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Reliability-Based Criteria for Load and Resistance Factor Design Code for Wood Bridges

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Recently AASHTO adopted a load and resistance factor design code for highway bridges. The new code provides a rational basis for the design of steel and concrete structures. However, the calibration was not done for wood bridges. Therefore, there is a need to fill this gap. The development of statistical models for wood bridge structures is discussed. Recent test results provided a considerable amount of new data for sawed wood and glulam components. Statistical methods provide a good tool for development of rational models for loads and resistance. Because of the random nature of load and resistance, reliability is a convenient measure of structural performance that also provides a rational basis for comparison of wood and other structural materials. The results of a recent project that led to development of rational design criteria for wood bridges are presented. The structural reliability of selected wooden bridges designed by the AASHTO codes are determined, and inadequacies in load distribution and material resistance in the current specifications are identified.

The objective of this study is to evaluate the reliability of wood bridges designed according to AASHTO Standard Specifications and the AASHTO Load and Resistance Factor Design (LRFD) Code. This evaluation is based on the reliability index β, the probabilistic measure of the structure's chance to fail. Two factors that have a large influence on β are the models used for load distribution and for structural resistance-the focus of this report.

Two broad types of structures are considered here: stringer bridges, which support a deck spanning either normal to (transverse decks) or parallel to (longitudinal decks) traffic, and deck bridges (Figures 1-3). Stringer bridges with longitudinal decks require transverse floor beams to support the deck and distribute load to longitudinal stringers.

Stringer bridges made of sawed lumber are typically short, spanning to a maximum of about 8 m (25 ft). Readily available stringers are 100 to 150 mm (4 to 6 in.) wide and 300 to 400 mm (12 to 16 in.) deep, and they are usually spaced 400 to 600 mm (16 to 24 in.) on center. Wood species of Douglas fir (DF) and southern pine are common.

Stringers of glulam can be manufactured with much greater depths and thus can span much greater distances and allow wider beam spacings. Spans from 6 to 24 m (20 to 80 ft) are common.

The stringer bridge can be fit with a variety of deck types. These include nail-laminated, spike-laminated, glued-laminated, and plank. Laminated decks on stringer bridges are made of vertical laminations, typically 50 mm (2 in.) thick and 100 to 300 mm (4 to 12 in.) deep, which are joined together by nails, glue, or spikes. These laminations are made into panels, which are usually 900 to 1500 mm (3 to 5 ft) wide. The designer may specify that these panels be interconnected or noninterconnected. Interconnected panels may be secured together by spikes, metal dowels, or stiffener beams to form a continuous deck surface, whereas noninterconnected panels are left independent of one another. Various deck grades are available. Attachment of the deck to stringers is made by nails, spikes, or special fasteners.

Additional systems include wood decks supported by steel stringers and cast-in-place concrete decks supported by wood stringers. However, only all-wood systems common to both the AASHTO standard and AASHTO LRFD are considered here.

Deck bridges can span economically to about 11 m (36 ft) and are 200 to 400 mm (8 to 16 in.) deep. The decks are similar to stringer bridge decks with the addition of two other types: the stress-laminated deck, in which the deck laminations are held together by transverse prestressing, and the continuous nail-lam deck, which is made of a single large panel that is constructed on site.

LOAD MODEL

The live-load model is developed on the basis of actual truck measurements. Extensive weigh-in-motion (WIM) measurements were carried out by researchers at the University of Michigan (*1*). The WIM measurement equipment was not visible to truck drivers, which allow unbiased results to be gathered as they include heavy permit trucks and illegally overloaded vehicles. The study provided statistical data on gross vehicle weights, axle weights, and axle spacings.

From the available data, lane loads were determined based on the method used in calibration of the 1994 LRFD code for steel and concrete structures. Results of traffic simulations indicate that, for interior stringers, the case with two fully correlated side-by-side trucks governs, with each truck equal to the maximum 2-month truck. Actual moments generated are expressed in terms of a bias factor (i.e., multiple) of an HS20 moment, which was found to vary with span length, from a low of 1.23 at 3 m (10 ft) to a high of 1.64 at 24 m (80 ft). The coefficient of variation (COV) similarly varies, from a high of 0.30 at **9** m (30 ft) to a low of 0.164 at 24 m (80 ft).

For decks, live-load consideration is focused on axle weights and wheel loads instead of whole vehicles. From the WIM survey, the mean weight for a two-tire wheel is taken as 100 kN (22 kips), and the COV is 0.25.

The pressure load on the deck from a tire can be estimated by the load divided by the tire contact area. Pezo et al. (*2*) constructed a statistical model to relate the net contact area with the ratio of the wheel load over the tire inflation pressure. The proposed formula for the tire contact area (TCA) in square inches is

$$\text{TCA} = 0.28905 + 1.0627(RA) - 0.00202(RA)^2 \tag{1}$$

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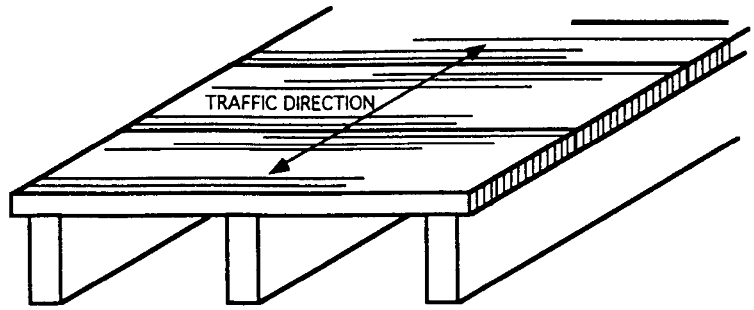
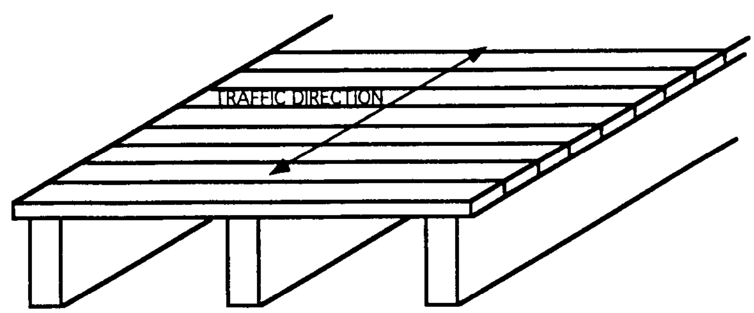


Laminated Deck Type



Plank Deck Type

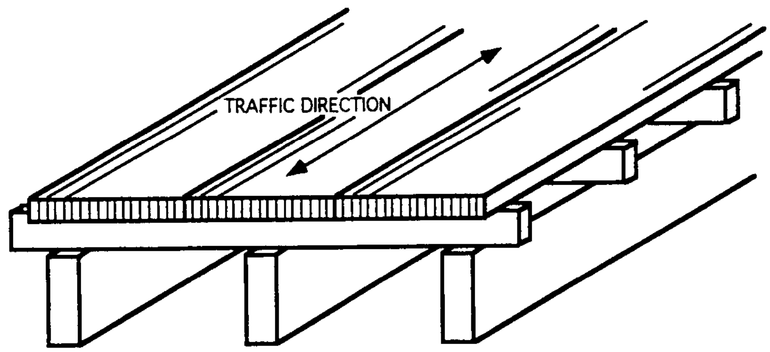
FIGURE 1 Stringer bridge with transverse deck.

where *RA* is the relative area in square inches. *RA* is equal to the wheel load force divided by tire inflation pressure. Based on further analysis of data obtained from surveys performed in Oregon and Texas (*3,4*), 860 MPa (120 psi) is used as the tire inflation pressure. Contact width varies very little regardless of inflation pressure and wheel load level. The typical contact width for a single tire is about 190 mm (7.5 in.). The tire contact area is assumed to be a rectangle with an area obtained from Equation 1 and a contact width of 190 mm (7.5 in.). The contact length can be determined by dividing the contact area by the contact width. With an expected mean maximum tire live load as indicated above (50 kN; 11 kips) distributed over the rectangular contact area, the length is 250 mm (10 in.) in the direction of traffic and the width is 190 mm (7.5 in.) in the other direction, with a resulting pressure load under the tire of 1 MPa (150 psi). The contact area for a two-tire wheel unit weighing 100 kN (22 kips) is then 500×250 mm (20×10 in.), with the 500-mm width resulting from two tire contact areas of 190 mm (7.5 in.) each with a 120-mm (5-in.) gap between, where 500 mm (20 in.) is in the direction of traffic, the typical wheel configuration on trucks.

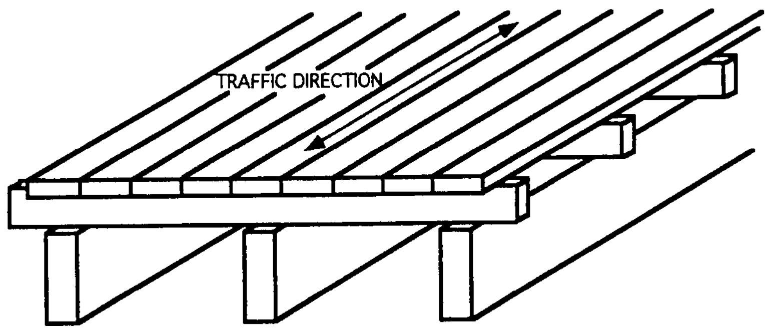
RESISTANCE MODEL

Structural Resistance

Actual force effects in the bridge components were determined by finite element method modeling. By varying mesh density, element type, and element proportions, models were first calibrated to existing experimental data. Spans and spacings were based on typical timber bridge dimensions. Stringer and deck stiffness were computed for typical components designed by the AASHTO LRFD code. Modeled bridges have no skew. are two lanes (about 9 m) wide, and are simple spans. The effects of miscellaneous bridge components, such as railings. diaphragms, and the wearing surface, are excluded. The wheel patch size as indicated above was applied as a pressure load under the area of interest to generate the maximum load effect; placement varied for bridge type and element considered (deck, longitudinal stringer, or transverse floor beam). Decks were modeled with solid elements. and stringers were modeled with beam elements. Standard orthotropic elastic material constants of wood were used in the analysis (5.6). Bridge span, deck type, stringer spacing, deck stiffness, and stringer stiffness were varied to produce a wide range of practical parameters. For sawed lumber stringer bridges with transverse decks. four spans (4.5, 6.0, 7.5, 9.0 m). three stringer spacings (0.4, 0.5, 0.6 m), and four stringer deck stiffness cases were combined to produce 48 models; for glulam stringer bridges with transverse decks, three spans (9, 15.2, 21.2 m), three stringer spacings (1.2, 1.5, 1.8 m), and four stiffness cases were combined to produce 36 models. For stringer bridges with longitudinal decks. three spans (3.6, 11, 18 m), three longitudinal beam spacings (1.2, 1.8, 2.4 m), and six transverse beam spacings (0.3, 0.6, 1.2, 1.8, 2.2, 2.4 m) were combined with eight stiffness combinations (two deck thicknesses, two longitudinal beam sections, and two transverse beam



Laminated Deck Type



Plank Deck Type

FIGURE 2 Stringer bridge with longitudinal deck.

sections) to produce 84 models (not all possible combinations were considered).

Empirical formulas were then written as functions of span, spacing, stiffness of components, and deck type that would predict the distribution factors (for girders) and stresses (for decks) found from finite element method analysis. These formulas formed the basis of the structural resistance model.

Material Resistance

The major parameter that determines the structural performance of wood components is the modulus of rupture (MOR). The statistical model of sawed lumber MOR is based on actual in-grade tests carried out by researchers in Canada (*7*) and test data were processed by Nowak (*8*). Glulam data were provided by the Forest Products Laboratory in Madison, Wisconsin (9). The flatwise use factor, the ratio of MOR for edge-wise and flat-wise loading, is based on the work of Stankiewicz and Nowak (*10*).

RELIABILITY ANALYSIS

The limit state considered is moment capacity. Failure is defined as the state at which the moment due to loads exceeds the moment carrying capacity; that is, stress on the extreme fiber equals or exceeds MOR. Let R represent the resistance (moment carrying capacity) and let Q represent the load effect (total moment applied to the considered element). Then the corresponding limit state function g can be written

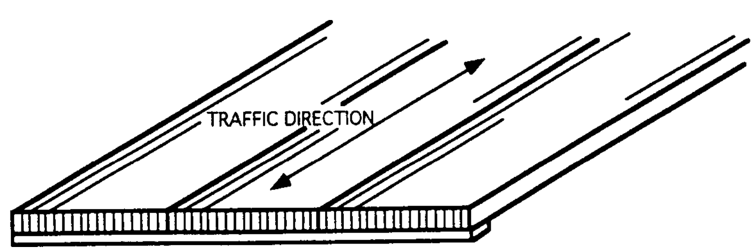


FIGURE 3 Deck bridge.

$$g = R - Q \tag{2}$$

If $g > 0$, the structure is safe; otherwise it fails. The probability of failure P_F is equal to

$$P_F = \text{Prob}(R - Q < 0) = \text{Prob}(g < 0) \tag{3}$$

The reliability index β is defined as a function of P_F:

$$\beta = -\Phi^{-1}(P_F) \tag{4}$$

where Φ^{-1} is the inverse standard normal distribution function.

If both R and Q are lognormal random variables, then β can be approximated (*11*) by

$$\beta = \ln(m_R/m_Q)/(V_R^2 + V_Q^2)^{1/2} \tag{5}$$

where

m_R = mean resistance,
m_Q = mean load,
V_R = COV of R, and
V_Q = COV of Q.

For the higher COVs (greater than 0.20) associated with the loads and resistances of wood bridges, a more exact formulation is needed (*12*) to estimate β:

$$\beta = [\ln(m_R) - 0.5\ln(V_R^2 + 1) - \ln(m_Q) + 0.5\ln(V_Q^2 + 1)]/[\ln(V_R^2 + 1) + \ln(V_Q^2 + 1)]^{1/2} \tag{6}$$

where

m_R = mean resistance,
m_Q = mean load,
V_R = COV of resistance, and
V_Q = COV of load.

The reliability analysis in this study is carried out with the procedure developed for calibration of the AASHTO LRFD. Load and resistance are treated as lognormal random variables, and reliability is measured in terms of the reliability index β. The analysis is performed for wood bridges designed according to the AASHTO standard and AASHTO LRFD. Components are designed for moment resistance only; shear and deflection are not considered. The reliability index is calculated with Equation 6.

Results are presented in Tables 1-4 Deck thickness and stringer sizes listed are nominal. Span is the maximum allowed by code with the given stringer and spacing combination. The laminated decks in Tables 1 and 2 are similar but not identical for both codes; the tables consider a 100-mm nail-laminated deck for AASHTO standard and a 150-mm spike-laminated deck for AASHTO LRFD. as AASHTO standard has no spike-lam provision and AASHTO LRFD has no nail-lam provision. The minimum nominal deck thickness for AASHTO LRFD (excluding planks) is 150 mm. which is thicker than that permitted in the AASHTO standard, although for the given stringer spacing a 100-mm deck is a more likely choice for designers. In Table 2, an impractical longitudinal stringer spacing results if sections smaller than 100 × 200 mm are specified in AASHTO LRFD. Table 3 considers transverse decks only, as there are no provisions for longitudinal glulam decks on transverse floor beams in the 1996 standard code. The deck thickness in Table 3 is 150 mm.

TABLE 1 Reliability Indices for Sawed Lumber Stringer Bridges with Transverse Decks

Stringer	Span (m)		Spacing (m)		Reliability Index, β: Deck		Reliability Index, β: Stringer	
(mm)	Stand.	LRFD	Stand.	LRFD	Stand.	LRFD	Stand.	LRFD
Laminated, Interconnected								
100x350	3.1	2.8	0.40	0.40	5.4	6.1	2.1	2.8
100x400	3.7	3.3	0.40	0.40	5.7	6.2	2.2	3.0
150x400	4.1	4.1	0.61	0.61	4.6	5.8	3.5	4.0
Laminated, Non-interconnected								
100x350	3.1	2.8	0.40	0.40	4.5	5.1	2.1	2.8
100x400	3.7	3.3	0.40	0.40	4.7	5.3	2.2	3.0
150x400	4.1	4.1	0.61	0.61	3.5	4.7	3.5	4.0
100x150 Plank								
100x350	3.2	2.9	0.35	0.35	6.0	6.3	2.3	2.7
100x400	3.3	2.8	0.45	0.45	5.6	5.9	2.3	2.9
150x400	4.8	4.6	0.45	0.45	5.6	5.7	3.7	4.4
100x250 Plank								
100x350	3.2	2.9	0.35	0.35	4.1	4.4	2.3	2.8
100x400	3.1	2.8	0.51	0.45	3.5	4.0	2.2	2.9
150x400	3.9	4.6	0.60	0.45	3.0	3.8	3.6	4.1

TABLE 2 Reliability Indices for Sawed Lumbar Stringer Bridges with Longitudinal Decks

Span (m)	Stringer (cm)		Stringer Spacing (m) Longitudinal		Stringer Spacing (m) Transverse		Reliability Index, β Deck		Reliability Index, β Stringer	
	Stand.	LRFD	Stand.	LRFD	Stand.	LRFD	Stand.	LRFD	Stand.	LRFD
Laminated, Interconnected										
4.5	10x35	10x35	1.47	1.57	1.06	1.06	3.5	5.5	3.8	4.2
4.5	10x30	10x30	1.91	1.97	1.06	1.06	3.4	5.6	3.5	4.1
9.1	10x30	10x30	1.91	1.97	1.06	1.06	3.1	5.3	3.0	3.4
Laminated, Non-Interconnected										
4.5	10x35	15x40	1.7	2.9	0.91	1.8	2.5	2.2	3.6	4.9
4.5	10x30	10x40	1.2	1.7	0.91	1.2	2.7	3.6	3.9	4.3
9.1	10x30	10x40	1.2	1.7	0.91	1.2	2.9	3.9	3.1	3.7
100x250 Plank										
4.5	10x15	10x20	1.0	1.1	0.35	0.45	6.6	7.2	2.8	3.0
9.1	10x15	10x20	1.0	1.1	0.35	0.45	6.7	7.4	3.2	3.4
4.5	10x20	N/A	1.7	N/A	0.35	N/A	6.1	N/A	1.9	N/A
100x300 Plank										
4.5	10x15	10x20	0.86	1.1	0.40	0.45	6.2	6.1	2.5	3.5
9.1	10x15	10x20	0.86	1.1	0.40	0.45	6.3	6.2	2.7	3.6
4.5	10x20	N/A	1.5	N/A	0.35	N/A	6.2	N/A	2.5	N/A

Given the restraints of the sawed lumber stringer sizes, in some cases stringer spacing less than the maximum allowed by code for the given deck type were chosen so the bridge length is a minimum of about 3 m (10 ft) (these cases were as follows: Table 1, all cases for laminated transverse decks and the case of a 100 × 350 mm stringer in transverse plank decks; in Table 3, for the LRFD code in the case of glulam interconnected decks; otherwise, stringer spacing is maximum allowed by code). For longitudinal decks, stringer size refers to transverse floor beams. All applicable adjustment factors provided in the codes were used to determine allowable design stress. Stringers are DF select structural, and decks are DF No. 1. Glulam decks are Type L2 (DF), and glulam stringers are 24F-V4 (DF).

CONCLUSIONS

In general, the results indicate that there are considerable differences in the reliability indices. These differences are attributable to several reasons.

1. There are significant differences in stresses predicted by the codes and from analytical results. The major load distribution effect for wood bridges that the code does not recognize is that a change in stiffness of a component results in a change in load to that component. This effect is accounted for in AASHTO LRFD for load distribution to steel or concrete girders with concrete decks based on the

TABLE 3 Reliability Indices of Glulam Stringer Bridges

Stringer (mm)	Span (m) Stand.	Span (m) LRFD	Spacing (m) Stand.	Spacing (m) LRFD	Reliability Index, β Deck Stand.	Reliability Index, β Deck LRFD	Reliability Index, β Stringer Stand.	Reliability Index, β Stringer LRFD
Interconnected								
150x910	5.9	4.8	2.2	2.2	4.4	5.0	3.2	4.1
150x1370	10.3	7.6	2.2	2.2	4.1	54	2.9	5.0
150x1830	14.3	11.2	2.2	2.2	3.8	4.2	2.8	4.6
Non-interconnected								
150x910	6.9	4.2	1.8	2.6	2.7	2.4	3.3	3.7
150x1370	11.8	7.5	1.8	2.6	2.4	3.2	2.9	4.4
150x1830	16.2	10.3	1.8	2.6	2.1	2.1	2.7	4.4

TABLE 4 Reliability Indices of Deck Bridges

Deck Depth (mm)	Interconnected Deck				Non-Interconnected Deck			
	Span (m)		Reliability Index, β		Span (m)		Reliability Index, β	
	Stand.	LRFD	Stand.	LRFD	Stand.	LRFD	Stand.	LRFD
Nail Laminated								
250	4.1	4.2	5.0	4.9	3.3	4.2	3.2	2.9
300	5.9	5.6	5.1	5.3	4.6	5.6	4.2	3.5
350	7.9	7.1	5.7	6.4	6.2	7.1	5.2	4.6
Glulam								
200	3.6	3.2	3.6	4.3	3.6	3.2	1.4	2.0
250	5.4	4.9	4.6	5.6	5.4	4.9	2.7	3.4
300	7.6	6.6	4.8	5.7	7.6	6.6	3.0	3.8

work of Zokaie et al. (*13*). Such an approach can be fashioned for wood stringers and decks. It should be noted that AASHTO LRFD requires the designer to carry out a refined analysis for wood structures that have component spacing less than 0.91 m (36 in.); there is no simplified distribution formula given. Because deck stresses are sensitive to small differences in even refined analytical approaches, accurate stresses are difficult to predict. This fact, coupled with the added burden of constructing a refined model by the designer, calls for a consistent, simplified approach.

2. Analytical results have shown large differences in the maximum stresses in interconnected and noninterconnected panels. This is primarily due to a significant stress increase when a wheel load is placed next to the edge of an interior noninterconnected panel, which has been found to be as high as 1.8 in some cases. Although these results have not been verified experimentally, they call for further study, as both codes do not adequately account for the magnitude of this stress increase. and AASHTO LRFD makes no stress calculation distinction among deck bridges with different degrees of interconnection between deck panels (e.g., a continuous laminated surface is treated the same as panels joined only by intermediate stiffener beams).

3. Load distribution to plank decks is another area of concern. In Table 1, note the large difference in β values between 100 × 150 mm (4 × 6 in.) and 100 × 250 mm (4 × 10 in.) transverse planks. This is due to the reduction in actual applied force to a 100 × 150 mm compared with a 100 × 250 mm plank. Considering a 250 × 500 mm (10 × 20 in.) wheel load patch, a 100 × 150 mm plank carries about 60 percent of the load of a 100 × 250 mm plank because of its smaller width; although a single 100 × 50 mm plank takes the entire wheel contact length of 250 mm, a single 100 × 150 mm plank can carry only 60 percent of that contact length, the remainder of which is carried by the adjacent plank. In the codes. however, both planks are assumed to take the entire wheel load.

4. The code's treatment of material resistance parameters is overly conservative. Experimental data (*10*) have shown that actual flat-use factors, the ratio of MOR when loaded flatwise to MOR when loaded edgewise, greatly exceed AASHTO Standard (1996) specified values. AASHTO LRFD specifies no flat-use factors (Table 5). Other differences, such as in size-effect factors, have also been found.

TABLE 5 Experimental and AASHTO Fiat-Use Factors

Size, mm (in.)	Experimental	AASHTO 1996
100x150 (4x6)	1.07	1.05
100x200 (4x8)	1.17	1.05
100x250 (4x10)	1.25	1.10
100x300 (4x12)	1.61	1.10

Each of these areas contributes to the inconsistency in code reliability. Adopting changes in the specifications, such as a more reasonable load distribution and analysis model and more accurate flat-use factors, can dramatically improve results and establish code reliability index consistency.

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