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SYNOPSIS A simple piezocone model combines spherical cavity expansion theory and modified Cam Clay concepts to represent both the corrected cone tip resistance (q_T) and penetration pore water pressure measured behind the tip (u_{bt}). In closed form, the undrained shear strength (s_u) is shown to be a function of the effective friction angle (ϕ'), the plastic volumetric strain ratio (Λ), and the piezocone parameter (q_T-u_{bt}). Parametric studies show that the model is relatively insensitive to variations in ϕ' and Λ, thereby simplifying its form for practical use. The method is applied to results from laboratory calibration chamber tests on kaolinitic clay, as well as field data from eight intact clay sites reported in the literature. In addition to in-situ PCPT records, these clay deposits have known profiles of s_u evaluated from laboratory isotropically and anisotropically-consolidated undrained triaxial compression tests.

INTRODUCTION

The use of in-situ tests for determining soil properties has been a major area of interest for the last decade. One of the most popular tools is the piezocone penetration test (PCPT) due to its unique characteristics of continuous profiling, fast operation, and relatively low cost. A conventional piezocone test provides three separate and continuous measurements: cone tip resistance (q_c), sleeve friction (f_s), and pore water pressure (u_m). The standard penetrometer has a 60° apex angle, 10 cm^2 of projected cone area, and 150 cm^2 sleeve. The cone is advanced at a constant rate of 20 mm/sec. The pore water pressure element is often located at one of three locations: (1) cone tip/face (u_t or u_1), (2) immediately behind the cone tip (u_{bt} or u_2), and (3) behind the friction sleeve (u_{bs} or u_3), as shown in Figure 1. Extensive testing by PCPTs has shown that $u_1 > u_2 > u_3$ (LaRochelle, et al. 1988; Mayne, et al. 1990). The Type 2 cone can be considered the standard, since the u_{bt} reading is required for correcting q_c to q_T (Lunne, et al. 1986; Powell et al., 1988).

The interpretation of undrained shear strength (s_u) using PCPT parameters has been investigated by several researchers. Konrad and Law (1987) provide a review of the primary approaches in this regard. The earliest theoretical derivations assumed a perfectly plastic medium in accordance with classical limit plasticity approach. Later, cavity expansion (CE) theories were adopted for determining the cone bearing factor (N_{kt}). Cavity expansion assumes an elasto-plastic medium in either spherical or cylindrical formulations (Vesić, 1972). For CE assessment of PCPT data, s_u may be determined from either the conventional net cone resistance:

$$s_u = (q_T-\sigma_{vo})/N_{kT} \qquad [1]$$

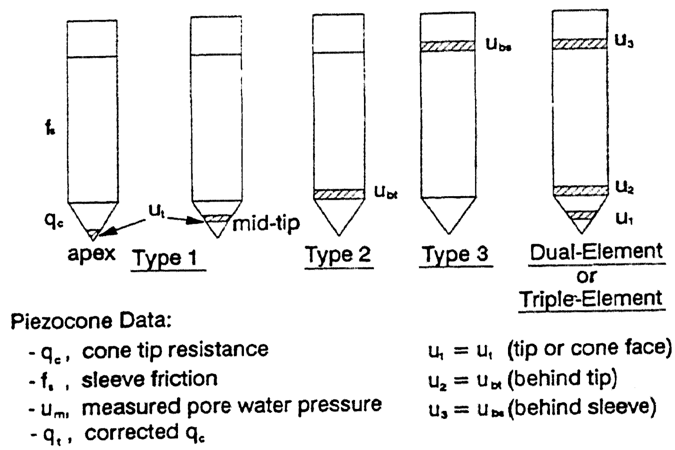

Figure 1. Generalized types of piezocones.

or excess pore water pressure:

$$s_u = \Delta u/N_{\Delta u} \qquad [2]$$

where N_{kT} and $N_{\Delta u}$ are cone bearing factors. Both N_{kT} and $N_{\Delta u}$ are shown to be functions of rigidity index, defined as the ratio of shear modulus to undrained strength (I_r=G/s_u). The determination of I_r requires an extra effort, either in the laboratory or in the field, therefore making this approach somewhat unattractive. For example, Konrad and Law (1987) incorporated spherical cavity expansion theory into an effective frictional model for assessing s_u. In this approach, additional parameters such as soil-steel friction (δ), pore water pressure ratio (α=u_t/u_{bt}), and relevant I_r are required, but not normally available.

In addition to the aforementioned closed-form approaches, numerical methods are also available for determining s_u from PCPT data. Baligh (1986) considered streamlines of soil flow around the cone utilizing the strain path method. Sandven (1990) used finite element computer programs and solved the problem numerically.

In each of these cases, a value of N_{kT} must be chosen before s_u can be determined from PCPT data. In many cases, this value is estimated from empirical correlations for practicality and the results are somewhat scattered. Various ranges of N_{kT} have been reported in the literature and backcalculated values between 7 and 32 are usually observed (Powell and Quarterman, 1988; Wroth, 1988).

Although the actual mechanism for soil failure around a penetrating cone is very complex, solving the problem with a simple closed-form approach is desirable for practical reasons. A new interpretation method, which combines spherical cavity expansion theory and modified Cam Clay, is derived herein for determining s_u.

In the proposed model, the pore water pressure measured immediately behind the tip (u_{bt}) is utilized. The s_u is expressed in terms of the PCPT parameter (q_T-u_{bt}), effective stress friction angle (ϕ'), and plastic volumetric strain ratio (Λ). The model also approximately accounts for the isotropic and anisotropic conditions during consolidation processes. Parametric studies are performed for verifying the sensitivity of parameters ϕ' and Λ within normal ranges, which result in a simple expression for practical use. Predictions are compared with the traditional N_{kT} reference values and results determined from isotropically and anisotropically-consolidated undrained triaxial compression tests.

MODEL DEVELOPMENT

By rearranging [1], the corrected cone tip resistance (q_T) is expressed in terms of the undrained shear strength (s_u):

$$q_T = N_{kT}s_u+P_o \qquad [3]$$

where P_o = in-situ total overburden stress and N_{kT} = cone bearing factor. If the spherical cavity expansion theory of Vesić (1977) is invoked, N_{kT} is simply:

$$N_{kT} = (4/3)(\ln I_r+1)+(\pi/2)+1 \qquad [4]$$

where $I_r = G/s_u$ = rigidity index. Values of N_{kT} from [4] are comparable to those from more sophisticated strain path analyses reported by Houlsby and Wroth (1989). Combining [3] and [4], the expression for the net cone tip resistance is given as:

$$(q_T-P_o) = (1.33\ln I_r+3.90)s_u \qquad [5]$$

Alternatively, I_r can be expressed in terms of (q_T-P_o) and s_u:

$$1.33\ln I_r = (q_T-P_o)/s_u-3.90 \qquad [6]$$

The excess pore water pressures ($\Delta u = u_{bt}-u_o$) generated during piezocone penetration may be expressed in terms of cavity expansion and critical-state concepts (Mayne and Bachus, 1988). These pressures are due to a combination of changes in octahedral and shear stresses:

$$\Delta u = \Delta u_{oct}+\Delta u_{shear} \qquad [7]$$

Using spherical cavity expansion theory to describe the octahedral component leads to:

$$\Delta u_{oct} = 1.33(\ln I_r)(s_u) \qquad [8]$$

Substituting [6] into [8] for eliminating I_r, the octahedral component of excess pore water pressures becomes:

$$\Delta u_{oct} = q_T-P_o-3.9s_u \qquad [9]$$

Assuming a constant P stress path for an isotropically-consolidated triaxial compression test (CIUC) as shown in Figure 2, the shear component of excess pore water pressures becomes:

$$\Delta u_{shear} = P_o'-P_f' \qquad [10]$$

where P_o' is the in-situ effective overburden stress and P_f' is the mean effective overburden stress at failure, $P_f' = 2s_u/M$. By substituting [9] and [10] into [7], the following is obtained:

$$u_{bt}-u_o = (q_T-P_o-3.9s_u)+(P_o'-2s_u/M) \qquad [11]$$

where $M = 6\sin\phi'/(3-\sin\phi')$. This expression results in an isotropic PCPT model for determining s_u:

$$(s_u)_{CIUC} = \frac{q_T-u_{bt}}{(2/M)+3.9} \qquad [12]$$

This simple isotropic model uses the modified Cam Clay concept where the corresponding normalized undrained shear strength ratio is given by:

$$(s_u/\sigma_{vo}')_{CIUC} = (M/2)(OCR/2)^{\Lambda} \qquad [13]$$

in which σ_{vo}' is the in-situ effective vertical stress and Λ is the plastic volumetric strain

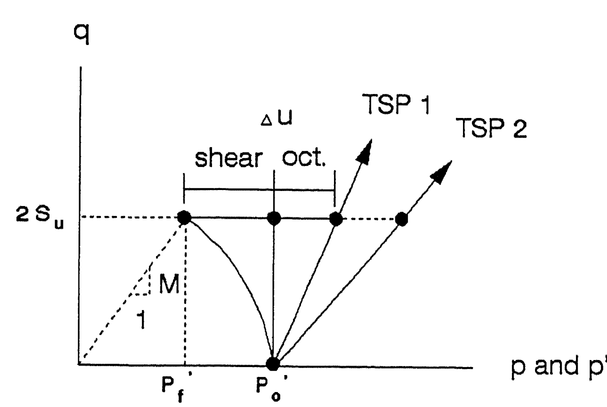

Figure 2. Interpretation of Excess Pore water Pressures Observed in Triaxial Compression Tests. (Wroth, 1984)

ratio. Since the actual stress state in the field is rarely isotropic, an anisotropic model for predicting s_u is desirable. Wroth (1984) derived a more complicated expression of normalized undrained shear strength corresponding to anisotropically-consolidated compression (CAUC):

$$(s_u/\sigma_{vo}')_{CAUC} = (b/2a)(OCR/2)^{\Lambda} \qquad [14a]$$

in which $a = (3-\sin\phi')/(6-4\sin\phi')$ [14b]

$$b = \sin\phi'(a^2+1)^{\Lambda} \qquad [14c]$$

By combining [13] and [14a], the ratio of anisotropic to isotropic strength becomes:

$$\frac{(s_u/\sigma_{vo}')_{CAUC}}{(s_u/\sigma_{vo}')_{CIUC}} = \frac{b}{aM} \qquad [15]$$

The factor (b/aM) is solely a function of ϕ' and Λ of the soil. For a typical value $\Lambda = 0.75$, the factor (b/aM) ranges from 0.96 at $\phi' = 20°$ to 0.76 at $\phi' = 40°$. Kulhawy and Mayne (1990) calibrated [15] against 48 intact clays, as shown in Figure 3. The available data indicate that the normalized undrained strength ratio for CAUC is lower than the ratio for CIUC. Subsequently, the s_u for the anisotropic compression mode can be expressed by:

$$(s_u)_{CAUC} = \left(\frac{q_T-u_{bt}}{(2/M)+3.9}\right)\left(\frac{b}{aM}\right) \qquad [16]$$

This may be alternatively expressed in the more simplified form:

$$s_u = (q_T-u_{bt})/N_{qu} \qquad [17]$$

where $N_{qu} = (2/M)+3.9$ for CIUC tests, and
$N_{qu} = [(2/M)+3.9](aM/b)$ for CAUC tests.

PARAMETRIC STUDY

Among the parameters presented in [17], q_T and u_{bt} are obtained directly from PCPT results, while N_{qu} is dependent on ϕ' and Λ of the soil. A parametric study was performed to investigate the significance of ϕ' and Λ in the model. Diaz-Rodriguez et al. (1992) reported a full range of ϕ' for natural clays worldwide from 17.5° to 43°. A review of 96 different sets of laboratory triaxial tests on clays compiled by Mayne (1980) indicates that $0.6 \le \Lambda \le 0.8$ for insensitive clays and $0.9 \le \Lambda \le 1.0$ for structured and cemented clays.

Figure 4 shows the theoretical value of N_{qu} over a wide range of ϕ' and Λ for both CIUC and CAUC conditions. For CAUC, it appears that N_{qu} is insensitive to variations of ϕ' and Λ within normal ranges. For the range of ϕ' and Λ mentioned above, N_{qu} varies from 6.0 to 7.2, which is considered an improvement over the expected wider range of N_{kT}. For CIUC, the value of N_{qu} is essentially independent of Λ and slightly decreases as ϕ' increases. Again, the value of N_{qu} varies only from 5.0 to 6.8 for the aforementioned range of ϕ' and Λ.

Parametric studies were performed using data from the eight sites listed in Table 1. Results from the studies, such as those for Lilla Mellösa and Gloucester sites shown in Figures 5 and 6, indicate that the model is not sensitive to either ϕ' or Λ. If average values of $\phi' = 30°$ and $\Lambda = 0.75$ are adopted, N_{qu} equals 5.5 and 6.5 for CIUC and CAUC, respectively. For engineering use, the following expression is recommended:

$$(s_u)_{CAUC} = (q_T-u_{bt})/6.5 \qquad [18]$$

since clays in nature are generally consolidated under anisotropic states of stress.

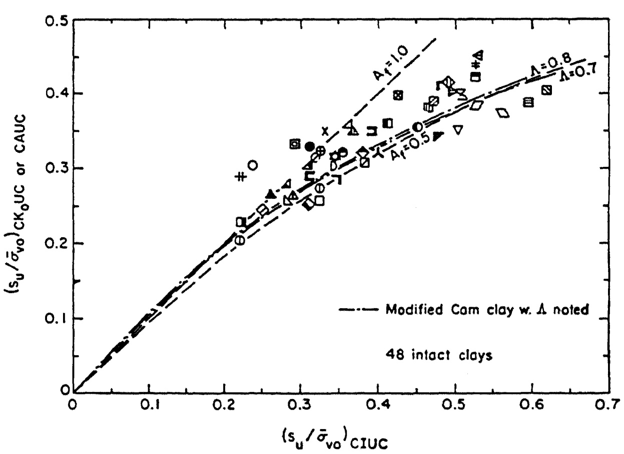

Figure 3. Comparison of Undrained Strength Ratio for NC Clays After Anisotropic and Isotropic Consolidations. (Kulhawy and Mayne, 1990)

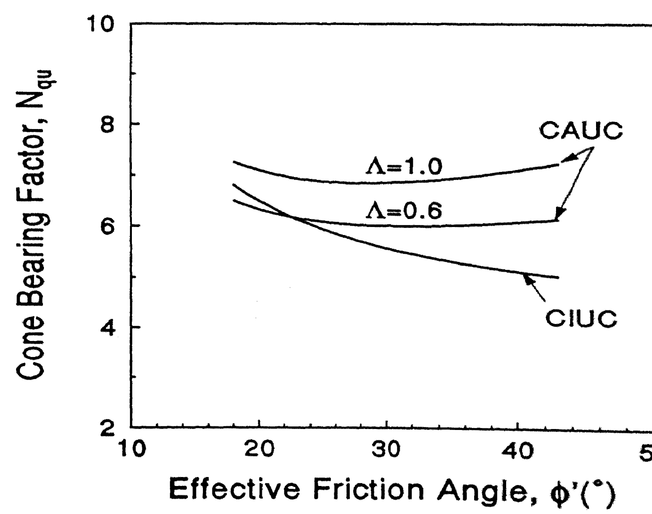

Figure 4. Bearing Cone Factor N_{qu} as a function of ϕ' and Λ.

Table 1. Summary of Piezocone Clay Sites and Relevant Information.

Site	Description	w_n	LL	PI	S_t	OCR	ϕ'	N_{KT}	N_{qu}	References
Lilla Mellösa, Sweden	NC, organic	100	95	65	15	1.2	30°	11.8	6.5	Larsson & Mulabdic (1991)
Gloucester, Ontario	NC, aged Leda	70	50	25	60±40	1.5	35°	10.8	6.3	Konrad & Law (1987)
St. Alban, Québec	Soft, aged	63	45	22	22	2.3	27°	10.0	5.9	Roy et al. (1982)
Bothkennar, U.K.	Soft NC	65	73	41	4-6	1-3	33°	13.2	8.4	Powell et al. (1988)
Onsøy, Norway	Soft NC, aged	63	65	37	6-9	1-4	34°	14.2	8.5	Lunne et al. (1986)
Valøya 4, Norway	MOC, very stiff	43	52	15	3-7	3-11	27°	10.3	5.7	Sandven (1990)
Yorktown, Virginia	MOC, stiff	31	31	4	4-8	4-12	38°	12.1	7.2	Mayne (1989)
Taranto, Italy	HOC, cemented	23	60	27	NA	20-40	28°	16.0	NA	Jamiolkowski et al. (1982)

Notes: NC - Normally Consolidated; LOC - Lightly Overconsolidated; MOC - Moderately Overconsolidated; HOC - Heavily Overconsolidated; N_{KT} - Backcalculated from $(q_T-\sigma_{vo})/s_u$; N_{qu} - Backcalculated from $(q_T-u_{bt})/s_u$

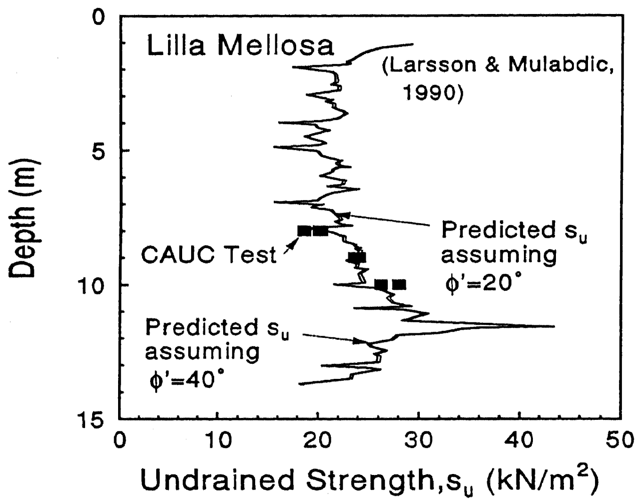

Figure 5. Parametric Effect of ϕ'.

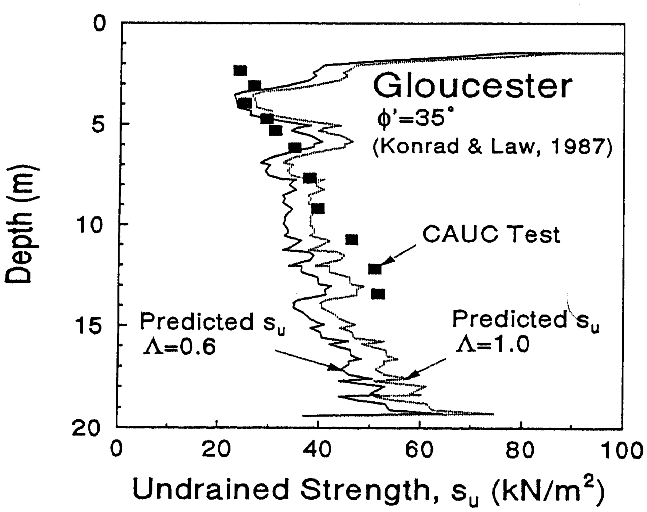

Figure 6. Parametric Effect of Λ.

LABORATORY STUDY

The newly proposed method for evaluating s_u from PCPT was applied to results from a laboratory-controlled testing program. In this event, a series of miniature in-situ tests were performed in prestressed kaolinitic clay during an experimental test program involving model foundation testing in a large fixed-wall calibration chamber. The deposit of clay was formed from a lean kaolinitic-silica slurry that was comprised of a 50-50 mixture of Peerless Clay No. 2 and very fine SuperSil 125 at an initial water content w_n = 66%. Resulting index properties were: LL = 33, PI = 11, CF = 33%, G_s = 2.65, and D_{50} = 0.006 mm. Additional details may be found in Mayne, Kulhawy, and Trautmann (1992) and Mayne (1992).

The slurry was pumped into a large cylindrical steel chamber having an inside diameter of 1.37 m and height of 2.13 m. Pneumatic pressure was applied to the top of a rigid piston and the slurry was consolidated one-dimensionally at $\Delta\sigma_p'$ = 48 kN/m^2 with double drainage permitted. After completion of primary consolidation, the specimen was rebounded to atmospheric conditions, resulting in a mechanically-overconsolidated profile with OCR = $(\Delta\sigma_p'+\sigma_{vo}')/\sigma_{vo}'$. A water reservoir maintained the "groundwater" level contiguous with the surface of the clay. After prestressing, the clay had an average water content w_n = 34.5 percent, e_o = 0.914, and γ_T = 18.2 kN/m^3.

A complementary suite of laboratory testing included triaxial, direct simple shear, oedometer, creep, isotropic consolidation, K_o tests, and fall cone tests on the material. Some of these test results are reported in McManus and Kulhawy (1991). Figure 7 shows the effective stress paths for CIUC triaxial tests on the material at four levels of induced OCR. The triaxial data indicate an effective stress friction angle ϕ' = 33.5° (or critical state failure parameter M = 1.35).

The results of conventional one-dimensional consolidation tests on a retrieved sample of the clay is presented in Figure 8. The interpreted σ_p' = 50 kN/m^2 is consistent with the known applied stress history to the deposit. Consolidation parameters derived from the oedometer testing include: C_c = 0.214, C_s = 0.028, and $C_{\alpha e}$ = 0.0067.

Miniature in-situ tests were conducted to evaluate the uniformity and consistency of the prestressed clay deposit (Mayne, 1992). These included vane shear, electric cone, two types of piezoprobe, as well as water content determinations. A motorized Wykeham-Farrance vane apparatus was used to perform the vane shear tests with a rectangular blade (12.7 mm diameter by 25.4 mm height). Undrained strengths measured by the vane were essentially constant with depth at s_{uv} = 8.51 ± 0.73 kPa. Consolidated water contents decreased from about 36% at the top to 34% at the bottom of the deposit.

Electric cone penetration tests were performed using a 23.3-mm diameter miniature penetrometer (Fugro-type geometry) with 60° apex to provide measurements of q_c. The cone has a net area ratio a = 0.88. Piezoprobe soundings were conducted using 19.1-mm diameter 60° tipped brass cones that were fitted with miniature Druck transducers and sintered brass porous elements. Two types of piezoprobes were built so that penetration pore water pressures could be measured at the tip (u_t) and behind the tip (u_{bt}). Figure 9 shows the records of the penetration tests in the prestressed clay deposit. The combined data from the cone and piezoprobes result in the equivalence of a piezocone sounding. The measured cone tip resistance (q_c) has been corrected to q_T to account for pore water pressure effects on equal areas of the cone geometry (Lunne et al., 1986).

A comparison of the measured triaxial compression strengths and predicted s_u profiles in the overconsolidated clay is shown in Figure 10. Measured values of s_u include the results from unconfined compression (UC) tests on retrieved samples at depths of 300, 600, and 900 mm, as

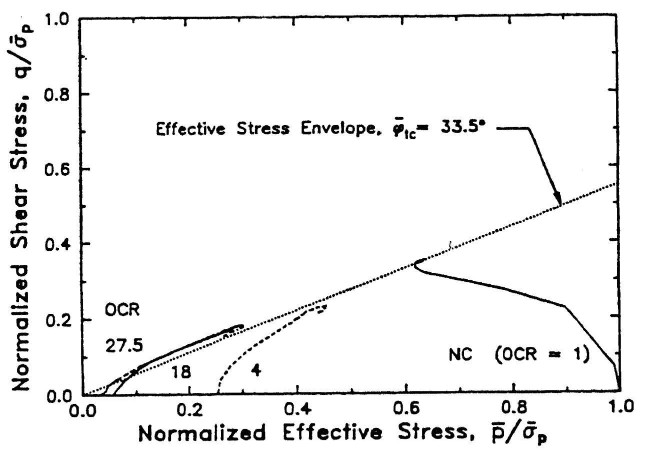

Figure 7. Effective Stress Paths for CIUC tests on Kaolinitic Clay.
(Source: McManus and Kulhawy, 1990)

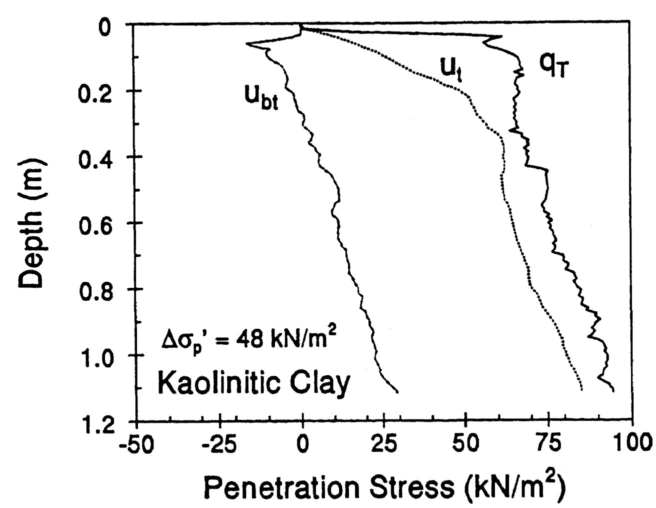

Figure 9. Results of Composite Piezocone Test from Miniature Electric Cone and Piezoprobe Soundings.

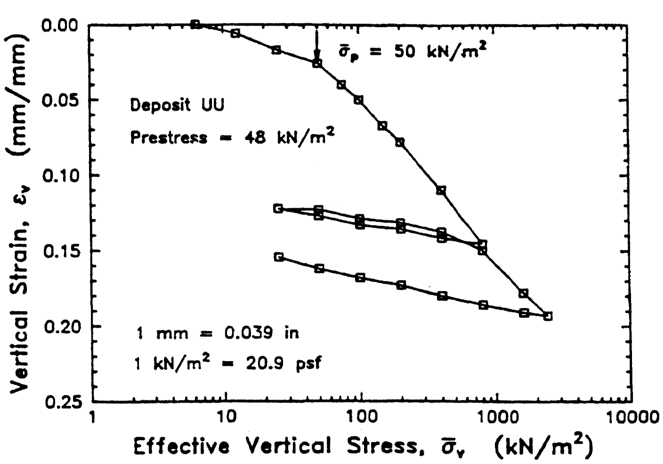

Figure 8. Oedometer Test Results on Prestressed Kaolinitic Clay.

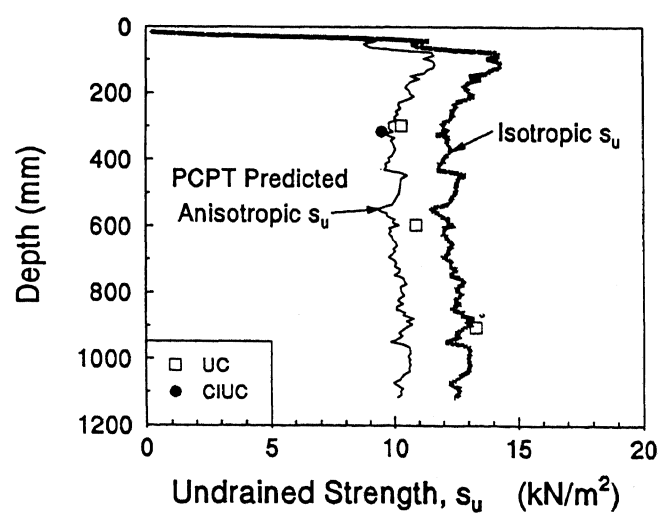

Figure 10. Measured-TC and PCPT Predicted Strengths of Kaolinitic Clay.

well as a SHANSEP value taken at the oedometer test depth and normalized relationship determined from the CIUC tests:

$$s_u/\sigma_{vo}' = 0.336\ OCR^{0.79} \qquad [19]$$

Reasonable agreement is seen for the range predicted by PCPT for isotropic and anisotropic undrained shear strengths.

CASE STUDIES

Eight well-documented sites selected from the geotechnical literature have been studied for the calibration of the proposed model. Table 1 summarizes the soil information and sources of references for these sites. The soils in these sites range from soft, sensitive, normally consolidated or lightly overconsolidated to very stiff, heavily overconsolidated clay deposits. The clays at St. Alban, Valøya, and Onsøy are considered to be moderately sensitive; St. Alban and Taranto are noted to be cemented. Bothkennar is insensitive and Yorktown consists of very sandy clays. As reported by the sources in Table 1, the average ϕ' for each site has been determined from laboratory triaxial compression tests.

The selection of the reference test is crucial in this study since s_u can vary over a wide range depending upon consolidation process, shearing mode, fabric, direction of loading, strain rate, stress rotation, and disturbance effects. Laboratory CIUC and CAUC tests have been selected where available, except for the Taranto site, in which the results from high quality unconsolidated undrained triaxial compression tests (UU) were available (Jamiolkowski et al., 1988). Reasons for choosing CIUC or CAUC tests as major reference tests include:

(1) The soil behavior beneath the cone tip is similar to that exhibited in triaxial compression.
(2) The consolidated undrained test (CU) is considered to be more reliable than the UU and unconfined compression (UC) tests regarding sampling disturbance and strain rate effects.
(3) Field tests such as field vane (FV), dilatometer (DMT), and self-boring pressuremeter (SBPMT) require further interpretation, and therefore, may not be appropriate for this study.

Figure 11 shows a series of predictions of s_u for six clays using [18]. This N_{qu} model and the conventional N_{kT} approach are compared and the result for the St. Alban site is presented in Figure 11(a). In general, [18] provides fairly reasonable profiles of s_u for Valøya, Yorktown, and Taranto sites; while slight over-predictions are evident for Bothkennar and Onsøy. It must be pointed out that the soils at each of these sites are essentially intact clays, therefore, this model may require further verification before application to fissured clays.

The value of N_{qu} was back-calculated for each site. The back-calculated N_{kT} from net cone resistances and undrained shear strengths were also obtained from PCPT data and CU tests.

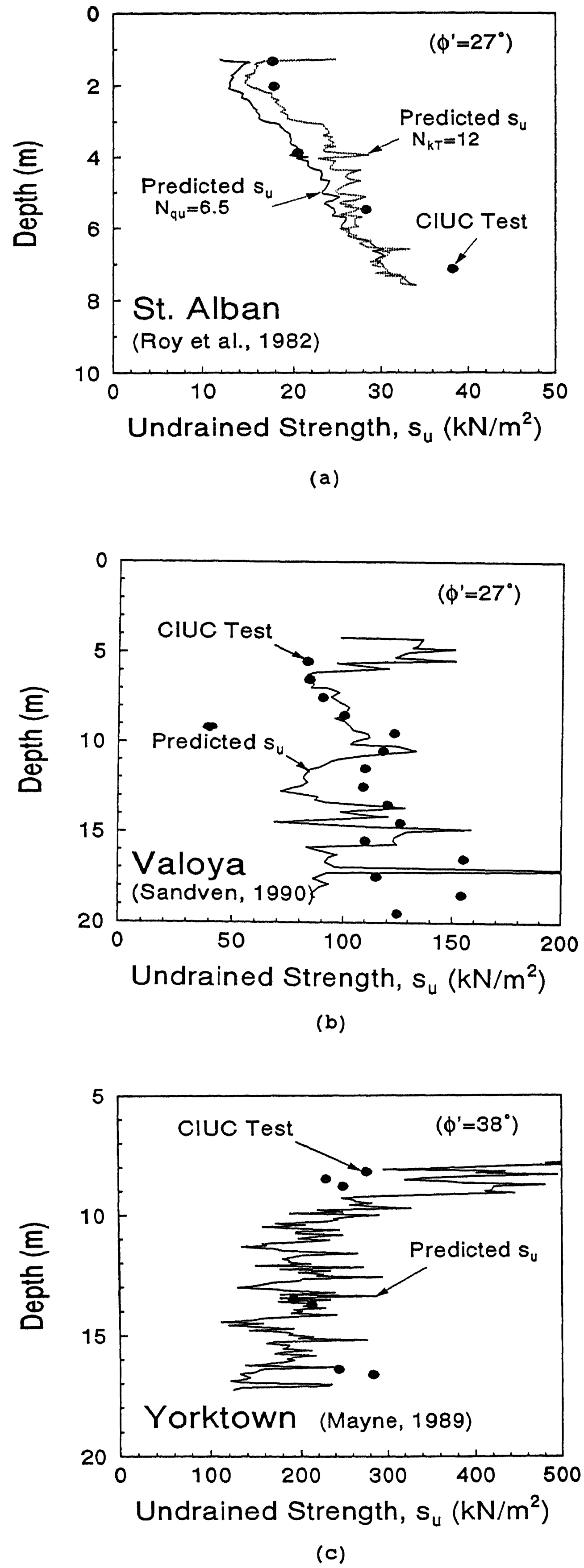

Figure 11. Applications of PCPT-s_u Profiles.

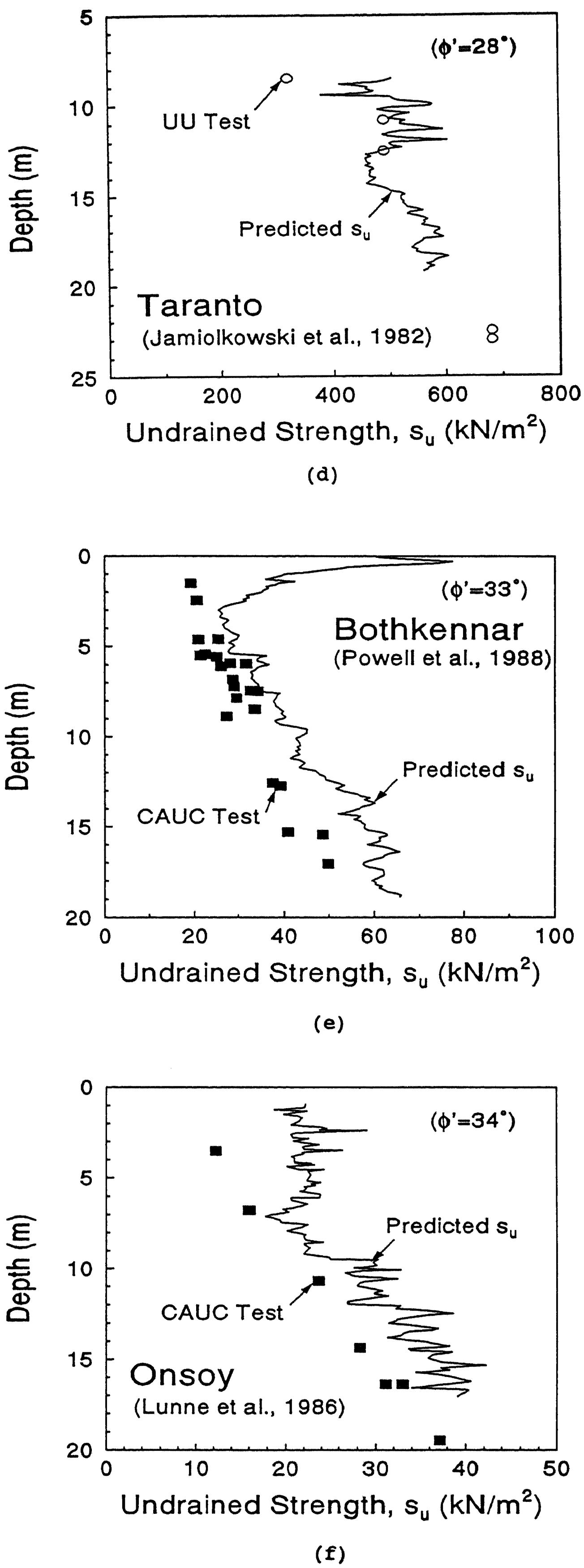

Figure 11. Applications of PCPT-s_u Profiles.

Results summarized in Table 1 indicate that, for the eight intact clay sites reviewed in this study, values of N_{kT} range from 10 to 16, while backcalculated values of N_{qu} consistently fall between 5.7 and 8.5. For fissured clays, Powell and Quarterman (1988) recommended $20 \leq N_{kT} \leq 30$.

It is well recognized that the value of N_{kT} is not a constant, instead, it varies depending upon the rigidity index I_r and subsequently the OCR of clay deposits. A wide range of N_{kT} varying from 7 to 32 has been reported by several researchers (Keaveny and Mitchell, 1986; Wroth, 1988; Powell and Quarterman, 1988). On the other hand, the proposed N_{qu} model shows a smaller range of cone bearing factor.

Both the conventional $(q_T-\sigma_{vo})$ approach and the proposed (q_T-u_{bt}) model are simple and convenient for practicing engineers. While they provide similar results, the latter makes use of another important PCPT measurement (u_{bt}). However, further calibration of the model is necessary. Additional factors such as K_o-induced anisotropy, stress rotation effects, soil fabric, fissuring, sensitivity, and strain rate must be evaluated for future improvements.

CONCLUSION

The development of a simple hybrid theory based on spherical cavity expansion and modified Cam Clay has been shown to approximately relate s_u to the PCPT parameter (q_T-u_{bt}). The predictions are relatively insensitive to ϕ' and Λ. Preliminary calibration of the model has shown a similar degree of satisfaction when compared to the conventional net cone resistance approach.

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