steel lifting bracket with chain loops was attached at the center of each panel to facilitate loading and unloading by typical knuckleboom loaders. Instead of using bolts or lag screws to attach the steel hardware to the glulam panels, 19 mm (0.75 in.) diameter steel dowels were placed through the glulam panels, welded to the steel plate, and then ground flush. This method of attachment eliminated exposed bolt heads that could be damaged during skidding operations. All steel plate, angles, and dowels conformed to ASTM A36 or ASTM A307 (ASTM, 1999). Since this bridge was projected to have a service life of approximately 10 years, steel hardware was not galvanized.

**Installation and Removal**
The glulam panels can be placed directly on stream banks without being interconnected. The bridge panels are placed by using the skidder’s grapple to pick up the panels, back over the stream, then lower them onto the stream banks as shown in Figure 1. The panels can also be winched into place. A gap is left between the panels so that the wheel lines of skidders match the center line of each panel. Logs can be placed between panels to prevent excessive debris from falling into streams during skidding operations. The bridge can be installed typically in less than 1 hour by on person. This includes skidding the bridge to the stream crossing site, placing the bridge deck panels, placing logs between the panels, removing the panels and logs, and skidding the panels back to a loading area. Since stream channels and stream banks are usually not disturbed during installation, there are little or no impacts on stream water quality.

**Cost**
Initial cost of the finished prototype glulam panels (including preservative treatment, installation of steel hardware, and delivery to the job site) was $9,300. Recent estimates for fabricating the bridge are approximately $8,000. Based on the actual deck area of 19.3 m² (208 ft²), the cost would be approximately $414/m² ($38/ft²). Installation and removal costs are conservatively estimated at approximately $165 per site. Using these data, if the bridge was installed at 50 different sites, the cost per site would be $325.

**Performance**
Taylor et al. (1996) provided a more detailed discussion of initial bridge performance. Overall, the bridge has performed well even though it has been subjected to wear typical of logging operations. The bridge has been well received by forest landowners and loggers because it is easy to install and remove and it reduces environmental impacts at stream crossings. Loggers that have used the bridge indicated that they preferred to use it over bridges with steel decks, because their skidder tires tended to slip less on the timber deck. Further refinement of the design may make its cost more competitive.

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2 All cost data are presented in US dollars.
T-SECTION GLULAM BRIDGE FOR TRUCK TRAFFIC

Design

The longitudinal deck timber bridge design discussed previously has been limited to spans of approximately 9 m (30 ft) due to practical limitations on thickness of the glulam deck panels. However, more efficient technology is needed to allow use of portable timber bridges on spans up to 15 m (50 ft). Therefore, Taylor and Ritter (1996) presented a longitudinal glulam deck bridge constructed in a double-tee cross section. A photograph of the bridge is shown in Figure 2.

The bridge consists of two longitudinal panels 10.7 m (35 ft) long and 1.8 m (6 ft) wide giving a total bridge width of approximately 3.6 m (12 ft) as shown in Figure 3. The design vehicle for the bridge was an AASHTO HS20 truck (AASHTO, 1993) with no specified deflection limitation. The panels are not interconnected; therefore, each panel carries one wheel line of the design vehicle. The panels were designed to be placed side by side on a timber sill, which can be placed directly on stream banks. Each panel was constructed in a double-tee cross section with dimensions given in Figure 3. Vertically-laminated flanges were 171 mm (6.75 in.) thick, 1.816 m (71.5 in.) wide, and were fabricated using No. 1 Southern Pine nominal 50 mm 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 mm by 305 mm (2 by 12 in.) lumber that met specifications for 302-24 tension laminations (AITC, 1993). 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 timber sill.

Interior wood diaphragms measuring 286 mm (11.25 in.) wide and 210 mm (8.25 in.) thick were provided between the webs at three locations along the length of the panels: one at each end, and one at midspan. In addition, to provide additional strength in the weak axis of the flange, 25 mm (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 panel third points. Additional reinforcing bars were placed horizontally through the diaphragms near the panel ends.

Curb rails were attached to steel angles, which were bolted to the outside edges of each flange. Rails were a single 140 mm (5.5 in.) deep, 127 mm (5 in.) wide, and 10.1 m (33 ft) long Southern Pine Combination 48 (AITC, 1993) glulam beam running the length of the bridge. For economic considerations, the curb was intended only for delineation purposes and was not designed as a structural rail.

A wearing surface was not provided on the bridge. However, steel angle was attached 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 steel plate 6 mm (0.25 in.) thick was bolted to the exposed end face of each web.
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 steel plate flange welded to one end. The eyes were installed in holes drilled through the bridge deck flanges and attached using 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 crane, loader, or backhoe. To assist in lifting and securing the panels at the site, additional steel plates, with chain loops welded to the plate, were bolted to the ends of each panel. 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.

Steel hardware was installed on the finished deck panels before they were shipped from the laminating plant. Deck panels were then shipped to a treating facility where they were preservative treated with creosote to a retention of 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 steel hardware and did not affect preservative penetration or retention in the wood. Installation of hardware before shipping to the treating facility allowed the finished bridge to be installed with no further fabrication or assembly on the part of the bridge owner.

**Installation**

Installation of the bridge was accomplished by using a crawler tractor to winch the panels into place. The panels were dragged to the site by the tractor. Then, the tractor crossed the stream and positioned itself on the opposite stream bank. After attaching a cable to the tractor and the bridge panel, each panel was winched across the stream. After positioning the panels, a timber sill was pushed under the ends of the panels using the tractor. Clearing the stream banks and placing the panels was completed in approximately 4 hours. After the panels were in place, wire ropes were secured to the chain loops at each of the bridge corners and to nearby trees. Bridge removal can be accomplished in a manner similar to installation. Another bridge of the same design has been installed using a backhoe to unload the bridge panels from a truck, carry them to the stream bank, and set them in place.

**Cost**

Cost for the materials, fabrication, treating, and shipping of the glulam bridge, timber sills, and cables was $14,825. Based on a deck area of 39.0 m$^2$ (420 ft$^2$), the cost was approximately $380/m$^2$ ($35/ft^2$). At the first installation of the bridge, the cost for labor and equipment to install and remove the bridge was approximately $1,095. Therefore, 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 timber sill, the total cost to install the bridge at 10 sites is $25,775 or $2,578 per site.

**Performance**

Taylor and Ritter (1996) provided a detailed discussion of a testing and monitoring program for the bridge. Overall, the bridge design has performed well. Although checks
have developed in the web laminations, these checks do not appear to have affected the structural adequacy of the bridge. The bridge was purchased for demonstration purposes and consequently, it has been used in a single location. Other bridges of the same design have been used successfully on multiple sites.

**STATIC VERSUS DYNAMIC DESIGN LOADS FOR PORTABLE BRIDGES**

Traditional design procedures for bridges account for both strength and serviceability criteria. Strength criteria dictate design loads or forces the bridge should safely support. Serviceability criteria mainly include prescribed deflection limitations for the bridge. Taylor et al. (1995) presented a matrix of general design criteria for portable bridges used on low-volume roads. Since portable bridges for forest roads do not typically use an additional wearing surface, the deflection limitations are not as critical as in highway bridge design. However, strength or load criteria are obviously of utmost concern in any bridge design.

**STATIC DESIGN LOADS**

Design vehicles for low-volume forest roads include highway vehicles and off-highway vehicles. In the matrix presented by Taylor et al. (1995), they indicated that some bridges may be designed specifically for off-highway vehicles such as wheeled-skidders or crawler tractors. Also, for bridges carrying truck traffic, they indicated that bridges should be designed for standard hypothetical design vehicles such as the AASHTO HS20 truck. Because portable bridges are needed for a wide variety of conditions and vehicle types, it seems appropriate to develop bridge designs for a specific set of design vehicles, such as the standard AASHTO trucks, which are used for the design of most highway bridges in the US. Therefore, research currently underway by Auburn University and the USDA Forest Service is developing methodology to determine the loads exerted by the various types of forestry equipment and then find an equivalent AASHTO truck that would result in similar levels of shear forces and bending moments. For example, a wheeled grapple skidder with an operating weight of 15,000 kg (33,000 lbs) and a wheelbase of 3.5 m (11.5 ft) may apply shear forces and bending moments similar to those of an AASHTO HS20 truck for bridge spans ranging from 3 m to 15 m. Therefore, when specifying the bridge requirements, the forest operations manager simply needs to specify that the bridge design vehicle is an AASHTO HS20 truck rather than a specific forest machine. When this work is completed, it will help forest operations personnel specify and or design portable bridges that fit a broad range of applications.

**DYNAMIC DESIGN LOADS**

The current methodology for designing timber bridges in the US is based on using static axle and wheel loads of the design vehicle. However, Wipf et al. (1996) presented research that showed that behavior of bridges under dynamic loading could be quite different from that under static loading. They measured the dynamic bridge response to trucks traveling at various speeds by recording deflection of the bridges with a high speed data acquisition system. They found that bridge deflections under dynamic loads were higher than those under static loads. Therefore, actual forces applied to the bridge by a
moving vehicle were greater than those assumed in a simple static analysis. The magnitudes of these forces are influenced by factors such as vehicle characteristics, bridge characteristics, vehicle speed, road conditions, and bridge entrance conditions. For example, if a truck encounters a large bump before driving onto the bridge, it will induce a new vibration mode in the truck (i.e., the truck will begin bouncing and pitching) and when the vehicle crosses the bridge, this will lead to higher dynamic loads than would have been observed in a static condition. Therefore, as part of the ongoing cooperative research by Auburn University and the USDA Forest Service, field tests were conducted to document the dynamic effects of vehicle loads and determine if design procedures need to account for these effects. The testing procedures and results are summarized next.

**Descriptions of Test Bridges**
The two bridges described earlier in the case studies were used for the dynamic tests. For the portable skidder bridge, the bridge was installed in a location specifically for the tests. A pit was constructed so that the bridge panels could be placed as if they were crossing a stream. The pit was constructed sufficiently deep to allow placement of the panels on timber sills and to allow placement of deflection sensors underneath the bridge panels. The pit measured 8.2 m (27 ft) long by 4.3 m (14 ft) wide by 0.6 m (2 ft) deep. After preparing the pit, timber sills measuring 127 mm (5 in.) thick by 457 mm (18 in.) wide were placed on the soil surface with a clear span between the inside edges of the sills of 7.6 m (25 ft). Each of the bridge panels were then placed on the timber sills with a gap between the panels of 0.9 m (3 ft) as in a typical bridge installation. Approaches to the bridge were leveled with a motorgrader before testing began.

The T-section truck bridge was tested as it was installed two years earlier. The ends of the T-section deck flanges were placed on timber sills that were laid on the stream banks as shown in Figure 2. However, due to the relatively short distance between the stream banks, the T-section webs also were resting on the banks. The distance between the edges of the bearings was approximately 6.9 m (22.5 ft).

**Instrumentation**
Dynamic response of the bridge panels was recorded during repeated passage of a wheeled skidder over the skidder bridge and a three-axle truck over the T-section bridge. Deflections of the bridge panels were measured at midspan and at locations immediately adjacent to the bearings using Celesco direct current displacement transducers (DCDTs). At each of the three transducer locations (i.e., midspan and at each bearing), multiple transducers were placed across the width of the bridge panels. For the skidder bridge tests, three DCDTs were placed across the width of the each panels: one DCDT positioned under the panel centerline and two DCDTs positioned 150 mm (6 in.) from the outside edges of the panel. For tests of the T-section bridge, four DCDT’s were placed across the width of each of the panels: two DCDTs positioned under the centerline of each of the webs and two DCDTs positioned 75 mm (3 in.) from the outside edges of the flanges. A PC-based data acquisition system was then used to record the deflection values from each DCDT. Data were recorded at a rate of 45 Hz.

**Test Procedures**
The dynamic deflection behavior for both bridges was determined using two different vehicles operating at three different speeds with two bridge entrance conditions. A Caterpillar 525 wheeled grapple skidder was used as the test vehicle for the skidder bridge. This skidder had an operating weight of 15,331 kg (33,800 lbs) and a wheelbase of 3.5 m (11.5 ft). The skidder was operated without carrying any logs. Axle weights were 8664 kg (19,060 lb) and 6686 kg (14709 lb) for the front and rear axles, respectively. Test runs of the skidder were made at 9.2 kph (5.7 mph), 13 kph (8 mph), and 23.5 kph (14.6 mph) with the wheel lines of the skidder centered over the longitudinal axis of the bridge panels. An artificial rough bridge approach was created by forming an earthen bump measuring approximately 200 mm (8 in.) wide by 100 mm (4 in.) thick and placing it approximately 300 mm (12 in.) away from the end of the bridge panel. Five test runs were made at each speed and entrance condition for each bridge panel.

A tandem-axle flatbed truck carrying a crawler tractor was used for tests of the T-section bridge. The truck had a wheelbase of 5.0 m (16.4 ft) and a gross weight of 17,600 kg (38,720 lbs) with front and rear tandem axle weights of 4163 kg (9160 lbs) and 13,436 kg (29,560 lbs), respectively. Test runs of the truck were made at 8 kph (5 mph), 16 kph (10 mph), and 24 kph (15 mph) with the wheel lines of the truck centered over the longitudinal axis of the bridge panels. An artificial rough bridge approach was created by placing a sawn timber measuring 200 mm (8 in.) wide by 100 mm (4 in.) thick and placing it approximately 300 mm (12 in.) away from the end of the bridge panel. Ten test runs were made at each speed and entrance condition for each panel; however, safety considerations prevented the truck from testing the rough approach at the 16 kph (10 mph) and 24 kph (15 mph) speeds.

To obtain a reference condition for comparison with the dynamic deflection values, static load tests were conducted for both bridges. In these tests, the vehicles were positioned on the bridge to obtain the maximum bending moment, then deflection values were recorded for all DCDTs.

**Data Analysis**

Plots of bridge deflection versus time were created for each DCDT in each test run and the static load tests. Since the bridges were installed on the timber sills, which were placed directly on uncompacted soil, measurable deflection was recorded by the DCDTs at the bearings. Therefore, to obtain a true net deflection value at midspan, the deflection readings from the bearings were subtracted from the midspan deflection values. From these methods, the net dynamic bridge deflection was determined.

Using the dynamic and static deflection data, a Dynamic Amplification Factor (DAF) was determined for each data stream recorded for each DCDT located at the midspan of the bridge. The DAF is found by:

\[
\text{DAF} = \frac{\text{Maximum Dynamic Deflection}}{\text{Static Deflection}}
\]
The DAF is a relatively simple term to help quantify the magnitude of dynamic bridge loads relative to static loads exerted by a given vehicle.

**Dynamic Test Results and Discussion**

Examples of typical dynamic deflection plots are shown in Figures 4 and 5 for the skidder bridge and the T-section bridge, respectively. These figures contain example plots for various speeds and entrance conditions.

Several interesting points are found in the plots. First, the plots are good illustrations of how bridge deflection can increase due to dynamic loads exerted by moving vehicles. In the plots shown, the dynamic deflection values are considerably greater than the values recorded in static load tests of the bridges. This increase in deflection indicates that the bridges are experiencing levels of dynamic loads that are considerably higher than those exerted by a static vehicle.

The plots also illustrate that in the dynamic bridge - vehicle system, there are two primary vibration modes present. The first mode is a relatively high-frequency, low-amplitude vibration that is the fundamental vibration mode of the bridge deck panel. The second mode is a relatively low-frequency, high-amplitude vibration mode due to the bounce and pitch of the vehicle. The plots of dynamic deflection for test runs with rough approaches show that the amplitude of vehicle vibration is significantly greater when the vehicle encountered the rough bridge entrance condition. Similar results would be observed if the road or trail conditions leading to the bridge were rough. The rough entrance conditions tested were not different from what could be found in many portable bridge installations on temporary skid trails or forest roads. Therefore, the deflection plots indicate that, for bridges installed on skid trail and forest roads, it would not be unusual to see significantly greater dynamic loads than the static load levels customarily used for design purposes.

The deflection data from each test run were used to calculate values of DAFs. Tables 1 and 2 include summary statistics for DAFs determined for the skidder bridge and the T-section bridge tests, respectively. Also, Figures 6 and 7 contain relative frequency histograms of the DAF data for the skidder bridge and the T-section bridge, respectively. The data in these tables and figures indicate that, overall, the levels of bridge deflection resulting from the dynamic loads were greater than those measured in static tests; i.e. all mean values of DAF were greater than 1.0. Also, although the mean values of DAF are not considerably different for different speeds, there are differences in the shapes of the histograms for DAF for different vehicle speeds. In the case where rough entrance conditions exist, the dynamic forces are considerably greater than static levels. Therefore, the moving vehicle exerts considerably more force on the bridge than we would assume with a static analysis.

The tables contain quantile information that may be used to modify our current design procedures. For example, to safely consider the effects of actual dynamic loads in the design of a portable bridge, we might want to use a dynamic load adjustment factor to
increase the traditional static vehicle loads. Also, for structural design, we typically choose design values for loads or material properties that are in the upper or lower tails of the respective probability distribution. Similarly, to choose an appropriate value of DAF for use in adjusting static design loads, we should consider a value in the upper tail of the distribution of DAF. If, for example, we choose the 90th percentile value of DAF for the dynamic amplification factor, this would result in increasing the current static design loads by a factor of approximately 1.6. This would be a significant increase over the design loads that are currently being used; therefore, additional study is necessary before recommending a final value for a dynamic amplification factor. However, based on the test data presented here, it is clear that the bridge designer should not ignore the higher levels of vehicle loads due to dynamic effects.

**SUMMARY**

Portable bridges are experiencing increased use in the U.S. due to several factors. These factors include pressure to reduce environmental impacts at forest road stream crossings, pressure to reduce construction and maintenance costs for stream crossings, and the desire to find innovative ways to access forest resources over inadequate roads and bridges.

Several different types of portable bridges are currently being used. New designs of portable timber bridges have proven to be cost effective and environmentally sensitive stream crossing structures. Two successful portable bridge case studies were presented in this paper: 1) a longitudinal glulam deck bridge designed for off-highway vehicles such as log skidders, and 2) a unique longitudinal T-section glulam deck bridge designed for log truck traffic. Both bridges have been used successfully in temporary forest road applications.

Finally, information was presented on dynamic loading effects on portable bridges. Data collected in load tests of the two glulam bridges indicate that vehicle loads and the resulting bridge deflections under moving vehicle loads were significantly greater than those observed for static vehicles. Rough bridge entrance conditions resulted in the highest levels of dynamic load amplification. Therefore, when designing the portable bridge, the engineer needs to account for dynamic load effects produced by typical forestry vehicles. Additional work is needed to determine the appropriate level of this dynamic adjustment.
REFERENCES


Table 1. Summary statistics for the Dynamic Amplification Factor (the ratio of dynamic bridge deflection to static bridge deflection) from dynamic load tests of the skidder bridge.

<table>
<thead>
<tr>
<th>Speed</th>
<th>Smooth</th>
<th>Rough</th>
<th>Smooth</th>
<th>Rough</th>
<th>Smooth</th>
<th>Rough</th>
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<td>30</td>
<td>30</td>
<td>30</td>
<td>30</td>
<td>30</td>
<td>180</td>
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<td>1.22</td>
<td>1.22</td>
<td>1.36</td>
<td>1.2</td>
<td>1.64</td>
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<td>9.1%</td>
<td>5.6%</td>
<td>6.8%</td>
<td>11.8%</td>
<td>10.0%</td>
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<td>1.36</td>
<td>1.61</td>
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Table 2. Summary statistics for the Dynamic Amplification Factor (the ratio of dynamic bridge deflection to static bridge deflection) from dynamic load tests of the T-section bridge.

<table>
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<tr>
<th>Speed</th>
<th>Smooth Approach</th>
<th>Rough Approach</th>
<th>Smooth Approach</th>
<th>Rough Approach</th>
<th>Smooth Approach</th>
<th>Rough Approach</th>
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<td>Coefficient of Variation</td>
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<td>21.8%</td>
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<td>1.61</td>
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<td>1.95</td>
<td>1.53</td>
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Figure 1. Photograph showing the removal of the portable longitudinal glulam deck bridge for skidder traffic.

Figure 2. Photograph of finished installation of the T-section glulam deck bridge.
Figure 3. Sketch of the portable longitudinal glued-laminated T-section glulam deck bridge.
Figure 4. Typical plots of dynamic bridge deflection versus time for test runs of the skidder bridge. Various test speeds and entrance conditions are shown.
Figure 5. Typical plots of dynamic bridge deflection versus time for test runs of the T-section bridge. Various test speeds and entrance conditions are shown.
Figure 6. Relative frequency histograms of the Dynamic Amplification Factor from the tests of the skidder bridge.
Figure 7. Relative frequency histograms of the Dynamic Amplification Factor from the tests of the T-section bridge.