United States
Department of
Agriculture

Forest Service

Forest Products
Laboratory

Iowa State
University

Dynamic Field Performance of Timber Bridges

Final Report: Summary and Conclusions

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Abstract

In order to better utilize and develop timber structures in transportation, the United States government implemented several national programs starting in the early 1990's. One specific need identified was investigating the dynamic field performance of timber bridges due to vehicular loading. Currently, the American Association of State Highway and Transportation Officials (AASHTO) recommend a dynamic load allowance of 0.165 for timber bridges. Previously however, AASHTO did not include a load allowance for timber bridges. To quantify the appropriate code values, research was needed to determine the dynamic characteristics of timber bridges and to study their dynamic performance with respect to time and bridge condition.
To fulfill this research need, five glued-laminated timber girder bridges and four longitudinal glued-laminated timber panel bridges were selected for testing. The testing involved loading the structures to obtain dynamic response data. The information collected relates to the dynamic deflection, acceleration, and overall condition assessment for all nine bridges. The results of the individual bridges are also compared with each other to determine the validity of the current AASHTO recommendations and to develop better design standards. In general, the nine bridges tested were found to have fundamental frequencies between 5 Hz and 11Hz as well as a dynamic load allowance less than 0.25. The bridges found to have dynamic amplifications above specified code values were also found to have physical characteristics (i.e., rough entrances) that caused the excessive dynamic amplification values.

Acknowledgments

We would like to express our sincere appreciation to the engineers and staff from the following organizations that assisted in the locating and structural monitoring of the nine glued-laminated timber bridges involved in this research:

Dane County Highway Dept., Engineer, and staff; Dane County, WI

Tuscaloosa County Highway Dept., Engineer, and staff; Tuscaloosa Co., AL

Chambers County Highway Dept., Engineer, and staff; Chambers Co., AL

Butler County Highway Dept., Engineer, and staff; Butler Co., AL

Franklin County Highway Dept., Engineer, and staff; Franklin Co., AL

Allegany Highway Dept., Engineer, and staff; Allegany Co., NY

Contents

Introduction

The development of engineered timber structures in transportation has seen an increase in the past decade. The primary reason for this increase is due to several national programs that have been implemented to develop and extend the use of timber bridges throughout the country by promoting timber through demonstrations and research. The 1988 Timber Bridge Initiative (TBI), passed by the U.S. Congress, was an important pilot program that initiated the increase. This legislation was implemented primarily to revitalize local economies in areas where rural communities depend upon natural resource management. There was also a push to start using the abundant supply of underutilized wood species for highway applications [13].

Although the TBI was the beginning of new research and design for timber bridges, the new era of timber technology actually began in the mid 1900's. The development of engineered timber and the refinement of preservatives enhanced the usage of timber as a building material. Currently, advancements in timber, such as standard design plans, construction standards, and full utilization of the advantageous material properties, have allowed timber to last longer, decrease construction time and cost, and increase span lengths. These factors have all been important in the renewal of the timber industry and making timber a competitive construction material.

In 1991, the United States Department of Agriculture (USDA) Forest Service/Forest Products Laboratory (FPL), in conjunction with the Federal Highway Administration (FHWA), again received significant funding under the Intermodal Surface Transportation Efficiency Act (ISTEA) to continue the research and technology advancements of timber structures. The program, now called National Wood in Transportation program (NWIT), is divided into three component program areas: construction of demonstration bridges, timber research, and technology transfer. The main goals of these three areas are to develop a better, more positive public awareness of timber, and to advance timber technology for future needs [13].

The timber bridge demonstration component of the NWIT program focuses on improving rural transportation infrastructure and stimulating local economies by using native wood species [12]. The demonstration bridges were also used to improve the perception of timber as a construction material. Currently, over 2,700 timber bridges, ranging from glued-laminated timber to timber arch bridges, have been built since 1988 through this program [13].

Due to previous lack of interest in and support of timber products, the design and construction technologies have not advanced as rapidly for timber as compared to steel and concrete. In order to accelerate the development of timber technologies, the research and technology transfer components of the NWIT program were established. Addressing the research and development needs of timber bridges allowed for newer designs and stronger, more durable products to be developed.

The joint research conducted by the FPL and FHWA has been divided into six categories. One of these is System Development and Design. This specific research area focuses on investigating the field performance of timber bridges to refine and improve design practices and current design codes. One identified research need is the investigation of dynamic field performance of timber bridges due to vehicular loading. To fulfill this need, Iowa State University (ISU), in cooperation with the FPL, has been involved in a three phase project to determine the dynamic characteristics for stress laminated bridges, glued-laminated girder bridges, and longitudinal glued-laminated panel bridges. As with bridges constructed of other materials, the design of all of these general bridge types must be completed with consideration of the dynamic forces imposed by passing vehicles. To account for this, a dynamic load allowance is typically applied to static live loads. Historically, the American Association of State Highway and Transportation Officials (AASHTO) has not included a load allowance for timber bridges due to the belief that the natural properties of timber resist most types of dynamic loading [1]. However, the current AASHTO- Load and Resistant Factor Design (LRFD) code recommends a dynamic load allowance (DLA) of 0.165, which is a 50% reduction of the load allowance for other bridge materials [2]. The three phase NWIT project focuses on collecting and analyzing field data that will be used to quantify the dynamic amplification factor (DAF), which correlates to 1+DLA, and general response for the three different types of timber bridge designs. These tests were designed to observe and study bridge deflections and vertical accelerations with varying vehicle velocities [16]. The information gained from the field testing can be used to not only quantify the DAF of the bridges, but to also allow for a better understanding of the overall dynamic behavior of timber bridges.

This report summarizes a series of nine individual reports that document the results of dynamic testing and performance of five glued-laminated girder bridges and four longitudinal glued-laminated panel bridges. All the bridges were field tested and inspected in 2003, with previous testing also taking place on three of the girder bridges and all four of the panel bridges in the mid 1990's. Each of these nine bridges are documented in detailed individual reports, entitled *Dynamic Field Performance of Timber Bridges 1-9* [25]-[33]. These reports describe the mid 1990 and 2003 collection and analysis of the bridge data, along with the results and relationships between DAF, frequency, and physical and structural condition. A brief discussion of each bridge, dynamic load allowance evaluation, and other bridge attributes found to affect the dynamic performance of the bridges are given herein.

Objective and Scope

The primary objective of this research was to determine the dynamic characteristics of timber bridges and to study their dynamic performance with respect to time and bridge condition. Comparisons with design code values will also be made.

Iowa State University, in conjunction with the FPL, has been investigating timber bridge dynamics, both analytically and physically, since the mid 1990's. However, very little research has been conducted on the dynamics of timber bridges elsewhere. To encompass all aspects of timber bridge dynamics and to obtain comprehensive conclusions, this research had three main objectives:

- to conduct physical bridge testing to study dynamic loading and response, including DAF, frequency content, and relationship to bridge condition.
- to verify codified dynamic load allowance criteria based on the field behavior of the timber superstructures tested.
- to correlate physical aspects of the bridge, including time effects and wearing surface conditions, with the obtained field data.

To satisfy these objectives, the project scope included three tasks: a literature review, field testing, and development of final conclusions. The literature review was conducted to learn about research that has been previously completed on the dynamics of bridges. By understanding what has been researched in the past, the current research can be built upon this without unneeded duplication. A brief summary of the literature review relating to bridge dynamic properties, wearing surface condition, and vehicle dynamic properties is presented in the next section. Field testing, conducted in both the 1990's and in 2003, focused on collecting behavior data and visually inspecting each bridge to document the condition for correlation with performance. The quantitative measurements and qualitative observations were then used for correlation studies and the development of the final conclusions and recommendations.

Literature Review

Although timber was once the material of choice for bridge engineers, the use of timber as a bridge material has been declining at a significant rate since 1982. This decline in use is mainly due to a misconception about timber bridge performance. Research conducted by Smith and Stanfill-McMillan found that the general perception amongst bridge engineers is that timber has shorter life, lower strength, higher maintenance, is environmentally unsafe, and is more difficult to design than other bridge materials. The research also found that this misconception mainly stems from observed sites where timber bridges are not designed to code standards and/or are not properly maintained. In areas where timber bridges have a high performance rating, the perception of timber as a building material is much better and have actually seen increases in the number of timber bridges being constructed [23]. These misconceptions have also resulted in the decline of timber bridge research and stagnated the development of technology within the timber industry. Dynamic evaluations of timber bridges, although limited, have been conducted both through field and analytical evaluations. A significant amount of research conducted on glued-laminated girder and glued-laminated longitudinal deck bridges took place at ISU and will be discussed in greater detail later in this report. However, the general findings from the ISU research concluded that the dynamic amplification factor was magnified when the vehicle was excited (i.e., body bounce and axle hop motion) from the conditions of the approach and/or entrance conditions of the bridge. The maximum DAF was also found to generally occur when the vehicle eccentrically loaded the bridge [9], [11], [24].

As previously stated, little research has been done exclusively on the dynamic response of timber bridges; however, since the 1950's, there has been an extensive amount of research conducted on bridge dynamics in general. McLean and Marsh stated in their synthesis, *Dynamic Impact Factors for Bridges*, that the "dynamic response of a bridge is the result of the modes of vibration of the bridge responding to the forcing function generated by a vehicle oscillating on its suspension system". Nearly all the research reviewed has concluded that the dynamic response of a bridge is highly dependent on the road surface roughness, vehicle characteristics, and the bridges characteristics. However, these three variables, along with several others, interact with each other making the dynamic characterization of a bridge very difficult [4], [5], [8], [14], [15], [18], [19], [20], [22]. McLean and Marsh, Bakht, and Paultre all state that due to this interaction of multiple variables, a simplistic correlation for bridge response cannot be obtained. Bakht goes on to state that the most efficient method for evaluating bridge dynamic behavior is to conduct full scale testing. Full scale testing, rather than analytical or laboratory evaluations, can quantify changes in the bridge performance and give instantaneous evaluation of the dynamic properties of the bridge. [4], [19].

Bridge dynamic characteristics obtained from testing primarily consist of the DAF, damping, natural frequencies, and forced vibrations. Although the bridge response due to dynamic loading is hard to obtain, several relationships for the listed characteristics exist. Several relationships relating span length to fundamental frequency have been established [8], [19]. The fundamental frequency has also been found to be heavily correlated with the DAF. However, this correlation also depends heavily on the frequency of the excitation force [5], [8], [22]. Due to these correlations, some bridge codes, including early AASHTO editions, recommended code values for DAF based only on the span length. Contradicting studies have found, however, that there is little to no correlation between DAF and bridge span. Other bridge codes have used a direct relationship between DAF and fundamental frequency. Most code values based on span length and frequency have now been simplified to a constant value [18].

Damping of a bridge has been found to be related to the specific material the bridge was constructed of, and is a function of geometry of the bridge. In general, long, straight, narrow bridges have lower damping than short curved bridges. Many studies show that damping does not affect the DAF of a bridge; however, contradicting research also states that damping does have an effect on the DAF [8], [18].

The vehicle properties and characteristics that primarily impact the dynamic response of a bridge are the tire stiffness, suspension, live load to dead load ratio, load position on the bridge, and initial conditions (e.g., body bounce and axle hop excitations) [5], [15]. The vehicle suspension type can significantly impact the dynamic response of the bridge. If the vehicle excitation frequencies match those of the bridge natural frequency, resonance vibration can occur. Air suspension systems have been found to have less of an impact on a bridge's DAF. This is due to the air suspension having lower natural frequencies than conventional leaf springs, the dynamic wheel loads were seen to be reduced, and higher viscous damping occurred [14]. It has also been noted that a vehicle loading a bridge eccentrically causes increased dynamic response [15], [24].

The wearing surface condition for both the approach and the bridge itself have been found to play a major role in how the vehicle gets excited and therefore excites the bridge. Undulations and irregularities in the approach surface essentially excite the vehicle creating the vehicle's initial dynamic conditions as it enters the bridge. The bridge wearing surface can then continue to stimulate the vehicle vibrations and dynamic forces. In general, the riding surface has been found to have a strong correlation with the DAF. Poor to medium pavement surfaces, especially on approaches, cause a significant increase in the DAF. This is important as not only does the pavement roughness excite the vehicle, but the vehicle's dynamic forces simultaneously increase the pavement roughness [3], [17].

The most severe impact wheel forces have been found to occur shortly after a vehicle enters the bridge. The source of the severe force is the initial conditions of the vehicle, which are highly influenced by the approach conditions [18].

Numerous factors such as wearing surface patches, misaligned expansion joints, snow/ice, and settlement have also been shown to cause vehicle excitation [5]. Several field tests were preformed by placing an artificial bump at the entrance of the bridge, where a rough approach was duplicated and the vehicles initial conditions were heavily excited. Results found that the DAF can be significantly higher due to an artificial bump [19]. It was also found that the bump not only excited the body bounce mode (2Hz to 5Hz), but also excited the axle hop mode (10Hz to 15Hz), therefore causing a second frequency with which the bridge resonated [8].

As one can see, the interaction between the bridge, the vehicle, and the road surface makes it very difficult to isolate each variable individually and creates a complex problem for analytical modeling of bridges. The interaction stems from the approach road surface causing the vehicle to "bounce" and create dynamic forces. The bridge response is then activated as the vehicle enters the bridge. The magnitude of the response depends highly on the vibrating frequency of the vehicle and the natural frequency of the bridge. Typically, bridges with natural frequencies in the range of 2 Hz to 5 Hz experience the largest amplifications. Depending on the surface roughness, however, the axle hop vibration can be excited and cause higher frequency bridges to have higher amplifications. When one also considers the less significant variables, not mentioned in detail, an extremely complicated interaction problem is created [5], [8], [15], [22]. Due to the complexity of the problem, many contradicting conclusions can be found in the technical literature.

Evaluation Methodology

The monitoring and evaluation plan for the bridges tested as part of this work were jointly developed by the FPL and ISU. The plan entailed investigating the moisture content, measuring bridge dynamic behavior, and visual inspections of typical bridges. Seven of the nine bridges were tested in both 1996 and in 2003 to allow for quantification of durability and change with time. The procedures/protocols followed in this investigation are described in the following sections.

Moisture Content

Moisture content is, generally, significant to the performance of timber bridges because as the moisture content changes the modulus of elasticity of timber changes. A change in moisture content can also lead to shrinkage or swelling of bridge components which can lead to damage of the structure if improperly designed.

In this study, moisture content data were only collected in 2003. A two prong electric resistance moisture meter was used for measuring the moisture at various locations on the underside of the subject bridges. The instrument prongs were 1.5 in. in length and fully driven into the timber components of interest.

Dynamic Behavior

In order to account for the load increase induced by the vehicle/bridge interaction, the DAF was evaluated for the bridges. When determining the DAF experimentally, midspan deflections are typically measured for both crawl and speeds up to the posted speed limit. The dynamic amplification, then, is the ratio defined as:

$$DA = \frac{\delta_{dyn} - \delta_{stat}}{\delta_{stat}} \qquad (1)$$

where δ_{dyn} = the maximum deflection of the vehicle traveling at normal speeds and δ_{stat} = maximum deflection of the vehicle traveling at crawl speeds. The amplification factor is then given by:

$$DAF = 1 + DA \qquad (2)$$

To determine the DAF for the bridges evaluated in this study, field load tests were conducted on the bridges. These tests were conducted with a fully loaded three axle dump truck that was driven over the bridge. Several passes were made with the truck starting with a crawl pass (< 5 mph) that was increased in 5 mph increments to the posted speed limit. Typically, a combination of three load cases were used to test each bridge. Figure 1 shows the three typical transverse concentric and eccentric load positions. Bridge behavior during the loading was recorded with string potentiometers located at the midspan of the spans of interest. The data from these potentiometers were collected by a data acquisition system (DAS) and stored on a PC laptop computer.

The frequency content is an important attribute for understanding the dynamic behavior of a bridge. As stated previously, the frequency can influence the DAF through interaction with the vehicle. The acceleration response of the bridge was measured by placing seismic accelerometers in three to four locations on the underside of the bridge. The locations of the accelerometers were selected to collect high quality data which could be used to obtain several vibration modes. The same DAS was used for collecting the accelerometer data as was used for the displacement data.

Condition Assessment

The condition assessment for each bridge involved visual inspection, making dimensional measurements, and taking photographs. During the visual inspections, the bridges were checked for decayed material, cracking in both the road surface and the laminates, abutment joint problems, and scanned for other conditions that might be harmful to overall performance. Measurements were taken of the bridges' overall dimensions along with measurements of other individual elements. Photographs were also taken to document the state of the bridges and to assist with future monitoring.

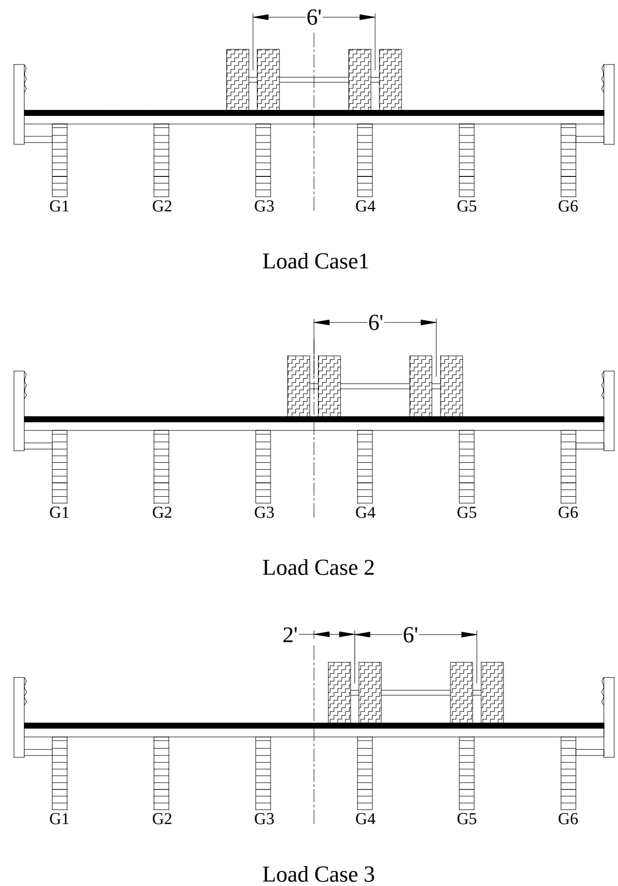


Figure 1. Bridge load positions.

Glued-Laminated Girder Bridge

Background

The most common type of timber bridge for both single and multiple spans is the glued-laminated girder bridge. Glued-laminated girder bridges are constructed of beams manufactured from nominal 1.5 in. thick lumber laminations bonded together with waterproof structural adhesive. The beams have standard widths ranging from 3 in. to 14.25 in. The beam depth is theoretically unlimited but is generally restricted by transportation or by pressure treating limitations. The clear span for girder bridges ranges from 20 ft to 80 ft with some applications up to 140 ft [21].

The advantages of glued-laminated girder bridges with respect to conventional saw lumber bridges are not only that they have longer span lengths with fewer beam lines, but also that they are easily constructed and have good longevity. Because glued-laminated bridges are constructed of prefabricated modular components, they are constructed in less time with less equipment and damage to the environment. With proper design and fabrication, the timber girder bridges have a service life of 50 years or more [21].

In order to study the DAF and other characteristics of glued-laminated girder bridges and to allow for verification of dynamic load allowances, five bridges that met the testing criteria were selected for field testing and inspection. The results from field testing and visual inspection of these five glued-laminated girder bridges are fully documented in *Dynamic Field Performance of Timber Bridges 1 – 5* and are the basis for the following sections [25]-[29].

Results

A brief discussion for each of the five bridges is presented in the following paragraphs. The characteristics, dynamic response, general performance, and condition at the time of testing are reported for each bridge. Following the summary of the individual bridges is a discussion of the overall dynamic performance of the five bridges when considered together.

Where applicable, the bridge's dynamic characteristics will be evaluated and compared with the dynamic load allowances found in various bridge codes. Figure 2 shows several new and old dynamic load allowances for timber bridges that exist from the United States and Canadian design codes [2], [6], [7]. In general, the dynamic load allowance values shown for timber represent a reduction of those specified for concrete and steel bridges. Also included in Fig. 2 are the DAFs of the five girder bridges tested and discussed herein. The experimental DAFs for the bridges seen are random and are within certain dynamic load allowances for some codes and exceed values from other codes. The differences between the DAFs may be attributed to several factors. The more obvious factors, as stated previously, are the road surface condition and the vehicle suspension; however, other factors such as moisture content, bridge design, and natural frequency can affect the response of the bridge. When applicable, these factors are discussed in the following paragraphs.

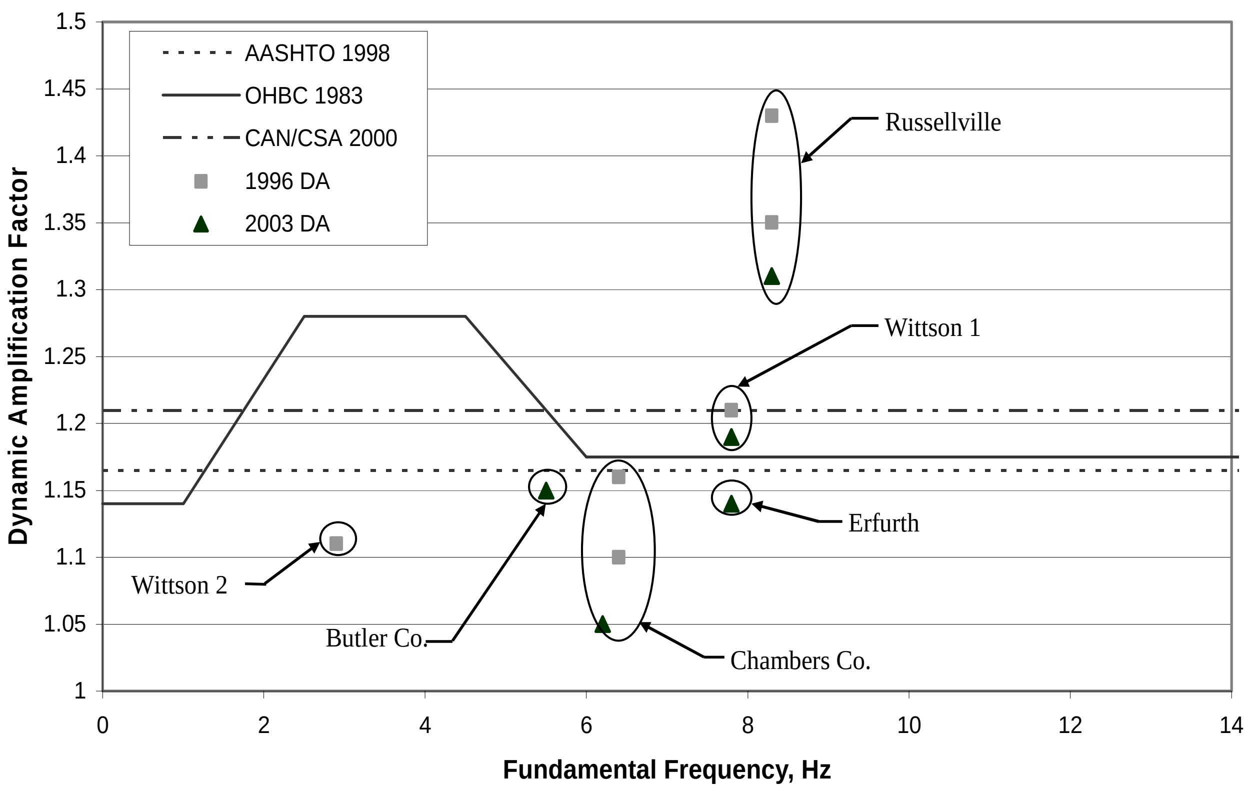


Figure 2. International impact factors for glued-laminated girder bridges.

The Chambers County Bridge, constructed in 1994, is located in east central Alabama. The bridge is a two lane, simply supported structure with span length of 51 ft - 6 in. The primary supporting elements of the bridge are six glued-laminated girders measuring 8.75 in. x 43 in. that are placed 5 ft on center. The bridge has 14 transverse glued-laminated deck panels. The deck panels have nominal dimensions of 5 in. x 49 in and are 28.5 ft in length. The panels are noninterconnected and have a nominal 3-in. thick course asphalt wearing surface.

The Chambers County Bridge was first tested in 1995. At this time the bridge was only one year old; therefore, all aspects of the bridge were in good condition. The bridge was dynamically loaded using Load Case 1 and 2. The maximum deflection for Load Case 1 was 0.536 in. This deflection occurred at 31.3 mph and created a DAF of 1.10. The maximum Load Case 2 deflection occurred at 34.8 mph and created a DAF of 1.16.

During the 1995 testing, four mode shapes were identified from the free vibration record. The fundamental frequency of the bridge was found to be 6.44 Hz. A finite element model of the bridge was also used to determine the frequency of the Chambers County Bridge. In general, the analytical modal computed the frequency of nearly all the vibration modes to be with 2.5 % of the observed field data.

The Chambers County Bridge was also field tested in 2003 using only Load Case 3. The condition of the structural elements of the bridge at the time of testing was rated as good. The wearing surface, however, was in poor condition with very rough approaches and significant transverse cracking. The bridge deck can be seen in Fig. 3. During the testing, a maximum deflection of 0.588 in. was obtained. This deflection occurred at 29.6 mph and created a DAF of 1.05. Possible higher DAF values could have been obtained from the bridge if higher testing speeds were possible. Figure 4 shows the DAF in relation to velocities for all three load cases and is typical of the collected data. The fundamental frequency from the 2003 testing was found to be 6.2 Hz.

Figure 3. Chambers County Bridge wearing surface.

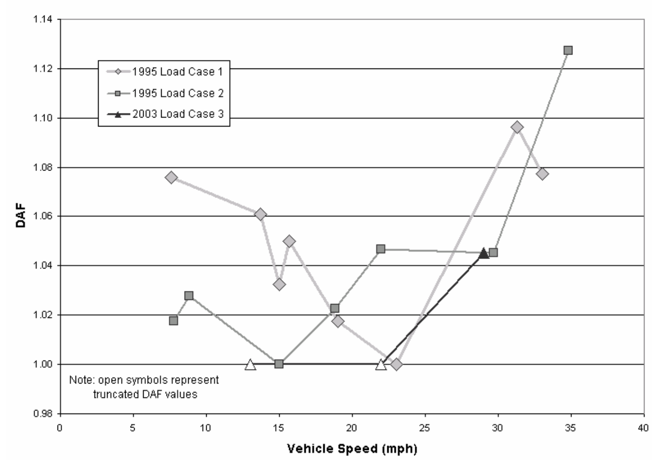


Figure 4. Chambers County Bridge DAF plot.

The Russellville Bridge, located in northwest Alabama, was also built in 1994. The bridge has four simply supported spans that carry two lanes of traffic. The 41 ft. – 6 in. long Span 1 was the focus of the testing and evaluation. Span 1 has five 6.75 in. x 41.5 in. girders spaced at 5 ft. centers. The deck consists of 5 in. x 48 in. x 24 ft – 6 in. glued laminated transverse deck panels. A nominal 3-in. asphaltic overlay is placed on top of the panels.

During the 1995 testing the bridge was characterized as being in good condition. It was noted, however, that the first deck panel created a natural bump at the abutment of the bridge. The bump was elevated approximately 0.75 in. above the natural road surface and is schematically illustrated in Fig. 5.

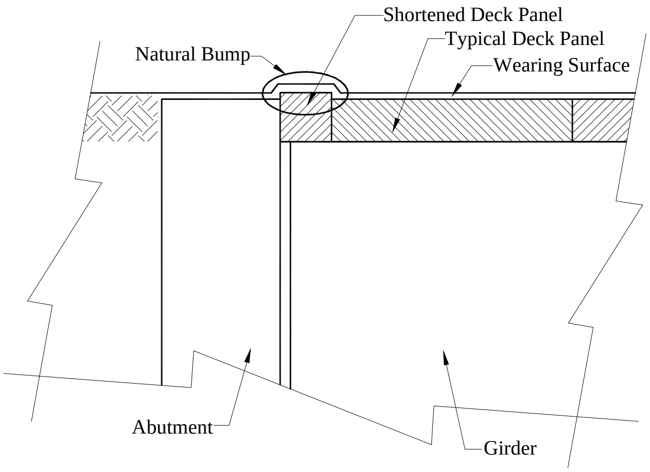


Figure 5. Bump at entrance of Russellville Bridge.

Load Cases 1 and 2 were used during testing of the Russellville Bridge. The maximum DAF that occurred for Load Case 1 was 1.43 and occurred at a speed of 33.3 mph. The Load Case 2 maximum DAF was 1.35 and had a deflection of 0.488 in. The natural frequency of the bridge was in the range of 7.8 Hz to 8.6 Hz. The high DAF values are attributed to the bump at the entrance of the bridge. The bump most likely excited the vehicle, causing larger wheel forces. Further, the bump could have excited the axle hop mode of vibration for the vehicle. The range for a vehicle's axle hop vibration is close to the natural frequency of the bridge and could lead to a near resonant response of the bridge causing the higher DAF.

The condition of the bridge during the 2003 test was found to be satisfactory. The natural bump at the entrance of the bridge still existed, although did not appear to have worsened over time. The wearing surface also had prominent transverse cracking typically aligning with the deck joints. The structural condition of bridge generally appeared to be in good condition.

The bridge was tested using Load Case 2 in 2003. The maximum DAF occurred as the truck crossed the bridge at 26.6 mph. The resulting deflection and DAF was 0.477 in. and 1.31, respectively. Once again, the high DAF is attributed to the natural bump at the entrance of the bridge. The natural frequency matched very closely to the frequency found in 1995- a range of 7.8 Hz to 8.6 Hz. A plot of the DAF results from both 1995 and 2003 is shown in Fig. 6.

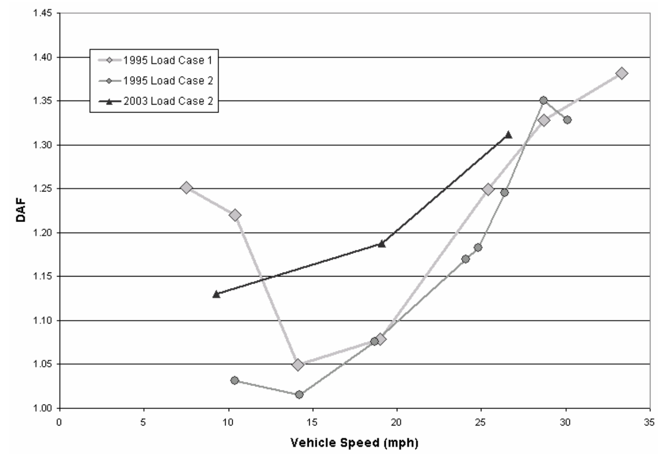


Figure 6. Russellville Bridge DAF plot.

The Wittson Bridge, built in 1994, carries one-way traffic for Old Jasper Road in central Alabama. The bridge has four simply supported spans carried by four 6.75 in. x 43 in. girders spaced 51 in. on center. The transverse deck panels have nominal dimensions of 5 in. x 51 in. wide with a length of 16 ft. The panels are noninterconnected and lag screwed to the girders. The wearing surface consists a nominal 3-in. course asphalt. For testing purposes only, Span 1 and Span 3 were instrumented. Span 1 is 50 ft – 7 in. long, while Span 3 is 102 ft long. Both accelerations and deflections were measured during the 1995 field testing for Span 1 and 3. The 2003 field testing also examined the accelerations for Spans 1 and 3; however, deflection measurements were only taken for Span 1. The truck path for all tests conducted in both 1995 and 2003 was based on Load Case 1 only due to limited bridge width.

Due to the bridge being relatively new in 1995, the bridge was characterized as being in good condition. The 1995 testing found the maximum deflection of Span 1 to be 0.499 in. This deflection created a DAF of 1.21. It should be noted, however, that the adjacent girder had a DAF of 1.10 for the same loading. The maximum DAF for Span 3 was found to be 1.11. A deflection of 1.017 in. at 10 mph caused this DAF.

The frequency content of Span 1 was difficult to obtain due to the interaction of the adjacent spans. However, a range for the fundamental frequency of Span 1 was found to be between 7.0 Hz and 7.8 Hz. Along with the field observations of Span 3, an analytical model was developed for comparison with the data. From the testing, three frequencies were obtained for Span 3. The three frequencies, with the first being the fundamental frequency, were 2.8 Hz, 8.8 Hz, and 10.6 Hz. An analytical model found frequencies to be within 9.4% of the experimental values. The corresponding analytical frequencies were 2.7 Hz, 9.5 Hz, and 11.7 Hz.

The overall condition of the bridge during the 2003 testing was satisfactory. The structural condition of the bridge was very good with no signs of decay or damage to the bridge elements. The wearing surface, however, was found to only be satisfactory. Cracking of the wearing surface was evident throughout the bridge with the most severe located on the three shorter spans (i.e., Span 1, 2, and 4). The short span cracking was dominated by transverse cracks generally located along the panel joints. Less cracking was evident in the long span (i.e., Span 3). It was also noted that, at the pier location between Span 1 and 2, there was no transverse cracking visible for several feet extending each direction from the pier.

The maximum dynamic response of Span1 from the 2003 testing was found to be very similar to the testing in 1995. The maximum DAF was found to be 1.19 at 37 mph. Figure 7 shows the DAF for the Wittson Bridge in both 1995 and 2003. The frequency was found to be 7.8 Hz, which is similar to the range found in 1995. The Span 3 frequency was found to be 2.9 Hz, which is also close to 1995 results.

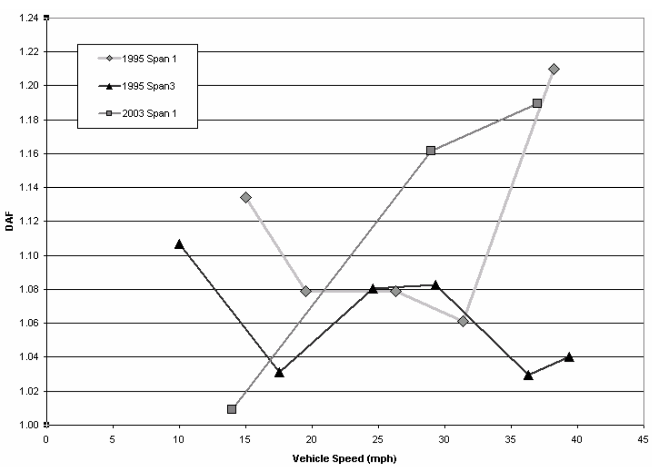


Figure 7. Wittson Bridge DAF plot.

The Butler County Bridge, located in southern Alabama, is a two span bridge supplying two-way traffic. The bridge was built in 1992. The shorter west span of the bridge is 24 ft long and supported by five 5 in. x 27.5 in. girders spaced 5 ft on center. The east span is 60 ft long with five girders measuring 8.625 in. x 48.125 in. spaced 5 ft on center. The bridge deck consists of noninterconnected panels measuring 5 in. x 48 in. x 24 ft – 7 in. long. A nominal 3-in. wearing surface covers the deck panels.

Field tests were conducted in 2003 on the west span of the Butler County Bridge. The condition of the bridge at that time was reported to be severe. The wearing surface was cracked in both the longitudinal and transverse direction with the most severe cracking in the transverse direction and varied from small cracks along the shoulder to cracks as large as 2.0 in. wide in the middle of the panels. These large cracks were located at deck joint locations. Figure 8 shows a typical example of the cracking and raveling of the deck surface. The deck panels of the bridge were found to be in poor condition. The panels were cupped and were separated by as much as 0.5 in. in some places. The upward cupping of the deck panels caused the bearing surface at the girders to be limited to only the center portion of each panel. The cupping also caused a rocking action of the individual panels as the vehicle traveled across the bridge. This rocking action is believed to be the cause of the severe deterioration of the wearing surface. Despite the deterioration of the deck, the girders appeared to generally be in good condition.

Figure 8. Butler County Bridge wearing surface cracking.

The results from field testing the Butler County Bridge found the maximum DAF to occur at girder G5 at a speed of 9.4 mph. The DAF was 1.15 with a deflection of 0.287 in. All other girders had DAF values very close to 1.00 for all test runs. Table 1 shows the deflections and DAF values for the girders in the zone of direct influence. The natural frequency of the bridge was found to be 5.47 Hz.

Table 1. DAF for selected girders of Butler County Bridge

Speed	Girder G3		Girder G4		Girder G5	
	Deflection*	DAF	Deflection*	DAF	Deflection*	DAF
Crawl	0.219	n/a	0.292	n/a	0.250	n/a
9.4	0.205	1.00^	0.290	1.00	0.287	1.15
16.4	0.213	1.00^	0.275	1.00^	0.259	1.03

* measured in inches.
^ truncated DAF values
n/a not applicable

The Erfurth Bridge, located in southern Wisconsin, supplies two-way traffic for a low volume county highway. The bridge, built in 1992, has twelve 14.25 in. x 23.5 in girders that make up the single span. The twelve girders are composed of two individual sections, one being 8.5 in. x 23.5 in. and the other 5.5 in. x 23.5 in. A cross-sectional sketch is shown in Fig. 9. The girders are nominally spaced at 2 ft – 7 in. The bridge deck consists of glued-laminated panels having nominal dimensions of 3.25 in. x 52 in. wide with a length of 31 ft – 11 in. The wearing surface is a nominal 3-in. coarse asphalt.

Testing took place in 2003 using Load Case 3 only. A maximum deflection of 0.903 in. was measured on one girder. This deflection, however, was twice as large as the adjacent girders. The reason for the large deflection could not be determined; however possible sources include varying gaps between deck panels and girders, differences in support conditions, and possible non-visible deterioration on the girder. The maximum DAF for the bridge occurred at 9.4 mph and was 1.14. Figure 10 shows the DAF for four of the girders in the zone of direct influence of the truck.

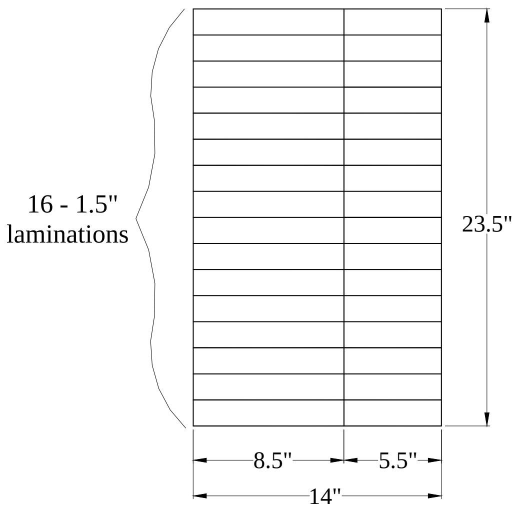


Figure 9. Cross section of Erfurth Bridge double girder.

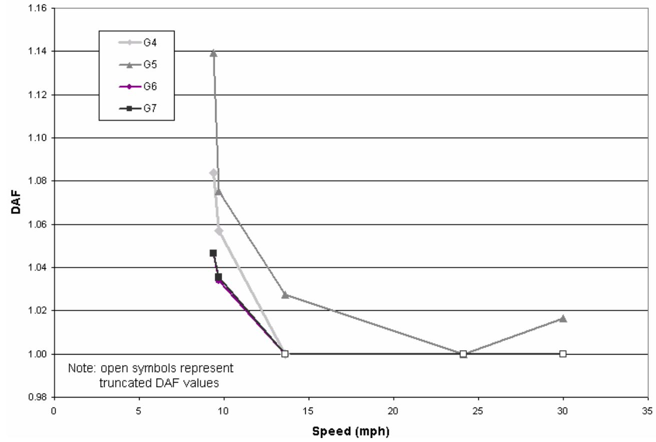


Figure 10. Erfurth Bridge DAF plot.

The wearing surface of the Erfurth Bridge at the time of testing was found to be in satisfactory condition. Significant transverse cracking existed along the length of the bridge. The transverse cracking was highly regular and was seen to generally occur above deck panel joints. Additionally, longitudinal cracking was found to be regular with the cracks aligning with the bridge girders. The wearing surface was found to be well maintained with the cracks being filled with bituminous sealant. Figure 11 shows the wearing surface.

Figure 11. Erfurth Bridge wearing surface.

Discussion

Four of the five bridges tested were found to have DAF within the code limits presented in Fig. 2. The Russellville Bridge was the only bridge that did not meet the codified requirements. The large DAF values for the Russellville Bridge are attributed to the natural bump at the entrance of the bridge. Tables 2 and 3 summarize the results of the dynamic testing and also list key physical attributes of each bridge.

With the exception of the Butler County Bridge, the condition of the wearing surface of the girder bridges was generally found to be satisfactory. All of the bridges had transverse cracking and the majority of these cracks aligned with the deck joints. The sizes of the cracks varied from being very small on the Erfurth Bridge, to being 2.0 in. or larger on the Butler County Bridge. Longitudinal cracking was also present, but more sporadic and occurring less frequently. Several reasons exist for the similar cracking patterns to exist in all the bridges. The severe transverse cracking that aligned with the deck joints is believed to be caused by a combination of girder and differential deck panel deflection, which maybe enhanced by the dynamic forces and vibration. The differential panel deflection correlated to the wearing surface deterioration has been further investigated by Wipf [34]. Due to the range of asphalt deterioration, however, further experimental studies are needed in this area.

The bridges that were tested in 1995 were found to have a similar, or a decrease in, DAF when tested in 2003. The most drastic change was with the Chambers County Bridge. The DAF from 1995 to 2003 decreased by approximately 9% while the wearing surface changed from excellent to poor condition. This finding leads one to consider the changes in the wearing surface conditions are less influential when compared to changes in the bridge and vehicle characteristics. The same phenomenon was also seen for the Russellville Bridge. The DAF for the bridge was very high in 1995 with a lower DAF value obtained in 2003. There was also a simultaneous decrease in the condition state of the wearing surface seen between the two testing periods. Several possible changes that could be attributed to the decrease in DAF for the bridges are changes in the abutment fixity, changes in girder continuity, changes in overall bridge stiffness, change in mass, and changes in the truck dynamic properties.

The natural frequencies determined from field testing were seen to have little change between the eight years of testing. This is likely due to the major timber components performing very well with very little deterioration or reduction in stiffness. It has been suggested by Chen, only a severely damaged structure will have a noticeable change in the mode frequencies [10]. This was apparently verified for the bridges tested in this study.

With the exception of Span 3 of the Wittson Bridge, the bridge spans tested had natural frequencies that were generally between the body bounce and axle hop frequencies associated with the test vehicles used. Since the frequency of the bridges did not match the natural frequency of most vehicles, it is likely the bridges will not experience excessively large deflections due to resonation with vehicles.

Table 2. Glued-laminated girder bridge information

Report Number	Bridge Name	No. of Spans	No. of Traffic Lanes	Span(s) Length (ft - in.)*	Traffic Width (ft - in.)	No. of Girders	Girder Size (in. x in.)	Girder Spacing (in.)	Deck Panel Size (in. x in.)
1	Chambers County	1	2	**51 - 6**	28 - 6	6	8.75 x 43	60	48 x 5
2	Russellville	4	2	**41 - 9** 42 - 6 42 - 6 41 - 9	24 - 6.5	5	6.75 x 41.5	60	48 x 5
3	Wittson	4	1	**50 - 7** 50 - 7 **102 - 0** 30 - 0	16 - 0	4	6.75 x 43 6.75 x 63.25	51	51 x 5
4	Butler County	2	2	**24 - 0** 60 - 0	24 - 7	5	5 x 27.5	60	48 x 5
5	Erfurth	1	2	**41 - 6**	31 - 11	12	14.25 x 23.5	31	52 x 3.25

*bold number denote spans that were tested.

Table 3. Glued-laminated girder bridge test results

Report Number	Bridge Name	Load Case	1995 Results Wearing surface condition	Max. Dynamic Deflection (in)*	Max. DAF	Fundamental Frequency (Hz)	2003 Results Wearing surface condition	Max. Dynamic Deflection (in.)*	Max. DAF	Fundamental Frequency (Hz)
1	Chambers County	1 2 3	Excellent	0.604 0.669 n/a	1.10 1.16 n/a	6.4	Poor: Rough approach; deteriorated asphalt.	n/a n/a 0.644	n/a n/a 1.05	6.2
2	Russellville	1 2	Good: Natural bump at entrance of bridge.	0.504 0.560	1.43 1.35	7.8 to 8.6	Satisfactory: Entrance bump & transverse cracking.	n/a 0.536	n/a 1.31	7.8 to 8.6
3	Wittson	1	Very good	0.628 (span 1) 1.28 (span 3)	1.21 (span 1) 1.11 (span 3)	7.0 to 7.8 (span 1) 2.8 (span 3)	Satisfactory: Transverse & longitudinal cracking.	0.625 (span 1)	1.19 (span 1)	7.8 (span 1) 2.9 (span 3)
4	Butler County	2	n/a	n/a	n/a	n/a	Serious: Severe cracking & deterioration	0.343	1.15	5.5
5	Erfurth	3	n/a	n/a	n/a	n/a	Satisfactory: Transverse & longitudinal cracking.	0.953	1.14	7.8

*Deflections have been adjusted to weight of design truck (HS20).
n/a not applicable

Longitudinal Glued-Laminated Panel Bridge

Background

Longitudinal glued-laminated panel bridges consist of glued-laminated panels, similar to those used for the decks of girder bridges, except the panels are placed parallel to traffic between the bridge abutments. The deck panels are generally noninterconnected. In addition to the panels, transverse stiffener beams are attached to the underside of the deck panels. Typically, these are intended to enhance the distribution of loads laterally across the bridge and to give overall continuity to the system. The panels can be used for either single span or multi-span bridges with practical clear span distances of approximately 35 ft [21].

Like the girder bridge, one advantage of panel bridges is that the longitudinal panels are prefabricated in a modular system. With prefabrication these systems have the required cuts and holes drilled prior to preservative treatment and the components are less susceptible to future decay. Panel bridges also have the advantage of having a low profile. This makes the bridge suitable for short spans that have clearance restrictions. Overall, the panel bridges are very economical, have good longevity, and have reduced erection time compared to other bridge materials.

The verification and quantification of the dynamic properties for glued-laminated panel bridges was completed by field testing four longitudinal glued-laminated panel timber bridges. All four of these bridges were located in southwest New York State in Allegany County. A detailed documentation of the results can be found in *Dynamic Field Performance of Timber Bridges 6 – 9* [30]-[33]. These four reports are the basis for the following sections.

Results

The following paragraphs discuss briefly each of the four bridges tested including the dynamic response, general performance, and condition at the time of testing. Following the individual bridge results will be a discussion of the overall dynamic performance of the four tested bridges with respect to the previously presented information. Where applicable, the results will be compared with other similar research completed with other bridge materials.

Figure 12 shows the dynamic load allowances from various design codes with respect to the experimental bridge amplifications found in both 1996 and 2003. As can be seen from Fig. 12, only three measured DAFs were above the various code requirements. Interestingly, all DAFs established in 2003 were below the established codes values and were seen to be less than the 1996 tests results.

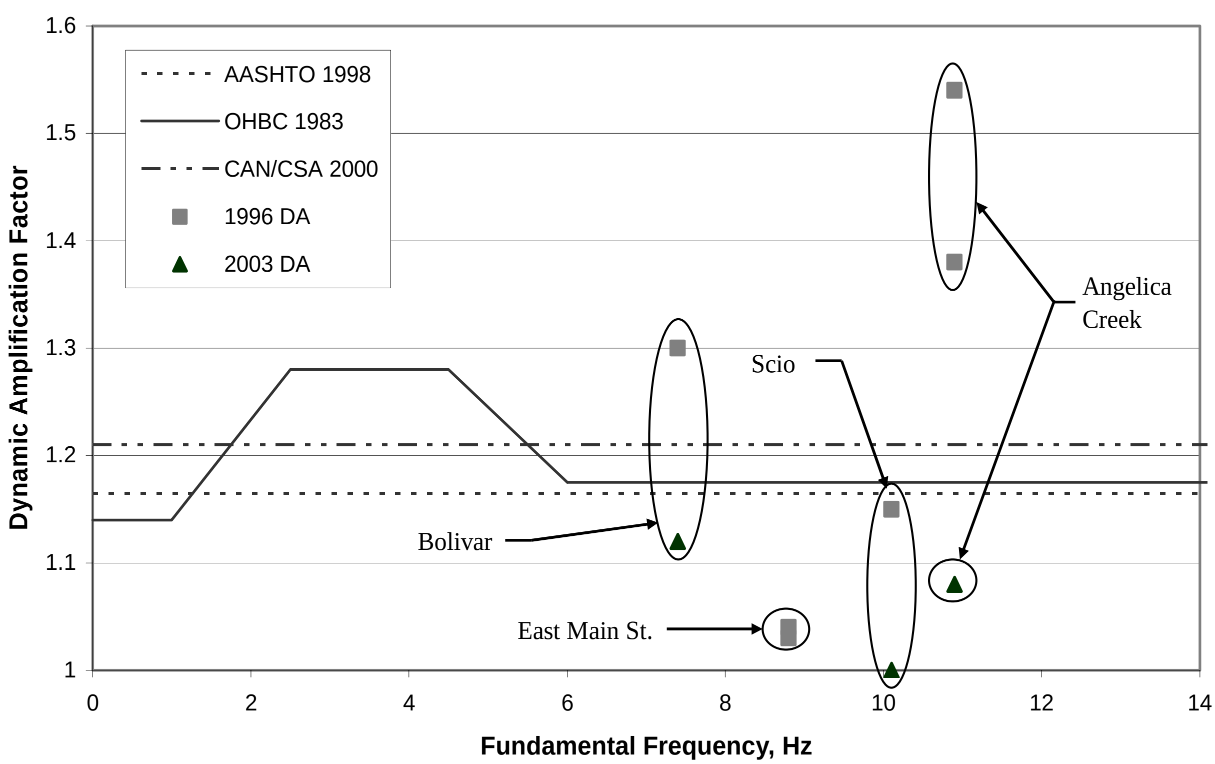


Figure 12. International impact factors for longitudinal glued-laminated panel bridges.

The Angelica Creek Bridge was constructed in 1982. This single span, simply supported bridge carries two-way traffic and has an AADT of approximately 260. The bridge has a clear span of 20 ft and a roadway width of 29 ft – 1 in. Seven deck panels, measuring 50 in. x 9 in. thick, span between the bridge abutments. The panels bear on T-shaped steel clips that are also used for attachment to the abutments. The bridge has stiffener beams located at the third points of the bridge. These stiffener beams are 6.75 in. x 8.25 in. and are connected to the panels at each panel joint via U-shaped brackets. The bridge has a nominal 2.5-in. wearing surface placed over a membrane.

The Angelica Creek Bridge was tested in 1996 using Load Case 2. Two separate tests were conducted each with two different load trucks denoted as Truck 12 and Truck 18. Truck 12 produced the smaller DAF of the two trucks. The maximum Truck 12 deflection was 0.710 in., producing a DAF of 1.38. Truck 18, which weighed 400 lbs more than Truck 12, had a maximum deflection of 0.81 in. This induced a DAF of 1.54. Although the magnitudes of the DAFs differed at various transducer locations, the DAF was seen to increase as the speed of the vehicle increased for all the transducers in the zone of direct influence. The natural frequency was found to be 10.9 Hz.

The bridge was characterized as being in fair condition during the 1996 testing. Although the approaches were in good condition with only small transverse cracks existing, the abutment and wearing surface of the bridge had several areas of deterioration and asphalt patching. At the entrance of the bridge, a 0.5–in. natural bump was formed by an asphalt patch. The patch varied in width from 28 in. to 40 in. along the direction of travel. This patch is believed to be related to the large DAF values. The Angelica Creek Bridge surface also had longitudinal cracking and some localized depressions in the asphalt. The longitudinal cracks generally aligned with the deck panel joints. The structural deck panels were found to be in good condition with no visible structural damage. The exterior panels were found to be warped concave up at the mid-width.

The testing that took place in 2003 also traversed Load Case 2. The maximum dynamic deflection that occurred during the testing was 0.56 in. at 14.8 mph. The resulting DAF was 1.08. This was also the maximum DAF that occurred for the bridge. Figure 13 shows the DAF history with respect to vehicle speed for both the 1995 and 2003 testing. Similar to that found in 1996 the fundamental frequency was found to be 10.9 Hz.

The bridge was in excellent condition during the 2003 testing. A new lift of asphalt had been placed on the approaches and bridge approximately two months prior to testing. This new asphalt surface, shown in Fig. 14, created an extremely smooth riding surface for the test vehicle. The deck panels were found to be in good condition with the end panels still being warped as previously mentioned.

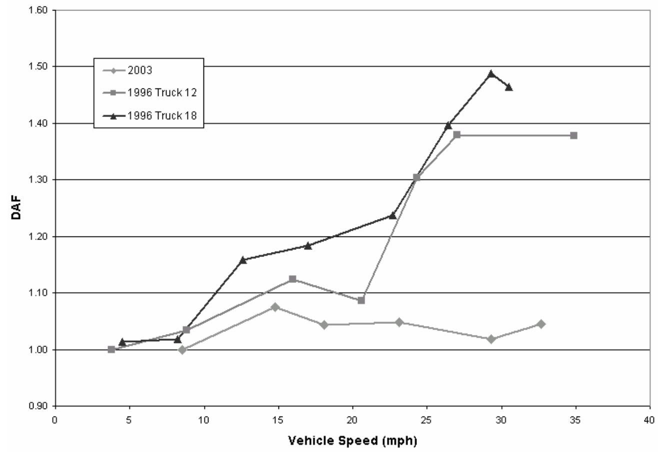


Figure 13. Angelica Creek Bridge DAF plot.

Figure 14. Angelica Creek Bridge 2003 wearing surface.

The East Main Street Bridge is located in the town of Angelica. Built in 1985, the bridge supplies two-way traffic for the low volume East Main Street. The bridge superstructure consist of eight glued-laminated panels that are 53 in. x 14 in. thick creating a bridge width of 35 ft – 10 in. The panels have a clear span of 29 ft – 7 in. with a bearing of 11 in. on each end. Four glued-laminated stiffener beams are placed at the fifth points of the bridge. The beams are attached to the underside of the panels at each panel joint via two through bolts. The bridge wearing surface consists of a nominal 3-in. thick course asphalt. The bridge also has heavy timber curbs connected to the edge deck panels with through bolts.

The condition of the bridge during the 1996 testing was found to be good. Small transverse cracks did exist in the approaches, but no potholes existed and the riding surface was reasonably smooth. There was, however, a 0.5 in. bump caused by an uneven road surface at the entrance to the bridge. The bridge wearing surface was also characterized as being in good condition. The structural deck was found to be in good condition with little to no visible damage or decay. The stiffener beams were in satisfactory condition. Vertical splits and section loss was taking place at the ends of the beams. One stiffener beam had a noticeable longitudinal crack under one of the panel joints. The crack had been repaired by bolting a 0.25 in. thick steel plate across the bottom of the cracked portion of the beam.

In 1996 the East Main Street Bridge was dynamically tested with both Truck 12 and Truck 18 on Load Case 3. The results from the Truck 12 testing found the maximum displacement to be 0.414 in. This deflection occurred at a speed of 35.8 mph and also created the maximum DAF of 1.03. Truck 18, which weighed 5600 lb less than Truck 12, had a maximum deflection of 0.390 in. This created a maximum DAF of 1.04. The fundamental frequency of the bridge was found to be 8.8 Hz. Figure 15 shows the DAF history with respect to vehicle speed for both Truck 12 and Truck 18 testing.

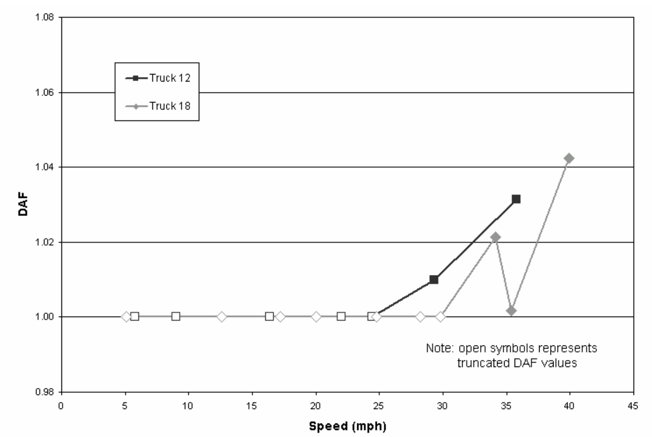


Figure 15. East Main Street Bridge DAF plot.

Due to traffic conditions during the 2003 testing only the natural frequency and a condition assessment of the bridge was able to be completed. The fundamental frequency was found to be 8.8 Hz. This corresponded very closely to the frequency obtained in 1996.

The bridge was found to be in good condition in 2003. The approaches showed no signs of excessive cracking or potholes. Transverse cracking did exist at the bridge abutments but the presence of a preexisting bump was not apparent. The bridge surface had two longitudinal cracks running the length of the bridge along with other areas of minor cracking. Two longitudinal cracks appeared to align with the deck panel joints. The east abutment cracking and typical longitudinal cracking is shown in Fig. 16.

Figure 16. East Main Street Bridge 2003 asphalt wearing surface.

The Bolivar Bridge, constructed in 1991, is a two-lane low volume bridge with an AADT of 360. The bridge is simply supported with a clear span of 27 ft – 8 in. The deck is composed of six 52 in. x 15 in. thick panels with a nominal 3-in. asphalt overlay. Three stiffener beams are attached to the underside of the deck panels at the quarter points and at midspan. The beams are attached using through bolts at all panel joint locations.

The test conducted in 1996 used load Truck 18 traversing Load Case 3. The maximum deflection for the bridge was 0.642 in. The maximum DAF of 1.30, however, occurred at different transducer location at the test speed of 29.7 mph. Three modes of vibration were identified for the Bolivar Bridge. The fundamental and other frequencies were, 7.4 Hz, 10.4 Hz, and 12.5 Hz, respectively.

The bridge was characterized as being in good condition during the testing. The approach and road surfaces were relatively smooth. The entrance abutment, however, was noted to have an asphalt patch and settlement that created a natural bump. The bump varied between 0.5 in. to 1 in. across the width of the bridge. The structural condition of the bridge was classified as being good. The bridge was noted to have considerable swelling and had caused the abutment connection clips to rotate and crush portions of the timber components.

The maximum 2003 deflection for the Bolivar Bridge was 0.536 in. The maximum DAF was found at an adjacent location resulting in a DAF of 1.12 and occurred at the maximum test speed. As can be seen from Fig. 17, if larger speeds were attained during the 2003 testing, a higher DAF could have possibly been obtained. The natural frequency was found to be 7.4 Hz, which was the same frequency found in 1996.

The bridge condition during the 2003 testing was rated as fair. The abutment of the bridge, as shown in Fig. 18, had several areas of asphalt patching and transverse cracking. Although the settlement was not as apparent as it was in 1996, there was still a small bump. The bridge wearing surface had several longitudinal cracks and potholes forming on the surface. The longitudinal cracks formed mainly along the panel joints and typically had transverse cracks extending from them. The primary structural members were rated as in satisfactory condition. Swelling and crushing was still present at the abutments and crushing was also noted at the stiffener beam connection locations.

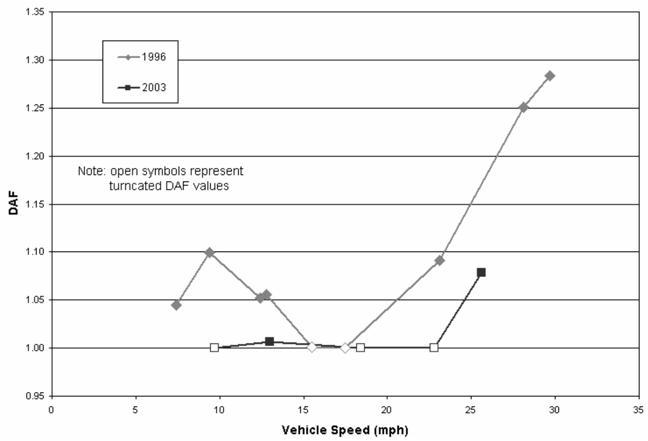


Figure 17. Bolivar Bridge DAF plot.

Figure 18. Bolivar Bridge 2003 entrance abutment.

The Scio Bridge is a simply supported, low volume structure that carries two-lanes of traffic. The bridge is located near the town of Scio and was built in 1984. The clear span of the bridge is 19 ft - 4 in. and has a roadway width of 31 ft – 2 in. The seven supporting deck panels are 52 in. x 9 in. thick with glued-laminated stiffener beams located at the quarter points and at midspan. The three stiffener beams are attached to the underside of the deck panels via two through bolts at the panel joints. Heavy timber curbs are attached to the edge panels. The wearing surface of the bridge consisted of a nominal 6–in. course asphalt.

The 1996 testing found a maximum deflection of 0.629 in. that took place at a speed of 31.0 mph. The maximum DAF was found to be 1.15; however, this occurred at an adjacent location. In general, the DAF was found to be 1.00 for speeds in the mid twenties. The DAF stayed above 1.00 only until reaching speeds in the mid thirties where it then went back down to 1.00. Two vibration modes were identified for the bridge. The frequencies were 10.1 Hz and 14.0 Hz.

The condition of the bridge during the 1996 testing found the wearing surface to be in satisfactory condition. The bridge was noted to have a 0.25–in. high asphalt patch that created a natural rough region at the east entrance of the bridge. A shallow depression in the approach surface was present 25 ft east of the bridge entrance. The depression extended approximately 4 ft along the length of the road and had a maximum depth of 0.5 in. The structural components of the bridge were in good condition with no significant problems noted.

During the 2003 testing of the Scio Bridge, seven truck passes were conducted for the same load position used in the 1996 testing. All seven dynamic truck passes yielded DAF values of 1.00. Figure 19 shows the DAF results for both the 1996 and the 2003 testing. Only one vibration mode was found for the Scio Bridge during the 2003 testing. The fundamental frequency was 10.1 Hz, which is the same frequency as was found in 1996.

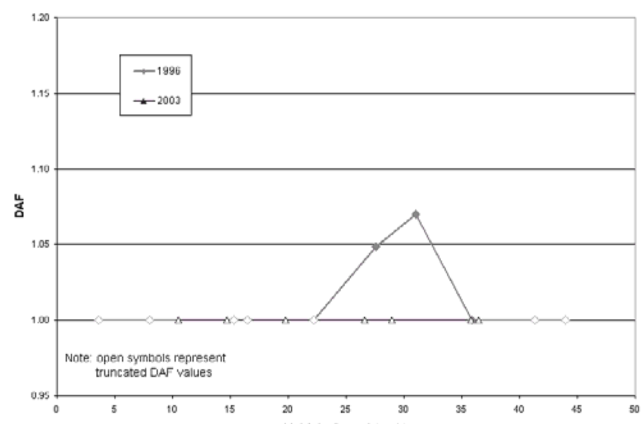
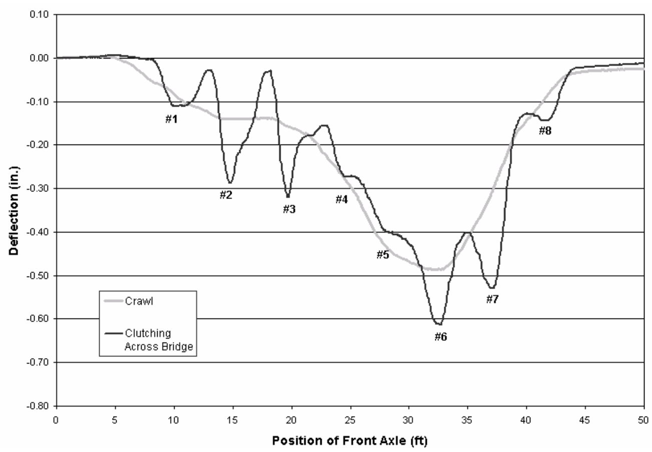


Figure 19. Scio Bridge DAF plot.

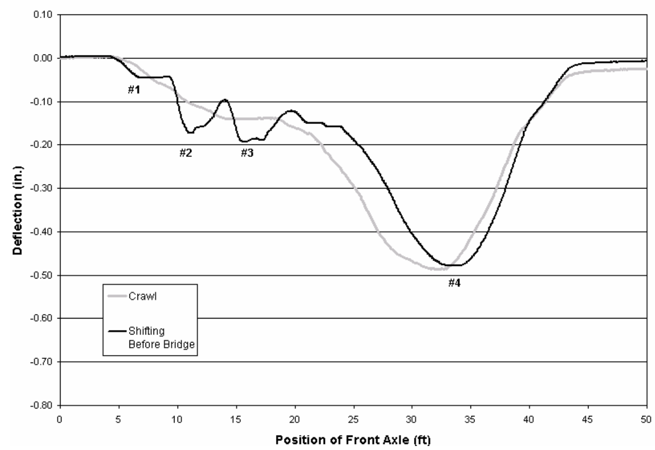
In addition to the standard dynamic truck passes, three non-conventional truck passes were made in 2003. The first non-conventional pass consisted of the driver traversing the bridge at approximately 3.5 mph and 'jamming the clutch' eight times over the length of the bridge. Jamming the clutch entailed pushing the truck clutch in and out to cause sudden decelerations and accelerations to the vehicle. The sudden speed changes of the truck induced a controlled bouncing of the truck as it traversed the bridge. The deflection plot of the truck pass can be seen in Fig. 20. A localized maximum DAF of 2.0 was achieved at points 2 and 3.

The second non-conventional truck pass consisted of the truck shifting gears just prior to entering the bridge. The truck was traveling approximately 5 mph and the shifting caused the truck to bounce just prior to entering the bridge. Figure 20 shows the deflection of the truck pass. A localized maximum DAF was found to be 1.7. The third truck pass was similar to truck pass two; however, the shifting took place while on the bridge. The deflection history of this loading is also shown in Fig. 20. It was seen that shifting while on the bridge had less of an impact than the other non-conventional truck passes; however, a localized maximum DAF was found to be 1.14 (as compared to 1.00 for the conventional passes). Table 4 shows all the local DAF values for the three truck passes.

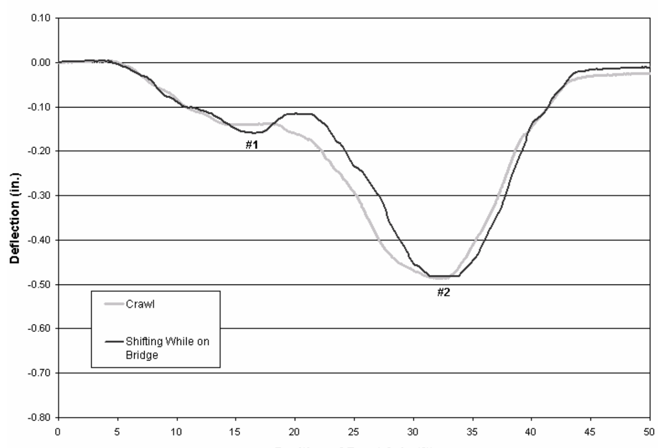
The condition of the wearing surface in 2003 was characterized as good. The approaches were also characterized as being in good condition with no rough regions noted. The entrance did, however, have slight transverse cracking. The cracking extended the full width of the bridge and created a depression of approximately 0.25 in. at both the entrance and exit of the bridge. The bridge wearing surface had several small/hairline longitudinal cracks along with one larger longitudinal crack running half the length of the bridge. The structural condition of the bridge was also characterized as good. The deck panels were performing well with little to no signs of decay or cracking. The stiffener beams, however, did have some crushing and splitting due to the connection bolts being too tight.



a. Jamming clutch across bridge.



b. Shifting prior to entering bridge.



c. Shifting while on bridge.

Figure 20. Scio Bridge dynamic deflections for non-conventional loading.

Table 4. Local DAF for three additional load cases

Peak Number	Jamming Clutch			Shifting Before Bridge			Shifting on Bridge		
	δ_{dyn}*	δ_{stat}*	DAF	δ_{dyn}*	δ_{stat}*	DAF	δ_{dyn}*	δ_{stat}*	DAF
1	-0.111	-0.082	1.35	-0.046	-0.027	1.70	-0.159	-0.140	1.14
2	-0.282	-0.141	2.00	-0.172	-0.106	1.62	-0.483	-0.485	1.00
3	-0.320	-0.160	2.00	-0.193	-0.140	1.38	n/a	n/a	n/a
4	-0.273	-0.280	1.00	-0.478	-0.463	1.03	n/a	n/a	n/a
5	-0.400	-0.440	1.00	n/a	n/a	n/a	n/a	n/a	n/a
6	-0.613	-0.484	1.27	n/a	n/a	n/a	n/a	n/a	n/a
7	-0.528	-0.305	1.73	n/a	n/a	n/a	n/a	n/a	n/a
8	0.143	-0.082	1.75	n/a	n/a	n/a	n/a	n/a	n/a

* measured in inches
n/a: Not applicable

Discussion

Two of the four panel bridges tested had all DAF values within code limits. Of the nine dynamic tests performed on the bridges, 66% of the DAF values obtained were within code limits. The Angelica Creek Bridge and Bolivar Bridge had DAF values that were above the specified code limits as was shown in Fig. 12. The large DAF values for both of these bridges are attributed to bumps at the entrances of the bridge during the 1996 testing. These values, however, were seen to decrease below the code recommendations in 2003. This decrease is attributed to the change in the approach conditions, the properties of the different testing vehicles, changes in the weight of the bridge due to increase in moisture levels, and changes in load distribution due to swelling of components. Similar to the girder bridges, the entrance and approach roughness heavily influenced the DAF values obtained for the four panel bridges. A summary of the results along with key physical characteristics is shown in Tables 5 and 6.

Overall, the four panel bridges had satisfactory wearing surfaces during the 1996 testing. Nearly all the bridges had some type of rough region or bump at the entrance of the bridge. These bumps were seen to cause high DAF values for the bridges. It was also noted that the bridges having more severe bumps also had higher DAFs. During the 2003 testing, however, the maximum DAF values decreased to be below code requirements for all bridges. In general, the surface conditions for the 2003 testing improved to be in good condition. Particularly, the Angelica Creek Bridge was found to have a significant improvement in the wearing surface and a dramatic decrease in DAF. This specific case, along with the general conditions of the other bridges, agrees with McLean and Marsh, Gupta, Cantieni, and several others authors who state that the approach surface condition is a very important factor and heavily influences the magnitude of the DAF obtained for a bridge [8], [15], [18].

Another factor also believed to decrease the DAF from 1996 to 2003 is changes in the bridges' structural performance. All four of the bridges, as stated in *Live Load Deflection of Timber Bridges,* had a decrease in overall live load deflection between the 1996 and 2003 testing. Angelica Creek Bridge and Bolivar Bridge also had a decrease in differential panel deflection between testing periods [34]. Dramatic visual changes in the bridges' conditions were not noticed between the testing periods. The changes in load carrying characteristics are believed to be partly caused by friction effects between adjacent panels and the stiffener beams. An increase in static friction between bridge components is believed to create enhanced continuity across the deck, thus causing less deflection. Changes in the static friction resulted from localized swelling of the members that occurred between testing periods. Other factors such as abutment fixity, daily environmental conditions (weather), and vehicle characteristics also are believed to affect the bridge response.

Longitudinal cracking along with some transverse cracking was found on nearly all the bridge surfaces during the 1996 and 2003 testing. The longitudinal cracking was primarily found to follow the panel joints with less severe transverse cracking in other areas. The longitudinal cracking of the panel bridges was seen to be analogous to the transverse cracking that was taking place on the girder bridge deck surfaces. Many of the same reasons stated in the discussion of the girder bridges apply to the panel bridges surface cracking.

The natural frequencies of the panel bridges were seen to have very little change between 1996 and 2003 testing. The panel bridges either did not have any large changes in stiffness and mass, or both the stiffness and mass changed proportionally in order for the natural frequency of the bridge to be similar during both testing periods.

The natural frequencies of the panel bridges were generally larger than that of the girder bridges. The panel bridges had shorter span lengths which is likely the cause for the higher frequencies. The higher frequencies of the panel bridges did keep them from resonating with the body bounce vibration of the vehicle; however, the panel bridges had natural frequencies generally close to the axle hop frequency. Although the axle hop vibration of a vehicle is more difficult to excite, the bridges have the possibility of resonating with the axle hop vibrations of a vehicle and thus creating larger deflections. As an example the Angelica Creek Bridge, having a bump at the entrance of the bridge in 1996, could have excited the vehicle axle hop vibration and caused resonating frequencies with the bridge. In 2003, due the smooth surface, the vehicle's axle hop vibration was not excited and therefore the dynamic deflection was not as large.

The three non-conventional truck passes for the Scio Bridge gave much higher DAF values than the traditional testing. The 2003 traditional testing had a maximum DAF of 1.00, while the three non-conventional maximum DAFs were 2.00, 1.70, and 1.14, respectively. It was also found that the second and third non-conventional truck passes, where the truck shifted gears, had reduced DAF values as the truck traversed the bridge. After the vehicles' vibration due to shifting had dissipated, the displacement appeared to be very similar to the traditional truck passes. It was also noted that, when the truck shifted prior to entering the bridge, the DAF was much higher than when shifting was done on the bridge. These results were similar to when the vehicle was excited by the approach/entrance surface. It appears as though exciting the truck before the bridge by shifting is analogous to that of the approach or entrance of the bridge exciting the vehicle, while shifting on the bridge is similar to the bridge surface exciting the vehicle. Similar to the girder bridges, the excitement before the bridge or at the bridge entrance seemed to have a larger impact on DAF magnitude than if the excitement occurred on the bridge.

Table 5. Longitudinal glued-laminated panel bridge information

Report Number	Bridge Name	No. of Spans	No. of Traffic Lanes	Span Length (ft - in.)	Traffic Width (ft - in.)	No. of Panels	Panel Size (in. x in.)	Stiffener Beam Location
6	Angelica Creek	1	2	20 - 0	28 - 3	7	50 x 9	Third Points
7	East Main St.	1	2	29 - 7	35 - 10	8	53 x 14	Fifth Points
8	Bolivar	1	2	27 - 8	26 - 1	6	52 x 15	Quarter Points & Midspan
9	Scio	1	2	19 - 4	32 - 1	7	52 x 9	Quarter Points & Midspan

Table 6. Longitudinal glued-laminated panel bridge test results

			1996 Results					2003 Results			
Report Number	Bridge Name	Load Case	Wearing surface condition	Truck	Max. Dynamic Deflection (in)*	Max. DAF	Fundamental Frequency (Hz)	Wearing surface condition	Max. Dynamic Deflection (in.)*	Max. DAF	Fundamental Frequency (Hz)
6	Angelica Creek	2	Fair: Entrance bump & long. cracking.	Truck 12 Truck 18	0.762 0.816	1.38 1.54	10.9	Excellent: Smooth surface.	0.572	1.08	10.9
7	East Main St.	3	Good: Small transverse cracks	Truck 12 Truck 18	0.392 0.400	1.03 1.04	8.8	Good: Localized areas of cracking	n/a	n/a	8.8
8	Bolivar	3	Good: Bump caused by settlement.	Truck 18	0.672	1.30	7.4	Fair: Entrance decay & long. cracking.	0.552	1.12	7.4
9	Scio	3	Satisfactory: Rough region at entrance.	Truck 12	0.629	1.15	10.1	Good: Entrance indent & cracking	0.577	1.00	10.1

*Deflections have been adjusted to weight of design truck (HS20).
n/a not applicable

Summary and Conclusion

The objective of this study was to determine the dynamic characteristics of glued-laminated timber bridges and to study their dynamic performance relative to time and bridge condition. To complete this objective, nine timber bridges were selected for testing. Five of the timber bridges were glued-laminated girder bridges with three of these bridges being tested in both 1995 and 2003. The four other bridges were glued-laminated panel bridges with all of them being evaluated in both 1996 and 2003. Each of the nine bridges is documented in detail in individual reports entitled *Dynamic Field Performance of Timber Bridges* [25]-[33]. Included within these reports are detailed descriptions of the bridge geometry and condition, field testing procedures, test results, and a discussion of the results.

Conclusion

In general, two components of the bridges were the focus of this report. The first was the bridge condition, including both wearing surface and structural condition. The structural condition of the bridges was found to be good during the testing periods. No excessive cracking, decay or preservative loss was found on any of the bridges. The panel bridges did have noticeable swelling of the individual panels. The wearing surface of the bridges and approaches were found to have conditions ranging from severe to good. The girder bridges were found to have transverse cracking that primarily aligned with the deck panel joints. The panel bridges had longitudinal cracking aligning with the panel joints. Many of the bridges were also found to have rough approaches and entrances

The second component investigated was the dynamic characteristics of the bridges. The performance of each individual bridge varied when compared with pertinent code requirements. As was seen in Figs. 2 and 12, not all DAF values obtained for the bridges were within code requirements. In most cases, however, naturally occurring physical characteristics of the bridges were determined to be the cause of the DAF values that exceeded the requirements. The DAF values of the timber bridges tested in the study, along with DAF values of concrete and steel bridges presented by Billings [5] are shown in Fig. 21. It can be seen that timber, although somewhat sporadic, generally has fundamental frequencies that lie between 5 Hz and 11 Hz as well as a DAF less than 1.25. It is also noted that nearly all the timber bridges tested have natural frequencies between the body bounce and axle hop vehicle frequencies. This frequency grouping is mainly due to the bridges all having short span lengths. When comparing timber with other materials it was found that, in general, the timber bridges appear to have DAF values less than steel values. However, it is also noted that the highest DAF shown in Fig. 21 is for a timber bridge. The concrete values are too random for meaningful comparisons.

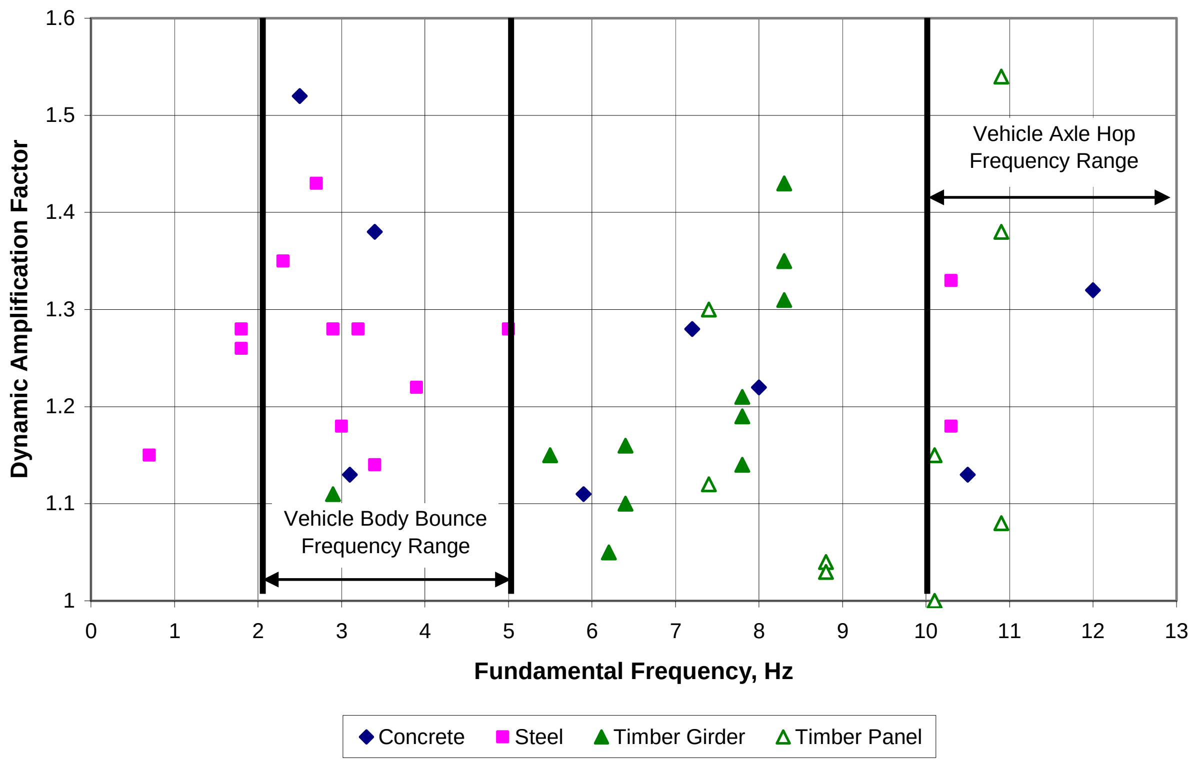


Figure 21. DAF for bridges constructed of various materials.

Based on the field evaluations and observations the following additional conclusions were made for both glued-laminated girder and longitudinal glued-laminated panel bridges:

1. The bridges tested during both testing periods were found to have a decrease in DAF from the mid-1990's to 2003; therefore, the age of the bridge does not necessarily relate to an increase in DAF.

2. The maximum dynamic deflection does not necessarily correlate to the maximum DAF.

3. A rough approach and/or entrance bump has a larger impact on the DAF than does the condition of the bridge wearing surface.

4. Bridge attributes such as moisture content and swelling appear to change the DAF values by primarily changing the overall structural stiffness; although the increase in bridge mass could possibly have a small effect.

5. The natural frequency of the bridges tested were generally unchanging with time and lie between the body bounce and axle hop frequency of typical heavy vehicles.

6. Due to the frequency range of the bridges tested, vehicle load resonance is unlikely.

7. Nine bridges were tested; from those test 20 different DAF values were determined. Sixty-one percent of the DAF values obtained were within AASHTO code requirements and 72% were within the Canadian code requirements. It was also noted that the bridges having DAF values exceeding code requirements generally had rough regions or natural bumps at the entrance of the bridge.

8. The average DAF for the girder bridges was 1.20. The average DAF for the panel bridges was 1.18. The overall average was 1.19, an approximate 42% reduction in the AASHTO load allowance (0.33) specified for steel and concrete materials.

9. Further research needs to be conducted on additional timber bridges to develop a knowledge base on varying timber structures and conditions.

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