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Live Load Distribution on Longitudinal Glued-Laminated Timber Deck Bridges

Final Report: Conclusions and Recommendations

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Abstract

Over the past few years the United States Department of Agriculture (USDA), Forest Products Laboratory (FPL), and the Federal Highway Administration (FHWA) have supported several research programs. This paper is a result of a study sponsored by FPL, with the objective of determining how truckloads are distributed to the deck panels of a longitudinal glued-laminated timber deck bridge. Currently, the American Association of State Highway and Transportation Officials (AASHTO) LRFD (load and resistance factor design) Bridge Design Specification provides live load distribution provisions for longitudinal glued-laminated timber deck-panel bridges.

The AASHTO LRFD live load distribution provisions for longitudinal glued-laminated timber deck bridges were based on the assumption that the bridge deck behaves as one slab and ignores the discontinuity of the bridge deck panels. This study investigated this assumption by using analytical models that validated field test data from several in-service bridges and data from a full-scale laboratory test bridge. The analytical models accounted for the effects of the interface between the deck panels as well as the effects of the transverse stiffener beams on the distribution of the live-load. The analytical live load distribution results above were compared with both the AASHTO LRFD and AASHTO Standard Specifications.

Keywords: timber, wood, bridges, girder, loads, load distribution

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SI conversion factors

Inch–pound unit	Conversion factor	SI unit
inch (in.)	25.4	millimeter (mm)
foot (ft)	0.3048	meter (m)
kip (1,000 lb)	× 4448.2	N (newton)
psi (lbf/in^2)	× 6894.8	Pa (pascal)
ksi (kip/in^2)	× 6.894	Pa (pascal)

In this paper 1 billion = 10^9

Live Load Distribution on Longitudinal Glued-Laminated Timber Deck Bridges

Final Report: Conclusions and Recommendations

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Introduction

The Forest Products Laboratory (FPL) has sponsored several research projects involving timber bridges, specifically longitudinal glued-laminated timber deck bridges, over the past three decades. Iowa State University (ISU) has contributed to this research by field testing several in-service longitudinal glued-laminated timber deck bridges under static and dynamic loading conditions. The research at ISU included collection of field data, testing of a full-scale bridge in the laboratory, and using analytical models to study the structural performance of these bridges. Over the past several years, the volume of collected field data has increased. However, to the authors' knowledge, these data have yet to confirm, or amend, the current bridge design provisions of the 2004 American Association of State and Highway Officials (AASHTO) load and resistance factor (LRFD) Bridge Design Specifications (AASHTO LRFD 2004). This paper focuses on these provisions, specifically the applicability of using the equivalent strip-width equations that are recommended to design slab bridges and this particular timber bridge type.

The 2004 AASHTO LRFD Specification recommends using the equivalent strip widths to design longitudinal glued-laminated timber deck bridges. Equivalent strip widths represent the partial width of the deck over which designers can assume uniform stresses from the effect of the live-load. This assumption simplifies the design process of slab bridges.

Longitudinal glued-laminated timber deck bridges are panelized systems. A typical cross-section view of a longitudinal glued-laminated timber deck bridge is shown in Figure 1. The deck consists of glued-laminated timber panels placed beside each other and connected with stiffener beams from beneath the bridge deck, as shown in Figure 2. The standard connection of the stiffener beam to the deck panels consists of a through bolt and is located near the edge of each deck panel. Optional to the through bolt connection, an aluminum bracket may also be used to connect the stiffener beam to the deck panels. The stiffener beams are considered as the main load-transfer mechanism from one panel to another.

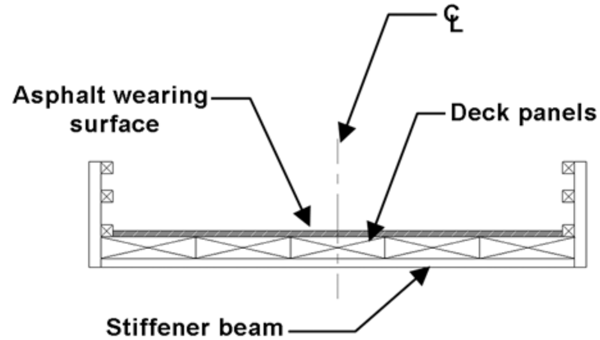

Figure 1. Cross section of a longitudinal deck bridge.

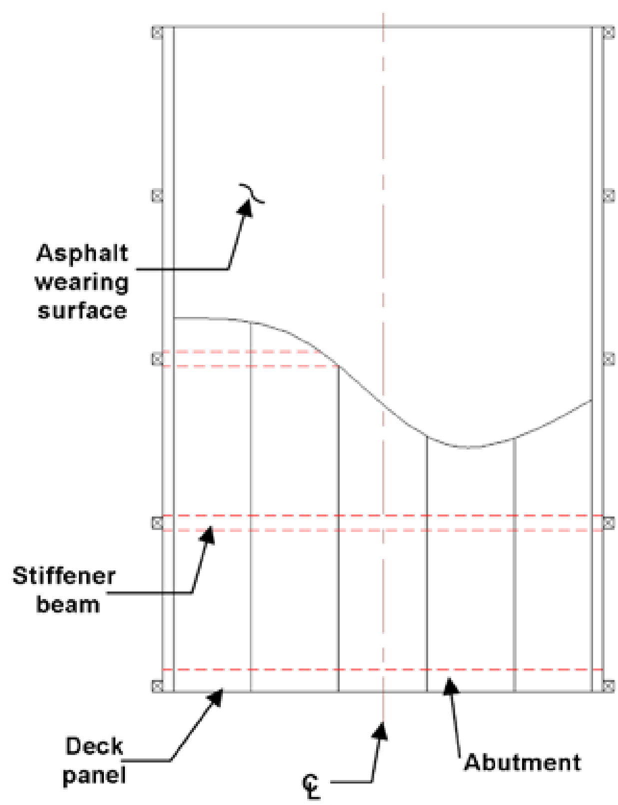

Figure 2. Plan view of a longitudinal deck bridge.

Objective and Scope

The overall objective of the study presented herein was to evaluate how an applied truck load is distributed among the deck panels of the longitudinal glued-laminated timber deck bridge system. This evaluation was attained by using test data from several in-service bridges, laboratory test bridges, and analytical results. These results were compared with the 2004 AASHTO LRFD and 1996 AASHTO Standard Specification live load distribution provisions for longitudinal glued-laminated timber deck bridges.

The objectives listed above were accomplished by completing the following five tasks:

1. Review the AASHTO Bridge Design Specifications and the associated load distribution criteria for longitudinal glued-laminated timber deck bridges. This review included both the AASHTO LRFD and AASHTO Standard Specifications.
2. Develop detailed analytical finite-element models to evaluate the structural performance of the longitudinal glued-laminated timber deck bridges. These analytical models account for the orthotropic behavior of timber material, the interface between the deck panels, and the connection between the deck panel and the stiffener beams.
3. The finite-element results were validated by comparing the analytical results of the deck panel deflections and live load distribution values to the data attained from field tests of the in-service bridges conducted by ISU researchers.
4. Study the influence of other parameters such as the interface between the deck panels, stiffener beam spacing, and the stiffener beam size on the distribution of live-load.
5. If required, develop live load distribution formulas. These formulas should be based on simplified methods or parametric equations using variables that are known during preliminary design.

Background

Simple live load distribution equations have appeared in the AASHTO Standard Bridge Design Specifications for many years. However, the AASHTO LFRD Bridge Design Specification introduced major revisions to the live load distribution provisions for slab-type bridges. Longitudinal glued-laminated timber deck panel bridges with spreader beams were included in these revisions.

The 1996 AASHTO Standard Specification (AASHTO 1996) live load distribution factors for longitudinal glued-laminated timber deck bridges were presented based on wheel loads, or half of the total axle load, carried by a single panel. The equations used for flexure design are listed in Table 1 for a panel under single or multiple truck loads. The AASHTO Standard Specification requires one stiffener beam to be placed at mid-span with all other stiffener beams placed at intervals of 10 ft or less. These stiffener beams are attached to the deck near the edges of the deck panels, typically with a bolted connection, and should have a stiffness of 80,000 kip/in^2 (ksi) or greater (AASHTO 1996).

Table 1—1996 AASHTO Standard Specification, wheel-load distribution factors[a]

Design loading	Equation for flexure[b]
One traffic lane	$\dfrac{W_p}{4.25+\dfrac{L}{28}}$ or $\dfrac{W_p}{5.50}$ whichever is greater
Two traffic lanes	$\dfrac{W_p}{3.75+\dfrac{L}{28}}$ or $\dfrac{W_p}{5.00}$ whichever is greater

[a]Source: AASHTO 3.25.3, 1996. AASHTO is the American Association of State Highway and Transportation Officials.
[b] W_p is width of panel (ft) ($3.5 \le W_p \le 4.5$),
and L, length of bridge, center of bearing to center of bearing (ft).

Table 2—2005 AASHTO LRFD Design Specification, equivalent strip width equations[a]

Design loading	Moment equation[b]
One traffic lane	$E = 10.0 + 5.0\sqrt{(L_1)(W_1)}$
Two or more traffic lanes	$E = 84.0 + 1.44\sqrt{(L_1)(W_1)} \le \dfrac{12.0W}{N_L}$

[a]Source: AASHTO LRFD 4.6.2.3 (2004). LRFD is load and resistance factor design.
[b]E is equivalent width (in.),
L_1 modified span length taken to the lesser of the actual span or 60 ft),
W_1 modified width of the bridge taken to be equal to the lesser of the actual width, or 60.0 for multi-lane loading, or 30.0 for single-lane loading (ft),
W physical edge-to-edge width of bridge (feet), and
N_L number of design lanes.

The 2004 AASHTO LRFD Bridge Design Specification (AASHTO LRFD 2004) provides equivalent strip-width equations for longitudinal glued-laminated timber deck bridges. The equivalent strip-width equations are based on lane loads, or full axle loads as shown in Table 2. These equations are also used to design reinforced concrete slab bridges and post-tensioned timber deck bridges. The AASHTO LRFD Specification requires one stiffener beam to be placed at intervals of 8 ft or less. The stiffener beam is connected with a through-bolt connection to the deck near the panel edges and should have a stiffness of 80,000 kip/in^2 (ksi) or greater (AASHTO LRFD 2004).

Multiple presence factors are included in the AASHTO Standard and LRFD Specification equations that are listed in Table 1 and Table 2, respectively. These factors account for uncertainties associated with the number of loaded lanes and are shown in Table 3. For example, for bridges with multiple

Table 3—AASHTO multiple presence "m" factors

Number of loaded lanes	Standard specification[a]	2005 LRFD[b]
1	1.0	1.2
2	1.0	1.0
3	0.9	0.85
> 3	0.75	0.65

[a]Source: American Association of State Highway and Transportation Officials (AASHTO) 1996.
[b]Source: load and resistance factor design, AASHTO LRFD 2004.

design lanes, it is unlikely that three adjacent lanes will be loaded at the same time. Therefore, the design load is decreased. For the single design-lane condition, the AASHTO LRFD multiple presence factor is greater than one to account for an overload condition (AASHTO LRFD 2004).

Literature Review

The 1996 AASHTO Standard Specification live load distribution provisions for longitudinal glued-laminated timber deck bridges, Table 1, were based on research performed by Sanders and others (1985). Sanders and others performed analytical studies to determine the load distribution characteristics of longitudinal glued-laminated timber deck bridges. The analytical models were created using SAP IV finite-element software (University of California, Berkeley 1974). In their work, Sanders and others used plate elements to model the deck panels and beam elements to model the stiffener beam. These elements were connected using rigid links. With the finite-element model, parametric studies were performed on bridges with span lengths from 9 to 33 ft, roadway widths from 16 to 40 ft, deck thickness from 6.75 to 12.25 in., and various stiffener-beam arrangements. Additionally, the width of the deck panels was varied from 42 to 54 in. (Sanders and others 1985).

Research of the longitudinal glued-laminated timber deck bridges was also conducted by Funke (1986). This research consisted of laboratory testing and analytical finite-element modeling using SAP IV finite-element software. The laboratory experiments were performed on full-scale bridges with a span length of 26 ft. Various stiffener beam, deck panel, and load-positioning arrangements were used in the laboratory testing. Laboratory results from this study verified the applicability of the live load distribution equations created by Sanders and others (1985). Favorable live load distribution behavior occurred when using at least three stiffener beams.

In the 1980s, the National Cooperative Highway Research Program (NCHRP) Project 12–26 (Zokaie and others 1991) developed live load distribution equations for slab bridges. The live load distribution equations documented in the NCHRP 12–26 report were the basis for the load-distribution provisions presented in the 2004 AASHTO LRFD Design Specifications. To develop equations with a wide range of applicability, a large database of bridges with various parameters was selected. The database consisted of 130 reinforced concrete slab bridges. Longitudinal glued-laminated timber deck bridges were not considered in NCHRP Project 12–26 (Zokaie and others 1991).

Zokaie and others (1991) used grillage models to evaluate the 130 reinforced concrete slab bridges. From these results, the authors of NCHRP 12–26 developed relationships to calculate the equivalent strip-width equations provided in Table 2 using grillage models. The grillage mesh consists of longitudinal and transverse beam elements. Load distribution factors were determined for each of the longitudinal beam elements, similar to the method used for girder-slab bridges. Dividing the load distribution factor by the width of the deck represented by the longitudinal beam element in the grillage model produces a moment distribution factor per unit width. The load distribution design width, or equivalent strip width, is determined by taking the inverse of this factor. Simply, the equivalent strip-width values can be determined using Equation (1). This equation allows one to relate live load distribution factors to equivalent strip widths. Edge-stiffening effects from guardrails, or barriers, were not included in the analysis (Zokaie and others 1991).

$$E_i = \frac{W_E}{DF_i} \quad (1)$$

where

DF_i	is	the lane load distribution factor of the *i*th longitudinal beam,
E_i		the equivalent strip width of the *i*th longitudinal beam (inches), and
W_E		the tributary width of longitudinal beam element.

Several analytical studies were performed on longitudinal glued-laminated timber deck bridges at ISU in recent years. Kurian (2001) conducted finite-element analyses to investigate the effects of several design parameters on the overall structural behavior of many in-service bridges. The parametric analyses performed by Kurian (2001) examined the effects of edge stiffening, boundary conditions, and the change in the timber modulus of elasticity. Kurian (2001) concluded that the modulus of elasticity of the deck material had a significant influence on bridge response when comparing the deflections attained from the analytical models with the field-test results. Kurian (2001) also noted that the influence of edge stiffening becomes insignificant to the panel deflections and stresses moving away from the exterior panels to the interior panels. Also in his study, Kurian (2001) focused only on deflection results and did not address load distribution.

Analysis of Longitudinal Glued-Laminated Timber Deck Bridges

The results reported herein were attained from detailed finite-element analyses. These analyses were carried out using the ANSYS finite-element software (Canonsburg, Pennsylvania 2004). ANSYS is a general-purpose finite-element program and was used to calculate deflections, stresses, and strains that are induced in several in-service longitudinal glued-laminated timber deck panel bridges under various truck loadings. To facilitate the construction of multiple finite-element models of various timber bridges, it was necessary to develop a preprocessor that simplifies the generation of such models. For this purpose, the ANSYS parametric design language (APDL) helped write the needed preprocessor. The preprocessor allows users with limited finite-element analysis knowledge to model longitudinal glued-laminated timber deck bridges. The preprocessor program accesses the information entered by the user to generate the finite-element model, as shown in Figure 3.

To execute the preprocessor, the user provides input parameters such as the span length, deck panel width, deck-panel thickness, material properties, the position and the magnitude of truckloads, and the bridge boundary conditions. In addition, the finite-element model constructed with the preprocessor allows the user to model the longitudinal glued-laminated timber deck bridges as either one single-deck panel or with individual deck panels. The deck panels may act as one single panel due to swelling of the deck panels. When modeling the individual deck panels, the program allows the user to utilize interface elements between the deck panels using nonlinear spring elements. The nonlinear spring elements allow the user to adjust the interaction of the deck panels by defining different coefficient of friction values to model the normal and sliding forces acting between the panels.

The finite-element model used solid brick elements to model the timber deck panels as well as the stiffener beam. This element allows one to incorporate the orthotropic timber material properties in the longitudinal (L), radial (R), and tangential (T) directions. The longitudinal modulus of elasticity is typically known. The orthotropic timber properties, related to the longitudinal modulus of elasticity used for this report are provided in the FPL 1999 Wood Handbook. The Wood Handbook provides the 12 constants required to represent the orthotropic properties of timber. The selected timber species was Douglas-fir, which is a typical softwood species used for glued-laminated timber beams.

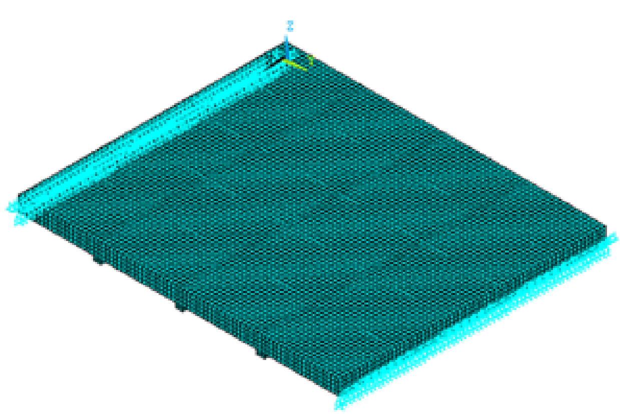

Figure 3. Three-dimensional rendering of the finite-element model.

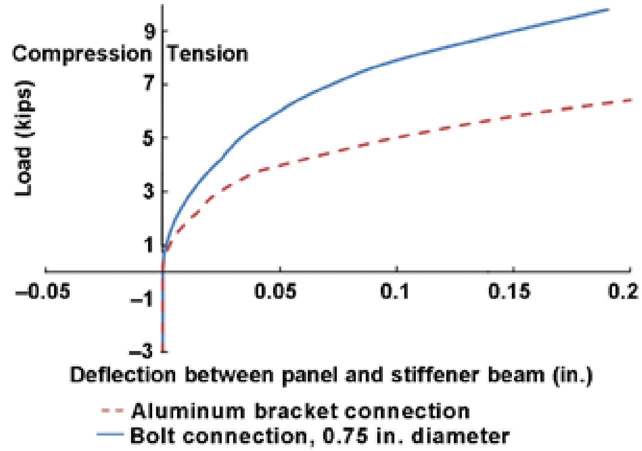

Figure 4. Load-deflection data used in the finite-element analysis, from Zokaie and others (1991).

The stiffener beam interaction with the deck panels varies over the width of the bridge. For this purpose, compression-only spring elements were used to idealize the interface between the panels and the stiffener beam. The stiffness of the spring element becomes zero when a gap exists between the deck panel and the stiffener beam. Additionally, tension-compression spring elements were used to model the through bolt, or aluminum bracket, connections that are required to connect the stiffener beam to the deck panels. The load displacement relationships of these connections, in tension, were determined from unpublished experimental test data provided by the Weyerhaeuser Company (Tacoma, Washington) (Hale 1978). The stiffness of the through bolt and aluminum bracket connections, when in compression, were assumed to be large and acted as a rigid connection. The tension-compression relationships of the aluminum bracket and through bolt connections are shown in Figure 4.

Analysis of In-Service Bridges

General

As previously mentioned, several in-service and laboratory longitudinal glued-laminated timber deck bridges were tested by ISU researchers. The collected data from these tests consisted of deflections that were recorded at the edges of each the deck panels. Longitudinally, these deflections were measured at, or near, the mid-span of each deck panel. The live load distribution factors of the in-service bridges, for each panel, were determined using Equation (2) (Hosteng 2004). In the work presented herein, these in-service live load distribution results were compared with the AASHTO Standard and LRFD live load distribution provisions. Additionally, the in-service deflection and live load distribution results were compared with the values attained using the finite-element modeling described above.

$$\mathrm{DF}_i = \frac{\Delta_i}{\Sigma_{i=1}^{n} \Delta i} \tag{2}$$

where

Δ_i	is	average deck panel deflection,
DF_i		lane load distribution fraction of the ith panel,
$\Sigma\Delta_i$		sum of average panel displacement, and
n		number of panels.

Angelica Bridge

Angelica Bridge in the Town of Angelica, New York, was tested by ISU researchers in 1996 and 2003 (Wipf and others 2004, page 10). The field test results presented herein were based on the 2003 results. This bridge has a span length of 21 ft by 4 in., a clear width of 28 ft by 3 in., and consists of seven glued-laminated deck panels. The deck panels have a width of 4 ft by 2 in. and a depth of 8.25 in. This bridge has two stiffener beams that are spaced at 7 ft by 6 in. (see Figure 5). The two stiffener beams are 6.875 in. wide and have a depth of 8.25 in. The stiffener beams were connected to the deck panels using through bolts. The asphalt-wearing surface on the deck panels was 2.5 in. thick.

The worst-case deflections and live load distribution factors from the field-test results were obtained when the test vehicle is located near the guardrail (see Fig. 5). As the truck moved transversely toward the center of the bridge, the deflection and live load distribution values would decrease. The controlling deflection results were created from the load case shown in Figures 5a and 5b. The test vehicle configuration is shown in Figure 6.

The field-test deflection results from the load position above were compared with the results attained using the finite-element analyses. Initially, the bridge was modeled with individual deck panels. However, this idealization resulted in larger overall deflections than those obtained from the field test. Notice from Figure 7 that the field-test results show minimal differential displacements between two adjacent deck panels. The maximum differential displacement between the panels is 0.037 in. Because of the small differential panel displacements, the bridge was then modeled as a single-deck panel. A combination of the swelling of the deck panels, close spacing of the stiffener beams, and the presence of the asphalt-wearing surface could be the reason the bridge behaves as a single panel.

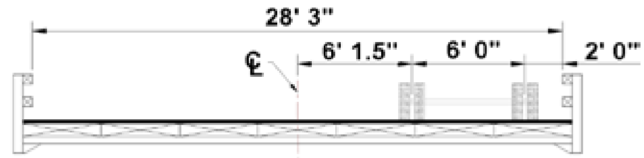

Figure 5a. Controlling transverse load position for Angelica Bridge.

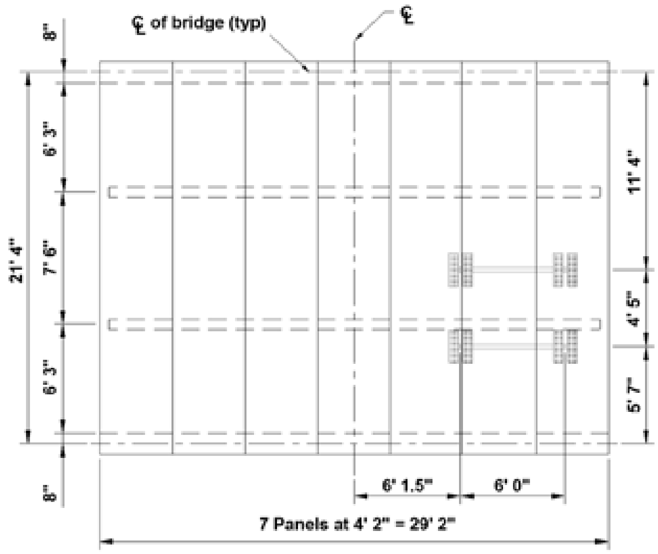

Figure 5b. Controlling load position for Angelica Bridge, plan view. (The front axle is outside the bridge.)

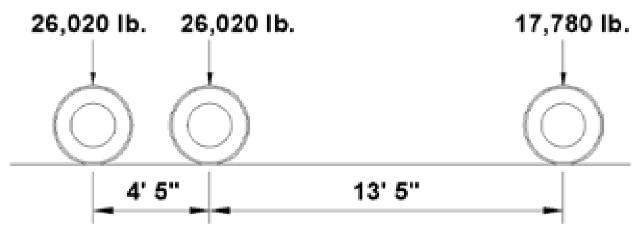

Figure 6. Angelica Bridge, test vehicle axle configuration.

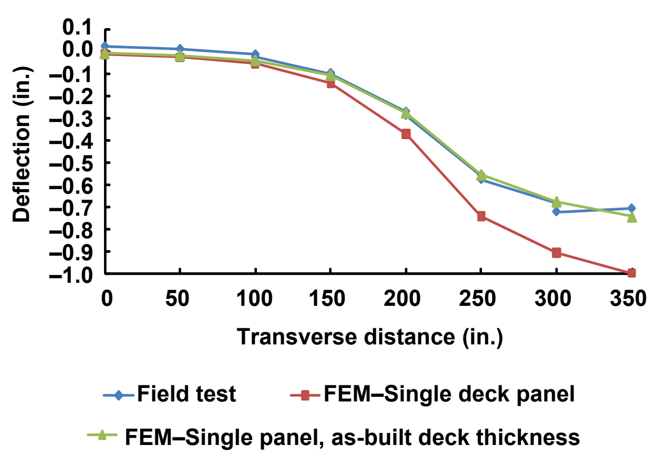

Figure 7. Deflection results for Angelica Bridge.

The effect of the asphalt-wearing surface was included in the analysis by assuming that the timber deck panel and the asphalt act compositely. Using strain compatibility, the modular ratio of the asphalt, and the timber deck panels, the thickness of the deck panels was increased by approximately 10%. Similar adjustment was used when analyzing the other in-service bridges.

The finite-element results obtained from modeling the deck as a single panel are shown in Figure 7. As can be noticed, the finite-element deflection results compare well with the field test results when modeling the as-built deck thickness, or when accounting for the asphalt-wearing surface.

The live load distribution factor results for Angelica Bridge are shown in Figure 8. For comparison with the field test and finite-element results, the 2004 AASHTO LRFD equivalent strip values would need to be converted to live load distribution factors per panel. From Table 2, the equivalent strip-width equation for a longitudinal glued laminated timber deck bridge under a single truck load is

$$E = 10.0 + 5.0\sqrt{(L_1)(W_1)} \quad (3)$$

Substituting the bridge length, L_1, and width, W_1, for Angelica Bridge into Equation (3), one will get the following equivalent strip-width value:

$$E = 10.0 + 5.0\sqrt{(21.33)(28.25)} = 132.74 \text{ in.} \quad (4)$$

This equation includes the 1.2 multiple presence factor per the AASHTO LRFD Specification (AASHTO LRFD 2004). To remove the multiple presence factor, one must multiply the equivalent strip-width value from above by 1.2:

$$E_{adj} = 132.74\ (1.2) = 159.28 \text{ in.} \quad (5)$$

Rearranging the equivalent strip-width and distribution factor relationship provided in Equation (1), provides Equation (6):

$$\mathrm{DF} = \frac{W_E}{E_{adj}} \quad (6)$$

where

DF is live load distribution factor converted from AASHTO LRFD equivalent strip width,

E_{adj} equivalent strip width the multiple presence factor removed, and

W_E tributary width longitudinal beam element, or width of the panel.

Using Equation (6), one can determine the AASHTO LRFD live load distribution factor for the width of the panel to be

$$\mathrm{DF} = \frac{50 \text{ in.}}{159.28 \text{ in.}} = 0.313$$

(without 1.2 multiple presence factor)

Figure 8 and Table 4 summarize live load distribution results for the Angelica Bridge when subjected to the load case shown in Figure 5. The finite-element single panel live load distribution factor results compare well to the field-test results. Accounting for effects of the wearing surface had minimal influence on the finite-element live load distribution results. Both the finite-element and the field-test results exceed the limits set by the AASHTO LRFD Specification when the multiple presence factor is removed. However, with the inclusion of the single-lane multiple presence

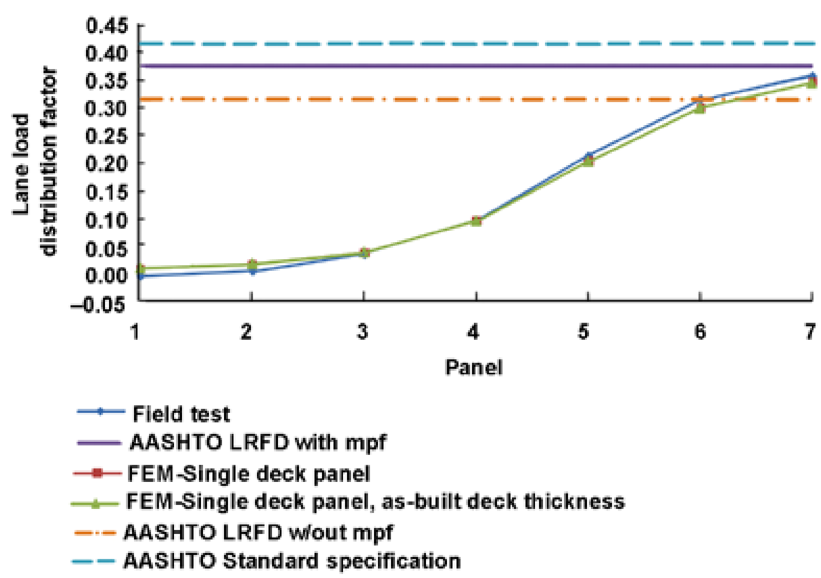

Figure 8. Live load distribution results for Angelica Bridge.

Table 4—Angelica Bridge, exterior panel live-load field test results

Technique	Live-load distribution factors
FEM[a]—single deck panel	0.345
AASHTO[b] Standard specification	0.416
AASHTO LRFD[c] with mpf[d]	0.376
AASHTO LRFD without mpf	0.313
Adjusted mpf by 0.9 reduction	0.338

[a] FEM is finite element model.
[b] AASHTO is American Association of State Highway and Transportation Officials,
[c] LRFD is load and resistance factor design.
[d] Mpf is multiple presence factor.

factor, the AASTHO LRFD Specification does provide conservative results.

East Main Street Bridge

East Main Street Bridge located in the Town of Angelica, New York, was tested by ISU researchers in 1996 and 2003 (Wipf and others 2004, page 9). The field-test results presented herein were based on the 2003 results. The bridge has a span length of 30 ft by 6 in., a clear width of 34 ft by 0 in., and consists of eight glued-laminated deck panels. The deck panels have a width of 4 ft by 5 in. and a depth of 14.25 in. This bridge has four stiffener beams, which are spaced at 6 ft by 0 in. The stiffener beams are 6.875 in. wide and have a depth of 4.5 in. The stiffener beams were connected to the deck panels with through bolts. The asphalt-wearing surface is 3.0 in. thick. The worst-case deflections and live load distribution factors from the field tests were created from the load case shown in Figures 9a and 9b. The test vehicle configuration is the same as shown in Figure 6.

The deflection and live load distribution factors for East Main Street Bridge are shown in Figures 10 and 11, respectively. These results are based on the load condition shown in Figures 9a and 9b. Unlike the previous bridge, edge-stiffening effects were observed in the exterior panels. Further adjustments were made to the finite-element as-built deck thickness results, incorporating edge-stiffening effects. This was accomplished using the results published by Anil (Kurian 2001). The adjustment was made by reducing the deflections using the difference between the results obtained with and without the railing system as documented by Anil (Kurian 2001). Similar to the previous bridge, the AASHTO LRFD equivalent strip-width values, with and without the multiple presence factor, were converted to a distribution factor. The controlling exterior panel live load distribution results are provided in Table 5. In addition, the AASHTO Standard Specification live load distribution factors from Table 1 were included in the results for East Main Street Bridge.

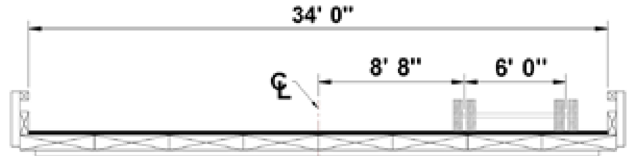

Figure 9a. Controlling transverse load position for East Main Street Bridge.

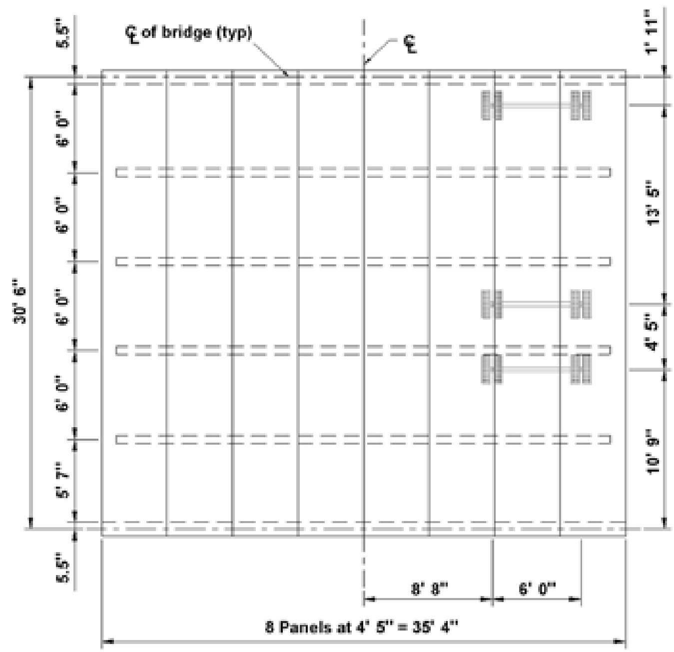

Figure 9b. Controlling load position for East Main Street Bridge, plan view.

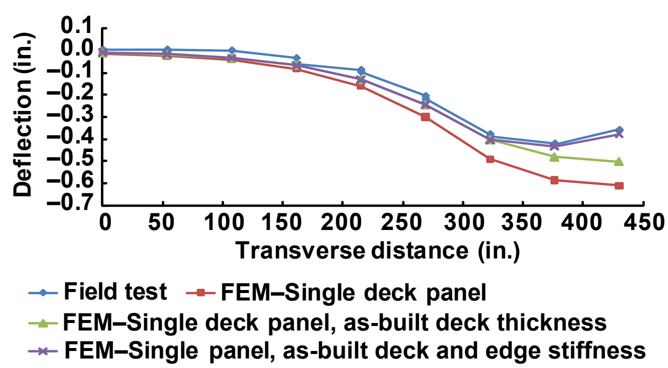

Figure 10. Deflection results, East Main Street Bridge.

Bolivar Bridge

Bolivar Bridge, located in the Town of Angelica, New York, was tested by ISU researchers in 1996 and 2003 (Wipf and others 2004, page 11). The field test results presented herein were based on the 2003 results. The bridge has a span length of 28 ft by 8 in., a clear width of 24 ft by 8 in., and consists of six glued-laminated deck panels. The deck panels have a width of 4 ft by 5 in. and a depth of 15.0 in. This bridge has three stiffener beams that are spaced at 7 ft by 6 in. The

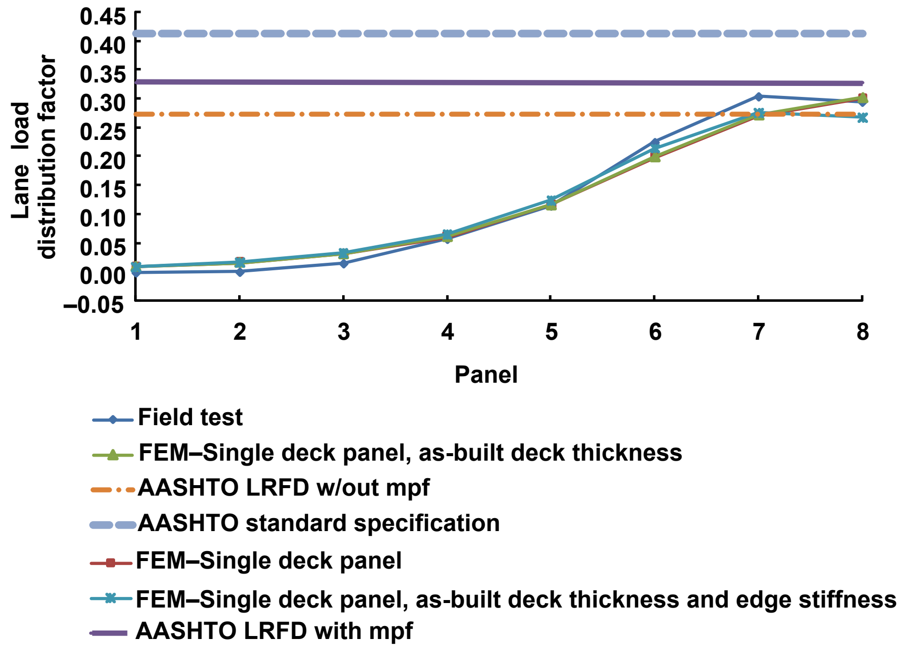

Figure 11. Live load distribution results for East Main Street Bridge.

Table 5—East Main Street Bridge, exterior panel live-load field test results

Technique	Live-load distribution factors
Field test	0.304
FEM—single deck panel[a]	0.301
AASHTO Standard specification[b]	0.414
AASHTO LRFD with mpf[c]	0.329
AASHTO LRFD without mpf[d]	0.274

[a] FEM is finite-element model.
[b] AASHTO is American Association of State Highway and Transportation Officials.
[c] LRFD is load and resistance factor design.
[d] Mpf is multiple presence factor.

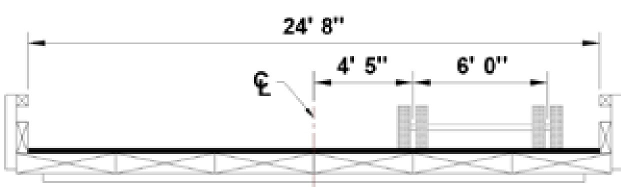

Figure 12a. Controlling transverse load position for Bolivar Bridge.

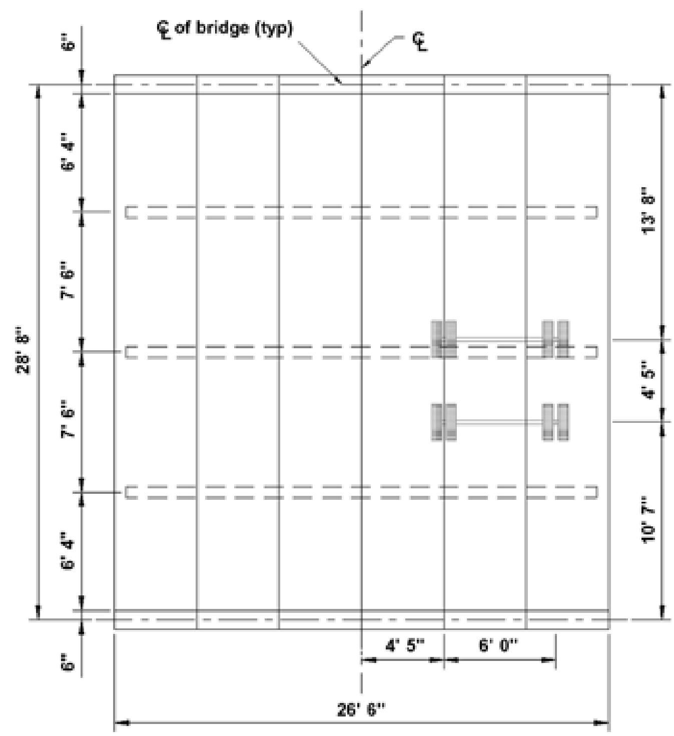

Figure 12b. Controlling load position for Bolivar Bridge, plan view.

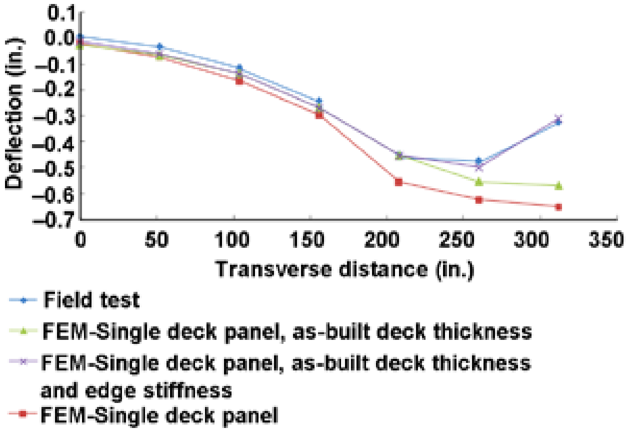

Figure 13. Deflection results for Bolivar Bridge.

two stiffener beams are 6.875 in. wide and have a depth of 4.5 in. The stiffener beams were connected to the deck panels with through bolts. The asphalt-wearing surface is 2.5 in. thick. The effect of the wearing surface was included in the analysis, as explained above. The guard-railing system consisted of timber posts and a glued-laminated timber panel barrier; they were not explicitly included in the finite-element model. The worst-case deflections and live load distribution factors from the field test results were created from the load case shown in Figures 12a and 12b. The test vehicle configuration is the same as shown in Figure 6.

The deflection and live load distribution factors for Bolivar Bridge are shown in Figures 13 and 14, respectively. These results are based on the load condition shown in Figures 12a and 12b. Edge-stiffening effects were observed in the exterior panels, and deflections were adjusted as described

Table 6—Bolivar Bridge, exterior panel live-load field test results

Technique	Live-load distribution factors
Field test	0.312
FEM—single deck panel[a]	0.310
AASHTO Standard specification[b]	0.411
AASHTO LRFD with mpf[c]	0.355
AASHTO LRFD without mpf[d]	0.296

[a] FEM is finite-element model.
[b] AASHTO is American Association of State Highway and Transportation Officials,
[c] LRFD is load and resistance factor design.
[d] Mpf is multiple presence factor.

Table 7—Scio Bridge, exterior panel live-load field test results

Technique	Live-load distribution factors
Field test	0.366
FEM—single deck panel[a]	0.338
AASHTO Standard specification[b]	0.447
AASHTO LRFD with mpf[c]	0.398
AASHTO LRFD without mpf[d]	0.331

[a] FEM is finite-element model.
[b] AASHTO is American Association of State Highway and Transportation Officials.
[c] LRFD is load and resistance factor design.
[d] Mpf is multiple presence factor.

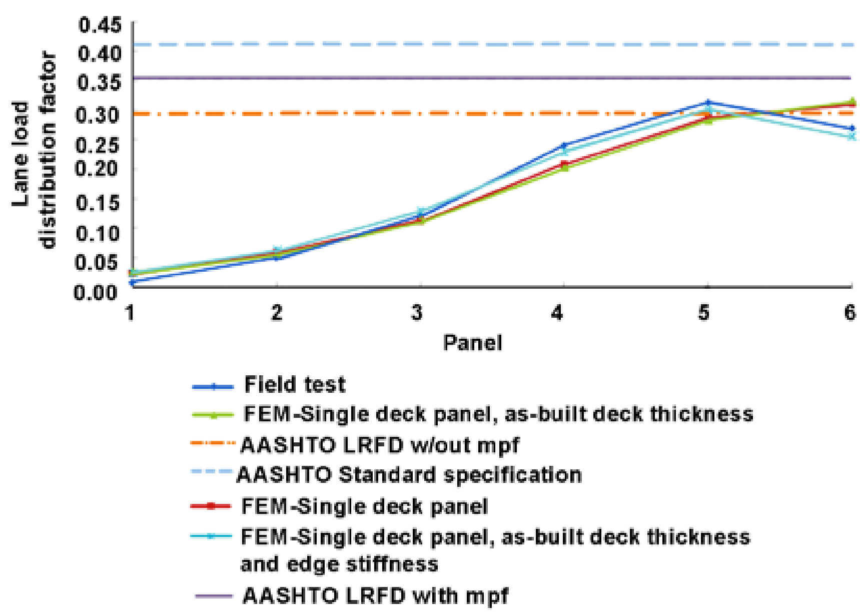

Figure 14. Live load distribution factor results for Bolivar Bridge.

previously. As before, the AASHTO LRFD equivalent strip-width values, with and without the multiple presence factor, were converted to distribution factors. The controlling exterior panel live load distribution results are provided in Table 6.

Scio Bridge

Scio Bridge in the Town of Angelica, New York, was tested by ISU researchers in 1996 and 2003 (Wipf and others 2004, page 12). The field test results presented herein were based on the 2003 results. The bridge has span length of 20 ft by 8 in., a clear width of 30 ft by 0 in., and consists of six glued-laminated deck panels. The deck panels have a width of 4 ft by 4 in. and a depth of 9.0 in. This bridge has three stiffener beams that are spaced at 7 ft by 6 in. The two stiffener beams are 6.875 in. wide and have a depth of 4.5 in. The stiffener beams were connected to the deck panels with the through bolt connection. The asphalt wearing surface is 6.0 in thick. The effect of the wearing surface was included in the analysis, as explained above. The guard railing system consisted of timber posts and a glued-laminated timber panel barrier. The worst-case deflections and live load distribution factors from the field test results were created from load case shown in Figure 15. The test vehicle configuration is the same as shown in Figure 6.

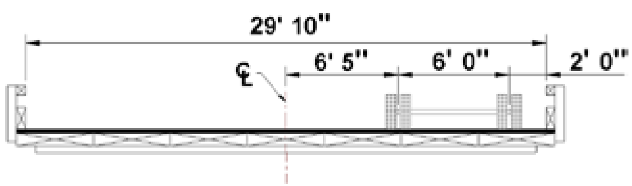

Figure 15a. Controlling transverse load position for Scio Bridge.

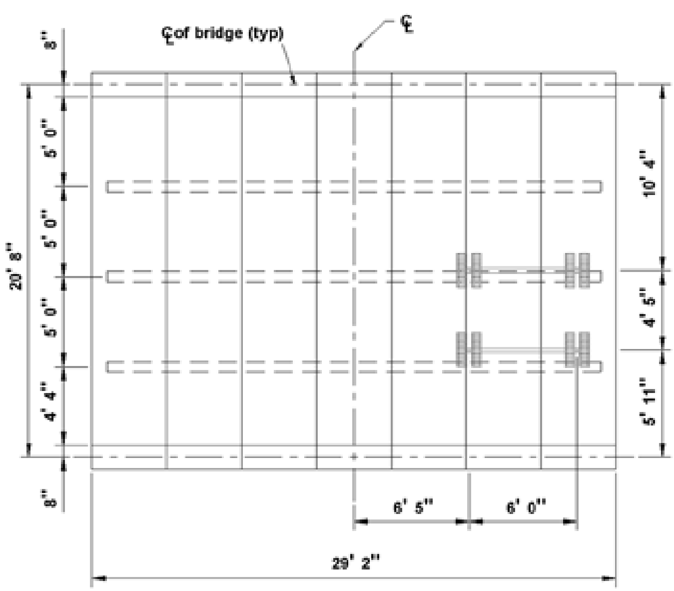

Figure 15b. Controlling load position for Scio Bridge, plan view.

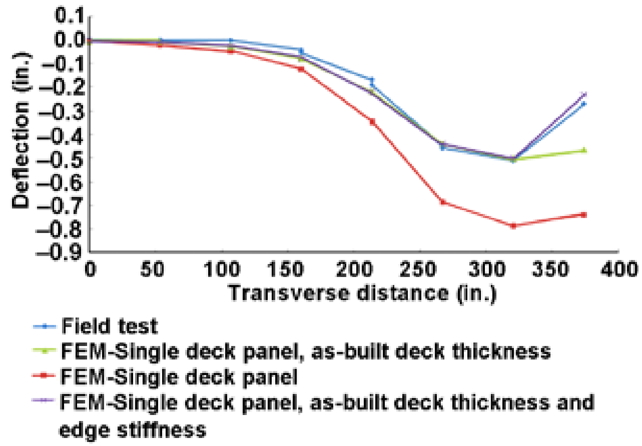

Figure 16. Deflection results, Scio Bridge.

The deflection and live load distribution factors for Scio Bridge are shown in Figures 16 and 17, respectively. These results are based on the load condition shown in Figures 15a and 15b. Edge-stiffening effects were observed in the exterior panels and the deflections were adjusted as described previously. As before, the AASHTO LRFD equivalent strip-width values, with and without the multiple presence factor, were converted to distribution factors. The controlling exterior panel live load distribution results are provided in Table 7.

Analysis of the Laboratory Test Bridge

General

The full-scale laboratory bridge tested by Funke (1986) was also analyzed. This allowed studying the behavior of the longitudinal glued-laminated timber deck-panel bridge without the influence of swelling, the asphalt-wearing surface, and edge-stiffening effects from guardrails or barriers. The laboratory test bridge had a span length of 26 ft by 0 in. This bridge set-up consisted of six deck panels with one stiffener beam located at the mid-span of the bridge. The deck panels were 4 ft by 0 in. wide and had an average depth of 10.72 in. The stiffener beam had a depth of 4.5 in. and a width of 6.75 in. The stiffener beam was connected to the deck panels with the through bolt connection described earlier. The load consists of a single HS20-44 design truck placed 30 in. from the edge of the deck as shown in Figure 18. Longitudinally, two axles were placed on the bridge. One axle was placed 2 ft by 6 in. from the center line of the abutment and the other axle was placed 14 ft by 0 in. from the first.

The laboratory bridge (Funke 1986) was analyzed as having individual deck panels and as one single-deck panel. When modeling the bridge with the individual deck panels, the nonlinear spring elements connecting the deck panels were assigned negligible coefficient of friction and stiffness

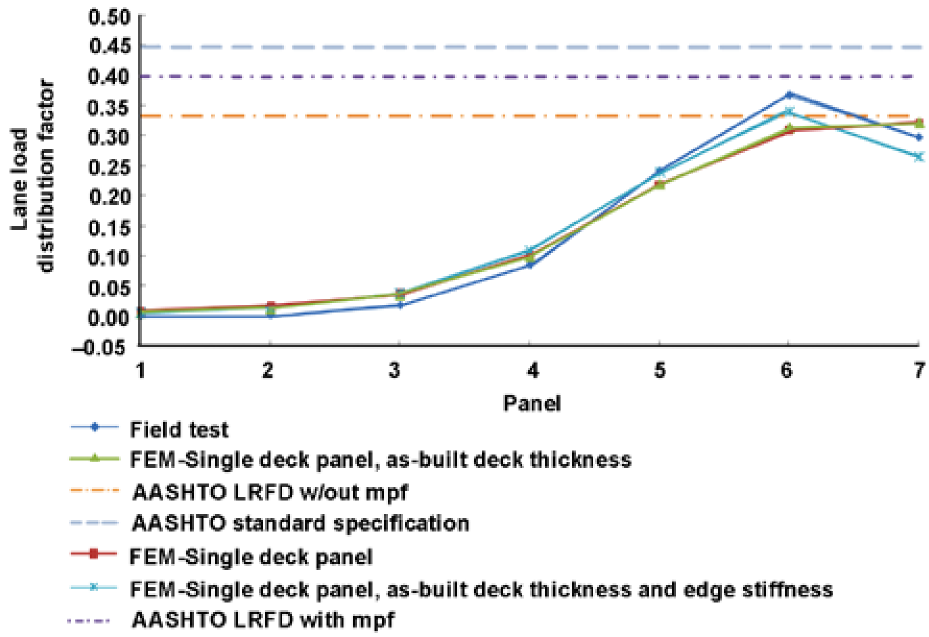

Figure 17. Live load distribution factor results, Scio Bridge.

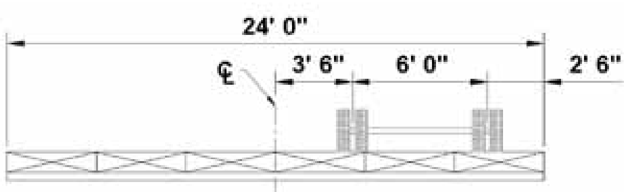

Figure 18. Laboratory test bridge 1TE6-A.

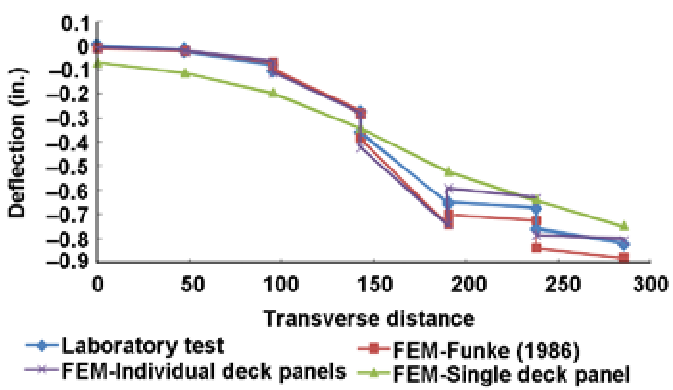

Figure 19. Laboratory test bridge 1TE6-A, deflection.

values, allowing the deck panels to slide freely. Therefore, the stiffener beam was the only path to transfer the load from panel to panel. As mentioned above, the stiffener beam was connected to the deck panels with through bolts. Therefore, the compression-tension force versus displacement values for the through bolt connection, shown in Figure 4, were used by the preprocessor described above.

The displacement results obtained from the analytical model compared well to the displacement resulted from the laboratory test (see Fig. 19). The individual deck panel finite- element results were within a 2% difference of the laboratory displacement results.

The live load distribution factor results for the laboratory test of the bridge, finite-element analyses, and AASHTO LRFD and Standard Specifications are shown in Figure 20. One can observe that the controlling live load distribution factor is located at the exterior panel. The individual deck panel finite-element results are within a 2% difference of the laboratory live load distribution results. The controlling live load distribution factor when modeling the deck as a single panel compared well to the AASHTO LRFD live load distribution value with the multiple presence factor removed. However, when modeling the deck as individual panels, the finite-element and field test results compared well to the AASHTO Standard specification limit shown in Figure 20. A summary of the controlling live load distribution factors from the different analyses is provided in Table 8.

From the live load distribution factor results of the laboratory bridge, one can notice that the deck of the bridge does not behave as a single panel structure because of the large differential displacement between the deck panels. This was expected due to the large spacing between the stiffener beams, absence of a wearing surface, and small friction between the deck panels. Additional finite-element trials were later performed to investigate the effects of the stiffener

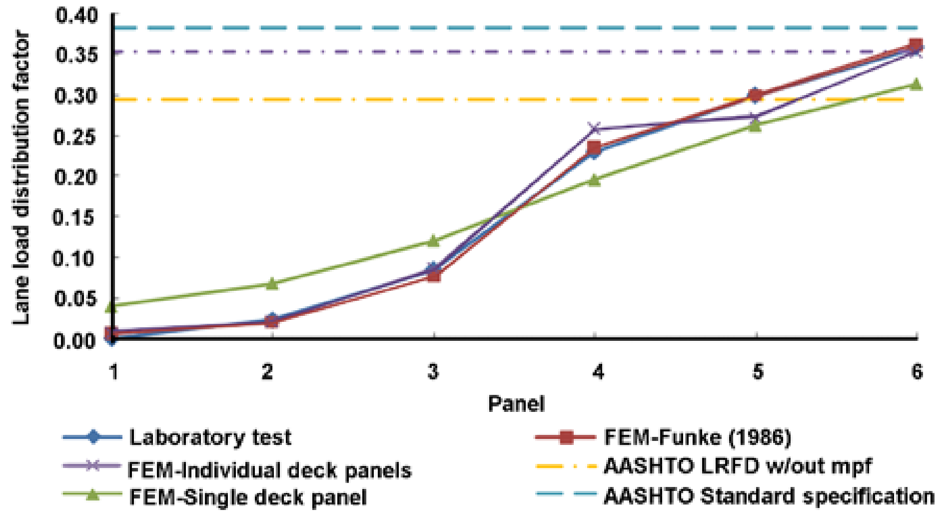

Figure 20. Laboratory test bridge 1TE6-A, live load distribution results.

Table 8—Laboratory Bridge, exterior panel live-load distribution results

Technique	Live-load distribution factor results
Laboratory test	0.359
FEM—individual deck panels[a]	0.368
FEM— single deck panel	0.311
AASHTO Standard specification[b]	0.384
AASHTO LRFD specification with mpf[c]	0.355
AASHTO LRFD specification without mpf[d]	0.296

[a] FEM is finite-element model.
[b] AASHTO is American Association of State Highway and Transportation Officials.
[c] LRFD is load and resistance factor design.
[d] Mpf is multiple presence factor.

Table 9—Stiffener beam parametric study

Parameter	Lane-load distribution factor
AASHTO[a] Standard specification	0.384
AASHTO LRFD[b] without mpf[c]	0.296
AASHTO LRFD with mpf	0.355
No stiffener beam	0.500
1 stiffener beam	0.368
2 stiffener beams	0.360
4 stiffener beams	0.351
(2×) stiffener beam depth	0.356
Single-deck panel	0.311

[a]AASHTO is is American Association of State Highway and Transportation Officials.
[b] LRFD is load and resistance factor design.
[c] Mpf is multiple presence factor.

beam spacing, stiffener beam size, and influence of friction on the laboratory bridge above.

Table 8 shows that the live load distribution factor obtained from the laboratory test agrees with that obtained using the 2004 AASHTO specification including the multiple presence factor. Also, the results in Table 8 illustrate that modeling the deck as individual panels resulted in higher distribution factors than those obtained from the test or the 2004 AASHTO codes.

Effects of Stiffener Beam Properties and Spacing

Using the laboratory test bridge, a parametric study was conducted to investigate the influence of the stiffener beam properties and spacing on the live load distribution results. Using the load configuration shown in Figure 18, the controlling live load distribution values were determined for the exterior panel. These results are listed in Table 9. One can observe how the load is distributed from the exterior to the adjacent panels as the number of stiffeners is increased. However, increasing the number of stiffener beams alone does not provide a result that fully converges to the results obtained, assuming the deck panel acts as a single-deck panel. Therefore, for the single panel action to occur a combination of swelling and close-stiffener beam spacing must be present.

The influence of the swelling on the behavior of the bridge is difficult to quantify. As the bridge panels swell, additional load is transferred to adjacent panels through friction forces. Similar to Table 9, additional trials were performed modifying the interaction of the deck panels. When modeling the bridge with the individual deck panels, the nonlinear spring elements connecting the deck panels were assigned large

Table 10—Stiffener beam parametric study including deck panel interaction

Parameter	Lane-load distribution factor
AASHTO Standard specification[a]	0.384
AASHTO LRFD without mpf[b,c]	0.296
AASHTO LRFD with mpf	0.355
No stiffener beam	0.408
1 stiffener beam	0.355
2 stiffener beams	0.341
4 stiffener beams	0.331
(2×) stiffener beam depth	0.341
Single deck panel	0.311

[a]AASHTO is American Association of State Highway and Transportation Officials.
[b] LRFD is load and resistance factor design.
[c] Mpf is multiple presence factor.

coefficient of friction and stiffness values. The controlling lane-load distribution results, for the controlling exterior deck panel, are shown in Table 10.

Comparing the results from Tables 9 and 10, one can observe the influence of the deck panel interaction with multiple stiffener-beam arrangements. Notice from Table 10 the 7% difference between the single-deck panel results and results using four stiffener beams, including the deck panel interaction. In the author's opinion, one could provide a transverse post-tensioning system to increase the deck panel interaction. This would aid in the distribution of load and assure the panelized system behaves similar to a single-deck panel structure.

Multiple Vehicle Loads

The above analyses focus on single-design truckloads. From these analyses, one can note that the in-service bridges perform similarly to a single-panel structure and compared reasonably well to the 2004 AASHTO LRFD live load distribution provisions. The AASHTO LRFD equivalent strip-width equations were further investigated by analyzing several bridges under multiple lane loads. These bridges were modeled for the finite-element analysis as a single or as individual deck panels. The effects from the asphalt-wearing surface and edge-stiffening effects from guardrails will be neglected.

The first bridge analyzed with two vehicle loads is shown in Figure 21. The bridge has a span length of 26 ft by 0 in. and a clear width of 24 ft by 0 in., similar in dimensions to the laboratory test bridge. The deck panels were 4 ft by 0 in. wide and had a depth of 10.72 in. Three stiffener beams were spaced at 6 ft by 6 in., each having a depth of 4.5 in. and a width of 6.75 in. As previously stated, the bridge was modeled as a single-deck panel, behaving similarly to the in-service bridges. The single-deck panel was divided into six sections, each having a tributary width of 4 ft by 0 in.

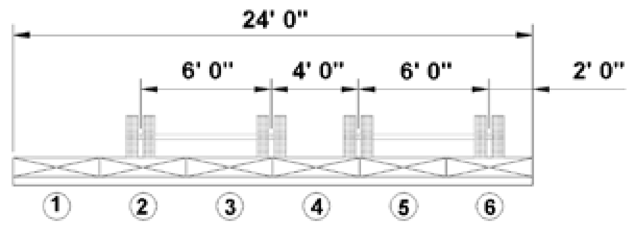

Figure 21. AASHTO LRFD tandem truck loading.

Table 11—Multiple lane load results

Panel number	Stress psi	Moment ft./kips	Equivalent width E (ft.)
1	706.61	54.13	20.32
2	1,002.98	76.84	14.32
3	1,095.64	83.94	13.10
4	1,285.36	98.47	11.17
5	1,283.70	98.35	11.18
6	1,367.50	104.77	10.50
Sum	6,741.79	516.50	–

The average stress and moment results for each of the six sections was used to determine the equivalent strip-width values, similar to a slab-girder bridge. The controlling beam-line moment of 275 ft/kips was due to the AASHTO LRFD tandem loading condition shown in Figure 2. The results are provided in Table 10.

Using the AASHTO 2004 LRFD code gives an equivalent strip-width value of 10.0 ft. This value agrees with the equivalent strip width calculated using the induced stresses in panel 6 (see Table 11).

The second bridge analyzed with two vehicle loads was the previously described East Main Street Bridge. As stated before, the bridge was modeled as a single-deck panel. Edge-stiffening effects were neglected, modeling the clear width of the bridge. The single-deck panel was divided into eight sections, the inner sections had a tributary width of 4 ft by 6 in. and the two outer sections had a tributary width of 3 ft by 5 in. The average stress and moment results for each of the eight sections were used to determine an equivalent strip-width value. The controlling beam-line moment of 331 ft/kips was due to the AASHTO LRFD tandem loading condition shown in Figure 22.

Using the AASHTO 2004 LRFD code gives an equivalent strip-width value of 10.6 ft. This value agrees with the equivalent strip width calculated using the induced stresses in panel 8 (see Table 12).

Conclusions

This research involved the evaluation of the existing live load distribution equations for longitudinal glued-laminated

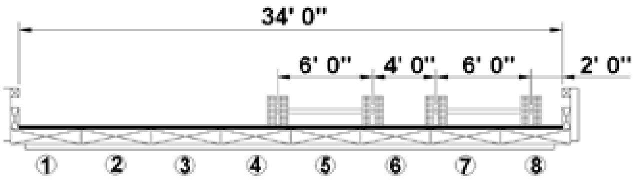

Figure 22. AASHTO LRFD (American Association of State Highway and Transportation Officials, load and resistance factor design) tandem truck loading, East Main Street Bridge.

Table 12—Multiple lane load results, East Main Street Bridge

Panel number	Stress psi (lb/in^2)	Moment (ft/kips)	Equivalent width *E* (ft)
1	108.48	12.40	93.68
2	159.42	23.43	63.74
3	271.77	39.95	37.39
4	521.12	76.60	19.50
5	703.69	103.44	14.44
6	877.06	128.93	11.59
7	907.26	133.37	11.20
8	993.96	113.64	10.22
Sum	4,542.76	631.77	–

timber deck bridges provided in the 2004 AASHTO LRFD Bridge Design Specification. This was accomplished by using analytical finite-element models, which were validated with test data from in-service and laboratory bridges. The test data consisted of deflections and live load distribution factors for each panel.

The analyses of the four in-service bridges illustrated that the decks of these bridges behaved as a single panel. The single-deck panel behavior of the in-service bridges could result from the effect of the stiffener beams and the swelling of the deck panels. Based on the analytical and in-service bridge results, the 2004 AASHTO LRFD live load distribution provisions for longitudinal glued-laminated timber bridges are acceptable. This was observed for both the single and multiple lane loading conditions.

Recommendations

Based on the analytical finite-element results and the comparison of the results above, the following can be recommended:

1. The AASHTO LRFD (2004) equivalent strip-width equations assume that the panelized structure behaves as a single-panel bridge. This assumption appears to be valid based on the performance of the in-service bridges. To assure that the panelized structure performs as a single panel, additional research should be performed on the panel-to-panel connections.
2. For newly constructed longitudinal glued-laminated timber deck bridges, their behavior will be similar to the laboratory bridge analyzed in this report. If the panel-to-panel connection is not investigated, as recommended above, one should consider using the AASHTO Standard Specification load distribution factors for design.
3. The effects of edge stiffening were observed at the in-service bridges. However, further study of the curb and guardrail should be conducted to aid in better understanding the edge-stiffening effects.

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