Observations on geotechnical reliability-based design development in North America

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ABSTRACT: There is little dispute that the current geotechnical design process could be improved significantly by integrating the various design components (loads, soil parameters, calculation models, and factors of safety) in a more logical and self-consistent way. Reliability-based design (RBD) is the only methodology available to date that can ensure self-consistency from both physical and probabilistic requirements and is compatible with the theoretical basis underlying structural design. A comparison of structural and geotechnical LRFD developments in North America was undertaken to re-focus on the necessity of using RBD to address basic design issues, as opposed to the prevailing emphasis on resistance factors. The importance of addressing various RBD issues specific to geotechnical engineering in a robust manner was illustrated with reference to a rigorous RBD study sponsored by EPRI.

1 INTRODUCTION

The basis for making a geotechnical design decision is not as well studied nor subjected to the same degree of formal scrutiny as structural design. Goble (1999) noted in an NCHRP Synthesis Report on Geotechnical Related Development and Implementation of Load and Resistance Factor Design (LRFD) Methods that the "education of geotechnical engineers strongly emphasizes the evaluation of soil and rock properties" and "the design process does not receive the emphasis that it does in structural engineering education". In fact, the first time the topic "Codes and Standards" was selected for formal discussion in a major ISSMGE conference was in 1989 (Ovesen 1989). Examination of current practice shows that procedures for selecting nominal soil strengths are not well-defined or followed uniformly. Some engineers use the mean value, while others use the most conservative of the measured strengths (Whitman, 1984). Different calculation methods are preferred in different localities or even by different engineers in the same locality (Goble 1999). The manner in which the factor of safety is incorporated in the design equation also is highly varied (Kulhawy 1984, 1996). Green & Becker (2001) made similar observations in a National Report on Limit State Design in Geotechnical Engineering in Canada.

Golder (1966) noted quite aptly in a discussion of the second Terzaghi Lecture by Arthur Casagrande that:

"We do not know how we make a decision."

Ironically, this view is rarely acknowledged publicly in the geotechnical engineering profession. The predominant view ranges from "If it is not broke, why fix it" (Green & Becker 2001) to a general feeling that conventional practice is perfectly adequate to do optimal design (Committee on Reliability Methods 1995, Kulhawy 1996). It must be emphasized at this point that the authors are not critical of existing practice, which has undoubtedly served the profession well for many years, but are critical of the reluctance to evaluate reliability-based design (RBD) methodologies that are capable of mitigating numerous logical inconsistencies inherent in current geotechnical design. No one is advocating total abandonment of existing practice for something entirely new. In fact, the reverse is probably closer to the truth - many aspects of current design practice would still appear in new RBD codes, albeit in a modified form (Kulhawy & Phoon 1996). It is also important to highlight that this paper is not concerned with the need for standardization of procedures for code harmonization and the merits/demerits associated with such "codification". However, it is a central theme in this paper that the loadings, calculation methods, and derivation of soil parameters must be made explicit and be calibrated together for resistance factors to be used in any meaningful way. More importantly, a fundamental change in mindset, similar to what has taken place in the structural community since the 1970s, is needed for the profession to take the next step. It is accurate to say that this change has not taken place in the geotechnical community in North America (Goble 1999, DiMaggio et al. 1999, Green & Becker 2001), although significant initiatives have been launched by major agencies in recent years such as OHBDC3 (Ministry of Transportation Ontario 1992), CAN/CSA-S472-92 (CSA 1992a), API RP 2A-LRFD (API 1993), EPRI (Phoon et al. 1995), NBCC (National Research Council of Canada 1995), AASHTO LRFD Bridge Code (AASHTO 1997), and CHBDC (CSA 2000).

Currently, the geotechnical community is mainly preoccupied with the transition from working or allowable stress design (WSD/ASD) to Load and Resistance Factor Design (LRFD). The term "LRFD" is used in a loose way to encompass methods that require all limit states to be checked using a specific multiple-factor format involving load and resistance factors. This term is used most widely in the United States and is equivalent to "Limit State Design (LSD)" in Canada. Both LRFD and LSD are philosophically akin to the partial factors approach commonly used in Europe, although a different multiple-factor format involving factored soil parameters is used. The emphasis in LRFD or its equivalent in Canada and Europe is primarily on the re-distribution of the original global factor safety in WSD into separate load and resistance factors (or soil parameter partial factors). The absence of strong analytical calibration and verification in Eurocode 7 (CEN/TC250 1994) and OHBDC3 (Ministry of Transportation Ontario 1992) is noted by DiMaggio et al. (1999) in an FHWA Report on "Geotechnical Engineering Practices in Canada and Europe". Paikowsky & Stenersen (2000) also noted a similar lack of data supporting current AASHTO LRFD specifications.

This paper advocates the need to re-focus on basic design issues rather than the format of the design check and the way in which the original global factor of safety is rearranged. There is little dispute that the current geotechnical design process could be improved significantly by integrating the various design components (loads, soil parameters, calculation models, and factors of safety) in a more logical and self-consistent way. Reliability-based design (RBD) is the only methodology available to date that can ensure self-consistency from both physical and probabilistic requirements and is compatible with the theoretical basis underlying structural design. The term "RBD" refers to any design methodology that is firmly founded on a rigorous reliability basis. It should be considered as a *necessary* theoretical basis for all geotechnical LRFD implementations. This basis is contrary to prevailing developments, in which LRFD is taken as a "given", while reliability calibration is relegated to a minor supporting role or even is optional. The technical details involved in RBD calibration and the use of LRFD as one possible simplified RBD format are given in another paper in these proceedings (Phoon & Kulhawy 2002). This paper focuses on the necessity of using RBD as a unifying framework and the importance of various RBD issues specific to geotechnical engineering.

2 HISTORICAL OVERVIEW

2.1 *Structural LRFD*

The classical structural reliability theory became widely known through a few influential publications such as Freudenthal (1947) and Pugsley (1955). The fundamental philosophy is that absolute reliability is an unattainable goal in the presence of uncertainty. Probability theory can provide a formal framework for developing design criteria that would ensure that the probability of "failure" (used herein to refer to exceeding of any prescribed limit state) is acceptably small. While the philosophy is elegant, the theory is mathematically intractable and numerically cumbersome. Cornell (1969) probably was the first to introduce the concept of a reliability index for simplified probabilistic design. Only second-moment information (mean and covariance) on uncertain parameters was needed and the computation was made simple by adopting the Gaussian model for random variables. However, the idea still was rather radical and could have been ignored if not for Lind (1971), who demonstrated that Cornell's reliability index could be used to derive load and resistance factors formally. The ability to repackage probabilistic design into a simplified multiple-factor design format with the same “look and feel” as existing design formats, while retaining theoretical rigor, is an important development from a practical point of view. To the authors' knowledge, LRFD was first implemented for steel building structures by Ravindra & Galambos (1978) using the theoretical basis established by Cornell (1969) and Lind (1971).

In the meantime, serious theoretical difficulties were encountered with Cornell's index, with the most severe being the problem of invariance. Cornell's index was found to vary when certain simple limit states were reformulated in a mechanically equivalent way. Although second-moment reliability-based structural design was becoming widely accepted in the early seventies, the goal of developing simplified design criteria firmly founded on a rigorous reliability basis remained elusive. This unsatisfactory condition was resolved eventually by Hasofer & Lind (1974), when they proved mathematically that the nearest distance of the limit state function from the origin of a standard Gaussian space is an invariant measure of reliability. This major theoretical breakthrough enforces invariance while retaining the practical second-moment simplification of Cornell's index. The last piece of significant addition to the theoretical repertoire for solving time-invariant reliability problems was the algorithm of Rackwitz & Fiessler (1978), which provided a practical and computationally efficient recipe for computing this reliability index with no restriction on the number of random variables. The reliability method proposed by Hasofer & Lind (1974) and its subsequent generalizations to handle non-Gaussian and correlated random variables commonly is called the First-Order Reliability Method (FORM). Ellingwood et al. (1980) were probably the first to apply FORM in a comprehensive way for simplified probabilistic design. Their study primarily presented load factors for buildings that were calibrated rationally using FORM and available statistical data.

The above review may not do justice to the voluminous research conducted in structural reliability over the past forty or so years. However, it does provide an overview of the historical development of structural LRFD and the accompanying key theoretical advances supporting this development. In the aftermath of recent natural hazards (e.g., Northridge and Kobe earthquakes), the structural engineering profession currently is focusing on performance-based design aimed at meeting client-specific performance goals, in addition to complying with local building codes (Wen 2000). Efficient techniques for solving time-dependent reliability problems are needed for such problems. Clearly, theoretical developments in structural reliability and applications to probabilistic design are still being pursued actively in the structural community.

2.2 Geotechnical LRFD

The development of geotechnical LRFD has taken a different track. One of the first efforts to rationalize foundation design can be attributed to Hansen (1965), who recommended separate checks for ultimate and serviceability limit states. In contrast, existing WSD often uses the global factor of safety for indirect control of serviceability. Hansen (1965) also recommended the use of partial factors for loads and soil parameters. These partial factors of safety were determined subjectively based on two guidelines: (a) a larger partial factor should be assigned to a more uncertain quantity, and (b) the partial coefficients should result in approximately the same design dimensions as that obtained from traditional practice. Ovesen (1989) highlighted the direct application of a partial factor to the source of uncertainty (soil parameter) as a notable improvement. The partial factors of safety suggested in 1965 were adopted in the Danish Code of Practice for Foundation Engineering (DGI 1978, 1985) with minor modifications. More recent implementations include the Canadian Foundation Engineering Manual, CFEM, third edition (Canadian Geotechnical Society 1992), Geoguide 1, second edition (Geotechnical Engineering Office 1993), and Eurocode 7 (CEN/ TC250 1994).

In North America, the factored resistance approach is the preferred design format. LRFD procedures for bridge superstructures and substructures were introduced in Canada in 1979 as part of the first edition of the Ontario Highway Bridge Design Code, OHBDC1 (Ministry of Transportation Ontario 1979, Green 1991). Green (1991) further noted that these procedures were "basically a simple rearrangement of factor of safety design provisions". The design of deep foundations for power generating stations (Ontario Hydro 1985) broadly followed the second edition, OHBDC2 (Ministry of Transportation Ontario 1983, Klym & Lee 1989). OHBDC is currently in its third edition (OHBDC3), but the foundation resistance factors are not based on reliability calibrations (Green & Becker 2001). In contrast, OHBDC for superstructures was calibrated using reliability theory in its second edition (Grouni & Nowak 1984). The target reliability indices selected were 3.5 for ultimate limit state and 1.0 for serviceability limit state (Nowak & Lind 1979).

For fixed offshore platforms, the Canadian standard for foundations, CAN/CSA-S472-92 (CSA 1992a), contains no specification of resistance factors for foundation design, although reliability-based LRFD is available for structural design (Been et al. 1993). Been et al. (1993) further noted that resistance factors were calibrated to the global factor of safety in an earlier 1989 draft commentary (CSA 1989) but were dropped in the 1992 version (CSA 1992b).

The main geotechnical design manual in Canada is the Canadian Foundation Engineering Manual, CFEM (Canadian Geotechnical Society 1992). As noted previously, the 1992 version (third edition) is based on the partial factors of safety approach, although the fourth edition currently under preparation is expected to be revised to be consistent with the factored resistance format (Green & Becker 2001). The partial factors of safety in the third edition were calibrated so that they result, on average, in overall factors of safety that are in agreement with existing practice (Meyerhof 1984). To the authors' knowledge, CFEM is the only design guide that indirectly recognizes the difficulty of using a single partial factor for each soil parameter to cover the wide range of design equations in which the same soil parameter can appear. Resistance modification factors and performance factors were recommended to ensure more reasonable agreement with existing practice. However, this procedure is not entirely successful, as noted by Baike (1985) and Valsangkar & Schriver (1991). The conflict between the need for simplicity or using small numbers of partial factors of safety, and the need to produce designs comparable with existing practice, does not appear to lend itself readily to simple solutions.

The development of LRFD for foundations in the 1995 National Building Code of Canada (NBCC) followed a semi-analytical approach (Becker 1996b). Becker (1996b) opined that a full reliability-based LRFD is difficult to apply because of a lack of statistical data and it is time-consuming and expensive. Therefore, the resistance factors for foundation design were calibrated to fit WSD and to be consistent with a lumped parameter lognormal reliability formula:

$$\beta = \frac{\log_e\left(m_{FS}\sqrt{\dfrac{1+COV_F^2}{1+COV_Q^2}}\right)}{\sqrt{\log_e\left[\left(1+COV_F^2\right)\left(1+COV_Q^2\right)\right]}} \tag{1}$$

in which β = reliability index, $m_{FS} = m_Q/m_F$ = mean factor of safety, m_Q and m_F = mean of capacity (Q) and load (F), $COV_Q = s_Q/m_Q$ = coefficient of variation (COV) of capacity, $COV_F = s_F/m_F$ = COV of load, and s_Q and s_F = standard deviation of capacity and load. Note that the reliability index is uniquely related to the probability of failure but in a highly nonlinear way as shown in Table 1. In the LRFD literature, the probability of failure is used less commonly because it carries the negative connotation of "failure". A target reliability index of 3.5 was used in NBCC for foundation design. As a reference, the NBCC for structural design was calibrated using a target β of 3.5 for ductile behavior with normal consequence of failure and a target β of 4.0 if either the consequence of failure is severe or the failure mode is brittle (Becker 1996b).

In the United States, the resistance factors for design of foundations in the AASHTO LRFD Bridge Code (AASHTO 1997) were derived from NCHRP Report 343 (Barker et al. 1991). The main rationale is to remove the inconsistency between load factor design for superstructures and allowable stress design for foundations, which has resulted in duplication of design efforts because two sets of loads must be evaluated (Rojiani et al. 1991). The resistance factors appear to be determined using a mixture of judgment, calibration with WSD, and reliability analysis. This is reported in Part 6 - Recommended Load Factor Design Specifications and Commentary, Section C 4.10.6 of NCHRP Report 343:

"Where statistical information was available, reliability theory, tempered in some cases with judgment, was used to derive the values of performance factors given in Tables 4.10.6-1 through 4.10.6-3. In cases where there was insufficient information for calibration using reliability theory, values of performance factors were chosen based on judgment, so that the design was consistent with that using ASD procedures …

In deriving the values of performance factors given in Tables 4.10.6-1 through 4.10.6-3, the target reliability indices were chosen as 2.0 to 2.5, 2.5 to 3.5, and 3.5, respectively, for driven piles, drilled shafts and spread footings."

Table 1. Relationship between reliability index (β) and probability of failure (p_f) [Source: US Army Corps of Engineers (1997), p. B-11].

Reliability index β	Probability of failure $p_f = \Phi(-\beta)$	Expected performance level
1.0	0.16	Hazardous
1.5	0.07	Unsatisfactory
2.0	0.023	Poor
2.5	0.006	Below average
3.0	0.001	Above average
4.0	0.00003	Good
5.0	0.0000003	High

Note: $\Phi(\cdot)$ = standard normal probability distribution

Reliability analysis seems to be used in a limited way using Equation 1 (Rojiani et al. 1991, Yoon & O'Neill 1997). The risk levels implied by an extensive range of existing calculation procedures (e.g., rational methods, semi-empirical methods, in-situ methods) formed the basis for the target reliability indices. Paikowsky & Stenersen (2000) further noted that the current AASHTO specifications "were developed using insufficient data, hence they utilized mostly back-calculated factors". Most interestingly, the target reliability index for bridge superstructures is 3.5 (Grubb 1997), which is much higher than the target reliability indices quoted in NCHRP Report 343. Research is currently in progress under Project NCHRP 24-17 to revise the driven pile and drilled shaft sections of AASHTO specifications (Paikowsky & Stenersen 2000).

For API RP 2A-LRFD (API 1993), foundations are treated as one of the structural elements in the RBD calibration process (Moses & Larrabee 1988). A lumped resistance parameter with a bias of 1.0 and a coefficient of variation (COV) of 20% was assumed for pile capacity. The main theoretical basis appeared to be Equation 1, although FORM was used for verification as well. The foundation resistance factor was adjusted to achieve an average reliability index of 2.2 for pile axial capacity. A lumped resistance model also was assumed for transmission line structure foundations in the ASCE Manual & Report 74 (Task Committee on Structural Loadings 1991) to preserve a common reliability calibration scheme for structural and foundation components. Resistance factors for a range of lumped resistance COVs (20 to 50%) and target probabilities of failure (0.25% to 1%) were presented. The range of probability of failure corresponds to reliability indices between 2.3 and 2.8. Geotechnical considerations were marginalized because API RP 2A-LRFD and ASCE Manual & Report 74 were focused on structural design. The first attempt to develop simplified RBD specifically for transmission line structure foundations with primary emphasis on geotechnical considerations was described in EPRI Report TR-105000 (Phoon et al. 1995). A number of geotechnical aspects in this study are of general applicability and will be discussed in greater detail later.

3 WHAT NEXT?

A comparison between the historical development of structural and geotechnical LRFD revealed a conspicuous difference. Structural LRFD is essentially the logical end-product of a philosophical shift in mindset to probabilistic design in the first instance and a simplification of rigorous reliability-based design into a familiar "look and feel" design format in the second. In stark contrast, geotechnical LRFD took place in an environment where the relevance of probabilistic design still is being debated (Committee on Reliability Methods 1995, Whitman 2000) and predominantly involved a rearrangement of existing global factors of safety into a new design format. More often than not, resistance factors are discussed without clear and explicit reference to the loads, the definition of the characteristic/nominal soil parameters, the method of transforming soil parameters to design parameters, and the bias in the calculation methods.

3.1 Simple design example

Figure 1 illustrates the four basic components (loads, soil parameters, calculation model, and safety factors) that a designer needs to consider for sizing the footings of a structure. A different design decision will be made whenever one or more of the components are changed. Kulhawy (1984, 1996) discussed the wide variability of practice using a simple example shown in Table 2. The problem is to calculate the design uplift capacity of a straight-sided drilled shaft in saturated clay, 1.5 m in diameter and 1.5 m deep, with average mobilized undrained shear strength along the vertical shaft surface equal to 38 kN/m^2 and a potential tip suction equal to 0.5 atmosphere acting over the tip area during undrained transient live loading. These data were given in 1984 to a number of experienced designers who were asked to compute the design capacity using their "normal" design practices with a factor of safety (FS) equal to 3 for illustration purposes only. This exercise resulted in the five different design assumptions and capacities depicted in the first three columns of Table 2. Column 4 gives the ratio of the available uplift capacity to the recommended design value, which can be interpreted as the "actual" FS, regardless of the nominal FS used in the design calculations.

Assumption 1 considered the side, tip, and weight components equally, while assumption 2 subtracted the weight from the design capacity to give a so-called "net design capacity". Assumptions 3 and 4 were similar to 1 and 2, except that the tip resistance was disregarded conservatively. Assumption 5 was an extreme case adopted by one designer, in which both the side and tip resistances were disregarded. However, this designer would only use a FS in this equation a bit greater than 1 (typically on the order of 1.25), which would give an "actual" FS = 9.8. It turned out that Assumption 3 was the most common.

For this simple case, in which all of the component capacities and the FS were given, the results still varied by almost a factor of two, disregarding the uncommon Assumption 5. If the designers had been free to select the procedure for computing each component capacity, as well as the FS, then even more variability could ensue. This exercise was repeated in 1995, with similar results.

Examination of most recent reference manuals and texts reveals no firm or rigorous advice on most of these issues. More often than not, several design equations are given to evaluate side resistance, a range of bearing capacity factors is cited, numerous lateral loading models are presented, etc. Then a variety of ways are cited to evaluate the geotechnical input properties necessary to do these calculations. Finally, typical FS values are quoted, most commonly in the range of 2 to 3, which is essentially the same as was done fifty years ago. Obviously, there have to be differences resulting from different designer engineers using different combinations of equations, property evaluation methodologies, and FS values. If there have been local calibrations of all of these factors with local load tests, then a sound design procedure could ensue. If not, then how sound is the

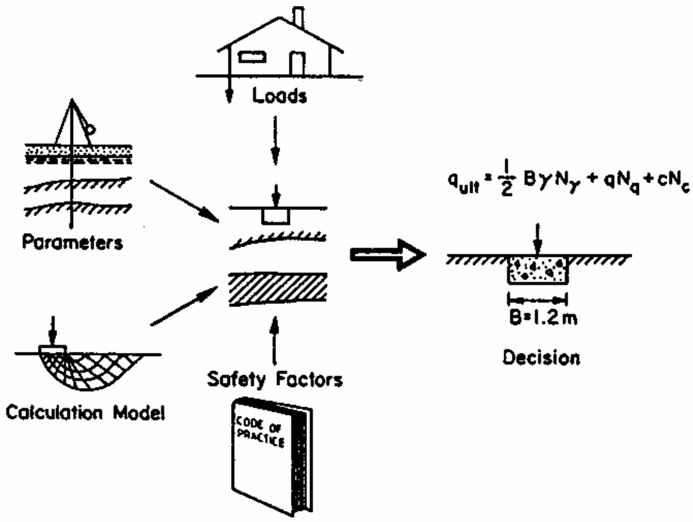


Figure 1. Components of geotechnical design (Modified from Ovesen 1989).

Table 2. Design capacity example (Source: Kulhawy 1984).

No.	Design Equation	Q_{ud} (kN) for FS = 3	Q_u/Q_{ud} "actual" FS
1	$Q_{ud} = (Q_{su} + Q_{tu} + W) / FS$	170.7	3.0
2	$Q_{ud} - W = (Q_{su} + Q_{tu}) / FS$	214.2	2.4
3	$Q_{ud} = (Q_{su} + W) / FS$	108.9	4.7
4	$Q_{ud} - W = Q_{su} / FS$	152.4	3.4
5	$Q_{ud} = W / FS$	21.8	23.5

Note: Q_{su} = side resistance = 261.8 kN, Q_{tu} = tip resistance = 184.4 kN, W = shaft weight = 65.3 kN, Q_{ud} = design uplift capacity, FS = factor of safety, Q_u = available uplift capacity = $Q_{su} + Q_{tu} + W$ = 511.6 kN design?

It is timely to pause and ask ourselves if a change in design format is a sufficiently strong reason to move away from a known WSD procedure to a largely untested LRFD procedure. If the theoretical reliability basis for code calibration remains relegated to a minor or optional role, no genuine compatibility with structural LRFD codes can be claimed, even if this is one of the major objectives in current geotechnical LRFD initiatives. Aside from prescribing resistance factors, the other design components are still essentially left to the discretion of the design engineers, which means that current geotechnical LRFD is subjected to the same inconsistencies inherent in WSD. If so, why change at all?

3.2 Limitations of "judgment"

In the meantime, it is accurate to say that the chasm between structural and geotechnical LRFD is widening with the advent of performance based design and the gradual shift from time-invariant reliability, which forms the theoretical underpinning for all existing structural RBD codes, toward time-dependent reliability. This is not to say that the geotechnical profession should follow the developments in structural reliability indiscriminately; the authors have cautioned against such a simplistic application since 1993 (Phoon et al. 1993). However, RBD is currently the only rational vehicle that has the potential to bring about solid improvement to the state of practice. Insufficient recognition is given to the propensity of making mistakes in reasoning with uncertain quantities.

An example given in Figure 2 illustrates one such common mistake. The problem statement for this example is to compute the probability of failure of a structural member, assuming that the load and strength are independent and that the probability of understrength is 1 in 10^2, while the probability of overload is 1 in 10^3. The intuitive answer might be 1 in 10^5, which represents the probability of having an overload and an understrength member. The reason this answer is incorrect is given in Figure 2, which shows that the combination of overload and understrength (area B) is only one of three possible scenarios in which the member can fail. The other two possibilities involve certain combinations of typical load with understrength member (area A) and overload with typical member strength (area C). In this example, the error happens to be on the unconservative side, because the actual probability of failure is larger than 1 in 10^5. The potential for making mistakes in manipulating uncertainties on an intuitive basis, especially in the case of real problems where large numbers of interdependent uncertain quantities are involved, is very real and must be avoided.

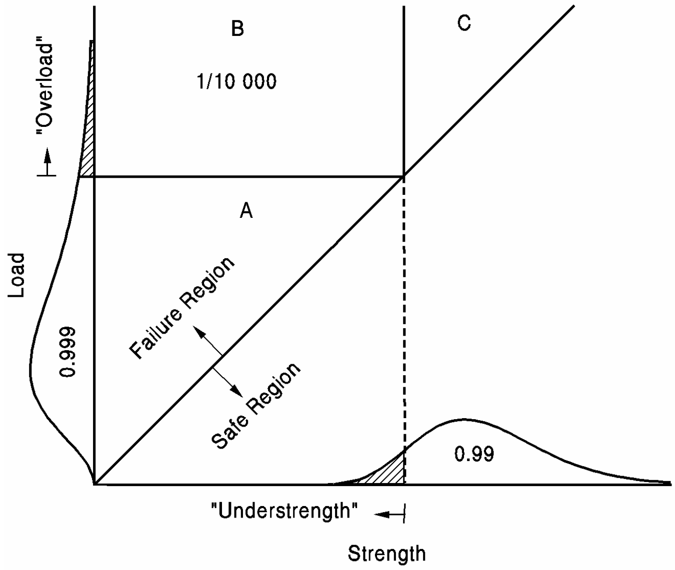


Figure 2. Evaluation Components of geotechnical design (Example from MacGregor 1976).

Another example in which intuition can be misleading is the common misconception that a larger partial factor should be assigned to a more uncertain soil parameter. This is not necessarily correct because the parameter may have little influence on the overall capacity equation. The magnitude of the partial factor therefore should depend on the uncertainty of the parameter and the sensitivity of the capacity equation to that parameter. This important factor has been highlighted by Simpson et al. (1981) as well. Reliability theory will correctly incorporate this important sensitivity issue in the computation of the reliability index.

The rest of this paper is devoted to important geotechnical considerations related to the implementation of rigorous RBD for foundation engineering. In particular, the authors hope to revise two widely held perceptions: (a) statistical data are lacking, and (b) there is no room to exercise judgment and creativity in probabilistic design.

4 GEOTECHNICAL VARIABILITY

The evaluation of soil and rock properties is one of the key design aspects that distinguishes geotechnical from structural engineering. None of the current geotechnical LRFD implementations consider this important issue explicitly. The purpose of this section is to highlight two important observations: (a) geotechnical variability is a complex attribute that needs careful evaluation, and (b) extensive statistical data are available for use as first-order estimates in RBD calibration and application.

There are three primary sources of geotechnical uncertainties: (a) inherent variabilities, (b) measurement uncertainties, and (c) transformation uncertainties (Phoon and Kulhawy 1999a). The first results primarily from the natural geologic processes that produced and continually modify the soil mass in-situ. The second is caused by equipment, procedural and/or operator, and random testing effects. Equipment effects result from inaccuracies in the measuring devices and variations in equipment geometries and systems employed for routine testing. Procedural and/or operator effects originate from the limitations in existing test standards and how they are followed. In general, tests that are highly operator-dependent and have complicated test procedures will have greater variability than those with simple procedures and little operator dependency, as described in detail elsewhere (Kulhawy & Trautmann 1996). Random testing error refers to the remaining scatter in the test results that is not assignable to specific testing parameters and is not caused by inherent soil variability.

Collectively, the first two sources can be described as data scatter as shown in Figure 3. These in-situ measurements also are influenced by statistical uncertainty or sampling error that results from limited amounts of information. This uncertainty can be minimized with increased testing, but it is commonly included within the measurement error at this time. The results for inherent soil variability are summarized in Table 3.

The third component of uncertainty is introduced when field or laboratory measurements are transformed into design soil properties using empirical or other correlation models (e.g., correlating the standard penetration test N value with the undrained shear strength). Obviously, the relative contribution of these components to the overall uncertainty in the design soil property depends on the site conditions, degree of equipment and procedural control, and quality of the correlation model. Therefore, soil property statistics that are determined from total variability analyses only can be applied to the specific set of circumstances (site conditions, measurement techniques, correlation models) for which the design soil properties were derived.

A comprehensive effort was undertaken to provide realistic soil statistics of sufficient generality to underpin current and future developments of practical RBD procedures (Phoon and Kulhawy, 1999a, 1999b; Kulhawy et al., 2000). Such an undertaking is clearly ambitious, but it is absolutely necessary if practicing engineers are to be convinced of the merits and soundness of the new RBD methodology. Green & Becker (2001) shared the same sentiment that "calibration procedures must be used in which the variability of the soil is fully recognized", but they did not seem to feel that statistics could be developed easily because of the site-specific nature of soil variability. This concern only is true for total variability analyses but does not apply to the general approach briefly outlined below. For each combination of soil type, measurement technique, and correlation model, the uncertainty in the design soil property is evaluated systematically by combining the appropriate component uncertainties using a simple second-moment probabilistic approach:

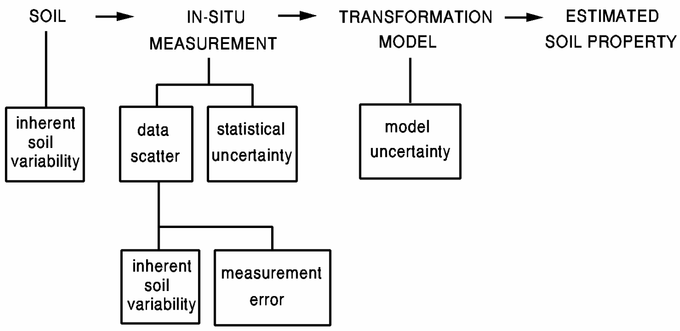


Figure 3. Uncertainty in soil property estimates.

$$s_{\xi d}^2 \approx \left(\frac{\partial T}{\partial w}\right)^2 s_w^2 + \left(\frac{\partial T}{\partial e}\right)^2 s_e^2 + \left(\frac{\partial T}{\partial \varepsilon}\right)^2 s_\varepsilon^2 \qquad (2)$$

in which $\xi_d = T(\xi_m, \varepsilon)$, $T(\cdot)$ = correlation function between test measurement (ξ_m) and design soil property (ξ_d), ε = transformation uncertainty, w = inherent variability, e = measurement error, and s^2 = variance. Useful guidelines on typical coefficients of variation of many common design soil strength properties have been summarized by Phoon and Kulhawy (1999b) and are given in Table 4 for reference.

5 KEY RELIABILITY ISSUES

5.1 Issues in need of attention

The primary focus of most geotechnical LRFD calibration is to produce designs that are consistent with existing practice using WSD. Various approaches such as judgment, rearrangement of traditional factor of safety, simplified reliability analysis, or some combinations thereof, with limited statistical support, have been used to derive geotechnical resistance factors. Although the resulting geotechnical LRFD codes look the same as their structural counterparts, they are fundamentally incompatible with reliability-based structural codes because one or more of the following key elements are missing:

a. The primary objective in structural RBD is to achieve a minimum target reliability index across a specified domain of interest (e.g., foundation geometries and types, loading modes, soil conditions, etc.). Structural RBD requires deliberate and explicit choices to be made on the target reliability index, scope of calibration domains, and representative designs populating each domain. This is philosophically different from the objective of achieving designs comparable to WSD.
b. The secondary objective in structural RBD is to increase uniformity of reliability across the domain of interest, which is rarely emphasized and verified in geotechnical LRFD. In fact, the typical use of a

Table 3. Approximate guidelines for inherent soil variability (Source: Phoon and Kulhawy 1999a).

Test type	Property[a]	Soil type	Mean	COV(%)
Lab strength	s_u(UC)	Clay	10-400 kN/m^2	20-55
	s_u(UU)	Clay	10-350 kN/m^2	10-30
	s_u(CIUC)	Clay	150-700 kN/m^2	20-40
	ϕ'	Clay & sand	20-40^o	5-15
CPT	q_T	Clay	0.5-2.5 MN/m^2	< 20
	q_c	Clay	0.5-2.0 MN/m^2	20-40
		Sand	0.5-30.0 MN/m^2	20-60
VST	s_u(VST)	Clay	5-400 kN/m^2	10-40
SPT	N	Clay & sand	10-70 blows/ft	25-50
DMT	A reading	Clay	100-450 kN/m^2	10-35
		Sand	60-1300 kN/m^2	20-50
	B reading	Clay	500-880 kN/m^2	10-35
		Sand	350-2400 kN/m^2	20-50
	I_D	Sand	1-8	20-60
	K_D	Sand	2-30	20-60
	E_D	Sand	10-50 MN/m^2	15-65
PMT	p_L	Clay	400-2800 kN/m^2	10-35
		Sand	1600-3500 kN/m^2	20-50
	E_{PMT}	Sand	5-15 MN/m^2	15-65
Lab index	w_n	Clay & silt	13-100 %	8-30
	w_L	Clay & silt	30-90 %	6-30
	w_P	Clay & silt	15-25 %	6-30
	PI	Clay & silt	10-40 %	b
	LI	Clay & silt	10 %	b
	γ, γ_d	Clay & silt	13-20 kN/m^3	< 10
	D_r	Sand	30-70 %	10-40[c] 50-70[d]

a - s_u = undrained shear strength; UC = unconfined compression test; UU = unconsolidated-undrained triaxial compression test; CIUC = consolidated isotropic undrained triaxial compression test; ϕ' = effective stress friction angle; q_T = corrected cone tip resistance; q_c = cone tip resistance; VST = vane shear test; N = standard penetration test blow count; A & B readings, I_D, K_D, & E_D = dilatometer A & B readings, material index, horizontal stress index, & modulus; p_L & E_{PMT} = pressuremeter limit stress & modulus; w_n = natural water content; w_L = liquid limit; w_P = plastic limit; PI = plasticity index; LI = liquidity index; γ & γ_d = total & dry unit weights; D_r = relative density

b - COV = (3-12%) / mean

c - total variability for direct method of determination

d - total variability for indirect determination using SPT values

single resistance factor for each loading mode is not adequate for this task, as will be illustrated in Section 6.3.

c. Soil variability is the most significant source of uncertainty, but it is not quantified in a robust way (if at all) and incorporated explicitly in the code calibration process.
d. Probabilistic load models compatible with the relevant structural codes are not spelled out clearly. It is unclear if the original structural load models have been used for code calibration. Load combinations definitely are not amenable to simplified reliability analysis in the form of Equation 1 unless they are approximated as some lumped load parameters.
e. Rigorous reliability analysis using FORM is not used as the main tool to integrate loads, soil parameters, and calculation models in a realistic and self-consistent way, both physically and probabilistically.
f. No guidelines on selection of nominal or characteristic soil parameters are usually given. It is also unclear how resistance factors will be affected by the site conditions, measurement techniques, and correlation models used to derive the relevant design parameters.

In the FHWA report on "Geotechnical Engineering Practices in Canada and Europe", DiMaggio et al. (1999) recommended a ten-step plan for future implementations of AASHTO geotechnical LRFD, which include modifying the code to include model and soil reliability factors, clearly defining the characteristic value of soil parameters, and adoption of reliability-based calibration.

Table 4. Guidelines for design soil property variability (Source: Phoon and Kulhawy 1999b).

Design property[a]	Test[b]	Soil type	Point COV[c] (%)	Spatial avg COV[c] (%)
s_u(UC)	Direct (lab)	Clay	20-55	10-40
s_u(UU)	Direct (lab)	Clay	10-35	7-25
s_u(CIUC)	Direct (lab)	Clay	20-45	10-30
s_u(field)	VST	Clay	15-50	15-50
s_u(UU)	q_T	Clay	30-40	30-35
s_u(CIUC)	q_T	Clay	35-50	35-40
s_u(UU)	N	Clay	40-60	40-55
s_u	K_D	Clay	30-55	30-55
s_u(field)	PI	Clay	30-55	-
ϕ'	Direct (lab)	All	7-20	6-20
ϕ'(TC)	q_T	Sand	10-15	10
ϕ'_{cv}	PI	Clay	15-20	15-20
K_o	Direct (SBPMT)	Clay	20-45	15-45
K_o	Direct (SBPMT)	Sand	25-55	20-55
K_o	K_D	Clay	35-50	35-50
K_o	N	Clay	40-75	-
E_{PMT}	Direct (PMT)	Sand	20-70	15-70
E_D	Direct (DMT)	Sand	15-70	10-70
E_{PMT}	N	Clay	85-95	85-95
E_D	N	Silt	40-60	35-55

a - s_u = undrained shear strength; UU = unconsolidated-undrained triaxial compression test; UC = unconfined compression test; CIUC = consolidated isotropic undrained triaxial compression test; s_u(field) = corrected s_u from vane shear test; ϕ' = effective stress friction angle; TC = triaxial compression; ϕ'_{cv} = constant volume ϕ'; K_o = in-situ horizontal stress coefficient; E_{PMT} = pressuremeter modulus; E_D = dilatometer modulus

b - VST = vane shear test; q_T = corrected cone tip resistance; N = standard penetration test blow count; K_D = dilatometer horizontal stress index; PI = plasticity index

c - refer to coefficient of variation (COV) equations in Phoon and Kulhawy (1999b)

In the "National Report on Limit State Design in Geotechnical Engineering: Canada", Green & Becker (2001) probably had the same thoughts in mind when they emphasized that "the real essence of design and key question is: *What is the process that leads to the development of the resistance value?*" (italics by original authors).

The development of a fully rigorous RBD code that can handle the entire range of geotechnical design problems and can address all geotechnical concerns thoroughly is admittedly difficult and perhaps premature. With the exception of foundations, insufficient field test data are available to perform a robust statistical assessment of the average bias in many geotechnical design models (e.g., earth-retaining systems). However, considerable progress in rigorous RBD calibration of foundations has been made in an EPRI study by Phoon et al. (1995).

If the geotechnical engineering profession is indeed determined to proceed with RBD calibration in the next development phase, which recent publications seem to indicate (DiMaggio et al. 1999, Green & Becker 2001), the authors propose using this EPRI study as a platform for discussion and framework for modification and/or improvement to suit superstructure-specific requirements.

5.2 Conceptual basis for EPRI study

The scope of the study (EPRI report TR-105000 by Phoon et al. 1995) is comparatively less ambitious than other geotechnical LRFD codes, which allows full rigor and realistic in-depth development of RBD formats to be undertaken. Several key contributions can be identified:

a. Resistance factors are based on rigorous FORM calibration, realistic geotechnical predictive models with known bias (not lumped resistance models), and probabilistic load models compatible with the relevant structural code.
b. Deformation factors for the serviceability limit state (SLS) are calibrated in the same consistent manner as the resistance factors for the ultimate limit state (ULS).
c. The target reliability index for the ULS (3.2) was higher than that for the SLS (2.6). Both target reliability indices were selected based on extensive studies of existing designs and are applicable to a variety of loading modes that are common for transmission line structure foundations.
d. Uniformity of reliability is maximized by partitioning the domain of interest into several smaller sub-domains and using a more appropriate Multiple Resistance Factor Design (MRFD) format.

e. Specific guidelines on assessment of geotechnical variabilities are provided based on an extensive compilation and synthesis of available soil statistics and correlations.
f. Resistance factors are functions of the quality of soil data.

The geotechnical inputs on design models and soil property evaluation entering into this reliability calibration exercise are taken from a series of EPRI reports spanning more than fifteen years of transmission line structure foundation research under Project RP1493. Section 4 of EPRI TR-105000, relating to geotechnical variabilities, represents a culmination of four reports pertaining to site characterization (Spry et al. 1988), measurement errors (Orchant et al. 1988), Bayesian updating (Filippas et al. 1988), and transformation models (Kulhawy et al. 1992). Therefore, the research efforts and data collection required to formulate these robust geotechnical RBD formats are considerable.

In his development of geotechnical resistance factors for NBCC, Becker (1996b) took the position that a full reliability-based LRFD is difficult to apply because of a lack of statistical data and it is time-consuming and expensive. The former observation is no longer true for foundation engineering, but the latter is still accurate. However, the authors believe that these difficulties should add impetus to research in this area rather than provide reasons for maintaining the status quo or settling for an empirical or semi-analytical LRFD approach, because the push for RBD methodologies will continue, even with little participation from our profession (Committee on Reliability Methods 1995).

6 PRACTICAL IMPLEMENTATION ISSUES

This section presents some of the rationale underlying the numerous decisions made in the development of practical RBD equations in EPRI TR-105000. The full implementation details of this RBD approach are given elsewhere (Phoon & Kulhawy 2002).

6.1 Lumped resistance model

The closed-form solution shown in Equation 1 has been used as a convenient basis for developing some geotechnical LRFD codes (e.g., Barker et al. 1991, Becker 1996b, Yoon & O'Neill 1997, Paikowsky & Stenersen 2000). To apply this closed-form solution, it is necessary to simplify the actual load and capacity components as single lumped parameters. For foundation engineering, such an approach is not robust from both physical and statistical considerations.

It is well-known that the capacity of a foundation is composed of different components that are physically distinctive. For example, the uplift capacity of a drilled shaft during undrained loading is composed of side resistance, tip suction, and self-weight. These components, in turn, are generally nonlinear functions of more fundamental design parameters, such as the foundation geometry and the soil physical properties. The relative contribution of each component to the overall capacity also is not constant. For example, the relative importance of tip suction decreases with increasing foundation depth, because the maximum suction stress is limited to about one atmosphere, while the self-weight and side resistance can increase with no apparent limit. In addition, the degrees of uncertainty associated with the evaluation of these components are different. For example, the weight of the shaft is almost deterministic in comparison to the undrained side resistance, because the COV of the unit weight of concrete is significantly smaller than the COV of the undrained shear strength.

The single random variable (Q) that encapsulates the whole foundation capacity is, therefore, a complex function of many deterministic and statistical parameters. Deterministic parameters include foundation depth and diameter, while statistical parameters include the mean and COV of the undrained shear strength and the tip suction stress. Because of its underlying complexity, robust statistics for Q can not be obtained easily for reliability assessment. A common strategy for estimating the statistics of Q is to use available load test data and normalize the predicted nominal capacity (Q_n) by the measured capacity (Q_m) for each load test as (e.g., DiGioia & Rojas-Gonzalez 1991):

$$R = Q_n/Q_m \tag{3}$$

in which R = capacity ratio. From a series of capacity ratios, the mean (m_R) and standard deviation (s_R) are evaluated, and the statistics of Q then can be approximated by:

$$m_Q \approx Q_n/m_R \tag{4a}$$

$$COV_Q \approx COV_R = s_R/m_R \tag{4b}$$

in which COV_Q and COV_R = coefficients of variation of Q and R, respectively.

Some values of m_R and COV_R for lateral loading are presented in Table 5. As expected, the capacity ratio statistics depend on the predictive model used in the calculation of the nominal capacity (Q_n). Another observation is that the capacity ratio statistics also are dependent on the characteristics of the load test database. This effect can be seen in Table 5 by comparing the statistics from an undifferentiated soil type database (DiGioia & Rojas-Gonzalez 1991) with those obtained from two other groups of load tests classified according to soil type (Chen & Kulhawy 1994). This lack of robustness in the statistics is a direct outcome of lumping many significant factors into a single parameter. The application of these lumped parameter statistics for reliability assessment clearly is not justifiable

Table 5. Variability of capacity ratio statistics for Reese (1974) and Hansen (1961) lateral loading models (Source: Phoon & Kulhawy 1996).

Calc. model	Database	Soil type	No. Tests	Capacity ratio, R Mean	COV
Reese et al. 1974	DiGioia & Rojas 1991	Mixed	11	0.65	0.41
	Chen & Kulhawy 1994	Clay	68	0.78	0.28
	Chen & Kulhawy 1994	Sand	65	1.00	0.41
Hansen 1961	DiGioia & Rojas 1991	Mixed	11	0.79	0.38
	Chen & Kulhawy 1994	Clay	68	0.57	0.32
	Chen & Kulhawy 1994	Sand	65	1.07	0.26

unless the foundation and geotechnical characteristics for the design situation are comparable to those in the database. This condition, however, is not easy to satisfy, because the characteristics of natural soil deposits are quite variable, and the level of site investigation performed for load tests usually is not representative of that performed for routine designs. To apply the lumped parameter approach correctly, it would appear necessary to conduct representative load tests and evaluate the capacity ratio statistics for each new design situation.

In a recent study on LRFD for dynamic pile capacity prediction methods, Paikowsky & Stenersen (2000) attempted to mitigate some of the above shortcomings by carefully studying the parameters that influence the lumped resistance model. The load test database is organized into a hierarchy of groups and sub-groups based on these influential parameters. These lumped capacity ratio statistics are potentially more robust (how much?) than those obtained from an undifferentiated database. However, it is uncertain how the sub-groups are correlated and, more importantly, the major problem of extrapolating statistics from load test scenarios to actual design scenarios with different quality of soil data remains unresolved.

6.2 Characteristic value

The goal of RBD calibration can be illustrated qualitatively using Figure 4. It can be seen that the calibrated factors are used to ensure consistent separation between the probability density functions describing the uncertain load and capacity. Figure 4 also clearly highlights the importance of defining the nominal (or characteristic) load and capacity precisely.

The need to define nominal values in an unambiguous way also can be demonstrated analytically by using a well-known approximation to Equation 1 (Rosenblueth & Esteva 1972):

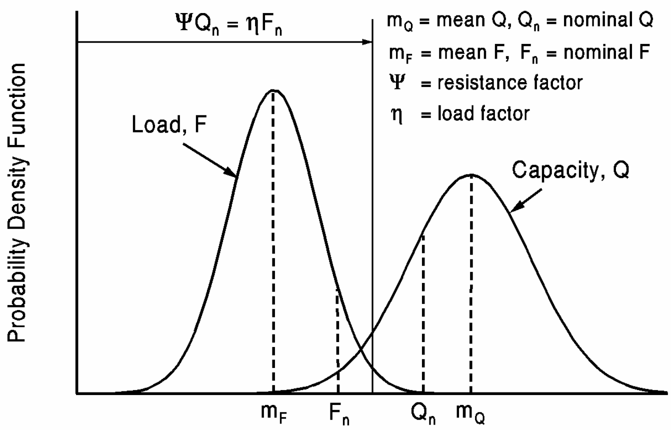


Figure 4. Simplified reliability-based design.

$$\beta = \frac{\log_e(m_{FS})}{\sqrt{COV_F^2 + COV_Q^2}} \tag{5}$$

To derive the LRFD format, the denominator in Equation 5 must be linearized as follows (Lind 1971):

$$\sqrt{COV_F^2 + COV_Q^2} \approx 0.75(COV_F + COV_Q) \tag{6}$$

Substituting Equation 6 into 5 and rearranging the terms to fit the LRFD format shown in Figure 4, it can be shown that:

$$\Psi = \frac{m_Q}{Q_n}\exp(-0.75\,\beta\,COV_Q) \tag{7a}$$

$$\eta = \frac{m_F}{F_n}\exp(+0.75\,\beta\,COV_F) \tag{7b}$$

in which Ψ = resistance factor and η = load factor. As to be expected, reliability-calibrated resistance and load factors are functions of the target reliability index (β) and the degree of uncertainty (COV_Q and COV_F). However, it is equally clear from Equation 7 that efforts to rationalize resistance and load factors using reliability calibration will be undermined completely by the failure to define Q_n and F_n.

Two important conclusions can be drawn from the above simple analytical demonstration:

a. It is imperative to define the nominal values in an unambiguous way *with reference to the probability distribution function*.
b. The definition of nominal values is unrelated to reliability analysis. However, practical issues, such as simplicity, familiarity, and compatibility with the existing design approach, are important considerations that will determine if the simplified RBD design approach can gain ready acceptance among practicing foundation engineers.

The former has been exhorted over many years (e.g., Been and Jefferies 1993, Dahlberg and Ronold 1993, Becker 1996a, Green & Becker 2001), but broad qualitative guidelines such as "cautious estimate of the value affecting the occurrence of the limit state" (CEN/TC250 1994) persist in the geotechnical LRFD literature. It is recognized that the current loose definition of nominal values mostly is intended to provide room for experienced engineers to exercise their skills and judgment in the selection of design parameters (Simpson & Driscoll 1998, Green & Becker 2001). In addition, the definition of nominal values appears to be resisted strongly by many geotechnical engineers in Canada and the United States (Green & Becker 2001). Simpson & Driscoll (1998) also noted in their commentary to Eurocode 7 that "their definition, in geotechnical terms, has been the most controversial topic in the whole process of drafting Eurocode 7".

The authors believe that much of the resistance and controversy arose because there is misunderstanding as to the *scope of the definition*. As far as RBD is concerned, nominal values must be definite only with reference to the probability distribution function (e.g., mean, mode, median, mean minus one standard deviation, etc.). There is absolutely no constraint on how the engineer should select representative design parameters from the usual geotechnical point of view. For example, an engineer familiar with a particular site may elect to use soil strength from a weak layer for some limit equilibrium analysis because he/she is confident that the failure surface passes mainly through this layer. If the mean value is selected as the nominal value, the engineer is required to estimate the mean strength of the layer of concern, but he/she is not required to disregard good geotechnical sense and be forced to estimate the average strength of the entire soil mass. The key point is that the engineer is not allowed to introduce additional conservatism into the design by using, for example, some lower bound value, because the uncertainty in the design parameters already is built rationally into the RBD equations. Contrary to popular belief (Boden 1981, Semple 1981, Bolton 1983, Fleming 1989), the application of experience, sound judgment, and soil mechanics still is needed for all aspects of geotechnical design. Human intuition is not suited for reasoning with uncertainties and only this aspect has been removed from the purview of the engineer. Clearly, judgment is not undermined; instead, it is focused on those aspects for which it is most suited.

This discussion on nominal values further emphasizes the importance of allowing engineers to select resistance factors based on the quality of data at hand. The EPRI study clearly demonstrates that resistance factors can be calibrated for broad categories of data quality (e.g., COV of undrained shear strength = 10-30%, 30-50%, 50-70%) without compromising on the uniformity of reliability achieved. Experienced engineers are expected to be able to choose the appropriate data quality category, even with limited statistical data supplemented with guidelines from Tables 2 and 3 and/or engineering judgment. Here is another excellent example of how engineering judgment could be used effectively with a RBD code calibrated specifically with geotechnical practice in mind.

In EPRI Report TR-105000, a decision was made to define nominal soil parameters at the mean for three reasons:

a. Geotechnical engineers prefer to assess foundation behavior using realistic parameters, so that they would have a physical feel for the problem, rather than perform a hypothetical computation using some statistically factored parameters (e.g., 5% exclusion limit).
b. Factored parameters inadvertently can produce values that are unrealistic or physically unrealizable.
c. The purpose of simplified RBD is to relieve practitioners from unfamiliar probability calculations so that attention can be focused on the geotechnical aspects of the problem. This objective could be partially undermined when statistical factoring is required.

Regardless of the choice, the location of the nominal values with respect to the probability distribution function (see Figure 4) must be specified if geotechnical LRFD were to be calibrated rationally using reliability analysis and if a prescribed target reliability were to be achieved consistently.

6.3 Partitioning and MRFD format

Aside from achieving a prescribed target reliability level, it is also desirable to keep the actual reliability close to the target for reasons of economy. In EPRI Report TR-105000, this second RBD objective is realized by partitioning the design domains and using a Multiple Resistance Factor Design (MRFD) format. The design of drilled shafts (bored piles) for uplift under undrained and drained loading will be used as an example; full details on the former are given elsewhere (Phoon et al. 2000). A different example on spread foundations is covered by another paper in these proceedings (Phoon & Kulhawy 2002).

Two simple design formats were selected for reliability calibration:

LRFD: $F_{50} \le \Psi_u Q_{un}$ (8)

MRFD: $F_{50} \le \Psi_{su} Q_{sun} + \Psi_{tu} Q_{tun} + \Psi_w W$ (9)

in which F_{50} = 50-year return period load, Q_{un} = nominal uplift capacity, Q_{sun} = nominal side resistance, Q_{tun} = nominal tip resistance, W = weight of foundation, and Ψ_u, Ψ_{su}, Ψ_{tu} and Ψ_w = resistance factors. Figures 5 and 6 show the reliability levels implicit in existing ULS designs under undrained and drained loading, respectively. Note that the existing WSD format is essentially the same as the LRFD format (Equation 8), because the reciprocal of the traditional factor of safety (FS) is equal to the resistance factor (Ψ_u). Therefore, the variation in the reliability index (β) or probability of failure (p_f) at a fixed FS is indicative that the LRFD format will produce fairly poor uniformity in reliability when it is applied over the entire design domain. For the drained case, the degree of non-uniformity in the reliability levels is very evident (Figure 6) because the in-situ coefficient of horizontal soil stress (K_o) is very influential and can be highly uncertain. For mean K_o = 2.0 and Ψ_u = 0.33 (or FS = 3), β varies from 2.81 to 3.39. It may be tempting to conclude that this variation is "small", but it must be emphasized that p_f (a more fundamental measure of risk) is very sensitive to changes in β. This sensitivity can be seen in Table 1. The variation of β noted above is equivalent to approximately one order of magnitude change in p_f (0.00035 to 0.0025).

In EPRI Report TR-105000, the uniformity in reliability was improved by using the following general calibration procedure:

a. Perform a parametric study on the variation of the reliability level with respect to each deterministic and statistical parameter in the design problem. Examples of deterministic parameters that control the design of drilled shafts include the diameter (B) and depth to diameter (D/B) ratio. Examples of statistical parameters include the mean and coefficient of variation (COV) of the undrained shear strength (s_u).
b. Partition the parameter space into several smaller domains. An example of a simple parameter space is shown in Figure 7. The reason for partitioning is to achieve greater uniformity in reliability over the full range of deterministic and statistical parameters. For those parameters identified in Step (a) as having a significant influence on the reliability level, the size of the partition clearly should be smaller. In addition, partitioning ideally should conform to existing geotechnical conventions.

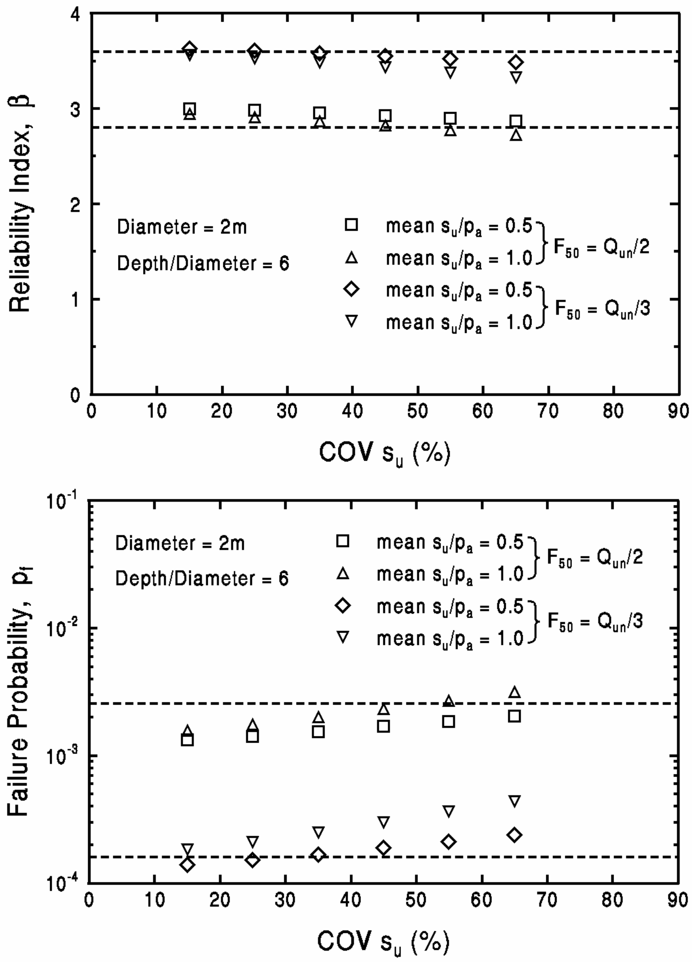


Figure 5. Reliability levels implicit in existing ultimate limit state design of drilled shafts in undrained uplift.

c. Select a set of representative points from each domain. Note that each point in the parameter space denotes a specific set of parameter values (Figure 7). Ideally, the set of representative points should capture the full range of variation in the reliability level over the whole domain.
d. Determine an acceptable foundation design for each point and evaluate the reliability levels in the designs. Foundation design is performed using the set of parameter values associated with each point, along with a simplified RBD format and a set of trial resistance factors. The reliability of the resulting foundation design then is evaluated using the FORM algorithm.
e. Quantify the deviations of the reliability levels from a pre-selected target reliability index, β_T. The following simple objective function can be used:

$$H(\Psi_{su}, \Psi_{tu}, \Psi_w) = \sum_{i=1}^{n} (\beta_i - \beta_T)^2 \quad (10)$$

in which $H(\cdot)$ = objective function to be minimized, n = number of points in the calibration domain, and β_i = reliability index for the ith point in the domain.

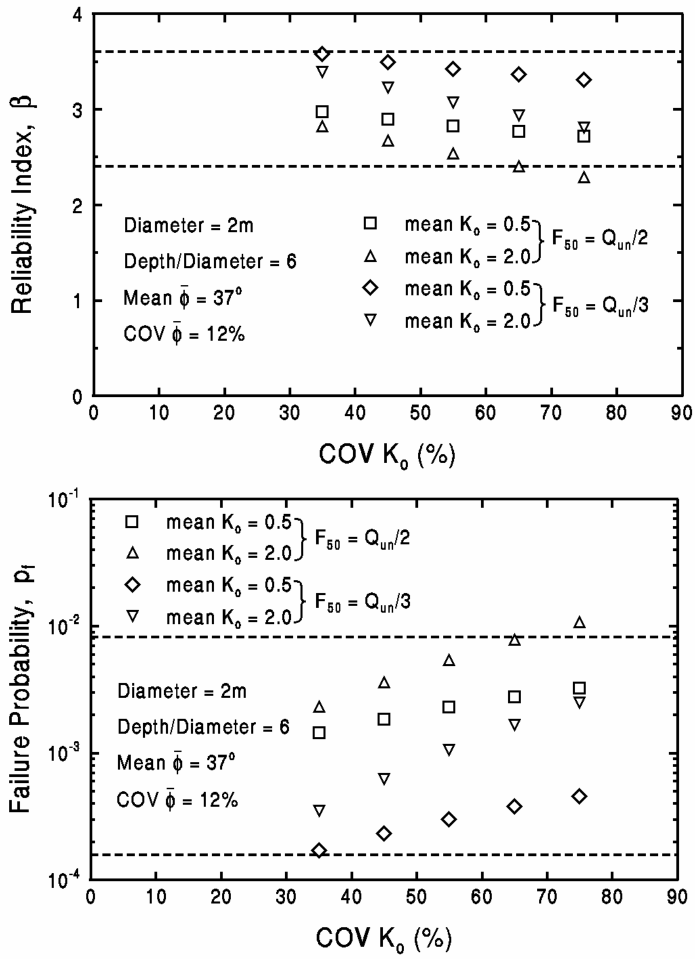


Figure 6. Reliability levels implicit in existing ultimate limit state design of drilled shafts in drained uplift.

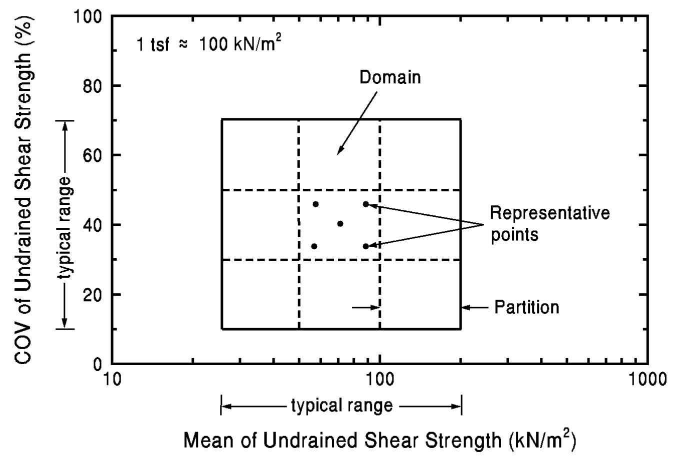


Figure 7. Partitioning of parameter space for calibration of resistance factors.

f. Adjust the resistance factors and repeat Steps (d) and (e) until the objective function is minimized. The set of resistance factors that minimizes the objective function (H) is the most desirable because the degree of uniformity in the reliability levels of all the designs in the domain is maximized. The following measure can be used to quantify the degree of uniformity that has been achieved:

$$\Delta\beta = \sqrt{\frac{H}{n}} \qquad (11)$$

g. Repeat Steps (c) to (f) for the other domains.

The results of the RBD calibration exercise for drilled shafts in undrained uplift loading are shown in Tables 6 and 7. Exact comparison with other LRFD resistance factors is difficult, but AASHTO recommends $\Psi_u = 0.55$ for uplift capacity of drilled shaft (α-method) in clay (NCHRP Report 343, Table 4.10.6-3) while OHBDC/CHBDC and NBCC recommends $\Psi_u = 0.3$ for tension capacity of deep foundations evaluated using static analysis (Green & Becker 2001). Note that the target reliability indices are 2.5 to 3.5 for AASHTO (probably closer to 2.5 as reported by Rojiani et al. 1991) and 3.5 for OHBDC/CHBDC/NBCC. The EPRI resistance factors vary from 0.34 to 0.44 depending on the quality of data, as shown in Table 6. They lie between AASHTO and OHBDC/CHBDC/NBCC factors, but are closer to the latter than the former, partially because the target reliability index is 3.2. More importantly, a single resistance factor cannot be expected to maintain uniform reliability over a wide and diverse range of design scenarios as shown in Figures 5 and 6.

The EPRI study shows that the use of a simple 3x3 partitioning on the mean and COV of undrained shear strength is sufficient to produce designs with distinctively more uniform reliability (compare Figures 5 and 8). The partitioning on the mean undrained shear strength also was selected to conform to existing geotechnical conventions, as noted previously. For the drained case, a 2x3 partitioning based on the mean and COV of K_o was used, as follows:

a. mean K_o = 0.3 - 1.0, 1.0 - 3.0
b. COV K_o = 30 - 50%, 50 - 70%, 70 - 90%

The improvement in the uniformity of the reliability index is quite significant as shown in Figure 9.

The EPRI study further recommended use of the Multiple Resistance Factor Design (MRFD) format for achieving a more consistent target reliability (Equation 9). The MRFD format is a natural generalization of the LRFD format that involves the application of one resistance factor to each component of the capacity rather than the overall capacity. MRFD is more physically meaningful for foundation design because the variability of each component can be significantly different. In addition, it achieves greater uniformity in reliability as shown in Figure 8. The superiority of this format over the LRFD format is more evident for the case of drilled shafts in drained uplift, as shown in Figure 9. This observation applies to all the loading modes and drainage conditions analyzed in the EPRI study.

Table 6. Undrained uplift resistance/deformation factors for drilled shafts designed using LRFD format (Source: Phoon et al. 1995, pp. 6-7 & 17).

Mean s_u (kN/m^2)	COV of s_u (%)	Ψ_u Resistance factor (ULS)	Ψ_u Deformation factor (SLS)
25 - 50 Medium clay	10 – 30	0.44	0.65
	30 – 50	0.43	0.63
	50 – 70	0.42	0.62
50 - 100 Stiff clay	10 – 30	0.43	0.64
	30 – 50	0.41	0.61
	50 – 70	0.39	0.58
100 - 200 Very stiff clay	10 – 30	0.40	0.61
	30 – 50	0.37	0.57
	50 – 70	0.34	0.52

Note: Target $\beta = 3.2$ for ULS and 2.6 for SLS

Table 7. Undrained uplift resistance factors (ULS) for drilled shafts designed using MRFD format (Source: Phoon et al. 1995, pp. 6-7).

Mean s_u (kN/m^2)	COV of s_u (%)	Ψ_{su}	Ψ_{tu}	Ψ_w
25 - 50 Medium clay	10 – 30	0.44	0.28	0.50
	30 – 50	0.41	0.31	0.52
	50 – 70	0.38	0.33	0.53
50 - 100 Stiff clay	10 – 30	0.40	0.35	0.56
	30 – 50	0.36	0.37	0.59
	50 – 70	0.32	0.40	0.62
100 - 200 Very stiff clay	10 – 30	0.35	0.42	0.66
	30 – 50	0.31	0.48	0.68
	50 – 70	0.26	0.51	0.72

Note: Target $\beta = 3.2$ for ULS

The use of a lumped parameter approach for the resistance side of the LRFD format arises, in part, from tradition. Historically, reliability-based design in the form of the LRFD format first was introduced by committees that were concerned primarily with structural loadings (e.g., Allen 1975, Ellingwood et al. 1980). It is only natural that the resistance side of the equation was left as a generic lumped parameter that can be tailored easily to suit the diverse strength formulae for different materials. There are no theoretical or practical objections to the use of the generalized MRFD format shown in Equation 9. However, the weight of tradition has led to the application of the LRFD format for foundation design, even though the generalized version is physically more appealing and is practically more useful for foundation design.

It also may be noted that the original rationale for applying more than one load factor is that the uncertainties involved in estimating dead and live loads are significantly different. The same situation clearly applies to foundation capacity where the uncertainties underlying different components, such as side resistance and foundation self-weight, are significantly different. Therefore, the use of the MRFD format is statistically more appealing as well.

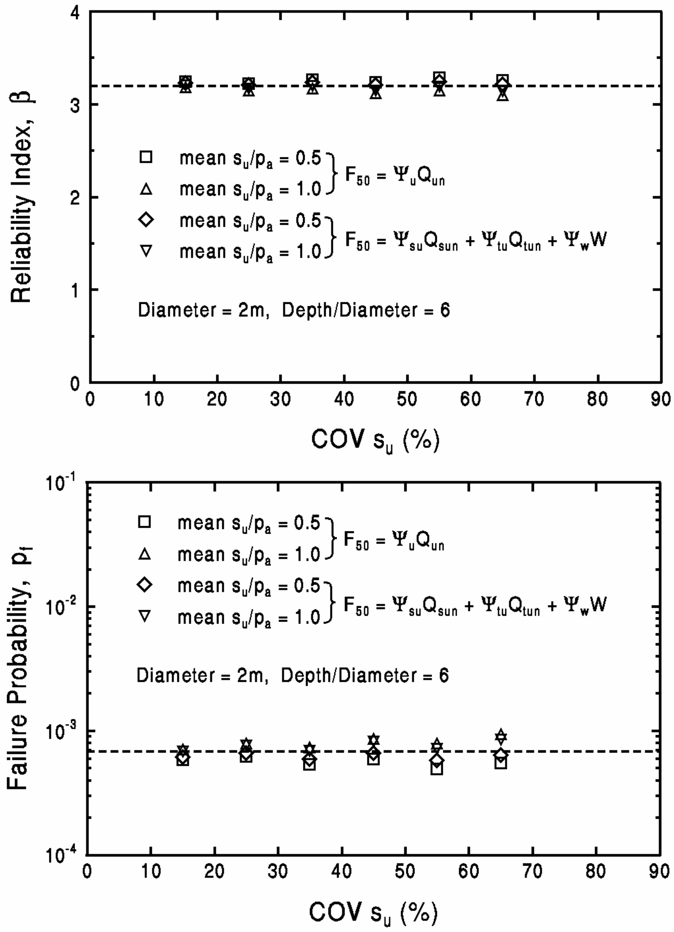
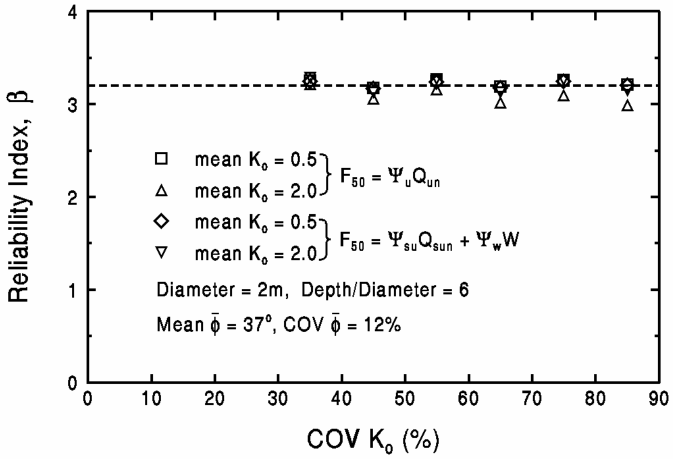


Figure 8. Performance of ultimate limit state RBD formats for drilled shafts in undrained uplift.

7 CLOSING THOUGHTS

Although existing geotechnical LRFD codes look the same as their structural counterparts, they are fundamentally incompatible with reliability-based structural codes primarily because: (a) many design aspects, such as the choice of nominal values, still are left to the discretion of the design engineer, (b) the main source of uncertainty (soil variability) is not explicitly considered, and (c) rigorous RBD calibration is absent. While the objective of maintaining continuity with past practice is an important one, it is difficult to justify moving away from a known WSD procedure to a largely untested LRFD procedure solely on this basis.

This paper demonstrates that all of the key components for implementing rigorous RBD in foundation engineering are in place. In particular, it is not true that statistics are difficult to develop because of the site-specific nature of soil variability. A simple second-moment probabilistic approach is available to combine uncertainties arising from site conditions, measurement techniques, and correlation models, in a general way. It also is not true that statistics are lacking. An extensive compilation and synthesis of available soil statistics and correlations are available for use as first-order estimates in RBD calibration and application.



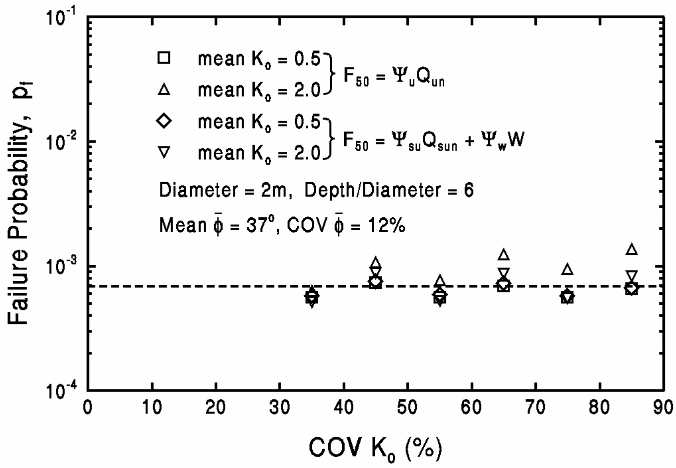


Figure 9. Performance of ultimate limit state RBD formats for drilled shafts in drained uplift.

There are various RBD issues specific to geotechnical engineering that should be considered carefully. The use of a lumped resistance model should be resisted despite its convenience, primarily because it is not robust and it is not possible to extrapolate statistics from load test scenarios to actual design scenarios where the quality of soil data is different. A better strategy is to use a first-order, a priori, predictive methodology and to calibrate the resistance factors as functions of soil data quality.

There appears to be some misunderstanding regarding the scope of the definition of nominal values. As far as RBD is concerned, nominal values must be definite only with reference to the probability distribution function. The engineer is allowed to select representative design parameters, from the usual geotechnical point of view, but is not allowed to introduce additional conservatism by using values smaller than the specified nominal value or vice-versa, because the effect of uncertainty already is accounted for rationally in RBD.

An important advantage of RBD is that the reliability index provides a consistent measure of risk that can be compared across different foundation types and loading modes. This advantage has been exploited by structural RBD to produce more economical designs by ensuring that the actual reliability achieved using simplified RBD formats is close to the target. The prevailing practice of using a single resistance factor for each loading mode cannot perform this task adequately. The use of partitioning and MRFD are encouraged to achieve this important RBD objective for foundation design.

With a properly calibrated simplified RBD format, the geotechnical engineer can focus on ground and construction evaluation in rigorous fashion, without having to agonize over use of the "right" design equation, how to select the elusive factor of safety in a rational and defensible manner, or how much conservatism should be applied to design parameters with highly variable uncertainties that are site-dependent. Although there is much research yet to be done in the RBD arena for the full range of geotechnical design conditions, we are now at the point where it really can be used in a rational and practical design mode.

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