Assessment of Current Load Factors for Use in Geotechnical Load and Resistance Factor Design

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Abstract: Load and resistance factor design (LRFD) is the standard structural design practice. In order for foundation design to be consistent with current structural design practice, the use of the same loads, load factors, and load combinations would be required. In this paper, we review the load factors presented in various LRFD codes from the United States, Canada, and Europe. A simple first-order second-moment (FOSM) reliability analysis is presented to determine appropriate ranges for the values of the load factors. These values are compared with those proposed in the codes. The comparisons between the analysis and the codes show that the values of load factors given in the codes generally fall within ranges consistent with the results of the FOSM analysis. However, it would be desirable for the successful development and adoption of the geotechnical component of LRFD codes to have uniformity of load-factor values across different codes for the loads that are common for virtually all civil structures.

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Introduction

Over the past four decades, a load and resistance factor design (LRFD) method was brought into practice in the United States with the adoption of the American Concrete Institute Building Design Code (ACI) in 1963 (Goble 1999). In structural design practice, LRFD is currently accepted worldwide along with a traditional design method, allowable stress design (ASD), or as it is also known, working stress design. With the trend toward the increased use of LRFD, new LRFD codes in the United States, Canada, and Europe (MOT 1992; API 1993; AASHTO 1994; ECS 1994; NRC 1995) have included the implementation of LRFD for geotechnical design over the past several years. Additionally, an ACI document in preparation also advocates LRFD design of shallow foundations. The AASHTO (1994, 1998) code proposes the use of the same loads, load factors, and load combinations for foundation design as those used in structural design. The resistance factors in the AASHTO code were calibrated for the same load factors used in the design of structural members. Since the load and resistance factors for structural design have been calibrated and adjusted through their use in practice over many years, it would be appropriate to use the same loads, load factors, and load combinations in foundation design to be consistent with current structural practice. Through the use of the same load factors, not only can a consistent design between superstructures and substructures be attained, but also the design process itself may be significantly simplified (Withiam et al. 1997).

The successful unification of the structural and geotechnical design processes may be achieved through the use of appropriate resistance factors in foundation LRFD, such that for the given set of load factors and load combinations, LRFD produces a design consistent with current practice, or even a more economic design for a desired reliability level. Compared with structural design, however, LRFD in foundation design is still new. To facilitate its general use in practice, continuous calibration efforts to determine the appropriate resistance factors, as was done for structural design codes, are desirable. While attempting to develop the resistance factors, a general understanding of the load factors proposed in current LRFD codes may provide a means to easily compare and evaluate resistance factors proposed recently or in the future. In this paper, load factors presented in various LRFD codes from the United States, Canada, and Europe are reviewed, and the similarities and differences between the values of load factors are assessed. A simple reliability analysis is conducted to determine an appropriate range for the values of load factors. The results of this analysis are then compared with the values presented in the reviewed codes. We conclude with recommendations on how to best develop LRFD for acceptance in geotechnical practice.

Load and Resistance Factor Design and Limit States

The basic design inequality for LRFD can be given as

$$\sum (\mathrm{LF})_i \cdot S_{ni} \leqslant \mathrm{RF} \cdot R_n \tag{1}$$

where LF, S_n, RF, and R_n = load factor, nominal load, resistance factor, and nominal resistance, respectively. The resistance is set

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Table 1. Load Factors

Loads	United States				Canada		Europe	
	AASHTO (1998)	ACI (1999)	AISC (1994)	API (1993)	MOT (1992)	NRC (1995)	DGI (1985)	ECS (1995)
Dead	1.25–1.95 (0.65–0.9)	1.4 (0.9)	1.2–1.4 (0.9)	1.1–1.3 (0.9)	1.1–1.5 (0.65–0.95)	1.25 (0.85)	1.0 (0.85)	1.0–1.35 (0.95)
Live	1.35–1.75	1.7	1.6	1.1–1.5 (0.8)	1.15–1.4	1.5	1.3	1.3–1.5
Wind	1.4	1.3	1.3	1.2–1.35	1.3	1.5	1.3	1.3–1.5
Seismic	1.0	1.4	1.0	0.9	1.3	1.0	1.0	1.0

Note: Values in parentheses apply when the load effects tend to resist failure for a given load combination.

such that the factored load effects do not exceed the factored resistance for predefined possible limit states. Here, the term ''limit state'' refers to any set of conditions that may produce unsatisfactory performance of the structural or geotechnical system. The limit states would be associated with the various loads and load combinations considered in the design. In general, limit states are grouped into two categories, ultimate limit states (ULS) and serviceability limit states (SLS). Ultimate limit states are associated with the concepts of danger (or lack of safety), usually involving structural damage that may lead to instability or collapse of the structure. An ULS may involve, for example, the rupture of critical parts of the structure, progressive collapse of a structural member, or instability due to deformations of the structure (MacGregor 1997). For foundations, the classical notion of a bearing capacity failure is clearly an ULS.

Serviceability limit states are defined as conditions that may undermine the function or service requirements (performance) of the structure under expected service or working loads (Becker 1996). Examples of serviceability limit states include cracking of architectural finishings or walls, excessive deformation (differential movement) of the superstructure, rupture of utility lines, or pavement cracking or undulation (which would lead to a ''rough ride'' on a bridge).

Load Factors Proposed by Load and Resistance Factor Design Codes in United States, Canada, and Europe

To review the load factors proposed by various LRFD codes, a total of eight bridge, building, and on-shore and offshore foundation LRFD codes from the United States, Canada, and Europe were collected. These were the following: ''AASHTO LRFD bridge design specifications'' (AASHTO 1998); ''Building code requirements for structural concrete'' (ACI 1999); ''LRFD specification for structural steel buildings'' (AISC 1994); ''Recommended practice for planning, designing, and constructing fixed offshore platforms-LRFD'' (API 1993); ''Ontario highway bridge design code'' (MOT 1992); ''National building code of Canada'' (NRC 1995); ''Code of practice for foundation engineering'' (DGI 1985); and ''Eurocode 1'' (ECS 1994). The load factors in the above codes have been determined through calibration processes either before or after the codes adopted LRFD for implementation in design practice. Code calibration may be done in several ways: using judgment and experience, fitting with traditional design codes (i.e., ASD), using reliability analysis based on rational probability theory, or using a combination of these approaches (Barker et al. 1991). The load and resistance factors in the LRFD codes of the United States and Canada have been primarily calibrated using probability theory, which has provided a theoretical basis for LRFD since the late 1960s in the United States. In Denmark and other European countries, the load and resistance factors in the codes have been mainly derived from fitting with previous codes and adjusted through their use in practice. In Denmark, limit states design (LSD) has been used for geotechnical applications since the 1960s.

There are many differences in the types of limit states considered for design and in the load types and load combinations defined for each limit state when comparing the bridge and offshore codes to the building codes. Usually, a greater number of limit states and load types apply to the design of special structures such as bridges and offshore foundations. However, certain types of loads appear in most design situations for all types of structures. These are dead loads, live loads, wind loads, and earthquake loads. In this study, load factors for these four major load types are considered. Some load types that are not considered include collision loads, snow and ice loads, and earth pressure loads.

Load Factors for Ultimate Limit States

Table 1 shows the ranges of values of load factors for ultimate limit states (ULS) in the codes discussed earlier. In general, for the bridge codes (MOT 1992; AASHTO 1998) and offshore foundation code (API 1993), the range of load factor values is rather wide compared with that for building or onshore foundation codes. For example, the range of values of load factors for dead loads in AASHTO and MOT extends from 1.25 to 1.95 and 1.1 to 1.5, respectively, whereas the range for the building codes, except ECS (1995), is 1.2 to 1.4. The values of live load factors in the bridge and offshore foundation codes lie between 1.1 and 1.75. The values of live load factors for the building codes, except ACI (1999), are in the range of 1.3 to 1.6.

Many different dead load types are considered in AASHTO (1998) and MOT (1992). These include the weight of the structural members, the weight of wearing surfaces such as asphalt, and earth pressure loads. A different value of the load factor is applied to each of these load types. For example, in AASHTO (1998), while the value of the load factor for structural components is 1.25, the load factor values for the weight of wearing surfaces and the vertical earth pressure applied to flexible buried structures are 1.5 and 1.95, respectively. The relatively high values of the load factors for the wearing surface weight and the earth pressure applied to buried structures reflect high variability in estimating the magnitude of the corresponding loads. On the other hand, the dead loads in the building codes such as ACI (1999) and NRC (1995) consist mostly of the weight of structural components, partitions, and all other materials incorporated into the building to be supported permanently by the structural components. The same load factor is used for all of these loads as they are all treated simply as dead loads. The rather wide ranges for the dead load factors in the bridge codes, therefore, are associated

Table 2. Load Factors and Gravity Load Combinations

Code	Representative gravity load combination
AASHTO (1998)	$1.25D+1.75L$
ACI (1999)	$1.4D+1.7L$
AISC (1994)	$1.2D+1.6L$
API (1993)	$1.3D+1.5L$
MOT (1992)	$1.2D+1.4L$
NRC (1995)	$1.25D+1.5L$
DGI (1985)	$1.0D+1.3L$
ECS (1995)	$1.35D+1.5L$

with the various types of dead loads accounted for in the design of bridges.

For the live loads in Table 1, the values for the load factors that are less than 1.0 apply when the load is used together with other transient loads (i.e., live, wind, or earthquake loads) in a load combination. This is based on the assumption that the simultaneous occurrence of the maximum value of each load is not likely, and some loads may counteract other loads when they occur together. To account for this, most codes, except the bridge codes (AASHTO and MOT), apply a load combination factor less than 1.0 when more than two different transient loads are used in a load combination. As an example, NRC (1995) proposes a value of 0.7 for the load combination factor when both a live and a wind load are present. In that case, therefore, 70% of each factored load effect for both the live and the wind loads are considered in design. That is

$$S=(\mathrm{LF})_D S_D+0.7[(\mathrm{LF})_L S_L+(\mathrm{LF})_W S_W] \quad (2)$$

The load combination factor usually varies with the number of transient loads that are present. That is, in the case where only one transient load applies, the value of the load combination factor is unity.

In the bridge codes (AASHTO and MOT), different values of the load factors are defined in different load combinations, instead of multiplying the proposed load factors for each load by the load combination factor. As an example, AASHTO defines one load combination when live load is present, but wind load is not

$$S=1.25S_D+1.75S_L \quad (3)$$

but defines another load combination when both live load and wind load are present

$$S=1.25S_D+1.35S_L+0.4S_W \quad (4)$$

To make comparisons between the values easier, the values of load factors for a representative load combination will be used. The load combination will be a gravity load combination (i.e., dead load plus live load). Table 2 shows a comparison of the gravity load combinations for the different codes. From Table 2, it can be seen that the variations among the codes for the values of load factors for dead and live loads fall into a relatively narrow range, 1.0 to 1.4 and 1.3 to 1.75, respectively. Excluding the values in the Danish foundation code (DGI 1985) from the comparison, the range of values for dead loads becomes even narrower (i.e., 1.2 to 1.4).

For wind and earthquake loads, the values of load factors for the different codes show comparatively better agreement than for gravity loads. The values of wind load factors vary from 1.2 to 1.5. For earthquake loads, the values of the load factors are 1.0 in most codes. Earthquake loads are site-dependent loads, which means that there may exist regional variations for design loads. Therefore, most codes state that nominal earthquake loads should be determined relatively conservatively and a value of 1.0 should be used for the earthquake load factor. This is done in order to avoid a load factor value that varies from site to site.

In summary, the comparisons show that the values of load factors for ULS are generally consistent for all the codes reviewed. A major difference appears in dead load factors between the building codes and bridge codes. Compared with the building codes, the bridge codes subdivide dead loads into more specific load types (e.g., vertical earth pressure applied to flexible buried structures), for which different values of load factor are used, resulting in wide ranges of load factor values. However, when considering a gravity load combination, the values fall within rather narrow ranges for all the codes.

Load Factors for Serviceability Limit States

Though ULSs are the focus of the current research, serviceability limit states (SLS) must be considered as well. Table 3 shows the values of load factors for serviceability limit states in the codes reviewed. SLSs are treated differently from ULSs. Load factors are applied in both cases, but resistance factors are not used for SLS checks. Instead, the settlements resulting from the factored loads must not exceed the allowable settlements. Load factors of unity are typically prescribed for SLS checks. The bridge codes, such as AASHTO (1998) and MOT (1991), use load factor values less than 1.0 for wind and live loads. In MOT, values of 0.7 and 0.75 apply to wind and live loads, respectively, while AASHTO uses a value of 0.3 for wind load factor.

The use of values less than 1.0 is derived from the reasoning that the time-dependent loads such as live and wind loads are not likely to remain at their maximum value for significant periods of time and therefore, factored loads for SLS checks will be less than the design loads. Furthermore, live loads considered in bridge designs are traffic loads that may be highly dependent on time compared with live loads in buildings that are mostly occupancy loads. Using a live load factor of 0.75, the MOT code accounts for the time-dependent characteristic of the traffic loads. However, the use of a load factor value of 1.0 may be more appropriate for SLS checks for foundations on granular soils, as the settlement of granular soils is immediate. This is not a problem for most codes, as load factors of 1.0 are used for SLS checks

Table 3. Load Factors for Serviceability Limit States

	United States			Canada		Europe	
Loads	AASHTO (1998)	ACI (1999)	AISC (1994)	MOT (1991)	NRC (1995)	DGI (1985)	ECS (1995)
Dead	1.0	1.0	1.0	1.0	1.0	1.0	1.0
Live	1.0	1.0	1.0	0.75	1.0	N/A[a]	1.0
Wind	0.3	1.0	1.0	0.7	1.0	N/A[a]	1.0

[a]Values for transient loads are given in the structural code.

in all of the codes, except the two bridge codes. Earthquake loads are not considered for SLS in the codes.

Simple Reliability Analysis

A simple reliability analysis was conducted to determine the appropriate ranges of the load factor values in ULS for the four different types of loads considered in this study. The method employed was the first-order second-moment (FOSM) method, assuming lognormal distributions for the design variables (i.e., load and resistance). This method was developed largely by Cornell (1969) and Lind (1971).

Loads may not be distributed lognormally; in fact, the exact distribution characteristics of loads are never known. The distribution used to model the loads should be the least biased distribution, using the given information. This given information is typically the mean and the variance (or coefficient of variation) of the loads. In order to determine which distribution is in fact the least biased, the principle of maximum entropy may be employed. This principle states that the least biased distribution is the distribution that maximizes entropy subject to the constraints imposed by the given information (Jaynes 1957). Entropy H for a discrete random variable is given by (Harr 1987)

$$H = -\sum p_i \ln p_i \tag{5}$$

where p_i = probability of event i. For a continuous random variable, entropy is given by (Harr 1987)

$$H = -\int_a^b f(x) \ln f(x) dx \tag{6}$$

where a and b = lower and upper limits, respectively, of the variable. The negative sign in each of these equations makes entropy positive. If the only data available about a variable are the values of the upper and lower limit, the principle of maximum entropy states that the uniform distribution (the distribution such that all values within the range of possible values are equally likely) is the least biased distribution (Harr 1987).

In geotechnical engineering, information about the mean and variance of a load or resistance is typically available, even though the exact distribution may not be known. The lower and upper limits of the load or resistance may be unknown. In this case, the principle of maximum entropy states that the normal distribution is the least biased distribution. However, the magnitudes of load and resistance found in geotechnical problems cannot take negative values. This firmly establishes a lower limit for both loads and resistances. The upper limit of the load or resistance is typically unknown. This is especially true for transient loads (i.e., live loads, wind loads, and earthquake loads), which can assume values that are extremely large, though quite improbable. These transient loads are typically modeled by load specification committees using more precise distributions, namely, the Type I or Type II extreme-value distributions (Ellingwood et al. 1980), but these distributions require more knowledge of the variable than simply the mean, variance, and minimum value. Therefore, these distributions do not represent the least biased distribution for the loads for the information generally available. Accordingly, the lognormal distribution better models transient loads, as it is fully characterized by its first two moments, allowing easier implementation in FOSM analysis. This leads to a distribution that is not only relatively simple to implement, but also gives reasonable results (MacGregor 1976). Moreover, the lognormal distribution better represents the product of several positive random variables, even if these variables are not themselves lognormally distributed. In load modeling, the nominal load itself may be modeled as the product of several components, each of which may also be modeled as a random variable. For example, wind loads are usually modeled as the product of wind speed and other empirical or experimental parameters that are treated as random variables (ASCE 7-95 1996). Occasionally, an engineer on a project will have detailed load information specific to that project. In this case, specific load factors could be developed or a more complex analysis could be used, if the effort is justified by the economics of the project.

An overall resistance is frequently modeled as the product of nominal resistance and several parameters to account for the different sources of uncertainty. In the design of a bridge structure, the overall resistance of a structural member is commonly modeled as the product of nominal resistance and a material factor, a fabrication factor, and an analysis factor, which are used to account for the uncertainties for the material strengths, component dimensions, and analytical models, respectively (Nowak and Grouni 1994). This can be expressed mathematically as

$$R = R_n \eta_m \eta_f \eta_a \tag{7}$$

where η_m = material factor that accounts for the uncertainty of the strength of the material; η_f = fabrication factor that accounts for the uncertainty of the size of the fabricated member (e.g., the variability of the size of formwork for cast in place concrete); and η_a = analysis factor that accounts for the uncertainty of the analytical model used to calculate resistance. Soil resistance for foundation design may also be modeled in several cases as the product of nominal resistance and several components that account for the uncertainties of inherent soil variability, measurement (testing), and analytical methods. Perhaps this is best illustrated by considering the general bearing capacity equation for clays

$$q_{bL} = (s_c d_c i_c b_c g_c) c N_c \tag{8}$$

which uses a series of multiplicative correction factors to model the bearing capacity of a shallow foundation. Measurement uncertainty would be seen in c, as cohesion is a soil strength parameter that must be measured using in situ testing, lab tests, or correlations with other measured parameters. Additional variability due to the inherent uncertainty of the bearing capacity equation itself would result in the analysis uncertainty.

In this context, the lognormal assumption for both loads and resistances appears to be reasonable, as both can be treated as the product of several random variables. The load effects and resistances of a structural or geotechnical system may then be expressed as shown in Fig. 1. Let the load effect S and the resistance R be random variables; then, failure (the attainment of an ULS) occurs when $\ln R - \ln S < 0$ (represented by the shaded area in Fig. 1). The probability of failure P_f can be written as

$$P_f = P[(\ln R - \ln S) < 0] \tag{9}$$

Assuming that the random variables, $\ln R$ and $\ln S$, are statistically independent, the mean $\overline{U}$ and standard deviation σ_U of $U = \ln R - \ln S$ are given by

$$\overline{U} = \overline{\ln R} - \overline{\ln S} \tag{10}$$

$$\sigma_U = \sqrt{\sigma_{\ln R}^2 + \sigma_{\ln S}^2} \tag{11}$$

The safety index or reliability index β, which is a relative measure of safety for a given system, can be expressed as a func-

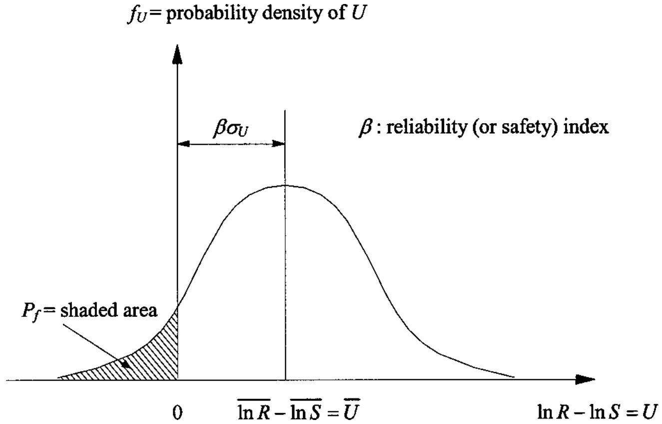

Fig. 1. Load effects, resistance, and reliability

tion of the mean and standard deviation of U (Fig. 1) (Allen 1975; MacGregor 1976; Becker 1996)

$$\beta = \frac{\overline{\ln R} - \overline{\ln S}}{\sqrt{\sigma^2_{\ln R} + \sigma^2_{\ln S}}} \tag{12}$$

For a lognormal distribution

$$\sigma^2_{\ln S} = \ln(1 + V_S^2), \quad \sigma^2_{\ln R} = \ln(1 + V_R^2) \tag{13}$$

where V_S and V_R = the coefficients of variation of S and R, respectively, defined as the ratio of the standard deviation to the mean. For small V_S or V_R (say, less than 0.6), the following expressions are acceptable approximations (MacGregor 1976):

$$V_S^2 \cong \sigma^2_{\ln S}, \quad V_R^2 \cong \sigma^2_{\ln R} \tag{14}$$

According to MacGregor (1976), the error in Eq. (14) is less than 2% for $V_R = 0.3$, increasing to about 10% for $V_R = 0.6$. For comparison, the reported values of V_R for various geotechnical properties and resistances lie in a wide range of about 0.05 to 0.85 (Becker 1996). Considering the mean values of the reported values, the range varies from about 0.1 to 0.5. The assumption of Eq. (14) overestimates the uncertainty of the resistance, and is therefore slightly conservative. Based on Eqs. (13) and (14), Eq. (12) can be rewritten as follows:

$$\overline{\ln R} - \overline{\ln S} \geqslant \beta \sqrt{V_S^2 + V_R^2} \tag{15}$$

Lind (1971) has shown that

$$\sqrt{V_S^2 + V_R^2} \cong \alpha V_S + \alpha V_R \tag{16}$$

where α = separation coefficient having values between 0.707 and 1.0 (depending on the value of the ratio V_R/V_s), and MacGregor (1976) has shown that

$$\overline{\ln R} - \overline{\ln S} \cong \ln\left(\frac{\overline{R}}{\overline{S}}\right) \tag{17}$$

which can be used to approximate Eq. (15). Taking Eqs. (16) and (17) into Eq. (15)

$$\ln(\overline{R}/\overline{S}) \geqslant \beta\alpha V_S + \beta\alpha V_R \tag{18}$$

or

$$\overline{R}/\overline{S} \geqslant e^{(\beta\alpha V_S + \beta\alpha V_R)} \tag{19}$$

Rearranging Eq. (19) gives

$$\overline{R}(e^{-\beta\alpha V_R}) \geqslant \overline{S}(e^{\beta\alpha V_S}) \tag{20}$$

The mean load effect $\overline{S}$ and resistance $\overline{R}$ can be defined as

$$\overline{S} = S_n k_S, \quad \overline{R} = R_n k_R \tag{21}$$

where S_n, R_n, k_S, and k_R = nominal load, the nominal resistance, and the bias factors (i.e., the ratio of mean to nominal) for load and resistance, respectively. Using Eq. (21), Eq. (20) can be rewritten in a form analogous to the LRFD fundamental equation

$$R_n k_R(e^{-\beta\alpha V_R}) \geqslant S_n k_S(e^{\beta\alpha V_S}) \tag{22}$$

or

$$\mathrm{RF} \cdot R_n \geqslant \mathrm{LF} \cdot S_n \tag{23}$$

where LF and RF = load factor and resistance factor, respectively. From Eqs. (22) and (23), the value of the load factor and the resistance factor can be calculated by

$$\mathrm{LF} = k_S e^{\beta\alpha V_S} \tag{24}$$

$$\mathrm{RF} = k_R e^{-\beta\alpha V_R} \tag{25}$$

With Eq. (24), if appropriate values of the parameters α, β, k_S, and V_S are known, the value of the load factor for each load type can be obtained in a simple manner. In most cases, however, the estimation of these parameters is difficult. This is so not only because α is a function of both the load effects and the resistance, but also because the values of k_S and V_S are not well known due to limited statistical data.

A similar derivation can be employed for determining load and resistance factors if the underlying distributions are normal. This will be useful for determining the load factor for dead load, as dead loads are typically modeled as having a normal distribution (Ellingwood et al. 1980). For normally distributed variables, the probability of failure is given by (Haldar and Mahadevan 2000)

$$P_f = P[(R - S) < 0] \tag{26}$$

The reliability index β is given by

$$\beta = \frac{\overline{R} - \overline{S}}{\sqrt{\sigma_R^2 + \sigma_S^2}} \tag{27}$$

Using the separation coefficient α, Eq. (27) can be written as

$$\beta = \frac{\overline{R} - \overline{S}}{\alpha(\sigma_R + \sigma_S)} \tag{28}$$

Rearranging Eq. (28) gives

$$\overline{R} - \alpha\beta\sigma_R = \overline{S} - \alpha\beta\sigma_S \tag{29}$$

Noting that $V_R = \sigma_R/\overline{R}$ and $V_S = \sigma_S/\overline{S}$

$$\overline{R}(1 - \alpha\beta\sigma_R) = \overline{S}(1 - \alpha\beta\sigma_S) \tag{30}$$

With $\overline{S} = S_n k_S$ and $\overline{R} = R_n k_R$

$$\mathrm{LF} = k_S(1 + \alpha\beta V_S) \tag{31}$$

$$\mathrm{RF} = k_R(1 - \alpha\beta V_R) \tag{32}$$

Selection of Parameters Used in Analysis

From Eq. (16), the separation coefficient α can be written as

$$\alpha = \frac{\sqrt{1 + (V_R/V_S)^2}}{1 + (V_R/V_S)} \tag{33}$$

The separation coefficient is a function of the ratio V_R/V_S. In other words, it is a function of the uncertainties in both the loads and the resistances. To derive a load factor based on Eq. (24),

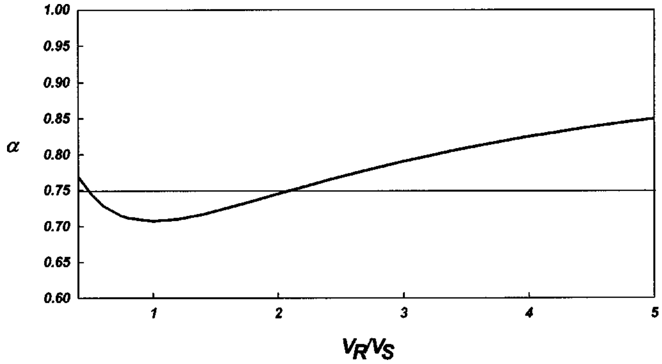

Fig. 2. Variation of separation coefficient α

therefore, a representative value of V_R/V_S should be chosen. Values of V_R range from about 0.1 to 0.5 as presented previously. The representative values of V_S reported in the literature range from 0.1 to 0.25 for dead, live, and wind load (Nowak 1994; Ellingwood and Tekie 1999). Hence, the corresponding ratio V_R/V_S for the reported ranges of V_R and V_S range from 0.4 to 5. For values of V_R/V_S between 0.4 and 5, the separation coefficient α takes values within the rather narrow range of about 0.7 to 0.85. Accordingly, a value of 0.75 was assumed for α in our analysis. This value has also been used by Becker (1996) and is consistent with the range as presented in Fig. 2. For comparison, load factor values obtained using α ranging from 0.7 to 0.85 are also examined.

The reliability index β is a relative measure of the degree of safety. As shown in Fig. 1, higher values of β are associated with smaller probabilities of failure, and vice versa. By using Eqs. (24) and (25), one can calculate the value of β for given values of the load and resistance factors and statistical parameters. Conversely, the load and the resistance factor can be determined for a given β (i.e., for a target reliability index) and for given statistical parameters. In fact, code calibration is the process in which the load and resistance factors are adjusted to obtain a desired level of reliability. The load effects *S* in Fig. 1 are usually the combination of load effects for several different load types according to the load combinations used. For instance, in a gravity load combination, a load effect *S* will be the combination of dead load effects and live load effects. In this case, the reliability index β is commonly calculated using the reliability equations, where statistical parameters such as V_S and V_R are the statistical parameters representative of the combined load effects (i.e., dead load and live load) and the overall resistance. Based on this approach, Ellingwood et al. (1980), after careful examination of β for common structural members, such as concrete, steel, and timber, reported that the representative values of reliability index β tend to fall within the range of 2.5 to 3.0 for both the gravity load and the gravity plus wind load combinations. These values for β are representative of the reliability associated with existing designs. He also suggested that, for gravity load, gravity plus wind load, and gravity plus earthquake load combinations, the representative target reliability indices β_T are 3.0, 2.5, and 1.75, respectively. These target reliability indices have been established after consideration of the reliability associated with current designs. Establishing target reliability indices based on current designs will lead to load factors that produce designs that are similar to current designs. This is desirable because the reliability indices can be refined later, if there is a need to refine them at all, in a cautious manner as the codes evolve. To derive the load factor for a particular load type using Eq. (24), therefore, the selection of different values of β for each load type would be required. In this analysis, based on Ellingwood's work, the values of β equal to 3.0 for dead load, 2.75 for live load, 2.5 for wind load, and 1.75 for earthquake loads were assumed.

Table 5. Values of k_S and V_S Assumed for Analysis

Loads	k_S	V_S
Dead	1.0–1.05	0.07–0.16
Live	0.95–1.05	0.2–0.3
Wind	0.85–0.9	0.15–0.25
Earthquake	0.25–0.35	0.9–1.1

For the evaluation of the values of k_S and V_S, extensive research has been performed over several decades of use of LRFD in structural design. For the time-variant loads such as live, wind, and earthquake loads, the values of k_S and V_S are normally obtained from time-stochastic modeling processes based on available recorded data (e.g., traffic survey data, wind speed data, or seismic acceleration coefficient). Table 4 shows the values of k_S and V_S reported by several researchers. As expected, the biases for gravity loads (i.e., dead load and live load) are relatively small. This means that gravity loads tend to be estimated rather accurately. Also note that the coefficient of variation for dead loads is quite low. On the other hand, values of V_S for earthquake loads are significantly higher than for other loads. Based on the data presented in Table 4, ranges of values for k_S and V_S are determined for each load type for use in the analysis of the present paper. The ranges of values used are presented in Table 5.

Comparison Between Results and Load Factors in Codes

Table 6 and Fig. 3 show the comparisons of the values of the load factors between the analysis and the codes. The load factors for beneficial dead loads were obtained using equations similar in form to Eqs. (25) and (32), namely

$$\mathrm{LF}=k_S e^{-\alpha\beta V_S} \tag{34}$$

for the lognormal distribution, and

$$\mathrm{LF}=k_S(1-\alpha\beta V_S) \tag{35}$$

Table 4. Ratio of Mean to Nominal Load k_S and Coefficient of Variation V_S

Loads	k_S	References	V_S	References
Dead	1.03–1.05	Nowak 1994; Ellingwood and Tekie 1999	0.08–0.15	Nowak 1994; Ellingwood and Tekie 1999
Live	1.0	Ellingwood and Tekie 1999	0.25	Ellingwood and Tekie 1999
Wind	0.875	Nowak 1994	0.20	Nowak 1994
Earthquake	0.3	Nowak 1994	0.7<	Ellingwood et al. 1980; Nowak 1994

Note: k_S and V_S in transient loads (i.e., live, wind, and earthquake loads) are maximum 50-year values.

Table 6. Comparison of Values of Load Factors from Analysis and from Codes

	Dead load	Live load	Wind load	Earthquake load
Analysis ($0.7<\alpha<0.85$)	1.16–1.58 (1.34); 0.66–0.91[a] (0.79)	1.40–2.12 (1.71)	1.11–1.53 (1.29)	0.75–1.80 (1.17)
Analysis ($\alpha=0.75$)	1.17–1.50 (1.33); 0.70–0.90[a] (0.79)	1.44–1.95 (1.68)	1.13–1.44 (1.28)	0.81–1.48 (1.12)
All codes	1.0–1.4[b] (1.24); 0.65–0.95[a] (0.80)	1.3–1.75[b] (1.53)	1.2–1.5 (1.36)	0.9–1.4 (1.08)
AASHTO, ACI, and AISC only	1.2–1.4[b] (1.28); 0.65–0.9 (0.86)	1.6–1.75[b] (1.68)	1.3–1.4 (1.33)	1.0–1.4 (1.13)

Note: Values in parentheses represent average values.

[a]Beneficial dead loads.

[b]Range for a representative gravity load combination, as presented in Table 2.

for the normal distribution, based on the reasoning that beneficial dead loads resist failure. These equations are similar to the resistance factor equations, except that the bias factor and coefficient of variation are for the beneficial load effects, not the resistances. These equations also differ from the standard load factor Eqs. (24) and (31) in that they are expressed in terms of $-\alpha\beta V_S$ instead of $\alpha\beta V_S$. This accounts for the beneficial nature of these loads. The values for load factors given in the codes are found to be reasonably consistent for all loads considered. A relatively wide range in earthquake load factors is mainly due to the values of V_S used in the analysis, which lie within a wide range. In the same table, for comparisons, average values for the ranges of each load are shown. For dead and live load, the values by the analysis are somewhat higher than those in all the codes. It is interesting to note, however, that when a comparison is made with the U.S. codes (i.e., AASHTO, ACI, and AISC), the average values from the analysis show relatively good agreement with the values from the codes, although the ranges given in the analysis are rather large (Table 6). For α varying from 0.7 to 0.85, the ranges become somewhat larger, but the only load factors affected significantly are those for earthquake loads. In some cases, the analysis supports the use of load factors that are higher than the load factors currently used in the codes. This can be seen in Fig. 3 for earthquake loads especially. This apparent unconservatism in the current codes is due to the underlying probability distribution for the loads. The current research uses the least biased distribution considering only the mean and variance of the loads along with the fact that the loads cannot be negative. The codes are based on more precise, and therefore more biased, distributions of the loads, using more information about the particular loads being considered. Upon considering this extra information, the code developers can arrive at a more precise load factor for a particular case. As can be seen from Fig. 3, these values always lie within the range determined by the current research.

Future Development of Geotechnical Load and Resistance Factor Design

As demonstrated by Eqs. (23), (24), (25), and (33), load and resistance factors are inexorably linked through the values of β, V_R, and V_S. This means that each code will assign different values to resistance factors, because of the different load factor values adopted. This adds to the complexity of LRFD compared with ASD. In ASD, engineers need only to understand the concept of the global factor of safety, which has been in use for at least a century. The safety factor for a footing, for example, typically would be in the range of 2 to 4, and the engineer selects the value to use in a design based on general guidelines. In LRFD, it is essential to use the values of LF and RF prescribed in the code, as well as a nominal resistance consistent with the LF and RF values. This requires an understanding of more complex concepts.

Acceptance of the LRFD approach hinges on making the method understandable to and usable by geotechnical engineers. The large array of different load factors currently in existence,

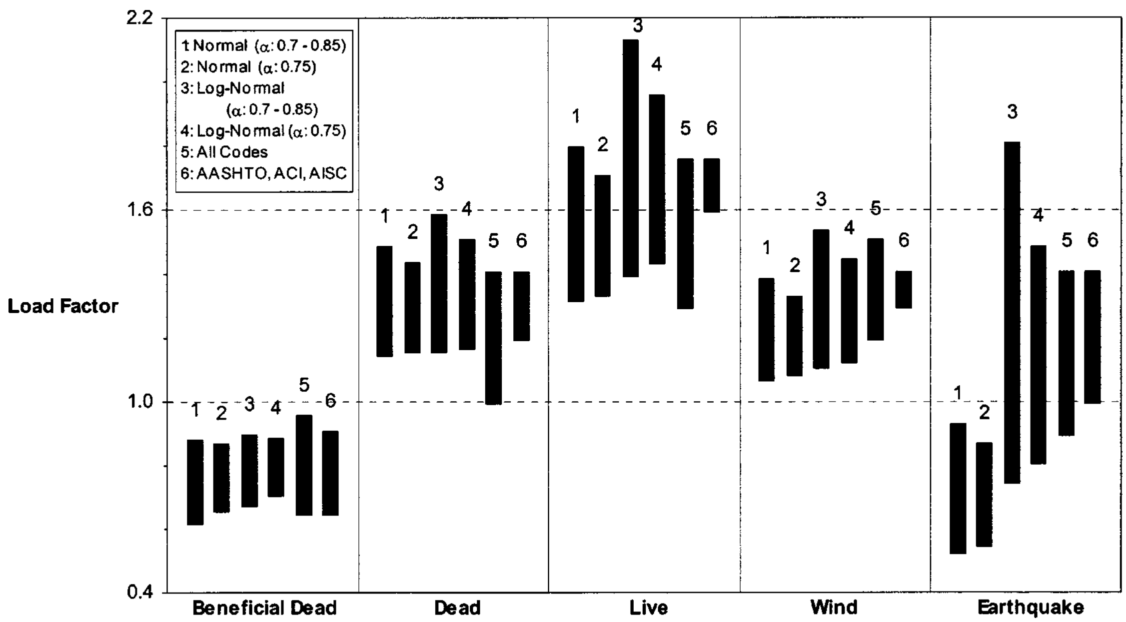

Fig. 3. Comparison of analysis and codes

which leads to a large number of different resistance factors, adds to the overall complexity of LRFD for the practicing engineer and ultimately discourages the use of this design method. Our analysis shows that, in general, the load factors proposed by different codes are all acceptable from a theoretical standpoint. Ideally, in order to facilitate the use of LRFD in routine practice, the leadership of the organizations responsible for each code would join in adopting a single set of load factors, at least for the primary loads, such as the four load types discussed in this paper (i.e., dead, live, wind, and earthquake loads). We recognize this is difficult to accomplish, as it involves overcoming nontechnical, political hurdles. The alternative is for engineers to become used to using different load and resistance factors when designing the same type of foundation element depending on the code controlling design.

Summary and Conclusions

The load factors proposed by various current structural and foundation LRFD codes were reviewed. Usually, a larger number of limit states, load types, and load combinations are considered in the bridge and offshore foundation design codes, compared with building and onshore foundation design codes. In this study, the load factors for four major load types (i.e., dead, live, wind, and earthquake loads) that control most design cases were examined and compared between the codes.

For ULSs, the load factor values fall within rather consistent ranges for most load types considered. Differences appear in the dead and the live load factors between the building and the bridge codes. For the bridge codes, the values of dead load factors lie within a relatively wide range. This is because, for bridge design, more types of loads are usually defined as dead loads, for which different values of load factors are used to account for the different degrees of uncertainty inherent in each load. While the use of a large number of different load factors adds to the complexity of a code, it also adds to the utility of the code. When a greater number of load factors are used, the uncertainties due to each load type are better separated. This separation of uncertainties is the ultimate goal of LRFD. The bridge codes also define different values of live load factors for different load combinations (i.e., different limit states) instead of using load combination factors to account for the reduced probability of a simultaneous occurrence of maximum values of several transient loads. When considering a gravity load combination, however, the values for the dead and the live load factors are reduced to a rather narrow range for all of the codes, resulting in ranges consistent with other load types examined.

For SLSs, some differences appear again between the bridge and building codes. While most codes prescribe the use of unfactored loads, AASHTO (1998) and MOT (1991) use values less than 1.0 for wind and both wind and live load factors, respectively. This reflects the differences in how each code prescribes the determination of the characteristic wind load, as well as the transient nature of the live load for bridges. However, an argument can be made against using load factors less than one, except when the foundation soil is clay.

A simple FOSM reliability analysis was implemented to find appropriate ranges of the load factor values for each load considered in this study. The analysis produced results consistent with all the codes reviewed, although the values produced lie in rather wide ranges due to the relatively wide range of the input parameters. The analysis shows even better agreement with the codes when considering only the U.S. codes (AASHTO, ACI, and AISC). The values presented in the U.S. codes lie in the middle of the acceptable range determined by the analysis, as summarized by Fig. 3. As the analysis uses the least biased distribution to model the loads, load factors for use in geotechnical LRFD should not lie outside the range determined by the current research unless that load factor applies to a specific type of load that is not considered in this research.

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Notation

The following symbols are used in this paper:

H = entropy;
k_R = bias factor of resistance;
k_S = bias factor of load effect;
LF = load factor;
$(\mathrm{LF})_D$ = dead load factor;
$(\mathrm{LF})_L$ = live load factor;
$(\mathrm{LF})_W$ = wind load factor;
P_f = probability of failure;
p_i = probability of event i;
R = resistance;
$\bar{R}$ = mean resistance;
RF = resistance factor;
R_n = nominal resistance;
S = load effects;
$\bar{S}$ = mean load effect;
S_D = nominal dead load effect;
S_L = nominal live load effect;
S_n = nominal load effect;
S_W = nominal wind load effect;
V_R = coefficient of variation of resistance;
V_S = coefficient of variation of load effect;
α = separation coefficient;
β = reliability index;
β_T = target reliability index;
η_a = analysis factor;
η_f = fabrication factor;
η_m = material factor; and
σ = standard deviation.

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