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Live Load Deflection of Timber Bridges

Final Report: Conclusions and Recommendations

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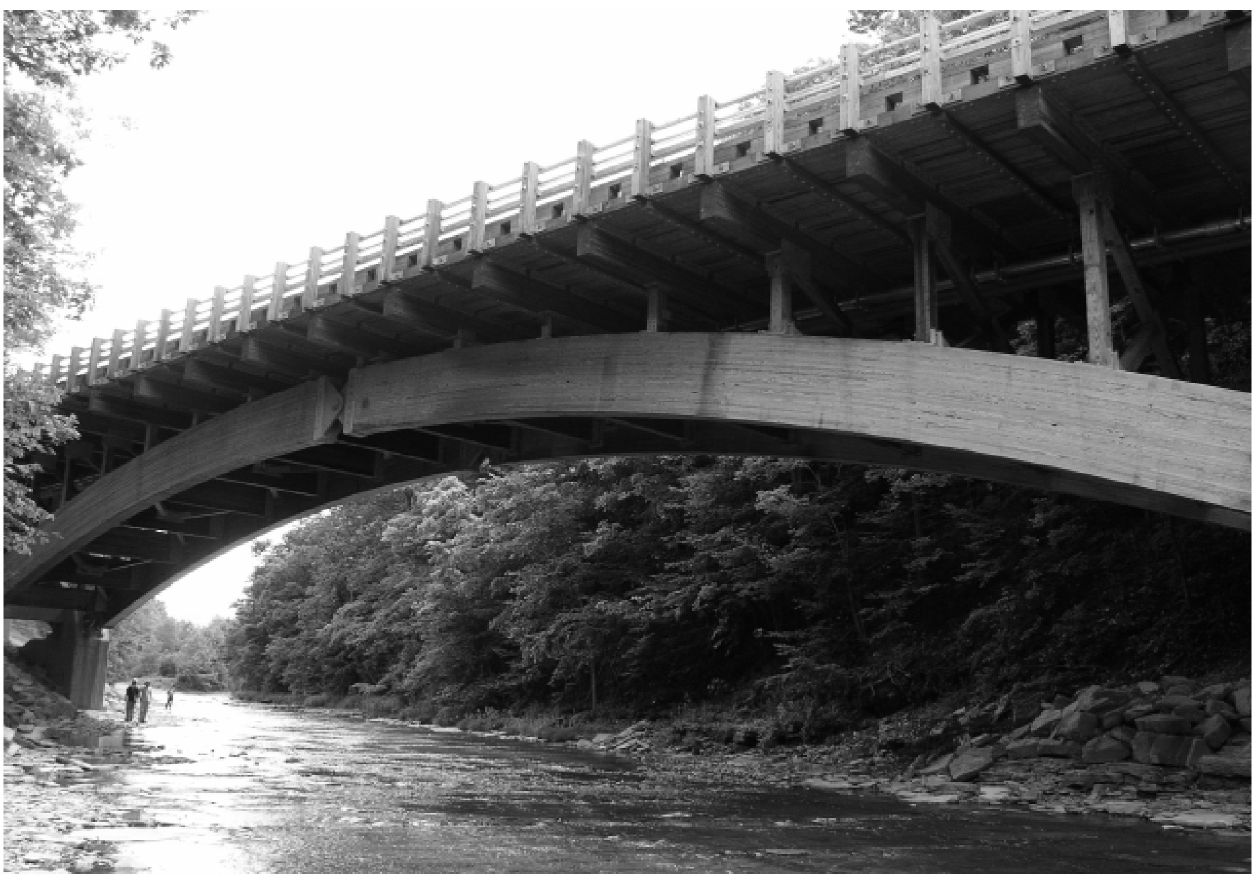

Abstract

In order to promote and increase the use of timber bridges in our nations transportation systems, the United States Department of Agriculture (USDA) and the Forest Products Laboratory funded research to develop design criteria to improve the design of glued-laminated timber bridges. This project is part of this research and is directed towards developing acceptable live load deflection criteria, which are based on the actual structural performance of these types of bridges. Specifically, the relationship between live load deflection and the condition of the asphalt wearing surfaces is of particular interest. To accomplish this, eight glued-laminated timber girder bridges and four longitudinal glued-laminated timber deck bridges were selected for testing. The performance of the bridges was investigated under live loading and analyzed in conjunction with the condition of the wearing surfaces gathered from field inspections. Testing involved loading the structures with fully loaded tandem axle dump trucks and gathering global and differential deflection data. Field tests revealed that the majority of the asphalt wearing surface deterioration was primarily the result of differential panel deflections.

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Contents

Introduction

In 1989, Congress passed the USDA Forest Service Timber Bridge Initiative, now the Wood In Transportation (WIT) Program, to rekindle bridge designers' interests in timber as a construction material. Their main objective was to boost local economies and enhance rural transportation systems using locally available wood species. The success of the program is seen not only in the increase in the number of timber bridges designed, but also in the advancement in timber bridge design specifications and engineered timber products.

"In an era when space-age materials are as ubiquitous as the cell phone and laptop computer, a new-old building technique is spanning the gap between nostalgia and the modern-day demand for high performance," said Craig Savoye, [27]. Timber bridges, once the prevalent choice of bridge designers, are making a comeback. In the last decade, approximately 2,500 timber bridges were built nationwide according to the Federal Highway Administration. The retro trend may be developing at an opportune time.

Research and development in timber design is the most significant reason for the increase in the number of timber bridges being built. In the past, most engineers' knowledge and/or experience with timber design were minimal to non-existent. This, along with the fact that timber design specifications were severely inadequate compared to those for reinforced concrete and steel, led to some insufficient designs and poor performance. In 1993, the American Association for State Highway and Transportation Officials (AASHTO) adopted the load and resistance factor design (LRFD) code for bridges. However, unlike concrete and steel for which an LRFD procedure has been available for several years, LRFD specifications for wood are still in developmental stages.

LRFD serviceability requirements for timber are one area in which there has been little research. In particular, a live load deflection criterion of span length divided by 425 is currently specified. However, this is considered an optional requirement and left to the designer's judgment. These deflection criteria based upon arbitrary limits and there exists a need for design criteria for timber superstructures and decks based upon actual structural behavior, user perception, and wearing surface performance.

As a result of the Timber Bridge Initiative and the Intermodal Surface Transportation Efficiency Act (ISTEA), passed by Congress in 1988 and 1991, respectively, funding was made available for timber bridge research. One portion of this research focuses on refining and developing design criteria for wood bridges. This project, to investigate and develop live load deflection criteria based on actual bridge performance, is part of that program and is a cooperative effort between Iowa State University, the USDA Forest Service, and the Forest Products Laboratory. The project looks at design criteria currently used by bridge engineers, the actual behavior and deterioration of timber bridges due to live load deflection, and the development of acceptable live load deflection criteria to be implemented into the design code immediately.

Timber has been used as a bridge material for centuries. However, many of the timber bridges in use today are deficient or in an advanced state of deterioration. This deficiency however, is not the result of wood being a inadequate bridge material. This poor performance is instead simply due to poor and inadequate design of timber bridges in the past.

Properly designed, modern timber bridges possess several advantages: durability and long life, simplicity of construction, prefabricated construction, high strength-to-weight ratio, cost competitive in small bridge construction, aesthetics, and immunity to the deteriorating effects of deicing chemicals like sodium chloride. The use of modern engineered lumber products in conjunction with a dependable and up to date design code will not only increase the number of timber bridges, but more importantly, the perception and quality of timber bridges.

This report summarizes the results of a series of twelve reports aimed at collecting, analyzing, developing and distributing information on the relationship between timber bridge deflection, wearing surface, and overall bridge performance. Field inspections in conjunction with data collected from field tests on 8 glued-laminated girder bridges and 4 glued-laminated panel bridges were used to investigate the deflection performance of such bridges and the subsequent effect on wearing surface performance. The results of these tests are found in 12 reports titled *Live Load Deflection of Timber Bridges* ([31]-[42]). The subsequent discussion briefly describes each bridge and its performance under static loading. Recommendations for limiting differential deflection, and common factors found to significantly affect the deterioration of the wearing surfaces are then presented based on the performances of the tested bridges.

Objective and Scope

The overall objective of this research was to study the relationship between live load deflection and asphalt wearing surface condition and to make recommendations for timber bridge live load deflection criteria. The project scope included data collection and analysis under static truck loading and studying its effect on the wearing surface and overall bridge performance. The results of this testing will be used to formulate recommendations for design specifications related to deflection criteria to be used on similar glued-laminated timber bridges.

In the past decade, extensive work by Iowa State University (ISU) and the Forest Products Laboratory (FPL) in Madison, WI has led to the advancement of existing design methodologies for various types of timber bridges. However, as discussed previously, little of this work has been directed towards serviceability issues. Hence, the objective of this study is three-fold:

1. To research the importance of and allowable limits for differential deflection as they relate to the long-term performance of timber bridges.

2. To develop live load deflection criteria for timber superstructures and decks based upon actual structural behavior, performance of wearing surfaces, and user perception.

3. To develop relationships between deflection data and specific deterioration modes using appropriate statistical models to obtain a complete picture of the source and significance of deflection-induced deterioration.

The scope of this project involves three general tasks: a literature review, an experimental field-testing program, and the development of a final report. A literature review was conducted to build on previous research data and avoid unnecessary repetition of data collection. Literature on the development of current deflection criteria, types of observed deflection induced deterioration, and the human perception of bridge deflection and vibration was collected and presented in brief. Field tests were done to monitor overall bridge behavior, total and differential deflection effects, and determine the source and significance of deflection induced deterioration.

Literature Review

Significant literature exists concerning timber bridges and promoting the future development and use of these types of bridges now and especially in the future. Researchers and engineers have tested, studied, and proven the effectiveness of timber bridges for use in the nation's, and world's, transportation systems. Effective or not, however, the perception of timber as a bridge material is still met with skepticism. [29] conducted significant research concerning the designers' perception of timber as a bridge material. The authors discovered that in locations where bridge ratings were low, so too were the designer's perception of timber as a bridge material. In addition, they found that poor design was the main reason these bridges became deficient in the first place. They concluded that perception of timber in the past has been low because a majority of our nations timber bridges were not designed to adequate standards.

In regards to serviceability criteria, little research has been done to develop timber specific design criteria for bridges. Currently, deflection criteria applied to timber bridges are based on arbitrary limits and are most often the same as those used for steel and concrete bridges. The biggest problem with serviceability limit states, according to [20], stems from the fact that they are defined by human perception rather than structural conditions. The authors comment that little has been done so far to develop design criteria for serviceability conditions; those criteria that do exist are recommendations rather than limits based on the performance and deterioration of the structure and are left to the engineers' judgment.

Research done by [24] found similar results to those of Nowak and Grouni. In his search for maximum recommended deflection-values, he found that they ranged from L/200 to L/1,200. [24] also recommends values for maximum deflection of timber members: L/360 for applied loads and L/240 for combination of applied loads and dead loads. The author suggests a more severe deflection limit in the presence of an asphalt wearing surface and/or pedestrian walkway.

An article by [43] discusses the history of bridge deflection criteria. The authors found that deflection criteria began with the railway bridge specification in 1871. Approximately sixty years later, deflection criteria recommended by the Bureau of Public Roads, which are similar to current limitations, seemed to be based on user discomfort. Recently, more severe limitations appear to have resulted from concern about deterioration of reinforced concrete decks. The author's conclusion was that discomfort and deck deterioration seem to be the only factors considered in developing current deflection criteria. The authors also stated that the static component of highway bridge deflection has negligible effect on human response; users only perceive bridge vibrations when they are standing or sitting on the bridge itself, or when they are in stationary vehicles.

The performance and deterioration of wearing surfaces on timber bridges due to the bridge's actual behavior has also posed questions and stimulated research by several bridge engineers and researchers. [17] conducted a research study to identify the primary mechanisms responsible for wear and surface deterioration of asphalt wearing surfaces. They looked at two basic bridge types: longitudinally nail-laminated superstructures and transverse nail-laminated decks supported by beam superstructures. From their inspection of the bridges, three common forms of mechanically induced pavement cracking were revealed in the wearing surfaces: cracking parallel to deck laminations, cracking parallel to deck panel joints, and transverse cracking over deck supports. In all cases, the cracking was due to some form of differential deflection, whether it was differential deflection between individual deck laminates, between deck panels, or between the deck and the supports. They concluded that the common factor in pavement deterioration is poor pavement design, and a more flexible pavement surface design was suggested to reduce surface cracking on these types of structures.

[30], also found that the long-term performance of timber bridges has often been deemed unsatisfactory due to cracking of the wearing surface. They, like [17], found that most of the cracking was due to differential deflection. Their research led to the development of a three-layered wearing surface system to prevent cracking in the road surface. This system performed well in the laboratory based on differential deflection at panel joints of 0.05 in. (1.27 mm), which is half of the differential deflection recommended in the design of deck panels, 0.1 in. (2.54 mm) [24].

Currently, most research testing of timber bridges has been done on stress-laminated and glued laminated timber bridges. Much of the work on stress-laminated timber deck bridges was done by [26] in 17 *Field Performance of Timber Bridges* reports. These reports showed that the performance of the wearing surfaces varied from bridge to bridge. They found several bridges had excellent performing wearing surfaces with little to no cracking, rutting, or heaving. They also found several of the bridges had poor performance of the asphalt wearing surface with significant cracking, heaving, and rutting along the wheel lines. The authors concluded the main reason for this variance was due to variations in asphalt mix design, as well as, poor asphalt mix design in the first place.

Additional work on stress-laminated timber deck bridges was done by [23]. The objective of that study was to evaluate the overall performance of the bridges under static loading conditions. The report stated three main conclusions: the bridge performed well under static loading, there were no indications of deterioration in the timber components, and the asphalt wearing surface was in good condition with only minor transverse reflective cracking over the bridge abutments.

Valuable information was obtained on behavior and analysis of timber bridges from research projects conducted at Iowa State University. In [14], stress-laminated timber bridge experiments were conducted after which a general-purpose finite element software program was constructed to model and analyze such bridges. [16] developed a similar finite element software program that is applicable to longitudinal glued laminated deck and girder bridges. Development of the model began by obtaining static deflection data for two longitudinal glued-laminated deck bridges and four glued-laminated girder bridges.

The New York State Department of Transportation initiated a research project to systematically study the possible correlation between bridge deck cracking and bridge vibration [4]. Data concerning the vibration and cracking of steel girder bridges with concrete decks were analyzed using statistical methods. Several remedial measures were considered, including modifying the deflection criteria recommended by AASHTO bridge design standards; a recommendation to further study this relation using quantitative data was made.

In a study done by [11], the authors found that much of the deterioration seen in bridges not only leads to loss of structural integrity, but that the two together are detrimental to the structure and its users. They proposed a need to combine inspection techniques for detecting localized flaws with a comprehensive assessment strategy estimating their cumulative effects on overall structural integrity and strength. Further, they believe that proper applications of nondestructive evaluation (NDE) techniques will present a more confident assessment of material properties and, in turn, structural integrity and residual capacity.

Glued-Laminated Timber Girder Bridges

Background

Timber structures are cost effective, easy to construct, have a long service life if properly preserved, and are relatively low maintenance. However, many of these structures have experienced rapid and repeated deterioration of the wearing surface, leading to the subsequent degradation of the underlying structural components. One of the more common types of timber bridges constructed is a glued-laminated timber girder bridge with a transverse glued-laminated timber deck. These structures vary in width depending on the number of traffic lanes, and range in length from 20 to greater than 100 ft.

To investigate the source of the deterioration commonly seen in the wearing surfaces on these types of bridges, eight bridges that met the needs of this project were located, inspected, and field-tested. Data collected from visual inspections and field load tests were analyzed and presented previously in eight individual reports titled *Live Load Deflection of Timber Bridges* ([31]-[42]). The results from these reports are the basis for this section of this report.

There were two types of wearing surfaces on the eight bridges tested. The majority of the bridges had asphalt wearing surfaces, which varied in depth from 2.5 to 6 in. The Badger Creek Bridge had a longitudinal plank wearing surface covering the entire width of the roadway. The Camp Creek Bridge incorporated a combination of both longitudinal planks and asphalt wearing surfaces, consisting of longitudinal planks in the wheel lines and asphalt over the remainder of the deck.

Results

The deflection performance of each bridge and the condition of its wearing surface related to that performance will be discussed in short individually followed by a discussion of the overall performance of the eight bridges and the effects of live load deflection on wearing surface condition. Deflection checks for bridges are evaluated based on deflection criteria typically of the form L/n, L being clear span in inches. Listed in Table 1 are the deflection criteria found in [1], [2], and [24], the current design specifications and guide manual. In addition, [24] also suggests limiting panel deflection relative to the girders as well as differential panel deflection to 0.1 in. A further reduction in this limit is suggested in the presence of an asphalt wearing surface. The performance of the subject bridges will frequently be compared to these criteria in the subsequent discussion.

For comparison, the value of n using the maximum measured deflections from all load cases investigated on these eight bridges are listed in Table 2. Since the deflection criteria found in the specifications are based on the design truck, the experimental n-values were normalized for comparative purposes, by total truck weight, to the design truck used for that specific bridge.

The large difference between the recommended deflection criteria and that obtained from the experimental n-values may be attributed to several factors. The girders may have been initially over designed to reduce deflections or the deflection limit state may not have controlled. Transverse load distribution from girder to girder via the deck panels may be greater than typically assumed in design. In addition, changes in moisture content, support conditions, and other factors may result in smaller deflections than those predicted in design. These factors will all be discussed in the following sections.

Table 1. Deflection Criteria.

Source	Deflection Criteria
Ref. [1]	L/500
Ref. [2]	L/425
Ref. [24]	L/360

Table 2. Experimental n-values, girder bridges.

Report Number	Bridge	Load Case	Experimental n-value (1995)	Experimental n-value (2002, 2003)
1	Badger Creek	1	-	1110
		2	-	1178
2	Camp Creek	1	-	1370
		2	-	1750
		3	-	2300
3	Lost Creek	1	-	2110
		2	-	2110
4	Erfurth	1	-	532
		2	-	542
5	Chambers County	1	-	1070
		2	-	795
		3	-	747
		4	948	912
		5	1143	1092
6	Russellville	1	-	1055
		2	-	995
		3	-	985
		4	1140	1470
		5	980	-
		6	1375	-
7	Wittson	1 (span 1)	-	728
		2 (span 1)	1521	1196
		3 (span 1)	1504	-
8	Butler County	1	-	840
		2	-	590
		3	-	588

The Erfurth Bridge, located near Mount Vernon, WI was selected for evaluation because of its relatively thin (3.5 in.) timber panel deck. For detailed plan and cross-sectional drawings of the Erfurth Bridge see [34]. In short, the structure is a two-lane bridge spanning approximately 40 ft with 12 glued-laminated girders spaced 31 in. on center and a transverse glued-laminated panel deck consisting of 4 ft – 4 in. by 3.5 in. panels. The 12 glued-laminated girders are each composed of two separate sections, which make a cross-section similar to that illustrated in Fig. 1.

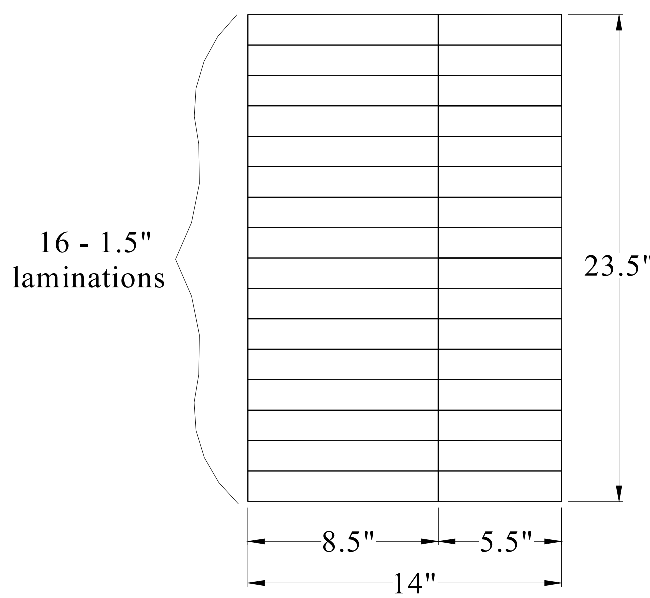

Figure 1. Erfurth Bridge girder cross-section.

The maximum girder deflection and maximum differential panel deflection were –0.85 in. and 0.03 in., respectively. Panel deflections measured relative to the girders were found to be less than 0.08 in. Note that both the differential panel deflection and the deflection of the panels relative to the girders were less than the recommended limit of 0.1 in.

Figure 2 illustrates the condition of the asphalt wearing surface on the Erfurth Bridge at the time of testing in 2003. The maximum deflection of –0.85 in. was measured at girder G4 in both test cases. Deflection of girder G5 was approximately 0.4 in., which is half of that at girder G4, and girder G3 had even smaller deflections. The exact source of the large difference in deflections between girder G4 and the adjacent girders was unable to be determined. Possible sources include: gaps between the deck panels and girders G3 and G5 but not G4, differences in support conditions and end restraints, localized deterioration of girder G4 which is not evident from visual inspection, as well as other factors. Inspection of the wearing surface found longitudinal cracks in the asphalt wearing surface above girder G5 immediately west of G4. The large differential girder deflection between girder G4 and G5 is believed to be the cause of the longitudinal cracking in that area shown in Fig. 2.

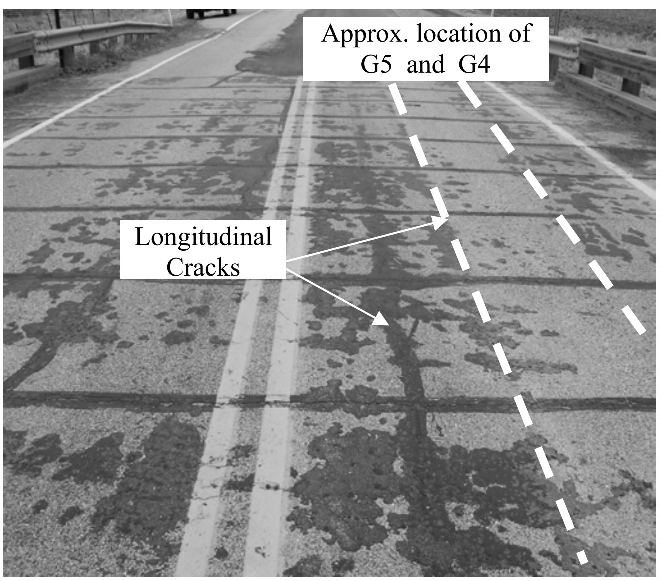

Figure 2. Erfurth Bridge wearing surface in 2003.

In addition to the longitudinal cracks above girder G5, transverse cracks located above each panel joint were evident as shown in Fig. 2. The transverse cracks were full depth and had been patched with sealant. The pattern of cracking suggests one or a combination of the following factors may be responsible: the relatively thin flexible deck, differential panel deflections, and/or the magnitude of the panel deflections relative to the girders. Although the differential panel deflections were typically significantly less than the recommended limit, it is believed that this deflection in combination with the above factors is the cause of the deterioration of the asphalt wearing surface. In addition, the presence of the miscellaneous cracking throughout the deck surface may be the result of the flexible deck and, as past research has shown, the asphalt mix design itself.

The Badger Creek Bridge [31], located near Mount Hood, OR spans 31 ft and consists of four glued-laminated girders, a transverse glued-laminated deck, and the longitudinal timber plank wearing surface shown in Fig. 3. The bridge showed no signs of deterioration, deflection-induced or otherwise, in the longitudinal plank wearing surface or in any other structural component. Inspection found that the panel joints on this bridge were difficult to locate and no signs of moisture ingress between the panels were evident, suggesting that there was a tight fit between the deck panels. Similarly, the panels appeared to be well seated on the girders with no visible gaps.

Maximum midspan girder deflections were less than approximately –0.3 in. Panel deflections relative to the girders and maximum differential panel deflections were found to be well within the 0.1 in. limit (differential panel deflections calculated from the measured deflections were typically less than 0.015 in). These relatively small differential panel deflections are possibly affected by the longitudinal plank wearing surface, which may reduce differential deflections by effectively transferring load longitudinally from panel to panel. Due to the lack of deterioration in the plank wearing surface, the differential panel deflections and live load deflection behavior of the bridge in general, do not appear to be affecting the condition of the wearing surface on the Badger Creek Bridge.

Figure 3. Badger Creek Bridge wearing surface in 2002.

The Camp Creek Bridge [32], located near Mount Hood, OR is a single lane bridge spanning 31 ft that uses longitudinal planks for the wearing surface along the wheel lines (see Fig 4). The remainder of the deck is covered by an asphalt wearing surface. As with the Badger Creek Bridge, the Camp Creek Bridge consisted of four glued-laminated timber girders and a transverse glued-laminated timber deck.

There were several uncharacteristic behaviors evident in the live load deflection of the Camp Creek Bridge. Deflections measured at midspan were typically less than those measured at ¼-span which is uncommon for this type of structure. In addition, the girder and panel deflections follow a stair step pattern upon initial loading, plateau at a peak deflection, and then resume the stair step pattern as the deflections decrease. These two behaviors may be caused by transfer of load longitudinally through the timber planks and/or swelling of the deck panels due to increases in moisture content. However, the cause is unknown.

From Table 2 it is evident that the deflection performance of the structure is within specified limits despite the uncharacteristic behavior. Maximum girder deflection for the Camp Creek Bridge, normalized to the design truck, was approximately –0.28 in. Load distribution factors approximated using the measured deflections were less than those calculated using the current design specifications. This suggests that the bridge resists deflections more effectively than anticipated in design. Additionally, a simple static analysis found that the support conditions are more like fixed ends than the pinned condition typically assumed in design.

Signs of deterioration were evident in both the longitudinal plank and asphalt wearing surfaces on this bridge. This deterioration is not believed to be directly the result of live load deflections. Rather, the deterioration appears to be from the weather and traffic wear, and possibly only compounded by live load deflections. The basis for this conclusion is that deterioration of the longitudinal planks was only evident in the exterior two planks in each wheel line but not in the interior planks. As with the longitudinal planks, the deterioration of the asphalt is not believed to be the result of load-induced deflections, but mainly to the detachment of the asphalt from the deck panels and wear from traffic and weather. In addition, accumulation of debris on the deck is possibly trapping moisture resulting in the accelerated deterioration of the deck panels and wearing surfaces.

a. Longitudinal planks on wheel lines.

b. Exposed deck panels due to asphalt deterioration.

Figure 4. Camp Creek Bridge wearing surface in 2002.

The Wittson Bridge [35], located northeast of Tuscaloosa, AL is a four-span bridge with variable span lengths and variable depth girders. One of the 50 ft spans and the 102 ft span were selected for testing, although limited data was able to be collected from the long span due to height restrictions and accessibility limitations. Comparison of data collected previously in 1995 with the data collected during the recent testing in 2003 indicates that the deflection performance of the bridge was variable. Comparing the n-values for the two Wittson Bridge spans in Table 2 with the recommended deflection criteria in Table 1, it is evident that the bridge was within acceptable limits in terms of girder deflections in both years. Maximum girder deflections for span 1 varied from approximately 0.5 in. to 1.0 in. in both 1995 and 2003 depending on the load case.

Despite the apparently acceptable deflection performance of the bridge, the pattern of transverse cracking on this bridge was uncommon compared to the other bridges tested in this research. Figure 5 illustrates the condition of the wearing surface on the Wittson Bridge. Transverse cracks were evident directly over the panel joints for approximately 85 percent of each span, but over approximately 10 ft on each side of the piers, the transverse cracking ceased. Continuity across the piers prevents stress reversals above the piers at the deck level, which is believed to be the source of the transverse cracking pattern shown in Fig. 5.

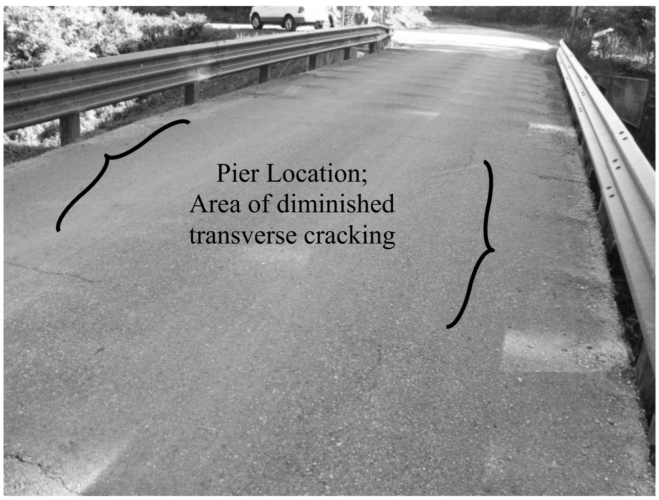

Figure 5. Wittson Bridge wearing surface in 2003.

Panel deflections relative to the girders were approximately 0.03 in. in 1995 and 0.004 in. in 2003. Maximum differential panel deflections on the short span were approximately 0.1 in. in 1995 and decreased by approximately 75 percent in 2003. Both the relative deflections as well as the differential panel deflections are within the acceptable limit of 0.1 in. Differential panel deflections in 2003 observed for the long span were negligible. The calculated differential panel deflections for spans 1 and 3 coupled with the pattern of transverse cracking, suggests that differential deflections may be affecting the deterioration of the wearing surface.

Significant stair stepping was also evident in the deflection pattern of the girders and deck panels. This stair stepping behavior and the decrease in differential panel deflections from 1995 to 2003 are believed to be due to increases in moisture content, and the subsequent swelling of the deck panels. The swelling increases the pressure between the panels, which in turn increases the contact friction between the panels. The repeated build up and release of friction between adjacent deck panels as the load passes over the bridge is likely the source of the stair stepping behavior. This repeated build up and release of forces between the deck panels may be another factor affecting the transverse cracking of the asphalt.

Longitudinal cracks were not evident in the asphalt wearing surface on the three short spans of the Wittson Bridge, but several significant longitudinal cracks were evident on span 3. These longitudinal cracks may be the result of differential girder deflection between the exterior two girders, which were on the order of 0.3 in.. However, the exact cause is unknown.

The Russellville Bridge [36], located near Russellville, AL is also a four-span bridge; however, unlike the Wittson Bridge each of the four spans is equal length. The bridge consists of four 42-ft spans, five glued-laminated girders, and a transverse glued-laminated deck. The Russellville Bridge is a two-lane structure with transverse cracks along the full length of the bridge at panel joint locations, and only minor longitudinal cracking. The condition of the wearing surface at the time of testing in 2003 is shown in Fig. 6.

Figure 6. Russellville Bridge wearing surface in 2003.

The presence of transverse cracks across the full length of the bridge suggests that the two four-span bridges behave differently under live loading. Comparison of the time-history deflections for both bridges indicates that both exhibit continuity across the piers. However, maximum midspan girder deflections for the Russellville Bridge are approximately half of those from the Wittson Bridge, for similar length spans. The one difference between the two that might result in differences in deterioration of the wearing surfaces over the piers is the girder spacing. The Russellville Bridge has a girder spacing of 5 ft; the Wittson Bridge has a girder spacing of 4 ft – 3 in., and both bridges have similar size deck panels. The greater distance between girders for the deck panels on the Russellville Bridge may produce greater differential deflections above the piers resulting in continuous deterioration of the wearing surface along the full length of the bridge. In addition, the effective deck span for the Russellville Bridge is 58.25 in. (clear span plus ½ the girder width or clear span plus the panel thickness, whichever is greater), exceeds the acceptable range of 50 – 57 in. for noninterconnected deck panels [24].

The Russellville bridge was found to have maximum panel deflections relative to the girders of approximately 0.01 in. in 1995 and 0.07 in. in 2003. Maximum differential panel deflection for the Russellville Bridge was approximately 0.2 in. in 1995 and 0.026 in. in 2003. Once again, swelling of the deck panels over time due to increases in moisture content is believed to be the cause of the decrease in differential panel deflections between 1995 and 2003. Additionally, the swelling of the panels is believed to be the source the stair stepping behavior evident in the Russellville deflection data.

Girder deflections for span 1 increased slightly from 1995 to 2003 from approximately –0.45 in. to –0.57 in., and maximum differential girder deflections were typically less than 0.2 in. Only minor longitudinal cracking was evident in the wearing surface, which may or may not be attributed to these differential girder deflections.

From the above information, the deterioration of the Russellville Bridge wearing surface is believed to be a result of the differential panel deflections, which may be impacted by the girder spacing. In addition, asphalt mix design may be affecting the deterioration of the wearing surface on the Russellville Bridge.

There were two more multi-span bridges tested as part of this work: the Lost Creek Bridge and the Butler County Bridge. The Butler County Bridge [38], located near Georgiana, AL consists of one 24-ft span and one 60-ft span; however, only the 24 ft span was tested. The structure is a two-lane bridge consisting of five glued-laminated girders and a transverse glued-laminated deck.

The problems inherent in the Butler Co. Bridge were immediately apparent upon initial inspection of the wearing surface and deck panels. Severe full-width transverse cracking was evident at every deck panel joint along the entire length of the bridge as shown in Fig. 7. The transverse cracks were approximately 2 in. wide and in most cases, the moisture barrier between the deck and the asphalt was visible and often severed. These cracks were found to be the result of severe cupping of the deck panels, concave upwards, which is believed to be the result of changes in moisture content, which may have been compounded by initially being slightly cupped at the time of installation. Figure 8 illustrates the cupping of the deck panels, and the resulting gaps between the deck and the girders. The cupping of the panels has resulted in gaps between adjacent deck panels of approximately 0.5 in.

Figure 7. Butler County Bridge wearing surface deterioration.

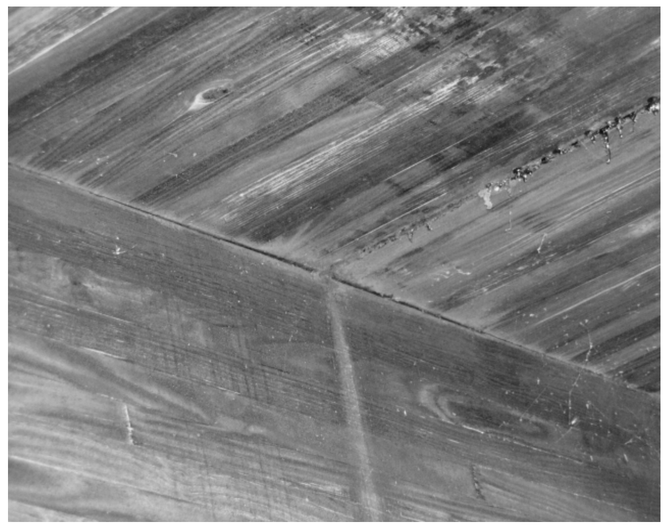

Figure 8. Cupping of deck panels on the Butler Co. Bridge.

The global deflection performance of the Butler Co. Bridge was within recommended limits as shown in Tables 1 and 2. Girders near and directly under the load resisted the greatest percentage of the load, while those away from the load resisted a significantly smaller percentage. Distribution factors approximated from the measured deflections suggest that the equations used to approximate the distribution factors in the design code are conservative for all girders, except the exterior girders when the load truck is positioned near the curb.

The deflection behavior of the Butler Co. deck panels is different from what is typically seen for bridges of this type, largely due to the cupping of the deck panels. Typically, the differential panel deflections will increase once for the passage of one axle. In the case of the Butler Co. Bridge, due to the cupping of the deck panels the differential panel deflections increase three times for the passage of one axle.

This behavior is illustrated in Fig. 9. The largest differential panel deflections occurred when one rear tandem axle was positioned similar to the top figure in Fig. 9 and the other rear tandem axle was positioned similar to the bottom figure in Fig. 9. This configuration produced differential deflections of approximately 0.18 in.

Panel deflections relative to the girders were approximately 0.15 in., and both these deflections and the calculated differential panel deflections were greater than the recommended limit of 0.1 in. The cupping of the panels not only increases the magnitude of the differential deflections, but just as important, also increases the number of stress reversals in the wearing surface. Thus, the cupping of the deck panels is both directly and indirectly responsible for the cracking, raveling, and disintegrating of the asphalt above the panel joints.

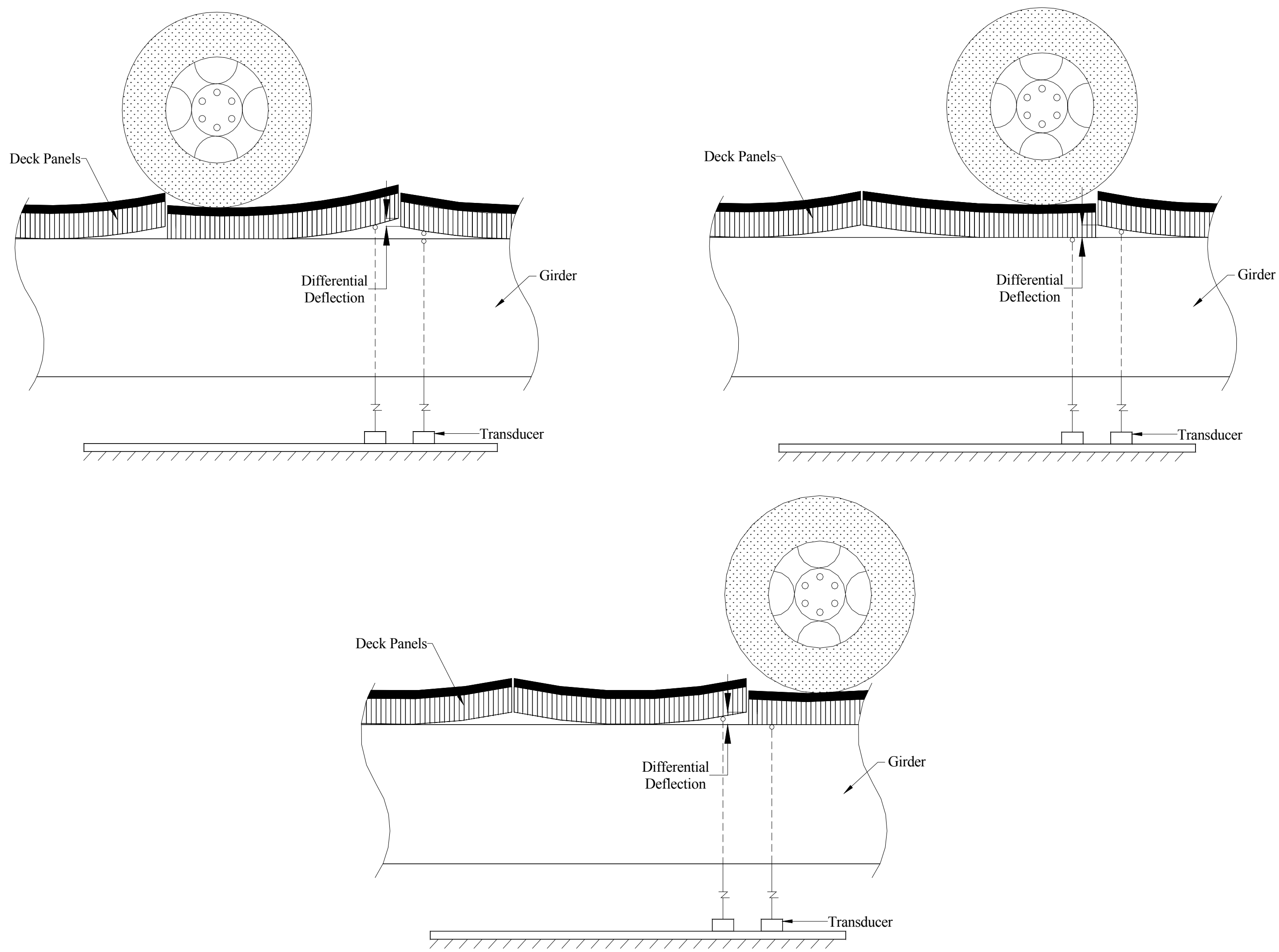

Figure 9. Illustration of differential panel deflection.

The Lost Creek Bridge [33], located near Mount Hood, OR is a three-span bridge consisting of a 47-ft main span and two 14 ft – 3 in. end spans that essentially cantilever over the piers. The structure is composed of three full-length glued-laminated girders, a transverse glued-laminated deck interconnected with steel dowels, has a timber side walk on one side, and a slight taper at one end.

The performance of the Lost Creek Bridge under static live loading was within recommended limits. Global girder and panel deflection were both within acceptable limits although as with the Wittson and Russellville Bridges, a stair step pattern was evident in the deflection diagrams. The stair step pattern is believed to be due to the swelling of the deck panels in combination with the presence of the steel dowels. Maximum differential girder deflection for the main span was approximately 0.13 in. when the load truck was positioned either near the curb or near the sidewalk.

Figure 10 shows the condition of the asphalt wearing surface on the Lost Creek Bridge. Due to complications in the field, differential panel deflections were unable to be determined. However, panel deflections were calculated relative to the girders and were approximately 0.05 in., which is less than the recommended limit of 0.1 in. Therefore, due to the lack of longitudinal and transverse cracking in the asphalt wearing surface along with the magnitude of the relative girder and panel deflections, the live load deflection behavior of the bridge does not appear to be affecting the condition of the wearing surface on the Lost Creek Bridge.

Figure 10. Lost Creek Bridge wearing surface in 2002.

The final glued-laminated girder bridge tested was the Chambers County Bridge [37], located near Auburn, AL. This structure spans 51 ft – 6 in., is a two-lane, single span bridge consisting of six girders with a transverse glued-laminated panel deck. Inspection of the wearing surface at the time of testing found several areas of deterioration, which has occurred in the three years since the asphalt had been placed in 2000. Judging by photographs from previous inspection reports similar levels of deterioration were evident in the wearing surface prior to 2000 as well. Figure 11 illustrates the condition of the asphalt wearing surface on the Chambers Co. Bridge in 2003. Deterioration of the asphalt ranged from transverse cracking above the panel joints, other minor transverse cracking, small potholes, and raveling. In addition to the deterioration of the wearing surface on the bridge, the asphalt approaches were also significantly deteriorated. The asphalt in one lane of one approach was almost completely deteriorated and had been replaced with limestone.

Figure 11. Chambers Co. Bridge wearing surface in 2003.

From Table 1 and 2, it is evident that the overall deflection performance of the structure is within recommended limits (maximum girder deflections were approximately –0.65 in. in 1995 and –0.85 in. in 2003). The increase in the girder deflection from 1995 to 2003 could be attributed to any number of factors and is not uncommon. More importantly, the larger deflection measured in 2003 is still within current recommended limitations. The lack of longitudinal cracking in the wearing surface is the only indication that the differential girder deflections, which were typically less than 0.25 in., are not a significant factor affecting the condition of the wearing surface.

However, transverse cracking over the panel joints suggests that relative and differential panel deflections are critical to the performance of the wearing surface. Maximum differential panel deflections calculated in 1995 and 2003 were 0.08 in. and 0.06 in., respectively. As with several other bridges, based on the level of reduction applied to the 0.1 in. limit due to the asphalt wearing surface, these deflections may or may not be significant. Panel deflections relative to the girders in both years were found to be less than the recommended limit of 0.1 in. Potholes found in the asphalt suggest that some other factor, in addition to differential panel deflection, is causing the deterioration of the wearing surface. Possible sources include: the asphalt mix design, asphalt placement procedures, accumulation of debris causing further deterioration, as well as other factors. The deterioration of the asphalt approaches, which could simply be the result of a poor sub-base, may also indicate that the asphalt mix is partially responsible. Considering all this, it is evident that differential panel deflection is at least partially responsible for the deterioration of the asphalt wearing surface, although other factors appear to be contributing as well.

Discussion

The structural performance of the eight glued-laminated girder bridges tested for this project, in terms of global deflection, was found to be adequate and within recommended limits for all load cases investigated. This is summarized in Tables 1, 2, and 3. Table 3 also lists all 8 bridges along with key physical attributes of each bridge, maximum girder and differential panel deflections from the 1995, 2002, and 2003 load tests, and the n-values calculated from those measured deflections.

Inspection of the bridges both before and concurrent with testing found signs of at least some level of deterioration in all components except the girders. Examples of the types of deterioration found include: cupping of the deck panels, splitting of longitudinal wearing planks, increases in moisture content of the deck panels, cracking and potholes in the asphalt, and gaps between the deck panels. From the collected data, live load deflection is believed to be partially responsible for the deterioration found in the wearing surfaces on these bridges.

There were two types of wearing surfaces used on the eight timber girder bridges tested, longitudinal planks and asphalt. The performances of these two different wearing surfaces under live loading was as different as the wearing surfaces themselves. The longitudinal plank wearing surfaces, utilized on the Badger Creek and Camp Creek Bridges, performed exceptionally well under live loading. Some signs of deterioration were evident in the longitudinal planks on the Camp Creek Bridge; however, this was determined to be the direct result of the weather conditions and traffic induced wear and, at most, indirectly by live load deflections. The Badger Creek Bridge showed no signs of deterioration of its wearing surface and differential panel deflections were found to be small, as shown in Table 3. These small differential panel deflections may be partly the result of the longitudinal planks themselves. The planks are not designed, nor intended, to distribute loads transversely or longitudinally on the structure. Nevertheless, these panels may be doing exactly that, transferring the load longitudinally from panel to panel, reducing differential panel deflections and thereby preventing deterioration of the planks themselves.

Although differential panel deflections were unable to be calculated for the Camp Creek Bridge, the similarities in span, girder size, panel size, and girder deflections with those of the Badger Creek Bridge suggests that differential panel deflections would be similar as well. However, this structure did exhibit some uncharacteristic behaviors in terms of deflection response to loading. These behaviors are believed to be due to the localized transfer of load longitudinally by the wearing planks, increased stiffness provided by the planks at the interior of the bridge, and the large curb sections providing additional stiffness to the exterior of the bridge. The nature of these behaviors and the deterioration evident in the planks and asphalt wearing surfaces on the deck suggest that factors other than live load induced deflections are responsible.

Based on the measured deflections and the condition of the wearing surfaces for both the Badger Creek and Camp Creek bridges, the performance of single lane glued-laminated timber girder bridges, which utilize longitudinal timber planks for a wearing surface, is above average. The longitudinal planks appear to have the affect of distributing the load longitudinally from panel to panel, thereby reducing the differential panel deflections. In addition, the timber planks appear to withstand live load deflections rather effectively.

For the most part, the condition of the asphalt wearing surfaces on the other six bridges was poor with one exception, the Lost Creek Bridge. The Lost Creek Bridge was the only bridge to have no signs of cracking, transverse or otherwise, in its asphalt wearing surface. The other five bridges had significant transverse cracking in the asphalt wearing surface along with minor transverse and longitudinal cracking as well.

Whether or not the lack of cracking in the wearing surface of the Lost Creek Bridge is due to small differential panel deflection is unknown since differential panel deflections were unable to be calculated. The possibility exists that the condition of the wearing surface is preserved by the design and resulting deflection performance of the bridge. The three-span continuous glued-laminated timber girders span the long center span and are cantilevered over the piers creating the short end spans. The abutments consist of timber backwalls attached to the girders with steel angles and through bolts. This configuration, large center span with short cantilever end spans, in addition to this being a single lane bridge, may possibly reduce the deflections and subsequently prohibit the development of cracks in the asphalt wearing surface. In addition, the presence of steel dowels connecting adjacent deck panels together may be reducing the differential panel deflections. As shown in Table 3, the maximum deflections for this bridge are small, both in magnitude as well as when comparing the corresponding n-value of 2032 with the recommended deflection limit of L/360.

The remaining five bridges (Erfurth, Chambers Co., Wittson, Russellville, and Butler Co.) had varied but significant levels of deterioration in their asphalt wearing surfaces. The majority of the deterioration was transverse cracking in the asphalt directly above the panel joints. For some bridges, these cracks were along each panel joint, for other bridges the cracks were only over some of the panel joints. Moreover, the cracks ranged in width from minor hairline cracks to cracks nearly 2 in. in width.

From the data in Table 3, it is evident that the live load deflection performance of the bridges is as varied as the levels of wearing surface deterioration. Overall, calculated n-values for the five bridges ranged from approximately 500 to nearly 2000 and differential panel deflections ranged from negligible to just over 0.2 in. The large variance in the n-values from one load case to another for an individual bridge is often attributed to the shift of the load truck from near the longitudinal centerline of the bridge to near the curb. Placement of the load toward the centerline of the bridge allows for load to be distributed to a greater number of girders than does placement of the load truck near the curbs, resulting in smaller deflections and larger n-values. The recommended limit on both panel deflection relative to the girders and differential panel deflection of 0.1 in. presented in [24] is intended to be used in addition to other deflection limits for bridges and a reduction in this limit is suggested when asphalt wearing surfaces are used on the bridge. Several bridges exceeded this recommended limit for differential panel deflection, but were within the limit for panel deflections relative to the girders.

To study the relationship between deterioration severity and other bridge characteristics, a scale was created to rate the deterioration of the wearing surfaces. Bridges with severe transverse cracking at each panel joint, such as the Butler Co. Bridge, were rated as a 2. Bridges with moderate transverse cracking as well as other minor cracking, such as the Chamber Co. Bridge, were rated as a 5. Bridges with minor or no cracking, such as the Lost Creek Bridge, were rated as a 9. These ratings are summarized in Table 3.

The largest differential panel deflections were found to be the result of two factors. First, in the case of the Butler Co. Bridge, the differential panel deflections and significant deterioration of the asphalt were found to be due to the cupping of the deck panels. The cupping of the deck panels results in multiple stress reversals in the wearing surface for each load that passes over the joint. Whereas flat panels typically only experience one stress reversal per load passage. Second, in the case of the Russellville Bridge, transverse cracks are believed to be the result of larger girder spacings. The wider girder spacing may result in larger differential panel deflections over the piers and subsequently transverse cracks over the piers.

Looking at all five bridges with asphalt wearing surfaces, several conclusions may be drawn from the results listed in Table 3. First, the magnitude of the girder deflections appears to be irrelevant, since girder deflections were only found to directly affect the deterioration of the wearing surface on one bridge, the Erfurth Bridge. However, the girder deflections relative to the span length, or the n-values, do provide some useful information. The one structure without transverse cracking in the wearing surface, the Lost Creek Bridge, had n-values near 2000. The n-values for those bridges with transverse cracking were typically lower than 1200. Third, based on the n-values and the differential panel deflections, neither large girder deflection alone nor large differential panel deflections alone appear to be the cause of the cracking seen in the asphalt wearing surfaces. Rather, the combination of large girder deflections with differential panel deflection of generally any magnitude appears to be the controlling factor. However, as mentioned previously, the asphalt mix design and other factors may also be affecting the transverse cracking seen in the tested bridges.

Recommendations

Recommendations for future design of glued-laminated timber girder bridges with transverse glued-laminated timber panel decks and asphalt wearing surfaces are as follows:

- Bridge deflections should be limited by a more strict limit than L/360; L/1500-L/2000 appears more appropriate
- A more strict limit on differential panel deflection should also be applied, possibly 0.05 in. or less, and should be applied in addition to the L/n limit
- Girder spacing should be made to be within the recommended limits
- Insure that the deck panels are initially flat and are attached and protected such that they will remain in the same condition as at the time of installation
- Research is needed to develop inexpensive, construction friendly, and effective methods to reduce differential panel deflections for both newly constructed and existing structures
- Deck systems that incorporate deck panels, longitudinal planks, and asphalt may be an effective means of reducting wearing surface deterioration
- Further investigation into the asphalt mix design may be necessary

Table 3. Glued-laminated girder bridge information.

Report Number	Bridge Name	# of Spans	# of Lanes	Span (s) (ft-in.) (tested spans in bold)	# of Girders	Girder Size (in.)	Girder Spacing (in.)	Panel Width (ft-in.)	Panel Depth (in.)	Curbs (Y/N)	2003 Truck Weight (lbs.)	Wearing Surface Type	Transverse Cracking (Y/N)^	Load Case	Exp. n-value (1995)*	Exp. n-value (2003)*	Maximum Girder Defl. (in.)*	Maximum Differential Panel Defl. (in.)*
1	Badger Creek	1	1	**30-11**	4	8.75x 30	48	4	5	Y	48,500	Long. Planks	N, 9	1	-	1110	-0.28	0.022
														2	-	1178	-0.25	
2	Camp Creek	1	1	**31-1**	4	8.75x 31.5	48	4	5	Y	48,500	Long. Planks	N, 7	1	-	1280	-0.28	-
														2	-	1621	-0.22	
														3	-	2026	-0.16	
3	Lost Creek	3	1	**14-3 / 47 / 14-3**	3	10.75 x49.5	90	4	7	Y	48,500	Asphalt	N, 9	1	-	2032	-0.27	-
														2	-	2032	-0.27	
4	Erfurth	1	2	**40-6**	12	14x 23.5	31	4-4	3.25	Y	67,700	Asphalt	Y, 4	1	-	515	-0.91	0.127
														2	-	535	-0.88	
5	Chambers County	1	2	**51-6**	6	8.75x 43	60	4-1	5	N	65,760	Asphalt	Y, 5	1	-	948	-0.64	0.088 ('95) 0.054 ('03)
														2	-	704	-0.85	
														3	-	662	-0.91	
														4	816	808	-0.74	
														5	984	968	-0.62	
6	Russellville	4	2	**41-9** / 41-9 / 41-9 / 41-9	5	6.75x 41.5	60	4-1	5	N	64,060	Asphalt	Y, 5	1	-	1000	-0.47	0.22 ('95) 0.034 ('03)
														2	-	745	-0.64	
														3	-	738	-0.64	
														4	1081	1103	-0.43	
														5	929	-	-0.52	
														6	1303	-	-0.37	
7	Wittson	4	1	**50**/50/ **102**/30	4	6.75x 43; 6.75x 63.25	51	4	5	N	67,900	Asphalt	Y, 6	1 (span 1)	-	604	-0.95	0.117 (span1 '95) 0.027 (span1 '03) 0.001 (span3 '03)
														2 (span 1)	1036	996	-0.58	
														3 (span 3)	938	-	-1.28	
8	Butler County	2	2	**24** / 60	5	5x 27.5	60	4	5	N	61,300	Asphalt	Y, 2	1	-	796	-0.34	0.176
														2	-	561	-0.48	
														3	-	557	-0.49	

*Values have been adjusted by total truck weight to the design truck for that bridge (HS20 or HS25).

^Deterioration of the wearing surface was rated on a scale from 1 – 9:

1 – severe deterioration of the entire wearing surface

5 – moderate deterioration of the wearing surface

9 – minor deterioration of the wearing surface

Longitudinal Glued-Laminated Timber Panel Bridges

Background

Another common type of glued-laminated timber bridge is the longitudinal panel bridge, which consists of glued-laminated panels spanning longitudinally from support to support as the main load resisting members. Transverse stiffener beams are attached to the underside of the panels to distribute loads from panel to panel and to minimize differential deflections. These structures are typically one or two lanes and are common for spans up to approximately 35 ft [24].

To investigate the source of the deterioration commonly seen in the wearing surfaces of these types of bridges, four bridges that met the needs of this project were located, inspected, and field-tested. Data collected from these visual inspections and field tests were analyzed and presented previously in four individual reports titled *Live Load Deflection of Timber Bridges* ([31]-[42]). The results from these reports are the basis for this section of this report.

All four of the bridges had asphalt wearing surfaces, which varied in depth from 2.5 to 6 in., timber guardrails, and three of the four had timber curbs. Three of the four bridges were founded on sheet pile abutments and the fourth on concrete abutments.

Results

The deflection performance of each bridge and the condition of its wearing surface related to that performance will be discussed in short individually followed by a discussion of the overall performance of the four bridges and the effects of live load deflection on wearing surface condition. Deflection checks for timber deck bridges are evaluated similar to girder bridges and are based on deflection criteria typically of the form L/n, L being clear span in inches. Listed in Table 1 are the deflection criteria found in [1], [2], and [24], the current design specifications and guide manual. In addition, [24] also suggests limiting differential panel deflection to 0.1 in. A further reduction in this limit is suggested in the presence of an asphalt wearing surface. The performance of the subject bridges will frequently be compared to these criteria in the subsequent discussion.

For comparison, the value of n using the maximum measured deflections from all load cases investigated on these four bridges are listed in Table 4. Since the deflection criteria found in the specifications are based on the design truck, the experimental n-values were normalized, by total truck weight, to the design truck used for that specific bridge.

Table 4. Experimental n-values, panel bridges.

Report Number	Bridge Name	Load Case	Exp. n-value (1996)	Exp. n-value (2003)
9	East Main St.	1	913	895
		2	788	808
		3	870	919
		4	748	778
10	Angelica Creek	1	390	417
		2	334	324
		3	383	386
		4	292	278
11	Bolivar	1	589	656
		2	589	670
		3	614	635
		4	572	620
12	Scio	1	342	412
		2	385	441
		3	360	408
		4	353	391

The East Main Street Bridge [39], located in Angelica, NY is a two lane structure spanning 30 ft – 6 in. and consists of eight 14.25 in. x 4 ft – 5 in. glued-laminated panels. Large glued-laminated curb sections are located on both sides of the bridge and double as guardrail. Figures 12 and 13 show the condition of the wearing surface in 2000 and 2003, respectively. Several full and partial length longitudinal cracks were found in the asphalt wearing surface at the time of testing in 2003 and photographs from previous inspections revealed similar levels of deterioration in the years prior to testing.

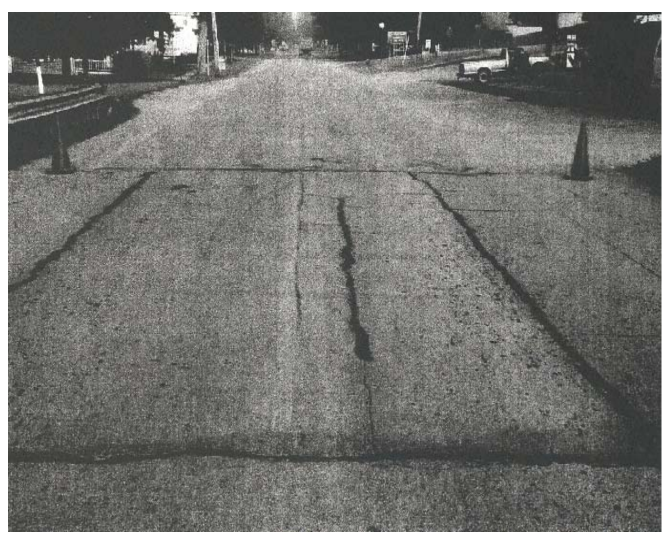

Figure 12. East Main St. Bridge wearing surface in 2000.

Figure 13. East Main St. Bridge wearing surface in 2003.

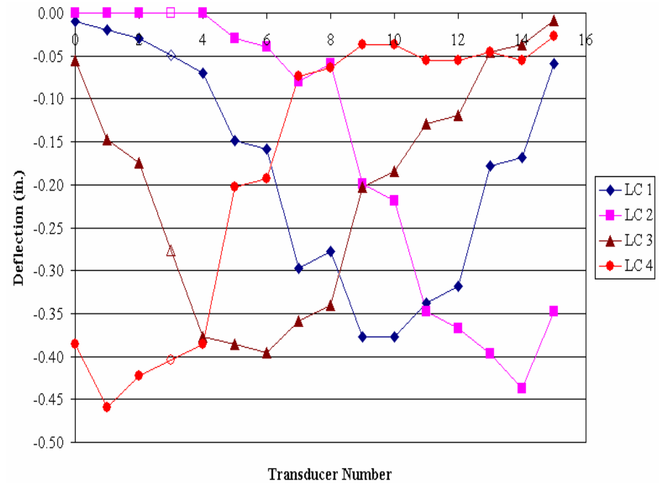

Figure 14. Maximum midspan panel deflections, East Main St. Bridge.

For both the 1996 and 2003 tests, the overall deflection performance of the bridge was found to be within acceptable limits. The deflections were relatively similar for both load tests, indicating that, structurally the bridge has incurred little deterioration. Maximum panel deflection measured in 2003 was –0.46 in.; this deflection corresponds to an n-value of 748, which is approximately twice the deflection limit. In addition, the distribution factors approximated using the measured deflections were found to be less (i.e., better lateral load distribution characteristics) than the wheel load factors (WLF) specified in [24] for the design of longitudinal deck panels. The small measured deflections and distribution factors suggest that the bridge is stiffer than assumed in design.

Slight variances in the deflection performance were evident when the load truck was shifted from the centerline position to the near curb. Larger overall deflections were measured when the truck was near the curb than when near the centerline. This is believed to be the result of the large curb sections and variances in load distribution. It is evident from Fig. 14 that transverse load distribution is more effective when the load is near centerline. In addition, Fig. 14 also indicates that the curbs tend to decrease the deflection of the exterior side of the exterior panels when the load truck is near centerline. The curbs appear to act as beams/girders located at the edges of the deck. Without the large curbs, the deflection of the exterior panels would be increased for the load cases involving the load truck near the curb.

Differential panel deflections were typically found to be less than 0.04 in. and were the greatest adjacent to the load truck. Compared to the 0.1 in. limit on differential panel deflection recommended by [24] the calculated differential deflections appear to be adequate. However, based on the condition of the asphalt wearing surface and the presence of the longitudinal cracks above the panel joints in 2000 and 2003, the differential deflections appear to be contributing to the deterioration of the wearing surface.

As discussed previously, load distribution via the transverse stiffener beams, although affected by the curbs, was found to be better than typically assumed in designed. In addition, the effectiveness of the stiffener beams to reduce differential panel deflections was investigated. Figure 15 illustrates the differential deflection along one panel joint both adjacent to and midway between two stiffener beams. Lines with open data points represent differential deflections midway between two stiffener beams; lines with solid data points represent differential deflections adjacent to the stiffener beams. The differential panel deflections shown in Fig. 15 indicate that the stiffener beams have little to no effect on reducing the differential panel deflections.

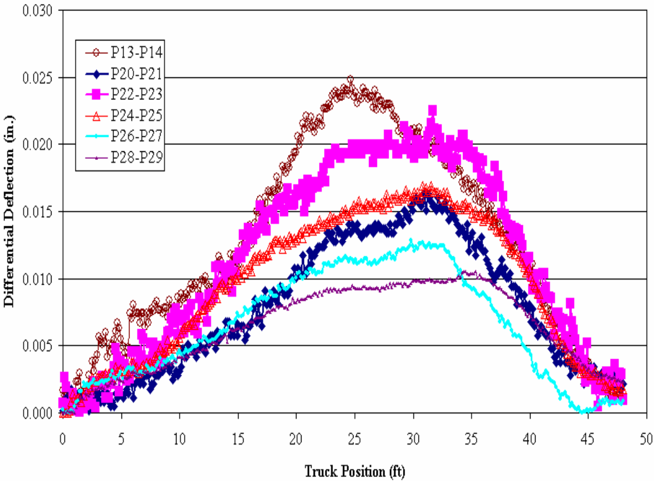

Figure 15. Differential panel deflections along joint J7, East Main St. Bridge.

In reference to the criteria in the current design specifications regarding global and relative panel deflection, the performance of the East Main St. Bridge was found to be within limits. However, based on the condition of the asphalt wearing surface, differential panel deflections do appear to be a factor contributing to the repeated and consistent deterioration of the asphalt wearing surface.

The Angelica Creek Bridge [40] is located just outside the town of Angelica, NY. This two lane bridge spans 21 ft – 4 in. and consists of seven 8.75 in. x 4 ft – 2 in. glued-laminated panels, timber guardrails, and has no curbs. At the time of testing in 2003, no cracking was evident in the wearing surface because a new layer of asphalt had been placed over the bridge shortly before testing. However, from photographs in previous inspection reports, several full and partial length longitudinal cracks were evident in the asphalt wearing surface. Figures 16 and 17 show the condition of the wearing surface in 1998 and 2003, respectively.

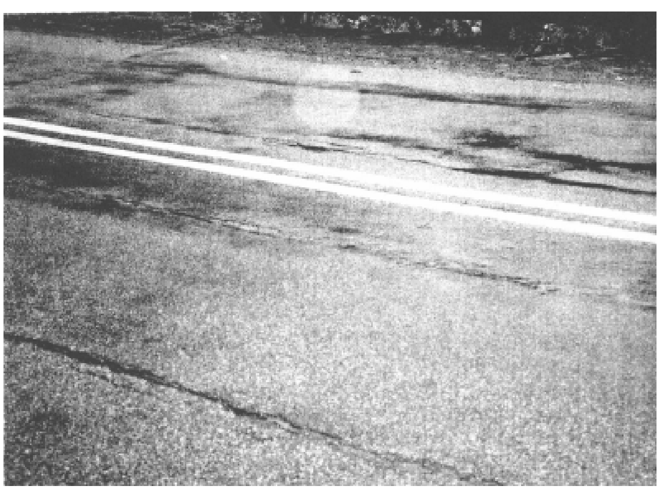

Figure 16. Angelica Creek Bridge wearing surface in 1998.

Figure 17. Angelica Creek wearing surface in 2003.

Cupping of the exterior panels (concave downward) was evident, but is not believed to be severely affecting the condition of the asphalt wearing surface since the level of cracking over the panel joints was consistent along the entire width of the bridge deck. The cupping of the exterior panels is believed to be the result of the expansion of the panels due to increases in moisture content that is restricted by the connection of the panels and stiffener beams to the base of the guardrail posts. This expansion and cupping of the exterior panels has also resulted in the rotation of the guardrail posts.

For both the 1996 and 2003 tests, the overall performance of the bridge varied compared to the specified deflection criteria. From Table 4 it is evident that the deflection of the bridge exceeds the acceptable limit of L/360 for load cases where the load truck is positioned near the curb. Figure 18 shows the distribution factors calculated for each load case in 2003, which are slightly less than the wheel load factors (WLF) specified in [24] for design of longitudinal deck panels. Figure 18 suggests that transverse load distribution, as well as other factors, may be affecting the deflection performance of the bridge.

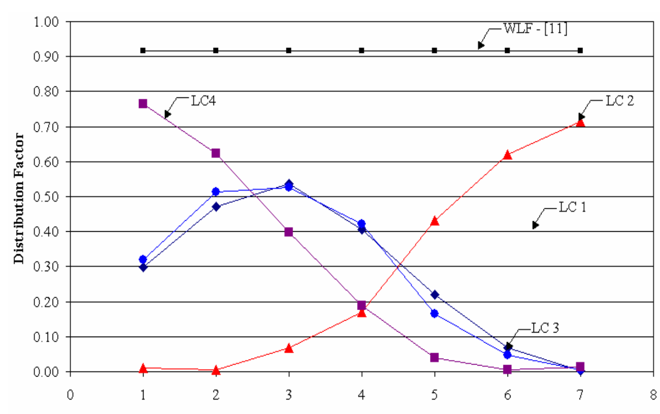

Figure 18. Experimental and codified distribution factors, Angelica Creek Bridge.

Despite the varied performance compared to the deflection criteria, little change was evident from 1996 to 2003 in the deflection data indicating that the performance of the bridge over the years has remained relatively consistent. Maximum deflections for load cases 1 – 4 in both 1996 and 2003 were approximately –0.6 in., -0.7 in., -0.6 in., and –0.8 in., respectively. Differential panel deflections in both 1996 and 2003 were less than 0.07 in. and were the greatest adjacent to the load truck. Compared to the 0.1 in. limit on differential panel deflection, the calculated differential deflections seem to be adequate. However, the significance of the differential deflections is more evident in the condition of the asphalt wearing surface and the presence of the longitudinal cracks above the panel joints in 1998. The significant longitudinal cracks located at each panel joint suggest that these differential panel deflections are at least one factor affecting the deterioration of the wearing surface.

In addition to the effectiveness of the stiffener beams to distribute loads, the effectiveness of the stiffener beams to reduce differential panel deflections was also investigated. Differential panel deflections calculated both adjacent to and midway between two stiffener beams indicate that the stiffener beams have little to no effect on the differential panel deflections.

Overall, the deflection performance of the bridge is dependant on the load position and is within limits when loaded within the normal traffic lanes. In addition, differential panel deflections appear to be one factor affecting the condition of the asphalt wearing surface. However, other factors may also be contributing to this deterioration.

The Bolivar Bridge [41], located near Bolivar, NY is a two-lane bridge spanning 28 ft – 8 in., is 26 ft – 1 in. wide, and consists of six 15 in. x 4 ft – 4 in. glued-laminated deck panels, large timber curbs, and metal guardrails. Several full and partial length longitudinal cracks were found in the asphalt wearing surface at the time of testing, and photographs from previous inspections revealed similar levels of deterioration in the years prior to testing. Figures 19 and 20 show the relatively poor condition of the wearing surface in 2000 and 2003, respectively. These cracks were found to be located directly above the panel joints, and there are locations of random transverse and longitudinal cracking in the wearing surface.

In addition to the wearing surface, the deck panels also showed signs of deterioration. Longitudinal splits were found in the underside of the deck panels at several locations. The most significant cracks were located between the end of the transverse stiffener beams and the edge of the deck. Additionally, expansion of the panels, which is believed to be due to increases in moisture content, has resulted in crushing of the exterior of the panels at the abutments against the sheet pile abutment walls. This expansion has also resulted in the shifting of the panels and subsequently, the rotation of the connectors that attach the panels to the abutment cap. In addition, significant gaps were evident between adjacent deck panels as well as between the panels and the transverse stiffener beams.

Figure 19. Bolivar Bridge wearing surface in 2000.

Figure 20. Bolivar Bridge wearing surface in 2003.

In terms of global deflection, the performance of the Bolivar Bridge in both 1996 and 2003 was within specified limits, as is evident from Table 4. Maximum panel deflection in 1996 was approximately –0.56 in. and in 2003 was approximately –0.52 in. The general shape of the deflection diagrams for both years suggest that load distribution via the transverse stiffener beams is adequate compared to the wheel load factors (WLF) specified in [24] for the design of longitudinal deck panels. Distribution factors calculated from the measured deflections were typically less than the codified WLF's.

Figure 21 illustrates the maximum midspan deflection of the panels in 2003. A significant decrease in deflection is evident between the transducer on the interior edge of the exterior panel and the transducer on the exterior edge of that same panel. The large curb sections located along the exterior edge of the exterior panels are likely the source of this decrease in deflection. The large curbs act as girders adding stiffness to the exterior of the bridge. In addition, the fact that the transverse stiffeners terminate at the middle of the exterior panels may reduce load distribution to the exterior panels, thereby reducing deflections.

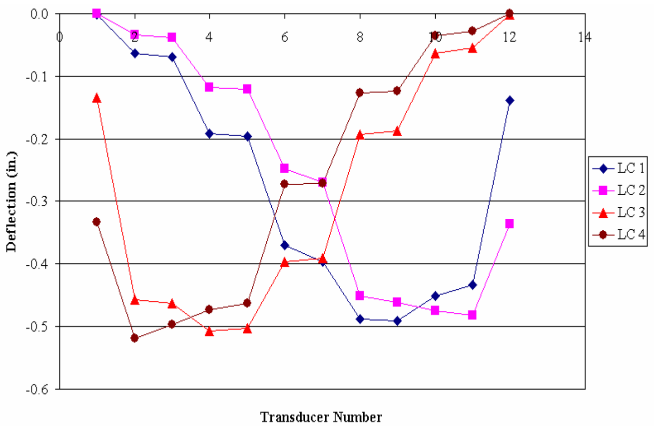

Figure 21. Maximum midspan panel deflections in 2003, Bolivar Bridge.

In addition to global panel deflection, differential panel deflections are equally important. Differential panel deflections in 1996 and 2003 were typically found to be less than 0.05 in. and 0.03 in., respectively, and were the greatest adjacent to the load truck. Compared to the 0.1 in. limit on differential panel deflection recommended by [24] the calculated differential deflections seem to be adequate. However, the significance of the differential deflections may be more evident from the condition of the asphalt wearing surface and the presence of the longitudinal cracks above the panel joints. The calculated differential deflections may be compounded by the behavior of the exterior panels and the curbs. Because of the curbs, the interior panels are in effect more flexible than the exterior panels resulting in larger differential panel deflections than might be expected for a similar structure without curbs. In addition, the gap between the panels and the stiffener beams may cause increases in the differential panel deflections, and consequently the deterioration of the wearing surface.

As discussed previously, load distribution via the transverse stiffener beams was found to be better than typically assumed in design. The effectiveness of the stiffener beams to reduce differential panel deflections, however, was found to be minimal.

In brief, the performance of the Bolivar Bridge under live loading is within current deflection limits. However, the combination of live load deflections, termination of the stiffener beams prior to the edge of the bridge, and the presence of the relatively large curbs on the deck appear to be affecting the condition of the asphalt wearing surface. Differential panel deflections induced by live loads and compounded by the presence of the curbs appear to be the source of the longitudinal cracks in the wearing surface. In addition, other factors may be contributing to this deterioration.

The last deck bridge tested in New York was the Scio Bridge [42], which is located near Scio, NY. This two lane bridge spans 20 ft – 8 in., is 31 ft – 2 in. wide, and consists of seven 9 in. x 4 ft – 4 in. glued-laminated timber panels. The bridge has large timber curbs, which also double as the guardrail, and an asphalt wearing surface.

There were only a few locations of deterioration evident in the asphalt wearing surface on the Scio Bridge. One such location is shown in Fig. 22. Evident in Fig. 22 is a longitudinal crack along the majority of the span located near the outside wheel line of the eastbound lane over a panel joint. Similar deterioration was also found in the westbound lane near the white line marking the lane boundary. In addition, transverse cracks were found at each abutment and a pothole approximately 6 in. in diameter was found near the northeast corner of the westbound lane. This structure is classified as a culvert due to its short span length, therefore, no inspection reports were available that document the condition of the wearing surface in previous years.

Figure 22. Scio Bridge wearing surface in 2003.

The deflection performance of the Scio Bridge in both 1996 and 2003 in terms of global panel deflection varied depending on load position, as is evident from Table 4. Maximum panel deflection in 1996 was approximately –0.68 in. and in 2003 was approximately –0.58 in. In 1996, the bridge failed to meet or narrowly met the design criteria for deflection for all load cases investigated. However, the deflection performance of the bridge improved in 2003 and was within specified limits for all load cases. This improvement in live load deflection performance is not uncommon and may be the result of changing support conditions, changes in moisture content, as well as other factors. The general shape of the deflection diagrams for both years suggest that load distribution via the transverse stiffener beams is adequate compared to the wheel load factors (WLF) specified in [24] for design of longitudinal deck panels. In addition, distribution factors calculated from the measured deflections were typically less than the codified WLF's.

A significant decrease in deflection was evident between the transducer on the interior edge of the exterior panel and the transducer on the exterior edge of that same panel as shown in Fig. 23. This decrease is likely caused by the large curb sections located along the exterior edge of the exterior panels. The large curbs act as girders adding stiffness to the exterior of the bridge.

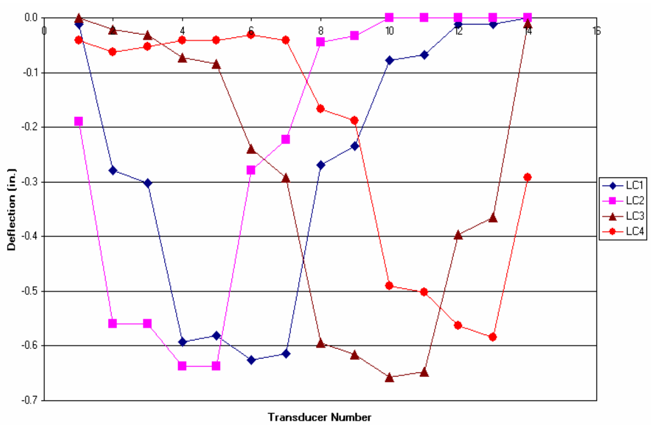

Figure 23. Maximum midspan panel deflections, Scio Bridge.

In addition to global panel deflection, differential panel deflections are equally important. Differential panel deflections in 1996 and 2003 were typically found to be less than 0.03 in. and 0.04 in., respectively, and were the greatest adjacent to the load truck. Compared to the 0.1 in. limit on differential panel deflection recommended by [24] the calculated differential deflections seem to be adequate. However, the presence of longitudinal cracks in the wearing surface suggests that the differential deflections may be significant. The calculated differential deflections may be compounded by the behavior of the exterior panels and the curbs. Because of the curbs, the interior panels are in effect more flexible than the exterior panels which results in larger differential panel deflections between the interior and exterior panels.

Load distribution via the transverse stiffener beams was found to be more adequate than typically assumed in designed as discussed previously. In addition, the effectiveness of the stiffener beams to reduce differential panel deflections was also investigated. Differential panel deflections calculated both adjacent to and midway between two stiffener beams indicated that the stiffener beams have little to no effect on the differential deflections.

In brief, the performance of the Scio Bridge under live loading has improved from 1996 to 2003. Failure to meet the recommended deflection criteria may be the result of the relatively thin 9 in. deck. However, the longitudinal cracking in the asphalt wearing surface was minimal, and the magnitude of the differential panel deflections were small compared to current recommended limits. The longitudinal cracks evident in the wearing surface may be the result of live load deflections, both global and differential, compounded by the additional stiffness along the edge of the bridge due to the curbs. In addition, the presence of potholes on the bridge deck suggests that other factors, such as asphalt mix design, may be contributing to the deterioration of the asphalt wearing surface.

Discussion

The structural performance of the four glued-laminated timber panel bridges tested for this project, in terms of global deflection, varied for each bridge as well as each load case. This is evident by the n-values listed in Table 4 and again in Table 5, which also lists key physical attributes of each bridge and maximum panel and differential panel deflections from the 1996 and 2003 load tests. The large difference between the recommended deflection criteria and that obtained from the experimental n-values may be attributed to several factors. The panels may have been initially over designed to reduce deflections or the deflection limit state may not have controlled. Transverse load distribution from panel to panel via the transverse stiffener beams may be better than typically assumed in design. Changes in moisture content, support conditions, and other factors may also result in smaller deflections than those predicted in design. In addition, it is believed that the presence of large timber curb sections provides significant additional stiffness to the bridge thereby reducing deflections and subsequently the calculated n-values.

Inspection of the bridges both before and after testing found signs of at least some level of deterioration in all components of the bridges. Examples of the types of deterioration found include: cupping of the deck panels, splitting of longitudinal deck panels, increases in moisture content of the deck panels, cracking and potholes in the asphalt, gaps between the deck panels, and localized crushing of the stiffener beams. From the collected data, live load deflection is believed to be partially responsible for the deterioration found in the wearing surfaces on these bridges, and possibly indirectly responsible for the other modes of deterioration evident.

Two of the four bridges were approximately 30 ft in length and the other two were approximately 20 ft in length. The two longer bridges had a difference in length of only 2 ft, but had similar sized longitudinal panels and large timber curbs. Although there were only subtle differences between these two bridges physically, a large difference in deflection performance is evident in Table 5. The maximum panel deflection for the East Main St. Bridge was approximately –0.45 in., which is approximately the minimum measured panel deflection for the Bolivar Bridge. However, the differential panel deflections for the two bridges were relatively similar for all but one load case.

Possible explanations for this variance in deflection performance are differences in the number of deck panels, the level of transverse load distribution, and slight differences in span length, as well as other factors. Figures 24 and 25 illustrate the maximum midspan panel deflection for the East Main St. and Bolivar bridges, respectively. Transducers were only installed on the outer edge of one exterior panel, therefore, the first transducer in Fig. 24 corresponds to the inside edge of the exterior panel. From these figures it is evident that load is not being distributed to the exterior two panels on the East Main St. Bridge when the load is near the opposite curb. This indicates that the six panels nearest the load truck resist the majority of the load for that particular load case; whereas for the Bolivar Bridge, the load appears to be distributed to all six panels. The deflection behavior just discussed indicates that the degree of load distribution is relatively similar for both bridges. For load cases where the load truck is near the centerline of the bridge, load distribution and the number of panels appears to be more of a factor.

For a given panel depth and width, classic beam theory would suggest that the longer span of the East Main St. Bridge would produce larger deflections. However, based on the measured deflections in Figs. 24 and 25, the span length does not appear to have a negative effect on the magnitude of the panel deflections. Other possible sources of the variance in deflection performance include: support conditions, changes in the material properties of the glued-laminated timber panels, as well as other factors. Regardless of the difference in the level of deflection performance, the level of deterioration evident in the wearing surfaces on these two bridges was comparable, as shown in Figs. 26 and 27.

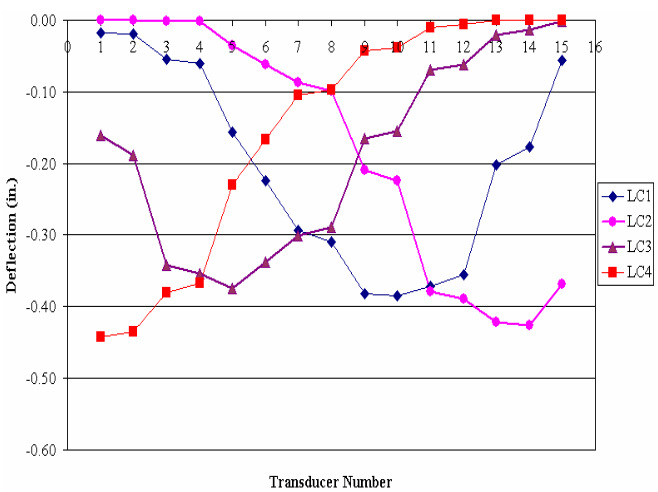

Figure 24. Maximum midspan deflection, East Main St. Bridge.

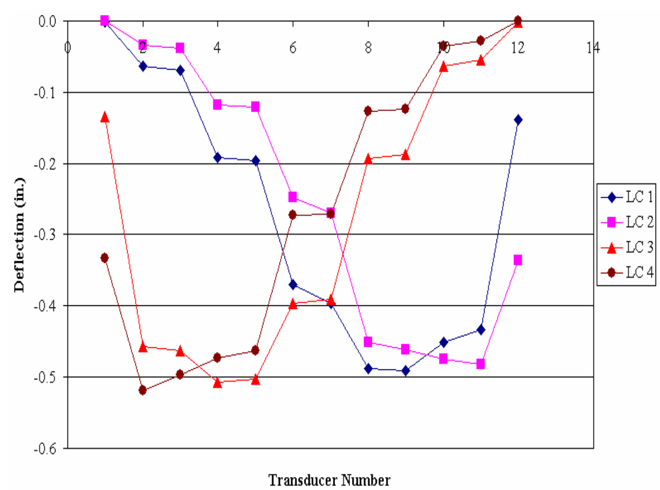

Figure 25. Maximum midspan deflection, Bolivar Bridge.

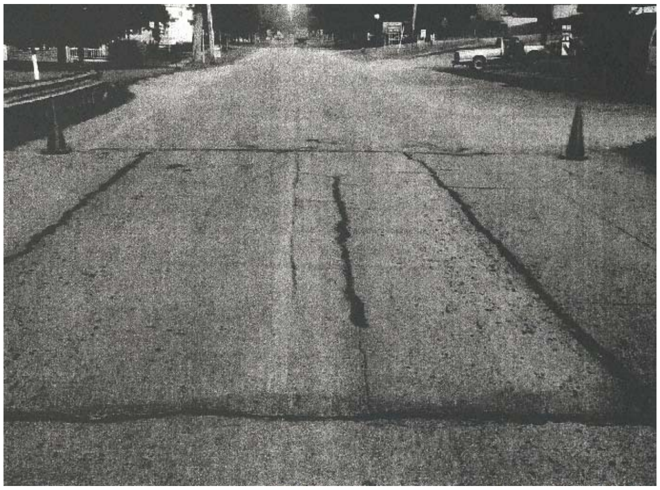

Figure 26. East Main Street Bridge wearing surface in 2000.

Figure 27. Bolivar Bridge wearing surface in 2000.

Comparing the two shorter bridges and their performance under live load, both similarities and differences are evident from Table 5. The similarities between the Angelica Creek and Scio bridges are with the span length, depth of panels, and the general differential panel deflections. The differences between the two bridges include: the presence of curbs on the Scio Bridge, the magnitude of the global panel deflections, the n-values, and the deterioration of the wearing surface.

One obvious factor affecting the deflection performance of these two bridges is evident in Figs. 28 and 29, which illustrate the maximum midspan panel deflection for the Angelica Creek and Scio bridges, respectively. These two figures indicate that the presence of the large curbs has a significant effect on the deflection of the bridge when the load truck is near the curbs. A similar reduction in deflection is evident at the exterior of the bridge when the load truck is near the longitudinal centerline. Thus, the presence of the large curb sections on the Scio Bridge is at least one factor affecting the deflection performance of this bridge.

The similarity in the differential panel deflections and deflection performance (n-values) of these two bridges along with the difference in the deterioration of the asphalt wearing surfaces suggest that differential panel deflections are not solely to blame for the longitudinal cracking evident in 1996 and 2003.

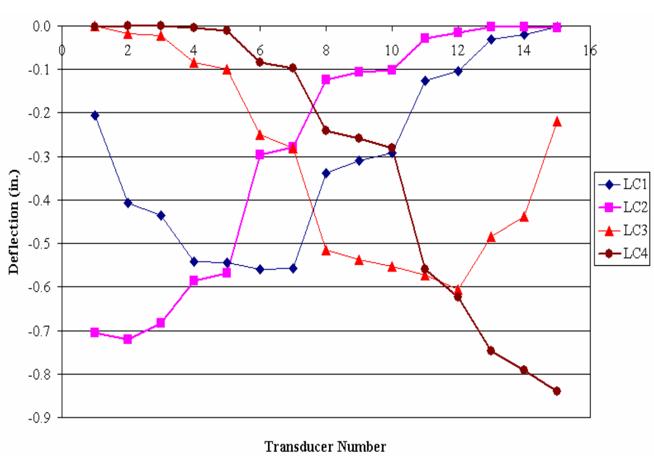

Figure 28. Maximum midspan deflection, Angelica Creek Bridge.

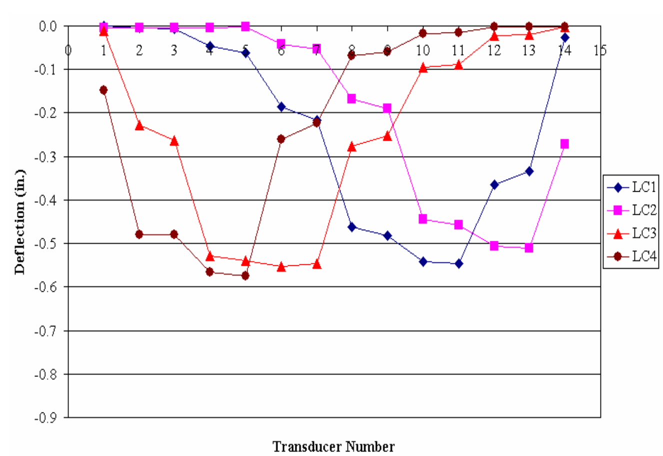

Figure 29. Maximum midspan deflection, Scio Bridge.

The following conclusions are drawn from the information listed in Table 5. Length of span does not appear to have an effect on the amount of deterioration sustained by the wearing surface. The longer bridges with typically deeper deck panels were within current deflection limits, while the performance of the shorter bridges with shallower deck panels varied from within limits to exceeding the limits. Regardless of length, the presence of large glued-laminated timber curbs has the effect of stiffening the exterior panels on the bridge, thereby affecting the global deflection of the bridge. This may result in larger differential panel deflections near the exterior of the bridge, but does not appear to be significantly affecting the condition of the wearing surface, as is evident from the Scio Bridge.

For the first three bridges listed in Table 5, which have relatively the same level of deterioration in the wearing surface, a broad spectrum of deflections are evident. Global panel deflections range from as small as –0.38 in. to as large as nearly –0.8 in. Thus, the magnitude of the global panel deflections themselves does not appear to be contributing to the deterioration of the wearing surface. In addition, these deflections relative to the span, or the n-values, also appear to have no correlation to the level of deterioration sustained by the wearing surface. This is evident by the low n-values and average rating for the wearing surface on the Angelica Creek Bridge and the high n-values and similarly average rating for the wearing surface on the East Main St. Bridge.

Not one of these factors alone is believed to result in the deterioration of the asphalt wearing surfaces on these bridges. The locations of the longitudinal cracks do however indicate that the differential panel deflections may be a significant factor. However, it is the repeated change in stress generated by these differential deflections, and not the magnitude of the differential deflections, that is believed to produce these cracks. This conclusion is based on the fact that longitudinal cracking was evident in previous years in the wearing surface of the Angelica Creek Bridge, yet no cracks were evident in the newly placed asphalt wearing surface at the time of testing in 2003. The global deflection of the panels possibly compounds this effect or causes deterioration to the structure other than in its wearing surface, but is not directly involved in the deterioration of the asphalt wearing surfaces.

In addition, the transverse stiffener beams appear to be an ineffective means of reducing differential panel deflection. However, they do adequately distribute loads laterally from panel to panel across the width of the bridge.

Recommendations

Recommendations for future design of glued-laminated timber deck bridges with asphalt wearing surfaces are as follows:

- The deflection limit of L/360 appears to be adequate for longitudinal glued-laminated timber panel bridges, but requires attention when using deck panels of approximately 9 in. depth
- Research is needed to develop more effective methods to reduce differential panel deflections and distribute loads transversely from panel to panel to be applied to both newly constructed and existing structures
- Insure that the deck panels are initially flat and that they are attached and protected such that they will remain in the same condition as at the time of installation
- Further investigation into the asphalt mix design may be necessary

Table 5. Glued-laminated deck bridge information.

Report Number	Bridge Name	# of Spans	# of Lanes	Span (ft-in.)	# of Panels	Panel Width (ft-in.)	Panel Depth (in.)	Curbs (Y/N)	2003 Truck Weight (lbs.)	Wearing Surface Type	Longitudinal Cracking (Y/N)^	Load Case	Exp. n-value (1996)*	Exp. n-value (2003)*	Maximum Panel Defl. (in.)*	Maximum Differential Panel Defl. (in.)*
9	East Main St.	1	2	30 - 6	8	4 - 5	14	Y	69,820	Asphalt	Y, 4	1	913	895	0.382	0.073
												2	788	808	0.426	0.025
												3	870	919	0.375	0.029
												4	748	778	0.442	0.013
10	Angelica Creek	1	2	21 - 3	7	4 - 2	9	N	69,820	Asphalt	Y, 4	1	390	417	0.561	0.032
												2	334	324	0.721	0.038
												3	383	386	0.606	0.041
												4	292	278	0.791	0.07
11	Bolivar	1	2	28 - 8	6	4 - 4	15	Y	69,820	Asphalt	Y, 4	1	589	656	0.491	0.029
												2	589	670	0.481	0.024
												3	614	635	0.507	0.011
												4	572	620	0.519	0.022
12	Scio	1	2	20 - 8	6	4 - 4	9	Y	69,820	Asphalt	Y, 7	1	342	412	0.546	0.033
												2	385	441	0.51	0.023
												3	360	408	0.552	0.037
												4	353	391	0.576	0.038

*Values have been adjusted by total truck weight to the design truck for that bridge (typically HS20).
^Deterioration of wearing surface rated on scale from 1 – 9

1 – severe deterioration of the entire wearing surface
5 – moderate deterioration of the wearing surface
9 – minor deterioration of the wearing surface

Conclusions

This research involved the inspection and testing of glued-laminated timber bridges to investigate the correlation between live load deflection and bridge condition, in particular the asphalt wearing surface condition. The majority of this research was conducted and presented in 12 reports entitled *Live Load Deflection of Timber Bridges* ([31]-[42]). These reports document the inspection and live load testing of eight glued-laminated timber girder bridges and four longitudinal glued-laminated deck bridges. The wearing surface condition and performance under live loading of all 12 bridges were combined and analyzed in this report to investigate the effectiveness of the current deflection criteria and to develop new criteria and design recommendations based on the results of the tests.

The common two wearing surfaces on the tested bridges were longitudinal timber planks and asphalt. Only two of the bridges had longitudinal plank wearing surfaces and the performance of these two bridges was well within the recommended limits. The timber planks are believed to be the source of some additional stiffness resulting in the small deflections. In addition, the planks were typically in excellent condition and not believed to be affected by live load deflections. Some deterioration was evident in the planks, but the nature of the deterioration suggests that factors other than live load deflection are the source.

The remainder of the bridges had asphalt wearing surfaces with varying levels of deterioration. Deterioration of the asphalt wearing surfaces was typically in the form of cracks along the panel joints (transverse on the girder bridges, longitudinal on the deck bridges) as well as potholes and miscellaneous longitudinal and transverse cracking. The level of cracking also varied. Some of the cracks above the panel joints were newly formed hairline cracks or hairline cracks that had not been spread due to further deterioration. Other cracks were nearly 2 in. in width with signs of raveling and exposure of the deck panels underneath.

Individually, the performance of the bridges varied compared to the current deflection criteria. All of the girder bridges were well within the recommended deflection limit of L/360, with n-values in the range of 500-2000. However, several of the decks on these bridges exceeded the recommended differential panel deflection limit of 0.1 in. In contrast, compared to the deflection limit of L/360, the deck bridges varied from consistently exceeding the limit, to consistently being within the limit from test to test, and all of the deck bridges satisfied the limit on differential panel deflection. Little correlation was found between the magnitude of the bridge deflections and the deterioration of the asphalt wearing surface. For instance, some girder bridges had minor cracking and differential panel deflections greater than the suggested limit, while others had much more significant cracking of the wearing surface with differential deflections within the recommended limit. Therefore, the repeated cycling, not the magnitude, of the differential deflections and resulting changes in stress in the asphalt layer are believed to be one of the main sources of wearing surface deterioration.

However, neither global nor differential panel deflections alone are believed to be the source of the deterioration of the wearing surface. Instead, from the test results it appears transverse cracking along the panel joints on timber girder bridges is more severe in the presence of both large girder deflections and generally any magnitude of differential panel deflection. Cupping of the deck panels, concave upward, significantly increases the effects of the differential panel deflections on the wearing surface. In addition, differential girder deflections appear to have resulted in longitudinal cracks in some of the bridges, but there does not appear to be a definitive threshold were the magnitude of the differential girder deflections is or is not significant. For the deck bridges, the combination of global panel deflections, differential panel deflections, and the presence of large timber curbs appear to be the most significant factor affecting the condition of the wearing surfaces. The large timber curbs behaved like beam/girders at the edges of the bridge and appeared to have an effect on transverse load distribution but no obvious effect on the magnitude of the differential panel deflections. The transverse stiffener beams were found to have a similar effect. Additionally, bridges without timber curbs were found to have larger panel deflections than bridges of similar geometry that had timber curbs.

Data collected from the 12 field tests and inspection reports from both past and present inspections indicate that the deterioration of the asphalt wearing surface appears to occur over time. Differential panel deflection does not appear severe enough to immediately affect the wearing surface. Rather, over time, the repeated change in stress at the deck level caused by the differential panel deflections is believed to result in the cracking of the asphalt above the panel joints. The effect of the differential panel deflections is compounded by the global deflection of the girders and panels on the girder and deck bridges, respectively.

The following conclusion and recommendations were made from the results of this research.

- A stiffer limit on girder deflection in the range of L/1500 is suggested
- A reduction in the differential panel deflection limit from 0.1 in. to approximately 0.05 in. is recommended
- The limits on girder spacing provided in [24] should be enforced and not exceeded

- Steps should be taken (possibly requiring further research), to reduce and prohibit the cupping of deck panels on these bridges

- A stiffer limit on differential panel deflection for longitudinal panel bridges is suggested; but may require further research involving structures with negligible wearing surface deterioration

- Development of more effective methods, which may be applied to both newly constructed and existing structures, to reduce differential panel deflections and distribute loads transversely from panel to panel is needed

- Deck systems incorporating deck panels, longitudinal planks, and asphalt may be an effective means of reducing deck deterioration

- Further investigation into the asphalt mix design may be necessary

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