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ENGINEERING JUDGMENT IN THE EVOLUTION FROM DETERMINISTIC TO RELIABILITY-BASED FOUNDATION DESIGN

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ABSTRACT: Engineering judgment has always played a predominant role in geotechnical design and construction. Until earlier this century, most of this judgment was based on experience and precedents. The role of judgment in geotechnical practice has undergone significant changes since World War II as a result of theoretical, experimental, and field developments in soil mechanics, and more recently, in reliability theory. A clarification of this latter change particularly is needed to avoid misunderstanding and misuse of the new reliability-based design (RBD) codes. This paper first provides a historical perspective of the traditional factor of safety approach. The fundamental importance of limit state design to RBD then is emphasized. Finally, an overview of RBD is presented, and the proper application of this new design approach is discussed, with an example given of the ultimate limit state design of drilled shafts under undrained uplift loading. Judgment issues from traditional approaches through RBD are interwoven where appropriate.

INTRODUCTION

Almost all engineers would agree that engineering judgment is indispensable to the successful practice of engineering. Since antiquity, engineering judgment has played a predominant role in geotechnical design and construction, although most of the early judgment was based on experience and precedents. A major change in engineering practice took place when scientific principles, such as stress analysis, were incorporated systematically into the design process. In geotechnical engineering in particular, significant advances were made following World War II primarily because of extensive theoretical, experimental, and field research. The advent of powerful and inexpensive computers in the last two decades has helped to provide further impetus to the expansion and adoption of theoretical analyses in geotechnical engineering practice. The role of

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engineering judgment has changed as a result of these developments, but the nature of this change often has been overlooked in the enthusiastic pursuit of more sophisticated analyses. Much has been written by notable engineers to highlight the danger of using theory indiscriminately, particularly in geotechnical engineering (e.g., Dunnicliff & Deere, 1984; Focht, 1994). For example, engineering judgment still is needed (and likely always will be!) in site characterization, selection of appropriate soil/rock parameters and methods of analysis, and critical evaluation of the results of analyses, measurements, and observations. The importance of engineering judgment clearly has not diminished with the growth of theory and computational tools. However, its role has become more focused on those design aspects that remained outside the scope of theoretical analyses.

At present, another significant change in engineering practice is taking place. Much of the impetus for this innovation arose from the widespread rethinking of structural safety concepts that was brought about by the boom in post-World War II construction (e.g., Freudenthal, 1947; Pugsley, 1955). Traditional deterministic design codes gradually are being phased out in favor of reliability-based design (RBD) codes that can provide a more consistent assurance of safety based on probabilistic analyses. Since the mid-1970s, a considerable number of these new design codes have been put into practice for routine structural design, for example, in the United Kingdom in 1972 (BSI-CP110), in Canada in 1974 (CSA-S136), in Denmark in 1978 (NKB-36), and in the U.S. in 1983 for concrete (ACI) and in 1986 for steel (AISC). In geotechnical engineering, a number of RBD codes also have been proposed recently for trial use (e.g., Barker et al. 1991; Berger & Goble 1992; Phoon et al. 1995).

The impact of these developments on the role of engineering judgment is analogous to that brought about by the introduction of scientific principles into engineering practice. In this continuing evolution, it must be realized that RBD is just another tool, but it is different from traditional deterministic design, even though the code equations from both methods have the same "look-and-feel". These differences can lead to misunderstanding and misuse of the new RBD codes. For these reasons, it is necessary to: (a) clarify how engineering judgment can be used properly so that it is compatible with RBD, and (b) identify those geotechnical safety aspects that are not amenable to probabilistic analysis. In this paper, an overview is given first of the traditional geotechnical design approach from the perspective of safety control. The philosophy of limit state design then is presented as the underlying framework for RBD. Finally, the basic principles of RBD are reviewed, and the proper application of this new design approach is discussed with an example of the ultimate limit state design of drilled shafts under undrained uplift loading. As described in the paper title, engineering judgment is interwoven throughout.

TRADITIONAL GEOTECHNICAL DESIGN PRACTICE

The presence of uncertainties and their significance in relation to design has long been appreciated (e.g., Casagrande 1965). The engineer recognizes, explicitly or otherwise, that there is always a chance of not achieving the design objective, which is to ensure that the system performs satisfactorily within a specified period of time. Traditionally, the geotechnical engineer relies primarily on factors of safety at the design stage to reduce the risk of potential adverse performance (collapse, excessive deformations, etc.). Factors of safety between 2 to 3 generally are considered to be adequate in foundation design (e.g., Focht & O'Neill 1985). However, these values can be misleading because, too often, factors of safety are recommended without reference to any other aspects of the design computational process, such as the loads and their evaluation, method of analysis (i.e., design equation), method of property evaluation (i.e., how do you select the undrained shear strength?), and so on. Other important considerations that affect the factor of safety include variations in the loads and material strengths, inaccuracies in the design equations, errors arising from poorly supervised construction, possible changes in the function of the structure from the original intent, unrecognized loads, and unforeseen in-situ conditions. The manner in which these background factors are listed should not be construed as suggesting that the engineer actually goes through the process of considering each of these factors separately and in explicit detail. The assessment of the traditional factor of safety is essentially subjective, requiring only global appreciation of the above factors against the backdrop of previous experience.

The sole reliance on engineering judgment to assess the factor of safety can lead to numerous inconsistencies. First, the traditional factor of safety suffers from a major flaw in that it is not unique. Depending on its definition, the factor of safety can vary significantly over a wide range, as shown in Table 1 for illustrative purposes. The problem examined in Table 1 is to compute the design capacity of a straight-sided drilled shaft in clay, 1.5 m in diameter and 1.5 m deep, with an average side resistance along the shaft equal to 36 kN/m^2 and a potential tip suction of 1/2 atmosphere operating during undrained transient live loading. Five possible design assumptions are included. The first applies the factor of safety (FS) uniformly to the sum of the side, tip, and weight components; the second applies the FS only to the side and tip components; the third is like the first, but disregarding the tip; the fourth is like the second, but disregarding the tip; and the fifth is ultra-conservative, considering only the weight. It is clear from Table 1 that a particular factor of safety is meaningful only with respect to a given design assumption and equation.

Another significant source of ambiguity lies in the relationship between the factor of safety and the underlying level of risk. A larger factor of safety does not necessarily imply a smaller level of risk, because its effect can be negated by the presence of larger uncertainties in the design environment. In addition, the effect of the factor of safety on the underlying risk level also is dependent on how conservative the selected design models and design parameters are.

In a broad sense, these issues generally are appreciated by most engineers. They can exert additional influences on the engineer's choice of the factor of safety but, in the absence of a theoretical framework, it is not likely that the risk of adverse performance can be reduced to a desired level consistently. Therefore, the main weakness in traditional practice, where assurance of safety is concerned, can be attributed to the lack of clarity in the relationship

TABLE 1. Design Capacity Example (Kulhawy 1984, p. 395)

Design Assumption	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 = weight of shaft = 65.3 kN, Q_u = available capacity = $Q_{su} + Q_{tu} + W$ = 511.6 kN, Q_{ud} = design uplift capacity, FS = factor of safety

between the method (factor of safety) and the objective (reduce design risk). To address this problem properly, an essential first step is to establish the design process on a more logical basis, known as limit state design.

PHILOSOPHY OF LIMIT STATE DESIGN

The original concept of limit state design refers to a design philosophy that entails the following three basic requirements: (a) identify all potential failure modes or limit states, (b) apply separate checks on each limit state, and (c) show that the occurrence of each limit state is sufficiently improbable. Conceptually, limit state design is not new. It is merely a logical formalization of the traditional design approach that would help facilitate the explicit recognition and treatment of engineering risks. In recent years, the rapid development of RBD has tended to overshadow the fundamental role of limit state design. Much attention has been focused on the consistent evaluation of safety margins using advanced probabilistic techniques (e.g., MacGregor 1989). Although the achievement of consistent safety margins is a highly desirable goal, it should not be overemphasized to the extent that the importance of the principles underlying limit state design become diminished.

This fundamental role of limit state design is particularly true for geotechnical engineering. The first step in limit state design, which involves the proper identification of potential foundation failure modes, is not always a trivial task (Mortensen 1983). This effort generally requires an appreciation of the interaction between the geologic environment, loading characteristics, and foundation response. Useful generalizations on which limit states are likely to dominate in typical foundation design situations are certainly possible, as in the case of structural design. The role of the geotechnical engineer in making adjustments to these generalizations on the basis of site-specific information is, however, indispensable as well. The need for engineering judgment in the selection of potential limit states is greater in foundation design than in structural design because in-situ conditions must be dealt with "as is" and might contain geologic "surprises". The danger of downplaying this aspect of limit state design in the fervor toward improving the computation and evaluation of safety margins in design can not be overemphasized.

The second step in limit state design is to check if any of the selected limit states has been violated. To accomplish this step, it is necessary to use a model that can predict the performance of the system from some measured parameters. In geotechnical engineering, this is not a straightforward task. Consider Figure 1, which is the essence of any type of prediction, geotechnical or otherwise. At one end of the process is the forcing function, which normally consists of loads in conventional foundation engineering. At the other end is the system response, which would be the prediction in an analysis or design situation. Between the forcing function (load) and the system response (prediction) is the model invoked to describe the system behavior, coupled with the properties needed for this particular model. Contrary to popular belief, the quality of geotechnical prediction does not necessarily increase with the level of sophistication in the model (Kulhawy 1992). A more important criterion for the quality of geotechnical prediction is whether the model and property are calibrated together for a specific load and subsequent prediction (Kulhawy 1992, 1994). Reasonable predictions often can be achieved using simple models, even though the type of behavior to be predicted is nominally beyond the capability of the models, as long as there are sufficient data to calibrate these models empirically. However, these models then would be restricted to the specific range of conditions in the calibration process. Extrapolation beyond these conditions can potentially result in erroneous predictions. Ideally, empirical calibration of this type should be applied judiciously by avoiding the use of overly simplistic models. Common examples of such an oversimplification are the sets of extensive correlations between the standard penetration test N-value and practically all types of geotechnical design parameters, as well as several design conditions such as footing settlement and bearing capacity. Although they lack generality, simple models will remain in use for quite some time because of our professional heritage that is replete with, and built upon, empirical correlations. The role of the geotechnical engineer in appreciating the complexities of soil behavior and recognizing the inherent limitations in the simplified models is clearly of considerable importance. The amount of attention paid to the evaluation of safety margins is essentially of little consequence if the engineer were to assess the soil properties incorrectly or to select an inappropriate model for design.

Third, the occurrence of each limit state must be shown to be sufficiently improbable. The philosophy of limit state design does not entail a preferred method of

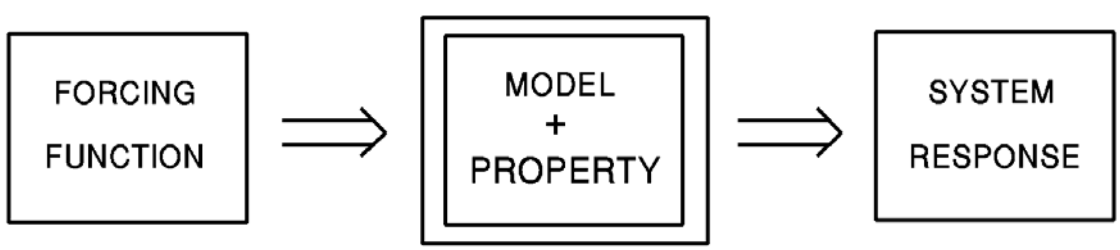


FIG. 1. Components of Geotechnical Prediction (Kulhawy 1994, p. 210)

ensuring safety. Since all engineering quantities (e.g., loads, strengths) are inherently uncertain to some extent, a logical approach is to formulate the above problem in the language of probability. The mathematical formalization of this aspect of limit state design using probabilistic methods constitutes the main thrust of RBD. Aside from probabilistic methods, less formal methods of ensuring safety, such as the partial factors of safety method (e.g., Danish Geotechnical Institute 1985; Technical Committee on Foundations 1992), have also been used within the framework of limit state design.

In summary, the control of safety in geotechnical design is distributed among more than one aspect of the design process. Although it is important to consider the effect of uncertainties in loads and strengths on the safety margins, it is nonetheless only one aspect of the problem of ensuring sufficient safety in the design. The other two aspects, identification of potential failure modes and the methodology of making geotechnical predictions, can be of paramount importance, although they may be less amenable to theoretical analyses.

RELIABILITY-BASED DESIGN

Overview of Reliability Theory

The principal difference between RBD and the traditional design approach lies in the application of reliability theory, which allows uncertainties to be quantified and manipulated consistently in a manner that is free from self-contradiction. A simple application of reliability theory is shown in Figure 2. Uncertain design quantities, such as the load (F) and the capacity (Q), are modeled as random variables, while design risk is quantified by the probability of failure (p_f). The basic reliability problem is to evaluate p_f from some pertinent statistics of F and Q, which typically include the mean (m_F or m_Q) and the standard deviation (s_F or s_Q). Note that the standard deviation provides a quantitative measure of the magnitude of uncertainty about the mean value.

A simple closed-form solution for p_f is available if Q and F are both normally distributed. For this condition, the safety margin (Q - F = M) also is normally distributed with the following mean (m_M) and standard deviation (s_M) (e.g., Melchers 1987):

$$m_M = m_Q - m_F \tag{1a}$$

$$s_M^2 = s_Q^2 + s_F^2 \tag{1b}$$

Once the probability distribution of M is known, the probability of failure (p_f) can be evaluated as (e.g., Melchers 1987):

$$p_f = \text{Prob}(Q < F) = \text{Prob}(Q - F < 0) = \text{Prob}(M < 0) = \Phi(- m_M/s_M) \tag{2}$$

in which Prob(·) = probability of an event and Φ(·) = standard normal cumulative function. Numerical values for Φ(·) are tabulated in many standard texts on reliability

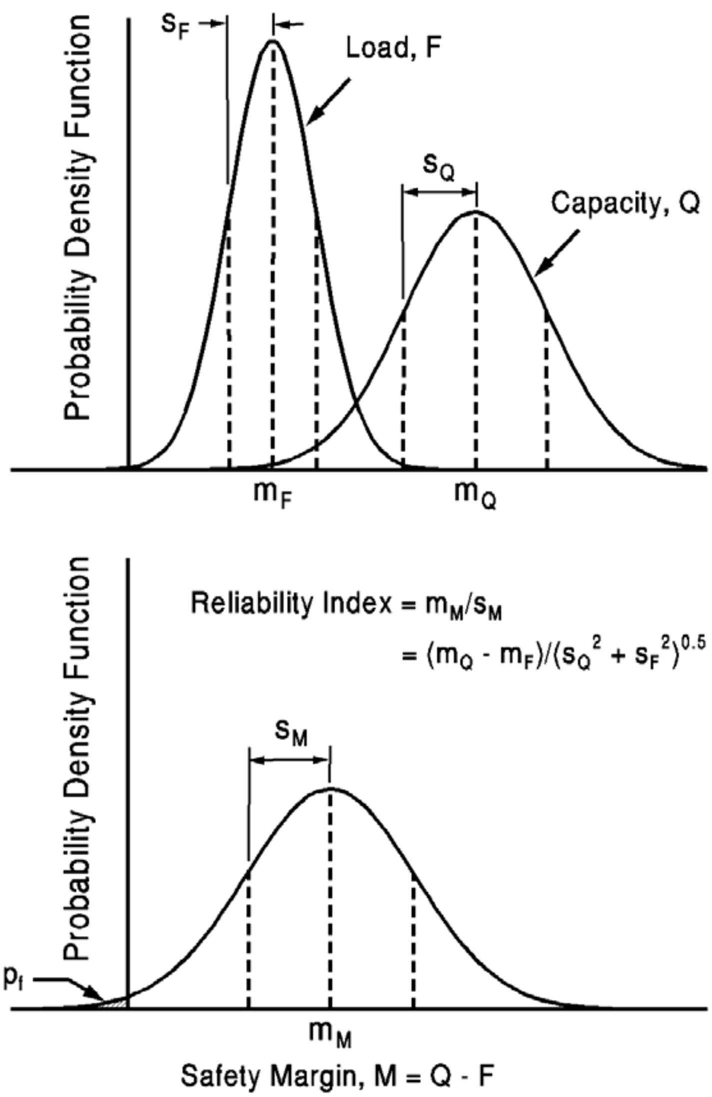


FIG. 2. Reliability Assessment for Two Normal Random Variables, Q and F

theory (e.g., Melchers, 1987). The probability of failure is cumbersome to use when its value becomes very small, and it carries the negative connotation of "failure". A more convenient (and perhaps more palatable) measure of design risk is the reliability index (β), which is defined as:

$$\beta = -\Phi^{-1}(p_f) \tag{3}$$

in which $\Phi^{-1}(\cdot)$ = inverse standard normal cumulative function. Note that β is not a new measure of design risk. It simply represents an alternative method for presenting p_f on a more convenient scale. A comparison of Equations 2 and 3 shows that the reliability index for the special case of two normal random variables is given by:

$$\beta = m_M / s_M = (m_Q - m_F) / (s_Q^2 + s_F^2)^{0.5} \quad (4)$$

The reliability indices for most structural and geotechnical components and systems lie between 1 and 4, corresponding to probabilities of failure ranging from about 16 to 0.003%, as shown in Table 2. Note that p_f decreases as β increases, but the variation is not linear. A proper understanding of these two terms and their interrelationship is essential, because they play a fundamental role in RBD.

Simplified RBD for Foundations

Once a reliability assessment technique is available, the process of RBD would involve evaluating the probabilities of failure of trial designs until an acceptable target value is achieved. While the approach is rigorous, it is not suitable for designs that are conducted on a routine basis. One of the main reasons for this limitation is that the reliability assessment of realistic geotechnical systems is more involved than that shown in Figure 2. The simple closed-form solution given by Equation 2 only is applicable to cases wherein the safety margin can be expressed as a linear sum of normal random variables. However, the capacity of most geotechnical systems is more suitably expressed as a nonlinear function of random design soil parameters (e.g., effective stress friction angle, in-situ horizontal stress coefficient, etc.) that generally are non-normal in nature. To evaluate p_f for this general case, fairly elaborate numerical procedures, such as the First-Order Reliability Method (FORM), are needed. A description of FORM for geotechnical engineering is given elsewhere (e.g., Phoon et al. 1995) and is beyond the scope of this paper. At the present time, it is safe to say that most geotechnical engineers would feel uncomfortable performing such elaborate calculations because of their lack of proficiency in probability theory (Whitman 1984).

All the existing implementations of RBD are based on a simplified approach that involves the use of multiple-factor formats for checking designs. The three main types of

TABLE 2. Relationship Between Reliability Index (β) and Probability of Failure (p_f)

Reliability Index, β	Probability of Failure, $p_f = \Phi(-\beta)$
1.0	0.159
1.5	0.0668
2.0	0.0228
2.5	0.00621
3.0	0.00135
3.5	0.000233
4.0	0.0000316

Note: $\Phi(\cdot)$ = standard normal probability distribution multiple-factor formats are: (a) partial factors of safety, (b) load and resistance factor design (LRFD), and (c) multiple resistance factor design (MRFD). Examples of these design formats are given below for uplift loading of a drilled shaft:

$$\eta F_n = Q_u(c_n/\gamma_c, \phi_n/\gamma_\phi) \quad (5a)$$

$$\eta F_n = \Psi_u Q_{un} \quad (5b)$$

$$\eta F_n = \Psi_{su} Q_{sun} + \Psi_{tu} Q_{tun} + \Psi_w W \quad (5c)$$

in which η = load factor, F_n = nominal design load, Q_u = uplift capacity, c_n = nominal cohesion, ϕ_n = nominal friction angle, γ_c and γ_ϕ = partial factors of safety, Q_{un} = nominal uplift capacity, Q_{sun} = nominal uplift side resistance, Q_{tun} = nominal uplift tip resistance, W = shaft weight, and Ψ_u, Ψ_{su}, Ψ_{tu}, and Ψ_w = resistance factors. The multiple factors in the simplified RBD equations are calibrated rigorously using reliability theory to produce designs that achieve a known level of reliability consistently. Details of the geotechnical calibration process are given elsewhere (e.g., Phoon et al. 1995).

In principle, any of the above formats or some combinations thereof can be used for calibration. The selection of an appropriate format is unrelated to reliability analysis. Practical issues, such as simplicity and compatibility with the existing design approach, are important considerations that will determine if the simplified RBD approach can gain ready acceptance among practicing engineers. At present, the partial factors of safety format (Equation 5a) has not been used for RBD because of three main shortcomings. First, a unique partial factor of safety can not be assigned to each soil property, because the effect of its uncertainty on the foundation capacity depends on the specific mathematical function in which it is embedded. Second, indiscriminate use of the partial factors of safety can produce factored soil property values that are unrealistic or physically unrealizable. Third, many 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 factored parameters (Duncan et al. 1989; Green 1993; Been et al. 1993).

This preference clearly is reflected in the traditional design approach, wherein the modification for uncertainty often is applied to the overall capacity using a global factor of safety (FS) as follows:

$$F_n = Q_{un}/FS \quad (6)$$

A comparison between Equation 6 and Equations 5b and 5c clearly shows that the LRFD and MRFD formats are compatible with the preferred method of applying safety factors. In fact, the load and resistance factors in the LRFD format can be related easily to the familiar global factor of safety as follows:

$$FS = \eta/\Psi_u \quad (7)$$

The corresponding relationship for the MRFD format is:

$$FS = \eta/(\Psi_{su}Q_{sun}/Q_{un} + \Psi_{tu}Q_{tun}/Q_{un} + \Psi_w W/Q_{un}) \quad (8)$$

Although Equation 8 is slightly more complicated, it still is readily amenable to simple calculations. These relationships are very important, because they provide the design engineer with a simple direct means of checking the new design formats against their traditional design experience.

RBD EXAMPLE

The development of a rigorous and robust RBD approach for geotechnical design, which also is simple to use, is no trivial task. Since the early 1980s, an extensive research study of this type has been in progress at Cornell University under the sponsorship of the Electric Power Research Institute and has focused on the needs of the electric utility industry. Extensive background information on site characterization, property evaluation, in-situ test correlations, etc. had to be developed as a prelude to the RBD methodology. This work is summarized elsewhere (Spry et al. 1988, Orchant et al. 1988, Filippas et al. 1988, Kulhawy et al. 1992). Building on these and other studies, ultimate and serviceability limit state RBD equations were developed for drilled shafts and spread foundations subjected to a variety of loading modes under both drained and undrained conditions (Phoon et al. 1995). The results of an extensive reliability calibration study for ultimate limit state design of drilled shafts under undrained uplift loading are presented in Tables 3 and 4 and are to be used with Equations 5b (LRFD) and 5c (MRFD). All other limit states, foundation types, loading modes, and drainage conditions addressed have similar types of results, with simple LRFD and MRFD

TABLE 3. Undrained Ultimate Uplift Resistance Factors For Drilled Shafts Designed Using $F_{50} = \Psi_u Q_{un}$ (Phoon et al. 1995, p. 6-7)

Clay	COV of s_u, (%)	Ψ_u
Medium	10 - 30	0.44
mean s_u = 25 to 50 kN/m^2	30 - 50	0.43
	50 - 70	0.42
Stiff	10 - 30	0.43
mean s_u = 50 to 100 kN/m^2	30 - 50	0.41
	50 - 70	0.39
Very Stiff	10 - 30	0.40
mean s_u = 100 to 200 kN/m^2	30 - 50	0.37
	50 - 70	0.34

Note: Target reliability index = 3.2

TABLE 4. Undrained Uplift Resistance Factors For Drilled Shafts Designed Using $F_{50} = \Psi_{su}Q_{sun} + \Psi_{tu}Q_{tun} + \Psi_w W$ (Phoon et al. 1995, p. 6-7)

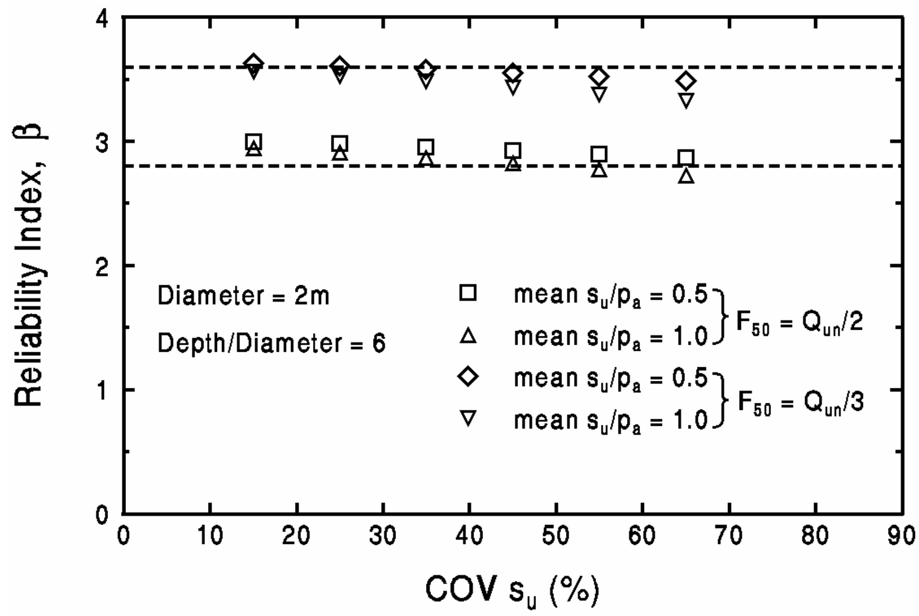
Clay	COV of s_u, (%)	Ψ_{su}	Ψ_{tu}	Ψ_w
Medium	10 - 30	0.44	0.28	0.50
mean s_u = 25 to 50 kN/m^2	30 - 50	0.41	0.31	0.52
	50 - 70	0.38	0.33	0.53
Stiff	10 - 30	0.40	0.35	0.56
mean s_u = 50 to 100 kN/m^2	30 - 50	0.36	0.37	0.59
	50 - 70	0.32	0.40	0.62
Very Stiff	10 - 30	0.35	0.42	0.66
mean s_u = 100 to 200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

Note: Target reliability index = 3.2

equations and corresponding tables of resistance factors. In these equations, the load factor is taken as unity, while the nominal load is defined as the 50-year return period load (F_{50}), which is typical for electrical transmission line structures. Note that the resistance factors depend on the clay consistency and the coefficient of variation (COV) of the undrained shear strength (s_u). The COV is an alternative measure of uncertainty that is defined as the ratio of the standard deviation to the mean. The clay consistency is classified broadly as medium, stiff, and very stiff, with corresponding mean s_u values of 25 to 50 kN/m^2, 50 to 100 kN/m^2, and 100 to 200 kN/m^2, respectively. Foundations are designed using these new RBD formats in the same way as in the traditional approach, with the exception that the rigorously-determined resistance factors shown in Tables 3 and 4 are used in place of an empirically-determined factor of safety.

Target Reliability Index

Before applying these resistance factors blindly in design, it is important to examine the target reliability index for which these resistance factors are calibrated. At the present time, there are no simple or straightforward procedures available to produce the "correct" or "true" target reliability index. However, important data that can be used to guide the selection of the target reliability index are the reliability indices implicit in existing designs (Ellingwood et al. 1980). An example of such data for ultimate limit state design of drilled shafts in undrained uplift is shown in Figure 3, in which a typical range of COV s_u, mean s_u normalized by atmospheric pressure (p_a), and global factor of safety are examined for a specific geometry. It can be seen that the reliability indices implicit in existing global factor of safety designs lie in the approximate range of 2.6 to 3.7. A target reliability index of 3.2 is representative of this range. Similar ultimate limit state



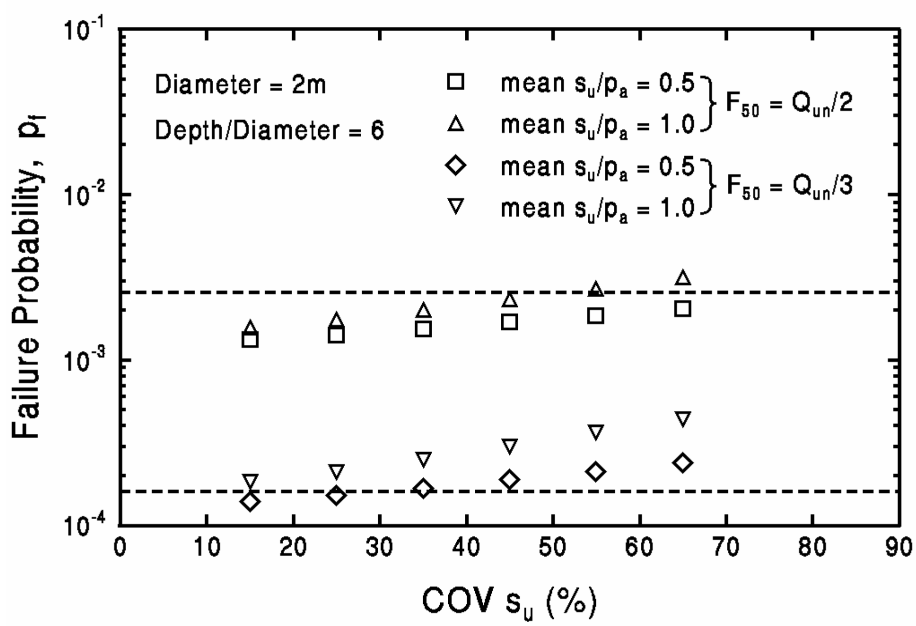


FIG. 3. Reliability Levels Implicit in Existing Ultimate Limit State Design of Drilled Shafts in Undrained Uplift

reliability studies for other parametric variations and loading modes under both drained and undrained conditions also strongly support the use of this target value (Phoon et al. 1995). While this approach is partially empirical, it does possess a major advantage of keeping the new design methodology compatible with the existing experience base.

Other important data to consider include the failure rates estimated from actual case histories. However, these failure rates can not be used directly for assessing the target reliability level, because the theoretical probability of failure obtained from reliability theory usually is one order of magnitude smaller than the actual failure rate (CIRIA 1977). This result is not surprising, because the safety of a design is not affected by uncertainties underlying design calculations alone. It also can be compromised severely by factors such as poor construction and human errors. An example of empirical rates of failure for civil engineering facilities and the related costs of failure is given in Figure 4. For foundations, the empirical rate of failure lies between 0.1 and 1%. This failure rate implies a theoretical probability of failure in the neighborhood of 0.01 to 0.1%. In terms of the reliability index, the currently accepted risk level, therefore, is between 3.1 and 3.7. A target reliability index of 3.2 also is consistent with this range.

The above discussion highlights some of the important considerations that are involved in the determination of the target reliability index. It is apparent that the target reliability index can not be adjusted casually without extensive prior calibration with existing practice. Different target reliability indices can be used for design, but specific guidelines always should be given on the conditions for which each target value applies. An example of a specific area in which a different target reliability index should be used is for serviceability limit state design (Phoon et al. 1995).

In the absence of specific guidelines, it might be possible for engineers to adjust the target reliability index to reflect some design conditions that already have been accounted for

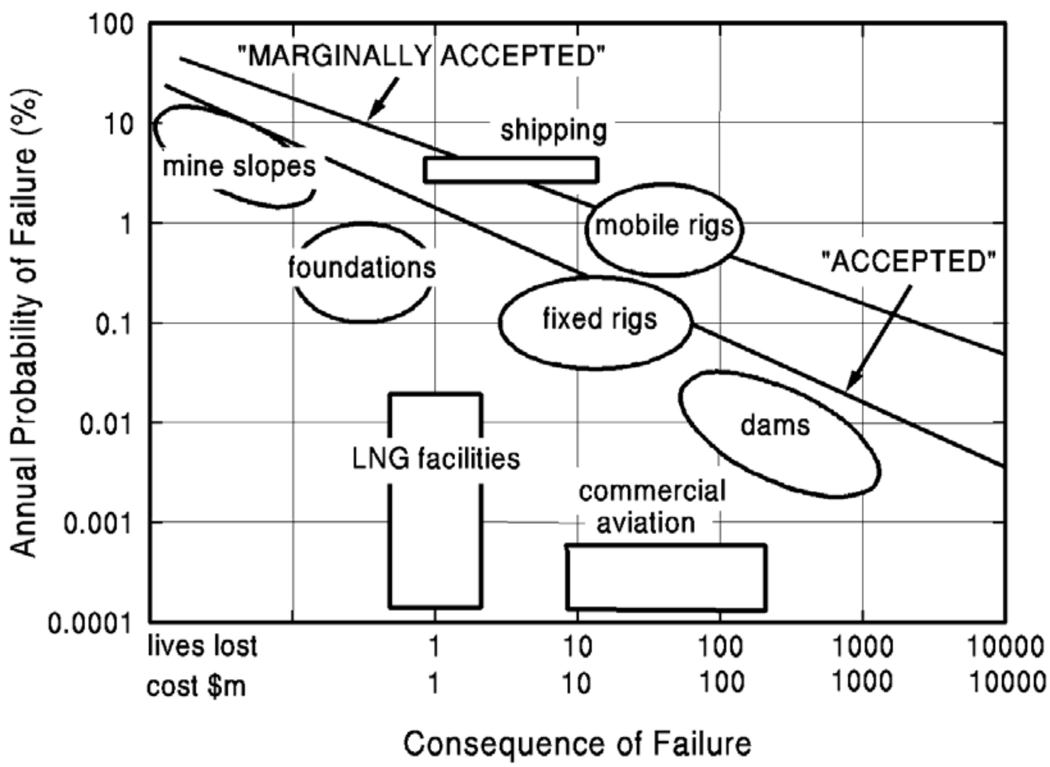


FIG. 4. Empirical Rates of Failure for Civil Engineering Facilities (Baecher 1987, p. 49)

in the calibration of the load and resistance factors. For example, an engineer might be tempted to use a different target reliability index for drained and undrained analysis, because the uncertainty in the in-situ horizontal stress coefficient might be judged to be higher than that in the undrained shear strength. Such intuitive adjustment of the safety level based on judgment alone is the norm in the traditional factor of safety design approach and for some overly-simplified RBD approaches that have been suggested. However, in a rigorous RBD approach, the difference between drained and undrained analysis already has been accounted for rationally in the resistance factors, and further adjustment of the target level would amount to "double-counting". Errors of this type are to be expected in the absence of proper guidance, because the typical RBD code user is not familiar with the details underlying the reliability calibration process. A proper appreciation of the target reliability index selection process particularly is important, because the target reliability index often has been mistaken (incorrectly) as the RBD equivalent of the empirical factor of safety.

Definition of Nominal Component

Aside from careful selection of the target reliability index, it also is important to define and understand precisely the nominal components shown in Equation 5 . The level of safety in a design clearly is governed by the product of the load and resistance factors and their respective nominal components. Two foundations can have widely different safety levels, even though the same set of resistance factors is applied, because one design might be based on average soil parameters while another could have accrued additional safety by using highly conservative soil parameters. This important aspect is not sufficiently well-emphasized in the RBD literature (CIRIA 1977; Been & Jefferies 1993).

The definitions of nominal soil strengths in the simplified RBD formats ideally should be consistent with those that are used in traditional foundation design practice. However, the existing procedures for selecting nominal soil strengths are not well-defined and certainly are not followed uniformly by all engineers. Some engineers use the mean value, while others use the most conservative of the measured strengths (Whitman 1984). Many other guidelines and rules-of-thumb exist. For example, Terzaghi and Peck (1948) recommended that the measured strength within a significant depth from each boring should be averaged, and then the smallest average should be used for design.

An alternative definition for the nominal value is based on the exclusion limit concept. The definition of a 5% exclusion limit is given in Figure 5. However, the use of a small exclusion limit probably is not appropriate for foundation design because of several reasons. First, the amount of data required for the reliable determination of a 5 to 10% exclusion limit typically is much larger than the number of measurements taken for a routine project (Been & Jefferies 1993). Second, the exclusion limit requires probability computations that are not currently performed in most foundation design. The main purpose of using a simplified RBD approach is to relieve practicing engineers from unfamiliar probability calculations so that they can focus on the geotechnical aspects of the problem. Use of the exclusion limit introduces unnecessary complications and partially undermines the objective of a simplified RBD approach. And third, the exclusion limit concept is less intuitive than that for the mean value.

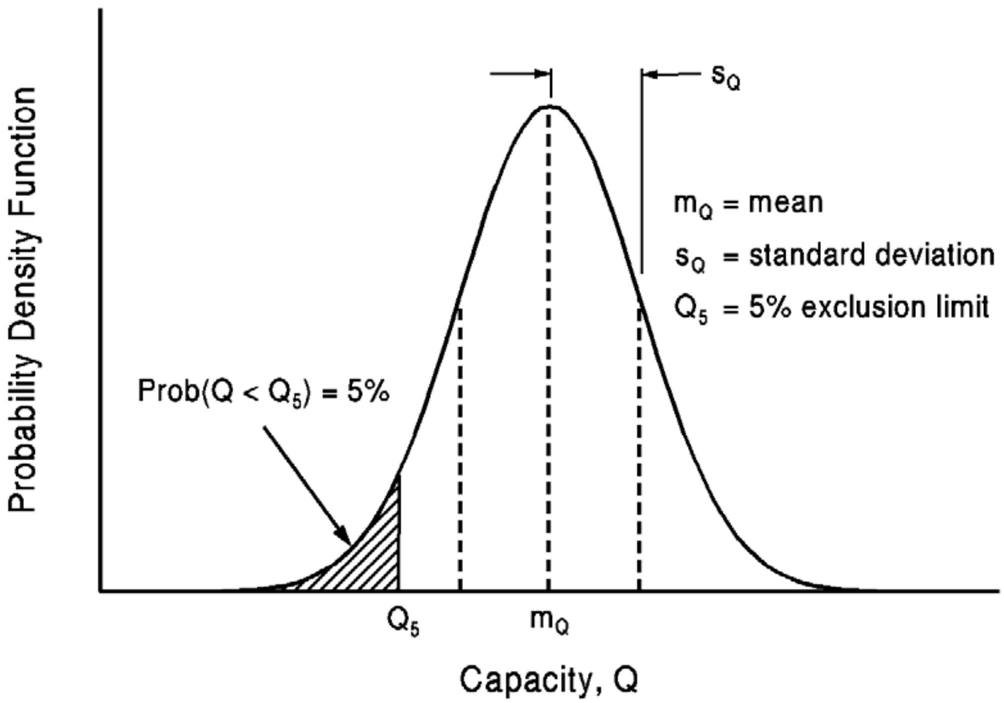


FIG. 5. Definition of 5 Percent Exclusion Limit

It is safe to say that most foundation engineers would feel more comfortable using the mean value, because they have a physical feel for that concept from their past experiences of working with realistic soil strength parameters. Regardless of the choice, it is important to emphasize that the definition of nominal values can not be left to the judgment of each individual engineer as is the case in traditional practice, if a uniform reliability level were to be maintained. It is our opinion that all nominal soil parameters should be defined at the mean for reasons of simplicity and compatibility with existing foundation design practice.

The other important nominal component is the load, which geotechnical engineers normally do not investigate in detail. Loading agendas can be rather complicated, so it is necessary to at least appreciate what these values mean. In many codes these days, loads are specified using the concept of a return period. For example, the ASCE loading guide for electrical transmission line structures (Task Committee on Structural Loading 1991) establishes the design loads for wind and other weather-related events at a return period of 50 years. The annual probability of exceeding the 50-year return period load is 1/50 or 2%. Other criteria are used by other organizations and for different types of structures and loads. The resistance factors used in RBD generally are related to the loading model and the definition of the nominal load as well.

Calibrations

Equations 5b and 5c and the corresponding Tables 3 and 4 look simple and just as easy to use as traditional design practice. That is the intent of any new or alternative design approach. However, there the similarities end. With RBD, rigorous calibrations are done of the design equations and all the input terms to achieve a target reliability index. Specified within this approach are the nominal load and resistances, target reliability index, design equation, and resistance factors, all calibrated together rigorously over a range of parameters using the First-Order Reliability Method (FORM). For the cases shown in Tables 3 and 4, the calibration parameter ranges were: wind speed = 30 to 50 m/s, shaft diameter = 1 to 3 m, shaft depth/diameter = 3 to 10, s_u = 25 to 200 kN/m^2, and COV of s_u = 10 to 70%. For each combination of parameters, unique resistance factors apply. However, it is impractical to list all of these factors in literally dozens of tables. Instead the results were scrutinized carefully, and it was found that the resistance factors could be "averaged" quite effectively over three ranges of s_u and three ranges of COV of s_u, as given in Tables 3 and 4.

The reliability indices for foundations obtained in this manner can not achieve the target reliability index exactly, because the same resistance factor is applied to a range of undrained shear strengths. However, with the three groupings selected, it was possible to reduce the average deviation from the target reliability index to a minimum, as shown in Table 5 and Figure 6. A comparison of the average deviations produced by the LRFD and MRFD formats also indicates that the MRFD format provides better reliability control. This observation basically is applicable to the other loading cases as well.

With these calibrations apparently so all-encompassing, one can ask the question whether engineering judgment is being usurped. The answer is an unequivocal "no". Instead, RBD causes us to focus our efforts and judgment on the important issues. First,

TABLE 5. Average Deviation From Target Ultimate Resistance Reliability Index For Drilled Shafts In Undrained Uplift (Phoon et al. 1995, p. 6-8)

Clay	COV of s_u, (%)	Average Reliability Deviation	
		Case 1[a]	Case 2[b]
Medium mean s_u = 25 to 50 kN/m^2	10 - 30	0.031	0.023
	30 - 50	0.042	0.034
	50 - 70	0.053	0.042
Stiff mean s_u = 50 to 100 kN/m^2	10 - 30	0.047	0.037
	30 - 50	0.068	0.054
	50 - 70	0.087	0.063
Very Stiff mean s_u = 100 to 200 kN/m^2	10 - 30	0.072	0.051
	30 - 50	0.102	0.074
	50 - 70	0.125	0.082

Note: Target reliability index = 3.2

a - designed using $F_{50} = \Psi_u Q_{un}$

b - designed using $F_{50} = \Psi_{su} Q_{sun} + \Psi_{tu} Q_{tun} + \Psi_w W$

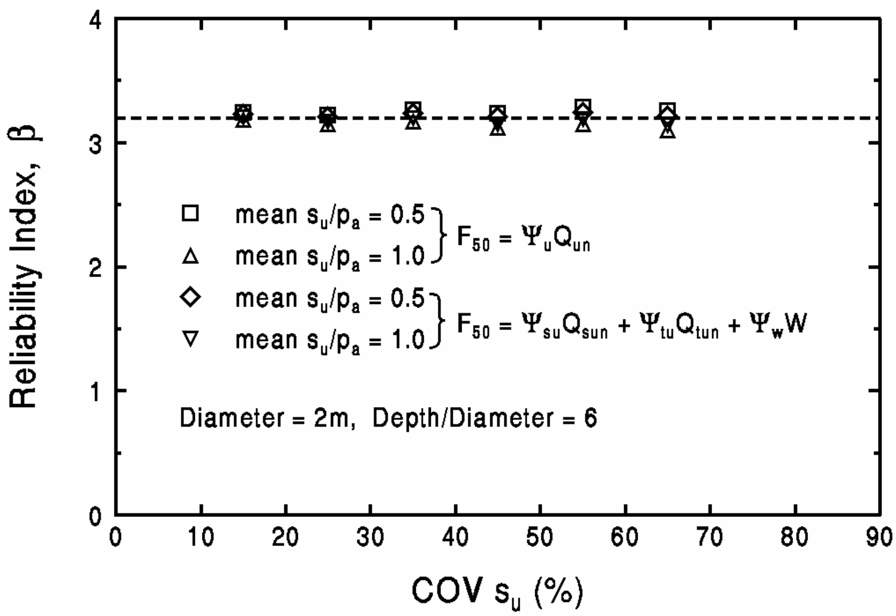


FIG. 6. Performance of Ultimate Limit State RBD Format for Drilled Shafts in Undrained Uplift

it forces us to agree on the load model, target reliability index, and design equation to use (at least for the time being). Second, it focuses our energies on evaluating the mean material properties and the variability (COV) in these properties for a given design situation. Guidance on these issues is beyond the scope of this paper, but detailed discussions are given elsewhere (Kulhawy 1992, Phoon et al. 1995, Phoon & Kulhawy 1996, Kulhawy & Trautmann 1996). It is sufficient to say that evaluation of the mean and COV for a particular boundary condition (shear, plane strain, extension, etc.) requires a careful assessment of all site, geologic, exploration, and testing variables. And third, given that the design engineer knows explicitly what is included in RBD, the design engineer then can enhance or modify the calculation results to include the intangible and/or unforeseen issues noted previously.

SUMMARY

Judgment has, and probably always will, play a critical role in geotechnical design, especially during the evolution from traditional deterministic design to new concepts of reliability-based design (RBD). In traditional geotechnical foundation design, the risk of adverse performance has been controlled by an empirical factor of safety at the design stage. However, this traditional design approach does not ensure a consistent level of safety, because the factor of safety is not well-defined, and its relationship to its underlying uncertainties is ambiguous. To address this problem in a more realistic fashion, an essential first step is to adopt limit state design. The relationship between limit state design and RBD is an intimate one. On one hand, the philosophy of limit states represents a logical and systematic approach to the process of engineering design. On the other hand, the formalization of one aspect of this whole process, which is the application of reliability theory to ensure that the occurrence of limit states is sufficiently improbable, constitutes the main thrust of RBD. From this perspective, limit state design represents a more fundamental approach. Undue emphasis on RBD at the expense of the other design aspects clearly must be avoided.

An overview of reliability theory and a simplified RBD approach is presented. The load and resistance factor design (LRFD) and multiple resistance factor design (MRFD) formats are shown to be suitable for reliability calibration, because they provide the design engineer with a simple direct means of checking the new design formats against their traditional design experience. Generally, the MRFD format is to be preferred. The proper use of these simplified RBD formats is discussed with reference to the ultimate limit state design of drilled shafts under undrained uplift loading. The two important aspects of this new design approach that can not be left entirely to the routine judgment of the design engineer are the: (a) selection of the target reliability index and (b) definition of the nominal quantities in the design equations.

The applications of these new concepts are explored in some detail, and it is stressed that judgment still has a very important role in the design process. However, the judgment issues shift largely from assessing empirical factors to defining material characteristics and uncertainties explicitly and to judging intangibles and unknowns implicitly. This process puts design within a more rigorous and consistent framework.

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