Load Distribution Criteria for Glued-Laminated Longitudinal Timber Deck Highway Bridges

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ABSTRACT

In recent years, changes in the load distribution criteria for highway bridges have been limited primarily to steel and concrete bridges, and only a few changes have occurred in timber bridge criteria. Several years ago, the AASHTO Bridge Committee approved the inclusion of criteria for glued-laminated stringer bridges. However, the glued-laminated, longitudinal, timber-deck bridge developed in recent years has been subject to code specifications that do not reflect the favorable load distribution characteristics of the bridge. Sufficient test data now exist to verify the distribution behavior. The purpose of this study was to develop criteria that more accurately represent the bridge behavior. The study was conducted in three phases. Phase One was a literature review of both analytical and experimental investigations of load distribution in timber-deck bridges. Phase Two analytically investigated the distribution characteristics of a broad range of glued-laminated, longitudinal, timber-deck bridges. The analytical procedure used was verified by comparison of theoretical results with results from actual field tests. In Phase Three, criteria for inclusion in the appropriate sections of the AASHTO Bridge Specifications were developed and have been approved by the AASHTO Bridge Committee. The new criteria more adequately reflect the load distribution of the longitudinal deck bridge.

During the last 25 years, many studies have been conducted to develop new or improved criteria for the distribution of wheel loads on highway bridges. These studies have been used to provide new criteria for improved and different types of construction and have resulted in a number of changes in existing criteria for steel and concrete bridges. For many years, the design criteria for timber bridges remained unchanged in the AASHTO Standard Specifications for Highway Bridges (1). Research in the area of timber bridge structures has led to recent changes in some sections of the specifications. Studies such as the one conducted by Iowa State University (ISU) for the American Institute of Timber Construction (AITC) in 1979 showed the benefits derived from glued-laminated bridge members. In this study, which was conducted on glued-laminated timber bridges with longitudinal stringers, load distribution criteria for the stringers were developed that showed improved distribution behavior over previous criteria. A study by McCutcheon et al. (2) described a design technique for the glued-laminated panels supported by stringers. As happened with the study by Sanders, AASHTO adopted McCutcheon's design procedure and thus included it in the bridge specifications.

The recent development of another type of timber bridge has created the need for additional design criteria. This glued-laminated longitudinal timber-deck panel bridge requires no stringers and can span distances approaching 40 ft. The system consists of panels approximately 48 in. wide, connected by stiffener beams placed transverse to the direction of the span (see Figure 1). Multiple panels are arranged to provide the required roadway width. No positive shear transfer device (e.g., dowels) except

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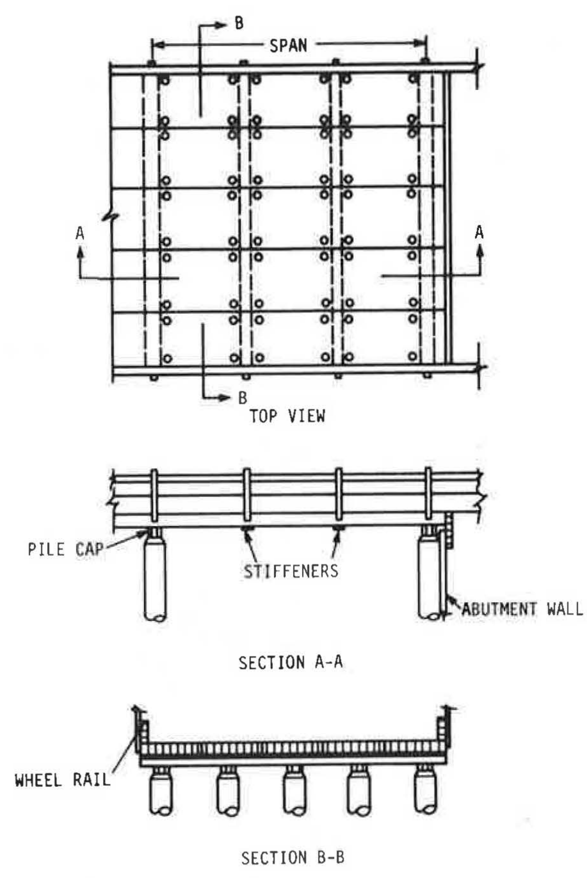


FIGURE 1 Glued laminated longitudinal timber deck highway bridge.

the stiffener beam is normally provided between panels. Presently, there are no clear design criteria for this type of bridge. Over the past few years, research by private industry, government laboratories, and universities has been conducted on this bridge type. With these experimental data, Iowa State University (ISU), under the sponsorship of AITC, conducted a study to develop design criteria for live-load distribution that more realistically reflected the behavior of this type of bridge and to standardize the design procedure.

BACKGROUND

Interest in timber construction for short-to medium-span bridges has increased in recent years and has brought new ideas for more economical designs for timber bridges. One such bridge type was developed in the 1970s by Stone at the Weyerhaeuser Company in Tacoma, Washington (unpublished data). The bridge has no stringers and, therefore, is useful in situations where adequate vertical clearance is a problem. Although many of these bridges have been constructed since their development, lack of clear and concise design criteria has probably precluded their use in some instances. The need for such design criteria has been widely known within the timber industry for some time.

The previously mentioned work by Stone involved full-scale bridge tests as well as tests on various components of the bridge to provide insight into bridge behavior. An analytical study was subsequently performed by Evans (unpublished data) using experimental data from the previously described tests to provide insight into a design technique for the bridge.

Although design criteria that are applicable to this bridge type currently appear in the AASHTO specifications, they are not clearly defined. Recent research efforts and experience show that these criteria do not seem to describe the bridge behavior realistically.

OBJECTIVE AND SCOPE

The research program was conducted in three phases. Phase One involved a thorough literature review of all documented research on the behavior of timber bridges in general and, more specifically, on the behavior of timber bridges composed of the glued-laminated longitudinal deck panels. Both analytical studies and field test studies were included in this review. As a result, a method of analysis was selected for the development of the analytical model to be used to study the bridge behavior.

Phase Two involved a survey of standard highway bridges that incorporated the glued-laminated longitudinal deck panels. The survey included discussions with design engineers and manufacturers who had experience in the design and construction of this type of bridge. The intent of the survey was to provide information on the sizes and structural configurations of the longitudinal bridges typically in use. From this information, it was possible to determine ranges of values of the key variables for use in the model study.

Phase Three used information from Phase Two for the development of an analytical model to study load distribution behavior. From the analytical study, design criteria were developed. The criteria use current distribution criteria formats and have been approved for inclusion in the AASHTO specifications (1).

LITERATURE REVIEW

Before an analytical model for studying the load distribution behavior of the bridge was developed, a review of standard highway bridges using glued-laminated longitudinal deck panels was performed. This review determined the practical limitations of the bridge geometrics and quantified the parameters that describe the bridge so that an accurate model of the bridge could be developed.

Glued-Laminated Longitudinal Timber Deck Studies

From the literature review, it was found that typical span lengths range from 9 to 36 ft for both single and multiple spans. The widths of the glued-laminated panels used in the bridges ranged from 44 to 62 in. However, the most common panel widths were between 48 and 52 in. Roadway widths varied from 16 to 36 ft and most commonly had two lanes. Designs for location and size of stiffener beam varied. In most cases, the arrangements and sizes were similar to the recommended values found in the Weyerhaeuser Design Tables (3) for longitudinal deck bridges. Four types (or variations) of stiffener beam connectors are available: aluminum brackets, thru-bolts, steel plates, and C-clips (Figure 2). The most common types of connectors being used are the aluminum brackets and thru-bolts. Four deck thicknesses are typically used: 6 3/4, 8 3/4, 10 3/4, and 12 1/4 in.

In the late 1970s, several studies provided valuable information about longitudinal deck bridge behavior. Evans conducted theoretical studies (unpublished) of single-span, low-profile, glued-laminated decks and stiffened deck panel systems. The studies involved the development of finite element models for use in predicting the behavior of the bridge. The analytical models were validated by comparison with test bridges 24 ft in length having various stiffener beam spacings. The model results compared well with the bridge test results. Two noteworthy conclusions from the study were that "for all cases investigated, a conservative design procedure results when a single standard wheel line is applied to a single panel and that panel is designed as a beam; i.e., the load is assumed to cause symmetric bending" and "present AASHTO code design requirements are overly conservative." In addition to providing information that illustrated the favorable behavior of the bridge with respect to load distribution, the study supported the validity of the finite element approach for modeling this bridge.

Hale performed field tests on both single-span and three-span stiffened longitudinal deck bridges (4-6). The tests determined the behavioral characteristics of various hardware and connectors that are typically used with the bridges. The tests also provided insight into the mechanism of the load transfer that occurs between the adjacent panels through the combined action of the stiffener beams and connectors.

Analytical Studies

Plate theory has been used most often in the modeling of timber bridges. Sanders (7) used orthotropic plate theory in a study of load distribution behavior in bridge stringers. The stringers were composed of steel and timber that supported timber decks. The complete system, including deck and stringers, was assigned appropriate orthotropic properties in the analysis.

McCutcheon (2) also utilized orthotropic plate theory to represent glued-laminated deck panels that

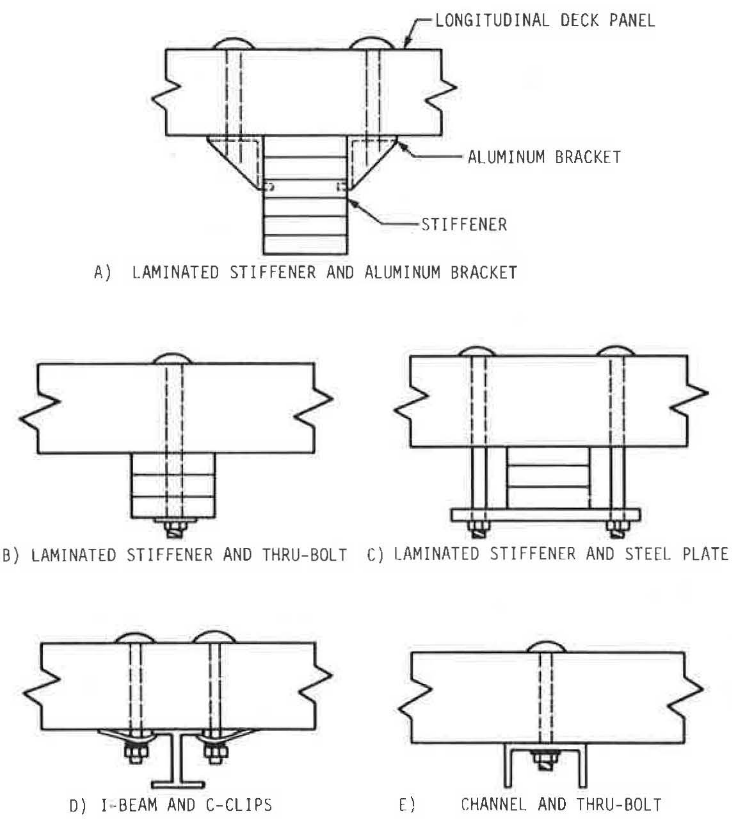


FIGURE 2 Types of transverse stiffener beam connections.

were supported by stringers. The panels were placed transverse to the stringers and connected to each other by dowels. The deck panels were analyzed as infinite strips under uniform rectangular load, where the load represented the wheel of the vehicle.

A third study, by Taylor et al. (8), utilized the same theory for another type of timber deck bridge. The bridge consisted of individual laminates that act together through a system of lateral post-tensioning. Laboratory tests were conducted on specimens that were post-tensioned to different levels of force, and orthotropic properties were determined. It was found that the plate properties were dependent on the level of post-tensioning force.

BEHAVIOR OF GLUED-LAMINATED BRIDGES

Glued-Laminated Bridge Idealization

On the basis of information obtained during Phase Two of this study, an analytical model was developed. This section discusses the model and the investigation of its validity as well as its sensitivity to selected parameters.

Finite element analysis was used in the development of the model for this study; plate theory was used to idealize the glued-laminated deck. A structural analysis program, SAP IV (9), commonly used for analysis of simple and complex structures, was used. For the model, plate and beam elements were used to simulate the various bridge members. Figure 3 shows a sketch of a model used to simulate a four-panel bridge with a single midspan stiffener. The deck panels were modeled as thin-plate elements having both longitudinal and transverse flexural rigidity as well as torsional rigidity. These plates are connected laterally to each other through a system of vertical elements that have axial stiffness only and simulate the deck connectors. The horizontal flexural elements represent the stiffener beams; the panels are connected only at the stiffener beams through the vertical elements. In this manner, lateral shear is transferred to adjacent panels through the interaction of the panel connectors and stiffener beams.

The longitudinal timber deck was analyzed assuming orthotropic plate behavior. The appropriate material matrix coefficients that define the timber panels were based on a report by Bodig et al. (10).

The remaining major components of the glued-laminated longitudinal deck highway bridge are the connectors and stiffener beams. Obviously, in the case of a connector in compression, load would be transferred primarily through the action of bearing between the panel and the stiffener beam. Previous analytical refinement of this interaction by Evans (unpublished data) produced no significant improvement over the model used in this study. Therefore, no attempt was made to refine the connector interaction model previously described.

Validation of Computer Model

The computer model's sensitivity to different magnitudes of the connector stiffness and type of stiffener beam was investigated and results were compared to full-scale test data obtained by Weyerhaeuser on 4-panel bridges made up of 24-ft simple spans (6). The panels were made of Douglas fir.

Two connector types (or variations of) are typically used for the type of bridge in question. In a study by Hale (4), five connector types shown in Figure 2 were tested in the laboratory to determine their structural characteristics. From the test

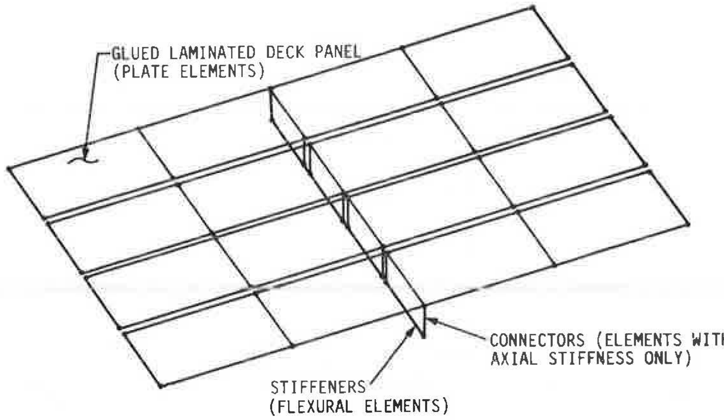


FIGURE 3 Finite element computer model of longitudinal glued-laminated bridge.

data, connector stiffness (K) was determined and subsequently used in the bridge model. The load-deflection data from the study indicated a linear relationship for the thru-bolt connection; its calculated stiffness was 667,000 lb per in. The aluminum bracket connection behaved in a nonlinear manner. In the interest of simplifying the behavior for use in modeling, Evans, in an earlier analytical study (unpublished), estimated the stiffness as 130,000 lb per in. The connector stiffness was varied within a wide range of values, and the model results were compared to data from selected full-scale tests. A typical result of this investigation is shown in Table 1, which shows deflections for a four-panel bridge for different connector stiffness. As may be seen, there is a significant behavioral difference in the bridge for the smaller stiffnesses considered. As noted, the smaller connector stiffness allows greater relative panel movement. In studying the deflection of the panels, it appears reasonable that the smaller stiffnesses also create less load sharing between adjacent panels. In the analytical study performed to determine load distribution behavior, the stiffness corresponding to the aluminum bracket connection was used, thus assuring that the developed design criteria would be conservative.

TABLE 1 Model Sensitivity Study for Variation of Connector Stiffness (K)—Weyerhaeuser Test No. 20[a]

Connector Stiffness (kips/in.)	Midspan Deflections (in.)							
	Panel 1		Panel 2		Panel 3		Panel 4	
	A	B	C	D	E	F	G	H
Experimental value[b]	.56	.45	.41	.37	.27	.16	.12	.04
1.3	.62	.54	.45	.64	.05	.04	.02	0
50	.56	.44	.40	.38	.26	.15	.13	.04
100	.54	.43	.40	.35	.28	.16	.14	.05
130	.54	.42	.40	.35	.29	.16	.15	.05
1000	.53	.41	.40	.32	.31	.17	.16	.06

[a]See Figure 5 for description of test.
[b]This value is based on the actual test; other values are from theoretical study.

The effect of the size (i.e., measure of stiffness) of the stiffener beam on model results was investigated. In the field tests performed by Hale (6), stiffener beams of various sizes and materials were tested including glued-laminated and steel members. However, the literature survey conducted earlier indicated only the use of rectangular glued-laminated stiffener beams, in which the sizes of the beams varied within a small range (in most cases they are approximately 5 to 7 in. deep).

The stiffener moment of inertia (I) was varied to determine the model sensitivity to this variable. Results were compared with data from the Weyerhaeuser full-scale bridge tests. Table 2 shows a typical result of the comparisons and indicates that the relatively large I's have a relatively small effect on the bridge behavior and that the range of I's selected did not have nearly as pronounced an effect on behavior as did the connector stiffnesses that were selected. These results were consistent with those found in an earlier study by Evans (unpublished data). In the analytical study, the use of a rectangular, glued-laminated, 5 x 7 in. stiffener beam was assumed.

TABLE 2 Model Sensitivity Study for Variation of Stiffener Beam Inertia (I)—Weyerhaeuser Test No. 20[a]

Stiffener Beam Inertia (in.^{4})	Midspan Deflections (in.)							
	Panel 1		Panel 2		Panel 3		Panel 4	
	A	B	C	D	E	F	G	H
Experimental value[b]	.56	.45	.41	.37	.27	.16	.12	.04
20	.53	.40	.39	.35	.34	.14	.14	.07
1,000	.53	.41	.41	.31	.30	.17	.17	.06
10,000	.53	.41	.41	.31	.29	.18	.17	.06

[a]See Figure 5 for description of test.
[b]This value is based on actual test; other values are from theoretical study.

After developing the finite element model and studying the sensitivity of various bridge parameters on its performance, a validation of the model using full-scale test data was performed. Although the sensitivity study discussed in the previous section also served to validate the model through comparisons made with full-scale test data, this section involved validations made using only the parameter values that were selected for use in the analytical study for load distribution. A typical comparison of results is shown in Figure 4. The results were equally good in all comparisons.

Although analytical model validation was made with bridges constructed of Douglas fir, the distri-

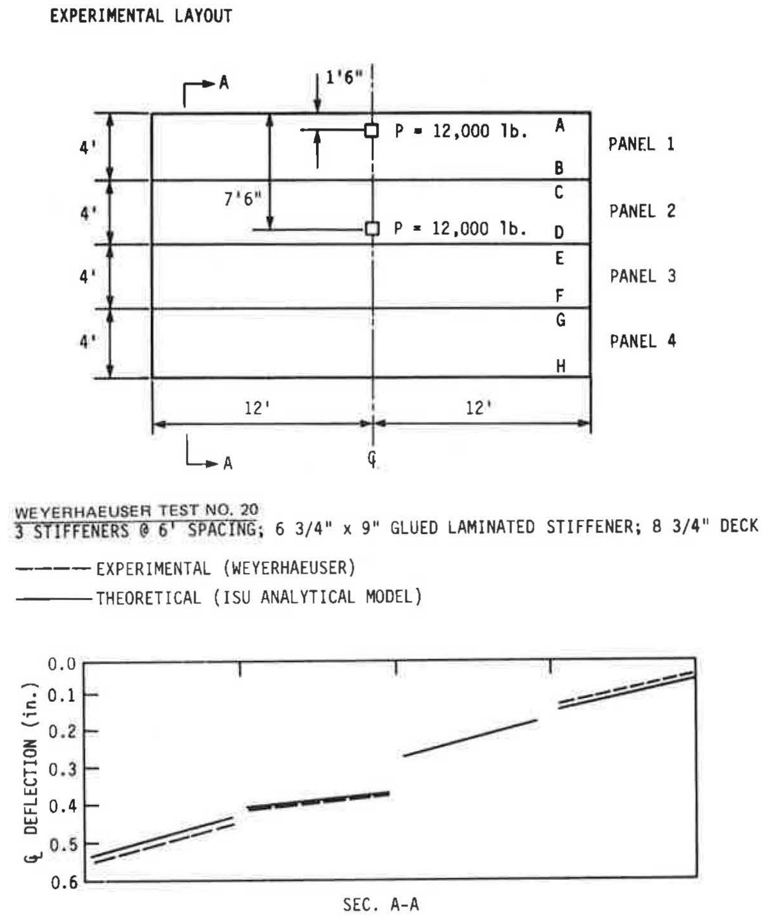


FIGURE 4 Comparison of theoretical data with experimental data.

bution criteria should be equally applicable to bridges constructed of southern pine. On the basis of results of the sensitivity studies, it is anticipated that load distribution behavior will be slightly more favorable for the bridge constructed of southern pine. Correspondingly, the panel deflections would be slightly greater.

ANALYTICAL STUDY

Deflection of Parameters

Once the finite element model was validated, Phase Three was initiated. This phase involved the use of an analytical study to quantify the load distribution behavior of the longitudinal deck bridges. On the basis of previous survey information of typical bridge sizes, a parameter program was set up that covered the range of values for the critical variables that describe the bridge behavior. Initially, a much fuller complement of simulations considering additional span lengths was considered for this study. However, after studying the results from the simulations of the lower and upper ranges of span length, and noting the similarity to the results obtained by Evans (unpublished data), it was decided that considerably fewer simulations were required to determine the bridge behavior.

The parametric program setup consisted of bridges with span lengths from 9 to 33 ft, roadway widths from 16 to 40 ft, deck thicknesses from 6 3/4 to 12 1/4 in., and various stiffener arrangements (see Table 3). These span lengths were initially applied to the study of single spans only; however, multiple spans were later considered. The loadings were placed for maximum effects for both the concentric and eccentric loading conditions. One-, two-, and three-lane roadways that were investigated are illustrated in Figure 5. The loadings were based on AASHTO standard specifications for critical positioning of traffic lanes.

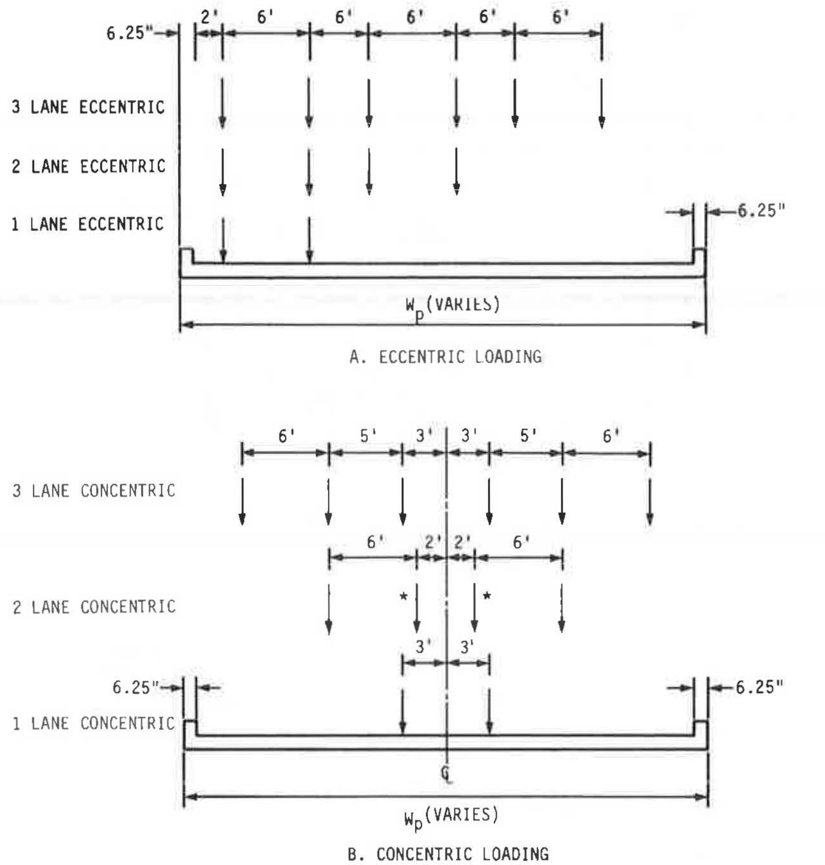
TABLE 3 Parameter Program for Single-Span Bridges

Bridge Span Length (ft)	Number of Stiffener Beams	Panel Thickness (in.)	Number of 48-in. Wide Panels[a]
9	1	6 3/4	4,7,8,9,10
15	2	8 3/4	4,7,8,9,10
21	3, 2	8 3/4, 10 3/4	4,7,8,9,10
27	3, 2	10 3/4, 12 1/4	4,7,8,9,10
33	5	12 1/4	4,7,8,9,10

[a]Bridges with 4 panels simulate single-lane roadways. Bridges with 7,8,9 panels simulate two-lane roadways. Bridges with 10 panels simulate three-lane roadways.

Results of Analytical Study

The current AASHTO format for load distribution, which uses a distribution factor (D) to represent load distribution behavior, was selected for this study because of its simplicity for use by the designer as well as for its convenience in representing distribution characteristics. A wheel-load fraction (W_p/D) was computed for each load case and



*When this load was located at a panel joint, the load was assumed to be carried equally by adjacent panels.

FIGURE 5 Loading cases considered.

bridge configuration where the term W_p represents the panel width. The distribution factor (D) is determined by comparison of the panel moment in the most heavily loaded panel (maximum moment) for a given loading condition with the average moment in the panels.

Thus,

D = bridge width/(M)(number of wheels for loading considered)

where M is the maximum panel moment/average moment in panels. A heavy concentration of moment in a panel would yield a large value for M and a corresponding small value for D. Therefore, a small D value represents poorer load distribution behavior.

The results of the analytical study with regard to single-span distribution of moment are shown in Table 4. As noted in the table, for single-lane bridges (4 panels), the critical loading case was obtained when eccentric loading was acting, whereas concentric loading gave critical values for D for multiple-lane bridges (7, 8, 9, and 10 panels). It is further noted that the width of the roadway had little effect on the load distribution behavior for multiple-lane roadways. The tendency to transfer lateral load was dissipated within the same distance regardless of the number of panels making up the bridge.

A limited number of two equal-span configurations were also investigated. The comprehensive study conducted for single-span distribution behavior provided adequate information to enable this limited study to predict multiple-span behavior. The lateral load distribution tendencies were similar to those for the single spans as shown in Table 5, with the multiple-span systems showing slightly worse behavior in most cases. The stiffer system caused by the continuity is the apparent cause of this behavior.

TABLE 4 Distribution Factors for Single-Span Moment (values shown are D in the Wheel Fraction W_p/D)

Number of Panels	Load Cases	Span Length in ft. (No. of Stiffeners)						
		9 (1)	15 (2)	21 (3)	21 (2)	27 (3)	27 (2)	33 (5)
4	1C[a]	5.21	6.07	7.04	6.36	–[c]	–	6.93
	1E[b]	4.84	5.18	5.59	5.38	5.69	5.58	6.00
7	1C	5.83	7.22	8.60	7.78	–	–	9.90
	2C	4.21	4.56	5.12	4.82	5.20	5.07	5.55
	1E	4.84	5.24	–	5.40	5.70	5.62	6.14
	2E	4.91	5.28	5.53	5.35	–	5.41	5.58
8	1C	5.33	6.40	–	6.92	–	–	–
	2C	4.64	4.92	5.45	5.07	5.30	5.27	5.91
	1E	4.85	5.23	5.69	5.40	5.59	5.62	6.15
	2E	4.91	5.28	5.52	5.35	–	5.40	5.57
9	1C	5.79	7.25	–	7.81	–	–	–
	2C	4.21	4.56	5.17	4.83	5.10	5.12	5.76
	1E	4.84	5.22	5.67	5.41	5.82	–	6.16
	2E	4.91	5.29	5.52	5.35	–	5.41	5.57
10	1C	5.33	6.41	–	6.92	–	–	–
	2C	4.65	4.92	5.48	5.08	5.22	5.29	6.02
	3C	4.56	4.95	–	5.10	–	–	–
	1E	4.84	5.23	–	5.41	5.65	5.62	6.20
	2E	4.91	5.28	5.53	5.36	5.52	5.41	–
	3E	4.91	5.29	5.36	5.36	–	5.36	5.60

[a]C = concentric load case.
[b]E = eccentric load case.
[c]Load case not reviewed.

The effect of distribution on shear was also con-

TABLE 5 Summary of Critical Distribution Factors for Moment and Shear

Span Type (Internal Force)	Span Length (ft) 9	15	21	27	33
Single Span (Moment)	4.24	4.56	4.82	5.07	5.55
Multiple Span (Moment)	4.19	4.50	4.90	5.13	5.30
Single Span (Shear)	4.24	4.68	5.08	5.42	6.13
Multiple Span (Shear)	4.13	4.27	4.54	4.90	5.05

sidered by calculating the reaction at the supports. The load cases considered were the same as those used for the lateral load moment distribution study. Values of D for shear were determined in a manner similar to those for moment (i.e., by comparing the maximum shear in the most heavily loaded panel for a given loading condition with the average shear in the panels). As noted for the longer-span lengths, the behavior was considerably better for shear than for moment when considering the single-span bridges. However, shear in the multiple span cases exhibited different behavior than this as shown in Table 5. Because these distribution factors were similar to those found for moment, and in the interest of simplifying the shear criteria, a decision was made to apply the moment distribution factors in the development of all shear criteria.

Figures 6 and 7 contain plots of the critical distribution values (D) versus the span length (L). As noted in the figures, the distribution behavior improves as span length increases. This behavior may be explained by considering the mechanics of load transfer performed by the various components of the longitudinal bridges. As the span length increases, the panel flexural stiffness decreases. This decreased stiffness causes a corresponding increase in the panel deflections, which causes increased interaction between the stiffener beams and adjacent panels. The cause of poorer distribution for the multiple span may also be attributed to the same factors, in that the multiple-span system is stiffer than the singlespan system of corresponding length. Although the study by Evans (unpublished data) attempted to quantify the bridge behavior in a different manner than that presented here, the basic results were similar to those found in this study. In Evans' study, the same trend toward improved distribution behavior with corresponding increase in span length was found.

DEVELOPMENT OF LOAD DISTRIBUTION CRITERIA

The proposed distribution criteria submitted to AASHTO, which appear in an appendix to this paper, were based on the results of the analytical study

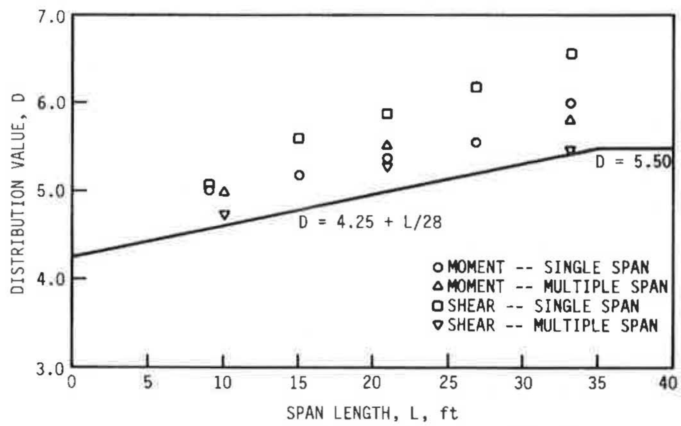


FIGURE 6 Plot of critical D values for all cases considered in parameter study for single-lane bridges.

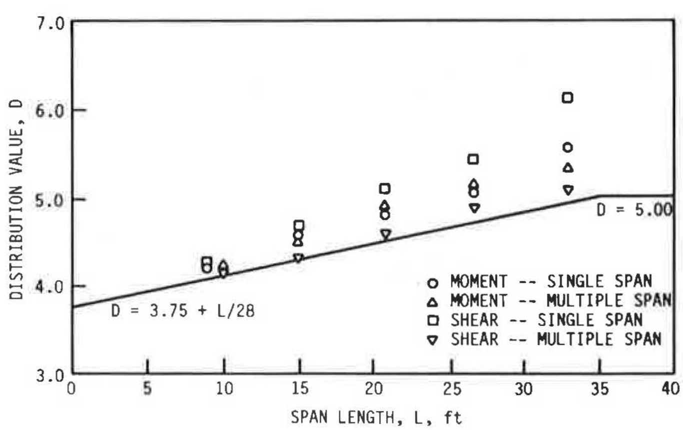


FIGURE 7 Plot of critical D values for all cases considered in parameter study for multiple-lane bridges.

just summarized. In this section, the previously mentioned criteria are developed and the effect of various parameters on their prediction performance are discussed. A design example may be found in a report by Sanders et al. (11).

Proposed Design Equation for Panels

As previously mentioned, the present AASHTO format for load distribution that incorporates a wheel fraction, W_p/D, to indicate wheel load percentage acting on a load carrying member, was also adopted for this study. As noted in the design equation in the Appendix, the distribution factor (D) is represented by a linear equation that includes the span length of the bridge. The equation was obtained from a plot of all of the critical D's for both shear and moment as a function of span length (see Figures 6 and 7). A somewhat arbitrary line (in that no set confidence limit was used considering all data points) was then drawn to represent a conservative relationship for D. This line represented adequate conservatism for design in lieu of the conservatism of the model and because the values plotted in Figures 6 and 7 were the critical values. The numerator term in the design equation (W_p) is the actual panel width and may vary. A discussion of the effect of varying panel widths is found in a later section of this paper.

Once the wheel fraction is determined, the panel is designed for the appropriate partial wheel load based on symmetrical bending. The full panel dimensions are used in determining the section properties.

Additional Design Considerations

In the proposed criteria (see Appendix), two additional design considerations are presented: deflection prediction and stiffener beam arrangement. The analytical study revealed that the maximum deflection in the bridge can be predicted by applying to a panel the moment wheel load fraction. The desired deflection is then calculated using conventional elastic analysis, assuming a symmetrically loaded beam. The deflections determined by this method agreed closely with the analytical results; comparisons of these deflections are shown in Tables 6 and 7. It should be noted that the values shown for the longer spans do not necessarily represent the absolute maximum deflection in the bridge. The deflections shown are due to the loads used only in the distribution study, which, for single spans, represent loads placed at midspan.

An empirical approach is used in the stiffener beam design. From the distribution study based on the beam spacings shown in Table 3, an investigation was made into the effects of varying the spacing. From these results, it was concluded that the

TABLE 6 Comparison of Theoretical Deflections with Calculated Deflections—Single Span

Span Length (ft)	Panel Thickness (in.)	Wheel Load Fraction	Theoretical Deflection (in.)	Deflection Based on Proposed Criteria[a] (in.)
9	6.75	0.982	0.16	0.16
15	8.75	0.933	0.31	0.33
21	10.75	0.889	0.43	0.47
27	12.25	0.848	0.59	0.64
33	12.25	0.812	0.97	1.11

[a]Deflection based on $\Delta = PL^3/48EI$
where E = 2050 ksi, P = 16^k, and Panel Width (W_p) = 4 ft.

TABLE 7 Comparison of Theoretical Deflections with Calculated Deflections—Multiple Span

Span Length (ft)	Panel Thickness (in.)	Wheel Load Fraction	Theoretical Deflection (in.)	Deflection Based on Proposed Criteria[a] (in.)
10	6.75	0.974	0.16	0.16
15	8.75	0.933	0.23	0.24
21	8.75	0.899	0.57	0.62
27	10.75	0.848	0.62	0.68
33	12.25	0.812	0.74	0.80

[a]Deflection based on $\Delta = 3\,PL^3/200EI$
where E = 2050 ksi, P = 16^k, and Panel Width (W_p) = 4 ft.

addition of beams over those shown in Table 3 had an insignificant effect on the distribution. In many cases, one stiffener beam at midspan gave results as favorable for midspan lateral distribution as for placing beams at the one-quarter points. It was definitely advantageous, from a lateral distribution standpoint, to have at least a midspan stiffener rather than stiffener beams at one-third points. On the basis of this information, the criteria specify that a beam shall be placed at midspan, with any additional beams spaced at intervals not to exceed 10 ft. Consideration of the relative panel deflection is left to the discretion of the design engineer.

It is also recommended that the stiffness factor, EI, of the stiffener beam shall be no less than 80,000 kips per in.². This value is representative of the stiffness of the most commonly used stiffener beams found in the literature survey. Because the analytical study showed that the size of the stiffener beam had insignificant effects on distribution of beams within a practical size range, however, this criterion is appropriate.

Effect of Panel Width and Stiffener Beam Spacing

After the design criteria were developed, an investigation was made of the effect that a practical range of parameter values might have on the design criteria prediction. Two parameters were considered: panel width and stiffener beam spacing. Initially, the panel width was thought to be a key factor in load distribution behavior because, as found in the model study, a major reason for differences in distribution was the change in relative position of wheel loads with respect to the panel edge. In other words, the lateral load position on a panel affects the interaction that the loaded panel has with the stiffener beams, which consequently affects the distribution of the load to adjacent panels. Subsequent sensitivity studies considering varying panel widths within practical ranges, however, showed only slight changes in distribution. The range of panel widths considered for the analytical study was 42 to 54 in. It was found that the distribution factor (D) was slightly larger for 42-in. panel bridges than for 48-in. panel bridges. Also, the D's for the 54-in. panel bridges were slightly smaller than for the 48-in. panel bridges. As a result of this study, the proposed criteria include the consideration of a panel width within the ranges mentioned above as part of the wheel fraction expression.

As may be seen in the proposed design equation for lateral load distribution, the panel width (W_p) is a variable. The panel is designed for symmetrical bending using the full panel width properties. The wider the panel, the larger the wheel fraction, W_p/D. There is compensation, however, for the design of the wide panel relative to the narrow panel. This is because of the wider panel's larger section modulus (S) and corresponding smaller stress. The result is that even though the two preceding cases result in different wheel fractions, similar flexural stresses are derived.

The stiffener beam arrangement and its effect on distribution behavior was also investigated. The stiffener beam spacing had a definite effect on relative panel displacements. For this reason, before deciding on beam spacing, the designer would certainly want to determine how much differential movement could be tolerated without damage to the deck surfacing. As far as distribution behavior is concerned, a stiffener beam placed at midspan of a single span results in the most favorable behavior. The primary effect of additional stiffener beams is that of limiting the differential movement between panels and, for this reason, the proposed criteria address stiffener-spacing limitations. The addition of stiffener beams beyond limits recommended in the criteria would not significantly improve distribution.

SUMMARY

There has been renewed interest in the use of timber bridges for secondary roadways. A recent development in the timber industry has been the glued-laminated, longitudinal, deck highway bridge. Present design criteria for this bridge are not clearly defined, nor do they truly represent the distribution behavior.

Full-scale tests were performed in the 1970s on the longitudinal deck bridge to study its behavior. Subsequent analytical studies were performed using this test data to study the bridge further in order to provide insight into a possible design technique.

The study presented here was performed in three phases, the ultimate goal of which was the development of load distribution design criteria that would be applicable to the longitudinal deck bridge. The design criteria that have been developed have been submitted to the AASHTO Bridge Subcommittee and approved for inclusion in the Bridge Design Specifications.

The first two phases of the study involved a review of literature and a survey of bridge parameters related to the longitudinal bridge. Parameters were defined from the results of the two phases, and in Phase 3, an analytical model was developed to study the load distribution behavior. The parameters identified in the first two phases of the study were used in the model. The sensitivity of various parameters (connector stiffness, stiffener beam size, etc.) on the bridge behavior was determined by using the analytical model. The analytical model was validated by comparison of its results to full-scale test data obtained by others.

The analytical study considered single- and multiple-lane bridges loaded with standard AASHTO loading. The placement of load was based on critical positioning of traffic lanes defined by AASHTO. The model results indicate that the load distribution behavior is primarily dependent on span length. As the span length increases, the load distribution behavior improves. The criteria contain a design equation that allows a determination to be made of a wheel load fraction that acts on the panel. The panel is then designed for this load on the basis of symmetrical bending.

CONCLUSIONS

Based on the results of this study, the following conclusions can be made:

1. The research contained herein supports the improved distribution criteria over the existing criteria.
2. The size of the stiffener beam within practical size ranges did not significantly affect distribution behavior.
3. The type of connectors typically used have an effect on relative panel displacements, and the thru-bolt connector is recommended.
4. Load distribution behavior at the midspan of single-span bridges is best when at least one stiffener beam is placed at midspan.
5. The lateral load distribution behavior of the bridge improves as span length increases.

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APPENDIX--Proposed Distribution Criteria

The following proposed specifications have been submitted to and approved by the AASHTO Operating Subcommittee on Bridges and Structures:

Change Section 3.25 on "DISTRIBUTION ON WHEEL LOADS ON TIMBER FLOORING" in AASHTO Standard Specifications for Highway Bridges as follows:

1. Add new Section 3.25.3 shown below;
2. Delete reference to glued laminated panels in 3.25.2.2;
3. Change title of 3.25.2 to read "PLANK AND NAIL LAMINATED LONGITUDINAL FLOORING;"
4. Change 3.25.3 to 3.25.4.

3.25.3 LONGITUDINAL GLUED LAMINATED TIMBER DECKS

3.25.3.1 BENDING MOMENT

In calculating bending moments in glued-laminated timber longitudinal decks, no longitudinal distribution of wheel loads shall be assumed. The lateral distribution shall be determined as follows.

The live load bending moment for each panel shall be determined by applying to the panel the fraction of a wheel load determined from the following equations:

TWO OR MORE TRAFFIC LANES

Load Fraction = $W_p/[3.75 + (L/28)]$ or $W_p/5.00$, whichever is greater.

ONE TRAFFIC LANE

Load Fraction = $W_p/[4.25 + (L/28)]$ or $W_p/5.50$, whichever is greater.

where W_p = Width of Panel in ft ($3.5 \leq W_p \leq 4.5$) and L = Length of span for simple-span bridges and the length of the shortest span for continuous bridges in ft.

3.25.3.2 SHEAR

When calculating the end shears and end reactions for each panel, no longitudinal distribution of the wheel loads shall be assumed. The lateral distribution of the wheel load at the supports shall be that determined by the equation:

Wheel Load Fraction per Panel = $W_p/4.00$ but not less than 1.

For wheel loads in other positions on the span, the lateral distribution for shear shall be determined by the method prescribed for moment.

3.25.3.3 DEFLECTIONS

The maximum deflection may be calculated by applying to the panel the wheel load fraction determined by the method prescribed for moment.

3.25.3.4 STIFFENER ARRANGEMENT

The transverse stiffeners shall be adequately attached to each panel, at points near the panel edges, with either steel plates, thru-bolts, C-clips, or aluminum brackets. The stiffener spacing required will depend upon the spacing needed in order to prevent differential panel movement; however, a stiffener shall be placed at midspan with additional stiffeners placed at intervals not to exceed 10 ft. The stiffness factor EI of the stiffeners shall not be less than 80,000 kips per in.[2]

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