From Casagrande's "Calculated Risk" to Reliability-Based Design in Foundation Engineering

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FRED H. KULHAWY

In October 1964, Arthur Casagrande presented the second Terzaghi Lecture, which was entitled "Role of the 'Calculated Risk' in Earthwork and Foundation Engineering."[1] This seminal lecture and the resulting paper candidly stated and illustrated that geotechnical uncertainty is quite prominent and that the geotechnical design process is fraught with pitfalls. This approach dramatically differed from what was (and still is) presented in most texts, which typically address uncertainty in design in a rather straightforward manner as if it were a deterministic quantity. I was duly impressed by this lecture, so when I was invited to present the Sixth Casagrande Lecture my topic was chosen easily. I would build on Casagrande's basic ideas of risk and examine how they have led to some modern concepts of reliability-based design (RBD) in foundation engineering. My particular focus is to provide a window into some of the thought processes that are necessary to appreciate and eventually apply RBD within foundation engineering and subsequently in other areas of geotechnical engineering.

Casagrande's "Calculated Risk" & Observations on 1960s Practice

In his paper, Casagrande classified design risks into two broad categories: human and engineering. Under human risks, he noted that most fall into the categories of:

- Unsatisfactory organization and division of responsibility;

- Unsatisfactory use of available knowledge and judgment; and,
- Corruption.

The engineering risks he classified as:

- Unknown; and,
- Calculated.

Casagrande then proceeded to note that, of these groupings, all but the calculated risk are purely qualitative and enter into the design process in a wholly subjective manner.

With regard to calculated risk, he spelled out the two following conditions that delineated his thoughts on what this kind of risk was about:[1]

- "The use of imperfect knowledge, guided by judgment and experience, to estimate the probable ranges for all pertinent quantities that enter into the solution of a problem."
- "The decision on an appropriate margin of safety, or degree of risk, taking into consideration economic factors and the magnitude of losses that would result from failure."

These heady statements capture much of the essence of geotechnical practice and also delineate the limitations of practice. To emphasize the importance of parts of these statements, there were many discussions to his paper, two of which are particularly pertinent. Stratton made the following very important observation:[2]

> "It is not casual experience and untested judgment that must be brought to bear, but the organized rather than the random use of available knowledge in the first instance and this in turn bulwarked by seasoned experience and proven judgment."

Perhaps even more important to our current premise is the following observation by Golder:[3]

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

These comments by exceptionally prominent practitioners point out that there was broad appreciation of the importance of uncertainties in geotechnical design and that design judgments perhaps had a more subjective than objective flair.

In contrast to the specific appreciation of uncertainties expressed above, "standard" texts of the time such as Peck *et al.*, Chellis and Teng adopted a more traditional stance and just explicitly stated values to use in design.[4-6] Typical values of the time were as follows:

- For bearing capacity, use a minimum factor of safety (FS) of 3 for dead load plus normal live load, or use 2 for dead load plus the maximum or extreme live load.
- For side resistance in clay, use 50 to 75 kN/m^2 (0.5 to 0.75 tsf in units of the time) and then apply a conservative FS.
- For side resistance in sand, use the normally consolidated coefficient of horizontal soil stress (K_{onc}), or 0.45, in the evaluation before applying the FS.

The recommendations for soil property evaluation also were simple and typically were as follows:

- Use the "cohesion" (undrained shear strength in modern parlance) equal to half the unconfined compressive strength (c equal to q_u divided by 2) and evaluate the effective stress friction angle (ϕ) from correlations with the Standard Penetration Test N-value or otherwise.

The rationale for selecting these values and parameters was not discussed in these or other period texts, and it has never been clear to me precisely how these values evolved and how conservative they really were intended to be.

Some Later Observations on Risk

Since the 1960s, there has been great interest and activity in probability theory and risk analysis. Much has been written on the subject, but practical applications in direct design practice have been few and far between. In the 17th Terzaghi Lecture, Whitman tackled the subject of "Evaluating Calculated Risk in Geotechnical Engineering."[7] While laying out a sound framework for risk analysis, he noted that "probability theory today is regarded with

doubt and even suspicion by the majority of geotechnical engineers."[7] This statement is still valid today, but at least more engineers are sympathetic to probabilistic concepts. Perhaps more disturbing was the observation he made that "the prospect of assigning a numerical value of risk presents a rather frightening dilemma to engineers and their clients."[7] This so-called dilemma actually represents a very fundamental problem in perception. Assigning a FS is, at least nominally, establishing an acceptable level of risk. However, it must be noted that a FS of 2 or 2.5 sounds better to the average person than a probability of failure of 1 or 2 percent. The former suggests a healthy cushion in the design, while the latter states directly that it could fail.

Whitman then proceeded to make the following observations about risk analysis in geotechnical engineering:[7]

> "(a) Not enough is known about soil or rock and its behavior in situ to do an evaluation of risk; (b) we will be criticized no matter how we do the analysis; but (c) we must proceed to make such studies. Learning how to do so in a meaningful and responsible manner is still a major challenge to the profession."

I believe that knowledge about ground characteristics and their uncertainty has improved sufficiently to make reasonable evaluations of risk in some classes of geotechnical problems. There will always be criticism for any new or "different" method of evaluation. Perhaps that is just human nature. However, such studies most certainly must continue. If they are not pursued, geotechnical knowledge and the profession will never advance.

Observations on Current Practice

Within the professional practice of geotechnical engineering, I detect there is a general complacency and a feeling that the state of knowledge is perfectly adequate to do optimal design. The National Research Council's Committee on Reliability Methods for Risk Mitigation in Geotechnical Engineering states this opinion well:[8]

> "However, in conventional geotechnical practice, such as foundation engineering and embankment dam design, probabilistic methods have not seen extensive use. In these older, more conventional areas of geotechnical engineering, well-developed, effective, and successful methods are available that embody the lessons learned from decades of professional practice. Many geotechnical engineers practicing in these traditional areas apparently have seen little need to change from using methods that have served them well to new and largely untried methods with questionable potential benefit."

I would not paint such a rosy picture of "conventional" geotechnical practice. Just because there are few foundation failures does not mean that design practice is optimal or even satisfactory. It could also mean that there is too much conservatism built into the design process or perhaps that the foundations have never had to support their design loads. Certainly other possible reasons could be given as well.

To illustrate the wide variability of practice, consider the simple design example that is outlined in Table 1, which presents an example of a straight-sided drilled shaft in saturated clay, 1.5 meters in diameter and 1.5 meters 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.[9] (The original data and calculations were in customary U.S. units, so some roundoff will be encountered if calculations are redone.) 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 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 1. Column four 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."

TABLE 1.
Design Capacity Example

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

Notes: 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
From Ref. 9

Assumptions 3 and 4 were similar to Assumptions 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 of 9.8. It turned out that Assumption 3 was the one most commonly used. (As a further point of interest, this exercise was conducted again recently, and the results were essentially the same.)

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.

A simple perusal of most recent references and texts does not provide 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, various ways are cited to evaluate the geotechnical input properties to do these calculations. Finally, typical FS values are given, most commonly in the range of 2 to 3 (which is essentially the same as was done in the 1960s). Obviously, there have to be differences resulting from different designers using different combinations of a design equation, property evaluation methodology and FS. If there have been local calibrations of all these factors with local load tests, then a sound design procedure could ensue. If not, then how sound is the design?

The Reliability-Based Design (RBD) Alternative

The design scenario described above poses many potential problems. The problems are not necessarily inherent in the individual components, but certainly in how they are linked together. RBD provides an alternative that has an advantage because the system components are evaluated together in a coherent fashion. More specifically, reliability analysis is the consistent evaluation of design risk using probability theory, and RBD is any design methodology that uses reliability analysis, explicitly or otherwise. Of course, in structural engineering, many engineers have embraced these concepts, and many structural codes already are based on RBD principles.

Consider the basic geotechnical evaluation process depicted in Figure 1. On the left is the *forcing function* — which could be a load, hydraulic gradient, chemical concentration or similar such item. This forcing function is applied to a geotechnical system that is represented by a model, coupled with necessary input properties, that should describe the sought-after behavior. The result of this process is the specific *system response* being considered or monitored. This process is imperfect, so the entire process usually is calibrated using field or other pertinent observations to "tweak" it

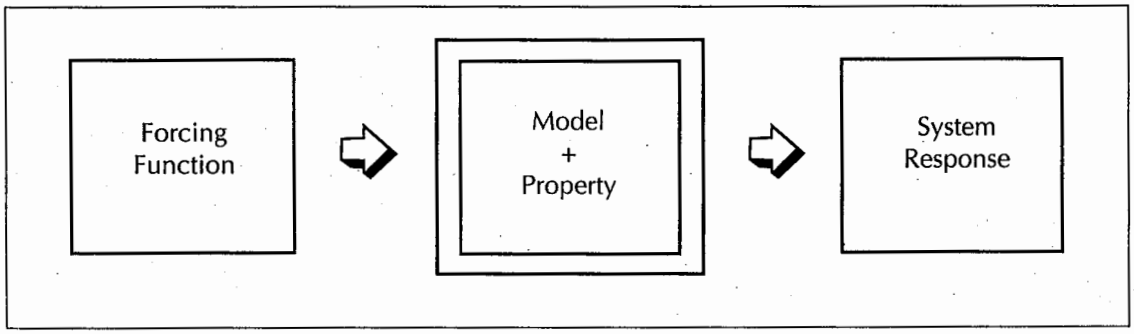

FIGURE 1. Components of the geotechnical evaluation process.

empirically so that it agrees with the actual observations. This physical calibration process is coupled with economic and safety/serviceability evaluations to implement the entire process in a design mode.

During the usual calibration process, all components are treated as deterministic quantities. A straightforward, easy-to-measure and unique forcing function is applied (such as dead loads or perhaps periodic repeating live loads in a pile load test). The system response commonly is monitored in limited fashion (such as butt or head movement and perhaps a few tell-tales in a pile load test). And then a model with associated properties is selected (which could range from a simple elastic continuum to very complicated, advanced models requiring many properties). The calibration process is such that either broad behavior is approximated in general (such as overall pile movements and load transfer) or specific aspects of behavior are approximated specifically (such as butt movement with load). In either case, deterministic empirical factors are developed for subsequent use in design.

In reality, the actual loads on the pile are complex and consist of dead, sustained live and transient live loads. The dead loads can be assessed reasonably accurately, but detailed assessment of the live loads is complicated. However, for design purposes, loads are established based on a number of assumptions of behavior. For example, in transmission line structure engineering, it is common to focus on the 50-year return period maximum wind, which has a mean coefficient of variation (COV) of the maximum wind speed on the order of 15 percent.[10] The COV is the standard deviation divided by the mean, both ideally for a large population, and essentially is a measure of the dispersion of the data.

The system response can have even more uncertainty. An example of this issue is presented in a study by Lambe, *et al.*, which examined the reliability of geotechnical predictions, primarily for earth dams and natural slopes, but it should still provide some insight into overall reliability.[11] Their evaluations suggested that geotechnical predictions can be made as follows:

- Predicted vertical displacement within ±50 percent of the measured vertical displacement;
- Horizontal displacement within ±150 percent;
- Change of pore water pressure within ±25 percent;
- Horizontal stress within ±50 percent;
- FS within ±25 percent; and,
- Flow within one order of magnitude.

The model to be selected normally is a simplification of actual behavior, primarily because in-situ geotechnical behavior is very complex. Finally, there is a great deal of uncertainty in the evaluation of the in-situ geologic and geotechnical regimes and their properties.

With all of these variabilities and uncertainties, it would seem to be natural for RBD to be applied extensively within geotechnical engineering. However, almost the opposite has been true. In contrast, structural engineers have embraced RBD openly for the last several decades, and much of structural design is now done using RBD codes. The load evaluation

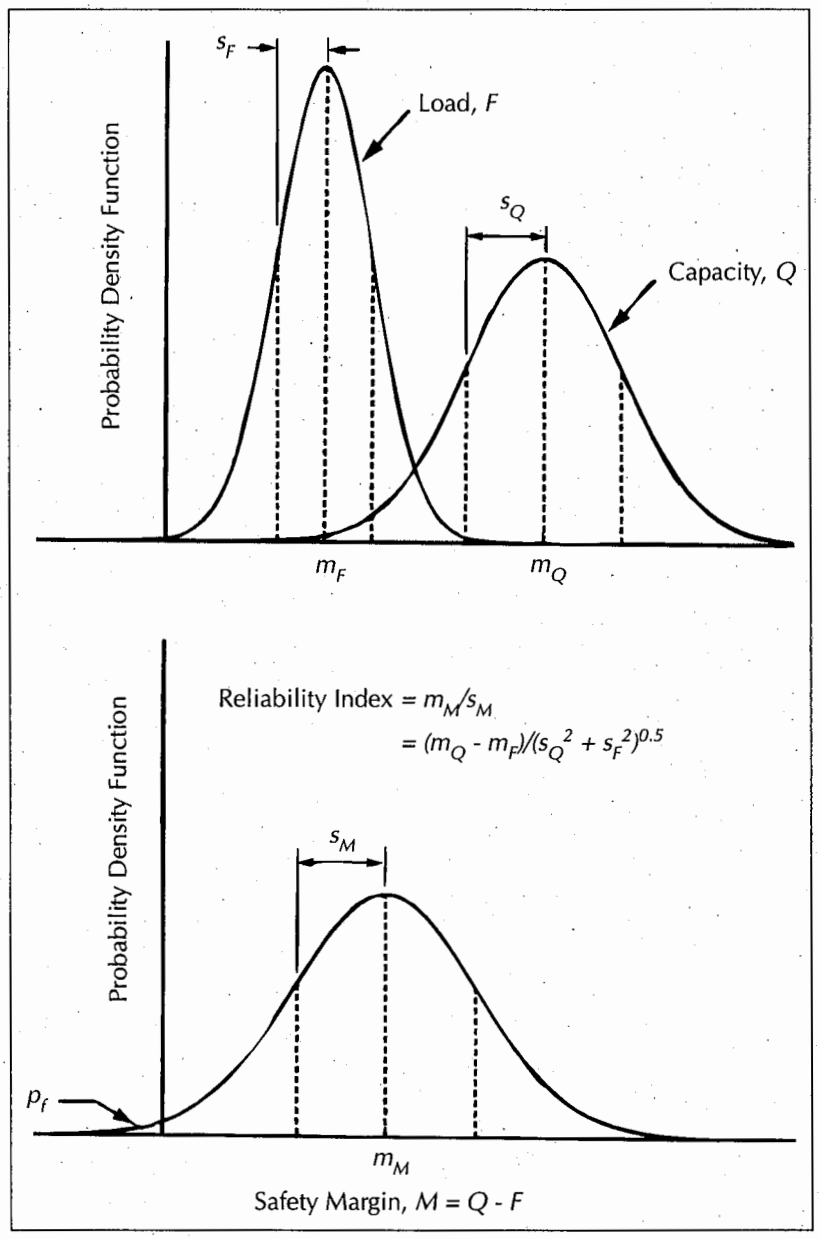

FIGURE 2. Reliability assessment for two normal random variables (Q and F).

and structural design are being done almost on a daily basis with reliability-based limit state design using load and resistance factors, while the geotechnical and foundation designs are being done using working stress design with unfactored (?) loads. Serious incompatibilities could develop with this situation.

In addition, the overall form and style of the structural RBD equations have largely been set. The de facto formulations to date have focused largely on a single resistance factor and on the resistance (capacity) having a relatively small COV that also is much smaller than the COV of the loads. These assumptions may not be appropriate for geotechnical design, because natural in-situ materials have to be considered. Also, there is much reliance on judgment in interpreting limited site information, as well as reliance on many empirical correlations. These factors generally do not enter into structural design and, therefore, the resistance evaluation is simpler.

Uncertainties have long been appreciated in geotechnical engineering, whether they are called *calculated risks* or some other term. Traditionally, a FS is used in design (which of course includes uncertainties), but the uncertainties are addressed only qualitatively and subjectively within the framework of the evaluation process described in Figure 1. RBD is an alternative that embraces the uncertainties in the loads and capacities directly by modeling them mathematically as random variables, and then quantifies design risk as a probability of failure. While there certainly are some sophisticated mathematical tools and manipulations required to address the uncertainties and reliability models directly, they would not be encountered in routine geotechnical design, just as they are not encountered in routine structural

EQUATION 1

$$P_f = Prob(Q < F) = Prob(Q - F < 0)$$

EQUATION 2

$$\beta = -\phi^{-1}(p_f)$$

design. Instead, the rigorous analyses are used as part of the calibration process that results in straightforward RBD equations that have a "look and feel" that is comparable to that of the conventional deterministic equations.

Some RBD Definitions

For simplicity, capacity (Q) and load (F) commonly are assumed to be characterized by normal distributions as shown in Figure 2. For these distributions, the key parameters are the mean (m) and standard deviation (s), with the COV given by s divided by m, and the safety margin (M) is given by Q minus F. Other forms of distributions are available, and they very well may be preferred in the future. However, at this time, the normal distribution generally is preferred because of its simplicity. In any case, the assumed distribution is a part of the calibration process.

With the distributions shown in Figure 2, the probability of failure (p_f) is given by Equation 1. The first case (with Q and F) is depicted by overlapping distributions, while the second (with M) is depicted by a negative safety margin.

As noted previously, the term *probability of failure* has a negative connotation. An alternative is a more positive sounding term — *reliability index* (β), which is related to p_f as shown in Equation 2, in which ϕ^{-1} is equal to the inverse normal cumulative function given in standard statistics texts. The resulting relationship is presented in Table 2. Note that the numerical values of β are perhaps more convenient to work with, but that the relationship between β and p_f is highly nonlinear. The value of β should not be mistaken as a FS.

TABLE 2.
Relationship Between Reliability Index & Probability of Failure

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

Incorporating Geotechnical Uncertainty

Geotechnical uncertainty has to be incorporated explicitly in a meaningful manner. Most available RBD codes tend to address capacity as a single lumped parameter with a single resistance factor. However, this process is not optimal, because most geotechnical resistance parameters are displacement-dependent. Consider, for example, the typical load-displacement behavior of a drilled shaft in compression as illustrated in Figure 3. In the figure, the tip and side constitute significantly different components of the shaft resistance with increasing displacement. Typically, the side resistance is mobilized at a displacement of 10 to 15 millimeters, while the tip resistance is mobilized at a displacement of 5 to 10 percent (or more) of the shaft diameter. In addition, the shaft weight is mobilized at the first onset of displacement. Clearly, different resistance factors are appropriate for these side, tip and weight components.

Next, the variability in the soil (or rock) properties used to predict these resistance components must be considered. Figure 4 illustrates the inherent variability of soil, which is the reality of in-situ conditions that must be dealt with. However, the trend of the desired property can be established for a specific layer, along with the deviation from this trend, as well as the scale of fluctuation — which roughly defines the interval wherein the deviations exhibit a strong correlation. This inherent soil variability is described elsewhere in more

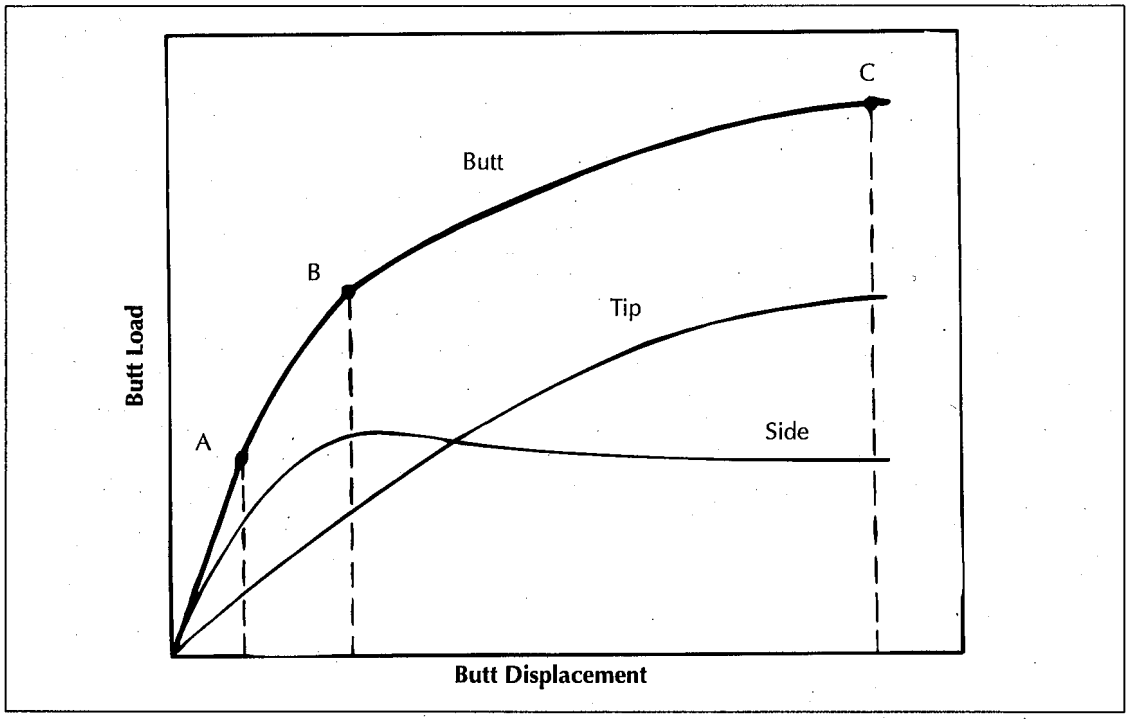

FIGURE 3. Typical load-displacement behavior of a drilled shaft in compression.

detail,[12] but it is only the first component of the overall uncertainty (as illustrated in Figure 5).

The second component is the measurement uncertainty that is attributed to data scatter and statistical uncertainty. Data scatter includes the inherent soil variability again and the measurement errors from equipment, procedural/operator and random testing effects. Equipment effects result from inaccuracies in the measuring devices and variations in equipment geometries and systems employed for routine geotechnical testing. Procedural/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.[14] 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. These in-situ measurements also are influenced by statistical uncertainty or by 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.[13]

The third component of uncertainty is introduced when field or laboratory measurements are transformed into design soil properties using empirical or other correlation models (for example, 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 clearly depends on the site conditions, the degree of equipment and procedural control, and the quality of the correlation model.

Unfortunately, there is no standard methodology for the systematic evaluation of geotechnical variability, and there is no broad-based compilation of soil property statistics that is suitable for general use. Because of these problems, a major research effort was undertaken to collect and synthesize statistical data on each of the major sources of the uncertainties described above.[10] For each combination of soil type, measurement technique and correlation model, the uncertainty in the design soil prop-

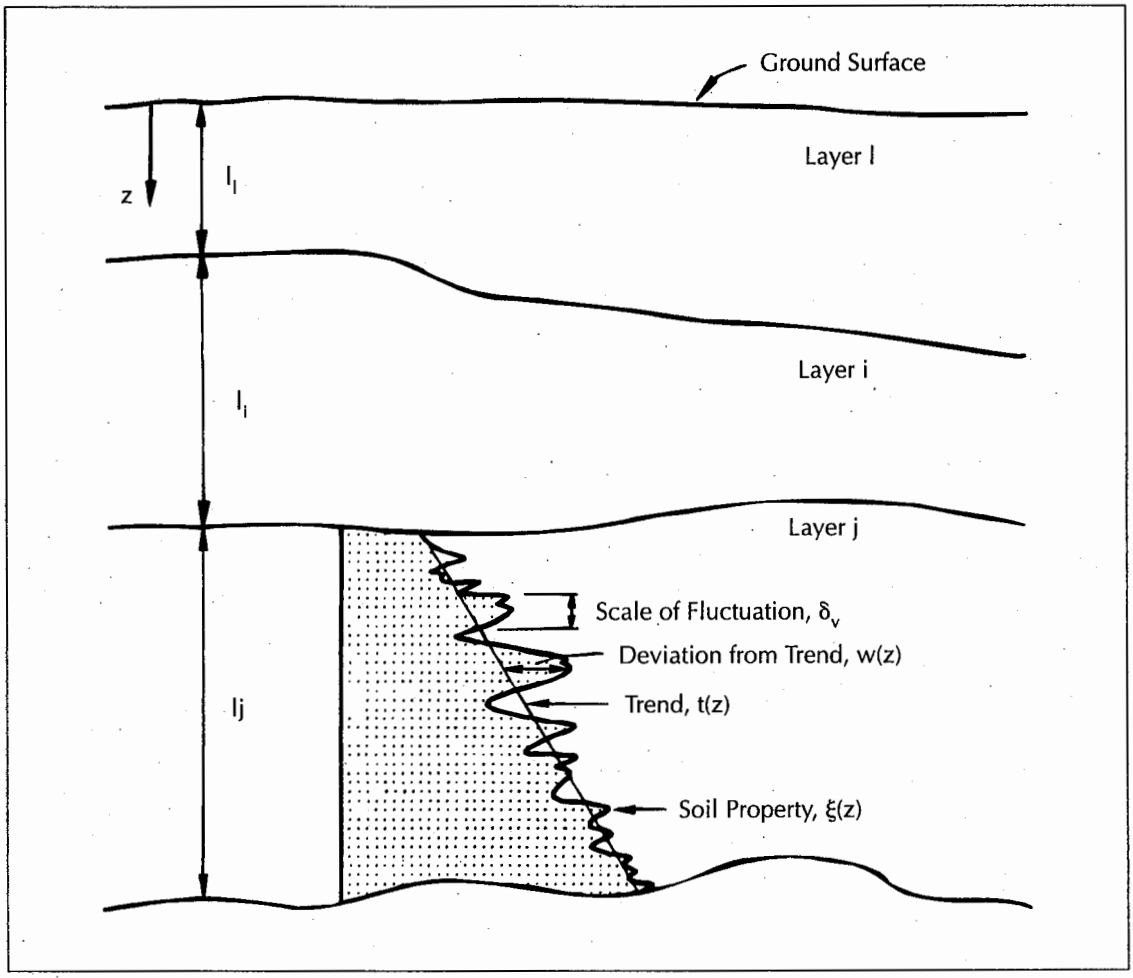

FIGURE 4. Inherent variability of soil.

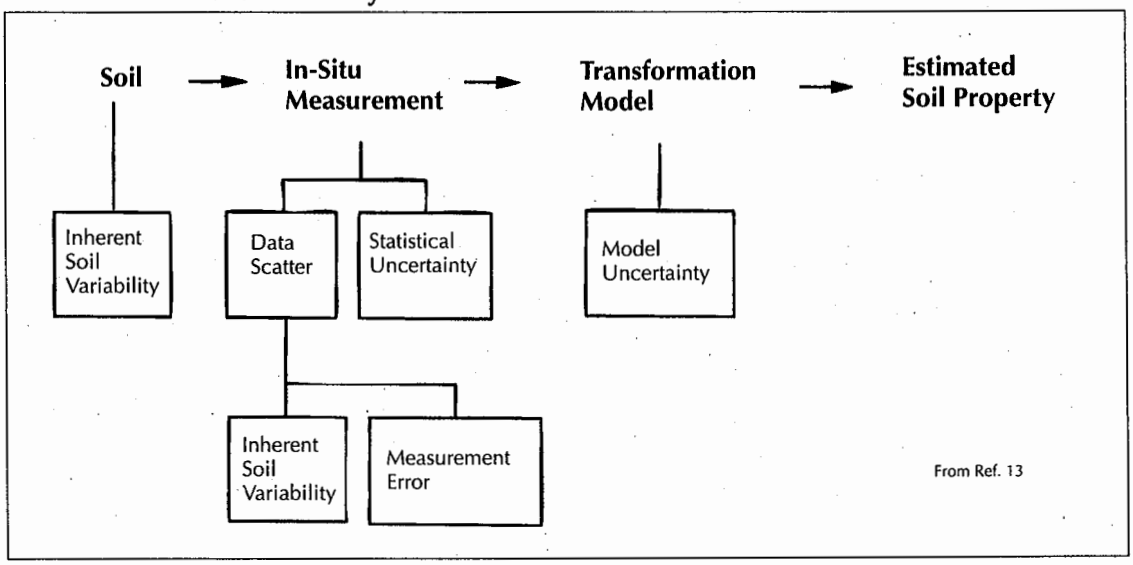

FIGURE 5. Uncertainty in soil property estimates.

TABLE 3.
Approximate Guidelines on the Variability of Some Soil Strength Properties

Design Property*	Test**	Soil Type	Point COV (%)
s_u(UC)	Direct (Lab)	Clay	20-55
s_u(UU)	Direct (Lab)	Clay	10-35
s_u(CIUC)	Direct (Lab)	Clay	20-45
s_u(Field)	VST	Clay	15-50
s_u(UU)	q_T	Clay	30-40****
s_u(CIUC)	q_T	Clay	35-50****
s_u(UU)	N	Clay	40-60
s_u***	K_D	Clay	30-55
s_u(Field)	PI	Clay	30-55
$\bar{\phi}$	Direct (Lab)	Clay, Sand	7-20
$\bar{\phi}$(TC)	q_T	Sand	10-15****
$\bar{\phi}_{cv}$	PI	Clay	15-20****

Notes: * s_u = undrained shear strength; UC = unconfined compression; UU = unconsolidated-undrained triaxial compression test; CIUC = consolidated isotropic undrained triaxial compression test; s_u(Field) = corrected s_u from vane shear test; TC = triaxial compression test; $\bar{\phi}$ = effective stress friction angle; $\bar{\phi}_{cv}$ = constant volume $\bar{\phi}$
** VST = vane shear test; q_T = corrected cone tip resistance; N = standard penetration test value; K_D = dilatometer horizontal stress index; PI = plasticity index
*** Mixture of s_u from UU, UC and VST
**** COV is a function of the mean (for details, see Ref. 10)
From Ref. 10

erty was determined rationally by combining the appropriate component uncertainties using second-moment probabilistic methods. General guidelines on the typical coefficients of variation of some common design soil strength properties are given in Table 3. Ideally, site-specific values for the COV would be determined, but this is only really possible on rather large projects or in areas where much geotechnical data are available from numerous local projects. Exact COV values are not needed in the RBD equations, so there should be no concern about evaluating these values precisely.

Formulation of RBD Equations

The basic objective of RBD is to ensure that the probability of failure of a component does not exceed an acceptable threshold level. While this objective clearly is satisfied if the probability of failure of a component lies far below this threshold, it is equally clear that this design also would not be economical. Therefore, a realistic design objective would be that the probability of failure should not depart significantly from the threshold. For the case illustrated in Figure 2, the RBD objective can be stated as in Equation 3, in which p_T is equal to the acceptable target probability of failure. In principle, a rational value for p_T can be determined from a cost-benefit analysis as shown in Figure 6. By evaluating the variation of the initial, maintenance and expected failure costs with p_f, it is theoretically possible to arrive at the most economical p_T for design.[15] However, this approach is not practical at this time because of the difficulties in estimating many types of failure costs (for example, the "cost" of human lives) and the effect of component failure on the system. A viable alternative is to set the value of p_T at a level that is consistent with the failure rates estimated from actual case histories. However, comparing the theoretical probability of failure from reliability computations with a value established from case histories is not

EQUATION 3

$$Prob(Q < F) \leq p_T$$

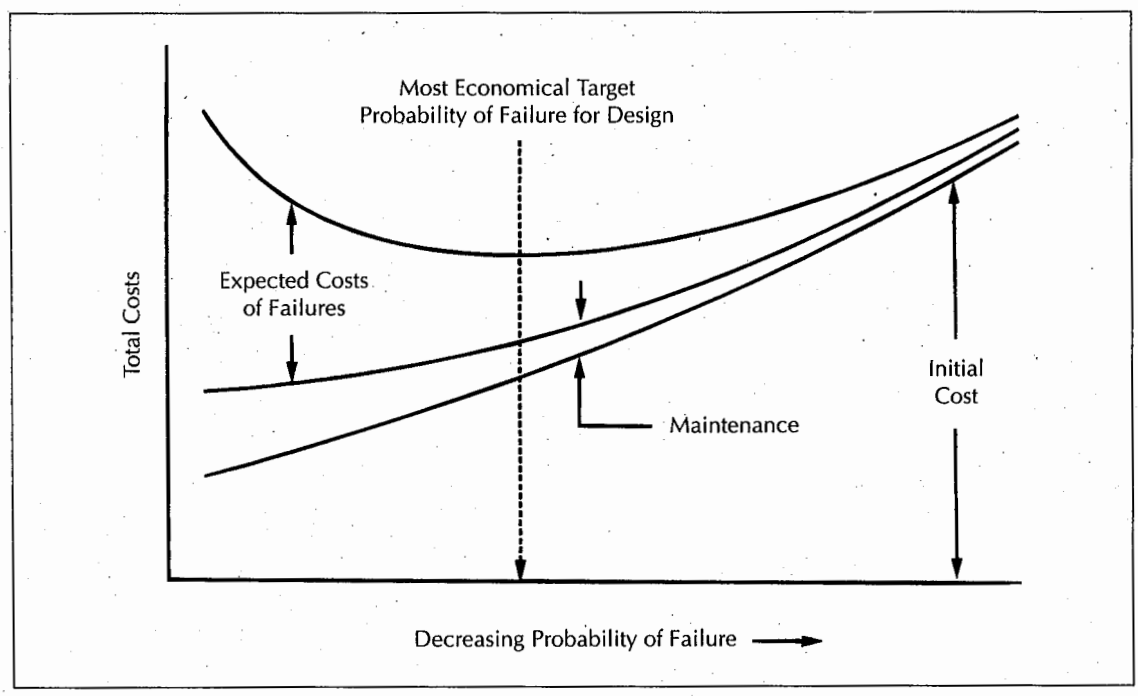

FIGURE 6. An illustrative cost-benefit analysis.

straightforward. It has been reported that the theoretical probability of failure usually is significantly smaller than the actual failure rate.[16] This result is to be expected because design safety is affected by more than the uncertainties in design calculations. It also can be compromised by factors such as poor construction and human errors.

Currently, the most widely used approach for selecting a target probability of failure for design is to calculate the theoretical probabilities of failure implicit in existing designs and to use those values as a broad initial basis for selecting an appropriate value of p_T.[17] While this approach is empirical, it has a major advantage of keeping the new design methodology compatible with the existing experience base. Furthermore, this approach is consistent with the evolutionary nature of codes and standards that requires changes to be made cautiously and deliberately.

The general approach for the calibration of p_T involves the following four steps:[10]

1. Select a set of representative design problems covering typical ranges of geometries, properties, etc.
2. Determine an acceptable solution to each problem based on existing methodology (such as the allowable stress method).
3. Evaluate the probability of failure for each design solution from Step 2 using a common reliability calculation scheme (for example, the first-order reliability method [FORM]) and a common set of probabilistic models.
4. Select an appropriate value for p_T based on the range of p_f computed in Step 3.

RBD using Equation 3 involves the repeated use of reliability assessment routines, such as FORM, to evaluate the probabilities of failure of trial designs until the computed probability of failure is reasonably close to the target threshold level. While this approach is rigorous, it is complicated and, therefore, not suitable for the routine design of conventional types of structures and foundations with no abnormal risks or with no unusual or exceptionally difficult ground or loading conditions. In a recent study, a simplified RBD methodology was developed that uses conventional lumped or multiple-factor formats for foundation design.[10] Equations 4 and 5 illustrate this meth-

EQUATION 4

$$F_{50} = \Psi_u Q_{un}$$

EQUATION 5

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

odology for uplift loading of a drilled shaft in which F_{50} is the 50-year return period design load (typical for electrical transmission line structure design), Q_{un} equals the nominal foundation uplift capacity, Q_{sun} equals the nominal uplift side resistance, Q_{tun} is the nominal uplift tip resistance, W is the weight of shaft, and Ψ_u, Ψ_{su}, Ψ_{tu} and Ψ_w are resistance factors. Comparable forms of the design equations were developed for other loading modes as well.

In principle, any reasonable format could be used for reliability calibration. However, practical issues such as simplicity, familiarity and compatibility with existing design approaches are important considerations in proposing a simplified RBD approach. The formats shown in Equations 4 and 5 clearly satisfy these issues. For example, the resistance factor in Equation 4 corresponds to the reciprocal of the FS in the traditional allowable stress approach. Equation 4 also is known as the *load and resistance factor design* (LRFD) format that already has been adopted widely in the structural community for RBD. Equation 5 is a broad generalization of Equation 4 that includes multiple resistance factors, one for each component of the capacity.

The resistance factors in Equations 4 and 5 were calibrated rigorously using FORM to produce designs that essentially achieve a target reliability index (β_T) equal to 3.2. This value is representative of those implicit in current designs assessed within the calibration process, is greater than that used for transmission line structure design and also is consistent with empirical rates of failure (accounting for the fact that actual failure rates are on the order of one order of magnitude higher than the theoretical rates). Details of the calibration process are given by Phoon, *et al.*[10] It should be noted that consistently better designs, much closer to β_T, were obtained with the multiple factor format, which therefore should be the preferred format for foundation engineering.

The results of an extensive reliability calibration study for the ultimate limit state (ULS) design of drilled shafts under undrained uplift loading are given in Table 4 and are to be used with Equation 5. All other limit states, foundation types, loading modes and drainage conditions addressed offer similar types of results, with simple RBD equations and corresponding tables of resistance factors.[10] Note that the resistance factors depend on the clay consistency

TABLE 4.
Undrained Uplift Resistance Factors for the Ultimate Limit State Design of Drilled Shafts (Using Equation 5)

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

Notes: Target reliability index = 3.2. From Ref. 14.

and the COV of the undrained shear strength (s_u). The clay consistency is classified broadly as medium, stiff and very stiff, with corresponding mean s_u values of 25 to 50, 50 to 100 and 100 to 200 kN/m^2, respectively. Foundations are designed using this new RBD format in much the same way as in the traditional approach, except that the rigorously determined resistance factors shown in Table 4 are used in place of empirically determined factors of safety. Additional details on this specific application of RBD can be found elsewhere.[10,18-20] Although not described herein, comparable design equations and factors have been developed for the serviceability limit state (SLS).[10]

RBD Advantages & Disadvantages

The simplified RBD methodology has three important advantages. First, the multiple-factor format is familiar to most practicing engineers, and it has the same "look-and-feel" of traditional design formats. Second, it is easy to use because the design engineer does not have to perform the elaborate probability computations that are necessary to develop the resistance factors in the RBD format. Finally, this RBD methodology satisfies the RBD objective stated in Equation 3, with the resistance and load factors calibrated to produce designs that essentially achieve a target reliability index consistently.

The primary disadvantage to using RBD might be the perceived loss of flexibility because the design engineer can not freely change the predictive model, the underlying probability distributions for loads and strengths, and the target reliability index. Rigorous re-calibrations would be needed if any of these factors were to be changed. However, this "perceived" loss does not end up being a real one for most designers. Most designers would probably welcome this type of approach. There would be far less agonizing over what equation to use and how to use it, what geotechnical parameters to obtain and how, and what factor of safety to use and why. The geotechnical engineer instead can focus on ground evaluation issues, unraveling the geologic setting and stratigraphy, establishing proper trend and mean values of the pertinent soil properties, assessing the variability or COV of the properties and addressing construction issues. With formats such as the one presented in Table 4, the engineer also can make a direct assessment of the economic value of "better" data that normally have a lower COV.

Concluding Comments

Much has occurred in risk assessment since Casagrande discussed his "calculated risk." Many researchers have been able to make valuable contributions to uncertainty assessment and RBD during this time, and we are now literally on the verge of a new foundation design methodology. RBD already has been embraced by structural engineers, and now it is becoming a reality in geotechnical engineering. However, the resulting RBD equations must be established in a manner that best optimizes geotechnical design. Generally, doing so means using multiple resistance factor formats and calibrating design equations with the realization that the resistance side of the equation can very well represent the greatest uncertainty in design. During this calibration process, there should be careful examination of how the geotechnical data are obtained, as well as the resulting property variability. Particular attention needs to be focused on the target reliability index, which can (and likely will) vary as a function of structure type and use, foundation type and use, and loading condition, as a minimum.

With a proper, simplified RBD format, the geotechnical engineer can focus on ground and construction evaluation in a rigorous fashion, without having to agonize (as in the past) over use of the "right" design equation or how to select the "proper" FS in a rational and defensible manner. Although there is much research yet to be done in the RBD arena for the full range of geotechnical design conditions, it really can be used now for specific cases in a rational and practical design mode.

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FRED H. KULHAWY *is Professor of Civil/Geotechnical Engineering at Cornell University, Ithaca, New York, where he heads the Geotechnical Engineering program. He is an internationally acclaimed educator, consultant and researcher. His teaching, research and practice focus on foundations, soil-structure interaction, soil and rock behavior, and geotechnical computer applications. He is the author or co-author of over 230 published technical papers and reports.*

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