

Evaluation of a Timber Bridge for the Secondary Road System Using FRP Reinforced Glued-Laminated Girders

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ABSTRACT

One of the goals that the Innovative Bridge Research and Construction (IRBC) program, sponsored by the Federal Highway Administration (FHWA), wishes to achieve through sponsored research projects is the concept of an economical, durable, short-span bridge that reduces construction time. Delaware County, Iowa took a step in that direction through the construction of a 64 ft. long bridge comprised of fiber-reinforced polymer (FRP) reinforced glued-laminated timber girders and a transverse glued-laminated timber deck. Prior funding for the design, construction (including materials), and monitoring/evaluation of this project was obtained through the IBRC program with the assistance of the Iowa Division of the FHWA. Although strengthening of timber bridges with FRP is not a new topic, limited information exists on the short- and long-term effectiveness of using FRP reinforcement for these types of structures. Thus, the objective of this project was to evaluate the in-service structural performance of a glued-laminated timber bridge strengthened with FRP plates. Field load tests and inspections were performed immediately after construction and each year for two years after construction of the bridge. The purpose was to establish a database of information to address both short- and long-term performance using FRP plates to reinforce glued-laminated timber girders. Structurally, the bridge performed within current specified limits, and no significant increase in stiffness was identified in the girders due to the presence of the FRP plates. Test results collected in 2006 show no noticeable difference compared to those collected two years earlier in 2004, and the FRP/timber bond showed no signs of deterioration. Cost still appears to be the limiting factor for this bridge design, although, as this design gains familiarity, that cost may decrease, making it a more attractive alternative.

Key words: FRP—glued-laminated timber—timber-FRP composite

INTRODUCTION

In recent years, the concept of an economical, durable, short-span bridge which reduces construction time has become more of a necessity to the county engineer than a goal. In addition, county engineers are seeking these same qualities in methods to repair and strengthen existing bridges on the secondary road system. This is just one of the goals that the Innovative Bridge Research and Construction (IRBC) program, sponsored by the Federal Highway Administration (FHWA), wishes to achieve through sponsored research projects.

Delaware County, Iowa is taking a step in that direction through the construction of a 64 ft. long bridge comprised of fiber-reinforced polymer (FRP) reinforced glued-laminated timber girders and a transverse glued-laminated timber deck. The bridge was designed by Matt Smith of Laminated Concepts, Inc. of New York state with technical guidance provided in the design and fabrication of the FRP reinforced girders by the University of Maine. The bridge was fabricated by Alamco Wood Products, Inc. of Albert Lea, MN. Prior funding for the design, construction (including materials), and monitoring/evaluation of this project has been obtained through the IBRC program with the assistance of Curtis Monk, Division Bridge Engineer with the Iowa Division of the FHWA.

This report summarizes the monitoring and evaluation of the structure conducted immediately following the construction of the bridge and throughout the first couple years after construction.

BACKGROUND

Significant research has been conducted in the past by various researchers and engineers concerning the design, construction, implementation, and effectiveness of using timber as a bridge construction material. Timber has been proven to be a viable option for today's transportation structures and, given the advancements in glued-laminated timber products, tomorrow's transportation structures as well.

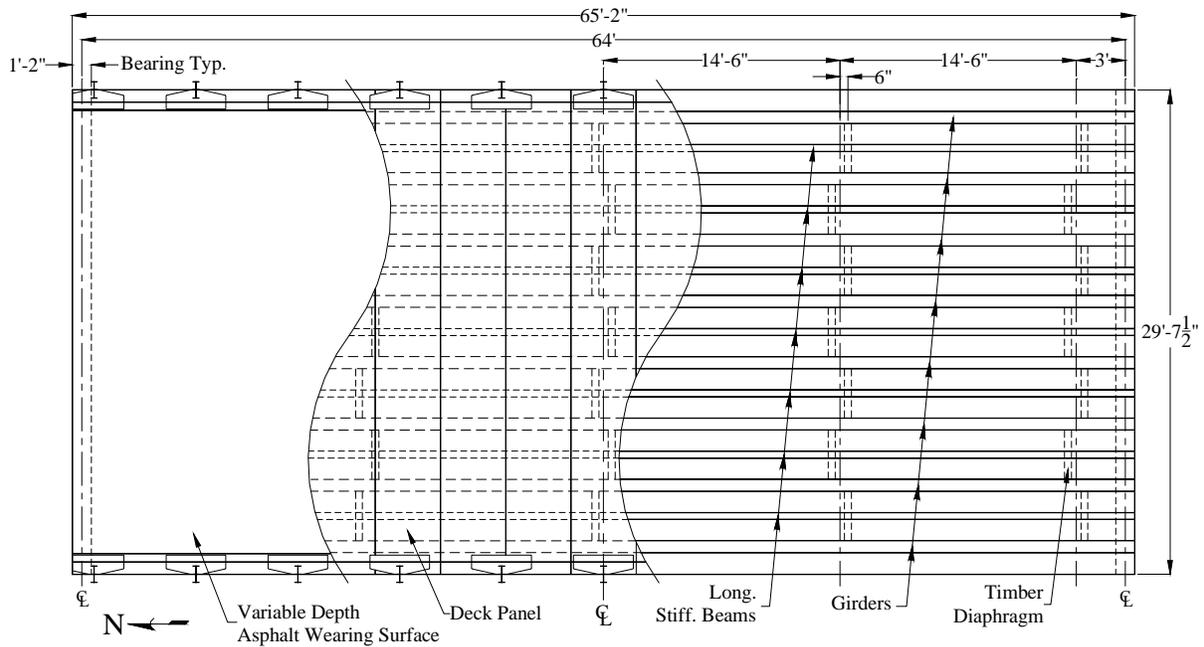
For centuries, engineers have been using timber bridges to span short crossings on our nation's secondary roads. However, changes in the dynamics and demand of traffic, hydraulics, aesthetics, and economics have forced today's engineer to be more creative and to develop methods to carry increased traffic across longer spans in a more efficient and cost-effective manner. In addition, engineers are seeking methods to improve or increase the strength and performance of current structures to handle the aforementioned changes in traffic, hydraulics, economics, and so forth.

The benefits of using FRP materials to strengthen and repair bridges has been realized and accomplished very effectively on various steel and concrete bridges in the past. Although strengthening of timber bridges with FRP is not a new topic, limited information exists on the short- and long-term effectiveness of using FRP reinforcement for these types of structures. Thus, the objective of this project was to evaluate the structural performance of the bridge in service. Additional field load tests and inspections will be performed over approximately the next two years to establish a database of information to address both short- and long-term performance using FRP plates to reinforce glued-laminated timber girders on a 64 ft. simple span bridge.

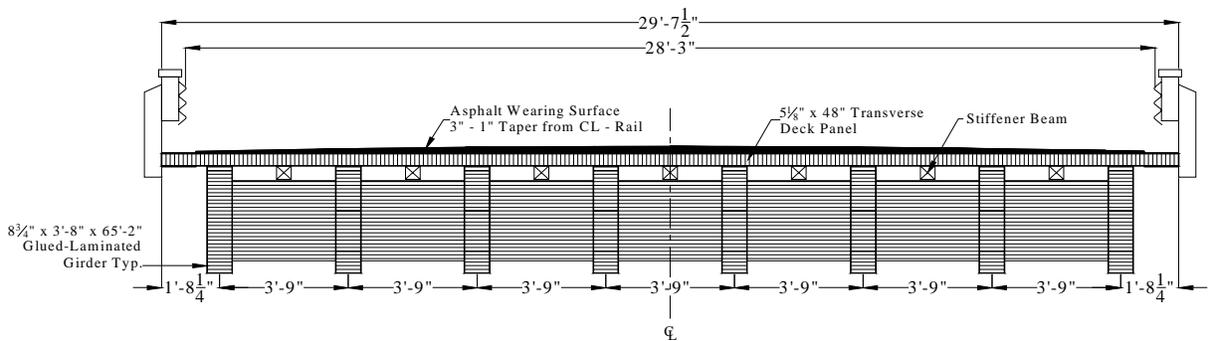
BRIDGE DESCRIPTION

The Delaware Co. Bridge is a two-lane, simple span, longitudinal glued-laminated girder bridge located on a low-volume gravel road spanning Lime Creek east of Ryan, IA. The glued-laminated timber girders and panels were fabricated at Alamco in Albert Lea, MN.

The bridge spans 64 ft., center to center of abutment, is 29 ft. 7.50 in. wide with zero degrees of skew and a roadway width of 28 ft.,3 in., as shown in Fig. 1. The superstructure consists of eight glued-laminated timber girders strengthened with FRP plates and a transverse glued-laminated timber deck. The eight girders measure 65 ft. 2 in. in length and consists of an 8.75 in. by 3 ft. 7.50 in. cross section of glued-laminated timber with a 0.50-in. thick by 8.75 in. wide FRP plate bonded to the tension laminate (see Figure 2). The girders are supported on either end by a 14 in. by 8.75 in. by 0.50 in. neoprene pad on the abutments' 14 in. seats. The supporting substructure consists of timber piles, timber abutment caps, and a timber plank abutment back wall.



a. Plan view



b. Cross section view

Figure 1. Delaware County Bridge

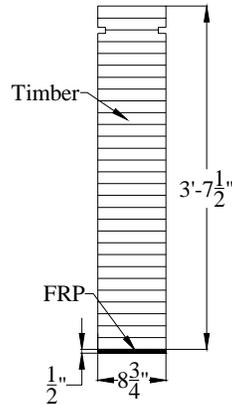


Figure 2. Girder cross section

The exterior girders on either side of the bridge are inset 1 ft. 8.25 in., measured from the outside edge of the deck to the centerline of the girder, and the center-to-center girder spacing is 3 ft. 9 in. Glued-laminated timber diaphragms are located at the abutments, 1/4 span, and midspan and longitudinal timber stiffener beams are bolted to the bottom of the deck midway between each girder.

The glued-laminated timber deck spans transverse to the girders and consists of nominal 5 in. by 48 in. panels measuring 29 ft. 7.50 in. in length. The panels are set against one another and are attached to the girders with metal s-clips bolted to the deck and inset into a groove in the girders. The guardrail is composed of a three-beam rail and steel posts attached to the deck with bolts; a variable depth asphalt wearing surface provides protection for the timber deck panels and also serves as the riding surface. Figure 3 illustrates the wearing surface utilized on this structure. No curbs are present on this bridge.



Figure 3. Delaware County Bridge wearing surface

EVALUATION METHODOLOGY

Girder and deck deflections were recorded at critical locations with the use of an Optim Megadec data acquisition system (DAS), a Dell laptop computer running TCS software for communication with the Megadec, and ratiometric displacement transducers. In addition, strains were recorded at critical locations with the use of the Bridge Diagnostics, Inc. (BDI) Intelliducers and Structural Testing System (STS).

Using the global deflection data collected, differential deflections and lateral load distribution factors were calculated during post-processing of the data. Differential deflections have been known to be a problematic area on these types of bridges, and approximate distribution factors provide some insight to the actual load distribution characteristics of the bridge compared to those assumed in the design.

INSTRUMENTATION

Figures 4 and 5 illustrate the positioning of the displacement and strain transducers on the Delaware Co. Bridge, respectively. Displacement transducers were installed on the underside of all eight girders at mid-span and on the underside of girders G1-G4 at quarter-span. Transducers were installed on the underside of the deck approximately 1 in. from the panel joint on both sides of the joint at multiple locations. The transducers on the girders were used to evaluate the global deflection performance of the structure, while those on the underside of the deck panels were used to determine the localized deflection performance of the deck panels.

Given the variable localized material properties of timber and the orthotropic nature of the material, there is some difficulty accurately measuring strains in the components of a timber bridge. However, with careful planning and gage application, strains in timber members may be measured and approximated to a degree that provides useful information about the behavior of the members and the structure as a whole.

Strain gages were positioned on girders G1 and G4 at mid- and quarter-spans in locations such that the distribution of stress over the girder cross section and the effectiveness of the bond between the glued-laminated timber and the FRP laminate could be evaluated. As shown in Figure 5, two gages were installed on one side of each girder several inches from its top and bottom, and one gage was installed on the bottom face of the FRP. In addition, strain gages were installed on the underside of two longitudinal stiffener beams in locations adjacent to the strain gages on the girders.

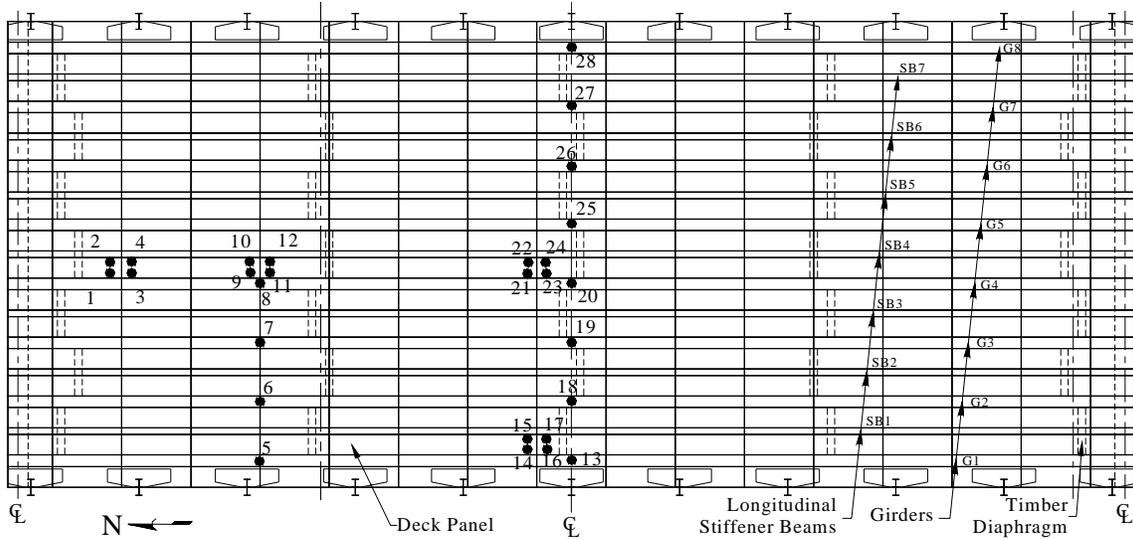


Figure 4. Delaware County Bridge displacement transducer locations

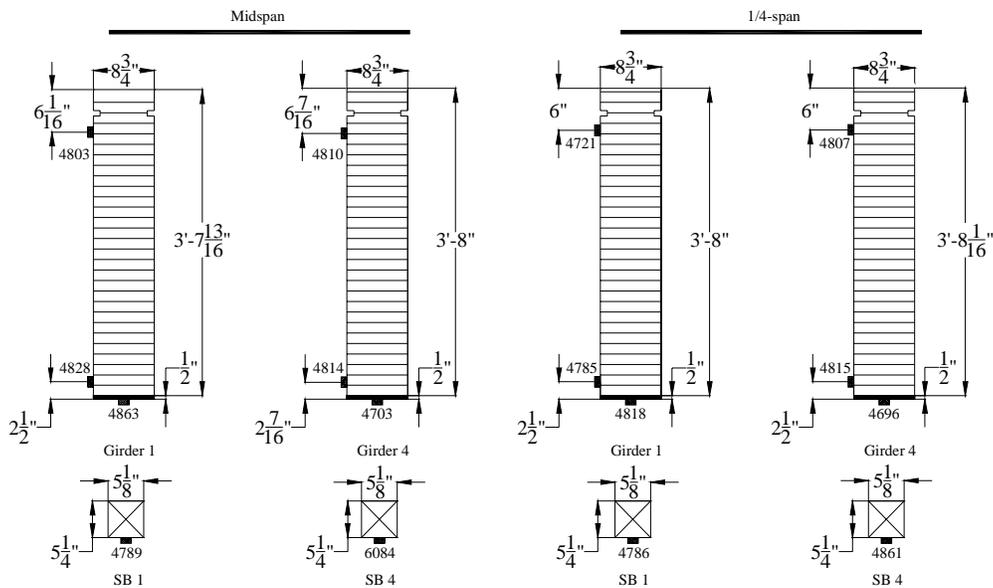


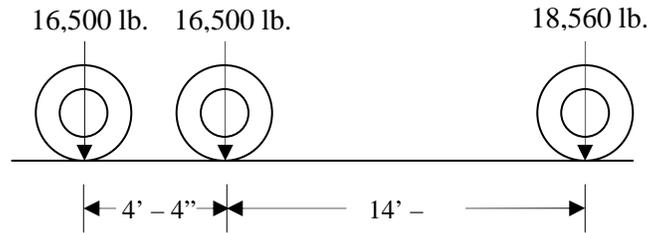
Figure 5. Delaware County Bridge strain gage locations

STATIC LOADING

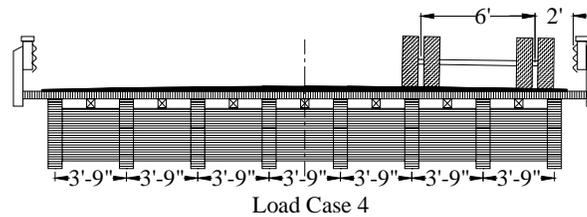
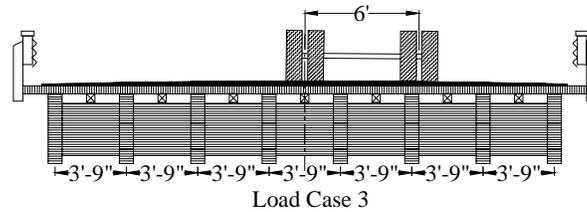
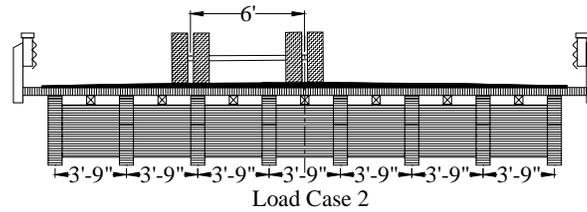
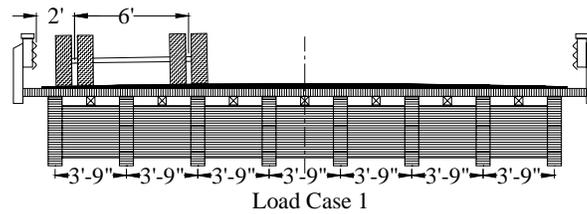
The bridge was loaded with a fully loaded (51,560 lb.) tandem axle dump truck provided by the Delaware County Secondary Roads Department. The vehicle configuration and axle loads are illustrated in Figure 6a. Two runs were completed for each of the four load cases investigated, which are illustrated in Figure 6b. The first load case consisted of the load truck driving southbound at crawl speed with the passenger-side wheel line offset 2 ft. from the west guardrail. The second load case consisted of the load truck driving southbound at crawl speed with the driver-side wheel line centered over the longitudinal centerline of the bridge. The third load case consisted of the load truck driving southbound at crawl speed with the passenger-side wheel line centered over the longitudinal centerline of the bridge. The fourth and

final load case consisted of the load truck driving southbound at crawl speed with the driver-side wheel line offset 2 ft. from the east guardrail.

The use of transversely symmetric load cases allows for verification that the load distribution characteristics are consistent across the width of the bridge and that the bridge is behaving as expected.



a. Vehicle configuration and axle loads



b. Load cases

Figure 6. Delaware County Bridge

CONDITION ASSESSMENT

Girders

Three condition assessments were conducted on the girders prior to load testing of the structure: first, pre-construction at the laminating plant; second, post-construction at the bridge site once the girders and deck panels had been set in place; and third, prior to load testing.

Pre-Construction

The initial inspection, conducted at the Alamco Wood Products, Inc. plant in Albert Lea, MN, found the glued-laminated girders were in excellent condition upon removal from the clamps. The girders were then planed, drilled, routed, and prepared for FRP installation.

Immediately following the fabrication of the girders, representatives from Iowa State University and Delaware County traveled to the plant site to witness the FRP installation. Mark Nahra, the Delaware County Engineer, was onsite to observe the installation of the FRP plate on the eight girders, which was performed by Justin Crouse, a research associate for the University of Maine. Installation of the 0.50 in. FRP plate transpired with no complications.

Figures 7 and 8 illustrate the condition of the glued-laminated girders before and after installation of the FRP, respectively.



Figure 7. Glued-laminated girders prior to FRP installation



Figure 8. Glued-laminated girders after FRP installation

Post-Construction

Delivery and installation of the girders went well with only one minor incident noted. During installation, girder G1 was apparently bumped, causing delamination of approximately the first 1 ft. of the FRP plate at the north end of the girder. Inspection revealed that the damage, which is difficult to identify, actually occurred in the timber itself and not the bond between the timber and the FRP. It appeared that approximately 1 in. of the girder was delaminated from the rest of the girder but not completely detached. It is anticipated that this will have little to no effect on the effectiveness of the FRP reinforcement, given the location of the damage.

Prior to Testing

There was no change found in the condition of the glued-laminated timber girders at the time of testing since the last inspection. All eight girders were well seated and no defects, other than the delamination which occurred during installation, were evident in the FRP reinforcement or glued-laminated timber portions of the girders. In addition, moisture content readings were taken, using an Elmhurst Moisture Meter with a 2 in. pin in both the deck panels and the girders prior to testing. In all locations the moisture content was approximately 15%.

Deck Panels

The deck panels are standard panels for this type of bridge construction, thus inspection prior to construction was not warranted; however, inspection of the deck panels was conducted post-construction and again prior to load testing.

Post-Construction

The initial inspection of the deck panels, immediately following placement on the girders, found no defects or problems in the glued-laminated deck panels themselves. The deck panels appeared to be relatively well seated on the girders in most locations; however, there were minor problems evident in the placement and attachment of the panels to the girders which may be significant in the future.

There were several locations where the deck panels were installed with gaps ranging from 0.25 in. to 0.50 in. between adjacent deck panels. However, there were also areas where the deck panels were set tightly against one another as desired.

Prior to Testing

Similar to the girders, there was no obvious change in the condition of the deck panels from the previous inspection. The deck panels had no areas of discoloration or damage and looked to be well-seated on the girders. However, the gaps between the girders were still evident at the edges of the deck where the panels were not covered by the wearing surface, and some of the s-clips had not yet been tightened in several locations.

Asphalt Wearing Surface

Prior to Testing

Currently, only one inspection has been completed on the asphalt wearing surface on the Delaware County Bridge. This initial inspection, conducted immediately before load testing of the bridge, resulted in some interesting findings.

First, there appeared to be no moisture-blocking membrane placed between the glued-laminated deck and the wearing surface to prevent moisture from coming in contact with the deck and underlying girders should there be a break in the wearing surface. Second, the wearing surface did not cover the entire deck surface but, instead, terminated just inside the metal base plates used to attach the guardrail posts to the deck, leaving approximately 1 ft. of the deck surface exposed. Third, the wearing surface was measured as approximately 3 in. in depth at centerline and tapered to approximately 1 in. in depth at the guardrail. Lastly, evident at each panel joint and at both abutments were small, noticeable, transverse cracks in the asphalt wearing surface. Figure 9 attempts to illustrate these cracks, although they are difficult to identify. As alluded to previously, these cracks will allow for infiltration of moisture to both the deck panels and girders, which may lead to further problems in the future.

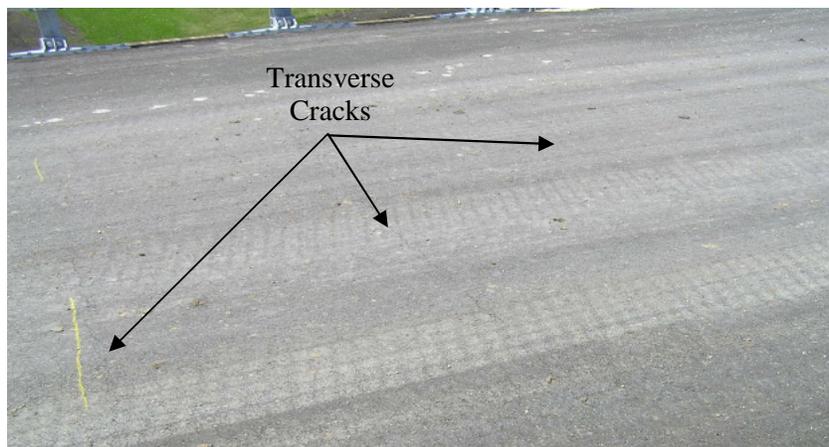


Figure 9. Transverse cracks at panel joints on Delaware County Bridge

RESULTS AND DISCUSSION

Global Deflection Performance

Illustrated in Figure 10 is the transverse global deflection of the bridge for all four load cases when the load truck is positioned longitudinally near mid-span. The maximum deflections were approximately -0.90 in. to -1.00 in. for load cases 1 and 4 at the exterior girders. When the load truck was positioned near the longitudinal centerline of the bridge; as in load cases 2 and 3, the maximum deflections were approximately -0.55 in. The decrease in the maximum deflection from load case 1 and 4 to load case 2 and 3 is likely the result of transverse load distribution characteristics associated with load position.

Current design specifications call for deflection checks of the form L/n , L being the clear span of the bridge. For this bridge, the American Association for State Highway Transportation Officials (AASHTO) Standard Specification (1996) specifies that global deflection of timber bridges be limited to 1.51 in.; the AASHTO LRFD (1998) specification limits the same deflection to 1.77 in.; the timber design manual, *Timber Bridges: Design, Construction, Inspection and Maintenance* (Ritter 1990), published by the Forest Service, limits this deflection to 2.09 in. The maximum deflections for the Delaware Co. Bridge for each load case are 0.72 in., 0.68 in., 0.41 in. and 0.40 in. after normalization to the standard HS-20 load truck for comparative purposes. This indicates that the deflection performance of the bridge is within the limits of the current design specifications and manual.

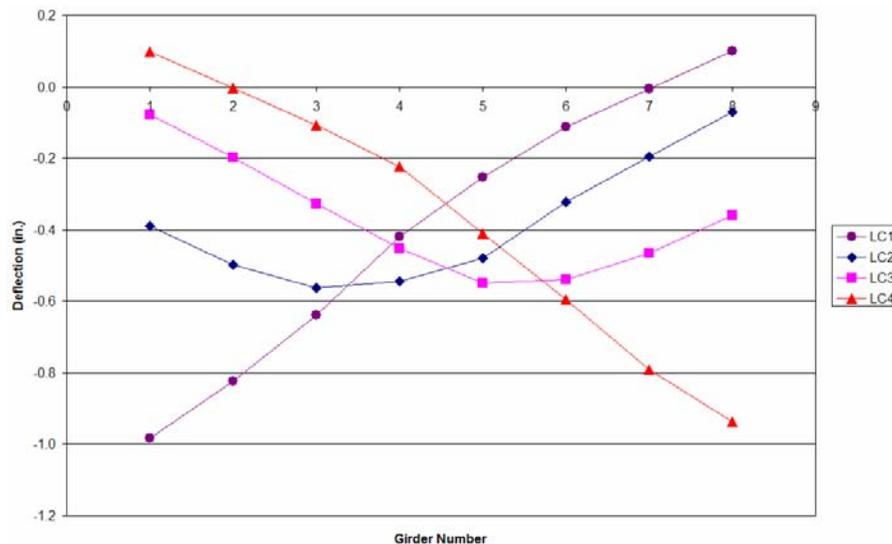


Figure 10. Girder deflection for load cases 1-4, load truck near mid-span

To investigate the level of girder end restraint, deflections were approximated from equations based on standard beam theory for a pinned-pinned condition and a fixed-fixed condition. These deflections were then compared with the measured deflections. Illustrated in Figure 11 are the calculated and measured deflections for one girder, which is representative of all girders for all four load cases. Since the experimental data trend line is closer to the fixed-fixed trend line than the pinned-pinned trend line, there is apparently some level of rotational restraint at the girder ends of the Delaware Co. Bridge.

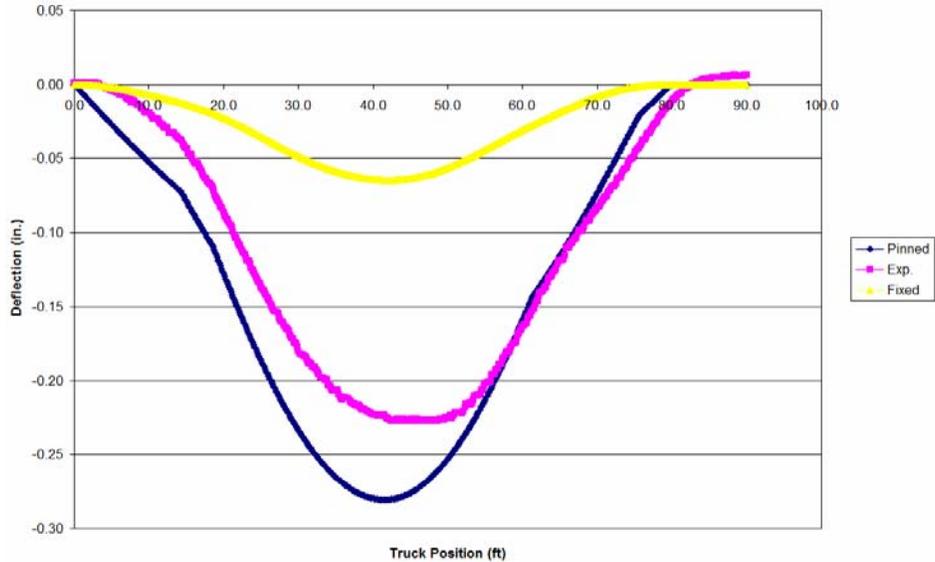


Figure 11. Calculated and experimental deflections for Delaware County Bridge

To investigate the load distribution characteristics of the bridge, approximate distribution factors were calculated from the measured deflections and compared with those computed with code equations. Using the assumption that all girders are of equal stiffness, an approximation of the distribution factors for each load case can be obtained from Equation. 1 using the physical test data. To obtain a more accurate estimate of the load distribution characteristics of the bridge, the distribution factors from two load cases were added together to obtain the approximate load distribution for the bridge with both lanes loaded, as assumed in the code.

$$DF_i = \frac{\Delta_i}{\sum_{i=1}^n \Delta_i} \quad (1)$$

where

- DF_i = distribution factor of the i th girder (lanes/girder)
- D_i = deflection of the i th girder
- SD_i = sum of all girder deflections
- n = number of girders

Illustrated in Figure 12 are the codified and experimental distribution factors for the Delaware County Bridge. From Figure 12 it is clear that the equations in [1], [2], and [3] used to calculate distribution factors are conservative.

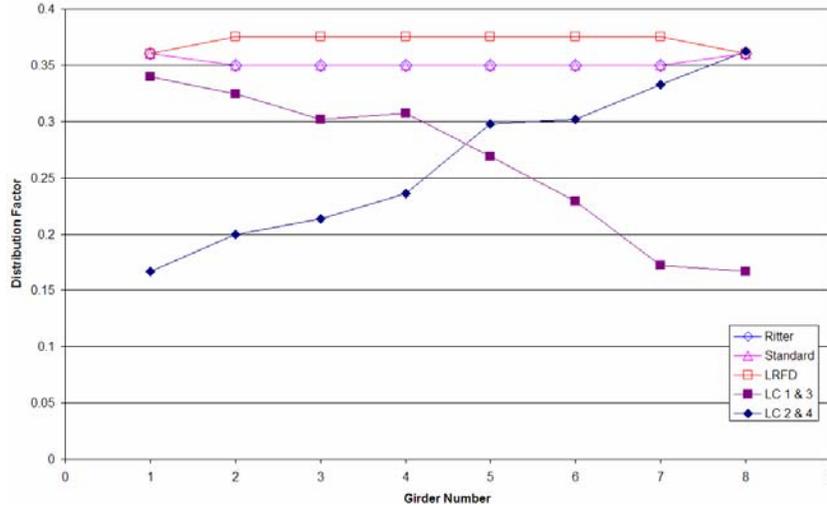


Figure 12. Experimental and codified distribution factors

Deck Panel Deflection Performance

In general, the deflection performance of the deck panels was satisfactory. Transversely, the deflection of the panels was as would be expected, with the deflection pattern of the panels following the same deflection pattern as the girders. This indicates that the deck panels possess adequate stiffness to effectively span between the girders. In previous research, it was determined that differential panel deflections were partially responsible for the deterioration of the wearing surface, specifically the cracking above the panel joints. Thus, the differential panel deflections and the effect the longitudinal stiffener beams have on these deflections are of particular interest.

The only specified limitation on this type of deflection is given in [3]. The manual states that relative deflection of the panels should be limited to 0.10 in., with a further reduction in this limit for deck panels supporting a pedestrian walkway or an asphalt wearing surface. Any reduction in the 0.10 in. limit is left up to the designer/engineer’s judgment and based solely on experience and user preference, not on the actual structural or serviceability performance of the bridge.

Listed in Table 1 are the maximum differential panel deflections for all eight locations for all four load cases. As shown in Table 1, the calculated differential panel deflections are significantly less than the 0.10 in. limit for all load cases investigated. The largest differential panel deflection calculated was 0.027 in. In addition, there appears to be little difference between the differential panel deflections calculated adjacent to the stiffener beams and those calculated midway between the stiffener beams and the girders. These small differential panel deflections suggest that the stiffener beams are limiting the magnitude of the differential panel deflections between the girders.

Table 1. Differential panel deflections of the Delaware County Bridge

Load Case	1-3 (in.)	2-4 (in.)	9-11 (in.)	10-12 (in.)	14-16 (in.)	15-17 (in.)	21-22 (in.)	23-24 (in.)
1	0.005	0.006	0.002	0.011	0.027	0.022	0.007	0.004
2	0.018	0.016	0.004	0.014	0.019	0.018	0.015	0.017
3	0.020	0.017	0.007	0.013	0.016	0.017	0.020	0.017
4	0.003	0.007	0.002	0.008	0.013	0.013	0.008	0.010

Strain

Using the measured strains from the live load tests, strain magnitudes were evaluated and compared with the code specified yield or ultimate strains (or in the case of the timber manual, the tabulated strains), and the location of the neutral axis and the strain distribution across the girder cross section were approximated.

Strain Magnitudes

The maximum measured strain at mid-span for girders G1 and G4 for all four load cases are listed in Table 2. The calculated strains from [3] are listed in Table 3 and were based on the tabulated bending stresses and modulus of elasticity for glued laminated timber. The tabulated bending stresses are 640 psi (C), 2,400 psi (T), and the tabulated modulus of elasticity is 1,800 ksi for the combination symbol 24F-V3. In addition, the ultimate strains of the FRP are also listed in Table 3. These values were calculated based on the published ultimate tensile strength and tensile modulus of elasticity and the ultimate compressive strength and compressive modulus of elasticity which are 130 ksi, 5,600 ksi, 102 ksi, and 5,400 ksi, respectively.

Based on the strain values listed in Tables 2 and 3, the strains in the timber load tests were below the tabulated tensile strain and less than approximately 60% of the tabulated compressive strain. Likewise, the measured strains in the FRP are below the specified ultimate strain of the FRP.

Table 2. Maximum measured strains (microstrain)

	Girder	Load Case (see Fig. 9)			
		1	2	3	4
Timber	G1 (Top)	228 (c)	110 (c)	26 (c)	34 (t)
	G1 (Bottom)	240 (t)	96 (t)	32 (t)	8 (c)
	G4 (Top)	93 (c)	178 (c)	195 (c)	102 (c)
	G4 (Bottom)	104 (t)	162 (t)	141 (t)	81 (t)
FRP	G1	340 (t)	123 (t)	22 (t)	-32 (c)
	G4	156 (t)	227 (t)	194 (t)	87 (t)

(c) – Compression
(t) – Tension

Table 3. Code specified tabulated and ultimate strains (microstrain)

	e_t	e_c
Timber (tabulated)	1,333	356
FRP (ultimate)	23,000	20,000

Neutral Axis Location

Using linear interpolation between the measured strains at the top of the girder and the bottom of the FRP laminate (Ref. Fig. 13 and Equation 2), the approximate neutral axis location for each instrumented location and each load case are listed in Table 4. Due to the deck connection detail, these calculations assume there is no composite action between the glued-laminated deck and the glued-laminated girders.

For load cases 1 and 2, the neutral axis is typically between 1 in. above and 2 in. below the mid-depth of the girder. For load cases 3 and 4, the neutral axis is typically between 1 in. and 6 in. below the mid-depth of the girder.

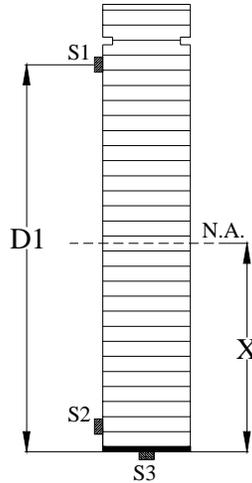


Figure 13. Girder neutral axis diagram

$$X = D1 * \left[\frac{S3}{(S3 - S1)} \right] \quad (2)$$

Table 4. Delaware County Bridge, neutral axis location (X) measured from the bottom of the FRP

Load Case	G1 – ¼ Span (in.)	G4 – ¼ Span (in.)	G1 – Midspan (in.)	G4 - Midspan (in.)
1	22.74	23.49	22.56	23.50
2	20.51	23.25	20.09	21.63
3	16.07	21.56	16.41	18.90
4	18.45	21.65	18.19	17.55

SUMMARY

This report summarizes work completed in three primary phases: (1) inspection of glued-laminated girder fabrication including the FRP installation, (2) condition inspections throughout construction of the bridge, and (3) the static field load testing of the bridge.

Phase 2 of the project was completed in two steps. The initial inspection of the structure was completed in early July 2004 prior to placement of the asphalt wearing surface. Overall, the structure looked sound, although there were two minor items worth noting. First, there were locations where there were gaps as wide as 0.50 in. between adjacent deck panels. There were also several s-clips (used to attach the deck panels to the girders) underneath the deck panels which were not completely tightened. The second inspection was completed immediately before testing of the structure in mid August 2004. This inspection revealed noticeable transverse cracks in the asphalt wearing surface above each of the panel joints. In addition, several of the s-clips under the deck were still found to be not completely tightened. The loose s-clips did not appear to be a significant problem at the time of testing but may allow the deck panels to become cupped in the future, leading to further deterioration of other parts of the structure including the wearing surface, girders, and guardrail.

Phase 3, static load testing of the bridge, was completed immediately following final inspection. The two-lane, single span Delaware Co. Bridge, composed of eight glued-laminated girders reinforced with FRP on the tension side and a transverse glued-laminated deck, performed well under static loading. Global

deflection of the structure was within current limits specified in AASHTO (1996), AASHTO LRFD (1998), and Ritter (1990). In addition, experimental lateral load distribution factors calculated from the field data suggests that the bridge distributes load more effectively than assumed in design.

From the load test data, there appeared to be no noticeable change in stiffness of girder G1, which had approximately the first foot of FRP on one end delaminated during bridge erection. The damage to the girder is only slightly obvious and only visible under close inspection of the girder. Since the damage was concentrated near the abutment, away from the tension zone of the girder, it is no surprise this had little effect on the response of the bridge during service loading.

The performance of the structure was equally adequate and within the limitations of AASHTO (1996), AASHTO LRFD (1998), and Ritter (1990) with regard to relative deflections such as differential panel deflections. For all load cases investigated, the calculated differential panel deflections were less than 0.03 in., which is approximately one-third of the 0.10 in. limit specified in Ritter (1990) for this type of deflection. The magnitudes of these deflections were found to be similar at locations adjacent to the longitudinal stiffener beams as well as midway between the stiffener beams and the girders. Based on differential panel deflections from similar bridges tested previously under similar loading configurations, the magnitude of these deflections appear to have been significantly reduced due in large part to the presence of the longitudinal stiffener beams.

Inspection prior to testing revealed transverse cracks in the asphalt wearing surface only weeks after placement of the wearing surface. The presence of these cracks along with small differential panel deflections indicates that there may be other factors affecting the condition of the asphalt-wearing surface on the bridge.

CONCLUSIONS

- The overall bridge structural performance under static live loading is adequate and within specified limits.
- Initial test results do not indicate a significant increase in stiffness resulting from the presence of the FRP reinforcement on the girders.
- There does not appear to be a decrease in the stiffness of girder G1 due to the delamination of the FRP which occurred during erection of the bridge.
- Regardless of the initial performance of girder G1 with the delaminated FRP, this girder should be inspected on a regular schedule to identify any further delamination or other changes resulting from the initial damage.
- The relative deflection performance of the deck panels with the longitudinal stiffener beams was well within specified limits; however, transverse cracking of the wearing surface above the deck panel joints appears to be occurring and is the result of unidentified factors.

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