Load- and Resistance-Factor Design of Concrete Highway Bridges

J. G. MacGregor, Department of Civil Engineering, University of Alberta, Edmonton

Load-factor design of concrete bridges has been included in the AASHTO specifications for highway bridges since 1973. This design method leads to savings in materials and a more uniform safety level for bridges. However, the serviceability limit states must be considered by designers. Five methods of specifying load and resistance factors are reviewed and compared, and a procedure for implementing a modern load- and resistance-factor design specification is presented. This procedure requires coordination between groups writing various parts of the design specification.

Since 1973 the AASHTO specifications for highway bridges (1) have allowed load-factor or strength design of concrete highway bridges. The load and resistance factors currently in use are based on older data (2, 3). During the nearly 20 years since the basic work on these factors was carried out, understanding of safety theory has increased immensely. As a result, it is now possible to derive load and resistance factors that will lead to more uniform safety than the factors currently in the AASHTO specifications.

This paper does not attempt to propose a specific set of load and resistance factors for a future bridge design specification. Rather, it reviews the advantages of load- and resistance-factor design and several possible ways of presenting these factors. A procedure for implementing them in the design of highway bridges is presented.

LIMIT-STATES DESIGN

Definitions

When a structure or structural element becomes unfit for its intended purpose it is said to have reached a limit state. For concrete highway bridges the limit states can be divided into two categories.

1. Ultimate limit states are related to a structural collapse of a member or a structure. Such a limit state should have a very low probability of occurrence because it may lead to loss of life and major financial losses such as (a) loss of equilibrium of part or all of the structure when considered as a rigid body (tipping or sliding), (b) rupture of critical parts of the structure leading to collapse (flexure, shear, bond), and (c) instability due to deformations of the structure (buckling).
2. Serviceability limit states are related to disruption of the functional use of the structure or damage to or deterioration of the structure. Since there is less danger of loss of life, a higher probability of occurrence may be tolerated than in the case of the ultimate limit states. For concrete highway bridges these can be subdivided into two further categories.

Type I. Serviceability limit states are limit states that are most critical under many repeated loadings close to the normal traffic load magnitude and include (a) fatigue and (b) vibration leading to discomfort of pedestrians on the bridge.

Type II. Serviceability limit states are limit states that result from an occasional heavy loading situation such as (a) excessive crack widths that lead to leaking of water through members and potential corrosion of reinforcement and (b) excessive deformation or permanent deformations that affect the appearance, riding quality, or drainage of the bridge.

For prestressed concrete bridges the cracking limit state may be restated in terms of a decompression limit state corresponding to zero stress in the least-compressed fiber and a crack-formation limit state corresponding to tensile stresses approaching the modulus of rupture of concrete. Thus, for example, the 1978 Model Code for Reinforced Concrete Structures (4) requires that the crack-formation limit state not be exceeded under frequent loads for members with reinforcement that is susceptible to corrosion (small-diameter bars and prestressing tendons) and members that are exposed to severe environments. In addition, the code requires that the cover to the reinforcement be increased in this case.

Load- and resistance-factor design is a process that involves

1. Identification of all modes of failure or ways in which the structure might fail to fulfill its intended purpose (limit states),
2. Determination of acceptable levels of safety against occurrence of each limit state, and
3. Consideration by the designer of the significant limit states.

In Europe and Canada this design process is referred to as limit-states design to emphasize the need to consider several limit states.

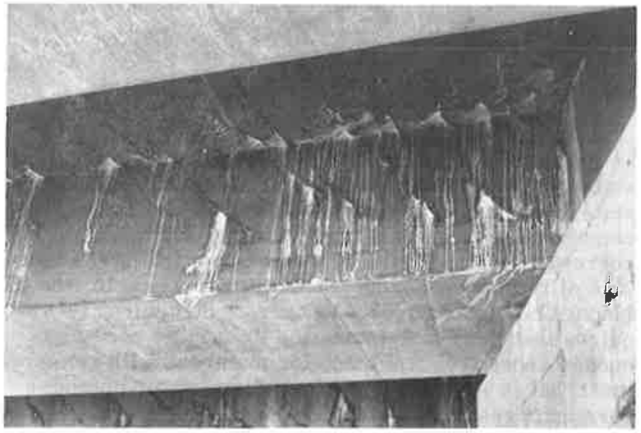
Reasons for Load- and Resistance-Factor Design

The load- and resistance-factor design procedure described in the previous section is, in effect, the traditional engineering design procedure formalized to require specific consideration of the various limit states. The three stages of the design process are emphasized in this paper because, all too often in the past, designers and specification writers have given their prime attention to the ultimate limit states and not enough to the factors that might render the bridge unsatisfactory in everyday use.

In the past, the use of allowable stresses led to combined factors of roughly $U/\phi = 2.5D + 2.5(L + I)$ for a beam with grade 60 steel. The 1977 AASHTO load and resistance factors correspond to $U/\phi = 1.44D + 2.41L$. The very high dead-load factor in working-stress design resulted in relatively low service-load stresses and a large overload capacity for live loads. This made serviceability a minor problem. The lower load factor for dead load in load- and resistance-factor or limit-states design leads to economy at the expense of an increased possibility of serviceability problems.

Figure 1 shows the underside of the negative moment region of a reinforced concrete box girder bridge with two spans, each roughly 30 m (100 ft). The white stains emanate from flexural and inclined cracks. This bridge was designed according to and satisfies the load-factor design provisions of the 1973 AASHTO specification. The bridge was opened to traffic in October several years ago and the photograph was taken the following summer. The owner of the bridge has expressed concern over the appearance of the bridge and the possibility of corrosion damage.

Figure 1. Stains from leakage through cracks in negative moment region of continuous box girder bridge.



In this region, the service dead-load moments were roughly four times the service live-load moments. As a result, the computed service dead-load stresses in the negative moment reinforcement would have been limited to about 131 MPa (19 000 lbf/in^2) if the bridge had been designed by allowable-stress design compared to computed service dead-load stresses of 193 MPa (28 000 lbf/in^2) in the bridge in question. Approximately 20 percent more reinforcement would have been required in an allowable-stress design. Similarly, an allowable-stress design would have required an additional 30 percent more stirrups than a load-factor design.

Both of these factors led to a savings in costs but also led to wider cracks at service loads than would have occurred in a bridge designed by allowable-stress design. It should be noted that the serviceability provisions in the 1977 AASHTO load-factor design section are far superior to the corresponding provisions in the 1973 specifications.

The major reason for presenting Figure 1 is to emphasize the fact that, because the ratio of dead to live load is high for concrete bridges, the use of load and resistance factors will lead to considerable savings. At the same time there is an increased need for explicit consideration of the serviceability limit states.

SAFETY PROVISIONS FOR STRUCTURAL DESIGN

Reasons for Safety Provisions

There are three basic reasons for including safety factors of some sort in structural design. First, the strengths of materials or elements may be lower than expected because of variability in material strengths, differences between the in situ strengths and the control strengths, geometrical errors in cross-sectional dimensions, errors in placement of reinforcement, errors in calculated strengths from assumptions in the design equations, and gross errors in design or construction. Second, overloads may be caused by variability in the loadings, uncertainties in calculation of load effects and load distributions, and gross errors in the structural analysis or in the control of loading on the bridge. Third, the consequences of failure will include replacement cost, potential loss of life, and costs to society in lost time or lost revenue. Frequently the consequences of failure will be reduced if the member that fails does so in a ductile manner that warns of the impending failure or if the structure is able to develop alternative load paths.

Basic Procedures for Setting Safety Margins

Three basic methods of defining structural safety will be reviewed very briefly to show their differences, strengths, and weaknesses. This will be followed by a review of the procedures currently used to define the safety of reinforced concrete structures in several codes and specifications.

Factor of Safety or Working-Stress Design Method

The factor of safety can be defined as

$$\text{Factor of safety} = \text{ultimate resistance/service load} = R/Q \qquad (1)$$

This implies that both of these quantities are well defined, each with a unique value. However, both the resistance (strength) R and the loads Q are variable, as shown schematically in Figure 2. As a result, Equation 1 lacks clarity. Two possible restatements are

$$\text{Central factor of safety} = \text{mean resistance/mean load} = \bar{R}/\bar{Q} \qquad (2)$$

and

$$\text{Nominal factor of safety} = \text{design resistance/service load} = R_d/Q_d \qquad (3)$$

where R_d is the capacity computed according to the design code and Q_d is the service load from the design specification. The intersection of the frequency diagrams for R and Q, shown as the shaded area in Figure 2, suggests that there is a probability that failure will occur under some combination of strength and load.

For reinforced concrete the factor of safety has been assumed to be related to the ratio of (a) yield strength of reinforcement to allowable steel stress or (b) concrete strength to allowable concrete stress.

This method of defining safety has three major drawbacks. It does not adequately account for the variability of loadings and resistances. Two extreme cases are illustrated in Figure 3. Although the central factor of safety is the same in both cases, case a has a very low probability of failure, as evidenced by the small overlap of the curves, while case b has a significantly higher probability of failure. In the traditional working-stress design of reinforced concrete, the greater variability of concrete compared to steel was recognized by using a higher factor of safety in ratio b than in ratio a above.

Another drawback is that it does not adequately account for variations in loadings that increase at different rates. A working-stress format assumes that all loadings will increase at the same rate. This is especially serious when dead loads and variable loads counteract each other. This problem has long been dealt with empirically in highway bridge codes by means of an overload provision that requires that the stresses from one double-weight truck not exceed 150 percent of the allowable stresses. There is no rational method of considering such things as type of failure or consequences of failure.

Maximum Probability of Failure Method

If R and Q represent the strengths and loads on a given structure, failure will occur if $Q > R$. Thus, the probability of failure is the probability that $Q > R$ or

$$P_f = P[(R - Q) < 0] \qquad (4)$$

We can define the new function $Y = R - Q$ that has a mean

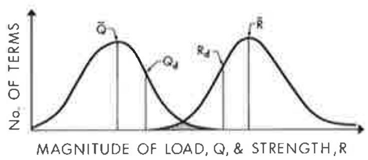


Figure 2. Variation of loads on and strengths of structures.

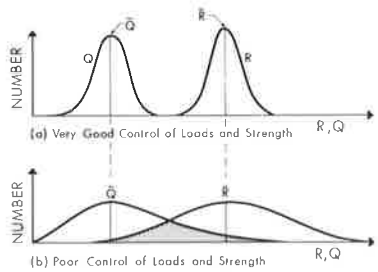


Figure 3. Effect of dispersion of loads and strengths on probability of failure for constant central factor of safety.

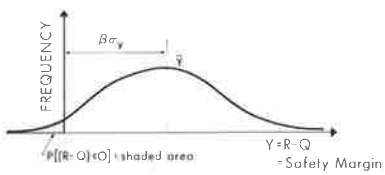


Figure 4. Definition of probability of failure and safety index.

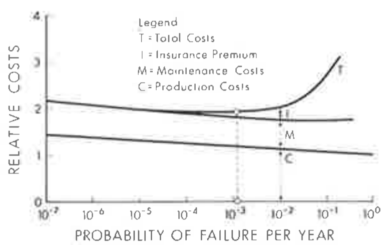


Figure 5. Evaluation of optimum (hypothetical) probability of failure.

of $\bar{Y}$ and a standard deviation σ_y. The probability of failure is represented by the shaded area in Figure 4. If the type of statistical distribution of Y is known, one can estimate the probability of failure from the number of standard deviations (β) by which the mean exceeds zero. For this reason β is called a safety index.

This procedure may be extended to give separate load factors and resistance factors for use in design and is the basis of current code proposals (5-7).

Minimum Cost Structure Including Cost of Failure

If one can estimate the probability of failure, then one can also calculate the total cost of a structure including the costs of failure (8). Thus, as shown in Figure 5, the total lifetime cost (T) of a bridge can be represented as the sum of the original construction costs (C), the maintenance costs (M), and the insurance costs (I). The insurance costs are related to the probability of failure and rise rapidly with it. Such a computation provides an estimate of the optimum probability of failure. Structural engineering design procedures based on this approach are still in their infancy.

Current Procedures for Defining Safety in Reinforced Concrete Building Codes

The safety provisions in the two leading reinforced concrete building codes are reviewed in this section. In both codes, design is based on load and resistance factors; the loads are amplified by load factors generally greater than one, and the strengths are reduced by resistance factors less than one. Thus, in both codes design is based on

$$\text{Factored resistance} \geq \Sigma \text{ factored load effects} \tag{5}$$

The two codes differ, however, in the manner in which the resistances are factored. The advantages of each approach are discussed briefly.

American Concrete Institute Building Code Requirements for 1963 Through 1977

The load factors and strength-reduction factors in the American Concrete Institute (ACI) code (2) were based on the assumption that, if the probability of understrength members is roughly 1 in 100 and the probability of overload is roughly 1 in 1000, the probability of an overload on an understrength member would be about 1 in 100 000. Load factors were derived to yield a probability of overload of 1 in 1000 for assumed building type loadings. After some adjustment by the ACI code committee, the basic ultimate load combination was stated as

$$U = 1.4D + 1.7L \tag{6}$$

By using values of concrete and steel strengths that correspond to a probability of 1 in 100 of understrength, the strengths of a number of typical sections were computed. The ratio of these strengths to those computed by using the nominal strengths used by the designer was called the strength-reduction factor (ϕ).

Because the consequences of a column failure in a building tend to be greater than the consequences of a beam or slab failure, the ϕ factor for columns was divided by 1.1. For tied columns the resulting ϕ factor was divided by 1.1 a second time to recognize the brittle mode of failure of tied columns compared to beams or spiral columns.

Thus, the ACI code's safety provisions represent an early (1958-1963) attempt to account for the possibility of understrength and overloads as well as the consequences of failure and mode of failure. The history of these provisions and a brief discussion of their statistical derivation are presented by MacGregor (7).

Comité Euro-International du Béton Model Code for Concrete Structures for 1978

The load factors and strength provisions in the Comité Euro-International du Béton code (CEB) (4) are based on a much more accurate and elaborate probabilistic analysis and on much better statistical data about loads and strengths than the ACI code was. This reflects the development of load-factor theory in the 15 years between 1963 and 1978.

In the CEB code, the load factors are roughly similar in format to those in the ACI code. On the other hand, the provisions for understrength are completely different. The strength of a cross section is computed by using design material strengths equal to f'_c/γ_c and f_y/γ_s, where γ_c and γ_s are material understrength factors with values of 1.5 and 1.15, respectively, for average construction quality. These values correspond roughly to the 1 in 1000 strengths of concrete and steel.

If the anticipated dimensional tolerances exceed normal practice, the designer is asked to reduce the effective depths used in calculations by the difference between the anticipated and the normal tolerances. Although there is provision in the CEB system for recognizing the consequences of failure or mode of failure, there is no intention at present to include these effects.

Comparison of ACI and CEB Safety Provisions

There is a major difference in philosophy between the ACI and CEB procedures for defining resistance factors. The ACI combines all the member understrength effects into one term (ϕ), which is intended to account for the effects of material understrengths, geometrical errors, consequences of member failure, and mode and warning of failure. This formulation would also allow consideration of errors in the design equations, although these were not specifically considered in deriving the ACI ϕ factors.

On the other hand, the CEB defines individual material understrength factors for concrete and steel. These are intended to account for the variability of material strength and the potential geometric errors associated with that material. This formulation implicitly recognizes that concrete strength varies more than steel strength. Thus, in the case of an eccentrically loaded column, for example, there is a relatively smooth transition from the situation where concrete controls the strength (small eccentricities) to the situation where steel controls the strength (large eccentricities). The ACI code has an awkward and empirical transition from one ϕ factor to another in this case.

The CEB safety provisions, as currently implemented, do not account for varying consequences of failure, different modes of failure (except as these are accounted for in the individual material understrength factors), or variability inherent in the design equations themselves.

To offset the drawbacks of these two individual methods, it would appear desirable to combine the best aspects of the ACI and CEB methods. Thus, design could be based on reduced material strengths f_c'/γ_c and f_y/γ_s to account for material variability and the brittle or ductile mode of failures resulting from crushing of concrete or yielding of steel, plus a ϕ factor to allow for equation error, consequences of failure, and so on. In such a procedure the material understrength factors for concrete and reinforcement would be roughly 1.50 and 1.15, as in the CEB, with a ϕ factor of 1.00 for flexure and possibly 0.90-0.95 for columns and shear.

Limit-States Design in Concrete Bridge Design Specifications

The safety provisions for concrete bridges in the 1977 AASHTO and the 1977 ACI-443 bridge specifications will be compared to the 1978 draft of the Ontario bridge design code. The safety provisions in the latter code differ somewhat in format and values from those in AASHTO or ACI-443.

AASHTO Standard Specifications for Highway Bridges

The AASHTO bridge specifications (1) allow service-load design and load-factor design of reinforced concrete bridges but only load-factor design for prestressed concrete bridges. The individual AASHTO load factors and coefficients are given in the specifications (1, Table 1.2.22). These are combined by using Equation 7 to get the corresponding loads for each design-load group.

$$\begin{aligned}\text{Group (N) load} = \alpha[&\beta_D D + \beta_L(L + I) + \beta_C CF + \beta_E E + \beta_B B + \beta_S SF \\ &+ \beta_W W + \beta_{WL} WL + \beta_L LF + \beta_R(R + S + T) \\ &+ \beta_{EQ} EQ + \beta_{ICE} ICE] \end{aligned} \quad (7)$$

where

N = group number,
α = load factor given in AASHTO specifications,
β = coefficient given in AASHTO specifications, and
D, L, etc. = dead, live, and other loading cases.

The basic load-factor case for design of a straight bridge superstructure is generally the group 1 load:

$$1.3[D + 1.67(L + I)] = 1.3D + 2.17(L + I) \quad (8)$$

The member strengths are multiplied by the following strength reduction factors (ϕ) for reinforced and prestressed concrete.

Concrete Type	Stress	ϕ Factor
Reinforced	Flexure	0.90
	Shear	0.85
	Spiral columns	0.75 increasing to 0.90
	Tied columns	0.70 increasing to 0.90
	Bearing	0.70
Prestressed	Flexure	
	Factory-produced precast prestressed members	1.0
	Post-tensioned cast-in-place members	0.95
	Shear	0.90

The load-factor design of reinforced concrete bridges in the 1977 AASHTO specifications grew out of the Strength and Serviceability Criteria, Reinforced Concrete Bridge Members, Ultimate Design, published in 1966 (3). This document adopted the ACI code ϕ factors and algebraically adjusted the load factors to give essentially the same safety factor for short-beam and slab spans as provided by the working stresses for reinforcement.

For reinforced concrete members designed according to the load-factor procedures, three serviceability checks must be made; one must consider

1. The possibility of fatigue failures,
2. The steel distribution to limit flexural crack widths under service loads, and
3. Deflections, either by limiting the span to depth ratio or by calculations. No limiting values are given.

It is not necessary to consider bridge vibrations.

In the case of prestressed bridges, the AASHTO specifications use the load factors presented earlier and use the more liberal resistance factors (ϕ) listed earlier. I am not aware of any published algebraic or statistical study justifying these ϕ factors. No serviceability requirements are given for prestressed concrete bridges.

ACI-443 Report

In 1977 ACI committee 443 issued the report Analysis and Design of Reinforced Bridge Structures (9). This report embodied essentially the same load factors as the 1977 AASHTO specifications, and the resistance factors for reinforced concrete were also the same as in AASHTO. For prestressed concrete, however, the ϕ factors in the ACI-443 report are

Stress	ϕ Factor
Bending	
Adequate quality control	0.95
Otherwise	0.90
Shear	0.90
Prestressed spiral columns	0.80
Prestressed tied columns	0.75
Bearing	0.75

Deflection and fatigue requirements are specified both for prestressed and reinforced concrete bridges, and crack-control provisions are specified for reinforced concrete bridges.

Ontario Highway Bridge Design Code

In preparation for metric bridge design, the Ministry of Transportation and Communications of the province of Ontario decided in 1976 to write a completely revised bridge design code (10). This was based on three major innovations. First, a new design-load scheme based on extensive truck-load surveys was incorporated. Second, new impact and bridge-vibration design rules were presented that were based on analyses and field measurements. The third and most important concept was the decision to present the entire code in a limit-states format by using probabilistically derived load and resistance factors.

The basic design section of the reinforced concrete section of the code for limit states in general says that "Structural components shall be proportioned to satisfy both the ultimate limit states and serviceability limit states requirements," that "The ultimate limit states to be considered shall be those of strength and stability," and that "The serviceability limit states to be considered shall be those of cracking, fatigue and deformation." For ultimate limit states the code requires that "Structural components shall be proportioned to have factored resistances which are greater than or equal to the force effects due to the factored loads as specified in Section 2."

The basic design equation for checking the ultimate limit states can be stated as:

$$\phi R/\gamma \geq (1.1D_1 + 1.2D_2 + 1.5D_3 + 1.35D_4) + (1.0P_p + 1.05P_s) + \psi(1.35L + 1.35I + \ldots) \quad (9)$$

where

- ϕ = capacity-reduction factor,
- R = strength computed by designer,
- γ = member or component importance factor taken to be equal to 1.0 (except that if, neglecting the resistance of this component, the remaining components of the structure are unable to support the dead loads, γ is taken as 1.1),
- D_1 = dead load of factory-produced structural components,
- D_2 = dead load of cast-in-field structural components and all nonstructural components other than a superimposed wearing surface,
- D_3 = dead load of superimposed wearing surface,
- D_4 = dead load of earth fill,
- P_p, P_s = primary and secondary forces due to prestressing,
- ψ = load-combination factor with values from 0.7 to 1.0 to reflect the probability of occurrence of certain live-load combinations,
- L = live load [the basic live load is a 676-kN (152 000-lbf) truck with five axles over a length of 18.3 m (60 ft)], and
- I = impact (the impact load varies from 0.15L to 0.45L as a function of the type of member, number of lanes loaded, and natural period of vibration).

Several new concepts are embodied in this design equation. Among them are the following.

1. The term γ provides additional safety against the failure of an essential member but does not penalize all columns, for example, as the ACI code does.
2. The load factors for dead load differ according to the variability of the dead load in question. Thus, the variability and load factor for asphalt are considerably higher than those for cast-in-place concrete.
3. Load factors are specified for prestressing forces.
4. With two exceptions, earth load and wind load, the load factor for a given load is constant. The differing probabilities of occurrence of various load combinations are accounted for by load-combination factors (ψ), which reduce the sums of the variable loads.
5. The load factors on live load and impact are relatively low but are based on much heavier design-vehicle loads, which offset this effect to some extent.

The factors for capacity reduction are lower than those in the AASHTO specifications, partly as a result of lower load factors. The following values are used for reinforced concrete.

Concrete Type	Stress	ϕ Factor
Reinforced	Flexure	0.80
	Shear and torsion	0.60
	Compression with flexure	0.60 increasing to 0.80
	Piles	0.60
	Bearing	0.60
Prestressed	Flexure	
	Cast-in-place members to 46 cm (18 in) deep	0.80
	Pretensioned members and cast-in-place members deeper than 46 cm	0.85
	Members of $\omega > 0.30$	0.60
	Shear and torsion	0.60
	Compression with flexure	0.60 increasing to value for flexure

IMPLEMENTATION OF A LOAD- AND RESISTANCE-FACTOR DESIGN SPECIFICATION FOR HIGHWAY BRIDGES

The following list is a series of decisions that must be made before a true limit-states code can be introduced for bridge design.

1. Scope: The first step in the derivation of a limit-states design code is to define the scope, that is, the class of structures it is to govern. This, presumably, should be for highway bridges. But should there be any limitation as to type of material, type of loading (normal highway loads versus abnormally heavy vehicles), span length, and so on?
2. Definition of limit states: The limit states to be considered should be specified as done in the Ontario code. In addition, it is necessary that a concise definition of each limit state be developed. Although the ultimate limit states are relatively well defined, this is not true for serviceability limit states. Thus, is a deflection of L/960 excessive? What angle change can be tolerated between adjacent members at supports? A more fundamental approach is to determine why it is necessary to limit deflections or angle changes. Is it done for aes-

thetic reasons or to improve bridge drainage or to avoid exciting the bouncing of vehicles close to the natural frequency of the vehicles?

Similarly, what crack widths can actually be tolerated? For example, the CEB code limits crack widths to 0.020 cm (0.008 in) for reinforced concrete with exterior exposure, not exposed to deicing chemicals. The AASHTO, ACI, and Ontario codes all allow 0.26 cm (0.011 in) for a similar exposure condition. Again, it is necessary to return to the basic question of why it is necessary to limit crack widths before arbitrarily selecting a width to be specified. Vibration of bridge superstructures can be approached in a manner similar to that of the Ontario code.

3. Determination of variability of resistances: Based on data on the variability of concrete, reinforcement, and dimensions as well as on data on the accuracy of calculation equations, it is possible to determine the variability of concrete members loaded in flexure, shear, axial compression, and so forth. Extensive research on this problem has already been carried out for concrete members. Additional research is under way at the University of Alberta and the National Bureau of Standards (11).

4. Definition of loads: The loads to be listed in the specification should be studied and documented. The manner in which these loads are described must be chosen. Thus, should the specified loads represent the mean lifetime maximum load, the mean maximum daily load, or some other load level? Data on the statistical distributions of these loads are required. Probabilistic studies of bridge loadings are discussed by Moses in another paper in this Record.

5. Selection of code format: Current design practice in North America and Europe is moving toward design codes that use load and resistance factors. The basic equation for design is Equation 5. Four possible formats for the left side of this equation have been presented. These are (a) the ACI format with a single resistance factor (ϕ) for all flexural failures, another for all shear failures, etc.; (b) the Ontario bridge design code format with ϕ/γ, which allows an increase in the level of safety if the consequences of a member or component failure are severe; (c) the CEB format, which is based on design strengths equal to $f'_c/1.5$ and $f_y/1.15$ without any ϕ factors as such; and (d) a combination of the above three methods that involves the use of design strengths and, in a few cases, ϕ factors.

Similarly, two possible formats have been presented for the right side: (a) the AASHTO load-factor matrix and (b) the Ontario system of separate unique factors for each type of load plus load-combination factors.

It is important that the format and values of the load factors be similar, if not identical, for all materials covered by a specification. It is desirable, but not essential, that the format of the capacity-reduction or resistance factors be similar for all materials.

6. Selection of the basic methodology to be used in computing load and resistance factors: The first stage in the selection of the safety computation methodology is to define the code objective. This may range from minimum overall cost including cost of failure for the class of structures considered, at one extreme, to a constant safety index (β) for all structural members, on the other. Once this objective has been selected, there will be several possible computational procedures among which to choose. For consistency, the same procedure should be used to derive the load and resistance factors for steel, concrete, and prestressed concrete bridges.

7. Selection of target reliability: This is generally approached from two points of view. The first involves using the methodology selected in step 6 to calculate the reliability attained in the design codes currently in use. This is called calibration. The results are generally expressed in terms of β. The second procedure involves estimating a probability of failure that would be acceptable to society and working backward to compute acceptable values of the target β. The results of these two procedures should be considered when selecting a target safety index for a code.

8. Testing of final results: Once load and resistance factors have been derived, they should be tested by using comparative designs to determine the changes introduced by the new factors. If these changes can be rationalized, they may be acceptable; if not, more study is required.

The procedure described above implicitly requires cooperation and interrelated effort by groups working on loads and on design specifications for concrete, prestressed concrete, and steel structures. This work must be coordinated by a relatively small, technically competent committee responsible to some agency such as AASHTO or the Federal Highway Administration. The Ontario bridge design code grew out of an integrated group of this kind. What must be avoided, if at all possible, is independent action by the concrete, steel, and timber communities that results in a multitude of self-serving code formats and philosophies.

SUMMARY AND CONCLUSIONS

Load- and resistance-factor design results in more uniform level of safety in bridge design. Because the excess dead-load capacity is reduced, it is possible to achieve considerable savings in amounts of reinforcement, compared to allowable stress design. On the other hand, the serviceability limit states become more critical. Thus, the designer must consider both ultimate limit states and serviceability limit states.

Several current code formats were reviewed to show the possible ways of presenting safety provisions in a bridge design specification. A procedure for implementing a modern load- and resistance-factor design specification is reviewed. The development of such a specification must be closely supervised by a group sufficiently aware of safety theory to realize the effects of their decisions and sufficiently representative to be able to lead all parts of the bridge design industry to a common philosophy and approach to safety problems.

REFERENCES

1. Standard Specifications for Highway Bridges. AASHTO, 12th Ed., 1977, 496 pp.
2. Building Code Requirements for Reinforced Concrete. Committee 318, ACI, Detroit, 1977, 102 pp.
3. E. L. Erickson and E. G. Paulet. Strength and Serviceability Criteria, Reinforced Concrete Bridge Members, Ultimate Design. Bureau of Public Roads, U.S. Department of Commerce, 1966.
4. Model Code for Reinforced Concrete Structures. Comité Euro-International du Béton, Paris, 1978, 300 pp.
5. C. A. Cornell. A Probability-Based Structural Code. Proc., ACI, Vol. 66, Dec. 1969, pp. 974-985.
6. W. W. C. Siu, S. R. Parimi, and N. C. Lind. Practical Approach to Code Calibration. Journal of Structural Division, Proc., ASCE, Vol. 101, No. ST7, July 1975, pp. 1469-1480.
7. J. G. MacGregor. Safety and Limit States Design for Reinforced Concrete. Canadian Journal of Civil Engineering, Vol. 3, No. 4, Dec. 1976, pp. 484-513.
8. H. Rüsch and R. Rackwitz. The Significance of the Concept of Probability of Failure as Applied to the Theory of Structural Safety. <u>In</u> Development—De-

sign—Construction: Commemorative Publication for the 100th Anniversary of Held and Franke Construction Corporation, Munich, 1972.
9. Analysis and Design of Reinforced Concrete Bridge Structures. Committee 443, ACI, Detroit, 1977, 116 pp.
10. Ontario Highway Bridge Design Code Draft for Public Comment. Ontario Ministry of Transportation and Communications, Toronto, Spring 1978, 460 pp.
11. B. Ellingwood. Reliability Basis of Load and Resistance Factors for Reinforced Concrete Design. National Bureau of Standards, Washington, DC, NBS Building Science Series 110, 1978, 95 pp.

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).

REFERENCES

12. C. N. Kostem. Overloading of Highway Bridges—A Parametric Study. Fritz Engineering Laboratory, Lehigh Univ., Bethlehem, PA, Rept. 378B.7, Aug. 1976.
13. C. N. Kostem. Overloading of Beam-Slab Type Highway Bridges. Fritz Engineering Laboratory, Lehigh Univ., Bethlehem, PA, Rept. 378B.8, July 1977.

Publication of this paper sponsored by Committee on Concrete Bridges.

Probabilistic Approaches to the Design of Steel Bridges

Theodore V. Galambos, Department of Civil Engineering, Washington University, St. Louis

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 se-