LOAD DISTRIBUTION IN GLULAM TIMBER HIGHWAY BRIDGES

W. W. Sanders, Jr.
Professor of Civil Engineering

Final Report Prepared for
American Institute of Timber Construction
February 1980

Engineering Research Institute
Iowa State University
Ames, Iowa 50011

ISU-ERI-A[illegible]
Project 1[illegible]

75 ϕ

TABLE OF CONTENTS

ABSTRACT

Although a number of changes have been made in the load distribution criteria for highway bridges to reflect improved and different types of construction, the changes have been primarily limited to steel and concrete bridges. Even though the glued-laminated (glulam) timber deck is a significant improvement over other timber decks in load distribution, changes have not been made in the criteria to reflect this increase.

The purpose of the study outlined in this report is to develop the needed changes in the criteria. The study was conducted in three phases, which included:

1. review of research, both analytical and experimental, on load distribution in timber deck and other solid deck bridges,
2. selection of an analytical procedure to study the distribution characteristics of a broad range of glulam timber deck bridges and verification of this procedure by comparison with actual field behavior,
3. development of proposed changes in the appropriate sections of the AASHTO Bridge Specifications so that they more adequately indicate the load distribution within bridges with glulam timber decks. A specific proposal is presented.

The appropriate background and development of the proposed criteria are presented. Comparisons are given with load distribution criteria for similar bridge types to support the proposals.

RECOMMENDATION TO AASHTO OPERATING
SUBCOMMITTEE ON BRIDGES AND STRUCTURES

Change Section 3 - Distribution of Loads in AASHTO "Standard Specifications for Highway Bridges" as follows:

Section 1.3.1 - Table 1.3.1(B)

Timber Floor - Add New Subsection

	One Lane	Two or More Lanes
Glued-laminated panels with glued-laminated stringers		
4" thick nominal	S/4.5	S/4.0
6" or more thick nominal	S/6.0	S/5.0
	If S exceeds 6' use footnote[2].	If S exceeds 7.5' use footnote[2].
Glued-laminated panels with steel stringers		
4" thick nominal	S/4.5	S/4.0
6" or more thick nominal	S/5.25	S/4.5
	If S exceeds 5.5' use footnote[2].	If S exceeds 7' use footnote[2].

ACKNOWLEDGMENTS

The research reported herein was conducted by the Engineering Research Institute of Iowa State University under the sponsorship of the American Institute of Timber Construction.

The assistance of Paul T. Nicholas, Director - Engineering Services, American Institute of Timber Construction, is gratefully acknowledged.

The comments of Keith Benthin, Chairman, Technical Committee on Timber Structures, and Veldo Goins, Chairman, Technical Committee on Loads and Load Distribution, AASHTO Operating Subcommittee on Bridges and Structures and members of these Technical Committees were appreciated.

SUMMARY REPORT

Introduction

During the last 20 years a number of 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 the criteria for steel and concrete bridges. However, even though the modern glued-laminated (glulam) deck bridge is a significant improvement over the nail-laminated or plank timber deck bridge, no modifications have been made in the distribution criteria for highway bridges with timber decks.

As a result, the American Institute of Timber Construction provided a grant to Iowa State University to undertake a study of the behavior of glulam bridges. It was the purpose of the study to develop new load distribution criteria that would more realistically reflect the true behavior of either steel beam or glulam timber beam highway bridges with these glulam timber decks. Since the current criteria reflect only the behavior of bridges with nail-laminated or plank decks, it should be expected that the proposed criteria will indicate improved distribution.

This summary report outlines the study, summarizes the current state of knowledge, and includes recommendations for a new distribution criteria for highway bridges with glulam timber panel decks. It is noted that the proposed distribution criteria will be considered at the 1980 Regional Meetings of the AASHTO Operating Subcommittee on Bridges and Structures.

Research Program

The research program was conducted in three phases. In the first phase a thorough review was made of all available research information on the behavior of timber bridges and on the development of load distribution in solid deck timber bridges. This review included those studies which consider analytically the behavior of the bridges [1,5,6,7,12] and those which include field test studies of actual bridges [3,4,5,6]. From this review a method of analysis was selected to be used in the further analytical studies of bridge behavior.

The second phase incorporated a survey of standard highway bridges with glulam decks [2,11]. From this survey ranges of key variables that reflect bridge behavior (W/L, aspect ratio; D_x and D_y, relative longitudinal and transverse stiffness) were selected.

The third phase consisted of an analytical study of the behavior of bridges encompassed in the variable ranges selected. From these results, a simplified procedure, incorporating current distribution criteria format, was developed and is proposed for inclusion in the AASHTO Bridge Specifications [8]. Comparison of the results of the analytical study and the proposed criteria were made with actual test results to confirm their validity. The effect of the proposed criteria on the design of two typical bridges is presented.

Background

In 1965, Iowa State University conducted a comprehensive study [7] of the distribution of wheel loads on highway bridges for the National

Cooperative Highway Research Program (NCHRP). This study included short and medium span bridges of the following types: beam and slab, multi-beam, and concrete box girder. The results of the study included an overall comprehensive review of the distribution criteria for concrete deck bridges and included recommendations for revisions in that criteria in the AASHTO Bridge Specifications [8].

The beam and slab bridges studied were similar in behavior to the timber bridges currently being studied. That is, the bridges were composed of a solid deck with individual supporting stringers. The analytical procedure used in the NCHRP study [7] was based on the orthotropic plate theory. Although several other methods of analysis have been proposed and used to study load distribution behavior of specific bridge types, the plate theory still is a valid procedure for evaluating a broad spectrum of bridges. Thus, it was used subsequently in the analytical phase of this study.

The key parameter reflecting behavior in the orthotropic plate theory is the relative stiffness parameter:

$$\theta = \frac{W}{2L}\sqrt[4]{\frac{D_x}{D_y}}$$

where W = bridge width, L = bridge span, D_x = flexural rigidity per unit width in X-direction, and D_y = flexural rigidity per unit width in y-direction. In effect, D_x is the flexural rigidity of the beams and D_y is the flexural rigidity of the timber deck. The stiffness parameter reflects the effect of the bridge aspect ratio (W/L) and the relative flexural stiffness of the transverse deck and the longitudinal beams (stringers). Consideration is also given in the procedure to the relative torsional stiffness of the elements.

In 1976, Sanders and Elleby conducted another load distribution review as a portion of a study [5,6] on design criteria for theater of operations highway bridges for the U.S. Army. That portion of the research was directed toward the determination of load distribution behavior for military highway bridges with particular emphasis on steel or timber (glulam) beam bridges with glulam decks. As a part of that study, the behavior of timber bridges from field tests of actual bridges was compared to behavior predicted by the previously used orthotropic plate theory. This comparison indicated the validity of the theory. Using that theory, the study indicated that the glulam deck gave significantly improved distribution compared to timber decks considered in the current AASHTO Bridge Specifications [8] and approached that of the concrete deck bridges.

Two other recent studies [1,3] of glulam timber deck bridges also show the better distribution of the glulam decks. Evans [1] conducted a comprehensive analytical study of glulam bridges. Hale [3] conducted a loading test of a full-size glulam timber bridge to determine load distribution. As a result of these studies, Evans [1] stated that "AASHTO Table 1.3.1(B) gives N to be 4.25 for a bridge of dimensions equivalent to that tested by Hale and those analyzed here. In view of the values calculated here, this appears to be overly conservative."

Stone [9,10] continued the previous studies [1,3] and refined the recommendations for changes in the AASHTO distribution criteria as they apply to timber deck bridges. His recommended distributions for glulam panels were:

(Section 1.3.1: Table 1.3.1(B): Timber Floors)

Glulam panel	One Lane	Two or More Lanes
6" nominal (5 1/8 Net)	S/5.0	S/5.0
8" nominal (6 3/4 Net)	S/6.0	S/6.0

Although their study was conducted on a steel beam bridge with large (5" × 10") floor planks (transverse) that served as the deck, Hilton and Ichter [4] found that the current AASHTO load distribution criteria were conservative in all cases for the interior girders. It was recommended that S/5 be used as the distribution factor for the bridge type studied. Since these are not solid decks (but individual planks) it would appear that a glulam (i.e., solid) timber deck of equivalent or greater thickness would have equal or better distribution.

In summary, it can be seen that all studies to date of timber deck bridges confirm the validity of the orthotropic plate theory for analyzing glulam timber bridges and strongly support an improved load distribution criteria for glulam panel deck highway bridges. Distribution factors approaching those of concrete slab on steel beam bridges do not appear unrealistic.

Bridge Parameters

A thorough study (second phase of the project) of the broad spectrum of AITC standard two-lane bridges [2] indicated that θ varied from 0.50 to 1.70 for the complete range of bridges; however, only when the aspect ratio (W/L) was low (i.e., wide/short bridge) did the θ values approach the upper limit. The bridges ranged in span from 24 to 80 ft with roadway widths from 26 to 34 ft. The designs were for both HS20 and HS15 loadings and included glulam decks on either steel girders or glulam girders. The ranges of θ values for the various AITC bridges are given in Table 1. A review of standard all-glulam bridges of the Minnesota Department of Transportation for designs with spans from 34 to 50 ft and roadway widths from 28 to 44 ft confirmed the ranges indicated from the AITC bridges. Additional study of the wider bridges (roadway widths greater than 34 ft) showed that θ values still are limited to about 1.5. Since a possible use of glulam decks is also in the rehabilitation of very narrow two-lane bridges, where roadway widths approach 20 ft, a special set of designs of this bridge configuration was made. The range of θ values for this study is also shown in Table 1.

In summary the θ values generally ranged from:

Glulam Girder - Glulam Deck

HS20 loading	0.4-1.0
HS15 loading	0.5-1.3

Steel Girder - Glulam Deck

HS20 loading	0.6-1.4
HS15 loading	0.7-1.5

A review of four standard Weyerhaeuser all-glulam two-lane bridges [11] showed θ values that ranged only from about 0.60 to 1.00. These bridges had a roadway width of 26 ft and spans from 19 to 60 ft. They were designed for H20 loadings. The θ values are within the ranges shown in Table 1.

Modifications of the AITC bridges were made so that they reflected typical single lane bridges (roadway width 16 ft). In this case the θ values were considerably less since θ is directly affected by the reduced bridge width. The ranges of θ values for these single lane bridges are also shown in Table 1.

Comparison of Theory and Field Test Results

Earlier it was noted that the orthotropic plate theory was selected as the analytical procedure to be used to determine the behavior of a broad range of glulam timber bridges. However, any theory should be verified with the results of field tests.

The orthotropic plate theory was used as the basis for a study of a broad range of glulam U.S. Army bridges [5,6]. As part of that study, Sanders and Elleby compared the findings of field tests of four timber deck / steel beam Army bridges with those from the orthotropic plate theory. This comparison demonstrated that the theory did favorably predict behavior if a torsional stiffness parameter (α) of 0.16 was used.

Since this study was also concerned with an all-glulam system (i.e., glulam panel deck / glulam stringers), a further verification of the validity of the orthotropic plate theory was made on such a

Table 1. Range of θ values for typical bridges.

Bridge Loading	Bridge Width, W 17 ft*	21 ft	27 ft	31 ft	35 ft
Glulam girder - Glulam deck					
HS20	0.35-0.55	0.40-0.70	0.55-0.90	0.60-1.00	0.75-1.10
HS15	0.40-0.70	0.50-0.85	0.65-1.10	0.75-1.25	0.85-1.35
Steel girder - Glulam deck					
HS20	0.50-0.70	0.60-0.90	0.80-1.15	0.90-1.30	1.00-1.50
HS15	0.60-1.80	0.65-1.10	0.85-1.40	1.00-1.60	1.10-1.70

*Single lane bridge; all others are multiple lane bridges (W > 36 ft may be loaded with three lanes).

bridge. In 1975, Hale [3] reported the results of the field test of a 40 ft span, two-lane panelized timber bridge. The bridge was constructed of 6 3/4 in. (8 in. nominal) glulam deck panels supported by five 8 3/4 in. × 43 1/2 in. glulam stringers spaced at 6 ft 4 in. centers. The span was 40 ft with an overall width of 29 ft 4 in. (roadway of 28 ft). The bridge was designed to AASHTO Standards [8] for an HS20 loading.

Loading the bridge with a simulated HS20 loading (single lane), Hale found that the heaviest loaded stringer carried about 36% of the total load. Using the same loading pattern and the appropriate stiffness parameters (α = 0.86, θ = 0.16) the orthotropic plate theory predicted 37% in the critical stringer.

Although field test data are somewhat limited, the results are sufficient to allow verification of the theory.

Analytical Study

As the final phase, a comprehensive analytical study of load distribution in bridges with critical values of the variables listed below was conducted. From previous data, it can be seen that the range of variables adequately covers the normal glulam timber bridge. The bridges were loaded with both single and multiple lanes of loading. The loadings were placed for maximum effect in both the central loading and eccentric loading condition for one- or two-lane bridges as shown in Fig. 1. In several cases three lanes were possible and a similar approach was used. These loadings reflect the new standards for number and position of lanes adopted by AASHTO [8] which call for critically positioning 12 ft lanes. The combination of variables was selected considering the designs that might be expected in actual practice:

θ = 0.25-1.50

W_R = 20-40 ft for multiple-lane bridges and 16 ft for single-lane bridges

L = 24-80 ft

(W_R = Roadway width)

In order to compare the results of the study with the current AASHTO distribution criteria, the value of D, the distribution factor, from the wheel load fraction formula, S/D, was determined for each case studied. D is determined from the comparison of the stringer moment in the heaviest

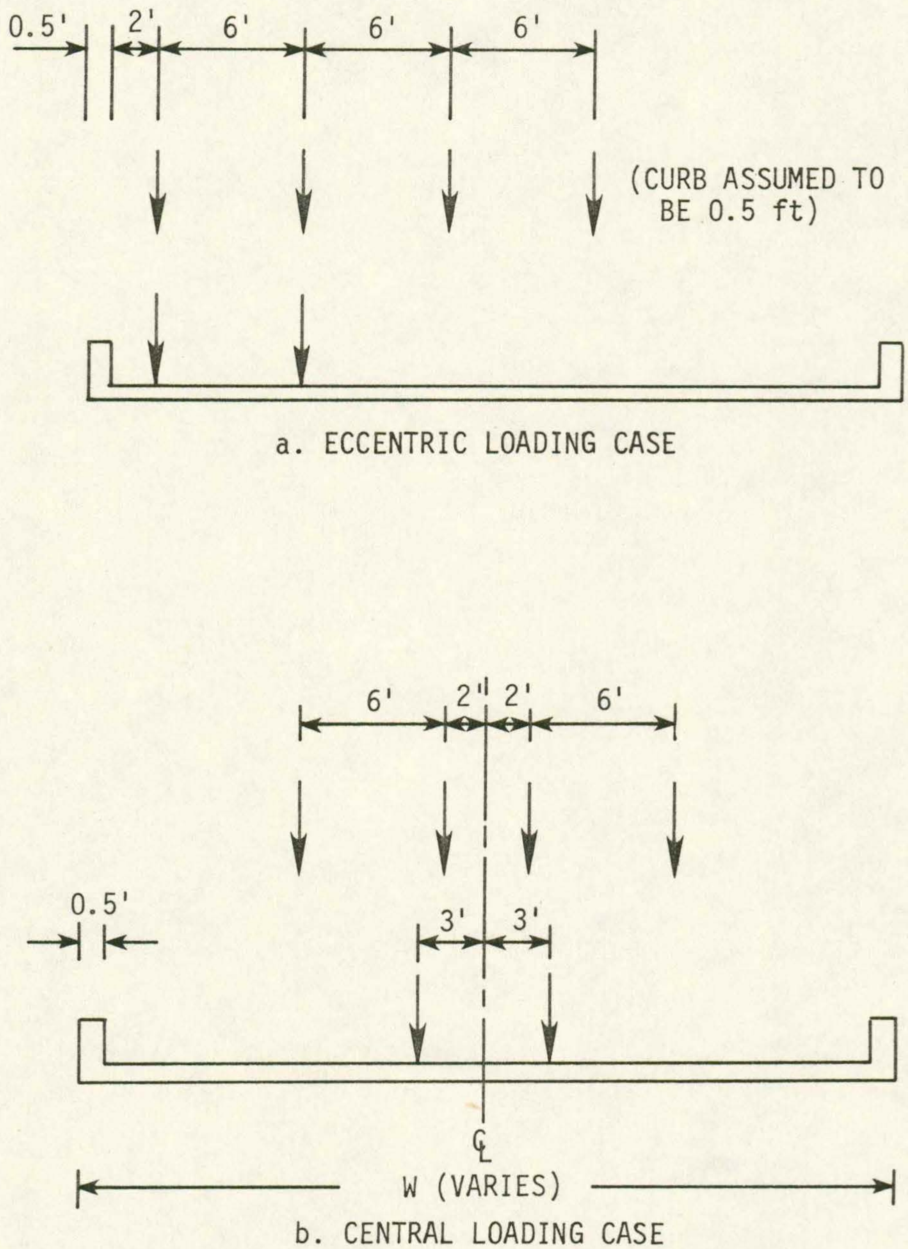

Fig. 1. Loading cases considered for two-lane bridges.

loaded interior stringer (maximum moment) for the loading condition to the average stringer moment. Thus,

$$M = \frac{\text{Maximum interior stringer moment}}{\text{Average stringer moment}}$$

and

$$D = \frac{\text{Bridge width}}{M\ (\text{Number of wheels})}$$

The results of the analytical study using the range of variables shown above are presented in Table 2 for multiple lane bridges and Table 3 for single lane bridges.

For multiple lane bridges (Table 2), in every case the loading of all lanes gave the critical value of D. In most cases, the central loading case controlled. The values of D, for the full range of θ and W values studied, varied from about 4.5 to 5.8. The lower values occurred at high θ values for the narrower bridges. It should be noted also that θ values larger than 1 are typical only of bridges with glulam deck panels <u>and</u> steel stringers. For all-glulam timber bridges (i.e., glulam deck and stringers) the values of θ range only between 0.4 and 1.3. If the limits of θ noted in Table 1 are considered when reviewing the load distribution factors, the D values ranged only from 4.9 to 5.8 for multiple lane bridges. The lower values of D actually would be limited to about 5.0 since an improved distribution criteria would result in slightly smaller stringers and lower θ values in newly designed bridges.

Table 3. Load distribution factors - single lane bridges (values shown are D in the fraction S/D for interior stringers).

	Bridge Width, W	
	17 ft	
θ	C*	E*
0.25	8.19	6.83
0.50	7.87	6.12
0.75	6.28	5.90

In all cases, $\alpha = 0.16$.

*C = central loading case; E = eccentric loading case.

In summary, for bridges designed for multiple lanes of loading, the values of D for interior stringers ranged as noted below for bridges with glulam timber deck panels:

With glulam timber stringers: D = 5.0-5.8

With steel stringers: D = 4.4-5.3.

The AASHTO Bridge Specifications [8] also consider bridges designed for one lane of traffic. Since these bridges would be significantly narrower than those noted earlier (two lanes), it should be expected that the range of θ values would be lower and that D values would be higher. The values of θ range from about 0.3 to 0.7 for all-glulam bridges and from about 0.5 to 0.8 for bridges with glulam decks and steel stringers.

Table 2. Load distribtuion factors - multiple land bridges (values shown are D in the fraction S/D for interior stringers).

	Bridge width, W									
	21 ft		27 ft		31 ft		35 ft		41 ft	
θ	C*	E*	C	E	C	E	C	E	C	E
0.50	5.49 (8.51)†	5.49 (5.71)	5.91 (10.35)	5.78 (7.15)	6.50 (11.46)	5.58 (7.75)	7.11 (12.55)	5.83 (8.40)	6.28 (14.24)	5.64 (8.39)
0.75	5.14 (7.32)	5.14 (5.25)	5.32 (8.62)	5.88 (6.65)	5.68 (9.64)	5.65 (7.08)	6.69 (10.18)	5.68 (7.57)	5.86 (11.33)	5.62 (7.32)
1.00	4.72 (6.31)	4.72 (5.06)	4.88 (7.45)	6.09 (6.41)	5.10 (8.13)	5.74 (6.71)	5.40 (8.64)	5.80 (7.12)	5.54 (9.49)	5.60 (6.41)
1.25			4.63 (6.75)	6.34 (6.28)	4.76 (7.29)	5.87 (6.60)	4.98 (7.70)	5.87 (6.80)	5.38 (8.35)	5.69 (6.41)
1.50			4.46 (6.30)	6.59 (6.20)	4.55 (6.72)	6.05 (6.28)	4.72 (7.06)	5.99 (6.54)	4.90 (7.56)	5.69 (6.15)

In all cases, $\alpha = 0.16$

*C = central loading case; E = eccentric loading case

†Values shown in parentheses represent one lane loaded. Other values are for two lanes or three lanes loaded, whichever is critical.

The values of the distribution factor, D, can be determined from a review of Table 3. The one-lane loading shown in Fig. 1 is also the loading that would be placed on a single lane bridge. If the range of θ values noted in Table 1 is considered it can be seen that D ranges from 5.9 to 8.0 with most values falling well above 6.0. If the revised criteria with better distribution is considered, a value of 6.0 would be a conservative minimum.

Development of Proposed Distribution Criteria

Using the results of the study just summarized, along with a review of the types of glulam deck bridges, the following distribution factors for interior stringers were developed for design usage. The value selected was based on the minimum value found in the analytical study for each stringer type (glulam timber or steel) for bridges with multiple lanes.

For multiple lane bridges, the distribution factors recommended are:

S/5.0 for glulam timber stringers

S/4.5 for steel stringers.

For single lane bridges, the recommended factors are:

S/6.0 for glulam timber stringers

S/5.25 for steel stringers.

The values listed above are for decks wtih thickness (nominal) larger than 6 in. For decks with 4 in. nominal thickness, no change is recommended.

A specific proposal in a form that can be adopted directly into the AASHTO Bridge Specifications is shown in the preface to this report. It is consistent with the results of the study and current AASHTO criteria for other types of bridges.

The current study also provided D values for exterior stringers. However, since the current criteria for these stringers is based on simple beam loading [8, Art. 1.3.1B], no revision is recommended as this is conservative and consistent for all bridge types.

Comparison With Other Bridge Types

In reviewing the current AASHTO load distribution criteria for other bridge types, it can be seen that the proposed criteria generally provide better distribution than that now specified for plank or strip timber floors and provide similar criteria to that for timber stringers with concrete floors.

As noted earlier, it would be expected that a properly formed solid glulam deck would provide better distribution than that of other timber flooring systems. This results from the improved rigidity of the glulam system. It is recognized, however, that a very thin (4 in.) deck is flexible and may not provide the higher distribution. Thus, for thin decks it has been recommended that the current criteria be retained.

The proposed criteria for all glulam bridges are generally consistent with those currently required for bridges with a concrete floor on timber stringers. It should be noted that since no limit is placed on the thickness of that concrete floor, the criteria should recognize that flexible (thin) concrete floors may be used. Only if a glulam floor is at least 6 in. thick and on stringers of a similar material are similar distributions allowed. If a thin glulam timber deck is used, it is recognized that a similar thin concrete floor may provide better distribution (for multiple lanes, S/4 for timber floor and S/5 for concrete floor). Additionally, the stiffness of the concrete floor at larger beam spacings is recognized in setting the limits [8] (see footnote 2 to Table 1.3.1(B)) when simple beam action is considered. These limits are lower for the glulam deck.

In summary, it is recognized that a properly designed thick concrete floor would provide better distribution than many timber floors. Only when a thicker glulam timber floor (6 in. or more) is used can the same distribution be used, and then the limit of S to which it can be used is less than that for concrete floors on timber stringers. Future research may provide different distribution factors for concrete floors of various thicknesses, but current criteria reflect the fact that no design criteria for the concrete floor are specified.

Effect of Proposed Criteria on Bridge Design

Since changes in criteria for glulam deck bridges with steel stringers are only slight, it is not expected that significant changes will result in these designs. Thus, for example, those bridges currently shown in the AITC standards [2] would still be typical.

However, the improved distribution in all-glulam structures should result in economies in the glulam stringers. Two typical bridges [2] were redesigned and a comparison is shown below:

1. Span = 40 ft   Road width = 34 ft   Deck = 5 1/8 in. net
   Loading = HS15

   Current design: 8 3/4 × 40 1/2 in. girders @ 6 ft 7 in. spacing

   Revised design: 8 3/4 × 37 1/2 in. girders @ 6 ft 7 in. spacing

2. Span = 60 ft   Roadway width = 28 ft   Deck = 6 3/4 in. net
   Loading = HS20

   Current design: 12 1/4 × 51 in. girders @ 6 ft 9 in. spacing

   Revised design: 12 1/4 × 48 in. girders @ 6 ft 9 in. spacing

Future Research

Over the last few years a number of researchers, along with members of the AASHTO Bridge Committee and bridge committees of the Transportation Research Board, have called for an overall review of load distribution in all types of highway bridges. The results of this study further reinforce that view. It can be seen that consideration of factors other than stringer spacing, such as the relative stiffness of the elements, can provide D values significantly higher than the minimums assumed for the simplified AASHTO criteria.

Summary and Conclusions

The purpose of the current research study was to develop more realistic load distribution criteria for glulam timber deck bridges with either glulam timber stringers or steel stringers. Reviews of past studies have indicated that a valid method of analysis for these bridges exists and that the distribution behavior of the glulam deck bridges (particularly if with glulam stringers) is close to that of steel stringer - concrete deck bridges. This similarity in behavior supports the suggestion of a higher distribution factor (D) for glulam deck bridges than now given for timber decks (planks, etc.).

A comprehensive analytical study was conducted and provided distribution factors for a broad range of glulam deck bridges that encompassed those configurations that can be expected in the field. The study resulted in proposals for revisions in load distribution criteria for glulam timber deck bridges. Both single and double lane bridges were considered. These proposals are presented along with supporting information.

The results of the study lead to the following conclusions:

1. The orthotropic plate theory is a valid procedure for the analysis of glulam timber deck bridges.
2. Extensive research studies of all types of timber deck bridges and, in particular, those with glulam decks strongly show that an increase in distribution (i.e., higher D values) is appropriate for glulam timber deck bridges. In a number of cases, the distribution is equivalent to that of concrete decks on steel stringers.
3. A comprehensive analytical study conducted for the research further reinforces the improved distribution for glulam decks

and provides a basis for a proposal for revisions in the AASHTO distribution criteria.

4. The improvement in distribution is particularly significant for bridges with glulam timber decks and glulam timber stringers.

REFERENCES

1. Evans, R. J., "Theoretical Analysis of Panelized Wood Bridges," Unpublished Report, Weyerhaeuser Co., Tacoma, 1976.

2. "Glulam Bridge Systems--Plans & Details," American Institute of Timber Construction, Englewood, Colorado, 1977.

3. Hale, C. ., "Field Test of a 40-ft Span, Two-Lane Weyerhaeuser Panelized Wood Bridge," Report No. RDR-045-1092, Weyerhaeuser Co., Tacoma, May 1975.

4. Hilton, M. H. and Ichter, L. L., "An Investigation of the Load Distribution on a Timber Deck-Steel Girder Bridge," Report 76-R30, Virginia Highway and Transportation Research Council, Charlottesville, December 1975 (Abridged in Transportation Research Record 645, 1977, pg. 20-22).

5. Knab, L. I., Sanders, W. W., et al., "Design Criteria for Theater of Operations Steel Highway Bridges," Volumes I and II, Technical Report M-195. Construction Engineering Research Laboratory, Champaign, Illinois, January 1977.

6. Knab, L. I., Moody, R. C., Sanders, W. W. and Elleby, H. A., "Design Criteria for Theater of Operations Glued-Laminated Timber Highway Bridges," Volumes I and II, Technical Report M-198, Construction Engineering Research Laboratory, Champaign, Illinois, January 1977.

7. Sanders, W. W., and Elleby, H. A., "Distribution of Wheel Loads on Highway Bridges," NCHRP Report 83, National Cooperative Highway Research Program, Washington, D.C. 1970.

8. "Standard Specifications for Highway Bridges," Twelfth Edition, American Association of State Highway and Transportation Officials, Washington, D.C., 1977.

9. Stone, M., "New Concepts for Short Span Panelized Bridge Design of Glulam Timber," Unpublished Report, Weyerhaeuser Co., Tacoma, 1977.

10. Stone, M. F., "New Concept Design for Glulam Timer Bridges," Preprint 3254, American Society of Civil Engineers Spring Convention, Pittsburgh, April 1978.

11. "Typical Glulam Timber Bridges," Unpublished Bridge Standards, Weyerhaeuser Co., Tacoma.

12. Wolford, C. W. and Ganga Roa, H.V.S, "Computer Design of Glued-Laminated Timber Bridge System", Special Research Report - Bridges, Dept. of Civil Engineering, West Virginia University, May 1975 (NTIS PB-247-802).