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Discussion

Celal N. Kostem, Lehigh University, Bethlehem, Pennsylvania

Before the load- and resistance-factor design concepts in bridge engineering are fully adopted, some issues require further clarification. Specifically, two of the related issues that have been indicated in MacGregor's paper are serviceability limits and overload provisions.

Quantification of type II serviceability limit states has not been uniformly agreed on by different code and specification writers. For example, in some instances the width and the depth of the cracks in the reinforced concrete deck slab are not being considered as an important parameter, if the prestressed beams supporting the deck are to remain elastic during the loading phase. This premise stems from the assumption that the beams will flex up and thus the cracks will be closed. Qualitative and quantitative decisions must be uniformly agreed on before type II serviceability limits (12) are used.

The research has also indicated that the overload provisions of AASHTO may be used as crude guidelines in the design of highway bridge superstructures. However, in view of the ever-expanding new overload configurations for vehicles, the extrapolation of the current AASHTO specifications into the load- and resistance-factor design approach may not be prudent. If new design concepts are to be developed, then the updating of the provisions pertaining to design overloading will be of great assistance to the bridge engineers (13).

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Probabilistic Approaches to the Design of Steel Bridges

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This paper demonstrates how a relatively simple first-order probabilistic method can be used to assess the reliability of the 1977 AASHTO specification for the design of steel bridges and how a consistent load- and resistance-factor design specification can be developed for steel bridge structures. It is shown that the AASHTO load-factor design method, as characterized by the safety index, is consistently reliable but that the reliability of the allowable-stress design method varies considerably. The paper also outlines the steps needed to generate a probability-based bridge design code and lists the available statistical data for steel members and connectors. The conclusions are that existing theory and data will allow development of a probabilistic design specification for steel bridges and that the format of such a specification does not differ greatly from the AASHTO load-factor design method.

Good structural design is a process of creating a load-carrying system that will perform as intended during its lifetime. The role of structural specifications, such as the AASHTO standard specification for highway bridges, is to set minimum requirements that ensure that the probability of system's malfunctioning is acceptably small.

The AASHTO specification accounts for expected overloads and normal uncertainties of loads and resistances by specifying load factors or factors of safety. Such a specification is a highly complex and sophisticated instrument that continues to evolve as a result of experience and research.

Changes are made by consensus agreement based on the combined judgment of the members of the various committees involved. Questions of safety, economy, and practicality are thoroughly explored before modifications are implemented. In this process, ideas of probability-based design decisions are seldom stated explicitly, but these ideas are nevertheless used implicitly. Recent emphasis in research on probability-based design (1) has made it possible to formulate design criteria on simple probabilistic concepts (2). At least two completed specifications for steel bridge structures now exist (January 1979) in proposal form (3, 4).

The purpose of this paper is to examine the current (1977) AASHTO specification in the light of these developments and to recommend research for implementing probabilistic concepts for the further evolution of steel bridge design criteria.

BASIC PROBABILISTIC CONCEPTS

A structure is "safe" if during its lifetime the most severe limit state caused by the effects of loading is not exceeded. A limit state signifies a limit of structural usefulness that, if exceeded, brings failure such as collapse, fracture, instability, excessive deflection (ultimate limit states) or permanent set, and excessive vibration (serviceability limit states).

A limit state is characterized by the resistance of the structure, and the action of loads (dead loads, vehicle loads, environmental loads) is characterized by a load effect. Bending moment, axial force, shear force, and torque are load effects.

If we represent resistance by R and the load effect by Q, then a structure is safe when $R > Q$, and the probability of exceeding a limit state is $P(R \leq Q)$ (Figure 1).

Implicit in this statement about probability is that R and Q are random variables and that R and Q are also independent variables. The latter is not exactly true, because a mild dependence must exist between the strength and the mass of the structure on the one hand and dynamic load-effect amplification on the other.

We do not have enough information to define the complete probability distribution of R and Q, but we are able to have fairly reliable estimates on the first two moments of the distribution, i.e., the means R_m and Q_m and the standard deviations σ_R and σ_Q, respectively. Based on this limited information, a design methodology called second-moment or first-order probabilistic design was developed (5-10), and steel design specifications for buildings (2, 11) and bridges (3, 4) have been proposed.

A structure is safe if $R > Q$, or, equivalently, if $\ell n\, R/Q > 0$. A schematic plot of $\ell n\, R/Q$ is shown in Figure 2, where the probability of failure is the shaded area when $\ell n\, R/Q$ is negative. For given statistical properties R_m, Q_m, $V_R = \sigma_R/R_m$, and $V_Q = \sigma_Q/Q_m$ (V's are coefficients of variation), the probability of failure (exceeding a limit state) is smaller if the whole curve $\ell n\, R/Q$ is shifted to the right, and it is larger if it is shifted to the left. This shift can be accomplished by varying the distance between the mean of the distribution $\ell n\, R/Q$ and the zero line (Figure 2) by varying the factor β. This factor is the safety index, and it has been used to compare the reliability of designs (5-11). If the exact probabilistic distributions of R and Q were known, one could relate β directly to a probability of exceeding a limit state. But because we do not know the distributions, β can be used (a) to comparatively assess the reliability of designs according to current design specifications and (b) to develop new design criteria that are uniformly reliable.

The safety index β can be approximated according to first-order probabilistic principles (10, 11) by the equation

$$\beta = (\ell n R_m/Q_m)/(V_R^2 + V_Q^2)^{1/2} \tag{1}$$

In what follows I shall examine the safety index implied in the allowable-stress design (ASD) method and the load-factor design (LFD) method of the AASHTO specification by means of an example of a simply supported multistringer steel bridge.

SAFETY INDEX FOR 1977 AASHTO SPECIFICATION

The bridge to be examined is a multistringer bridge with laterally braced compact steel wide-flange beams. The

Figure 1. Schematic distribution of R and Q.

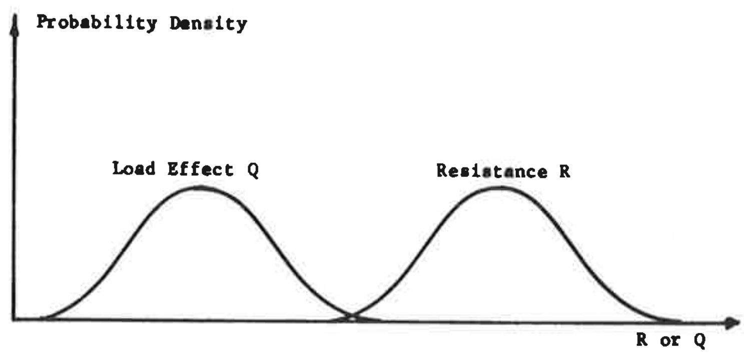
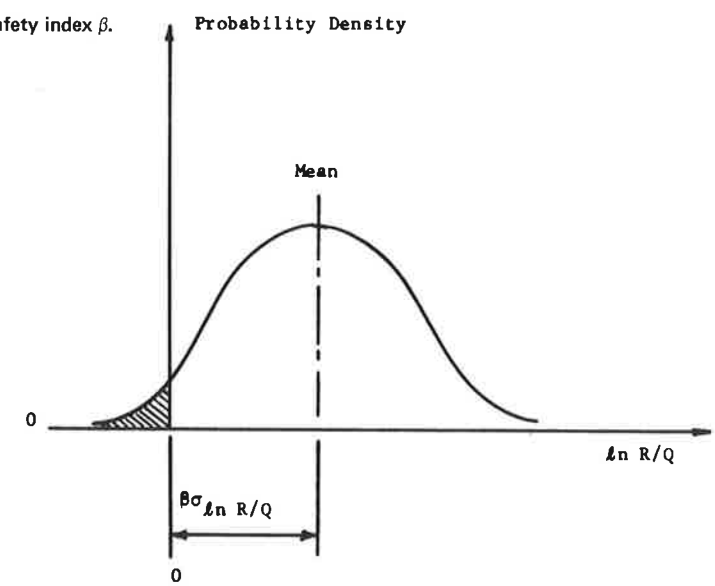


Figure 2. Definition of safety index β.



controlling limit state will be assumed to be the flexural capacity.

The resistance is equal to the plastic moment (M_p), and the mean resistance (R_m) (10, 11) is

$$R_m = Z F_y (P_m M_m F_m) \quad (2)$$

where

Z = nominal plastic modulus,
F_y = minimum specified yield stress,
P = a random parameter representing the professional factor,
M = a random parameter representing the actual yield stress or the material factor, and
F = a random parameter representing the actual plastic modulus or the fabrication factor.

P, M, and F represent the relationship between test and theory. These random parameters have mean values and coefficients of variation. These were determined from the results of tests and measurements (10, 11, 13) to be

$P_m = 1.02$, $V_P = 0.06$
$M_m = 1.05$, $V_M = 0.10$
$F_m = 1.00$, $V_F = 0.05$

Therefore

$$R_m = 1.02 \times 1.05 \times 1.00\, ZF_y \quad (3)$$

and

$$V_R = (V_P^2 + V_M^2 + V_F^2)^{1/2} = 0.13 \quad (4)$$

The maximum bending moment due to dead load and vehicle load, including impact and load distribution, is the load effect Q. Its mean value (4) is

$$Q_m = D_m + L_n(L_m A_m I_m) \quad (5)$$

and its coefficient of variation is

$$V_Q = [D_m^2 V_D^2 + L_n^2 (L_m A_m I_m)^2 (V_L^2 + V_A^2 + V_I^2)]^{1/2} \div [D_m + L_n(L_m A_m I_m)] \quad (6)$$

In these equations

D_m = mean dead-load moment,
L_n = maximum moment due to the standard AASHTO vehicle (including the effect of the AASHTO impact formula and the lateral load-distribution factor),
L_m = ratio of the mean maximum lifetime moment due to vehicles to the maximum moment due to the standard AASHTO vehicle,
A_m = ratio of the mean actual lateral load-distribution factor to the AASHTO lateral load-distribution factor, and
I_m = ratio of the mean actual dynamic amplification to the AASHTO impact factor.

A study of the available statistics (4) has resulted in the following values for these factors:

$D_m = D_n$, $V_D = 0.06$
$L_m = 1.10$, $V_L = 0.11$
$A_m = 1.0$, $V_A = 0.15$
$I_m = 1.0$, $V_I = 0.10$

where D_n is the nominal dead load.

These statistical data are estimates, and further careful study is required (see the paper by Moses in this Record). Also, it is possible to use different models to represent loading. However, our data were arrived at, after careful study of the available data, by using reasonable judgment where these data were uncertain (4).

The next step in the analysis is an evaluation of the safety index inherent in the ASD portion of the AASHTO specification.

According to AASHTO Specification section 1.7.1, the required elastic modulus is

$$S = (D_n + L_n)/0.55\, F_y \quad (7)$$

The elastic and the plastic moduli, S and Z, respectively, are related by the shape factor $f = Z/S$; for beam-type sections the mean value according to Beedle (14) of f is 1.14. The coefficient of variation is 0.02, which is negligible in its effect on $V_R = 0.13$. Thus

$$Z = 1.14\, [(D_n + L_n)/0.55\, F_y] \quad (8)$$

Substituting the ASD-required Z into the equation for R_m (Equation 3), we obtain

$$R_m = 1.02 \times 1.05 \times 1.00 \left\{ 1.14\, [(D_n + L_n)/0.55\, F_y] \right\} F_y \quad (9)$$

or

$$R_m = 2.22\, (L_n)[(D_n/L_n) + 1] \quad (10)$$

and

$$V_R = 0.13 \quad (11)$$

From Equation 5

$$Q_m = L_n\, [(D_n/L_n) + L_m A_m I_m] = L_n [(D_n/L_n) + 1.1 \times 1.0 \times 1.0] \quad (12)$$

or

$$Q_m = L_n\, [(D_n/L_n) + 1.1] \quad (13)$$

and thus

$$R_m/Q_m = 2.22\, [(D_n/L_n) + 1]/[(D_n/L_n) + 1.1] \quad (14)$$

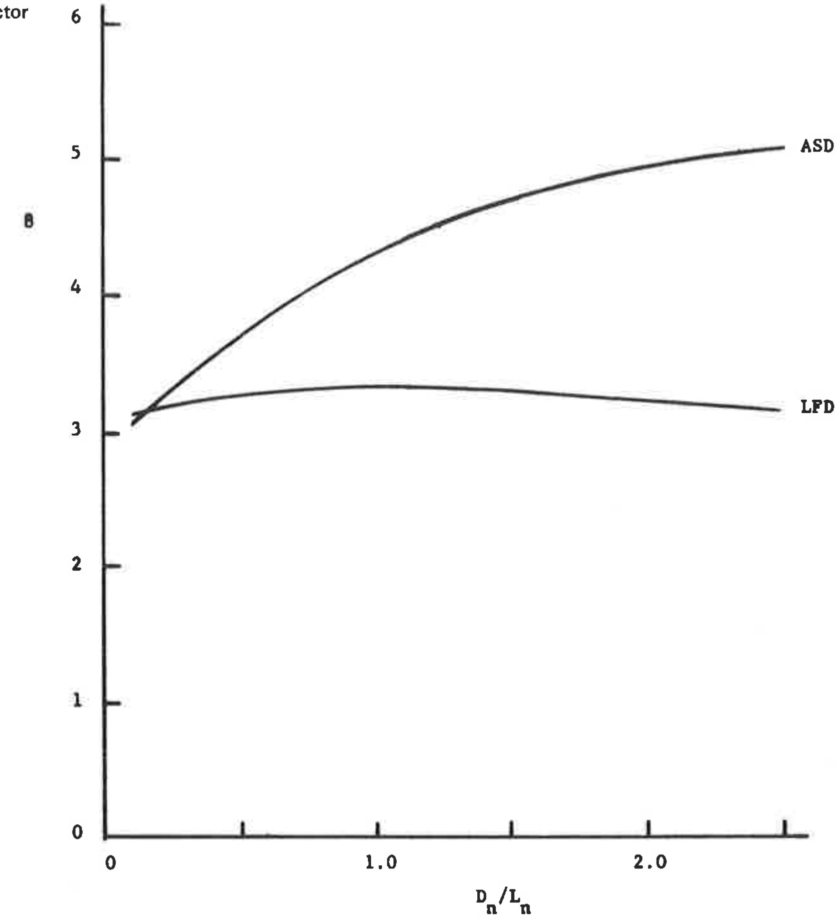
From Equation 6

$$\begin{aligned} V_Q &= \{L_n^2 [(D_n/L_n)^2 V_D^2 + (L_m A_m I_m)^2 (V_L^2 + V_A^2 + V_I^2)\}^{1/2} \div L_m [(D_n/L_n) + L_m A_m I_m] \\ &= [(D_m/L_m)^2 \times 0.06^2 + (1.1 \times 1.0 \times 1.0)^2 (0.11^2 + 0.15^2 + 0.10^2)]^{1/2}/[(D_n/L_n) + 1.1 \times 1.0 \times 1.0] \\ &= [0.0036\, (D_n/L_n)^2 + 0.05397]^{1/2}/[(D_n/L_n) + 1.1] \end{aligned} \quad (15)$$

The various components for determining β are now estimated (i.e., R_m/Q_m, V_Q, and V_R), and β can be calculated by using Equation 1. The variation of β with the nominal dead-to-live load ratio is shown in Figure 3 for the range of $0.1 \leq D_n/L_n \leq 2.5$. The safety index β according to this calibration for compact steel multistringer bridges designed by the 1977 AASHTO ASD method varies from about 3.0 to 5.0, and the reliability increases as the dead-to-live load ratio increases.

This range corresponds to a difference of approximately two orders of magnitude for the probability of exceeding the limit state of forming a plastic hinge (13), and it indicates that such bridges are overdesigned even when D_n/L_n is in a practically reasonable range. This fact has long been recognized, and the AASHTO specification provides an alternate LFD method based on the recommendations of Vincent (15).

Figure 3. β **for allowable-stress design and load-factor design.**



According to the LFD method, the required plastic modulus is

$$Z = 1.3\{[D_n + (5/3\ L_n)]/F_y\} \qquad (16)$$

Substitution of Z from Equation 16 into Equation 2 provides the calibration, and the final expressions for obtaining β from Equation 1 are as follows:

$$R_m/Q_m = [1.05 \times 1.02 \times 1.3\ (D_n/L_n) + (5/3)]/[(D_n/L_n) + 1.1] \qquad (17)$$

V_R equals 0.13 and V_Q is determined from Equation 15, just as it was for the previously calibrated ASD method. The variation of β with D_n/L_n is shown in Figure 3, where it is evident that Vincent's calibration, which proceeded from nonprobabilistic premises, was excellent and resulted in a nearly uniform β of approximately 3.2 for the entire D_n/L_n range. It is interesting to note that $\beta = 3.0$ was used as the basic safety index for members for steel building structures (2, 10, 11).

DESIGN OF LOAD AND RESISTANCE FACTORS

The calculations in the previous section of this paper have shown that the AASHTO LFD method provides nearly uniform reliability through the use of multiple load factors. This approach can be generalized further by the method known as load- and resistance-factor design (LRFD) or, in Canada, as limit-states design. The general form of the design equations can be expressed by

$$\sum_{i=1}^{n} \gamma_i Q_i \leq \phi R_n \qquad (18)$$

for each load combination and limit state. The left side of Equation 18 is the sum of the factored load effects, where γ_i is the load factor ($\gamma > 1.0$) and Q_i is the corresponding load effect from a given load source. The right side of Equation 18 is the product of the resistance factor ϕ ($\phi < 1.0$) and the resistance R_n for the particular limit state under consideration, determined for minimum specified material properties and for nominal sectional properties.

For dead-plus-live vehicle loads, for example, and for compact multistringer steel bridges, Equation 18 can be written as

$$\gamma_D M_D + \gamma_L M_L \leq \phi Z F_y \qquad (19)$$

where γ_D and γ_L are the dead- and live-load factors, respectively, and M_D and M_L are the corresponding moments.

The AASHTO LFD design formula (Equation 16) can be reproduced from Equation 19 if $\gamma_D/\phi = 1.3$ and $\gamma_L/\phi = 1.3 \times 5/3 = 2.167$ is used. The Ontario limit-states bridge design code (3) is somewhat more complex in that additional factors are introduced for importance and load combination and a distinction is made in the dead load between factory-produced elements and cast-in-place elements of the bridge (e.g., the girder weight versus the slab) and the loads due to the asphalt wearing surface (3).

For our example, the proposed Ontario code would have the following values:

$$1.15\ M_{D1} + 1.25\ M_{D2} + 1.7\ M_{D3} + 1.35\ M_L \leq 0.9\ ZF_y \qquad (20)$$

where M_{D1}, M_{D2}, and M_{D3} are moments due to the weight of the steel girders, the concrete slab, and the asphalt wearing surface, respectively, and M_L is the moment due to the vehicle plus its dynamic amplification; 1.15, 1.25, 1.7, and 1.35, respectively, are the corresponding load factors γ_i, and $\phi = 0.9$. Concerning the live-load factor $\gamma_L = 1.35$, it should be realized that the vehicle and impact models the Ontario code has adopted are different from those of AASHTO.

Experience and calibration (see Figure 3) have indicated that the multiple γ and ϕ factors, incorporated into the design format by whatever name (LRFD, LSD, or LFD), are consistently more reliable and economical than the traditional ASD method. The γ and ϕ factors are determined by some sort of calibration to a subdomain of existing structures that are deemed by experience and judgment to be just adequate for both safety and economy.

This was done in the development of the LFD method for the AASHTO specification (15) by accepting that a standard bridge 15 m (50 ft) long was just right. In the development of the Ontario LSD code a much more sophisticated iterative method was chosen (3), where γ and ϕ factors were assumed and then adjusted to give uniform safety index (β) values according to Equation 1. This latter approach is much more versatile in that calibration can be achieved over a wider domain of the appropriate design parameters, and thus better economy and more uniform reliability can be achieved.

Yet another method was used to obtain the ϕ and γ factors in the development of LRFD design criteria for steel building structures (2, 10, 11). In this method the underlying values of β (Equation 1) in the current (1978) American Institute of Steel Construction (AISC) design code were determined for steel beams, columns, and connectors. From this calibration process it was decided by judgment that $\beta = 3$ and $\beta = 4.5$ should be used as the basis for designing members and connections, respectively. The larger value of β for connections reflected the requirement that connection failure should be less likely than member failure. The ϕ and γ factors were then obtained from the relationships

$$\phi = (R_n/R_m) \exp - \alpha\beta V_R \quad (21)$$

$$\gamma_D = 1 - \alpha\beta V_D \quad (22)$$

$$\gamma_L = 1 - \alpha\beta V_L \quad (23)$$

The value of $\alpha = 0.55$ was obtained from error optimization for the design parameter domains appropriate for steel buildings, and it is thus not appropriate for bridges. However, the form of the equations for ϕ and γ suggests that these factors are dependent on the selected value of β (target value) and on the load and resistance statistics represented by the means and the coefficients of variation. The determination of ϕ and γ can proceed according to various schemes, such as by (a) iteration and recalibration (the Ontario code), (b) selection of one α through error optimization (LRFD for steel building structures), or (c) the methods used by Ellingwood (8) and Rackwitz (9).

PROBABILITY-BASED DESIGN FOR STEEL BRIDGES

The design of steel bridges can be accomplished by using load- and resistance-factor design (or limit-states design), which is characterized by a design criterion of the form of Equation 18, with resistance factors ϕ for each limit state and load factors γ for each load effect. Multiple equations are employed to account for the various load combinations. Through the judicious choice of the ϕ and γ factors it is possible to achieve uniform reliability through the whole domain of design parameters. These factors are most easily obtained by calibration and by using one of several ways in which first-order probabilistic principles can be applied (3, 8, 10).

The following steps are involved in the formulation of a unified bridge design specification:

1. Development of a rational load model (see the paper by Moses in this Record for further details on this subject), including the definition of consistent, realistic, and practically applicable concepts for design vehicles, load distribution, impact, overload, service load for vibration and fatigue, and load combination;
2. Collection and evaluation of the statistical data (distribution, if possible, but at least the mean and the coefficient of variation) for the loads due to the mass of the structure, the vehicles, and the environment;
3. Evaluation of the relevant statistical data for the resistance of the structural elements and members;
4. Development of the load and resistance factors through judgment by using calibration with the methods of first-order probability theory;
5. Development of design criteria for vibration, deflection, damage, fatigue, and brittle fracture by using first-order probability theory; and
6. Development of control methods for progressive collapse (e.g., a fracture control plan) by using first-order probability theory.

While it would appear that this is a formidable list of work to be done, it is evident to me that most of the background work has already been done and that it is necessary only to put it together. This is by no means a simple or an easy task, but the point to be emphasized is that it can be and has already been done (2, 3, 4, 12).

What is actually available for use? Moses, in his paper in this Record, has presented the current thinking on loads, load models, load statistics, and so on, and MacGregor, whose paper also appears in this Record, has summarized the statistical data on concrete structures. Probabilistic methodology, ranging from relatively simple to very sophisticated, exists and has been successfully used in developing LSD or LRFD codes (8-10).

Probability-based methods of design for fatigue and brittle fracture have also been developed (16) and applied (4). Statistical data on steel structures have been collected and evaluated and were used in the formulation of the LRFD criteria for steel structures (2).

Much prior research is thus available, and it is quite feasible to produce a probability-based bridge design code. Thanks to the prior introduction of LFD into the AASHTO code, the final new criteria will be quite familiar to the designer.

STATISTICAL DATA ON STEEL STRUCTURES

The relevant statistical data for steel structures are characterized by the mean and the standard deviation of the resistance R as follows:

$$R_m = R_n (P_m M_m F_m) \quad (24)$$

and

$$V_R = (V_P^2 + V_M^2 + V_F^2)^{1/2} \quad (25)$$

The available research literature was evaluated in connection with the development of LRFD criteria for steel building structures (2), and R_m and V_R were determined for beams (17), columns (10, 11, 13), beam-

Table 1. Member resistance statistics.

Member Type	Nominal Resistance	Resistance Value P_m	M_m	F_m	$P_mM_mF_m$	V_P	V_M	V_F	V_R
Composite Beam									
Solid slab	Plastic moment of composite section	0.99	1.07	1.0	1.06	0.08	0.10	0.05	0.14
Slab on formed steel deck	Plastic moment of composite section	1.01	1.07	1.0	1.08	0.08	0.10	0.05	0.14
Elastic beams	Lateral-torsional buckling	1.03	1.0	1.0	1.03	0.09	0.06	0.05	0.12
Inelastic beams	Lateral-torsional buckling	1.06	1.05	1.0	1.11	0.09	0.10	0.05	0.14
Plastic beams	Determinate beams, uniform moment, M_P	1.02	1.05	1.0	1.07	0.06	0.10	0.05	0.13
Plastic beams	Determinate beams, moment gradient, M_P	1.24	1.05	1.0	1.30	0.10	0.10	0.05	0.15
Plastic beams	Indeterminate beams, simple frames, M_P	1.06	1.05	1.0	1.11	0.07	0.10	0.06	0.13
Plate girders	Maximum moment capacity	1.03	1.05	1.0	1.08	0.05	0.10	0.05	0.12
Plate girders	Web shear capacity	1.03	1.10	1.0	1.13	0.11	0.11	0.05	0.16
Beam column	Ultimate capacity interaction equation	1.06	1.05	1.0	1.07	0.10	0.10	0.05	0.15

Table 2. Connector resistance statistics.

Connector Type	Nominal Resistance	Resistance Value P_m	M_m	F_m	$P_mM_mF_m$	V_P	V_M	V_F	V_R
Fillet welds	$0.6\ F_{EXX}\ A_w$	1.40	1.05	1.0	1.47	0.10	0.04	0.15	0.18
High-strength bolts, tension									
A325	$F_u\ A_{SA}$	1.0	1.20	1.0	1.20	0	0.07	0.05	0.09
A490	$F_u\ A_{SA}$	1.0	1.07	1.0	1.07	0	0.02	0.05	0.05
High-strength bolts, shear									
A325	$0.6\ F_u\ A_{SA}$	1.04	1.20	1.0	1.25	0.05	0.07	0.05	0.10
A490	$0.6\ F_u\ A_{SA}$	1.04	1.07	1.0	1.11	0.05	0.02	0.05	0.07
High-strength bolts, tension and shear									
A325	$S^2 + (0.6T)^2 = (0.6\ F_u\ A_{SA})$	1.05	1.20	1.0	1.26	0.10	0.07	0.05	0.13
A490	$S^2 + (0.6T)^2 = (0.6\ F_u\ A_{SA})$	1.05	1.07	1.0	1.12	0.10	0.02	0.05	0.11
A307 bolt, shear	$0.6\ F_u\ A_{SA}$	1.17	1.10	1.0	1.29	0.1	0.1	0.05	0.11

columns (18), connectors (19), composite beams (20), and plate girders (21). Statistical data on material properties were also collected and evaluated (22).

Typical results from these reports for members are presented in Table 1 and for connectors in Table 2. The material properties and their values (1 kPa = 0.145 lbf/in²) are listed below.

Property	Mean Value	Coefficient of Variation
Modulus of elasticity in tension and compression (kPa)	200 000	0.06
Modulus of elasticity in shear (kPa)	77 000	0.06
Poisson's ratio	0.30	0.03
F_y in flanges of W members	1.05	0.10
F_y in webs of W members	1.10	0.11
Yield stress in shear	0.64	0.10
Strain hardening modulus (kPa)	2800	0.25
Compressive residual stress in flanges of rolled sections (kPa)	70	0.5

These data derive from the research on steel structures and as such are directly usable for the development of the resistance factor ϕ and for calibration for steel bridges. Additional data could be drawn from research on steel bridges, especially fatigue data [Knab and others (4) have, for example, a collection of relevant fatigue information] and statistics from research on curved and box girder bridges.

SUMMARY AND CONCLUSIONS

This report has shown how relatively simple first-order probabilistic methods can be used to assess the reliability of the current AASHTO specification and to develop a consistent LRFD specification for steel bridge structures. It was demonstrated that the AASHTO LFD method provides a consistent reliability (safety) index but that the AASHTO ASD method does not (Figure 3).

It is concluded that there is sufficient statistical information on steel structures available to allow a probability-based design method to be developed. It is possible to derive resistance factors for members and connectors used in steel bridges. Furthermore, the probability-based method need not be significantly different in format and design use from the LFD method in the AASHTO specification.

ACKNOWLEDGMENT

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Discussion

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As Galambos points out in his paper, there are several steps to be undertaken in the development of a unified bridge design specification. Three possible problem areas need to be identified for a better appreciation of which investigations must be conducted and which decisions must be reached. The areas that need further clarification are (a) ratio of the mean maximum lifetime moment due to vehicles to the maximum moment due to the standard AASHTO vehicle (L_m), (b) ratio of the mean actual lateral load-distribution factor to the AASHTO lateral load-distribution factor (A_m), and (c) ratio of the mean actual dynamic amplification to the AASHTO impact factor (I_m).

Research on overlaoding of highway bridges has clearly demonstrated that, in view of the variability or illegality of overloads, configurations tend to vary substantially (23). Assuming that this is the desirable direction to take, the coefficient of variation that will account for this variability cannot be controlled unless definite measures are taken to classify the overload configurations and permit their growth in an orderly fashion.

The ratio of the mean actual lateral load-distribution factor to the AASHTO lateral load-distribution factor has already been found to show a great variation according to the bridge geometry. The inclusion of other variables, such as skew, will make V_A a very large quantity. Research has indicated that the AASHTO distribution factors are far from realistic (24, 25). Before the mean value and the variation that will account for the use of the distribution factor approach are established, interpretative research needs to be initiated and completed.

The dynamic amplification versus AASHTO impact-factor considerations require further clarifications. The field testing of simple-span prestressed concrete beam-slab highway bridges has indicated that the AASHTO impact factor provides a conservative estimate for the gross bridge behavior (26). However, if the dynamic amplification of individual beams is considered, then the AASHTO impact factor ceases to be conservative for all loading cases. It is therefore essential that specification writers clarify whether the impact factor is considered for the total superstructure in general or is also applicable to all components of the superstructure, one by one.

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Probabilistic Approaches to Bridge Design Loads

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Load uncertainties form a major part of the probabilistic basis for structural design. This paper is part of a state-of-the-art report on probabilistic design for bridges. The emphasis is on vehicle-induced loads for short- and medium-span bridges. Unlike environmental loads, vehicle loads can evolve according to imposed regulatory controls. Loads for railway bridges are relatively well controlled, and methods for deriving probabilistic load factors for repetitive spectra (fatigue) and maximum lifetime loading (strength) have been presented. Statistical data for highway bridges, however, are relatively sparse, especially for extreme vehicle weights and spacing configurations. Computer simulation and convolution models for deriving probabilistic load factors for strength are described, and safety indices for fatigue—including uncertainties in vehicle weights, truck dimensions, impact, headway, volume, girder distribution, and material fatigue life—are also presented. These probabilistic approaches are limited by the available data base and the continuing growth in truck loads. The paper also presents a system-reliability approach to model behavior beyond the simple element capacity check. Bridge damage versus load curves should be investigated for different geometries and configurations. Superimposing load and resistance probability distribution will provide damage costs for deriving optimum load factors. Because load growth would be partially absorbed in the bridge performance range between initial element failure and extreme damage, design ductility and redundancy need lower factors. Methods for establishing these goals are discussed.

An important part of a probabilistic approach to bridge safety is describing loads, which are caused by construction, erection, fabrication, settlement, temperature, traffic, and environmental forces. The AASHTO specification (1), for example, lists 10 loading combinations. Significant research under the general heading of structural reliability has described various load phenomena and different load and combination factors (2). For example, environmental forces such as wind, waves, earthquakes, and flooding are often described in terms of statistical parameters such as return periods (3). The major emphasis here will therefore be on a probabilistic framework and description of traffic loadings on bridges, especially short- and medium-span highway and railway bridges where the most uncertain load phenomenon is the live-load effect of traffic.

This paper describes the uncertainties of traffic loading and analyzes load spectra for fatigue and maximum lifetime load for a strength design. A careful distinction is made between load pattern and load factor. Both highway and railroad engineers have devised theoretical load patterns to provide envelopes for bending moment and shear that arise from existing or expected vehicle combinations. For example, AASHTO specifies a design vehicle with a variable wheel base intended to envelop moment and shear for simple and continuous spans over a wide range of lengths (1). Similarly, the theoretical Cooper loading is used for railway bridges to simplify the "bookkeeping" associated with checking strength and serviceability at various locations in a bridge. Other load patterns that provide better consistency for bending and shear response have also been invented for longer spans (4).

Load factors, however, as distinguished from the load pattern, are a quantity by which the calculated load effects are multiplied to provide safety margins that cover uncertainties such as high loads from heavy, closely spaced vehicles; approximations in the statistical analysis; and dynamic response. Further, as part of a reliability analysis, load factors can incorporate uncertainties in judgment, analysis, and failure consequences. A basic feature of an analysis of reliability or probabilistic safety is that load factors relate directly to load uncertainties that have different factors appropriate for dead load, live load, wind, and other forces. Reduction in loads is needed when a multiplicity of load effects combine. At the present stage of reliability applications, a measure of risk such as a safety index β is usually employed (5). For a simple check of strength capacity, the safety index can be written in the popular additive form as (6)

$$\beta = \text{mean safety margin/variance of safety margin} = (\bar{R} - \bar{S})/(\sigma_R^2 + \sigma_S^2)^{1/2} \qquad (1)$$

where $\bar{S}(\bar{R})$ and $\sigma_S(\sigma_R)$ are the mean and standard deviation of the load (strength) respectively. This format (2) gives rise to a load factor γ as

$$\gamma = 1 + \alpha\beta V_S \qquad (2)$$

where α is the separation factor arising from the square-root quantity in Equation 1 and V_s is the nondimensional load coefficient of variation (standard deviation divided by the mean). In code writing, each partial factor (load and resistance, or performance factor) is a function of its respective coefficient of variation V and a common safety index β that produces decreasing risk with increasing value. Load-combination factors may also be formulated (2) by extension

$$S = S_1 + S_2 + \ldots + S_i + \ldots + S_n \qquad (3)$$

where S_i is the ith load effect. Each load factor γ_i can be expressed as