U.S. Customary-SI Conversion Factors

In accordance with the October, 1970 action of the ASCE Board of Direction, which stated that all publications of the Society should list all measurements in both U.S. Customary and SI (International System) units, the following list contains conversion factors to enable readers to compute the SI unit values of measurements. A complete guide to the SI system and its use has been published by the American Society for Testing and Materials. Copies of this publication (ASTM E-380) can be purchased from ASCE at a price of $3.00 each; orders must be prepaid.

All authors of *Journal* papers are being asked to prepare their papers in this dual-unit format. To provide preliminary assistance to authors, the following list of conversion factors and guides are recommended by the ASCE Committee on Metrication.

To convert	To	Multiply by
inches (in.)	millimeters (mm)	25.4
feet (ft)	meters (m)	0.305
yards (yd)	meters (m)	0.914
miles (miles)	kilometers (km)	1.61
square inches (sq in.)	square millimeters (mm^2)	645
square feet (sq ft)	square meters (m^2)	0.093
square yards (sq yd)	square meters (m^2)	0.836
square miles (sq miles)	square kilometers (km^2)	2.59
acres (acre)	hectares (ha)	0.405
cubic inches (cu in.)	cubic millimeters (mm^3)	16.4×10^3
cubic feet (cu ft)	cubic meters (m^3)	0.028
cubic yards (cu yd)	cubic meters (m^3)	0.765
pounds mass (lbm)	kilograms (kg)	0.454
tons (ton) mass	kilograms (kg)	907
pounds force (lbf)	newtons (N)	4.45
kilograms force (kgf)	newtons (N)	9.81
pounds force per square foot (psf)	pascals (Pa)	47.9
pounds force per square inch (psi)	kilopascals (kPa)	6.89
U.S. gallons (gal)	liters (L)	3.79
acre-feet (acre-ft)	cubic meters (m^3)	1.23×10^3

Probability Based Load Criteria: Assessment of Current Design Practice

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Abstract: This is the first of two papers that describe a study conducted to develop probability-based load factors and load combinations suitable for use with the loads specified in American National Standard A58 on design loads and with all common construction materials and technologies. The first part of the study, described in this paper, involved the selection of a probabilistic methodology for performing the necessary reliability analyses and the collection and examination of statistical data on structural resistance and loads. Levels of reliability implied by the use of current design standards and specifications for common design situations in which performance generally is felt to be satisfactory were then estimated.

Introduction

Standards for structural design contain requirements to insure that structures perform satisfactorily under the effect of various loads. These provisions, which include load factors, resistance factors, allowable stresses, and deflection limits, have evolved more or less subjectively through extensive successful and unsuccessful professional experience, examination of available experimental data, theory, and judgment. In the United States, such safety and serviceability criteria traditionally have been developed for different materials and methods of construction by interested professional groups and trade associations with diverse design philosophies (3,5,19–23). As a consequence, these criteria do not insure consistent levels of safety and performance for different structures. Moreover, such criteria may be inappropriate for designs which employ new materials or structural schemes where little basis for judgment may exist.

This diversity considerably complicates the design process. The different treatment of structural loads in each specification tends to cause confusion and creates the need to perform separate analyses for the same structure when more than one construction material is used. To simplify this it would be desirable

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Note.—Discussion open until October 1, 1982. Separate discussions should be submitted for the individual papers in this symposium. To extend the closing date one month, a written request must be filed with the Manager of Technical and Professional Publications, ASCE. Manuscript was submitted for review for possible publication on January 15, 1981. This paper is part of the Journal of the Structural Division, Proceedings of the American Society of Civil Engineers, ©ASCE, Vol. 108, No. ST5, May, 1982. ISSN 0044-8001/82/0005-0959/$01.00.

to develop uniform load factors and load-combination rules which could be included in standards governing loads and general design for all structural materials (6). Individual material specification-writing groups then would select suitable resistance criteria compatible with the general provisions.

American National Standard A58, Building Code Requirements for Minimum Design Loads in Buildings and Other Structures (4), defines the magnitudes of dead, live, wind, snow, and earthquake loads suitable for inclusion in building codes and other regulatory documents. In the past, the ANSI A58 Standard has been concerned only with structural loadings, and its specified loads are used in an unfactored form in allowable stress specifications (3,19,21-23), or with load factors in strength design specifications (5,21). For allowable stress design it specifies an array of load combinations which are widely used. With the advent of ultimate strength and limit states design in concrete, steel, and other materials, it would seem that the A58 Standard is the logical place for the material-independent load criteria to appear since it is a national standard in which the approval process requires that all concerned interests have an opportunity to review and comment on the provisions prior to their implementation.

The contemporary trend in standards development is to use probabilistic concepts as the basis for selecting criteria for design (1). As examples of this trend, we might cite ANSI Standard A58.1-1972 (4) in which the snow load and wind load are specified on the basis of an annual probability of their being exceeded. More recent examples include development of Load and Resistance Factor Design (LRFD) criteria for the design of steel building structures using probabilistic methods (1,12), and studies relating to the probabilistic design of reinforced concrete structures (8,10,14,17). This approach recognizes that many of the uncertainties which necessitate the use of safety factors are random.

This paper is the first of two that describe a study (9) conducted to develop probability-based load criteria, including load factors and load combinations, for possible use in a future edition of the A58 Standard. The specific objectives of this study were: (1) To recommend a set of load factors and corresponding load combinations which would be appropriate for all types of building materials (e.g., steel, concrete, heavy timber, engineered masonry, cold-formed steel and aluminum); and (2) to provide a methodology for the material specification groups to select resistance criteria consistent with these load factors and their own specific objectives.

With the standards development process in the United States, no one group has overall responsibility for setting appropriate levels of safety and serviceability for structural design. Thus, the approach here was to develop load criteria that would enable material specification writers to prescribe resistance criteria that would lead, in an average sense, to designs similar to those currently obtained. This approach maintains the necessary continuity in standards and, at the same time, enables certain inconsistencies in current standards to be reduced.

The following is an abbreviated description of the approach taken to achieve the stated objectives:

1. Step 1.—Estimate the level of reliability implied by the use of the current design standards and specifications (e.g., ACI Standard 318, AISC Specifications, and loads from ANSI Standard A58.1-1972) for common structural members and elements (beams, columns, walls, connections) using a particular common reliability calculation scheme and the best available estimates of probability distributions and parameters.
2. Step 2.—Observe reliability levels over practical ranges of materials, load combinations, and failure criteria, and select values representative of current design practice.
3. Step 3.—Using the observed reliability levels as a guide, determine load factors consistent with general desired reliability levels and the selected safety checking format.
4. Step 4.—Develop procedures to enable material specification writing groups to determine resistance criteria consistent with a desired reliability level and the load factors developed in Step 3 without further computer operations.

This paper deals with Steps 1 and 2. Steps 3 and 4 are elaborated upon in the second and following article (10).

Reliability Analysis Procedure

The conceptual framework for probability-based design throughout this study is provided by classical reliability theory (2). The loads and resistances are assumed to be statistical variables and the necessary statistical information is assumed to be available. While this approach furnishes a sound theoretical basis for the evaluation of structural reliability, a number of conceptual and operational difficulties in its use have led to the development of first-order, second-moment (FOSM) methods, so called because of the way they characterize the uncertainty in the variables and the linearization underlying the reliability analysis (11,17).

A mathematical model is first defined which relates the resistance and load effect variables for the specific limit state of interest:

$$g(X_1, X_2, \ldots X_m) = 0 \qquad (1)$$

in which X_i = resistance and load variables, and failure is said to occur when $g < 0$ for the specific ultimate or serviceability limit state. Then the reliability associated with any given design is checked by identifying a point $(X_1, X_2 \ldots X_m)$ satisfying Eq. 1 such that

$$X_i = \bar{X}_i - \alpha_i \beta \sigma_i \qquad (2a)$$

$$\alpha_i = \frac{\sigma_i \dfrac{\partial g}{\partial X_i}}{\sqrt{\sum \left(\sigma_i \dfrac{\partial g}{\partial X_i} \right)^2}} \qquad (2b)$$

The mean and standard deviation of X_i are $\bar{X}_i$ and σ_i, respectively, and β is defined as the reliability index. The partial derivatives in Eq. 2b are evaluated at the point defined by Eq. 2a. This point is called the checking or design point. It is found by searching iteratively for the direction cosines, α_i, that minimize β.

The reliability analysis is shown graphically in Figs. 1(a) and 1(b) for a two-variable problem. The reliability index, β, may be visualized by reformulating Eq. 1 in terms of (uncorrelated) reduced variables x_i, $i = 1, \ldots, m$, that have zero means and unit standard deviations (Fig. 1(*b*)). In this reduced variable coordinate system, β is the shortest distance from the origin to the failure surface. Note from Fig. 1 that this procedure is equivalent to linearizing the limit state equation at the point defined by Eq. 2*a* and computing the reliability associated with the linearized rather than the original limit state. Linearization on the failure surface rather than at the point defined by the mean values avoids the problems which arose in early reliability analysis procedures in which β depended on the formulation of $g(\cdot)$. This is important in situations where loads counteract one another and where Eq. 1 is strongly nonlinear.

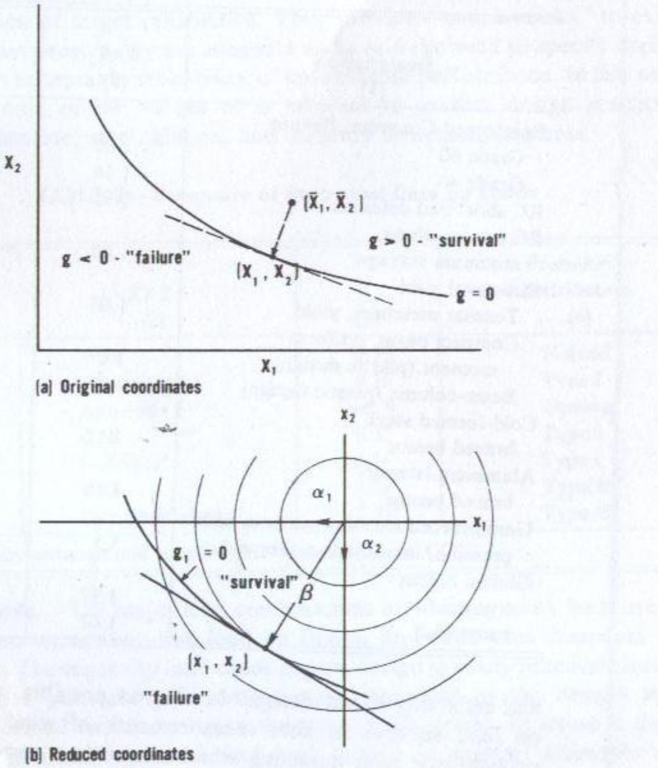

FIG. 1.—Safety Analysis in Original and Reduced Variable Coordinates

When information on the probability distributions of the *X*-variables in Eq. 1 is available, in addition to estimates of their means and variances, this information may be included in the analysis in an approximate way that does not entail the multidimensional integrations required in classical reliability analysis (11). Many structural problems involve random variables that are clearly nonnormal (7,18). The basic idea is to transform nonnormal variables to equivalent normal variables prior to the solution of Eqs. 1 and 2. Intuitively, this transforma-

tion should take place at a point close to that where failure is most likely, that is to say, at a point on the failure surface. Herein $\bar{X}_i$ and σ_i of the equivalent normal variable are chosen so that at the checking point, X_i, the cumulative probability distribution and probability density function of the actual and approximating normal variable are equal (11). Having determined $\bar{X}_i$ and σ_i of the equivalent normal distributions, the solution proceeds exactly as in Eqs. 2*a* and 2*b*.

The failure criterion $g(X_1, X_2, \ldots, X_m) < 0$ may be replaced by the checking equation

$$g(\gamma_1 X_{n_1}, \gamma_2 X_{n_2}, \ldots, \gamma_m X_{n_m}) = 0 \quad \ldots \quad (3)$$

for purposes of design. The factors $\gamma_i = X_i / X_{n_i}$, $i = 1, \ldots, m$ are partial safety factors corresponding to a prescribed reliability index, β, and X_{n_i} are nominal values of the load or resistance parameters specified in standards. In the context of American National Standard A58, the X_{n_i} would correspond to the load levels in current (1972) or proposed (1981) versions of the standard. Similarly, for steel, concrete, and other construction materials, the X_n would correspond to the capacity calculated using nominal material and cross-sectional properties specified in current standards (e.g. F_y = 36,000 psi (248 N/mm²) A36 steel) along with a rational formula based on an analytical or experimental model, or both, of structural behavior. Note from Eq. 3 that probability-based design criteria depend on the way the loads and the resistances themselves are specified in the relevant standards.

Most structural loads vary with time. If the structural member is subjected to only one time-varying load in addition to its dead load, the reliability may be determined by considering the combination of the dead load with the maximum time-varying load during some reference period of time *T*, considered appropriate for design. Frequently, however, more than one time-varying load acts on a structure. When more than one time-varying load acts it is extremely unlikely that each load will reach its peak lifetime value at the same moment. Consequently, a structural component can be designed for a total load which is less than the sum of the peak loads; this currently is recognized in ANSI Standard A58.1-1972, section 4.2.

Conceptually, these load combinations should be dealt with by applying the theory of stochastic processes which accounts for the stochastic nature and correlation of the loads in space and in time. For practical reliability analyses, however, it is necessary to work with random variable representations of the load rather than random process representations. Perhaps the simplest approach for treating combinations of load assumes that the maximum of the combined loads will occur when one of the loads is at its maximum value while other loads assume their instantaneous or arbitrary-point-in-time (apt) values (24). In other words, the maximum, *Z*, of a sum of time varying loads, X_i, during the reference period, *T*, is

$$Z = \max_i \left[\max_T X_i + \sum_{j \neq i} X_j \right] \quad \ldots \quad (4)$$

The arbitrary point-in-time value of load typically is much less than the nominal value; this is illustrated later in connection with the analysis of load data. If there are *n* loads in the limit state equation, in general it is necessary to consider

n distinct load combinations in computing reliability. Recent research on load combinations (13,26) has shown that Eq. 4 is a good approximation in many practical cases, although it tends to be uncoservative in instances where the probability of a joint occurrence of more than one maximum load is not negligible.

Analysis of Statistical Data

Data on both structural resistance and load variables are required in order to develop probability-based design criteria, and numerous research programs conducted over the past eight years or so have focused on one or more aspects of this problem. The basic information required is the probability distribution of each load and resistance variable and estimates of its mean and standard deviation or coefficient of variation. The mean and coefficient of variation of these basic variables should be representative of values that would be expected in actual structures in situ. While frequently there are sufficient data to obtain a reasonable estimate of the probability distribution, in many other cases this must be assumed on the basis of physical argument or for convenience.

In the context of the FOSM approach to reliability, the concept of uncertainty, exemplified by variability or scatter in a variable, is conveyed through the variance or the coefficient of variation (COV). The uncertainties used in the reliability analyses would include inherent statistical variability in the basic strength and load parameters, and additional sources of uncertainty that arise due to the modeling and prediction errors and incomplete information (2). Included in these modeling uncertainties would be errors in estimating the parameters of the distribution functions, mathematical idealizations of structural capacity and of the actual load processes, uncertainties in calculation, and deviations in the applications of the various load or material specifications from the idealized cases considered in their development. While occasionally there may be some data available with which to estimate these later uncertainty measures, frequently they must be estimated on the basis of professional judgment and experience.

In most instances in this study, the basic resistance variable was taken as structural action, i.e., the strength of the structural member in question. The basic load variable was the load effect, dimensionally consistent with the resistance. These can be used directly in the reliability analysis when the failure criterion is formulated as the linear combination of resistance and load variables. The linear formulation is quite common in practice and has been used for most of the reliability studies conducted over the last several years. The analysis of these data for resistances and loads is described in subsequent paragraphs.

Means, COVs, and probability distributions for structural resistances have been determined from test data on the strength of materials and on dimensions of structural members, from laboratory tests of full-scale members under idealized load conditions and, in some cases, where a clearly defined analytical model exists, through Monte Carlo simulation. A representative sampling of these data which summarize results of numerous research programs conducted over the last several years is given in Table 1. Further study may be found in Ref. 9. The mean values are normalized with respect to the nominal resistance, which is based on the model used to predict the resistance in the appropriate material specification. The statistics include factors which reflect the effects of modeling and of fabrication. A substantial amount of data exist for steel

and reinforced concrete structural elements (8,9,12,14,15). Less usable data are available for brick and concrete masonry, and for timber construction (9). The strength of glulam beams and columns in part depends on the duration of load. The load duration effect is presently an active research area. The tendency is for the strength to increase as the load duration decreases. This is reflected in the values for mean/nominal given for glulam beams in Table 1.

Means, COVs, and probability distributions of 50 yr maximum and arbitrary-point-in-time load effects are summarized in Table 2. The load subcommittees within ANSI Committee A58 provided many of these estimates. By and large, these statistical studies are a synthesis of values reported in numerous previous studies on structural loads and load models, behavior of structural members,

TABLE 1.—Summary of Statistical Data on Resistance

Designation (1)	$\bar{R}/R_n$ (2)	V_R (3)	Probability distribution (4)
Reinforced Concrete, flexure			
Grade 60	1.05	0.11	Normal
Grade 40	1.14	0.14	Normal
RC short tied columns	0.95	0.14	Normal
RC beams, shear, minimum stirrups	1.00	0.19	Normal
Structural steel			
Tension members, yield	1.05	0.11	Lognormal
Compact beam, uniform moment (plastic design)	1.07	0.13	Lognormal
Beam-column (plastic design)	1.07	0.15	Lognormal
Cold-formed steel, braced beams	1.17	0.17	Lognormal
Aluminum-laterally braced beams	1.10	0.08	Lognormal
Unreinforced masonry walls in compression, inspected workmanship	5.3	0.18	Lognormal
Glulam beams			
live load	1.97	0.18	Weibull
snow load	1.62	0.18	Weibull

and reliability-based design. Insofar as possible, the load statistics are based on load surveys in situ, measurements of wind pressure on buildings, and probabilistic load modeling which converts a survey load to a maximum load used for purposes of reliability analysis and design. In addition to the basic variability in the load, uncertainty arises from the load model which transforms the actual spatially and temporally varying load into a statically equivalent uniformly distributed load which can be used for design purposes. Uncertainties also arise in the analysis which transforms the uniformly distributed load to a load effect, including two-dimensional idealization of three-dimensional structures, fixity of supports, rigidity of connections, and continuity (9). These uncertainties are included in the COVs listed in Table 2. As described in Ref. 9, live, wind, and snow load statistics have been determined by fitting the extreme value distributions used in the reliability analysis work to the upper percentiles of the distributions obtained through either Monte Carlo simulation or numerical integration.

Reliabilities Inherent in Existing Standards

Target reliabilities (or safety indices), along with the statistical information summarized in the previous section, provide the basis for probability-based design criteria. Target reliabilities may be established by reviewing the levels of reliability inherent in those existing standards which have resulted in the past in satisfactory performance. While strict reliance on these values may allow inconsistencies and certain undesirable features of current design practice to carry over into the probability-based criteria, they are useful as a guide to the selection of target reliabilities. They provide "bench-marks" to existing procedures that presumably are adequate and avoid the need to specify explicitly and justify an acceptable probability of unfavorable performance. In this section we review some of the values of β inherent in present design practice for reinforced concrete, steel, glulam, and masonry structural elements.

TABLE 2.—Summary of Statistical Data on Loads

Load (1)	$\bar{X}/X_n$ (2)	V_X (3)	Probability distribution (4)
D	1.05	0.10	Normal
L	Appendix I	0.25	Type I
L_{apt}	Appendix I	Ref. 9	Gamma
W	0.78	0.37	Type I
W_{apt}	(−0.021)[a]	(18.7)[a]	Type I
S	0.82	0.26	Type II
E	(site dependent)[a]	(2.3)[a]	Type II

[a]Characteristic extreme and shape rather than mean and COV.

Gravity Loads.—The major load combinations involving gravity loads are dead plus maximum occupancy live load on floors, and dead plus maximum snow load on roofs. These gravity load cases govern design in many practical situations and they are a particularly important case, inasmuch as the designs appear to have performed satisfactorily in the past. Each design situation is defined by a set of nominal resistance and load values. In present allowable stress specifications (3,19–23)

$$\frac{R_n}{\text{F.S.}} = D_n + L_n \,(\text{or} \quad D_n + S_n) \qquad (5a)$$

in which F.S. is the factor of safety. In plastic design of steel structures (21)

$$R_n = 1.7(D_n + L_n) \qquad (5b)$$

In reinforced concrete structures

$$\phi R_n = 1.4\,D_n + 1.7\,L_n \qquad (5c)$$

Reliability indices associated with current flexural design of steel and reinforced concrete beams for dead plus live load are shown in Fig. 2 as functions of load ratio L_o/D_n. The live load, L_o, is the basic unreduced uniform live load specified in Table 1 of ANSI A58.1-1972 (4). From this figure it is evident that the variation of β for such beams is remarkably similar. In each case, β tends to decrease as L_o/D_n or S_n/D_n increases. Note that β for steel beams deviates more from the ideal constant reliability than β for reinforced concrete. This is because only one factor of safety is applied to the combination of loads in steel design, whereas reinforced concrete design uses a partial factor on each of the individual load effects which reflects the relative magnitude of the variability of that load effect. When viewing the similarity in reliability, it should be kept in mind, however, that reinforced concrete beams have practical ranges of L_o/D_n or S_n/D_n of 0.5 to 1.5, while for steel beams this range is from 1 to 2. As shown in Fig. 2, the significant load ratios for steel beams are thus shifted to the right with regard to concrete beams. Representative

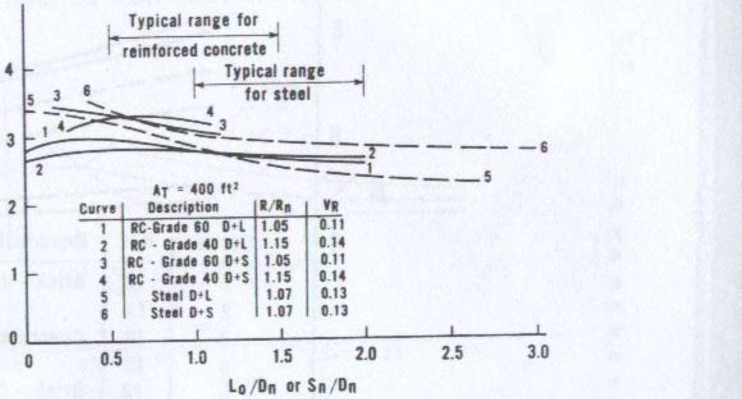

FIG. 2.—Reliability Index for Steel and Reinforced Concrete Beams (100 sq ft = 9.3 m²)

values for β are thus 2.8 and 3.2 for concrete beams and 2.5 and 2.9 for steel beams for, respectively, the $D + L$ and $D + S$ combinations.

While the reliability index for typical steel and concrete structural members under gravity loads is in the vicinity of 3, β for typical nonreinforced brick and concrete masonry walls, and columns under compression and bending, appears to be considerably higher. For example, for walls in compression built with inspected workmanship with a typical $L_o/D_n = 0.5$, $\beta = 7.4$ for brick masonry, and $\beta = 6.2$ for concrete masonry (see Fig. 3). The reliability of uninspected masonry, with its higher COV and lower $\bar{R}/R_n$, is considerably less; uninspected workmanship causes β for the same brick wall to decrease to 4.7. As bending increases (M/Pt in excess of 1/6), β begins to diminish, falling to about 3 when M/Pt reaches the maximum allowable value of 1/3 (9). Reliability indices for reinforced masonry columns in compression are between 6 and 7 (9).

Safety indices for glulam beams designed for gravity loads are given in Fig. 4. Among members of the same stress, grade, and species the margin of reliability may vary considerably. The sensitivity of β to distribution of R does not appear large at typical ratios of L_o/D_n and S_n/D_n.

Other typical values of β for the $D + L$ combination for metal, concrete, masonry, and timber structures are summarized in Table 3. The reliabilities shown in Table 3 depend on $\bar{R}/R_n$, V_R, the cdf, and the typical load ratios and supported areas for each member. These factors combine to reduce the range in reliabilities for steel and concrete structural members that might otherwise be expected on the basis of the statistics presented in Table 1. It might be observed that for reinforced concrete beams, the β value for shear is less than for flexure. The relatively small difference in resistance factor for flexure and shear in ACI 318-77 (0.9 and 0.85, respectively) is, in the opinion of the writers,

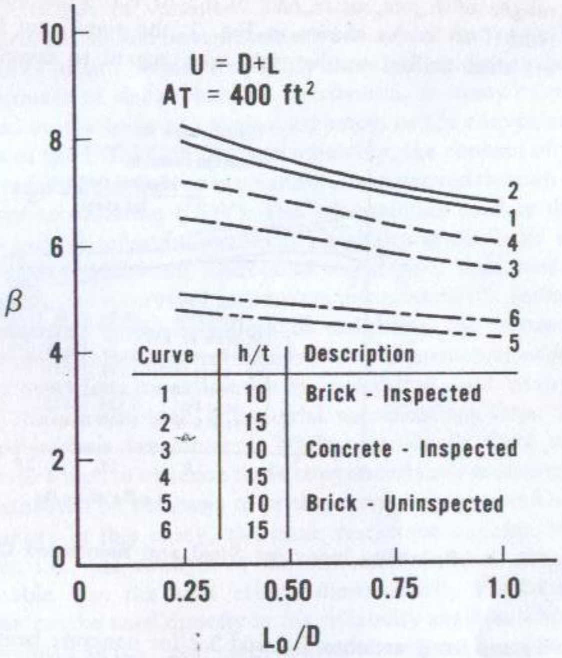

FIG. 3.—Reliability Index for Nonreinforced Masonry Walls (100 sq ft = 9.3 m^2)

inadequate to compensate for the higher variability in shear capacity and increased reliability that is desired. Current design criteria for reinforced concrete and steel beams appear to result in similar levels of reliability; as competition between these two primary construction materials in the United States traditionally has been very keen, this is not especially surprising.

Gravity and Environmental Loads.—The major load combinations are dead, live, and wind ($D + L + W$) and dead, live, and earthquake ($D + L + E$). Attention first will be directed to the combinations $D + L_{apt} + W$ and $D + L + W_{apt}$, where L and W define the maximum lifetime magnitudes, and L_{apt} and W_{apt} are the arbitrary-point-in-time values.

The variation of β with various L_o/D_n and W_n/D_n ratios is shown in Figs.

5 and 6 for steel and reinforced concrete beams, respectively. In the analysis, R_n is determined for each design situation according to

For allowable stress design,

$$\left(\frac{4}{3}\right)\frac{R_n}{\text{F.S.}} = D_n + L_n + W_n \quad \text{(6a)}$$

For plastic design in steel,

$$R_n = 1.3\,(D_n + L_n + W_n) \quad \text{(6b)}$$

For reinforced concrete design,

$$0.9\,R_n = 0.75\,(1.4\,D_n + 1.7\,L_n + 1.7\,W_n) \quad \text{(6c)}$$

In all cases, however, R_n must not be less than that required by the $D + L$ combination. The effect of the rate of loading has been included in the analysis by multiplying $\bar{R}/R_n$ by 1.10 for steel members and 1.05 for concrete members.

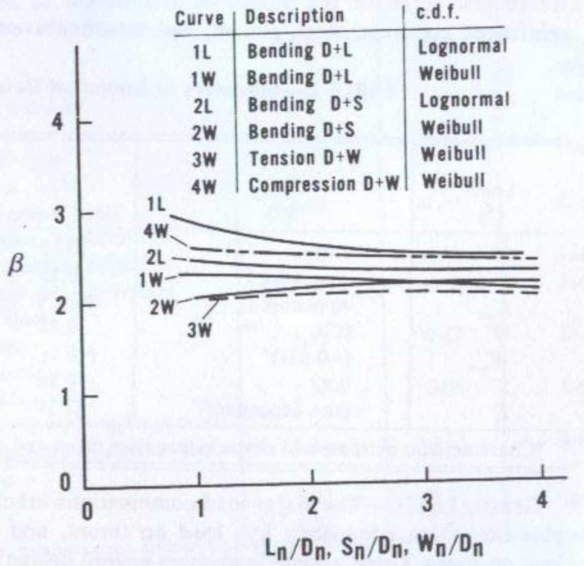

FIG. 4.—Reliability Index for Glued-Laminated Wood Members

This difference accounts for the relatively higher dead load component of the total load effect in the concrete structures. For steel beams, this gives essentially the mill test yield stress rather than the static yield stress as the basic material variable.

From Figs. 5 and 6 it can be seen that β decreases as W_n/D_n increases. While the curves in Figs. 5 and 6 are for beams, the results are similar for other types of members for which the resistance statistics are similar. Where wind contributes the major load component, the value of β approaches a value of 2. With greater live and dead load, the value of β increases to that of the $D + L$ case. In general, the wind load combinations result in a somewhat lower apparent reliability in current practice than the gravity load combinations. This is due to the 1/3 increase in allowable stress (or the use of 3/4 of the total factored loads) in all of the current standards which were used in this evaluation. It is not obvious from this analysis whether current criteria for gravity loads are too conservative or criteria for wind loads are not conservative enough, at least according to conventional safety checking methods. It is possible that the reliability of structures under wind is only apparently less because of such factors as load-sharing by cladding and load redistribution among members.

There are a number of difficulties in analyzing the reliability of structures against earthquake loads. These derive from the fact that the seismic provisions for static, linearly elastic member strength design are not well related to the behavior of the structural system that would occur under the design ground accelerations. The limit state implicitly addressed is not first yield of a cross

TABLE 3.—Summary of Reliabilities Associated With Current Standards ($D + L$), $L_n/D_n = 1.0$ Except as Noted

Member (1)	Reliability, β (2)
R/C beam, Grade 60, medium ρ	2.8
R/C beam, Grade 40, medium ρ	2.8
Cast-in-place, post-tensioned beam, low ρ	3.0
Plant precast, pre-tensioned beam, low ρ	3.6
Short tied R/C column, compression failure	3.4
Short spiral R/C column, compression failure	3.0
Shear strength of R/C beam 2 times minimum stirrups	2.4
Steel tension member, limit state yield (fracture)[a]	2.5 (3.4)
Compact steel beam[a]	3.1
Steel column, $\lambda = 0.5$	3.1
Fillet welds[a]	3.9
A325 bolts, shear[a]	4.4
Aluminum columns ($L_n/D_n = 5$)	2.8

[a] $L_n/D_n = 2$.

section but some less well-defined building behavior such as life-threatening damage to structural elements. In an attempt to make reliability estimates that could be compared with other load combinations, the recent ATC recommendations (25) were taken as a means of relating member yield to building failure. The link is provided by the "ductility factor," in ATC terminology, which is used to reduce base shears to design values and reflects the ductility of members under dynamic loads and the toughness of the system. It is used here to reduce predicted loads to reflect the same phenomena.

Typical values of β for the earthquake loading case $D + L + E$ are given in Fig. 7 for two locations, Boston, Mass., and Los Angeles, Calif., and for steel and concrete beams and columns. The strain rate effects also have been incorporated in this analysis. Due to the high variability of the earthquake loads as compared with the variability of dead loads, the $\beta - vs - E_n/D_n$ curves flatten out rapidly to values which reflect essentially only the contribution of

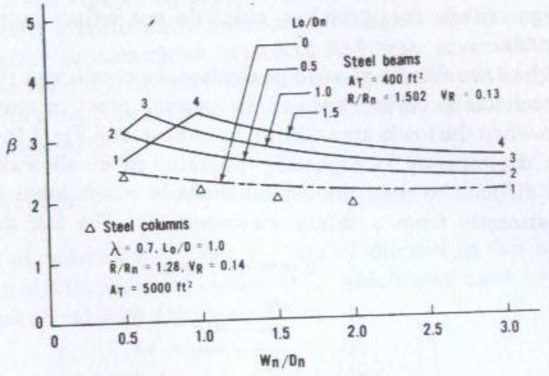

FIG. 5.—Reliability Index for Steel Members—Gravity Plus Wind Loads (100 sq ft = 9.3 m²)

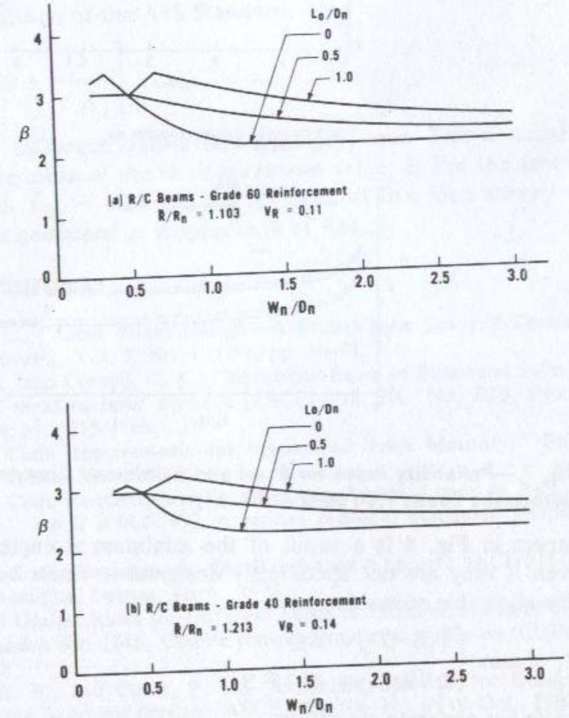

FIG. 6.—Reliability Index for Reinforced Concrete Beams—Gravity Plus Wind Load [A_T = 400 sq ft (37 m²)]

the earthquake load effect. Reliability indices for $D + L + E$ are lower than for $D + L + W$. While the difference between beams and columns is small, the effect of geographic location on β is pronounced, implying that current design criteria for these two cities do not reflect properly the relative seismic hazards.

When the effects of wind or earthquake counteract the effect of gravity loads, the reliability indices implied by current practice tend to be somewhat lower than when the loads are additive, as indicated in Fig. 8 for the $W - D$ combination. The discrepancy is especially pronounced in allowable stress formats where it is difficult to treat the combinations in which loads are added and subtracted consistently from a safety viewpoint (9). The left descending branch of the

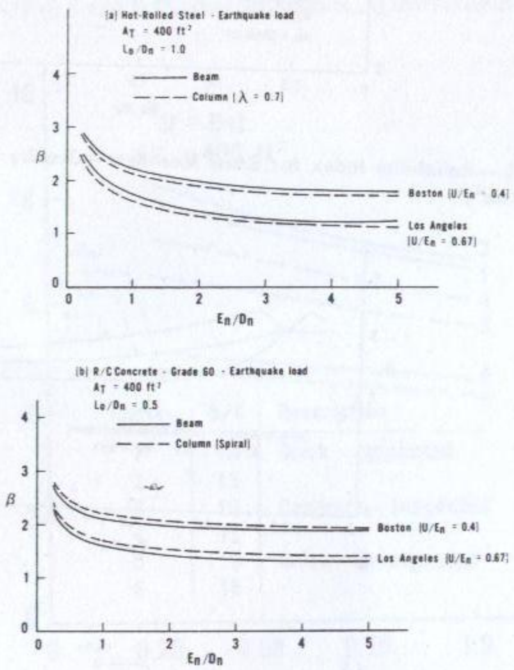

FIG. 7.—Reliability Index for Steel and Reinforced Concrete Members—Gravity Plus Earthquake Loads (100 sq ft = 9.3 m²)

curves in Fig. 8 is a result of the minimum strength that the members have even if they are not specifically designed to resist counteracting load effects. For allowable stress design

$$R_n = \max \begin{cases} \text{F.S.}\,(D_n + L_n) \\ 0.75\,(D_n - W_n)\,\text{F.S.} \end{cases} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (7a)$$

while for concrete structures

$$\phi R_n = \max \begin{cases} 1.4\,D_n + 1.7\,L_n \\ 0.9\,D_n - 1.3\,W_n \end{cases} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (7b)$$

The reliability of concrete beams and columns generally is closer to that for

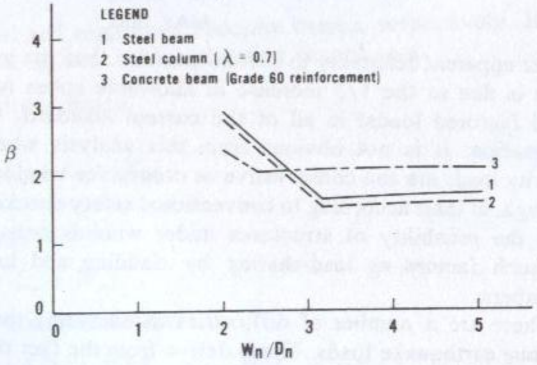

FIG. 8.—Reliability Index for Steel and Reinforced Concrete Members—Counteracting Loads

the additive load cases because the individual load effects are factored and because Eq. 7*b* recognizes the possibility of a dead load less than the nominal value.

Conclusions

The examination of reliabilities of existing design criteria indicates that the reliability index varies according to material, member type and failure mode, and load combination. It appears that the β levels associated with current designs are smaller for load combinations which include load effects due to wind or earthquake than for load combinations with gravity loads only.

The reason for the difference is the use of the one-third increase of the allowable stress when one of the load contributors is wind or earthquake. Apparently, this increase does more than to account for the improbability of the simultaneous occurrence of two maximum lifetime loads, since this effect is included in the probabilistic model used in the analysis. It can be argued that the lower reliability of the load combinations involving wind or earthquake is only apparent. In the case of live loads due to occupancy, the load consists of multiple discrete sources which tend to affect the structure or its individual members, while wind or earthquake loads affect the entire building system, including both structural and nonstructural elements.

Design codes deal explicitly with structural members, and the probabilistic analysis of this paper followed the same approach. If the current design codes implicitly include an allowance for systems reliability, be that load sharing between structural and nonstructural components, multiple load paths, or ductility, then the actual reliability can be determined only by a probabilistic analysis of the system. Such an analysis is still in the research stage and was not applied in this study. Instead, it was assumed, for the time being and pending further research on systems reliability, that the same implicit allowance for overall effects should also underlie the selection of target βs for the development of probabilistic load factors. Thus, different target reliabilities were chosen for gravity load combinations and wind and earthquake load combinations, as indicated in the following. This decision is based on the assumption that the differences of β in current design are only apparent, as witnessed by the fact that present designs for main force-resisting members subjected to gravity and wind or earthquake loading appear to be performing satisfactorily. While this solution is not entirely satisfying from the analytical point of view, it permits the development of load factors which would not upset and perhaps distort present practice in an undesirable way.

On the other hand, it also could be argued that present practice demands structures which are either overdesigned under gravity loads (live or snow loads), or, conversely, underdesigned under combinations involving wind or earthquake loads. In either case, selection of a single target, β, would result in load factors which would drastically change present practice. The writers did not feel justified, on the basis of analyses performed so far, to follow this alternative, especially in view of satisfactory performance of current designs. However, the dilemma posed by the differences in β is by no means resolved, and both researchers and code-writing groups should be challenged to undertake serious efforts to resolve it.

It was not the purpose of this study to make specific recommendations to material specification groups as to what values of ϕ to use or to prescribe precisely what reliabilities their resistance criteria should be targeted on. Such decisions are the responsibility of specification committees which have the necessary expertise on material performance. Nevertheless, it is necessary to have an idea of where these reliabilities are in current design and where they are likely to fall, in order to perform the necessary calculations leading to a coordinated set of specific load factor values. It should be emphasized that this procedure actually places little restriction on material specification groups because even after the load factor values have been determined, the final design reliabilities may be adjusted through an appropriate selection of resistance factor values or other materials criteria.

The analysis of reliabilities associated with current acceptable practice showed that for many steel, reinforced concrete, and glulam flexural and compression members, β tends to fall within the range of 2.5 to 3.5 for the gravity load combinations (considered on the basis of a 50 yr reference period). Values of β for combinations involving wind and earthquake loads are in the range 1.75 to 2.5. Accordingly, the target β_o chosen for purposes of deriving the load criteria for $D + L$ and $D + S$, is 3.0; for $D + L + W$, $\beta_o = 2.5$; for $D + L + E$, $\beta_o = 2.0$; and for counteracting loads, $\beta_o = 2.0$. Generally speaking, these values are slightly more conservative than indicated by current practice, when the transient load is large, in comparison with the permanent load and less conservative when the permanent load is a major component.

This paper has described a reliability analysis procedure for developing probability based design criteria, summarized the available data on structural capacity and loads, and shown where current standards stand with respect to one another from a reliability viewpoint. This establishes a datum from which probability-based design criteria can evolve. The following paper (10) shows how practical loading criteria can be developed using this information.

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Appendix I.—Live Load

Two values of nominal live load, L_n, are of interest in this study. The first is the value in ANSI Standard A58.1-1972, which was used to determine the values of β associated with existing practice:

$$L_n = L_o \left\{ 1 - \min \left[0.0008\, A_T,\ 0.6,\ 0.23 \left(1 + \frac{D_n}{L_o} \right) \right] \right\} \quad (8)$$

in which A_T = tributary area; and L_o = basic unreduced live load given in Table 1 of ANSI A58.1-1972. The second nominal live load is that proposed for the 1980 edition of the A58 Standard:

$$L_n = L_o \left[0.25 + \frac{15}{\sqrt{A}} \right] \quad (9)$$

in which A = influence (rather than tributary) area. This nominal value is also identical to the mean of the 50 yr maximum value, $\bar{L}$. For the arbitrary-point-in-time live load, $\bar{L}_{apt} \approx 0.24\, L_o$, on the basis of live load survey data. Further information is contained in Appendix A of Ref. 9.

Appendix II.—References

1. Allen, D. E., "Limit States Design—A Probabilistic Study," *Canadian Journal of Civil Engineering*, Vol. 2, No. 1, 1976, pp. 36–49.
2. Ang, A. H., and Cornell, C. A., "Reliability Bases of Structural Safety and Design," *Journal of the Structural Division*, ASCE, Vol. 100, No. ST9, Proc. Paper 10777, Sept., 1974, pp. 1755–1769.
3. "Building Code Requirements for Engineered Brick Masonry," Brick Institute of America, McLean, Va., 1969.
4. "Building Code Requirements for Minimum Design Loads in Buildings and Other Structures," *ANSI A58.1-1972*, American National Standards Institute, New York, N.Y., 1972.
5. "Building Code Requirements for Reinforced Concrete (ACI 318-77)," American Concrete Institute, Detroit, Mich., 1977.
6. "Common Unified Rules for Different Types of Construction and Material," *Bulletin D' Information No. 124E*, Comite Euro-International du Beton (CEB), Paris, France, Apr., 1978.
7. Corotis, R. B., and Chalk, P. L., "Probability Models for Design Live Loads," *Journal of the Structural Division*, ASCE, Vol. 106, No. ST10, Oct., 1980, pp. 2017–2033.
8. Ellingwood, B., "Reliability Basis of Load and Resistance Factors for Reinforced Concrete Design," *National Bureau of Standards Building Science Series 110*, National Bureau of Standards, Washington, D.C., Feb., 1978.
9. Ellingwood, B., Galambos, T. V., MacGregor, J. C., and Cornell, C. A., "Development of a Probability Based Load Criterion for American National Standard A58," *National Bureau of Standards Special Publication No. 577*, National Bureau of Standards, Washington, D.C., June, 1980.

10. Ellingwood, B., MacGregor, J. G., Galambos, T. V., and Cornell, C. A., "Probability Based Load Criteria—Load Factors and Load Combinations," *Journal of the Structural Division*, ASCE, Vol. 108, No. ST5, Proc. Paper 17068, May, 1982, pp. 978–997.
11. "First Order Reliability Concepts for Design Codes," *Bulletin D' Information No. 112*, Comite Europeen du Beton, Munich, Germany, July, 1976.
12. Galambos, T. V., and Ravindra, M. K., "Load and Resistance Factor Design," *Journal of the Structural Division*, ASCE, ST9, Proc. Paper 14008, Sept., 1978, pp. 1335–1336.
13. Larabee, R., and Cornell, C. A., "A Combination Procedure for a Wide Class of Loading Processes," *Probabilistic Mechanics and Structural Reliability*, ASCE, New York, N.Y., Jan., 1979.
14. MacGregor, J. G., "Safety and Limit States Design for Reinforced Concrete," *Canadian Journal of Civil Engineering*, Vol. 3, No. 4, Dec., 1976, pp. 484–513.
15. Mirza, S. A., and MacGregor, J. G., "A Statistical Study of Variables Affecting the Strength of Reinforced Normal Weight Concrete Members," *Structural Engineering Report No. 58*, Department of Civil Engineering, University of Alberta, Edmonton, Alberta, Canada, Dec., 1976.
16. Rackwitz, R., and Fiessler, B., "Note on Discrete Safety Checking When Using Non-Normal Stochastic Models for Basic Variables," Loads Project Working Session, Massachusetts Institute of Technology, Cambridge, Mass., June, 1976.
17. "Rationalisation of Safety and Serviceability Factors in Structural Codes," *CIRIA Report 63*, Construction Industry Research & Information Assoc., London, UK, 1977.
18. Simiu, E., and Filliben, J., "Statistical Analysis of Extreme Winds," National Bureau of Standards Technical Note No. 868, National Bureau of Standards, Washington, D.C., June, 1975.
19. "Specifications for Aluminum Structures," The Aluminum Association, Washington, D.C., Third Edition, Apr., 1976.
20. "Specifications for the Design and Construction of Load Bearing Concrete Masonry," National Concrete Masonry Association, Arlington, Va., 1968.
21. "Specifications for the Design, Fabrication and Erection of Steel Buildings," American Institute of Steel Construction, New York, N.Y., Nov., 1978.
22. "Specification for the Design of Cold-Formed Steel Structural Members," American Iron and Steel Institute, Washington, D.C., 1968.
23. "Standard Specifications for Structural Glued-Laminated Timber of Douglas Fir, Western Larch, Southern Pine and California Redwood," American Institute of Timber Construction, Englewood, Colo., 1974.
24. Turkstra, C. J., "Theory of Structural Design Decisions," *Solid Mechanics Study No 2*, University of Waterloo, Waterloo, Ontario, Canada, 1972.
25. "Tentative Provisions for the Development of Seismic Regulations for Buildings," *ATC 3-06*, Applied Technology Council, SEAOC, Palo Alto, Calif., June, 1978.
26. Wen, Y. K., "Statistical Combination of Extreme Loads," *Journal of the Structural Division*, ASCE, Vol. 103, No. ST6, Proc. Paper 12930, May, 1977, pp. 1079–1095.

Appendix III.—Notation

The following symbols are used in this paper:

A = influence area;
A_T = tributary area;
D = dead load;
E = earthquake;
F.S. = factor of safety;
F_y = yield stress;
g = limit state design function;
L = live load;
L_o = basic specified live load;
n = subscript denoting nominal value;

R = resistance;
S = snow load;
T = reference period;
V = coefficient of variation;
W = wind load;
X = design variable;
Z = time varying load variable;
α_i = factor defined by Eq. 2b;
β = reliability index;
γ = load factor;
ϕ = resistance factor;
λ = column slenderness parameter; and
ρ = reinforcement ratio.