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Load and Resistance Factor Calibration for Wood Bridges

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LRFD Calibration For Wood Bridges

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Abstract

The paper presents the calibration procedure and background data for the development of design code provisions for wood bridges. The structural types considered include sawn lumber stringers, glued-laminated girders, and various wood deck types. Load and resistance parameters are treated as random variables, and therefore, the structural performance is measured in terms of the reliability index. The statistical parameters of dead load and live (traffic) load, are based on the results of previous studies. Material resistance is taken from the available test data, which includes consideration of the post-elastic response. The resistance of components and structural systems is based on the available experimental data and finite element analysis results. Statistical parameters of resistance are computed for deck and girder subsystems as well as individual components. The reliability analysis was performed for wood bridges designed according to the AASHTO Standard Specifications and a significant variation in reliability indices was observed. The recommended load and resistance factors are provided that result in consistent levels of reliability at the target levels.

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Introduction

In 1993 AASHTO adopted a new load and resistance factor design (LRFD) code for highway bridges. The new code provides a rational basis for the design of steel and concrete structures. Although wood bridge design was also included in LRFD format, the calibration was not carried out for these structures (*Nowak 1995; 1999*). Therefore, there was a concern about the consistency of the reliability level for wood structures.

Previous studies showed that the reliability index for wood bridge components can be significantly different than for steel or concrete structures *(Nowak 1991)*. The degree of variation for wood properties varies depending on dimensions, load duration, moisture content and other parameters. In case of wood bridges, it is important to consider the structural system or subsystem as well as individual elements/components.

In general, a design code is calibrated by: 1) designing a range of structures according to current code procedures; 2) identifying random variables and developing load and resistance models based on the statistical parameters of actual loads and resistances; 3) choosing an appropriate reliability technique and computing reliability indices for the code-designed structures using the load and resistance models developed; 4) identifying target reliability indices from the results, usually such that the most typical structures represent the target indices; 5) suggesting adjustments to current code design procedures that would minimize variations in reliability index among structural components of a similar type.

The objective of this study is to complete the calibration process and determine appropriate design parameters for wood bridges. This research fills this gap and provides recommendations that result in a consistent level of reliability for wood bridges.

Structural Types Considered

The calibration work is performed for selected representative types of wood bridges. In particular, simple span, two-lane, non-skewed bridges with wooden components of short to medium spans, from 4m to 25m (13 to 80 ft), are considered. In general, there are two types of wood bridges: structures that span by beams (stringers or girders) or structures that span by a deck.

Stringer bridges made of sawn lumber are typically short, spanning to a maximum of about 8m (25'). Readily available sawn lumber stringers are usually from 100 to 150mm (4 to 6") wide and 300 to 400mm (12 to 16") deep, and these sizes often limit spacing to no more than 400 to 600mm (16 to 24") on center. However, the use of greater widths such as 20mm (8") and larger depths may allow stringer spacing to be increased, until ultimately limited by deck capacity. Stringers of glulam can be manufactured with much greater depths and widths, and can thus span much greater distances and allow wider beam spacing. Spans from 6 to 24m (20 to 80') are common.

The stringers support various wood deck types, which may be glued-laminated (glulam), nail-laminated (nail-lam), spike-laminated (spike-lam), plank (4" x 6", 4" x 8", 4" x 10", and 4" x 12"), stress-laminated (stress-lam), and reinforced concrete (non-composite). Laminated decks are made of vertical laminations, typically 50mm (2") thick and 100 to 300mm (4 to 12") deep, which are joined together by nails, glue, spikes, or transversely prestressed. The latter method is typically used for deck rather than stringer bridges, however. Laminations are made into panels that are usually from 900 to 1,500mm (3 to 5') wide. The designer may specify that these panels either be interconnected or non-interconnected (in a direction parallel to the laminations). Interconnected panels may be secured together by spikes, metal dowels, or stiffener beams, to form a continuous deck surface, while non-interconnected panels are left independent of one another, although in some cases the Code requires that transverse stiffener beams be used to provide some continuity. As with stringers, various wood species and commercial grades of deck laminations are available. Attachment of the deck to stringers is made by nails, spikes, or special fasteners. The structures may have decks running either perpendicular or parallel to traffic. Stringer bridges with longitudinal decks require transverse floor-beams to support the deck and distribute load to longitudinal stringers. Diagrams of these structures are presented in Fig. 1-2.

Deck bridges can economically span to about 11m (36'), and are from 200 to 400mm (8 to 16") deep (Fig. 3). The deck types are similar to those of the stringer bridge decks, with the addition of the continuous nail-lam deck, which is made of a single large panel, constructed on site. This deck type, as well as all of the stringer bridge deck types described above, are considered.

Load Models

Dead load typically constitutes from 10-20% of the total load effect on wood bridges. Dead load parameters are taken to be consistent with those used to calibrate the steel and concrete sections of the LRFD Code *(Nowak 1999, 1993).* The considered statistical parameters include the ratio of mean to nominal (design) value, called the bias factor, λ, and coefficient of variation, V, that is the ratio of standard deviation to the mean. For wood and concrete (deck) components, bias factor λ is 1.05 and coefficient of variation V is 0.10; for steel (girders), λ = 1.03 and V = 0.08; and for asphalt, mean thickness is taken as 90mm and V = 0.25. Dead load is taken as normally distributed.

The live load model is based on the available truck survey data as used in the calibration of the AASHTO Code *(Nowak 1999, 1993).* The analysis of live load involves the determination of the load in each lane and load distribution to components. The probabilities of a simultaneous occurrence of more than one truck in adjacent lanes and a multiple truck occurrence in the same lane, were considered with various degrees of correlation between truck weights. For most wood bridges, however, only a single truck per lane needs to be considered, as the typical short spans result in the probability of two trucks in the same lane unlikely or even impossible. The simulations indicated that for bridges with girder spacing of 1.2m - 2.4m (4-8 ft), two fully correlated trucks side-by-side govern. For the maximum 75 year moment, the results of analysis indicated that each truck in this combination is equivalent to the maximum two month truck. That is, considering the various combinations of single and two side-by-side truck weights and probabilities of occurrence for each combination, two side-by-side trucks of equal weight which are each the weight of the maximum single truck expected to pass in a two-month period, govern the load model in the reliability analysis. Bias factors were calculated as the ratio of mean maximum moment and design moment (applied to the entire bridge) specified in the Code, for various time periods. It was found that bias factor varies with span length. For spans up to 30m (100 ft), the results are shown in Figure 4 for 1 year and 75 year periods. The coefficient of variation is shown in Figure 5. Live load is approximately lognormal.

As wood strength is affected by load duration, the live load duration is calculated for various time periods. Three values of the average daily truck traffic (ADTT) are considered: low with ADTT = 500, medium with ADTT = 1,000, and high with ADTT = 3,000. It is assumed that the percentage of the actual heavy trucks (only very heavy vehicles need to be considered) is 20%, and this corresponds to 100, 200, and 600 trucks per day for the three considered traffic volumes, respectively. Note that these are high ADTT values for typical wood bridges, which are usually located on low-volume roads and may experience only a fraction of the traffic volume that highway bridges do. However, as current design procedures stipulate no restriction as to the use of wood bridges with regard to traffic volume, for code calibration purposes it would be unconservative to base load duration on low traffic volume roads only. Considering various span lengths and posted speed limits, it is assumed that the average duration of truck passage is about 1 second. For a typical simple span wood bridge, the load effect (bending moment) gradually increases from zero to maximum at midspan, then reduces back to zero. The actual duration of maximum live load effect is lower than crossing time and, therefore, on average exposure to the maximum live load effect is assumed equal to 0.5 seconds. In most cases, this is a conservative assumption as the effected portion of the influence line for many components of wood bridges is smaller than the whole span length. Therefore, the live load duration (corresponding to very heavy trucks) for 75 year period and for the three considered traffic volumes is:

(a) low ADTT = (100 trucks)(0.5 sec)(365 days)(75 years) = 15 days

(b) medium ADTT = (200 trucks)(0.5 sec)(365 days)(75 years) = 30 days

(c) high ADTT = (600 trucks)(0.5 sec)(365 days)(75 years) = 90 days

Although wood bridges are typically located on low volume roads, in the reliability analysis it is conservatively assumed that the live load duration is 2 months (between medium and high traffic volumes).

For short spans, live load is caused by axle loads or even wheel loads. Therefore, the live load model is determined by variations in wheel load rather than the entire truck or axle. Statistical parameters for wheel load are derived from existing survey data (*Nowak et al. 1994*). Based on axle load taken from field measurements on bridges located in Michigan, as well as State Police citation files for overload vehicles, the maximum observed axle load for a 1-year interval is close to 200 kN (40 kips), which produces 50 kN (10 kips) per wheel (2 tires per wheel). Therefore, in this calibration, the mean maximum one year value for a wheel load is taken as 50 kN (10 kips). The coefficient of variation is taken as 0.15 *(Nowak et al. 1994).*

Tire contact area is an important consideration for live load distribution to short span components. Based on the measurements reported by Pezo et al. *(1989)* and Sebaaly (*1992*), the transverse dimension (width) of the contact area is 185 mm (7.5 in) for each tire, with a 125 mm (5 in) gap between tires for a dual tire wheel. A nearly linear relationship exists between the wheel load and length of the contact area. For a 50 kN (10 kips) wheel load, tire length is approximately 250 mm (10 in). Therefore, in this study, the contact area for a single tire is considered as a rectangle of 180 mm x 250 mm (7.5x10in), and for a dual tire, a rectangle of 250 mm x 500 mm (10x20 in) (the gap is ignored).

In the AASHTO Standard *(1996)*, dynamic load is not considered for wood bridges. In AASHTO LRFD (*1998*), dynamic load is specified at 50% of the corresponding value specified for concrete and steel girders. Field measurements reviewed for the development of the AASHTO LRFD Code indicated the presence of a dynamic load effect in timber bridges *(Nowak and Eamon 2001).* It was also observed that the load effect was lower than that for other materials. Dynamic load is associated with a very short duration, much shorter than the static portion of live load. However, the strength of wood can be considerably larger for shorter time periods. Because of these observations, and of a lack of more detailed test data, the increase in component strength is not considered in the calibration process, but the dynamic load is taken as zero.

Material Resistance Models

The deterministic models of resistance are summarized by Ritter (*1990*). The major mechanical properties of wood are modulus of rupture (MOR), modulus of elasticity (MOE), and shear strength. These properties are subject of a considerable variation, and the statistical parameters depend on dimensions, species, grade, moisture content, and load duration.

For various grades and sizes of sawn lumber, a considerable data base was developed by Madsen and Nielsen (*1978a, 1978b*). For Douglas Fir, bias factors, with respect to tabulated strength values listed in the 1996 LRFD Manual for Engineered Wood Construction (*1996*) vary from 1.41-1.98 for select grade and 1.76-2.88 for grades 1 and 2, while coefficient of variation ranges from 0.17-0.27 for select and 0.23 to 0.30 for grade 1 and 2. The higher variations correspond to sections with largest depth/width ratios. Resistance is taken as a lognormal random variable.

For glulam girders, the statistical parameters for strength are taken from the report by Ellingwood et al. (*1980*), based on the test results obtained by the USDA Forest Products Laboratory on beams with Douglas Fir (DF) and Southern Pine (SP) with horizontally oriented laminating. The resulting bias factor is from approximately 2-3, with an average of 2.5, and coefficient of variation is from 0.10-0.25, with the average of 0.15. For bias factor calculation, the nominal (tabulated) value of resistance (MOR) is as specified by the National Design Specification for Wood Construction (*1991*). For glulam decks, where laminations are vertical rather than horizontal, data is provided by Hernandez et al. (*1995*), where bias factors ranged from 2.99 to 3.15, and coefficient of variation ranged from 0.20 to 0.25. Resistance is taken as a lognormal random variable.

As an increased moisture content may cause reduction of MOR. The LRFD Manual for Engineered Wood Construction (*1996*) specifies a wet service factor C_M to be applied to MOR where the moisture content exceeds 19% for sawn lumber and 16% for glulam. It is reasonable to expect that the actual effect of moisture content on MOR and other properties may follow a continuous curve, rather than a sharp change at a particular moisture level. However, due to a lack of availability of sufficient additional data, in this study it is assumed that the mean moisture content effect is as specified in the LRFD Manual for Engineered Wood Construction.

MOR of sawn lumber members is affected by whether the load is applied to the broad face (flatwise loading) or narrow face (edgewise loading) of the member. The results of flatwise versus edgewise loading on deck planks are described by Stankiewicz and Nowak (*1997*). Tests were performed on Red Pine, sizes 4x6, 4x8, 4x10 and 4x12. Results indicate that, if members are loaded flatwise, mean MOR is increased by a factor of 1.14 (for 4x6) to 1.50 (for 4x12), over that of edgewise loading, depending on section proportions. The experimental values are higher than the design values specified for NDS, which vary from 1.05 (4x6) to 1.10 (4x12). Flat-wise strength increases are primarily the result of flaws in the wood that may result in little change of section properties when members are loaded flatwise, while for an edge-wise loaded member, the same size flaw occupies a higher proportion of section width and thus may weaken it substantially. Coefficient of variation ranges from 0.25 to 0.31, where wider sections have the lowest variation.

The variability of modulus of elasticity (MOE) is described by Nowak (*1983*). It is considered as a lognormal distribution, with a coefficient of variation of 0.20. MOE is partially correlated with MOR. The correlation can be described with MOE as a linear function of MOR, as shown in Eq. 1.

$$\text{MOE} = [0.15\ (\text{MOR}) + 0.7]\ 1000 \tag{1}$$

From the standpoint of reliability, this relationship is important as in a system of wood components (such as a deck with multiple laminations), where the weakest (less-stiff) members absorb less force, increasing the reliability of the system.

Variation of dimensions is negligible. Form the measurements performed by Madsen and Nielsen (*1978a, 1978b*), the coefficient of variation is about 0.01. The bias factor varies from 0.97 to 1.04.

Structural Resistance Models

The current AASHTO LRFD Code (*1998*) girder distribution factor (GDF) formulas for wood bridges are given as a function of girder spacing only. The accuracy provided by this method is insufficient for developing a suitable resistance model. The GDF formulas for steel or concrete girders supporting a concrete deck predict load distribution well for a certain range of idealized structures, regardless of material. However, these formulas lose accuracy when girder spacing less than 1.1m or spans greater than 6m are considered. Many wood bridges have beam spacing and spans less than these values. Therefore, in this study, load distribution to stringers is based on finite element analysis.

The considered spans are from 4.5-21m (15-70 ft) with girder spacing from 0.4m-1.8m (16-72in). Standard wood material properties were used, and typical girder and deck stiffness parameters were used for the spans studied. Beams were represented with beam elements, and the deck was represented with hexahedral elements. Mesh density was chosen such that further refinements resulted in insignificant changes in girder moments. Beams were attached directly to the underside of the deck (as non-composite action is assumed for wood bridges, depth of beam from deck is not important). The wheel patches of two AASHTO design trucks, either HS-

20 or the design tandem, whichever governed, were loaded side-by-side on the bridge in the position that would generate maximum GDF to any interior girder. At larger girder spacing, the models closely matched the results of the AASHTO LRFD Code formula, as well as earlier research (*Nowak 1999; Bakht et al. 1985*).

An additional factor affecting load distribution is the post-elastic response of wood. Studies by Sexsmith, et al. (*1979*) have produced data that describe the behavior of wood as it is stressed until failure. Although wood does not exhibit plastic behavior such as steel, the small softening effect that is present may be important because as a wood component loses stiffness before failure, it allows for some load redistribution to stiffer, less-stressed members. Idealized stress-strain curves were developed for this project by analyzing the actual load and deflection results of Sexsmith's tests.

To further study these effects, four typical bridges were modeled in detail using the finite element method. These spans ranged from 15-30' (4.5-9m) and stringer spacing from 16"-72" (400-1,800mm) In each case, a nonlinear analysis was performed using the stress-strain relationships developed above. In general, it was found that if a load effect is large enough to cause a single stringer to reach MOR, it immediately loses practically all load carrying capacity. When load is redistributed to the remaining members, now fewer in number, entire bridge collapse is usually inevitable. With wood, stringer bridge capacity does not significantly benefit by considering this small softening effect (of the cases studies, the average increase in moment capacity was 1%, while the maximum increase was found to be 2%), as the load redistribution throughout the system is slight as failure approaches. However, this effect, in addition to the correlation between MOE and MOR, presented above, results in a decrease in variation of capacity of a stringer subsystem.

Based on results of analysis, for closely-spaced sawn lumber stringers (16-24", 400-600mm), a subsystem of three stringers tends to relatively equally share load when two trucks are side-by-side. For wider girder spacing, however, such as that for glulam girder bridges (1.5-2.4 m, 5-8ft), only one girder substantially resists a wheel load. Based on model simulations, coefficient of variation V of the 3-stringer subsystem is taken to be 0.15 (typical component V = 0.23), while for spacings much greater than 24" (600mm) (glulam girder bridges), coefficient of variation is not reduced from component V.

For decks and deck bridges, a single vehicle wheel will load a number of laminations simultaneously, and the statistical parameters of resistance for this subsystem must be considered as well. Although determining GDF is relatively insensitive to modeling technique, deck behavior is much less reliably predicted by analysis. Therefore, for this study, the existing experimental data were used to develop a model for deck resistance. In particular, the deflection profiles from a number of field tests were examined *(Bakht 1988; Wacker and Ritter 1992, 1995; Ritter et al. 1995; Lee et al. 1996*).

For nail-lam decks, after years of service, it was observed that there is a very limited load sharing effect. Based on these observations, for timber decks, a sub-system with a width of 750mm (30 in) is considered, as shown in Fig. 6. This is similar in size to the tire contact area, and in loosened up decks (after years of service), this is the area of uniform deflection. Here the girder distribution factor (GDF) is approximately 0.80-0.85 for two lanes loaded. These values were obtained by calculating the areas under typical deflection curves from existing experimental data. For a typical deck lamination subsystem, coefficient of variation is taken as 0.15 (0.32 for a typical single lamination).

A similar sub-system (with the width of 900mm, 36 in) is considered for stressed and glulam decks. The girder distribution factor is 0.45-0.55. The statistical parameters of resistance of a stressed sub-system are based on the test data obtained by Sexsmith et al (1979). The mean moment carrying capacity (resistance) of the sub-system (500 mm, 20 in wide) is equal to the sum of mean capacities of individual elements (boards). The mean MOR of a system is the same as that for an individual element. However, the coefficient of variation is 0.10 (for typical single lamination V = 0.32).

For glulam deck systems, no specific data on coefficient of variation are available. Studies have shown, however, that glulam decks display a similar, slightly more stiff transverse behavior as compared to stress-lam decks (*Batchelor et al. 1979, 1981; Bakht 1988*). Coefficient of variation is therefore conservatively based on stress-lam data.

For plank decks, based on a previous study by Eamon et al. (*2000*), it is assumed that the wheel load is resisted by the planks under the tire-deck contact area. The contact area is 250mm x 500mm (10" x 20"). For plank widths less than 250mm (10"), adjacent planks can share the load, and the distribution of load is proportional to the contact area for each plank. For common plank widths, approximately two planks can share the load. Here coefficient of variation is taken as 0.20 (for a typical single plank V=0.20).

Reliability Analysis

The reliability analysis is performed for a flexural limit state. Although components of wood bridges can be subjected to other load effects such as shear and torsion, there is currently insufficient test data available for resistance parameters to be reliably developed for these failure modes. However, flexural failures typically govern for the primary load carrying members of stringer bridges, although this is not always true for deck bridges.

As live load dominates, load effect is taken to be a lognormal random variable. Resistance test data for individual components indicates that the distribution of load carrying capacity can be approximated by a lognormal function, in particular this applies to the lower tail of the cumulative distribution function. For stringers, glulam and deck subsystems, resistances are taken as normal.

In this study, the reliability of components is calculated using a first order, second moment method for lognormal random variables *(Nowak and Collins 2000)*. For subsystems, reliability is calculated using the Rackwitz-Fiessler procedure *(Nowak and Collins 2000)*. A summary of analysis results is presented in Tables 1 and 2 for wood bridges designed according to AASHTO Standard (*1996*) and AASHTO LRFD Code (*1998*), respectively. In general, variation in reliability index is significant. For components, beta ranged from 2.1-3.1 for bridges designed according to the AASHTO Standard and from 1.7-3.1 for the AASHTO LRFD Code. For subsystems, beta ranged from 3.1-6.4 for the Standard Specifications and 3.1-4.3 for the LRFD Code.

Results Of Calibration

Based on these results, target reliability indices are selected. For sawn lumber stringers as components, the recommended target reliability is $\beta_T = 3.0$, and for a subsystem of sawn lumber stringers $\beta_T = 4.0$. For glulam girders as components, $\beta_T = 3.5$, and for subsystem $\beta_T = 3.75$; for nail lam decks, component $\beta_T = 2.0$ and subsystem $\beta_T = 3.5$; for stressed wood decks, component $\beta_T = 1.75$ and subsystem $\beta_T = 3.5$; for plank decks, component $\beta_T = 2.75$ and subsystem $\beta_T = 3.5$. For the various components considered, target reliability indices are chosen within the range of results such that the typical designs represent the target values. Note that for a system of parallel components, such as a stringer or deck system, target indices are higher than that of a single component, as there is a decreased probability of system failure relative to single component failure. Here there is no attempt to specify a new Code safety level, but rather to even out the differences in reliability found in different configurations (for example, as a result of bridge span, stringer spacing, deck thickness, wood species, etc.) of the same type of design. To achieve the target indices, the following design provisions are recommended for the AASHTO LRFD Code:

1. Use load factors specified by the AASHTO LRFD Code (*1998*).
2. Use the material strength values specified in the LRFD Manual for Engineered Wood Construction (*1996*).
3. The load duration corresponding to live load is two months, so the material strength values must be multiplied by the load duration factor of 0.80. If, at the considered location, the duration of extreme values of live load exceeds a two month period, then the load duration factor may have to be decreased.
4. The moisture content (wet service) factor must be applied to bridge components.
5. Dynamic load may be neglected.

The reliability analysis was performed for various values of resistance factors, all rounded to the nearest 0.05. The recommended resistance factors are selected based on closeness to the target reliability indices. The results are as follows: for flexure $\phi = 0.85$, for compression $\phi = 0.90$, for tension $\phi = 0.80$, for shear/torsion $\phi = 0.75$, and for connections $\phi = 0.65$.

Reliability indices were calculated for wood bridges designed according to the recommended provisions and they are shown in Table 3. The result is a more uniform level of safety for both components and subsystems, while all beta values are equal to or above the recommended target levels.

Conclusions

The calibration of the design code for wood bridges resulted in recommended load and resistance factors and other suggested changes in the current version of the AASHTO LRFD Code (*1998*). It was observed that the reliability indices for wood bridges designed to the current code have a considerable degree of variation. It is recommended to use the material strength values specified in the LRFD Manual for Engineered Wood Construction (*1996*). For the components resisting live load, material strength must be reduced by using the load duration factor for two months, 0.80. Moisture content factor must be applied to bridge components. It is also recommended to neglect the dynamic load for wood bridges.

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References

AASHTO (1998), "LRFD Bridge Design Specifications," American Association of State Highway and Transportation Officials, Washington, D.C..

AASHTO (1996), "Standard Specifications for Highway Bridges," American Association of State Highway and Transportation Officials, Washington, D.C.

Bakht, B (1988). "Load Distribution in Laminated Timber Decks," ASCE Journal of Structural Engineering, Vol 114, No. 7, July, pp. 1551-1570.

Bakht, B. and Jaeger, L (1985). Bridge Analysis Simplified. New York:McGraw-Hill.

Batchelor, B, Van Dalen, K, Hachborn, A, Lee, E (1979). Structural characteristics of transversely prestressed laminated wood bridge decks. Report 8305. Kingston, ON, Can: Queen's University.

Batchelor, B, Van Dalen, K, Morrison, T, Taylor, R.J. (1981). Structural characteristics of red pine and hem-fir in prestressed laminated wood bridge decks. Report 23122. Kingston, ON, Can: Queen's University.

Eamon, C., Nowak, A.S., Ritter, M., Murphy, T. (2000). "Reliability-based Criteria for LRFD Code for Wood Bridges," Transportation Research Record, No. 1696 Vol. 1 pp. 316-322.

Ellingwood, B., Galambos, T.V., MacGregor, J.G., Cornell, C.A. (1980) "Development of a Probability Based Load Criterion for American National Standard A58," Special Publication 577, National Bureau of Standards, June.

Hernandez, R., Moody, R.C. and Falk, R.H. (1995). "Fiber Stress Values for Design of Glulam Timber Utility Structures;" Pub. USDA FPL, Research Paper FPL-RP-532, August.

Lee, P.H., Ritter, M.A., and Wacker, J.P. (1996). Field Performance of Timber Bridges 10: Sanaborn Brook Stress-Laminated Deck Bridge. FPL Research Paper FPL-RP-555, USDA Forest Products Laboratory, Madison, WI.

LRFD Manual for Engineered Wood Construction (1996). The Engineered Wood Association, Tacoma, WA.

Madsen, B. and Nielsen, P.C. (1978a). "In-Grade Testing in Beams and Stringers," Department of Civil Engineering, University of British Columbia, Vancouver, August.

Madsen, B. and Nielsen, P.C. (1978b). "In-Grade Testing—Bending Tests in Canada, June 1977 to May 1978," Structural Research Series, Report No. 25, Department of Civil Engineering, University of British Columbia, Vancouver, August.

National Design Specification for Wood Construction (1991). National Forest Products Association, Washington, DC.

Nowak, A. S., (1983). "Modeling Properties of Timber Stringers," report UMEE 83R1, University of Michigan, May.

Nowak, A.S, (1991). "Reliability Analysis for Wood Bridges," Transportation Research Record, No. 1291, Vol. 1, pp. 315-327.

Nowak, A.S. and Collins, K.R. (2000). "Reliability of Structures", McGraw Hill, New York.

Nowak, A.S. and Eamon, C.D (2000). "LRFD Calibration for Wood Bridges," Project 96-RJVA-2822, Report submitted to US Dept. of Agriculture Forest Service, Forest Products Laboratory, April.

Nowak, A.S. (1999). "Calibration of LRFD Bridge Design Code", NCHRP Report 368, Transportation Research Board, Washington, D.C.

Nowak, A.S. (1995). "Calibration of LRFD Bridge Design Code", ASCE Journal of Structural Engineering, Vol. 121, No. 8, pp. 1245-1251.

Nowak, A.S. (1993). "Live Load Model for Highway Bridges", Journal of Structural Safety, Vol. 13, Nos. 1+2, December, pp. 53-66.

Nowak, A.S., Kim, S-J., Laman, J., Saraf, V. and Sokolik, A.F. (1994). "Truck Loads on Selected Bridges in the Detroit Area", Final Report submitted to MDOT, November.

Pezo, R. F., Marshek, K. M., and Hudson, W. R. (1989). "Truck Tire Pavement Contact Pressure Distribution Characteristics for the Bias Goodyear 18-22.5, the Radial Michelin 275/80R/24.5, and the Radial Michelin 255/70R/22.5, and the Radial Goodyear 11R24.5 Tires", Research Report Number FHWA/TX-90+1190-2F, Center for Transportation Research, University of Texas, Austin, September.

Ritter, M. (1990). "Timber Bridges, Design, Construction, Inspection, and Maintenance", Report, USDA Forest Service.

Ritter, Michael A, Wacker, James P, and Tice, Everett D. (1995). Field Performance of Timber Brides: 2. Cooper Creek Stress-Laminated Deck Bridge. FPL Research Paper FPL-RP-536. USDA Forest Products Laboratory, Madison, WI.

Sebaaly, P. E. (1992). "Pavement Damage as Related to Tires, Pressures, Axle Loads, and Configurations," Vehicle, Tire, Pavement Interface, ASTM STP1164, J. J. Henry and J. C. Wambold, eds., American Society for Testing and Materials, Philadelphia, pp. 54-68.

Sexsmith, R.G., Boyle, P.D., Rovner, B., and Abbott, R.A. (1979). "Load Sharing in Vertically Laminated, Post-Tensioned Bridge Decking," Technical Report No. 6, Western Forest Products Laboratory, Vancouver, British Columbia.

Stankiewicz, P. R. and Nowak. (1997). "Material Testing for Wood Plank Decks", UMCEE 97-10, Report submitted to US Forest Service, April.

Wacker, J. P. and Ritter, M. A. (1992). "Field Performance of Timber Bridges: 1. Teal Riveer Stress-Laminated Deck Bridge." FPL Research Paper FPL-RP-515, USDA Forest Products Laboratory, Madison, WI.

Wacker, J. P. and Ritter, M. A. (1995). Field Performance of Timber Bridges: Graves Crossing Stress Laminated Deck Bridge. FPL Research Paper FPL-RP-539. USDA Forest Products Laboratory, Madison, WI.

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Table 1. Range of Reliability Indices for AASHTO Standard (*9*)

Type of Structure	β Single Element	β Subsystem
Sawn Lumber Stringers	2.27-2.47	3.11-3.38
Glulam Girders	3.08-4.02	3.37-4.39
Nail-lam Deck	2.14-2.29	3.90-4.17
Stressed Deck	2.63-2.77	6.05-6.39
Plank Deck	2.84-3.08	3.75-4.08

Table 2. Reliability Indices for AASHTO LRFD Code (*1*)

Type of Structure	β Single Element	β Subsystem
Sawn Lumber Stringers	2.96-3.09	4.07-4.25
Glulam Girders	2.80-3.13	3.06-3.43
Spike-lam Deck	1.73-1.82	3.16-3.34
Stressed Deck	1.42-1.51	3.29-3.52
Plank Deck	2.38-2.52	3.16-3.34

Table 3. Reliability Indices using Recommended Design Provisions

Type of Structure	β Single Element	β Subsystem
Sawn Lumber Stringers	2.96-3.09	4.08-4.26
Glulam Girders	3.33-3.62	3.65-3.97
Spike-lam Deck	2.02-2.12	3.69-3.87
Stressed Deck	1.71-1.81	3.97-4.19
Plank Deck	2.78-2.92	3.69-3.87

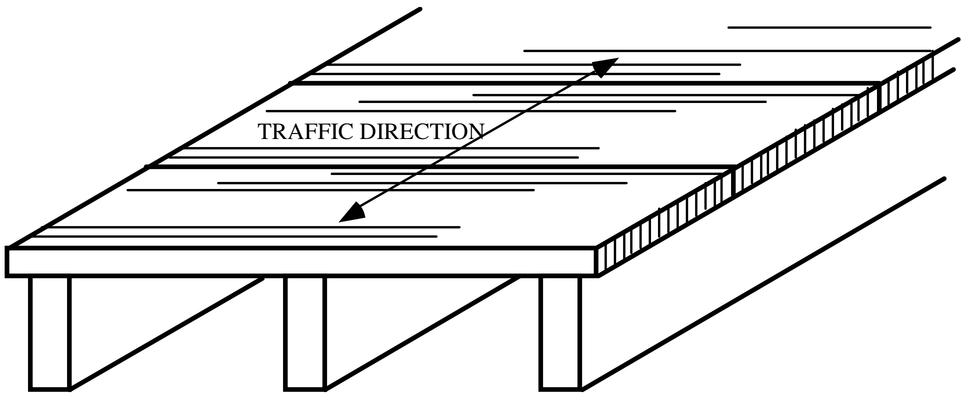

Figure 1. Stringer Bridge, Deck Perpendicular to Traffic.

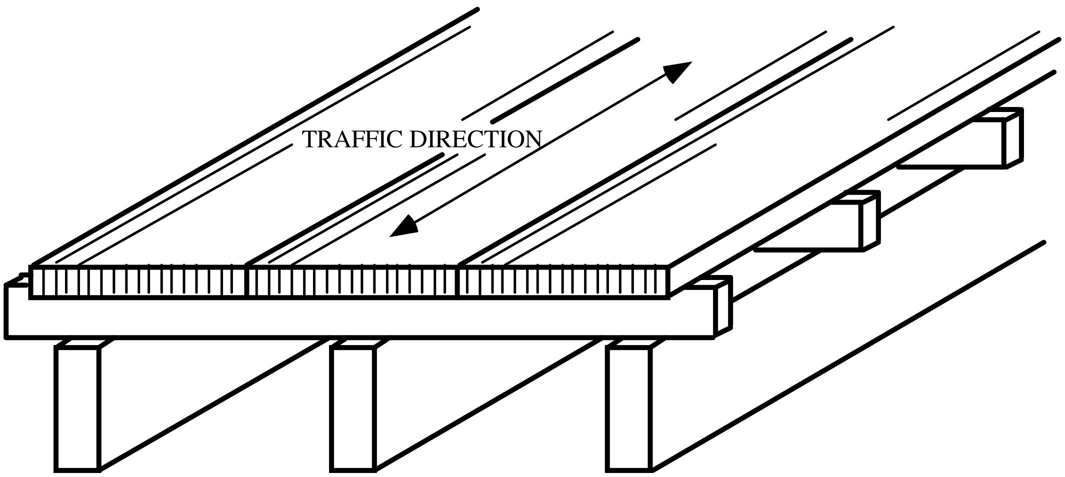

Figure 2. Stringer Bridge, Deck Parallel to Traffic.

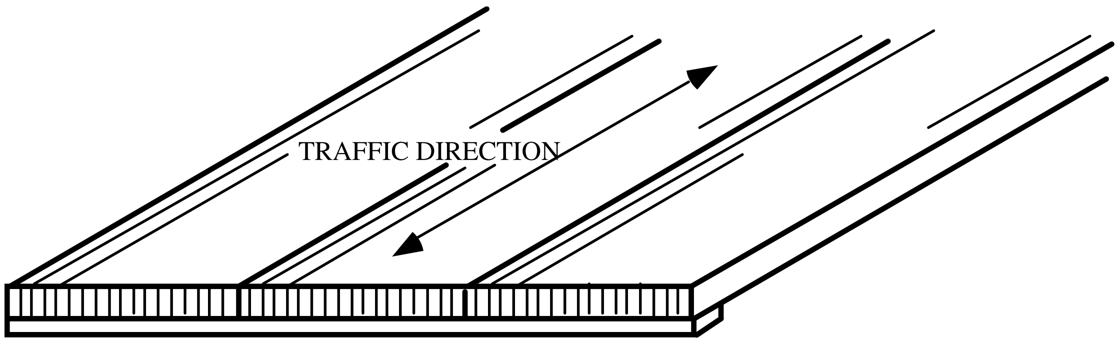

Figure 3. Deck Bridge.

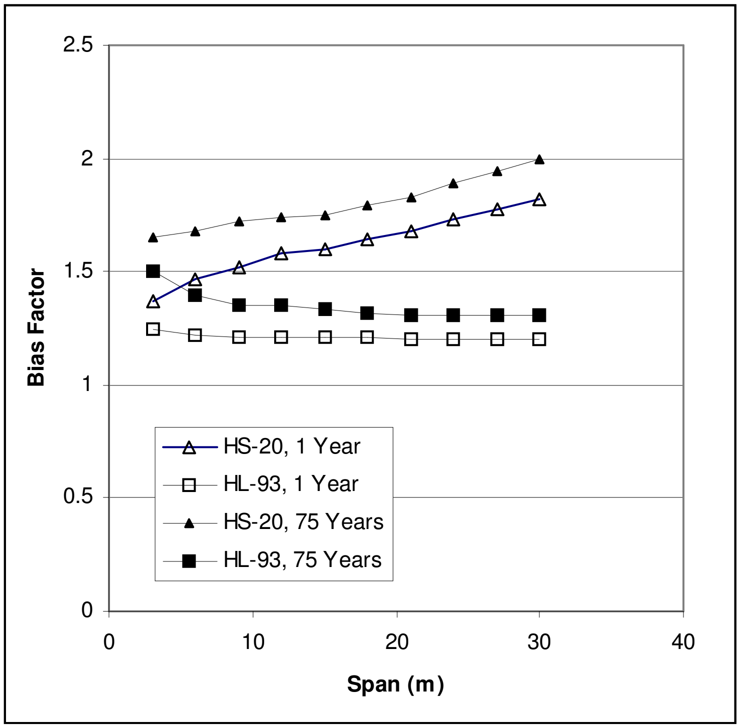

Figure 4. Live Load Bias Factors.

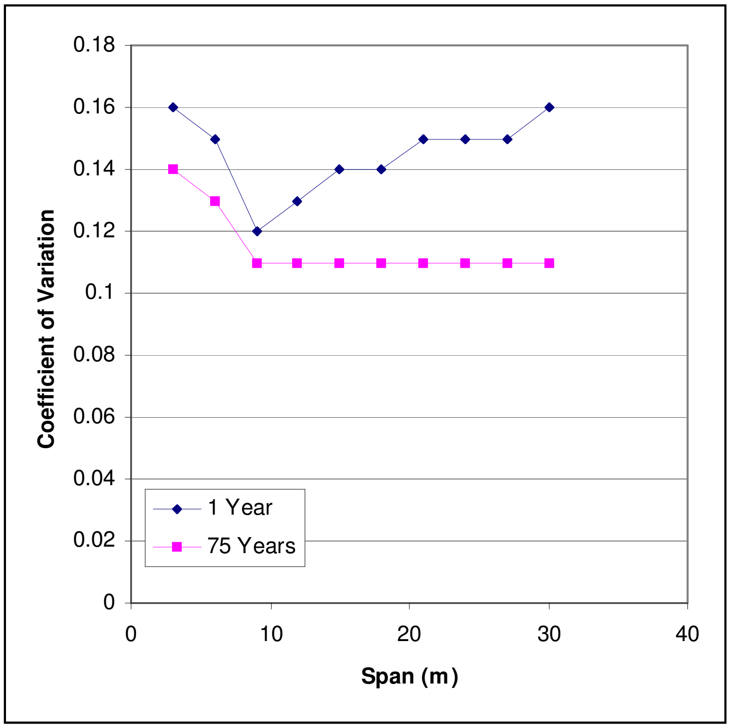

Figure 5. Coefficient of Variation of Live Load

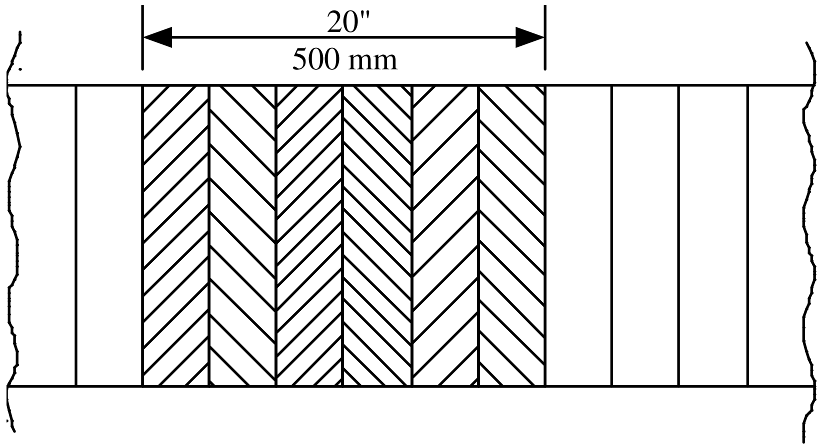

Figure 6. Deck Subsystem