Load and Resistance Factor Design

THEODORE V. GALAMBOS

Load and Resistance Factor Design, abbreviated as LRFD, is a scheme of designing steel structures and structural components which is different from the traditionally used allowable stress format, as can be seen by comparing the following two inequalities:

$$R_n/F.S. \geq \sum_{1}^{i} Q_{ni} \quad (1)$$

$$\phi R_n \geq \sum_{1}^{i} \gamma_i Q_{ni} \quad (2)$$

The first of these inequalities represents the allowable stress case, while the second represents the LRFD design criterion. The left side in each case is the *design strength*, and the right is the *required strength*.* The term R_n defines the *normal strength* as given by an equation in a specification, and Q_{ni} is the *load effect* (i.e., a computed stress or a force such as bending moment, shear force axial force, etc.) determined by structural analysis for the loads acting on the structure (e.g., live load, dead load, wind load, etc.). The term *F.S.* represents the *Factor of Safety*, ϕ is termed the *resistance factor*, and the γ_i's are the *load factors* associated with each load effect Q_i.

The coefficients $F.S. > 1.0$, $\phi < 1.0$, and $\gamma_i > 1.0$ all serve the same purpose; they account for the uncertainties inherent in the determination of the nominal strength and the load effects due to natural variation in the loads, the material properties, the accuracy of the theory, the precision of the analysis, etc.

The fundamental difference between LRFD and the allowable stress design method is, then, that the latter employs one factor (i.e., the Factor of Safety), while the former uses one factor with the resistance and one factor each for the different load effect types. LRFD, by employing more factors, recognizes the fact that, for example, beam theory is more accurate than column theory (e.g., in Ref. 1, $\phi = 0.85$ for beams and $\phi = 0.75$ for columns), or that the uncertainties of the dead load are smaller than those of the live load (e.g., in Ref. 2, $\gamma_D = 1.2$ and $\gamma_L = 1.6$). LRFD thus has the potential of providing more consistency, simply because it uses more than one factor.

The purpose of this paper is to describe the development of an LRFD specification for steel structures.

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* *The terms in italics in this paragraph are the adopted terms used in Refs. 1 and 2.*

SIMPLIFIED PROBABILISTIC MODEL

The strength R of a structural member and the load effect Q are both random parameters, since their actual values cannot be determined with certainty (Fig. 1). The strength of a structure, often referred to also as its *resistance*, is defined in a popular sense as the maximum force that it can sustain before it fails. Since *failure* is a term that is associated with *collapse*, it is more useful, in the context of structural behavior, to define strength as the force under which a clearly defined *limit state* is attained. Such limit states are, for example, the plastic mechanism, the plastic moment, the overall or component buckling load, fracture, fatigue, deflection, vibration, etc. Not all of these limit states cause "collapse" in the popular sense, and so it is appropriate to define strength as "the limit state which determines the boundary of structural usefulness."

Structural behavior is thus satisfactory if $Q \leq R$, while on the contrary, $Q > R$ is unacceptable. Since Q and R are random, it is theoretically not possible to state with certainty

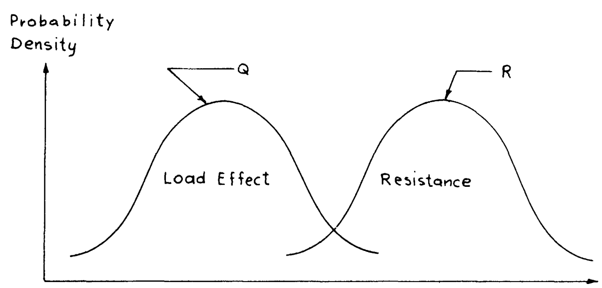

Fig. 1. Probabilistic description of Q and R

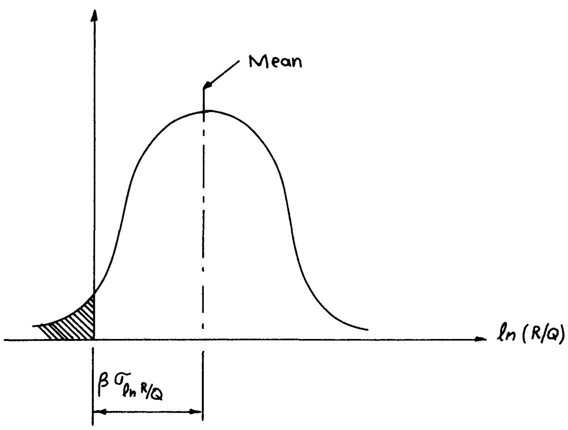

Fig. 2. Definition of the Reliability Index

for any structure that $Q \leq R$. Even for the most carefully designed and constructed structure there is a small but finite chance that $Q > R$, i.e., that the limit state can be exceeded. A satisfactory structural design specification is one which minimizes this chance to an acceptably low level.

It is possible, by using a simplified probabilistic approach, to quantify the statistical parameters which describe the probability of exceeding a limit state. Figure 2 is an identical representation of Fig. 1, using as the abscissa the ratio $ln(R/Q)$. When $ln(R/Q) < 1$, the limit state has been exceeded, and the shaded area in Fig. 2 is the probability of this event. Since the probabilistic distributions of R and Q are not known very precisely, a method has been devised which operates only with the mean and the standard deviation of the random parameters.[3] This method is called the *First-Order Second-Moment* probabilistic analysis.[4] More refined methods, which also take into account the distribution, are also available and have been used (Ref. 4) in the actual development of the load factors proposed in Ref. 2.

According to the first-order probabilistic method, one can determine a *reliability index* β:[5]

$$\beta = \frac{ln(\overline{R}/\overline{Q})}{\sqrt{V_R{}^2 + V_Q{}^2}} \tag{3}$$

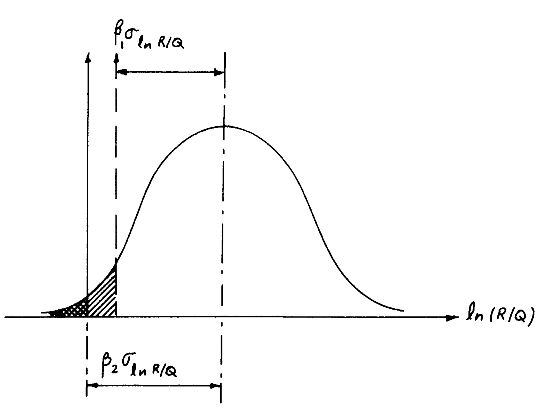

Fig. 3. Comparative description of the Reliability Index

where $\overline{R}$ and $\overline{Q}$ are the mean values and V_R and V_Q are the coefficients of variation ($V_R = \sigma_R/\overline{R}$, $V_Q = \sigma_Q/\overline{Q}$, where σ denotes the standard deviation; see Ref. 6) of R and Q, respectively. It can be seen in Fig. 2 that the magnitude of β effectively positions the coordinates with respect to the distribution curve. When β is larger, the probability of exceeding the limit state is smaller, i.e., the "reliability" is increased; the converse is true when β decreases (see Fig. 3).

The reliability index β can thus serve as a comparative measure of reliability between various design methods, types of members, and types of loading, and it has generally been preferred to use β, rather than the probability of exceeding the limit state, in developing the new LRFD specifications (Refs. 1, 4, 7, 8, for example). Typical values of β encountered range from 2 to 6, each increase of one unit corresponding very roughly to one order of magnitude of decrease in the probability of exceeding a limit state.

EXAMPLE OF THE DETERMINATION OF THE RELIABILITY INDEX

The use of Eq. (3) will be illustrated next. This is not a design office exercise (more will be said about that later), but a scheme whereby actual designs were "calibrated" prior to the development of the resistance and load factors which are to be used in the design office.

A *compact* two-span continuous beam will be used for illustration.

The mean strength and the coefficient of variation of a compact beam is equal to:[5]

$$\overline{R} = R_n\ (\overline{P}\,\overline{M}\,\overline{F}) \tag{4}$$

$$V_R = (V_R{}^2 + V_M{}^2 + V_F{}^2)^{1/2} \tag{5}$$

The term R_n is the nominal strength

$$R_n = M_p = Z_x\, F_y \tag{6}$$

where M_P is the nominal plastic moment based on the handbook value of the plastic section modulus Z_x and the specified yield stress F_y.

The coefficients $\overline{P}$, $\overline{M}$, $\overline{F}$ (representing abbreviations for Professional, Material, Fabrication) are mean values of the following random parameter ratios:

P = test/prediction
M = actual static yield stress/specified yield stress
F = actual Z_x/handbook Z_x

All available data were examined and, based on the interpretation of these data, it was decided that the following statistical values appropriately represent the total population of compact steel beams:

$\overline{P} = 1.06$, $V_p = 0.07$ (41 indeterminate beam tests, Ref. 9)
$\overline{M} = 1.05$, $V_M = 0.10$ (Ref. 10)
$\overline{F} = 1.00$, $V_F = 0.05$ (Ref. 5)

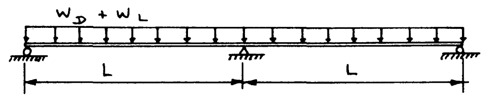

Fig. 4. Two-span beam example

Obviously there is a certain element of judgment involved in these values, but they represent the best that a number of experienced people could come up with on the basis of the available information. Substitution of these values into Eqs. (4) and (5) gives $\overline{R} = 1.17 Z_x F_y$ and $V_R = 0.13$.

Plastic analysis of the beam in Fig. 4 gives:[11]

$$M_P = \frac{(w_D + w_L)L^2}{11.66} \tag{7}$$

The right side of Eq. (7) is the load effect Q. It consists of the dead and the live load effect, and it can be written as:

$$Q = Q_D + Q_L \tag{8}$$

The mean and the standard deviation are then:[6]

$$\overline{Q} = \overline{Q}_D + \overline{Q}_L \tag{9}$$

$$V_Q = (V_D{}^2 \overline{Q}_D{}^2 + V_L{}^2 \overline{Q}_L{}^2)^{1/2} / (\overline{Q}_D + \overline{Q}_L) \tag{10}$$

A study of the load effect statistics (Ref. 2) gives the following values:

$$\overline{Q}_D = 1.05 Q_{nD}\ , \quad V_D = 0.10 \tag{11}$$

$$\overline{Q}_L = Q_{nL}\ , \quad V_L = 0.25 \tag{12}$$

where the subscript n relates to the nominal code-specified dead and live loads given in Ref. 2. The statistics include the uncertainties due to the loads themselves, as well as the effects of translating the loads into the idealized load effects and the idealization inherent in the uniform distribution. The occupancy live load Q_{nL} is equal to:[2]

$$Q_{nL} = Q_{oL}\,[0.25 + 15/\sqrt{A_I}] \tag{13}$$

where Q_{oL} is the load effect due to the basic code specified uniformly distributed live load intensity, and the bracket is a *live load reduction factor* dependent on the *influence area* A_I ($A_I = 2A_T$ and $A_I = 4A_T$ for beams and columns, respectively, where A_T is the *tributary area*).

The following data are specified for the given problem:

Dead load intensity: 57.5 psf
Basic live load intensity: 50 psf
Beam length between centers of supports: 28 ft
Beam spacing: 32 ft

For the tributary area of $28 \times 32 = 896$ ft^2, the reduced live load is 30 psf, and

$$\overline{Q}_D = \frac{28^2(1.05 \times 57.5 \times 32)(12)}{11.66 \times 1000} = 1559 \text{ kip-in.}$$

$\overline{Q}_L = 755$ kip-in.; $\overline{Q} = 2333$ kip-in.; $V_Q = 0.11$

Design according to the 1978 AISC Specification, Part 1

$w_D = 57.5 \times 32 = 1840$ plf

Live load reduction for $A_T = 896$ ft^2 and $D/L = 57.5/50$: 0.49

$w_L = 50(1 - 0.49)(32) = 809$ plf

$M_{max} = wL^2/8 = 3115$ kip-in. (at center support)

$$S_{req} = \frac{0.9 \times 3115}{0.66 \times 36} = 118 \text{ in.}^3$$

Use W21x62, $S_x = 127$ in.^{3}; $Z_x = 144$ in.^{3}

Thus $\overline{R} = 1.17 \times 144 \times 36 = 6065$ kip-in.

With this value of $\overline{R}$, and with $V_R = 0.13$, $\overline{Q} = 2333$ kip-in. and $V_Q = 0.11$, the reliability index [Eq. (3)] is: $\beta = 5.6$.

Design according to the 1978 AISC Specification, Part 2

$w_D = 1.7 \times 1840 = 3128$ plf

$w_L = 1.7 \times 809 = 1387$ plf

$$Z_{req} = \frac{(4.515 \times 28^2)(12)}{11.66 \times 36} = 101 \text{ in.}^3$$

Use W18x50, $Z_x = 101$ in.^{3}

The resulting β is 3.5.

Design according to the proposed AISC LRFD criteria (Ref. 1)

The design criterion is Eq. (2).

$$\phi R_n \geq 1.2 Q_{nD} + 1.6 Q_{nL} \tag{14}$$

$\phi = 0.85$ for beams

$W_D = 1.2 \times 57.5 \times 32 = 2208$ plf

$W_L = 1.6 \times 30 \times 32 = 1536$ plf

$$Z_{req} = \frac{3.744 \times 28^2 \times 12}{11.66 \times 36 \times 0.85} = 99 \text{ in.}^3$$

Use W18x50: $Z_x = 101$ in.^{3}

The resulting β is 3.5.

This simple exercise showed how the relative reliability of several designs can be compared and that the design of this particular beam according to Part 1 of the AISC Specification is conservative in comparison to the designs according to Part 2 and the LRFD method. Similar studies of thousands of different structure elements form the basis of the LRFD criteria.

RESEARCH ON THE DEVELOPMENT OF LRFD

The ideas of multiple factors and of probabilistic approaches to design are not new. No precisely documented historical account is intended here (see Refs. 4, 5, and 12 for such a treatise), but a brief general discussion will be provided to put the present status of LRFD into focus.

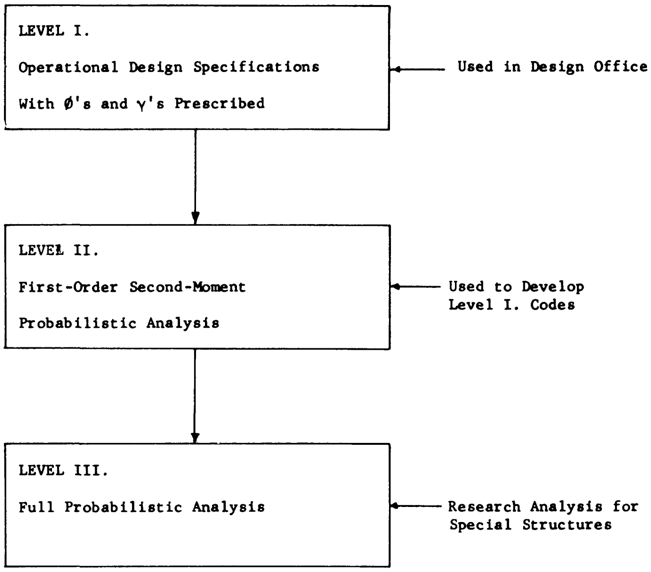

Fig. 5. Hierarchies of probabilistic methods

Designers were always aware of the uncertainties inherent in the design process, and this explains the evolution of the "factor of safety." Experience with aircraft design during World War II indicated that the probabilistic nature of the design parameters could somehow be quantified. The basic notions of this quantitative probabilistic approach were formulated in the 1950's by a series of papers authored by Freudenthal, and the premises advanced in these papers still hold true today.[13]

At the same time the idea of using multiple factors was suggested in England by Pugsley,[14] and a specification using them was formulated in the Soviet Union. Simultaneously, discussions within the American Concrete Institute eventually resulted in the familiar ACI ultimate design method, with its ϕ-factors and its load factors.

By the beginning of the 1960's, then, there was available a comprehensive probabilistic theory, and there were a number of design codes in use which used multiple factors arrived at by intuitive means. The 1960's and early 1970's brought about first the simple, and then an increasingly more sophisticated, First-Order Second-Moment method, whereby multiple design factors could be generated from statistical data and using probabilistic theory. There were many contributors to this development, e.g., J. R. Benjamin, C. A. Cornell, N. C. Lind, A. Ang, E. Basler, N. Ditlevsen, to mention only a few, and the first design specification for steel structures based on this approach was issued in 1974 in Canada.[8] At present (1981) the various approaches are ordered into hierarchies, as shown in Fig. 5. Many countries and regions of the world are presently either adopting or considering the adoption of LRFD type codes.

In the U.S. the development of LRFD for steel structures also has its origins in plastic design for buildings,[11] and in the formulation of Load Factor Design for steel bridges.[15] Research on the LRFD method for steel building structures, intended as a companion to the AISC Specification, started in 1969 at Washington University, and by 1978 a draft of such a specification was published.[16] These tentative LRFD criteria had also been tested in several design offices.[17]

It became obvious early in the LRFD research that the major difficulty, once the method and the format were agreed upon, was the treatment of the loads and the decisions on the magnitudes of the load factors. There was a clear need for a broader stance on this matter than could

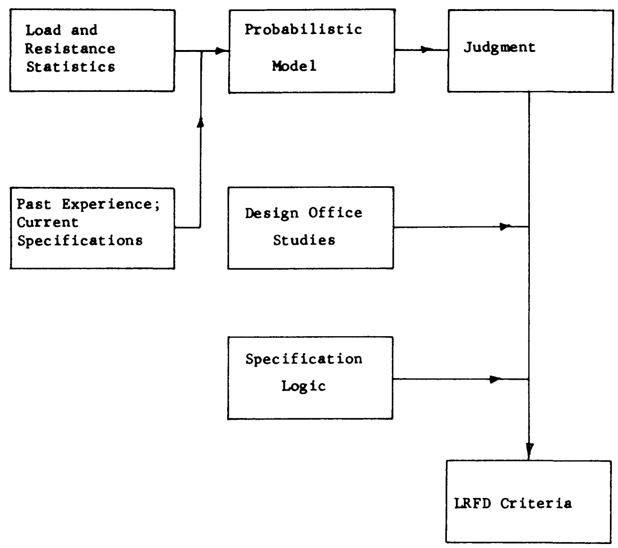

Fig. 6. Flowchart of LRFD specification

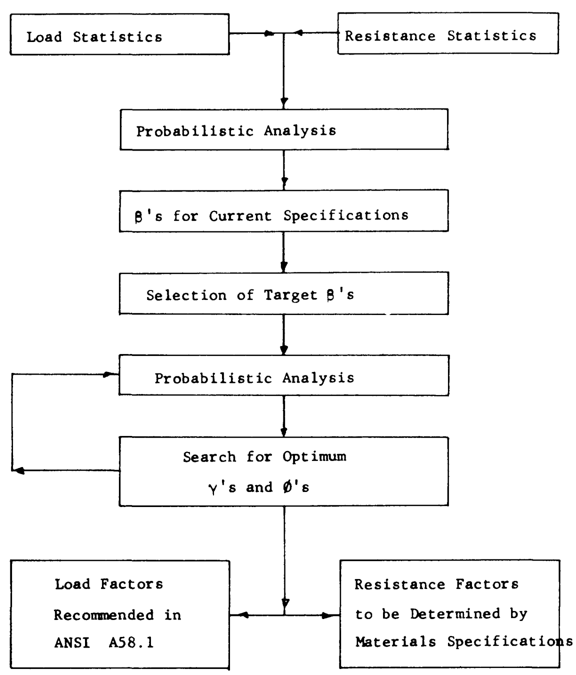

Fig. 7. Flowchart of load and resistance factor development

be taken by one single materials-oriented specification group, and consequently a study was initiated under the umbrella of the ANSI A58 Load Factor Subcommittee at the National Bureau of Standards during 1979. The aim of this research was to arrive at load factors which would be commonly applicable to all structural materials: hot-rolled and cold-formed steel, aluminum, reinforced and prestressed concrete, timber, and masonry. The recommendations of this research were published in 1980.[4] The load factors γ incorporated into the draft of the AISC LRFD Specification[1] are the same factors which have been recommended in ANSI A58.1-81.[2]

The schematic process of arriving at the LRFD Specification is flow-charted in Figs. 6 and 7. These charts show, in a very simplistic way, the ingredients of arriving at the new criteria. One should note that judgment and experience play a crucial role in the process. It is by no means a one-way procedure which is based solely on probabilistic methods. Such methods provide a set of tools which permit the decision makers to make rational rather than intuitive judgments.

THE ANSI A58.1 LOAD FACTORS

The recommended common load factors, as developed for the ANSI load code, were based on the load statistics given in Table 1, where the nominal loads are those of the ANSI A58.1 standard. These load statistics were obtained from the literature on loads by the various subcommittees of ANSI A58 responsible for the various load types, e.g., live load, wind load, etc. Resistance data were obtained from the literature on the various materials involved. The data for steel structures were taken from a series of papers and research reports generated in the research at Washington University, where similar information was also collected on cold-formed and aluminum structures. A sampling of the data for hot-rolled steel structural members is given in Tables 2, 3, and 4.

On the basis of the available load and resistance statistics and on their (1979) current structural design specifications, the values of the reliability index β inherent in all types of materials used in building construction were determined, much in the same way as the example illustrated above for

Table 1. Load Statistics

Load Type	Mean Value	Coeff. of Var.	Distribution Type
Dead	$1.05\ D_n$	0.10	Normal
Max. Lifetime Live	L_n	0.25	Type I
A.P.T. Live	$0.24\ L_o$	0.8–0.4	Gamma
Max. Lifetime Wind	$0.78\ W_n$	0.37	Type I
Max. Lifetime Snow	$0.82\ S_n$	0.26	Type II
Max. Annual Snow	$0.20\ S_n$	0.73	Lognormal

Notations:
A.P.T. = Arbitrary-Point-in-Time
Lifetime = 50 yr.
D_n, W_n, S_n, L_o = Code-specified load intensities (ANSI-A58.1)
$L_n = L_o(0.25 + 15/\sqrt{A_I})$
A_I = Influence area (2 A_T for beams, 4 A_T for columns)
A_T = Tributary area

Table 2. Material Property Statistics

Property	Mean Value	$\overline{M}$	V_M
Static yield stress, flanges	$1.05\ F_y$	1.05	0.10
Static yield stress, webs, plates	$1.10\ F_y$	1.10	0.11
Modulus of elasticity	E	1.00	0.06
Static yield stress in shear	$1.11\ F_y/\sqrt{3}$	1.11	0.10
Poisson's ratio	0.3	1.00	0.03
Tensile strength	$1.10\ F_u$	1.10	0.11
Tensile strength of weld, σ_u	$1.05\ F_{EXX}$	1.05	0.04
Shear strength of weld	$0.84\ \sigma_u$	0.84	0.10
Tensile strength of H.S.S. bolt, A325, σ_u	$1.20\ F_u$	1.20	0.07
Tensile strength of H.S.S. bolt, A490, σ_u	$1.07\ F_u$	1.07	0.02
Shear strength of H.S.S. bolt	$0.625\ \sigma_u$	0.625	0.05

Notations:
F_y = Specified yield stress
F_u = Specified tensile strength
F_{EXX} = Specified tensile strength of weld metal

Table 3. Modeling Statistics

Type Member	Model	$\overline{P}$	V_P
Tension member	A_gF_y or A_eF_u	1.00	0
Compact W-beam:			
Uniform moment	M_p	1.02	0.06
Continuous	Mechanism	1.06	0.07
W-beam, limit state LTB:			
Elastic	S_xF_{cr}	1.03	0.09
Inelastic	Fig. 10	1.06	0.09
Beam-column	Interaction Eq.	1.02	0.10
Plate girder:			
Flexure	M_u	1.03	0.05
Shear	V_u	1.03	0.11
Compact composite beam	M_u	0.99	0.08

Notations:
A_g = Gross area
A_e = Effective net area
M_p = Plastic moment
S_x = Elastic section modulus
F_{cr} = Elastic critical stress
LTB = Lateral-torsional buckling
M_u = Ultimate moment capacity
V_u = Ultimate shear capacity

Table 4. Resistance Statistics

Type Member	$\overline{R}$	V_R
Tension member, yield	1.05	0.11
Tension member, fracture	1.10	0.11
Compact beam, uniform moment	1.07	0.13
Compact beam, continuous	1.11	0.13
Elastic beam, LTB	1.03	0.12
Inelastic beam, LTB	1.11	0.14
Beam-column	1.07	0.15
Plate girder, flexure	1.08	0.12
Plate girder, shear	1.14	0.16
Compact composite beam	1.04	0.14

Notation:
LTB = Lateral-torsional buckling

Table 5. Recommended Load Factors

$1.4D_n$
$1.2D_n + 1.6L_n$
$1.2D_n + 1.6S_n + (0.5L_n \text{ or } 0.8W_n)$
$1.2D_n + 1.3W_n + (0.5L_n)$
$1.2D_n + 1.5E_n + (0.5L_n \text{ or } 0.2S_n)$
$0.9D_n - (1.3W_n \text{ or } 1.5E_n)$

Notations:
D = Dead load
L = Live load due to occupancy
W = Wind load (50 yr mean recurrence interval map)
S = Snow load (50 yr mean recurrence interval map)
E = Earthquake load

the two-span beam of Fig. 4, but using a more advanced method.[4]

The resulting β's were then examined, and on the basis of this across-materials survey, it was determined that, on the average, structural design in the U.S. inherently had the following reliability index values:

Gravity loads only: $\beta = 3.0$
Gravity plus wind loads: $\beta = 2.5$

While there was considerable fluctuation, these values turned up most frequently,[4] and the decision was made by the ANSI A58 subgroup working on the problem to base the new load factors on these values of β as target reliabilities. Accordingly a new cycle of probabilistic analyses was made (see Fig. 7), and the load factors listed in Table 8 were finally developed; these were recommended for adoption in the new ANSI A58.1 standard.[2]

The load combinations are of the following general form:

$$1.2D_n + \gamma_i Q_{in} + \sum_{j \neq i} \gamma Q_{jn} \tag{15}$$

where Q_{in} is the dominant transient load effect, with γ_i based on the load taking on its maximum lifetime (50 yr) value, and Q_{jn} are the other transient load effects, with the γ_j's being determined for the *arbitrary-point-in-time* values of the loads. For example, the combination

$$1.2D_n + 1.6S_n + 0.8W_n$$

stipulates the maximum snow load in the 50 yr lifetime of the structure, while the wind load is based on the maximum annual value.

The load factors and load combinations in Table 5 are presently (1981) being balloted for adoption in the ANSI load standard.[2] They are meant to apply to building structures made from any of the traditional building materials.

RESISTANCE FACTORS

The loads and the load factors in the LRFD design criterion [right side of Eq. (2)] are now set by ANSI, and the task of the materials specification groups is then to develop ϕR_n values which are consistent with the target reliabilities inherent in the load factors. Reference 4 gives the probability-based method, as well as charts and a computer program to facilitate this task. The resistance factors ϕ are to be developed for the load combination $1.2D_n + 1.6L_n$ and for a target reliability index of $\beta = 3.0$.

The following simple example, based on the simplified approach of Eq. (3), will illustrate the method. For a simply supported beam of compact shape, Ref. 9 provides the following statistical data:

$$\overline{P} = 1.02,\ V_P = 0.06,\ \overline{M} = 1.05,$$
$$V_M = 0.1,\ \overline{F} = 1.00,\ V_F = 0.05$$

Therefore, using Eqs. (4) and (5), $\overline{R} = 1.07R_n$ and $V_R = 0.13$. The load effect data is: $\overline{Q} = 1.05D_n + L_n$, where $L_n = L_o(0.25 + 15/\sqrt{2A_T})$ [Eqs. (9) and (13)], and V_Q is given by Eq. (10). Noting that $\overline{D} = 1.05D_n$, $V_D = 0.1$. $\overline{L} = L_n$, $V_L = 0.25$ (Table 1), and letting

$$X = 0.25 + 15/\sqrt{2A_T} \tag{16}$$

then

$$\overline{Q} = D_n\ (1.05 + L_oX/D_n) \tag{17}$$

$$V_Q = [0.011025 + 0.0625\ (L_oX/D_n)^2]^{1/2}/\overline{Q} \tag{18}$$

For the design according to the AISC 1978 Specification, Part 2:

$$R_n = 1.7D_n(1 + L_oX'/D_n) \tag{19}$$

where

$$X' = 1 - 0.0008\ A_T \geq 0.4 \text{ or } 1 - 0.23(1 + D_n/L_o) \tag{20}$$

whichever is larger.

For the design according to the AISC LRFD Specification,

$$R_n = D_n(1.2 + 1.6L_oX/D_n)/\phi \quad (21)$$

Substituting $\bar{R}/R_n$, R_n, $\bar{Q}$, V_R and V_Q into Eq. (3), the variation of β with the tributary area and the live-to-dead load ratio L_o/D_n can be determined, as seen in Figs. 8 and 9 for a value of $\phi = 0.85$. It is seen that the LRFD design gives a nearly constant reliability of approximately $\beta = 3.0$ over the whole range of parameters.

By performing similar analyses on other types of structural members and components, ϕ-factors were derived, and some of these are given in Table 6. It should be noted that these ϕ-factors provide roughly $\beta = 3.0$ for members, and $\beta = 4.0$ for connectors. This is consistent with the traditional practice of providing higher factors of safety for the latter.

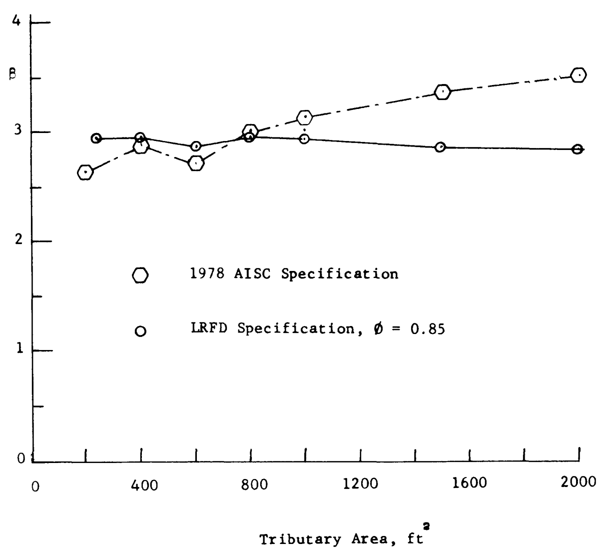

Fig. 8. Reliability Index for simply supported compact wide-flange beam, live-to-dead load ratio, $L_o/D_n = 1.0$

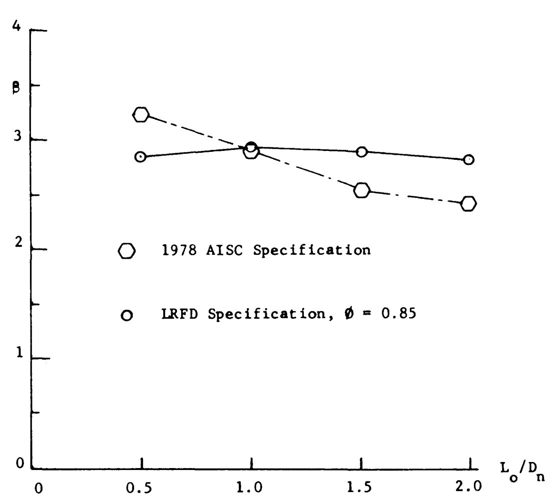

Fig. 9. Reliability Index for simply supporte d compact wide-flange beam, tributary area = 400 ft²

Table 6. Tentatively Recommended Resistance Factors

Type Member	ϕ*
Tension member, limit state: yield	0.90
Tension member, limit state: fracture	0.70
Columns, rolled W sections	0.75
Columns, all other type sections	0.70
Beams, all types and all limit states	0.85
Fillet welds	0.75
H.S.S. bolts, tension	0.75
H.S.S. bolts, shear	0.65

* These are not necessarily the values which will be finally adopted.

The ϕ-factors listed in Table 6 are not necessarily the values which will finally appear in the AISC LRFD Specification. They are at present (May 1981) still under discussion. The point to be made here, however, is that, whatever final value they are assigned, the method presented here will permit an evaluation of the consequences on the reliability index β.

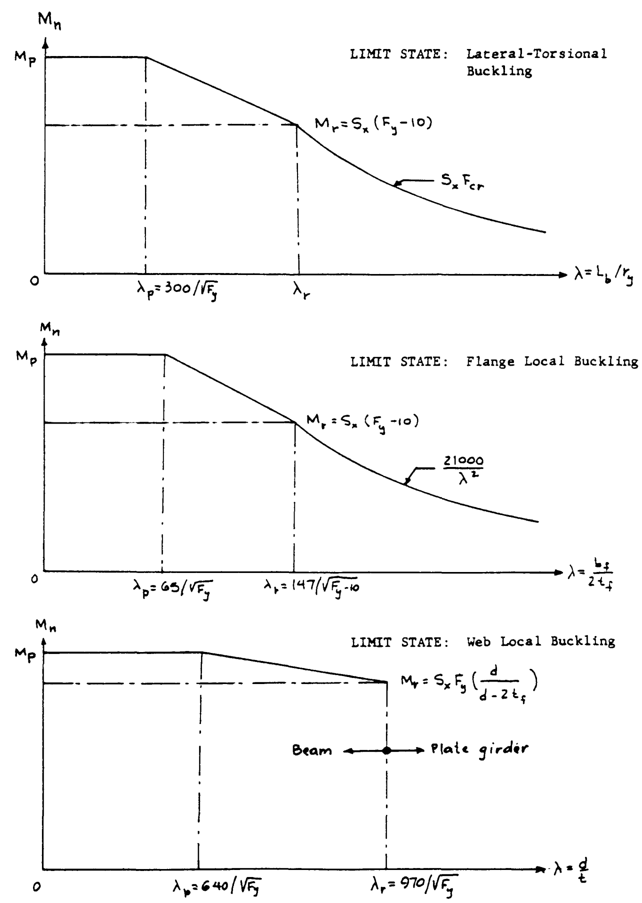

Fig. 10. Nominal strength of wide-flange beams under uniform moment

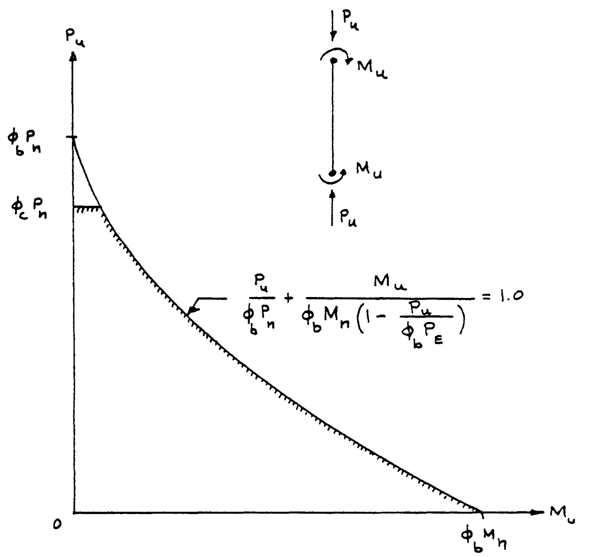

Fig. 11. LRFD design method for wide-flange beam-columns in bending about the major axis

THE AISC LRFD SPECIFICATION

The LRFD Specification is now (May 1981) ready to be debated by the Specification Advisory Committee of the AISC. The draft has been put together by Professor Steven Fenves of Carnegie-Mellon University and a number of Task Committees of the AISC.

The document is an entirely self-contained specification which encompasses all the parts of the well-known 1978 AISC Specification. It is arranged in accordance with the decision table logic developed by Professor Fenves. It is subdivided by members (e.g., tension members, compression members, flexure members, connections), and each type element is given the appropriate resistance factor ϕ and the nominal resistance R_n for each applicable limit state. The ϕ-factors are determined by the probabilistic method described earlier in this paper. The applicable load factors are those which were recommended for the ANSI load standard.[2]

The AISC LRFD Specification also has, in addition to the arrangement and the LRFD format, a number of other new features. It is not the intent here to enumerate these in detail, and only a few will be mentioned: beams (Fig. 10) and beam-columns (Fig. 11) will be treated differently from the 1978 AISC Specification; composite beam design will be based on ultimate strength concepts and, for the first time, the Specification will contain provisions for the design of composite columns.

SUMMARY AND CONCLUSIONS

An attempt has been made here to summarize briefly the existing development of the AISC LRFD Specification. Emphasis was placed on the numerous sources on which this document stands. It is one of the several new LRFD-type specifications which are now (1981) appearing throughout the world. These specifications employ several resistance factors and load factors to account for the various types of uncertainties which underlie design. The reliability is to be interpreted as being "notional," i.e., it is a comparative concept. It should not be confused with actual structural failures, which are the result of errors and omissions. Only the natural statistical variation of the parameters is included, and, as in other traditional specifications, human errors must be guarded against by other control measures.

Basically, the LRFD Specification attempts, within the limits of the first-order probability theory used, to provide designs across the whole design parameter space, which have an approximately consistent reliability under a given load combination. For the first time a method is provided, and load factors are proposed, which would permit the design of building structures of all structural material types to be based on a common approach.

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