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ON THE EVALUATION OF STATIC SOIL PROPERTIES

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ABSTRACT: Soil material properties are complex entities that can be evaluated in a number of ways. This paper focuses on procedures to evaluate these properties in a consistent and reproducible manner. Attention is brought to the modeling category, laboratory or field measurement type, field boundary condition, uncertainty issues in the soil, measurements, and models, and the overall property representation. General criteria are given to represent soil properties in a rational manner.

INTRODUCTION

The evaluation of soil properties is no trivial task. In fact, with the maturing of the geotechnical engineering discipline and the attendant accumulation of knowledge, a general realization has come about that static soil properties are highly variable entities that must be evaluated carefully for specific purposes. This realization is already pervasive throughout the research community that focuses on laboratory or in-situ measurements or predictions of properties, but it is only developing slowly in other sectors of practice. During the past fifteen years, a number of major overview papers have been written that have traced this steady accumulation of knowledge (e.g., Ladd, et al., 1977; Wroth, 1984; Wroth and Houlsby, 1985; Jamiolkowski, et al., 1985; Jamiolkowski, et al., 1991), and a major design manual on estimating soil properties also has been prepared (Kulhawy and Mayne, 1990). These documents have shown an increasing sophistication in property evaluation, including careful matching of test and prototype variables and a direct awareness of variability (or uncertainty) in the property evaluation process. However, with the increasing sophistication comes more demand and responsibility for assessing all components of the property evaluation process. In this paper, an overview will be presented of the key issues involved in the evaluation of static soil properties. Although the focus is on slope and embankment problems, the overview and resulting philosophy are general and can be applied to most geotechnical design problems.

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SOME BASIC DEFINITIONS

All of us intuitively understand what a property is, but it often is worthwhile to review the "standard" definitions before exploring the topic further. Table 1 extracts the pertinent technical definitions from three well-known and respected sources used extensively in North America. Although all three sources use different words, the underlying pervasive theme is "uniqueness of an attribute" that can be evaluated quantitatively from experiments. This type of definition is perhaps overly optimistic when the material in question is soil, because most soil performance properties (e.g., strength, modulus, etc.) are nonlinear and stress-dependent, as a minimum.

Table 1. Some Basic Definitions of a Property

WEBSTER'S UNABRIDGED DICTIONARY (Gove, 1966)
"quality or trait belonging to a person or thing"
"attribute, characteristic, or distinguishing mark common to all members of a class or species"

RANDOM HOUSE UNABRIDGED DICTIONARY (Flexner, 1987)
"essential or distinctive attribute or quality of a thing"

DICTIONARY OF GEOLOGICAL TERMS (AGI, 1962)
"system characteristic that can be evaluated quantitatively from experiments"

Instead, properties ought to be viewed within their specific context. Consider, for instance, the situation depicted in Figure 1, which simply shows that a prediction is made from a load by using a model and a property together. To make a sound prediction, the model and property should be of parallel form, as given in Table 2. Simple models are paired with simple tests, while sophisticated models are paired with sophisticated tests. However, contrary to many popular views, increasing the level of sophistication does not necessarily increase the quality of the prediction. If the model and property are calibrated together for a given loading and subsequent prediction, then all three modeling categories should be comparable, as long as the type of behavior to be predicted is legitimately within the capability of the model. For example, to predict the elastic response of a soil mass, only Category III modeling is necessary, although all three categories should give the same results. Category I modeling would represent overkill in this example.

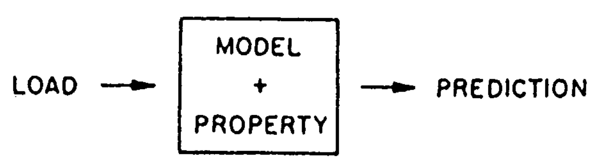


Figure 1. Components of Geotechnical Prediction

Table 2. Categories of Analytical Methods for Soil Modeling

Category	Main Features of Models	Determination of Soil Properties
I	Very advanced models using nonlinear elastic-plastic time-dependent laws that possibly incorporate anisotropic behavior	Only from sophisticated laboratory tests, with the exception of variables that must be obtained from in-situ tests
II	Advanced models using constitutive incremental elastic-plastic laws and nonlinear elastic relationships	Laboratory tests that are only a little more sophisticated than conventional tests; in-situ tests also appropriate
III	Simple continuum, such as isotropic elastic continuum, including layering and empirical models	Conventional laboratory and in-situ tests

(Kulhawy and Mayne, 1990, as adapted from Jamiolkowski, et al., 1985)

Conversely, if a rather sophisticated problem involving nonlinearity, soil yielding, and time-dependency was to be evaluated, Category I modeling would be appropriate (if the needed analytical models and laboratory tests actually are available). Categories II and III could be used, if sufficient data were available to calibrate these model levels empirically. But then these models would be applicable only for the specific range of conditions within which the empirical calibration was done. Extrapolation beyond these conditions would be inappropriate and potentially misleading.

Attempts might also be made to use models and properties in non-parallel form, such as a Category I or II model and Category III properties. However, these pairs then would need new empirical calibrations of their own, because those developed for parallel form modeling would not be appropriate. Much non-parallel evaluation is done in geotechnical engineering, largely because it is part of our professional heritage that is steeped in empirical correlations. An extreme example of this thought process is the almost unnatural obsession of segments of the geotechnical community for attempting to correlate virtually all types of predictions to the standard penetration test N-value or the cone penetration test q_c-value. Poulos and Brown (1986) put this thought process in perspective as follows:

> "The geotechnical parallel to the mythological quest for the Holy Grail is the search for a means by which geotechnical properties of a soil or rock may be determined straightforwardly and reliably from relatively simple in-situ tests."

In the remainder of this paper, oversimplification of this type will be avoided, parallel modeling will be adopted, and the appropriate level of modeling sophistication will be assumed.

BASIS FOR PROPERTY EVALUATION

Properties are of two main types: index and performance. Index properties are those that describe the soil constituent characteristics in some way, such as water content, gradation, plasticity, etc. Performance properties represent how the soil responds to some imposed gradient, such as gravity, boundary load, hydraulic head, etc. Index properties are determined in a rather straightforward manner, from tests on specimens sampled in the field, and are of two broad types, listed below:

(a) direct laboratory measurements
(b) indirect laboratory or field visual-manual procedures for gradation and plasticity

However, with performance property evaluation, no less than ten different procedures have been used to date, as summarized in Table 3. With this amount of diversity, it is inevitable that variations occur in predicted properties.

It is perhaps most important to realize with these performance property evaluations that different results should be expected. After all, different tests, procedures, or models are being invoked and all have a different basis. Consider the test representations shown in Figure 2. All could be used to estimate a soil strength, and most could be used to evaluate stress-deformation behavior. But the results will not be the same because the boundary conditions, modes of loading, and stress paths differ, as a minimum. The overview references cited previously cover these issues very well.

These same issues also carry over into the field, where different boundary conditions, stress paths, etc. will apply. Figure 3 illustrates a few common cases related to embankments, walls, and slopes. It is clear from these cases that no one type of test usually addresses the actual field conditions. Instead, a combination might be warranted.

The issues raised in Table 3 and Figures 2 and 3 are not new, and they do not represent an academic exercise. They are real, and they are important, even if they are not incorporated explicitly in much of current practice. Where much of the problem has been is that the information has been piecemeal. Advances in assessing loads, improving models, evaluating properties, and making predictions have been done largely independently, and therefore the overall process shown in Figure 1 is probably not in truly parallel form within different segments of design practice. Instead, there is a general tendency to estimate loads on the high side and properties on the low side, which then will lead to a conservative prediction. The goal really should be to make a direct prediction that is accurate and economical, but which addresses the uncertainties in the problem in a specific and realistic manner.

Table 3. Methods for Evaluating Soil Performance Properties

DIRECT MEASUREMENTS ON SPECIMENS SAMPLED IN FIELD
- (a) using laboratory tests that attempt to simulate the appropriate boundary conditions
- (b) using field tests that attempt to simulate the appropriate boundary conditions

EMPIRICAL CORRELATIONS WITH DIRECT MEASUREMENTS
- (c) through index properties, such as s_u vs. plasticity index
- (d) using field tests that do not simulate the boundary conditions, but still exhibit the general trends, such as s_u vs. N
- (e) through normalized relationships with other performance properties, such as estimating E from E/s_u vs. plasticity index

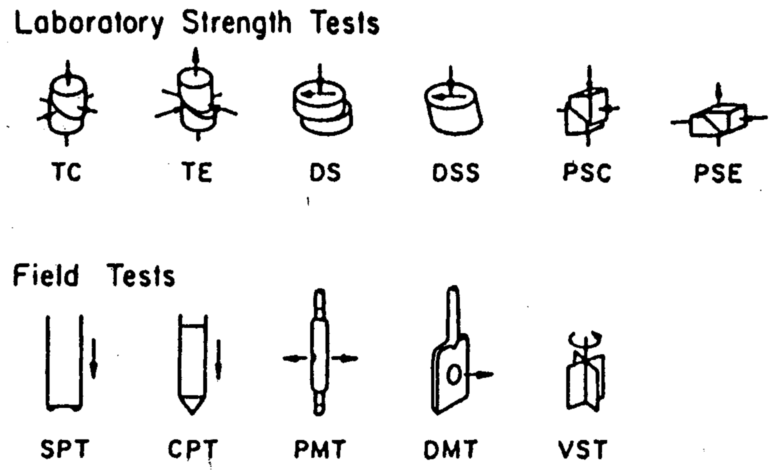
BACK-CALCULATIONS FROM LOAD TESTS, EITHER REDUCED OR FULL-SCALE
- (f) using appropriate parallel modeling, from which the property and its step-by-step variation with load is computed
- (g) by calibration with non-parallel modeling between the load and prediction to estimate an overall "average" or "representative" property [e.g., using an elastic model to estimate a working load level of E_{25} or E_{50} from tests that obviously are nonlinear]

THEORETICAL OR OBSERVATIONAL METHODS
- (h) using well-founded theoretical models, such as Cam clay and modified Cam clay, some properties can be predicted directly
- (i) using published summaries, based on description and consistency alone, of property values that are "typical", "average", or "representative"
- (j) using assumptions of material behavior by a particular model that then dictate other properties, such as assuming undrained, saturated, isotropic, linear behavior, which then dictates that $\nu = 0.5$ and therefore $E = 3G$

s_u = undrained shear strength
N = standard penetration test value
E = Young's modulus [E_x = value at x% failure]
ν = Poisson's ratio
G = shear modulus

PROPERTY EVALUATION STRATEGY

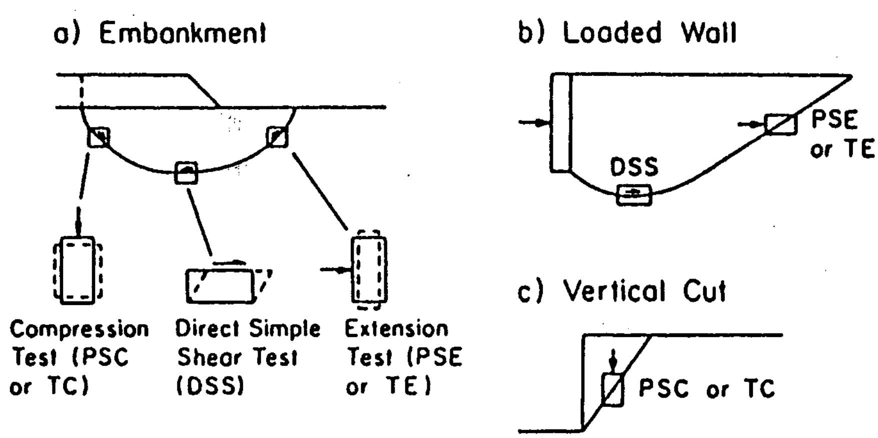
A soil performance property is a specific attribute that describes a particular type of response at a given time under a prescribed set of boundary conditions, stress paths, etc. With manufactured materials, the property evaluation is more straightforward because the material composition is controlled. A particular property should occur for a specific composition, within some relatively small manufacturing or testing variability. However, this variability often is so small that the property can be considered to be deterministic. This property status in Figure 1 greatly simplifies making a prediction, be-



SYMBOLS:

TC - triaxial compression	SPT - standard penetration test
TE - triaxial extension	CPT - cone penetration test
DS - direct shear	PMT - pressuremeter test
DSS - direct simple shear	DMT - dilatometer test
PSC - plane strain compression	VST - vane shear test
PSE - plane strain extension	

Figure 2. Common Laboratory Strength Tests and Field Tests



Note: Plane strain tests (PSC/PSE) used for long features
Triaxial tests (TC/TE) used for near symmetrical features
Direct shear (DS) normally substituted for DSS to evaluate $\bar{\phi}$

Figure 3. Relevance of Laboratory Strength Tests to Field Conditions

cause the key variable then is the load. However, in most geotechnical problems, the property is the key variable and the load variability is of lesser importance.

The first issue to be addressed in soil property evaluation is that of representative sampling. Figure 4 illustrates the relative volumes

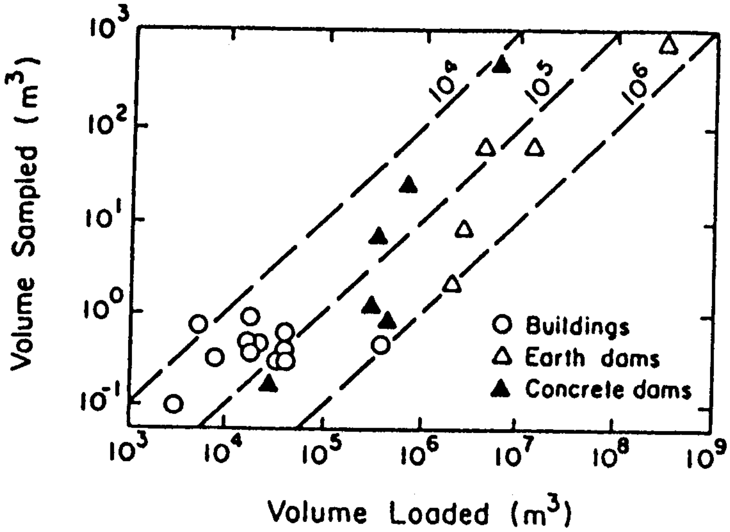


Figure 4. Illustrative Soil Volumes Sampled versus Loaded (Price and Knill, 1974)

sampled and loaded for several geotechnical problems. As can be seen, 10^4 to 10^6 times more soil volume commonly is loaded than is tested. With a ratio this large, it is obvious that careful strategies need to be implemented to obtain representative samples of the soil that is being loaded. Geologic inference and use of prior information are particularly useful in this process (e.g., Spry, et al., 1988). These are very important items, but they are beyond the scope of this paper.

Assuming representative sampling, the next issue is evaluating the components of uncertainty, outlined in Figure 5 for a typical in-situ test. As shown, there are uncertainties in the soil itself, in the measurements, and in the model used to transform the measurements into an estimated soil property. The inherent soil variability results from depositional, compositional, environmental, and diagenetic factors, as noted schematically in Figure 6. A typical case is illustrated in Figure 7, showing both horizontal and vertical variability. An adequate exploration would delineate the layering, and normally there would be more vertical than horizontal variability. Within a

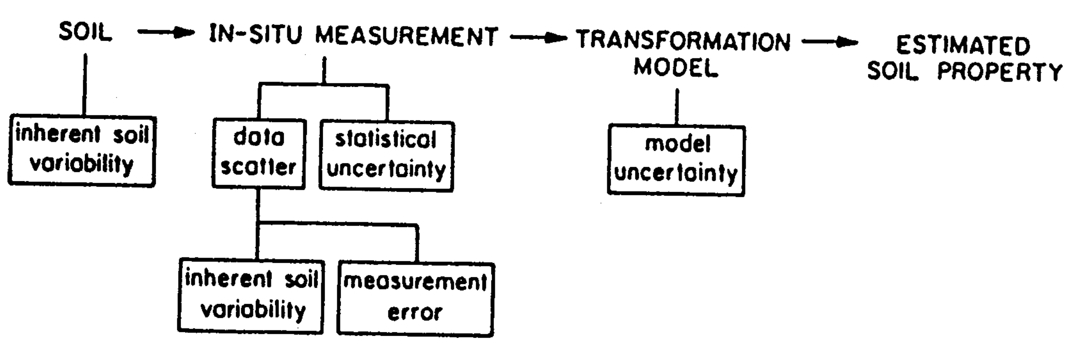


Figure 5. Uncertainty in Soil Property Estimates

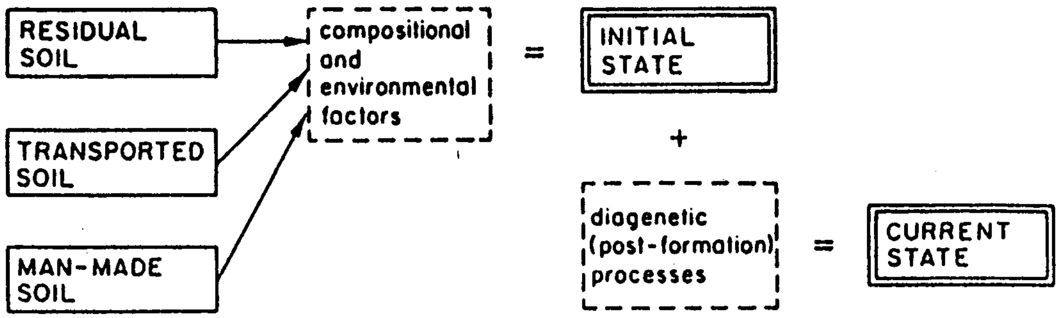


Figure 6. Soil Structure Evolution (Kulhawy, et al., 1989)

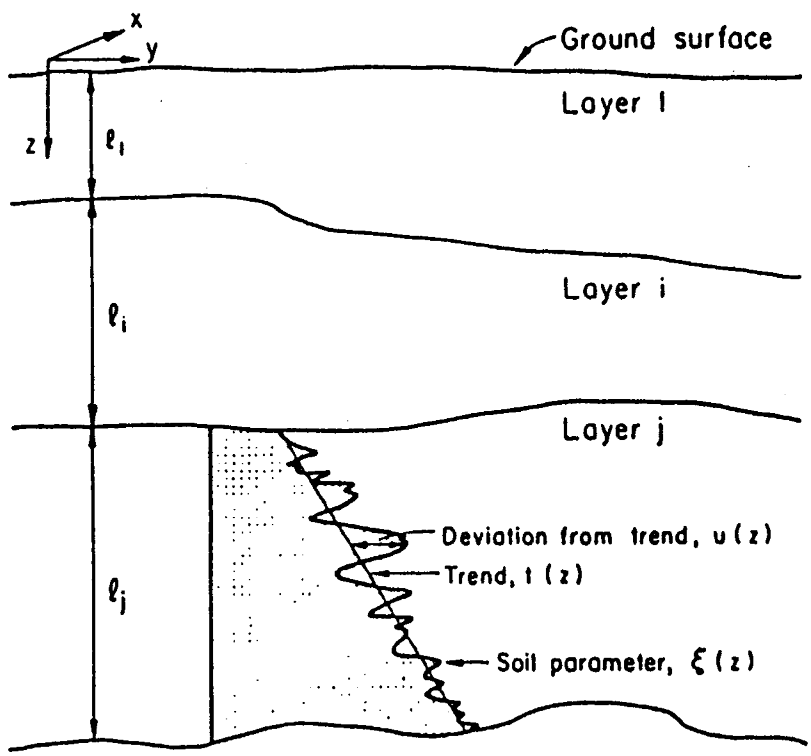


Figure 7. Soil Profile with Property Variation

given layer, any in-situ parameter, $\xi(z)$, changes with depth. This change can be modeled conveniently by treating $\xi(z)$ as a random variable composed of a trend, $t(z)$, and a deviation about the trend, $u(z)$, which represents the inherent soil variability (e.g., Filippas, et al., 1988).

Then an in-situ measurement can be made by some test (Figure 5). The value obtained could be close to or far from the actual in-situ parameter, depending on the data scatter and statistical uncertainty. The data scatter includes the inherent soil variability and measurement errors that result from equipment, procedural/operator, and random test effects (Orchant, et al., 1988). These effects vary considerably among different test types. Also included within the in-situ measurement is the statistical uncertainty or sampling error that results from limited information. This uncertainty can be minimized with increased testing, but it is commonly included within the measurement error at this time.

The in-situ measurement then can be used to estimate a soil property by invoking a transformation model, as illustrated in Figure 8, using the undrained shear strength, $s_u(z)$, as an example. With this transformation, $s_u(z)$ is estimated as follows:

$$s_u(z) = a + (b + M)\ \xi(z) \tag{1}$$

using a linear model with intercept a, slope b, and uncertainty M. The in-situ parameter $\xi(z)$ is given by a trend and deviations about the trend, as noted previously. The data scatter and statistical uncertainty add to the measurement uncertainty, e. These three parameters (t, u, e) collectively represent the "actual" in-situ measurements, giving the following for $s_u(z)$:

$$s_u(z) = a + (b + M)\ [t(z) + u(z) + e(z)] \tag{2}$$

in which a and b represent the transformation model means for the intercept and slope, t represents the mean of the in-situ parameter trend function, and M, u, and e are homogeneous random variables with normal distribution (for convenience) defined by a variance and zero mean. Incorporating random variables makes the mathematical manipulations a bit cumbersome, but these procedures are well-established (e.g., DeGroot, 1986; Filippas, et al., 1988).

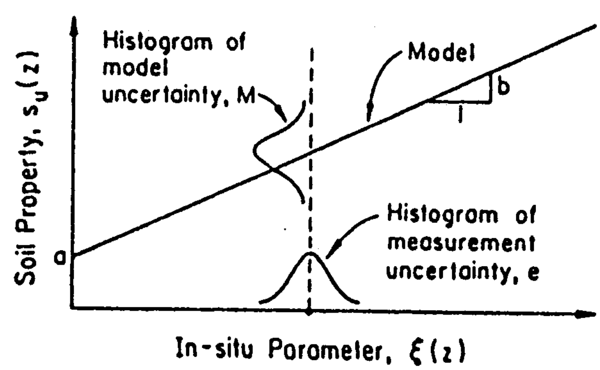


Figure 8. Characteristics of Transformation Model (Kulhawy and Grigoriu, 1987)

There would be analogous forms of Figure 8 and Equation 2 for other in-situ tests or desired properties. For laboratory soil property testing, these general forms also would be appropriate. The in-situ measurement uncertainty would be replaced by uncertainties resulting from sampling, disturbance, trimming, equipment variables, etc. The model uncertainty would be zero (with a = 0 and b = 1) if the test conditions are the same as those desired. However, if test conditions are to be different, such as if plane strain behavior is to be predicted from triaxial tests, then the model uncertainty again must be included.

EVALUATION OF SOIL PERFORMANCE PROPERTY TERMS

Equation 2 and Figure 8 provide a general basis for evaluating the soil performance properties. Although the concepts portrayed are simple, research has not quantified all of the terms well. In fact, it might even be argued that research perhaps has only clouded the issues for the practitioner who does not have ready access to all of the current research data. In this section, an evaluation of the terms in Equation 2 will be presented on a selected basis. A complete evaluation for all test types, soil properties, etc. is an enormous task worthy of a text, and it is well beyond the scope of this paper.

Trend Function. Trend functions of many types have been addressed in the geotechnical literature. Figure 9 illustrates several ways that the trend in a soil parameter can be evaluated. The actual parameter variation is shown in Figure 9a, indicating a common type of trend with depth, while three normalizing procedures are shown in Figures 9b, 9c, and 9d. The first represents normalizing by subtracting the trend function directly, the second normalizes the parameter by the effective overburden stress, and the third incorporates the effective overburden stress and some function of the overconsolidation ratio, OCR. The third procedure is potentially more useful because the normalizing includes unit weight variations, water table location, and stress history, all of which influence any in-situ performance parameter.

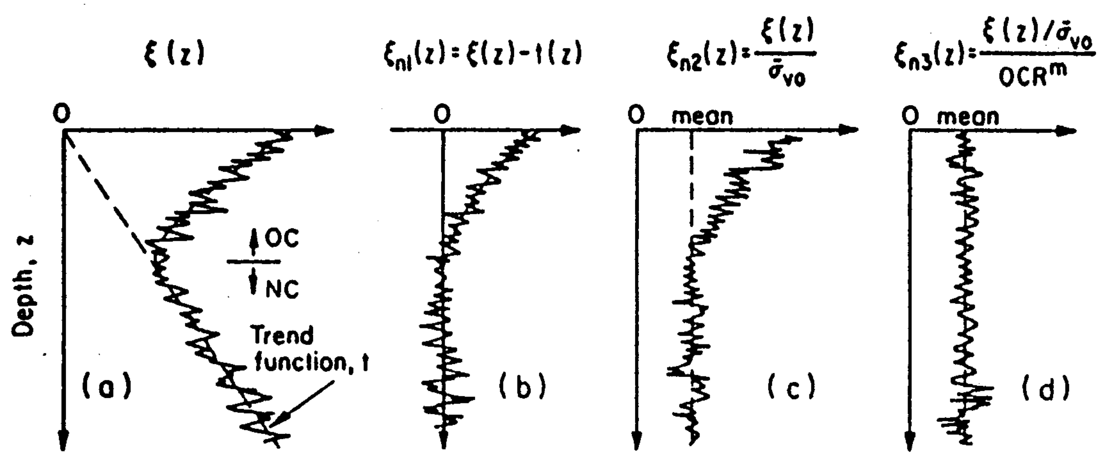


Figure 9. Variation of In-Situ Parameter With Trend Functions

One of the better known relationships of this type is given below:

$$(s_u/\bar{\sigma}_{vo})_{OC} = (s_u/\bar{\sigma}_{vo})_{NC}\ OCR^m \qquad (3)$$

in which s_u = undrained shear strength, OCR = overconsolidation ratio, $\bar{\sigma}_{vo}$ = effective overburden stress, OC = overconsolidated, NC = normally consolidated and m = exponent, typically on the order of 0.8. This relationship is based on both theory and extensive experimental evidence (e.g., Jamiolkowski, et al., 1985; Wroth and Houlsby, 1985). These traits are necessary for a general trend function that is based on sound first principles. Purely empirical correlations are discussed later. An examination of the overview references cited at the beginning of this paper will give many other relationships describing trend functions with depth or stress for other material properties. Some are well-developed; some are not.

<u>Inherent Soil Variability</u>. A number of studies have focused on the inherent soil variability in an effort to evaluate the coefficient of variation, COV (standard deviation/mean), of soil properties. These property studies have been highly variable, ranging from as few as 5 samples in some cases to as many as 790 in others. However, most studies generally included more than 20 samples. Table 4 summarizes the COV data available, disregarding several obvious outliers. As can be seen, the index properties have a relatively low COV. For comparison, the COV for the compressive strength of concrete and the tensile strength of steel is on the order of 6% (Harr, 1977).

Table 4. Coefficient of Variation (COV) for Available Data

	Property	No. Studies	Mean COV w/o Outliers (%)
Index	- natural water content, w_n	18	17.7
	- liquid limit, w_L	28	11.3
	- plastic limit, w_p	27	11.3
	- initial void ratio, e_i	14	19.8
	- unit weight, γ	12	7.1
Performance	- effective stress friction angle, $\bar{\phi}$	20	12.6
	- tangent of $\bar{\phi}$	7	11.3
	- undrained shear strength, s_u	38	33.8
	- compression index, C_c	8	37.0

(Kulhawy, et al., 1991)

For the performance properties, the COV for the effective stress friction angle is rather low and within the index property range. However, the COV for the undrained shear strength and the compression index are quite high. Higher COV parameters generally will warrant a higher factor of safety in analysis and design, because there is more uncertainty in the properties.

<u>Measurement Errors</u>. Measurement errors can be introduced by equipment, procedural/operator, and random test effects. Orchant, et al. (1988) conducted a detailed examination of the available comparative studies for seven of the relatively common in-situ test methods. The results of this study are shown in Table 5, giving the COV for each test effect, the total COV, and the likely range in the COV. This table suggests that, all other factors being equal, standard penetration test measurements are likely to be less reliable than the other test results because there is more inherent uncertainty in the test itself.

<u>Transformation Model</u>. Transformation model development has taken many different forms depending on the type of measurement and the correlated property. For example, the following expression is used to

Table 5. Estimates of In-Situ Test Variability

Test[a]	COV[b] Equipment	COV (%) Procedure	COV (%) Random	COV[c] (%) Total	COV[d] (%) Range
SPT	5[e] to 75[f]	5[e] to 75[f]	12 to 15	14[e] to 100[f]	15 to 45
MCPT	5	10[g] to 15[h]	10[g] to 15[h]	15[g] to 22[h]	15 to 25
ECPT	3	5	5[g] to 10[h]	7[g] to 12[h]	5 to 15
VST	5	8	10	14	10 to 20
DMT	5	5	8	11	5 to 15
PMT	5	12	10	16	10 to 20[i]
SBPMT	8	15	8	19	15 to 25[i]

a - See Figure 2 for test notation (M = mechanical, E = electrical, SB = self-boring)
b - COV = standard deviation/mean
c - COV(Total) = $[\text{COV(Equipment)}^2 + \text{COV(Procedure)}^2 + \text{COV(Random)}^2]^{1/2}$
d - Because of limited data and judgment involved in estimating COV, ranges represent probable magnitudes of test measurement error
e - Best case scenario for SPT test conditions
f - Worst case scenario for SPT test conditions
g - Tip resistance CPT measurements
h - Side resistance CPT measurements
i - Results may differ for p_o, p_f, and p_L, but data are insufficient to clarify this issue
(Orchant, et al., 1988)

estimate the undrained shear strength, s_u, from the corrected cone tip resistance, q_T:

$$s_u/\bar{\sigma}_{vo} = D\ (q_T - \sigma_{vo})/\bar{\sigma}_{vo} \tag{4}$$

in which $\bar{\sigma}_{vo}$ and σ_{vo} = effective and total overburden stresses and D = model slope (equivalent to b in Figure 8) for an intercept a = 0. D is more commonly expressed as $1/N_K$, in which N_K = cone factor. Many analogous equations are given in the literature for other soil properties and other in-situ tests.

The cone factor (or model slope) can be determined three ways: (a) theory, (b) empirical data fitting, or (c) probabilistic modeling. Numerous theories have been proposed for evaluating N_K, including bearing capacity, cavity expansion, steady penetration, and finite element formulations. From these theories, values of N_K from about 5 to about 20 can be calculated. There is no general agreement on the "correct" theory to date, but there is some strong support for the cavity expansion theory proposed by Vesić (1977).

Empirical data fitting also has been used extensively, in which the normalized s_u is plotted versus the normalized q_T. The interpreted fit then is designated as D or $1/N_K$. Using this procedure, N_K values have been reported as low as 4.5 and as high as 75. The majority of cases report N_K from 10 to 30. This large spread is to be expected because of the nature of empirical fitting. The normalized q_T is plotted versus the normalized s_u, assuming that both are deterministic values. Then the slope is evaluated, normally assuming a zero intercept. This evaluation could be done by straight regression techniques, by weighted regression, in which some data are considered to be more "accurate", or just by subjective "eyeballing", in which some conservative interpretation is likely to enter. By adopting this type of approach, the inherent soil variability, measurement errors, and model uncertainty are largely disregarded, at least quantitatively. The resulting N_K value is adopted and, more often than not, the spread of the data is forgotten. This spread should always be noted with any data interpretation, as noted in Figure 10 in a rather direct manner. A further problem with most empirical plots is the mixing of different quality data, cone types, methods of obtaining q_T, and tests to measure s_u. All of these should and will give different results, and therefore they should not be compared directly.

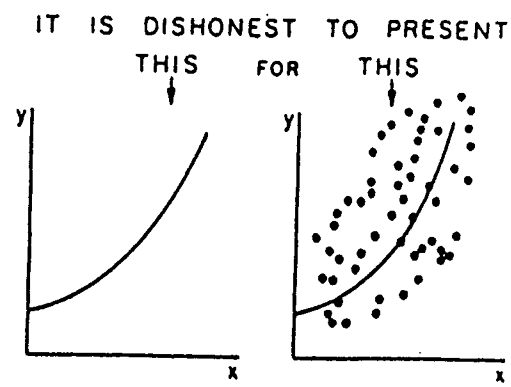


Figure 10. Importance of Proper Data Presentation (Moroney, 1956)

Probabilistic modeling also can be done, and it should be the preferred approach when a generally accepted theory is not available. It is also the only way to truly "prove" a theory. In this approach, a substantial data base is required of relatively high quality data for both the in-situ parameter, $(q_T - \sigma_{vo})/\bar{\sigma}_{vo}$, and the soil property, $s_u/\bar{\sigma}_{vo}$. For these data, statistically significant means for both the parameter and the property then are obtained, from which D (and therefore N_K) is determined, along with its statistical uncertainty.

In a comprehensive study of this type (Kulhawy, et al., 1992), it was found that D = 0.0789 (N_K = 12.7) and COV = 35.0% using the CIUC triaxial test to evaluate s_u. For the field vane shear test to evaluate s_u, D = 0.0906 (N_K = 11.0) and COV = 40.4%. These values for N_K are consistent with cavity expansion theory (Vesić, 1977), and the COV values are consistent with those that should be expected for s_u correlations.

Available Correlations. A large number of correlations are available in the literature. Tables 6 and 7 summarize the availability of correlations for a range of soil properties developed from a theory, laboratory test measurement, or field test measurement. Only a few of the test correlations incorporate the rigor of Figure 8 and Equation 2. The majority are empirical. As such, considerable uncertainty is to be expected, at least as large as that indicated in Tables 4 and 5.

A fair degree of caution should always be exercised when using any empirical correlation because two (or more) items are being linked together that are not directly related. Consider, for instance, the SPT N-value in Table 6. The N-value is the dynamic driving resistance of a particular size sampler, yet it has been correlated with the soil consistency, vertical and horizontal stress state, strength, and modulus. While these characteristics undoubtedly influence the N-value indirectly, it is too much to expect that they can be predicted "accurately".

One must also be cautious of the "age" of an empirical correlation. Generally speaking, these correlations change with time as understanding of the correlated terms improves. Young or immature correlations see rapid and sometimes dramatic changes, while more mature correlations might only see modest adjustments in the pertinent coefficients. For example, an empirical relationship was suggested in 1948 indicating a direct dependence of the relative density, D_r, on the N-value, as follows (Terzaghi and Peck, 1948):

$$D_r = f\ (N) \tag{5}$$

Through subsequent years of research efforts by many, the D_r-N relationship now takes the form (Kulhawy and Mayne, 1990, 1991):

$$D_r = f\ (N,\ C_{ER},\ C_B,\ C_S,\ C_R,\ C_N,\ C_P,\ C_A,\ C_{OCR}) \tag{6}$$

in which the corrections are as follows: energy ratio (C_{ER}), borehole diameter (C_B), sampling method (C_S), rod length (C_R), overburden stress (C_N), particle size (C_P), aging (C_A), and overconsolidation (C_{OCR}). Subsequent changes are likely to result only in modest adjustments to one or more of the correction coefficients. Use of Equation 5 at this point in time would be inappropriate, except for use in historical perspectives.

Importance of Standardization. As noted previously, part of the problem with empirical relationships is the mixing of different quality data, variations of a particular in-situ test type, and different tests to measure the reference soil property in the laboratory. Data quality can be screened, and only data of a minimum standard should be used in empirical correlations. With ASTM or other standardizations, the in-situ test equipment variations will be minimal. For example, SPT samplers should be of the same geometry, and dilatometer blades are all of the same type. Unfortunately, there is still a fair amount of variation with cone types.

With the laboratory reference test, Wroth (1984) and others have long cried out for the isotropically-consolidated triaxial compression test to be the basic standard. This test is logical because it approximately replicates the soil structure and stress states in the field with a minimum of disturbance, and it is a test that can be con-

Table 6. Available Correlations for Cohesive Soils

Property Category	Soil Property	Lab or Theory Correlation	Field Test Correlation[a]					
			SPT	CPT	CPTU	PMT	DMT	VST
Basic Characterization	Classification	X	-	X	X	-	X	-
	Unit weight, γ	X	-	-	-	-	-	-
	Consistency	-	X	X	-	-	-	-
In-Situ Stress	Preconsolidation stress, $\bar{\sigma}_p$	X	X	X	X	X	X	X
	Overconsolidation ratio, OCR	X	X	-	X	-	X	X
	Coef. of horizontal soil stress, K_o	X	X	X	X	X	X	-
Strength	Effective stress friction angle, $\bar{\phi}$	X	-	-	-	-	-	-
	Undrained shear strength, s_u	X	X	X	X	X	X	X
Deformability	Poisson's ratio, ν	X	-	-	-	-	-	-
	Young's modulus, E	X	-	-	-	X	-	-
	Compression indices, $C_c + C_{ur}$	X	-	-	-	-	-	-
	Constrained modulus, M	X	X	X	-	-	X	-
	Coef. of consolidation, c_v	X	-	-	X	-	X	-
	Coef. of secondary comp., C_α	X	-	-	-	-	-	-
Permeability	Hydraulic conductivity, k	X	-	-	-	-	-	-

a - See Figure 2 for test notation (CPTU = piezocone)
(from Kulhawy and Mayne, 1990)

Table 7. Available Correlations for Cohesionless Soils

Property Category	Soil Property	Lab or Theory Correlation	Field Test Correlation[a]				
			SPT	CPT	CPTU	PMT	DMT
Basic Characterization	Classification	X	-	X	X	-	X
	Unit weight, γ	X	-	-	-	-	-
	Relative density, D_r	-	X	X	X	-	X
In-Situ Stress	Coef. of horizontal soil stress, K_o	X	-	X	-	X	X
Strength	Effective stress friction angle, $\bar{\phi}$	X	X	X	-	X	X
Deformability	Poisson's ratio, ν	X	-	-	-	-	-
	Young's modulus, E	X	X	-	-	X	X
	Constrained modulus, M	X	-	X	-	-	-
	Compression index, C_c	X	-	-	-	-	-
Permeability	Hydraulic conductivity, k	X	-	-	-	-	-
Liquefaction	Cyclic stress ratio, $\tau_{av}/\bar{\sigma}_{vo}$	-	X	X	-	-	X

a - See Figure 2 for test notation (CPTU - piezocone)
(from Kulhawy and Mayne, 1990)

ducted using standard equipment, in a straightforward manner, in virtually any soils laboratory. For drained tests, correlations then would be referenced to the effective stress friction angle, $\bar{\phi}_{tc}$. For undrained tests, the reference would be the normalized undrained strength ratio, $(s_u/\bar{\sigma}_{vo})_{CIUC}$.

Table 8 illustrates the mean relative values of the friction angles among the major test types. As can be seen, the triaxial compression value is, almost always, the minimum $\bar{\phi}$, and therefore it will represent the conservative value. For all practical purposes, there is no difference in $\bar{\phi}_{tc}$ for either isotropic or anisotropic consolidation.

Table 8. Mean Relative Values of Effective Stress Friction Angles for Cohesionless Soils

Test Type	Friction Angle
Triaxial compression (TC)	$1.0\ \bar{\phi}_{tc}$
Triaxial extension (TE)	$1.12\ \bar{\phi}_{tc}$
Plane strain compression (PSC)	$1.12\ \bar{\phi}_{tc}$
Plane strain extension (PSE)	1.12 (for PSC/TC) x 1.12 (for TE/TC) $= 1.25\ \bar{\phi}_{tc}$
Direct Shear (DS)	$\tan^{-1}[\tan\bar{\phi}_{psc}\cos\bar{\phi}_{cv}]^a$ or $\tan^{-1}[\tan(1.12\ \bar{\phi}_{tc})\cos\bar{\phi}_{cv}]$

a - $\bar{\phi}_{cv}$ refers to the fully-softened or critical void ratio state (Kulhawy and Mayne, 1990)

Figure 11 illustrates the mean normalized undrained strength ratios for the major laboratory tests. For this figure, the reference strength ratio is given by the modified Cam clay model as follows (e.g., Wroth and Houlsby, 1985):

$$(s_u/\bar{\sigma}_{vo})_{CIUC} = 0.5\ \bar{M}\ (0.5)^{\Lambda} \qquad (7)$$

in which $\bar{M} = 6\sin\bar{\phi}_{tc}/(3 - \sin\bar{\phi}_{tc})$ and Λ = critical state parameter = 0.72 for compression (Kulhawy and Mayne, 1990). This relationship is applicable for relatively unstructured soils. For sensitive, cemented, and other structured fine-grained soils, Equation 7 tends to be a lower bound. As can be seen, the reference undrained strength ratio normally is the maximum value, and therefore it will represent the unconservative value. For undrained loading, considerable attention must be paid to the appropriate test simulation.

SOIL PROPERTY PREDICTION

The previous sections of this paper prescribed a general approach for property evaluation, from which it is clear that there is a dynamic and evolving process that is leading to a robust design methodol-

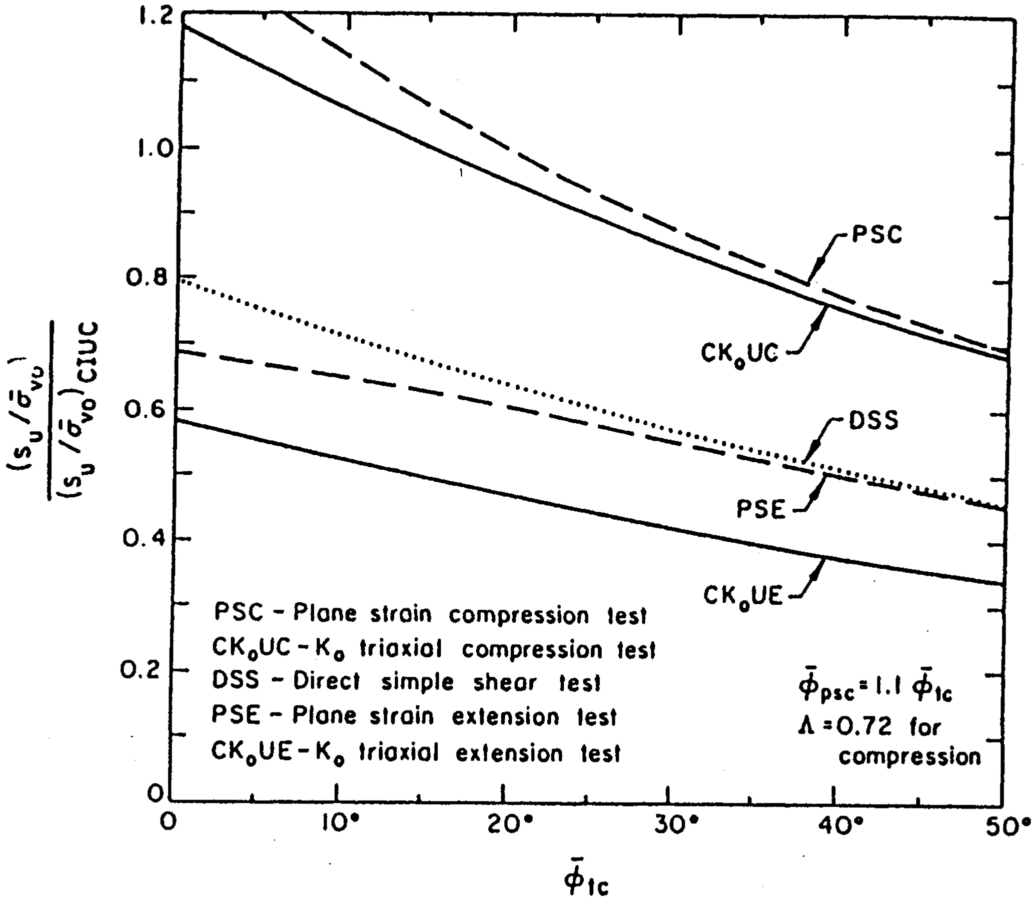


Figure 11. Mean Normalized Undrained Strength Ratios for Major Laboratory Tests (Kulhawy and Mayne, 1990)

ogy. Pieces of the approach are well-established, others are still in the research arena, and a truly useful, generalized, reliability-based design model is yet to be developed. Until all of these pieces are in place, reliance must be placed on our traditional approaches that have resulted from a curious blend of theory, laboratory or field measurement, observation, experience, precedent, prescience, and the wisdom of our predecessors. Some who do not understand this process have called it the geotechnical "crystal ball" or "geo-mysticism".

Figure 12 outlines the processes for soil property selection in a rational, up-to-date, nine-step procedure. Step 1 involves developing a sound perspective of geotechnical tradition on which to discuss the issue of predictions. Key references in this regard are the writings of Casagrande (1965), Lambe (1973), and Peck (Dunnicliff and Deere, 1984). Step 2 focuses on developing information related to precedents and prior knowledge. A summary of experience on the same class of problem is a valuable reference. Step 3 includes reviewing the Step 1 and 2 information and making an initial first-order "guesstimate" of the properties involved. Factors outlined in Figure 6 should play an influential role.

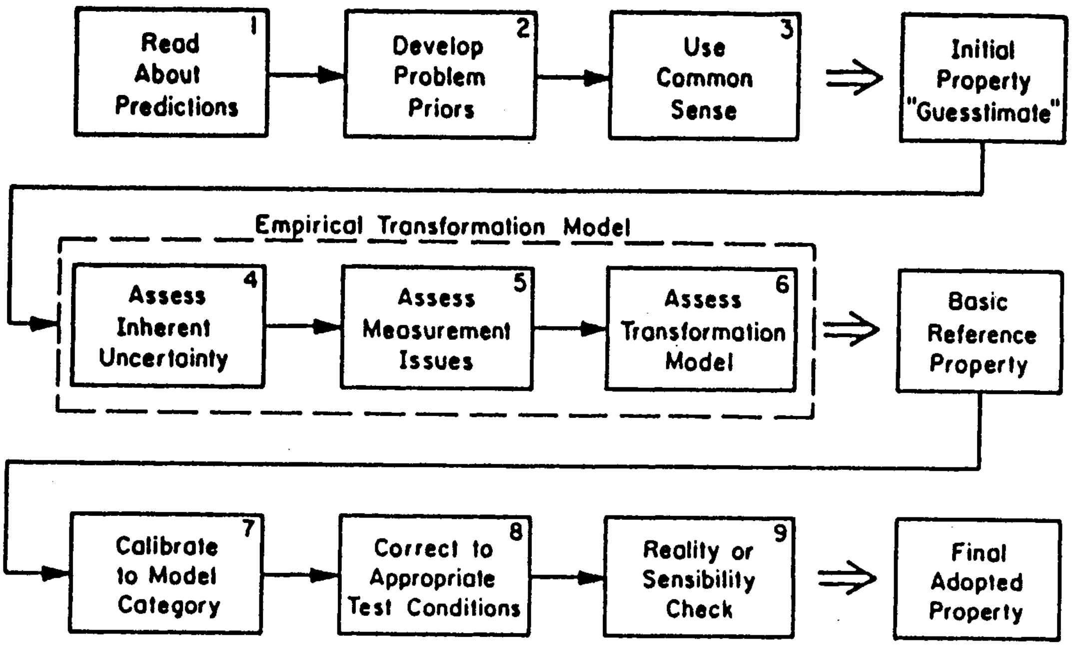


Figure 12. "Modernized" Flow Chart for Property Selection

Steps 4 through 6 have been described previously and result in a quantitative assessment of a basic material property, such as a strength related to the triaxial compression test. These steps together could be described by an empirical transformation model, which likely would have somewhat larger uncertainty because of several factors being lumped together. The result is a basic reference property, normalized to a standard test, such as strength being normalized to the triaxial compression test.

Step 7 is calibration to a particular category of model, attempting to take into account the level of complexity or simplicity involved. Step 8 addresses corrections to appropriate test conditions, as illustrated in Figure 3 and quantified in Table 8 and Figure 11. Step 9 is a check to ensure that the property, as initially estimated and subsequently modified, is still within the property range to be expected. If all checks are passed, the final adopted property is to be used.

SUMMARY

Soil performance properties are highly variable entities that must be evaluated specifically within a particular design context. Criteria are given for property evaluation, and a general methodology is presented to evaluate properties in a consistent manner. The approach builds on current practice and purports to achieve a more rational, reliability-based procedure in the future.

ACKNOWLEDGMENTS

The concepts expressed were developed largely in studies of site investigation and foundation reliability for the Electric Power Research Institute, Palo Alto, CA. V.J. Longo was the EPRI Project Manager. H.E. Stewart and M.D. Grigoriu provided valuable review comments. L. Mayes prepared the text, and A. Avcisoy prepared the figures.

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PREDICTIVE SOIL MECHANICS (WROTH MEMORIAL SYMP.), ED.
G.T. HOULSBY AND A.N. SCHOFIELD, THOMAS TELFORD, LONDON, 1993

Some thoughts on the evaluation of undrained shear strength for design

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Initial observations are made on the rationale for total stress analyses and the use of the undrained shear strength, s_u. It is shown that different s_u values are appropriate for different field loading conditions and that there are many uncertainties in s_u as a material property. Another call is made to adopt the CIUC test as the standard test of reference for evaluating s_u. Results of comprehensive studies are presented that show the relative comparisons among the CIUC and other major test types. Finally, illustrative comparisons are presented to show the relative undrained strength ratios for some different field loading conditions.

Introduction

Evaluation of the undrained shear strength of fine-grained soils is no trivial task. This fact has long been known within the research community that focuses on soil properties, but it is only slowly being realized in many other sectors of practice. During the past fifteen years, a number of major overview papers have been written that have traced the steady accumulation of knowledge on soil properties (e.g. Ladd et al., 1977; Wroth, 1984; Wroth and Houlsby, 1985; Jamiolkowski et al., 1985; Jamiolkowski et al., 1991), and a major design manual on estimating soil properties also has been prepared (Kulhawy and Mayne, 1990). These documents have shown an increasing sophistication in all types of property evaluation, specifically including a careful matching of test and prototype variables and a direct awareness of variability (or uncertainty) in the property evaluation process.

Peter Wroth was a major player in the development of this knowledge, and he was a strong proponent of modelling test and prototype conditions as realistically as possible and of developing minimum standards of reference in testing. In the many discussions (or mini-debates?) that we had on these subjects, Peter always impressed upon me the need to focus on these basic issues. Continuing in the spirit of these discussions, this paper focuses on some key issues in evaluating the undrained shear strength of fine-grained soils for design purposes. For simplicity, the soil is assumed to be saturated and relatively

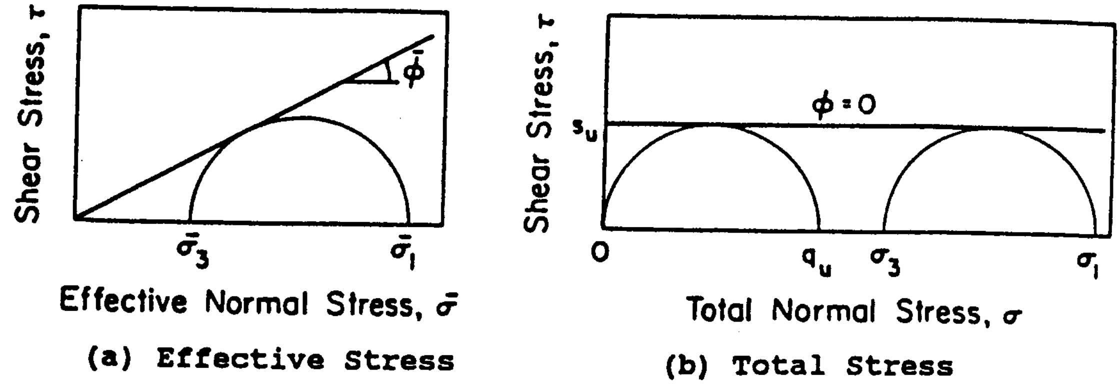

Fig. 1. Idealized Coulomb–Mohr failure envelopes

unstructured. Special behavioural issues dealing with sensitive, cemented, and other structured soils are beyond the scope of this paper.

Basic characterization

In geotechnical engineering analyses involving fine-grained soils, either effective stress or total stress methods can be used. Total stress methods normally are adopted because of (implied) simplicity. However, the failure of all soils actually occurs on the effective stress failure envelope shown in Fig. 1(*a*). Loading generates excess pore water stresses (Δu) that change the original effective stresses and, in turn, influence the stress state relative to the envelope defined by the effective stress friction angle ($\bar{\phi}$). Since the total stress loading path and the developed excess pore water stresses (Δu) may not be known with confidence, a total stress analysis with $\phi = 0$ and s_u = undrained shear strength, as shown in Fig. 1(*b*), provides a simple and idealized analysis alternative. However, it must be remembered that s_u incorporates both $\bar{\phi}$ and Δu, and it varies with the initial or in-situ effective stress level.

The undrained shear strength may very well be the most widely used parameter for characterizing fine-grained soils. In some circles, it is even portrayed as a fundamental material property, which it isn't. Instead, it is a measured soil response during undrained loading that assumes zero volume change. As such, s_u is affected by the mode of testing, boundary

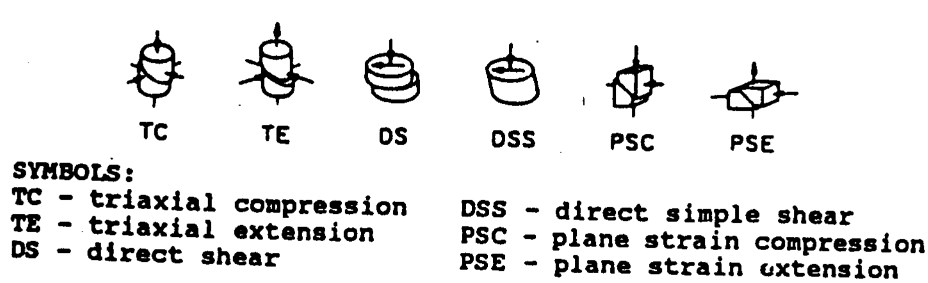

Fig. 2. Common laboratory strength tests

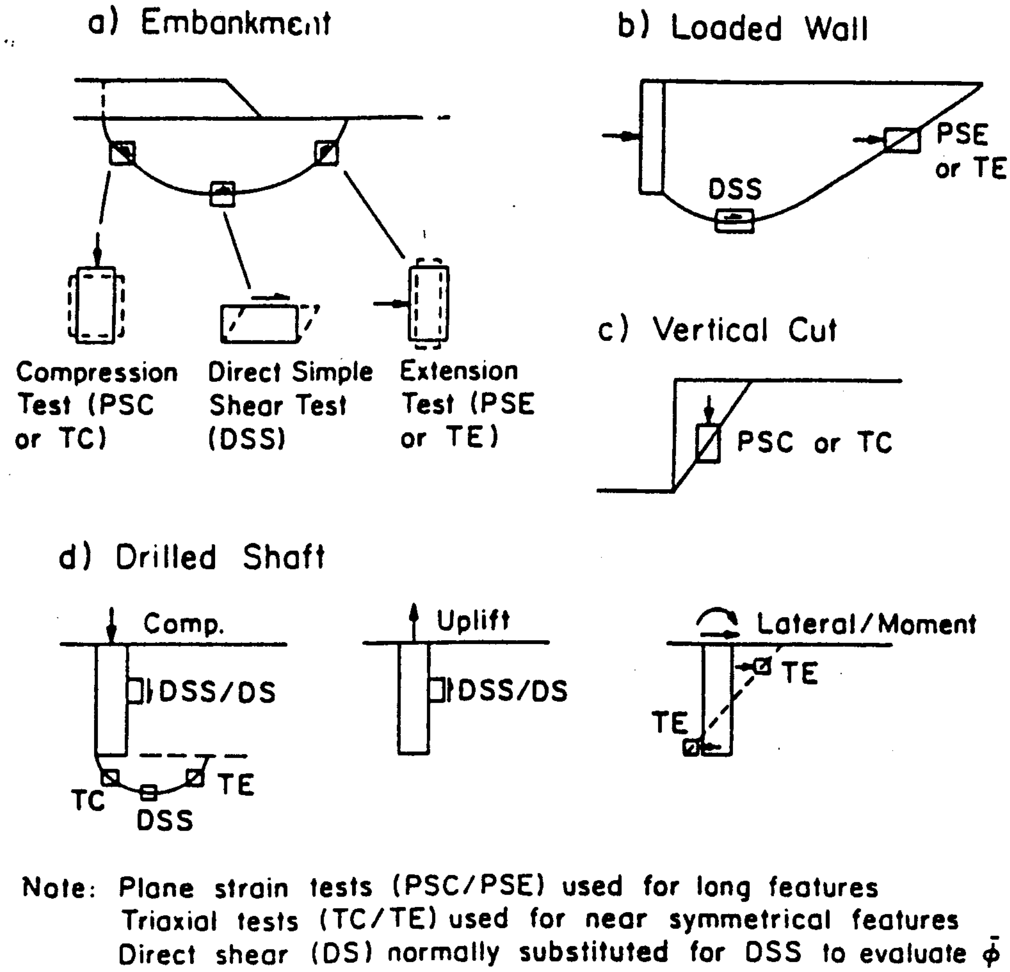

Fig. 3. Relevance of laboratory strength tests to field conditions

conditions, rate of loading, initial stress level, and other variables. Consequently, s_u is and should be different for different test types. Consider the test representations depicted in Fig. 2. All could be used to estimate a soil strength, but the results should be different. The overview references cited previously cover these issues very well.

These same points carry over into the field, where different boundary conditions, stress paths, etc. also will apply. Figure 3 illustrates a few common cases related to embankments, walls, slopes, and drilled shaft foundations. It is clear from this figure that no one type of test usually addresses the actual field conditions. Instead, a combination commonly is warranted.

The issues raised above are not new, and they do not represent an academic exercise. They are real, and they have important implications in practice because, if the 'wrong' test is used to characterize a particular field situation, there could be significant implications on the actual factor of safety in contrast with the perceived value.

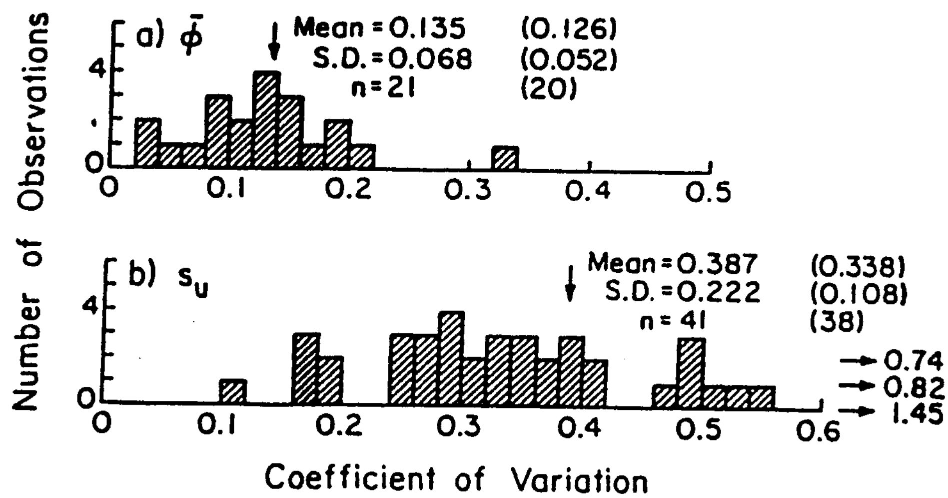

Fig. 4. Histograms of soil strength parameters (Kulhawy et al., 1991)

Uncertainties in strength parameters

Uncertainties are always introduced in any evaluation process. Some represent the inherent material or property variability, some represent measurement errors, and some represent modelling inadequacies or inaccuracies. A complete evaluation of these issues is well beyond the scope of this modest paper and our general state of knowledge at this time. However, a general framework for assessing these uncertainties is given by Kulhawy (1992).

As a first-order assessment of uncertainty, one can evaluate the coefficient of variation, COV (standard deviation/mean), of soil strength properties. Figure 4 summarizes data for both $\bar{\phi}$ and s_u, as reported in the literature. The databases for these studies have been highly variable, ranging from as few as 5 samples in some cases to as many as 295 in others. For $\bar{\phi}$, the range was 5 to 81; for s_u, the range was 10 to 295. A further complicating factor is the lack of control of data in any literature survey. Undoubtedly, there is mixing of test types and testing procedures in these data, so the summary in Fig. 4 is likely to be a bit on the high side in addressing the variability. For each parameter, the mean, standard deviation (S.D.), and number of samples (n) are given. The parenthesized values represent the mean, S.D., and n without the several high values that appear to be outside of the main populations. For comparison, the COV for the compressive strength of concrete and the tensile strength of steel is about 6% (Harr, 1977).

As can be seen, the COV for $\bar{\phi}$ is relatively low and is about double that for concrete or steel. However, the COV for s_u is quite large, necessarily indicating more uncertainty in the property. This significant difference does not necessarily imply that there is more uncertainty in total stress analyses. Consider, for instance, the geotechnical prediction model shown in Fig. 5. To make a prediction from a given load, the

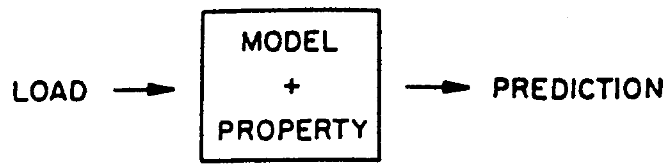

Fig. 5. Components of geotechnical prediction

model and property must be considered or calibrated together. For total stress analyses, the models may be simple, with little uncertainty, but the properties have significant uncertainty. The reverse tends to be true with undrained effective stress analyses. Therefore, with total stress analyses, it is much more important that the properties replicate the prototype conditions.

Importance of a standard 'test of reference'

As can be seen in Fig. 3, quite a number of different types of tests and equipment might be needed for a particular design condition. However, this level of testing is likely to be excessive in common and routine design cases. Therefore, it is both appropriate and convenient to establish a standard 'test of reference' that would be applicable in some design cases and would be simple and expedient from a commercial testing standpoint. The test that was recommended by Wroth (1984) and others is the isotropically consolidated, triaxial compression test for undrained loading (CIUC). This test is logical for high-quality field samples because it satisfies the above criteria, re-establishes a state of stress in the soil that is approximately consistent with the overburden stress, minimizes the sampling disturbance effects, and includes a reconsolidation phase.

It should be noted that most soils in situ actually will be consolidated anisotropically. This difference in consolidation stresses has no appreciable influence on $\bar{\phi}_{tc}$, the effective stress friction angle in triaxial compression (e.g. Kulhawy and Mayne, 1990), but it does influence s_u, as will be shown shortly.

There also are simpler forms of triaxial test that are available, such as the unconsolidated, undrained (UU) triaxial and unconfined compression (UC) tests. However, many detailed studies (e.g. Ladd et al., 1977; Tavenas and Leroueil, 1987) have shown that the UU and UC tests often are in gross error because of sampling disturbance effects, incorrect initial shear stress level, and omission of a reconsolidation phase. Based on studies such as these, the CIUC test also should be considered to be the minimum quality laboratory test for evaluating s_u.

With the CIUC test as the standard reference, the results of all other tests can be compared simply and conveniently. Since s_u is stress-dependent, its value typically is normalized by the vertical effective overburden stress ($\bar{\sigma}_{vo}$) at the depth where s_u is evaluated. The result is the undrained strength ratio $(s_u/\bar{\sigma}_{vo})_{CIUC}$.

Based on an evaluation of analytical expressions and a detailed comparison of available undrained strength data for the major test types, Kulhawy and Mayne (1990) developed the mean normalized undrained strength ratios shown in Fig. 6. For this figure, the reference strength ratio is given, to sufficient accuracy (S.D. $\approx$ 0.05), by the modified Cam clay model as follows (e.g. Wroth and Houlsby, 1985):

$$(s_u/\bar{\sigma}_{vo})_{CIUC} = 0.5\,M\,(0.5)^{\Lambda} \tag{1}$$

in which $M = 6 \sin \bar{\phi}_{tc}/(3 - \sin \bar{\phi}_{tc})$ and Λ = critical state parameter. This relationship works rather well for relatively unstructured soils. For sensitive, cemented, and other structured fine-grained soils, eqn. (1) tends to be a lower bound. Figure 6 represents an illustrative comparison of a very extensive set of databases, presented here for the specific

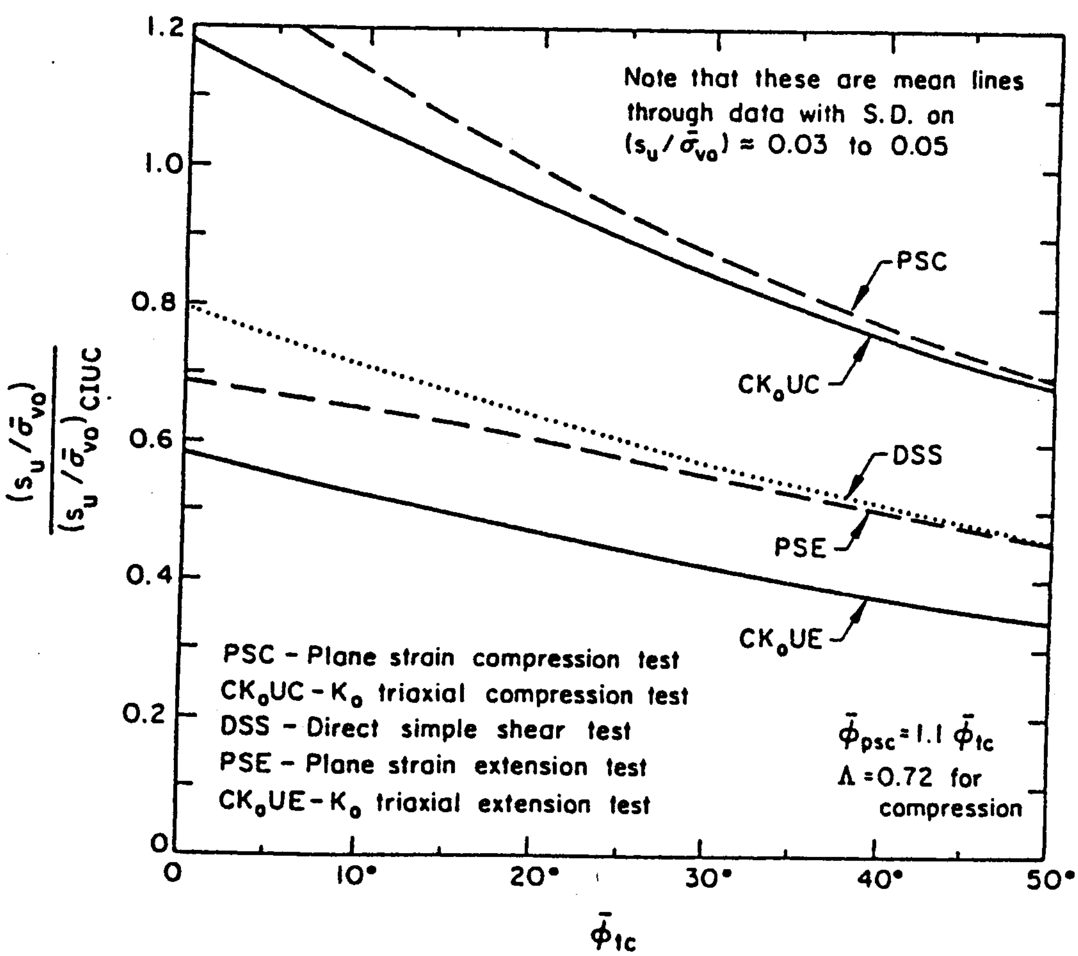

Fig. 6. Mean normalized undrained strength ratios for major laboratory tests (Kulhawy and Mayne, 1990)

cases of $\bar{\phi}_{psc}$ and Λ shown. The detailed comparisons are given by Kulhawy and Mayne (1990), along with appropriate modifications for testing rate, overconsolidation, and other test specifics.

As can be seen in Fig. 6 on a relative basis, considerable variation occurs among the test types. For the typical range of $\bar{\phi}_{tc}$ from 20° to 40°, the CIUC value will always be greater than all of the others, and therefore it may be unconservative to use directly. In addition, the other test values in compression are roughly double those in extension.

Although not recommended for any future work because of the problems cited previously, the UU test is the only strength documentation for many sites evaluated in the past. For this reason, it is of interest to examine the interrelationships between the UU and CIUC tests, as shown in Fig. 7. In this figure, the data were grouped by overconsolidation ratio, OCR, as follows: normally consolidated, NC ($1.0 < OCR < 1.3$); lightly overconsolidated, LOC ($1.3 < OCR < 3.0$); moderately overconsolidated, MOC ($3 < OCR < 10$); and heavily overconsolidated, HOC ($OCR > 10$).

Figure 7 represents reasonably homogeneous soil deposits and 'well-conditioned' data. As such, it may be interpreted as near the upper bound in quality (i.e. minimum S.D.) in the interrelationships. Following this preamble, it is clear that there is a well-defined relationship between the UU and CIUC s_u values. In the NC range, UU values may only be ½ the CIUC values. However, for the HOC range, UU values can exceed the CIUC values. This general behaviour can be predicted by

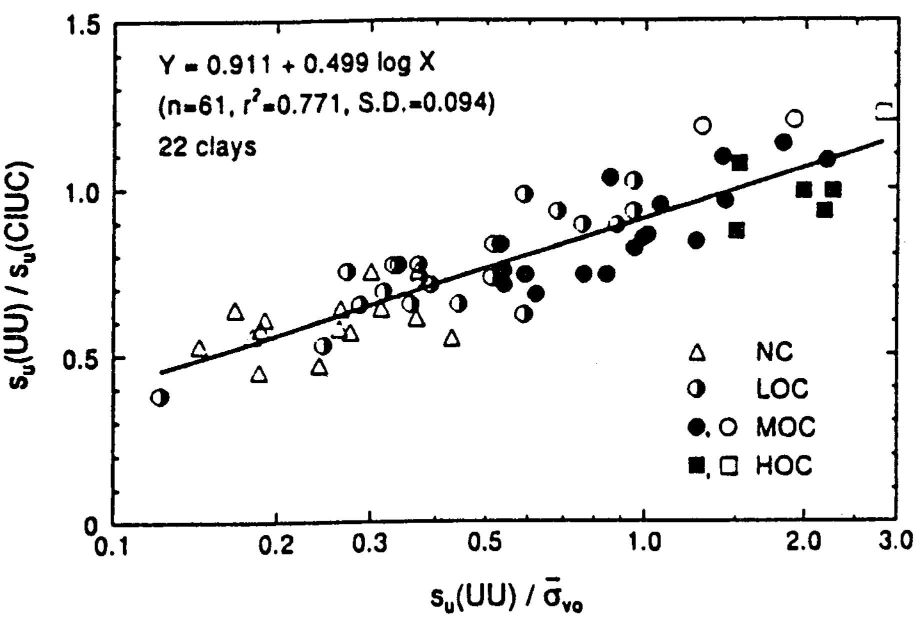

Fig. 7. Comparison of undrained strengths from UU and CIUC Tests (Chen and Kulhawy, 1993)

simple stress path evaluation that takes into account the effects of sampling and sample disturbance (e.g. Lambe and Whitman, 1969).

Comparison of s_u for different field loading conditions

Figure 3 portrayed the applicable undrained strength tests for a variety of field loading conditions, and Fig. 6 demonstrated the relationships between specific laboratory strength tests and the CIUC reference test. By integrating these two data sets, a comparison of the field loading conditions can be made with the CIUC test by field case type, as given in Table 1. This table shows that the normalized undrained strength ratio for design is always less than one. Therefore, if the CIUC results are used directly in design, then the results will be unconservative because the actual operative strength is less than the CIUC value.

However, if the analyses are based on UU test results instead of CIUC test results, then the situation changes. For NC soils, $s_u(UU)/s_u(CIUC) \approx 0.6$. Dividing the results in Table 1 by 0.6 would give a value of $(s_u/\bar{\sigma}_{vo})/(s_u/\bar{\sigma}_{vo})_{UU}$ closer to 1.0 for 5 of the 6 loading conditions portrayed, indicating compensating errors leading to an apparently acceptable result, as long as the UU data are representative and reliable.

For rational design, compensating errors of this type should not be relied upon, and lower grade tests simply are not acceptable for modern (informed) practice. Direct recognition of the boundary conditions and their relationship to the CIUC test results should be introduced into the design explicitly.

Summary

Evaluation of the undrained shear strength (s_u) of fine-grained soils should be based on sound geotechnical principles. Ample evidence

Table 1. Illustrative undrained strength ratio comparisons computed for different field loading conditions

Field Loading Condition	$(s_u/\bar{\sigma}_{vo})/(s_u/\bar{\sigma}_{vo})_{CIUC}$		
	$\bar{\phi}_{tc} = 20°$	30°	40°
Long Embankment (PSC + DSS + PSE)	0.75	0.67	0.60
Long Wall (DSS + PSE)	0.62	0.57	0.51
Short Vertical Cut (CK_oUC)	0.94	0.85	0.75
Shaft Bearing Capacity (CK_oUC + DSS + CK_oUE)	0.68	0.62	0.55
Shaft Side Resistance (DSS)	0.64	0.58	0.51
Shaft Lateral Load (CK_oUE)	0.47	0.42	0.38

exists that s_u varies greatly with the test conditions and other factors. A standard 'test of reference' should be adopted for all future work, and this test should be the CIUC triaxial test. Correlations to other site, geometry, or load-specific conditions can be made through simple correlations developed from extensive research studies. Prior usage of the UU test gave compensating errors that nearly 'corrected' the strength results adequately. However, the UU results are subject to many vagaries, and they cannot be depended on. The CIUC should be the minimum quality of test.

Acknowledgements

The concepts expressed were developed largely during geotechnical studies for the Electric Power Research Institute, Palo Alto, California. V.J. Longo was the EPRI Project Manager. Many discussions with H.E. Stewart and P.W. Mayne crystallized the concepts, and H.E. Stewart provided many useful review comments on this paper. L. Mayes prepared the text, and A. Avcisoy prepared the figures.

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