Evaluation of undrained shear strength of soft New Orleans clay using piezocone

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ABSTRACT: Cone penetration test (CPT) or piezocone penetration test (PCPT or CPTu) has long been used to estimate the undrained shear strength of clays. However, the undrained shear strength itself is not a unique measure of soil strength; different laboratory or in-situ tests may give different undrained shear strengths for the same soils. On the other hand, different cone factors associated with different interpretation methods are used to correlate the undrained shear strength with CPT or PCPT measurements. This paper presents a statistical analysis of CPT or PCPT based methods to correlate undrained shear strength with laboratory triaxial unconsolidated undrained compression results of soft deposits in the New Orleans area. The suggested cone factors may be valuable to practitioners to evaluate undrained shear strengths for New Orleans soft clays using CPT or PCPT results. On the other hand, the relationship without total overburden pressure correction shows excellent correlation with the laboratory measured shear strengths. By looking at the failure mode as a triaxial compression rather than a cavity expansion, it may not be necessary for the total overburden pressure explicitly included in the undrained shear strength determination.

1 INTRODUCTION

Traditional cone penetration tests (CPT) or the piezocone penetration test with pore pressure measurements (PCPT) are becoming increasingly popular nowadays to assist subsurface investigations. One of the applications of CPT or PCPT soundings is to estimate the undrained shear strength of clayey deposits. Most PCPT can measure tip resistance (q_c), sleeve friction (f_s) and pore water pressure (u_1, at the cone tip or u_2, behind the cone base) simultaneously (Tumay, et al., 1981; Zuidberg, et.al, 1982). Although all the above measurements may be correlated to the undrained shear strength, the tip resistance is usually the most frequently used quantity to estimate the undrained shear strength. It is widely accepted that the undrained shear strength may be estimated using the following bearing capacity equation (Terzaghi, 1943):

$$s_u = \frac{q_c - \sigma_v}{N_k} \tag{1}$$

where s_u is the undrained shear strength and q_c is the measured tip resistance and σ_v is the total overburden pressure. N_k is the cone factor, which a value of circa 9 has been proposed based on measurements and analysis (Meyerhof, 1951; Skempton, 1951; Tumay et.al, 1982). On the other hand, some researches tended to simply use a fraction of the measured tip resistance as the undrained shear strength:

$$s_u = q_c/N_c \tag{2}$$

where s_u is the undrained shear strength, q_c is the measured tip resistance while N_c is the cone factor. Sanglerat (1972) observed that N_c may range from 10 to 20 and often very close to a value of 15 for the soft clays of the Annecy area in France. Due to the geometry of the cone penetrometer, it is recommended that the measured tip resistance be corrected for the pore pressure, u_2, measured behind the cone base (Campanella et al., 1983). With q_t designated as the corrected tip resistance accounted for pore pressure effect, the above equations (1) and (2) can be re-written as equations (3) and (4), respectively as follows:

$$s_u = \frac{q_t - \sigma_v}{N_{kt}} \tag{3}$$

$$s_u = q_t/N_c \tag{4}$$

More recently, as the pore pressure measurement becomes more and more popular, the undrained shear strength may be evaluated based on the excess pore pressure (Δu) measured behind the cone (u_2) using the following:

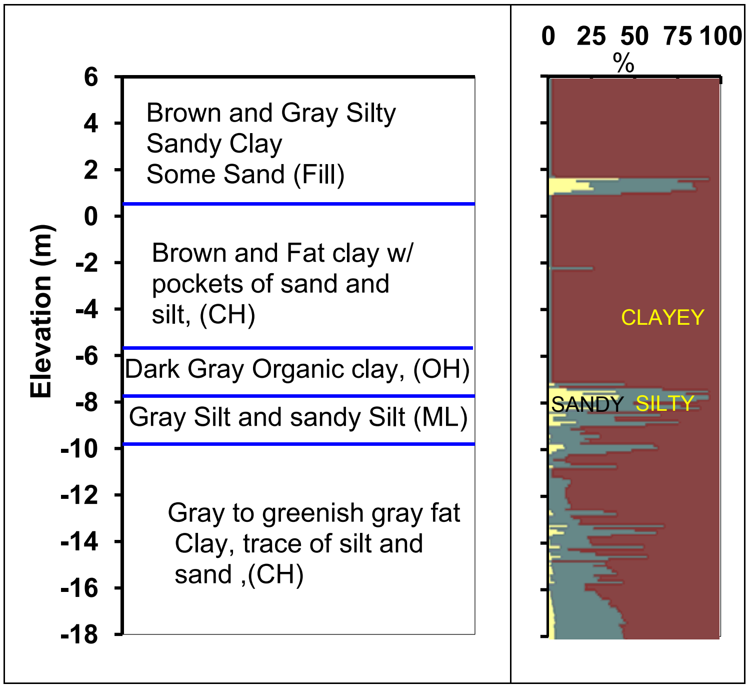
$$s_u = \Delta u/N_{\Delta u} \tag{5}$$

where $N_{\Delta u}$ varies from 7 to 10 (Robertson and Robertson, 2006). The purpose of this paper is to estimate the undrained shear strength using equations (3), (4) and (5) by correlating with laboratory test results based on a test site in New Orleans. The soil shear strengths were determined by laboratory triaxial unconsolidated undrained compression tests (UU). A statistical analysis is performed to determine the best fit cone factors for the test site in New Orleans using equations (3), (4) and (5).

2 SUBSURFACE CONDITIONS

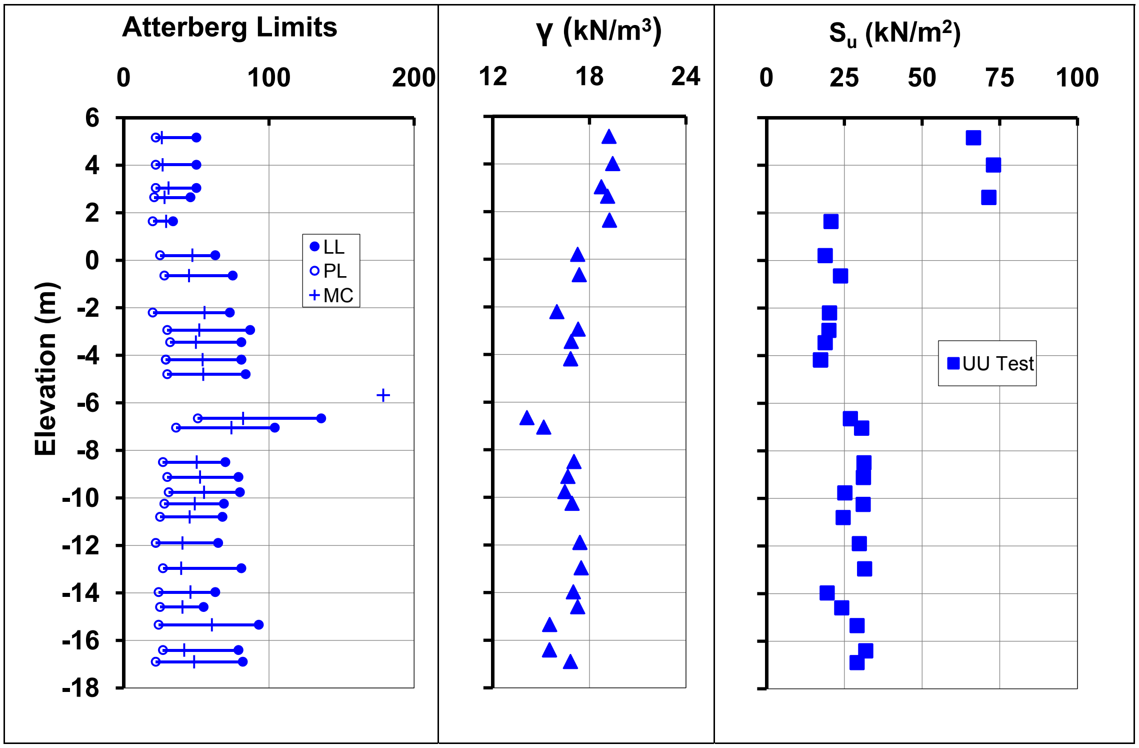
Nine borings and PCPT's were drilled or pushed along the centerline of the levee over a distance of 2,000 ft located in east New Orleans, Louisiana. Shelby tubes were used to retrieve undisturbed samples continuously at the boring location and the undisturbed samples were later transported to the laboratory for index and strength tests. The top of the explorations is at approximate elevation +6 m and the bottom of the explorations is located at about elevation -18 m. The water table appears to be near elevation +2.4 m. The natural ground surface in the area is believed to be near elevation 0 to +2 m, above which fill materials (silty and sandy clay) were placed to raise the existing levees. Similar soil stratifications were identified in all the nine borings and PCPT's. A representative boring log is presented in Figure 1a. At this boring location, the natural deposits are predominantly high plasticity fat clays; except that an organic clay layer appeared to be present between approximate El -6 to -8 m and that a silt and sandy silt layer is located between approximate El -8 to -10 m. The associated CPT based probability soil types are shown in Figure 1b, using the computerized probabilistic soil classification technique by Zhang and Tumay (1999). The PCPT based soil classification is generally in agreement with those identified in the borings.

The index property tests include moisture content, Atterberg limits and unit weight. The strength tests include numerous triaxial unconsolidated undrained compression tests (UU). The lab test results at this boring location are shown in Figures 2a through 2c. The PCPT measurements (tip resistance, sleeve friction, pore water pressure (u_2) and friction ratio, R_f) are presented in Figures 3a through 3c. Since this study focuses on the cone factors based on statistical methods, all the laboratory UU test data presented in Figure 4a are plotted against effective overburden. For each UU test, the adjacent PCPT measurements at the same depth are presented in Figure 4b (net tip resistance versus effective overburden) and Figure 4c (excess pore pressure versus effective overburden). Since this study is interested in the cone correlations (cone factors) in natural deposits, the test results within the fill materials were not included in Figures 4a through 4c. Similarly, the statistical (regression) analysis presented in the next section does not contain test data from those fill materials, either.



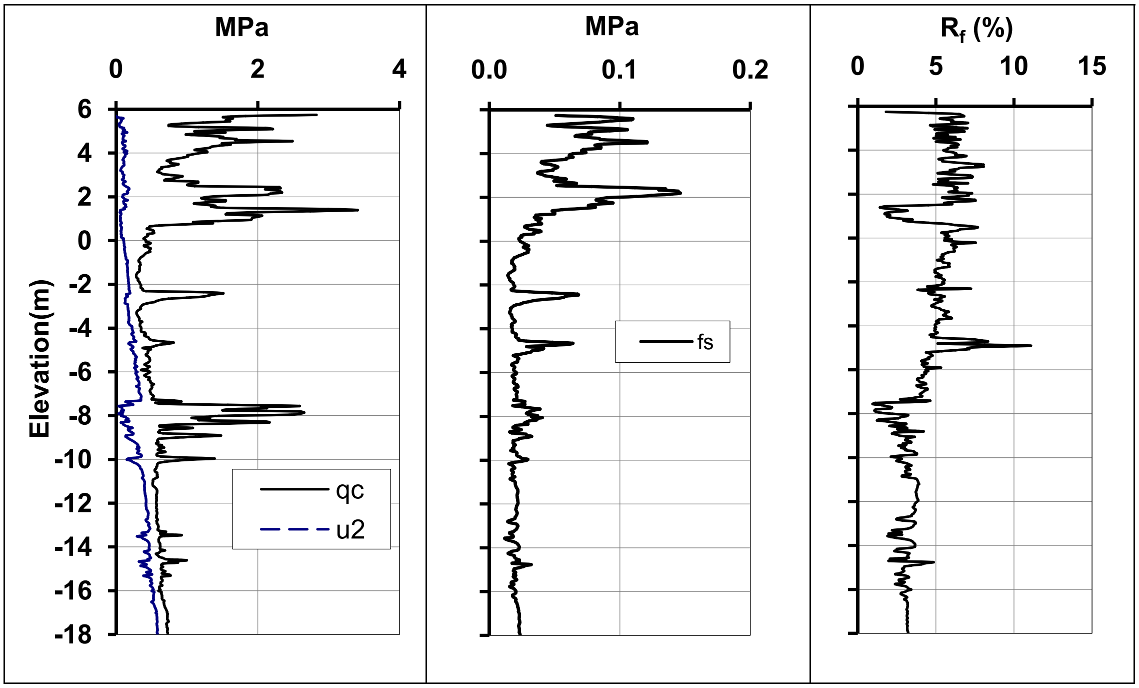
(1a) (1b)

Figure 1: (a) Boring Log; (b) CPT Probability Soil Type Percentage



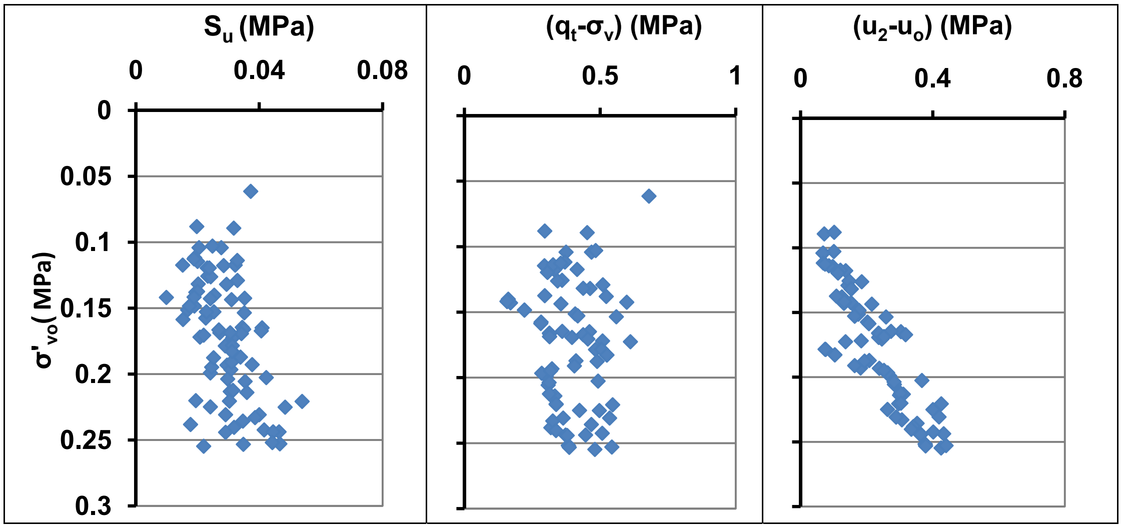
(2a) (2b) (2c)

Figure 2: (a) Atterberg Limits and Moisture Content; (b) Moist Unit Weight; (c) Undrained Shear Strength from Lab UU Tests



(3a) (3b) (3c)

Figure 3: (a) Tip Resistance and Pore Pressure; (b) Sleeve Friction; (c) Friction Ratio



(4a) (4b) (4c)

Figure 4: UU Strengths and PCPT Measurements vs Effective Overburden

3 RESULTS AND DISCUSSIONS

Table 1 summarizes the regression model parameters for the different correlations. n is the total number of data points used in the regression model. Coefficient of determination (R^2) generally indicates the index of fit, which means how much the sum of squared errors can be reduced by using a regression line compared to baseline model of always predicting mean. Mathematically, it can be expressed as

$$R^2 = \frac{SSR}{SST} = \frac{\sum(\widehat{Yi}-\bar{Y})^2}{(Yi-\bar{Y})^2} \qquad (6)$$

where $\bar{Y}$ the mean of the observed dependent variables , $\widehat{Yi}$ is the predicted variable corresponding to dependent variable X_i, SSR denotes regression sum of square and SST is total sum of square. Thus for linear correlations, higher the value of R^2 better the fit of data against regression line. For perfect linear fit, R^2 would be 1. However, regression model in this study restrict the intercept and regression line passes through origin.

$$\hat{Y} = \beta X_i + \varepsilon \qquad (7)$$

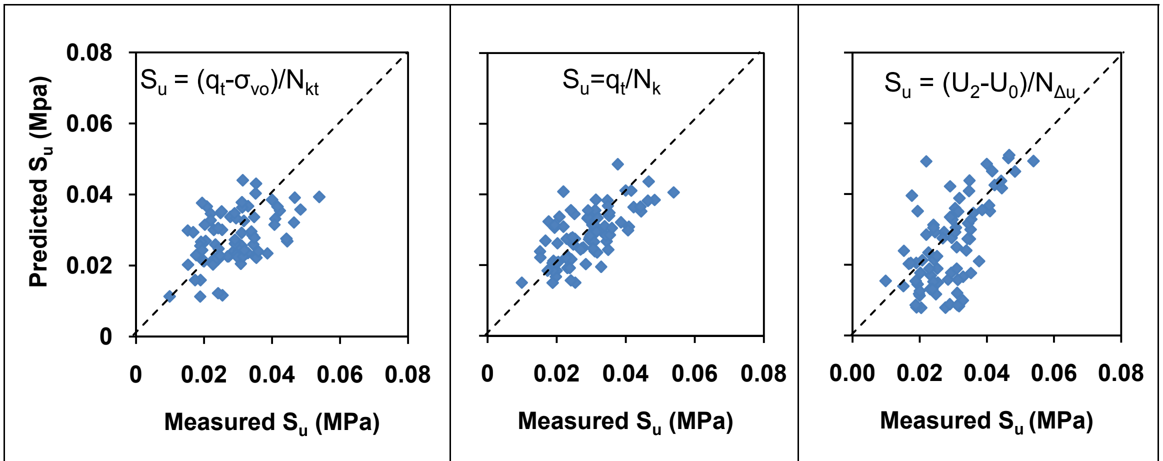
As such R^2 reported in the table are not directly comparable to that obtained with non restricted model (Kvalseth, 1985):

$$\hat{Y} = \beta_o + \beta X_i + \varepsilon \qquad (8)$$

Figure 5a, 5b and 5c present the predicted versus measured undrained shear strength with statistical analysis by equations (3), (4) and (5), respectively. A 45° line represents the best fit line and points scattered below or over this line indicate under-estimation or over-estimation of undrained shear strength using proposed correlations. In general the PCPT predicted shear strengths correlate well with the triaxial unconsolidated undrained test results with the cone factors shown in Table 1. The cone factor (N_{kt}) of 13.9 for equation (3) is between 10 to 15, which aligns with the normally consolidated clay range pointed out by de Ruiter (1982). The cone factor (N_c) of 23 for equation (4) is greater than 20, which is slightly out of the bound of 10 to 20 as observed by Sanglerat (1972). It's believed that the reason is due to the use of pore pressure corrected tip resistance. The cone factor of 8.6 using pore pressure based equation (5) is within the 7 to 10 range as pointed out by Robertson and Robertson (2006). The coefficient of determination (R^2) is 0.92, 0.95 and 0.88 for equations (3), (4) and (5) respectively. Based on the regression analysis results, it appears that equation (4), which has no overburden correction, gives the least scattered correlation based on the tested data at the site. Equation (3), which has taken into account the overburden pressure, has a slightly more scattered correlation than equation (4) by comparing the R^2 values. The correlation based on excess pore pressures (equation (5)) shows more scatter compared to equations (3) and (4) when evaluating the undrained shear strengths at the selected site. Considering equation (3) is much more widely used than equation (4), the fact that equation (4) actually yields a slightly better statistical correlation in this study is rather interesting to the authors.

Table 1: Summary of Regression Analysis

Equation No.	Model	n	R^2	Cone Factor
(3)	$s_u = \frac{q_t - \sigma_v}{N_{kt}}$	82	0.92	$N_{kt} = 13.9$
(4)	$s_u = \frac{q_t}{N_c}$	82	0.95	$N_c = 23$
(5)	$s_u = \frac{u_2 - u_0}{N_{\Delta u}}$	82	0.88	$N_{\Delta u} = 8.6$



5(a) 5(b) 5(c)

Figure 5: Predicted vs Measured Undrained Shear Strength

The total overburden playing a role in the CPT or PCPT undrained shear strength estimation has come from both bearing capacity and cavity expansion based interpretations (Meyerhof, 1961; Vesic, 1972). Compared to bearing capacity theory, cavity expansion theory is more favored by many researchers since it can take into account the soil deformation characteristics (Yu and Mitchell, 1998). However, the cavity expansion theory has its own limitations. Cavity expansion theories do not reflect the shape and scale effects experienced in field observations (Tumay et al., 1982). It is essentially a one-dimensional theory and thus restricts the dependence of field variables (i.e., displacements, strains, stresses, and pore pressures) to the radial coordinates only. Many researchers agree that the soil underneath the cone tip may fail similar to the triaxial compression failure mode (Keaveny and Mitchell, 1986). The undrained shear strength (S_u) can typically be expressed as:

$$s_u = (\sigma_1 - \sigma_3)/2 \qquad (9)$$

where σ_1 is the major principal stress while σ_3 is the minor principal stress. If the cone tip resistance can simply be considered the principal stress increase, $\Delta\sigma_1$, and if the minor principal stress increase, $\Delta\sigma_3$, can be assumed to be a fraction of $\Delta\sigma_1$, equation (9) may be re-written as

$$s_u = (\sigma_{1o} - \sigma_{3o})/2 + (q_t - Kq_t)/2 \qquad (10)$$

where σ_{1o} and σ_{3o} are in-situ major and minor principal stresses, respectively. K is a constant which relates the minor principal stress increase to major principal stress increase during cone advancing. A rigorous determination of K obviously should include appropriate soil behavior modeling, which requires further study in the future. If the initial stress is isotropic or difference between σ_{1o} and σ_{3o} is not significant, equation (10) will take the form of equation (4) with the cone factor N_c as

$$N_c = 2/(1-K) \qquad (11)$$

A range of N_c of 10 to 20 will correspond to a range of K of 0.8 to 0.9. Again, an analytical solution of K requires further study of soil behavior in a triaxial compression modeling.

4 CONCLUSIONS

Based on a case study of soft soils in New Orleans area with limited testing data, an attempt was made to correlate the PCPT based undrained shear strengths with those measured in the laboratory by triaxial unconsolidated undrained compression (UU) tests. It appears that both tip resistance and pore pressure based predictions are generally in agreement with the UU test results. The best-fit cone factors listed in Table 1 may be used to evaluate soil undrained strengths in the greater New Orleans area. The statistical (regression) analysis shows that equation (4), which has no overburden correction, gives slightly less scattered correlation than equation (3) which explicitly includes the total overburden pressure. The pore pressure based predictions (equation (5)) seems more scattered than those that are tip-resistance-based (equations (3) and

(4)). Since the overburden pressure has been implicitly reflected in the measured tip resistance, the explicit inclusion of total overburden pressure may not be necessary. Equation (10) illustrate the relationship of undrained shear strength to cone tip resistance by assuming soils experiencing a triaxial compression. As such, the initial stress anisotropy may not be significant and equation (11) may be used to calculate the cone factor N_c along with equation (4) to evaluate undrained shear strength from PCPT measurements. The constant K is the fraction of horizontal stress increase versus vertical stress increase in triaxial compression. A rigorous determination of K will require further study and before such a study becomes available, a preliminary range of 0.8 to 0.9 may be justified by past experience.

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