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Design Criteria for Portable Timber Bridge Systems: Static versus Dynamic Loads

by

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Summary:
Design criteria are needed specifically for portable bridges to insure that they are safe and cost effective. This paper discusses different portable bridge categories and their general design criteria. Specific emphasis is given to quantifying the effects of dynamic live loads on portable bridge design. Results from static and dynamic load tests of two portable timber bridges showed that dynamic loads can be significantly greater than static loads. Under smooth bridge entrance conditions, the mean dynamic bridge deflections were 1.13 times greater than static bridge deflections. Under rough bridge entrance conditions, mean dynamic bridge deflections were 1.44 times greater than static bridge deflections.

Keywords: Portable bridge, glued-laminated timber, design criteria, dynamic loads, forest road

DESIGN CRITERIA FOR PORTABLE TIMBER BRIDGE SYSTEMS: STATIC VERSUS DYNAMIC LOADS

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ABSTRACT

Interest in portable bridge systems has increased in the U.S. due to heightened awareness of the need to reduce environmental impacts and costs associated with road stream crossings. However, design criteria are needed for portable bridges to insure that they are safe and cost effective. This paper discusses different portable bridge categories and their general design criteria. Specific emphasis is given to quantifying the effects of dynamic live loads on portable bridge design. Results from static and dynamic load tests of two portable timber bridges demonstrated that dynamic loads can be significantly greater than static loads. Under smooth bridge entrance conditions, the mean dynamic bridge deflections were 1.13 times greater than static bridge deflections. Under rough bridge entrance conditions, mean dynamic bridge deflections were 1.44 times greater than static bridge deflections.

INTRODUCTION

There is considerable interest in the U.S. for portable bridge systems designed for forestry and related natural resources industries as well as other more traditional applications, such as for military or construction uses. 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.

Access to our forests and other natural resources requires an extensive roadway network over a wide range of geographical conditions. Environmental concerns are the primary reason for the current interest in portable bridges used in forestry and natural resource applications. 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. Taylor et al. (1999) reviewed several studies that documented significant increases in sediment levels downstream from stream crossings.

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 Tomatore et al. (1996) reported that proper installation of portable bridges could significantly reduce levels of sediment introduced into streams compared to other crossings. In

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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 can be 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. However, to insure that the portable bridge is cost effective and is designed with adequate levels of safety, research is needed to accurately characterize design loads. The objective of this paper is to quantify vehicle live loads on typical designs of portable timber bridges. Emphasis is placed on quantifying the effects of dynamic loads on portable timber bridges.

BACKGROUND ON PORTABLE BRIDGE DESIGN CONSIDERATIONS

Design procedures for timber bridges used in permanent highway applications in the United States can be found in the American Association of State Highway and Transportation Officials (AASHTO) Standard Specifications for Highway Bridges (1993) and the design manual by Ritter (1990). Little previous research, however, has been conducted on appropriate design procedures for portable timber bridges. Knab et al. (1977) studied military theater-of-operations glulam bridges with design lives of 2 to 5 years. They concluded that using civilian design procedures, which are generally based on design lives of 50 to 75 years with relatively high levels of reliability, could result in unnecessarily conservative and uneconomical designs for the limited performance needs of temporary bridges. Using results from reliability analyses, they developed new design procedures and modification factors for allowable stresses that would result in adequate levels of structural safety for glulam girder bridges. They concluded that modification factors could be used to increase allowable bending, shear, and compression stresses for these temporary military bridges.

Other work by GangaRao and Zelina (1988) examined the design specifications for low-volume civilian roads. They concluded that the use of urban highway standards for low-volume road bridges results in overly conservative and uneconomical designs. They defined low-volume roads as those with maximum two-directional ADT of 200 vehicles or maximum two-directional Average Daily Truck Traffic (ADTT) of approximately 30 trucks per day. They suggested that allowable stresses for steel and concrete structures might be increased for such roads and that deflection limits might be relaxed for steel bridges. They did not recommend changing the deflection criteria of L/400 and L/300, where L is the bridge span, for low-volume concrete or timber bridges, respectively.

These research results of Knab et al. (1977) and GangaRao and Zelina (1988) indicate that applying AASHTO design procedures to portable bridges on low-volume roads may result in overly conservative designs. The designer must consider that in many cases, the design life of such a bridge may only be 5 to 10 years. Therefore, it may be possible to make changes such as increasing the load duration factor above the value of 1.15 that is currently specified by AASHTO.

Taylor et al. (1995) proposed a matrix of design criteria for different types of portable bridges, depending on their intended use. The matrix was developed for bridges used on three different types of roads: sub-low volume, low volume, and high volume. Franklin (1999) modified this matrix to include four classes of bridge applications: two cases of sub-low volume, and one case each of low-volume and high-volume roads. The matrix is presented in Table 1. The sub-low volume bridge, Case 1, would include bridges intended for use by forestry machines such as wheeled skidders. The sub-low volume bridge, Case 2, would include bridges intended for log truck traffic, with very low traffic rates as might be expected for a single logging operation. The low volume bridge would be intended for use on main forest roads with higher traffic rates. The high-volume bridge would be intended for use in highway applications where it was serving as a bypass for permanent bridges that were under construction.

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. The bridge design criteria outlined in Table 1 contain several differences from the criteria typically used for permanent bridges. 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. Table 1 indicates that some bridges may be designed specifically for off-highway vehicles such as wheeled-skidders. Also, for bridges carrying truck traffic, bridges generally 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 (Franklin, 1999). 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 (10 ft to 50 ft). 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

Glulam Bridge for Forestry Skidder Traffic.

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/m^3 (12 lb/ft^3) 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.

In typical installations, the glulam panels can be placed directly on stream banks without being interconnected. A gap is left between the panels so that the wheel lines of skidders match the center line of each panel. For the dynamic load tests of the bridge, it 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. Figure 2 shows the bridge as it was installed for the dynamic tests.

T-Section Glulam Bridge for Truck Traffic.

Taylor and Ritter (1996) and Morgan et al. (1999) presented a longitudinal glulam deck bridge constructed in a double-tee cross section. The bridge, shown in Figure 3, 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). The design vehicle for the bridge was an AASHTO HS20 truck 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 by Taylor and Ritter (1996). 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. All steel plate, angles, lag screws, and bolts conformed to ASTM A36 or ASTM A307. 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).

This bridge was tested as it was installed two years earlier near Moulton, Alabama. The ends of the T-section deck flanges were placed on timber sills that were laid on the stream banks. 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 tandem-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 Model PT101 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. Figures 4 and 5 show placement of the DCDTs for the skidder bridge and for the T-section bridge, respectively. 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 the 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 as shown in Figure 6. 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 (14,709 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 as shown in Figure 7. 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 using 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 deflections were 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.

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:

$$\mathrm{DAF} = \frac{\text{Maximum Dynamic Deflection}}{\text{Static Deflection}}$$

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 8 and 9 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. Figure 10 illustrates how, after the vehicle encounters a rough bridge entrance condition, the vehicle and its axles can bounce, which in turn can lead to greater loads on the bridge.

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 2 and 3 include summary statistics for DAFs determined for the skidder bridge and the T-section bridge tests, respectively. Also, Figures 11 and 12 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. For the tests of the skidder bridge, values of DAF ranged from 0.90 to 1.49 and 0.93 to 1.93 for the smooth and rough entrance conditions, respectively. Mean values of DAF were 1.15 and 1.41 for the smooth and rough entrance condition tests of the skidder bridge, respectively.

For the T-section bridge, values of DAF ranged from 0.81 to 1.95 and 1.22 to 2.06 for the smooth and rough entrance conditions, respectively. Mean values of DAF were 1.19 and 1.53 for the smooth and rough entrance condition tests of the bridge, respectively.

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 95th percentile value of DAF for the dynamic amplification factor, this would result in increasing the current static design loads by a factor of approximately 1.65 or 1.7. However, upon a closer examination of the test conditions, the tests of the skidder bridge that were conducted at the higher speeds are not representative of common operating speeds for wheeled skidders. Therefore, if we only consider the tests conducted at speeds of approximately 8 kph (5 mph), the 95th percentile of DAF for both smooth and rough conditions is approximately 1.37, which would suggest the use of a DAF of approximately 1.4 for designing the sub-low volume bridges. After examining the data from the test of the T-section bridge, which would typically be used in the low or high volume bridge categories, the 95th percentile of DAF for all speeds and approaches was 1.64. Since it is conceivable for this bridge to experience all of the speeds and entrance conditions tested, it may be advisable to use the higher DAF value of approximately 1.6 for design purposes.

Using these suggested values of DAF to adjust the design loads 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 timber bridges are currently being used that have proven to be cost effective and environmentally sensitive stream crossing structures.

Current timber bridge design methods do not specifically account for dynamic effects from vehicle live loads. However, recent research on highway bridges indicated that dynamic loads can be greater than static loads. Therefore, to further refine design criteria for portable bridges, dynamic load tests were conducted on two portable timber bridges: one designed for forestry skidder traffic and one designed for log truck traffic.

Data collected in these dynamic load tests indicate that vehicle loads and the resulting bridge deflections under moving vehicle loads were significantly greater than those observed for static vehicles. For smooth bridge entrance conditions, mean bridge deflection values were 1.15 and 1.19 times greater than static deflections for the skidder bridge and the T-section bridge, respectively. Rough bridge entrance conditions resulted in the highest levels of dynamic load amplification. For the rough bridge entrance conditions, mean bridge deflection values were 1.4 and 1.5 times greater than static deflections for the skidder bridge and the T-section bridge, respectively. 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 for design procedures.

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Table 1. Suggested categories for portable timber bridges and their respective general design criteria.

Criterion	Case 1	Case 2	Case 3	Case 4
	Sub-Low Vol. A	Sub-Low Vol. B	Low Volume	High Volume
Design Life	5 years	5 years	10 years	25 years
Traffic Type	Off-highway/ Forestry Vehicles i.e. skidders	Off-highway/ Trucks i.e. log trucks	Trucks/ Light Vehicles	Unlimited
Average Daily Traffic	50	15	100	500
Design Speed	8 kph (5 mph)	8 kph (5 mph)	8 kph (5 mph)	40 kph (25 mph)
Load	Off-highway	HS 20	Off-highway	HS 20 or HS 25
Load Application Period	6 months	24 months	24 months	36 months
Deflection	none	none	none	AASHTO
Span Type	simple	simple	simple	simple

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 skidder bridge.

	9.2 kph Smooth	9.2 kph Rough	13 kph Smooth	13 kph Rough	23.5 kph Smooth	23.5 kph Rough	All Speeds Smooth and Rough
N	30	30	30	30	30	30	180
Mean	1.15	1.22	1.22	1.36	1.2	1.64	1.29
Coefficient of Variation	10.9%	9.1%	5.6%	6.8%	11.8%	10.0%	15.7%
Maximum	1.36	1.40	1.36	1.61	1.49	1.93	1.93
Minimum	0.90	0.93	1.09	1.14	0.92	1.24	0.90
90th Percentile	1.28	1.35	1.29	1.47	1.36	1.84	1.60
95th Percentile	1.31	1.37	1.34	1.49	1.37	1.88	1.71

Table 3. 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.

	8 kph Smooth Approach	8 kph Rough Approach	16 kph Smooth Approach	24 kph Smooth Approach	All Speeds and Approaches Combined
N	80	40	80	80	280
Mean	1.16	1.53	1.18	0.99	1.17
Coefficient of Variation	12.4%	11.8%	20.6%	19.6%	21.8
Maximum	1.61	2.06	1.95	1.53	2.06
Minimum	0.81	1.22	0.63	0.69	0.63
90^{th} Percentile	1.36	1.81	1.47	1.26	1.50
95^{th} Percentile	1.42	1.83	1.73	1.37	1.64

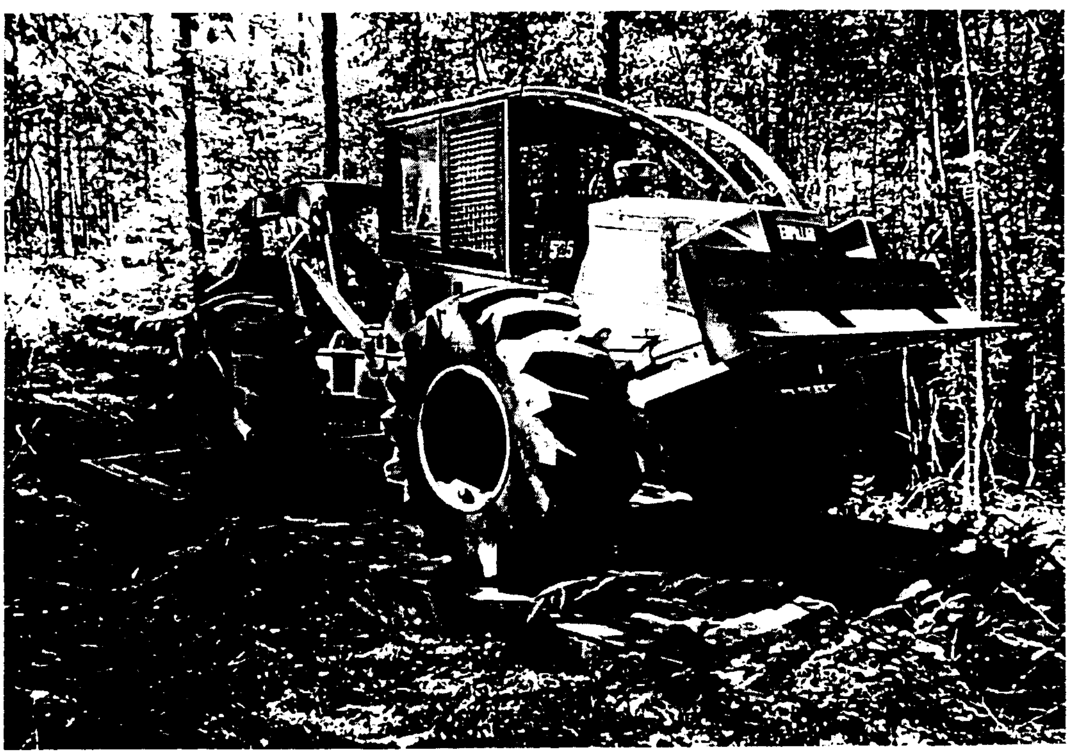

Figure 1. Typical installation of the portable longitudinal glulam deck bridge for skidder traffic.

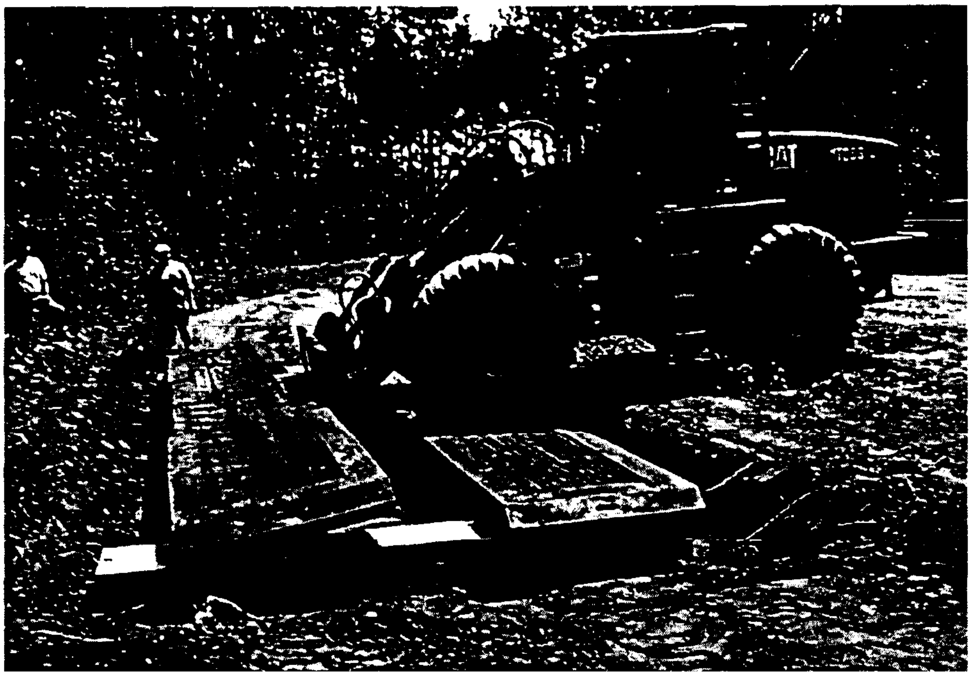

Figure 2. Portable glulam skidder bridge as installed for the dynamic load tests.

Figure 3. Finished installation of the T-section glulam deck bridge.

Figure 4. Placement of DCDT's under midspan of skidder bridge.

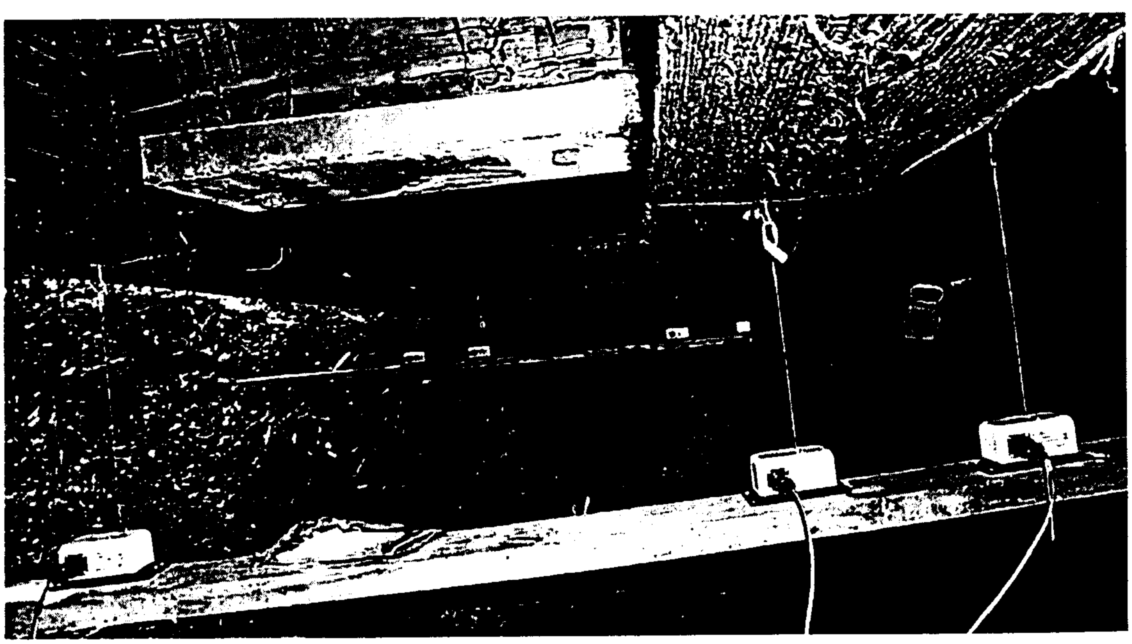

Figure 5. Placement of DCDT's under midspan and bearing of T-section bridge.

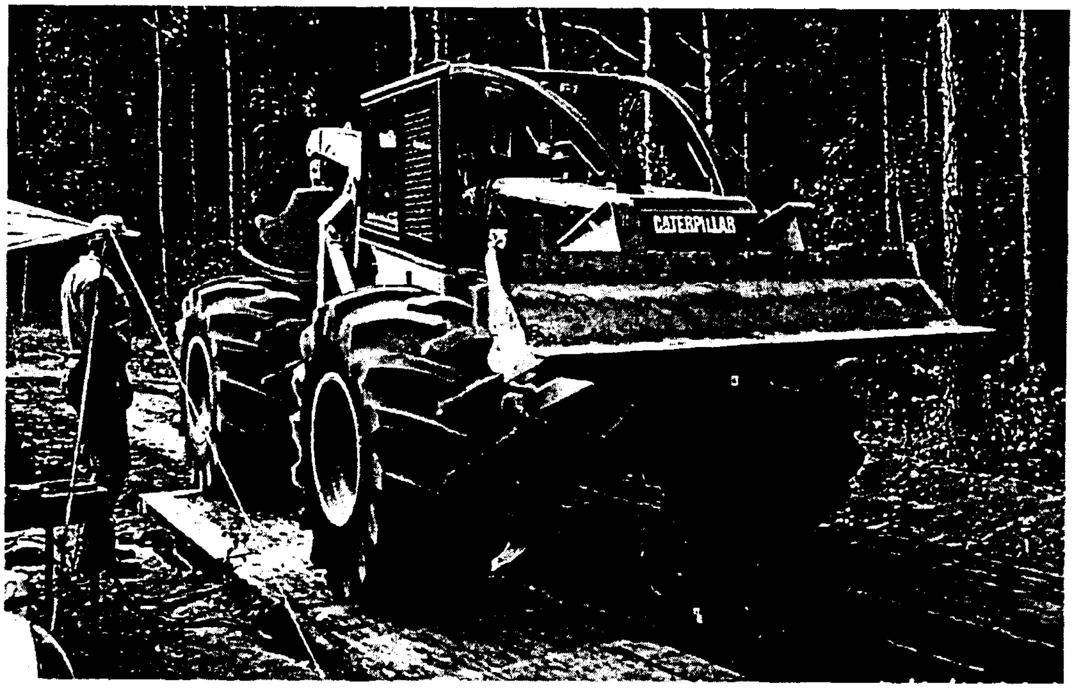

Figure 6. Wheeled skidder crossing bridge during dynamic testing of skidder bridge.

Figure 7. Tandem-axle truck crossing bridge during dynamic testing of the T-section bridge.

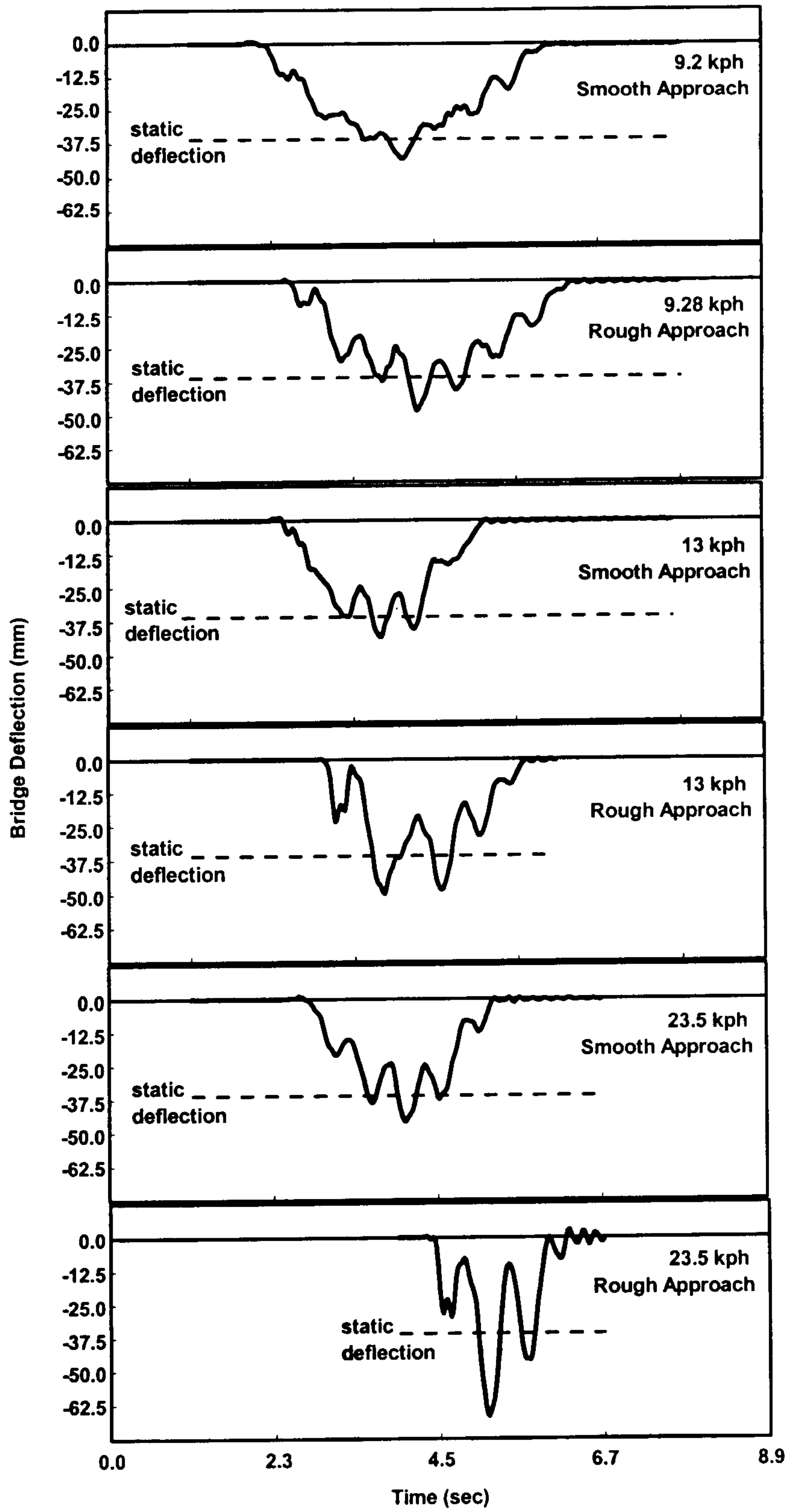

Figure 8. Typical plots of dynamic bridge deflection versus time for test runs of the skidder bridge. Various test speeds and entrance conditions are shown.

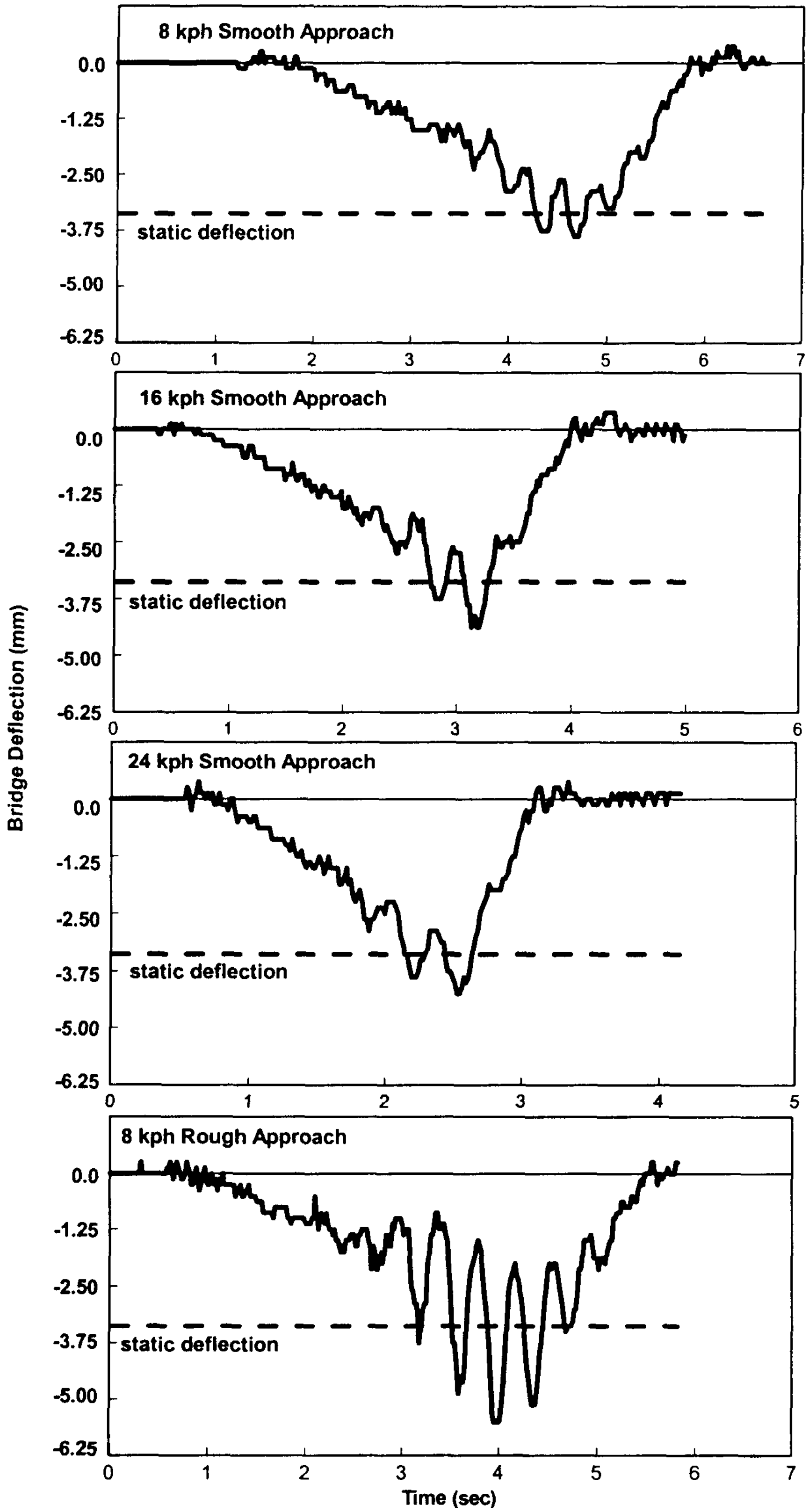

Figure 9. Typical plots of dynamic bridge deflection versus time for test runs of the T-section bridge. Various test speeds and entrance conditions are shown.

Figure 10. Example of one of the dynamic test runs over the T-section bridge with the rough entrance condition. The front axle of the rear tandem is not contacting the bridge deck.

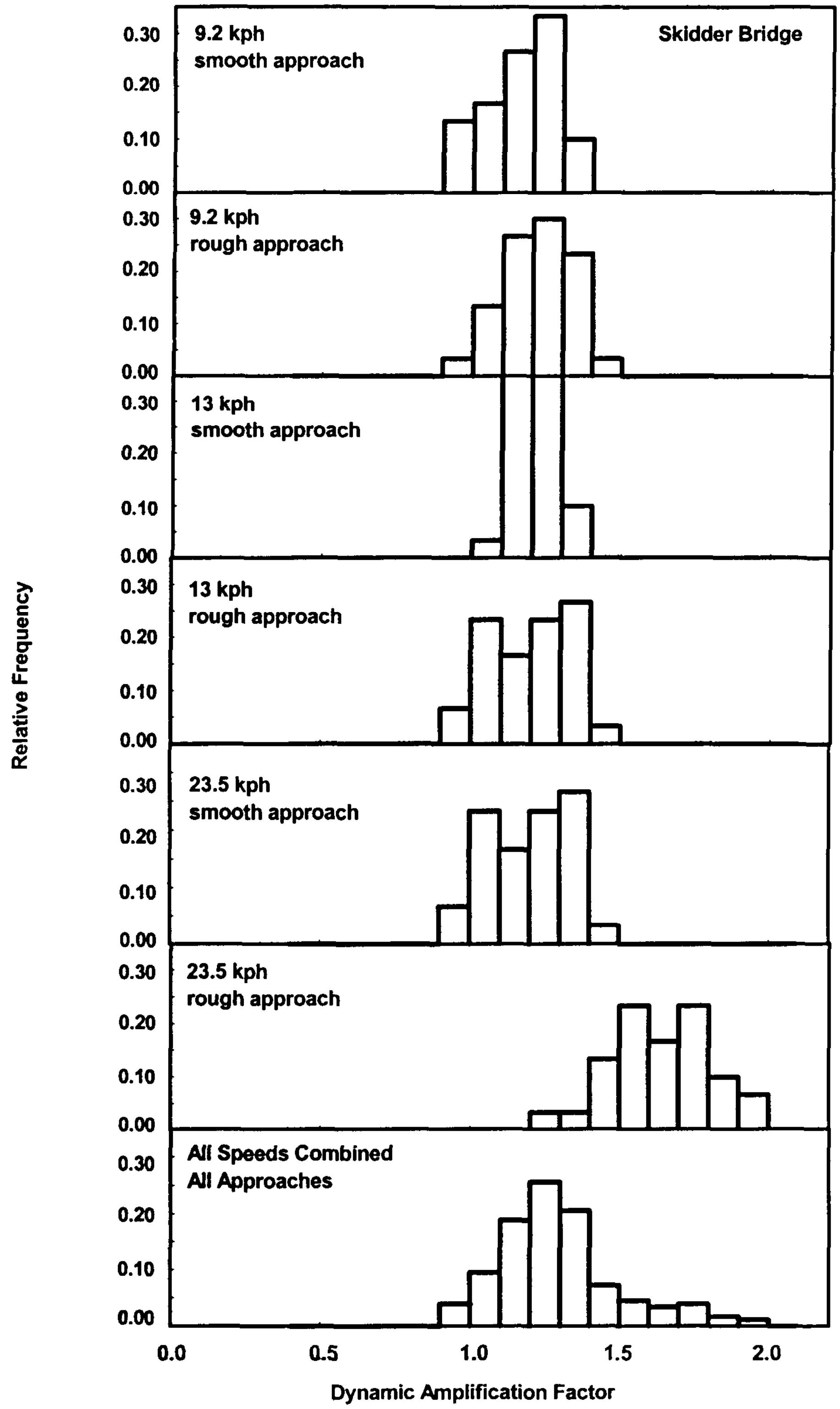

Figure 11. Relative frequency histograms of the Dynamic Amplification Factor from the tests of the skidder bridge.

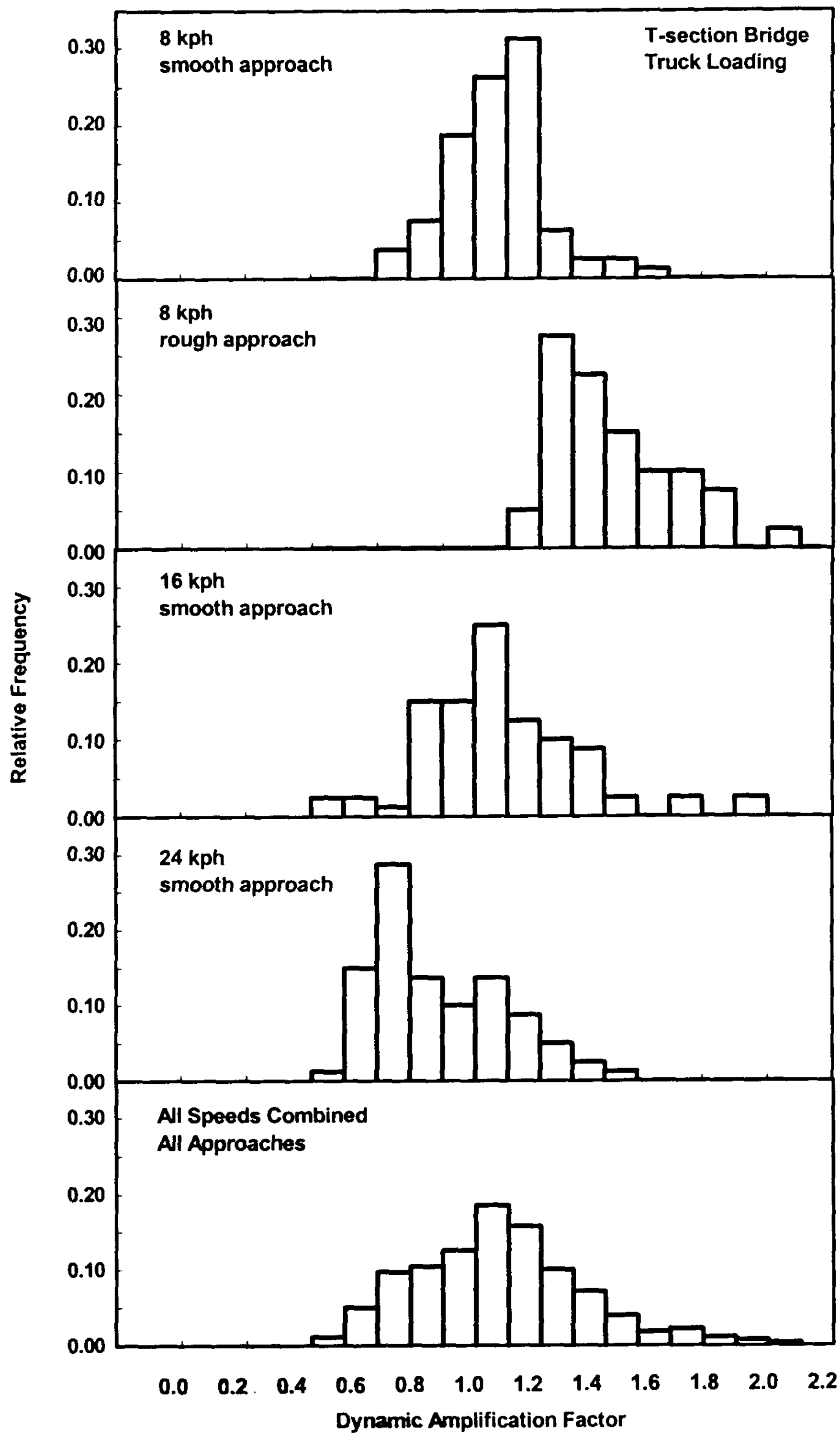

Figure 12. Relative frequency histograms of the Dynamic Amplification Factor from the tests of the T-section bridge.