Effective Stress Friction Angle of Normally Consolidated and Overconsolidated Intact Clays from Piezocone Tests

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ABSTRACT: The effective stress friction angle (ϕ′) is an important fundamental property for all soil types. A modified effective stress limit plasticity solution is presented in this paper for the evaluation of ϕ′ of normally-consolidated to overconsolidated clays from piezocone penetration tests (CPTu). The solution takes account of stress history effect by introducing the equivalent stress concept from critical state soil mechanics (CSSM). Values of ϕ′ obtained from laboratory triaxial compression tests on high quality samples are taken as the benchmark reference. The method is applicable to clays that are intact, insensitive, and inorganic. Example results of piezocones performed in normally consolidated kaolin in the laboratory and field tests on overconsolidated clay from Alaska are presented to elaborate application of the solution. A compiled database from 132 piezocone soundings in intact clays at field sites, 1-g chamber tests, and centrifuge series is compared with triaxial tests to show the full range of 18° < ϕ′< 45° of natural and artificial clays.

KEYWORDS: Clay, Friction angle, Piezocone, Effective stress friction angle, Overconsolidation ratio

1. INTRODUCTION

Piezocone testing (CPTu) involves the vertical pushing of an instrumented electronic probe into the soil at a constant rate of penetration of 20 mm/s to collect data with depth. For the standard CPTu, three separate measurements are obtained with depth: cone tip resistance (q_c), sleeve friction (f_s), and penetration porewater pressure at the shoulder position (u_2). Readings are generally taken at 1 or 2 second intervals, thus data are essentially continuous. The measured q_c is converted to total cone tip resistance (q_t) using $q_t = q_c + (1 - a_{net})\cdot u_2$, where a_{net} is defined as net area ratio (Campanella and Robertson 1988). Details concerning CPTu equipment, field test procedures, and reporting of acquired data are given by Lunne et al. (1997). Interpretative methods for assessing the stratigraphy, soil types, and selected geoparameters are covered elsewhere (Mayne 2007; Schnaid 2009; Robertson & Cabal 2016).

1.1 Effective strength of soil

It is well-recognized that soil behavior is controlled by effective stresses, as established by the famous "principle of effective stress" in which the effective stress (σ') acting on a soil element is calculated from two components, total stress (σ) and porewater pressure (u) according to:

$$\sigma' = \sigma - u \tag{1}$$

As shown by critical state soil mechanics (CSSM), the fundamental strength of all soils is represented by the effective stress strength angle, ϕ' (Schofield & Wroth 1968; Wood 1990; Diaz-Rodriguez et al. 1992; Leroueil & Hight, 2003; Mayne et al. 2009). Moreover, in terms of the common Mohr-Coulomb strength criterion, there are two strength parameters: ϕ' = effective friction angle and c' = effective cohesion intercept, where shear stress τ and the effective normal stress σ' are expressed in the following formula:

$$\tau = c' + \sigma' \tan\phi' \tag{2}$$

For vast majority of soils that are uncemented, particularly soft clays and silts, the value of c' can be taken as zero.

Following the release of the classical reference book on triaxial testing by Bishop & Henkel (1962), geotechnical laboratories quickly adopted the more versatile triaxial test over the older direct shear box, since it offered direct control of vertical and lateral effective stresses, drainage conditions, and measurement of porewater pressures. Figure 1 shows a schematic plot of the deviator stress $q = \sigma_1 - \sigma_3$ versus the mean effective stress $p' = (\sigma_1' + 2\sigma_3')/3$ and the effective stress friction angle ϕ' can be determined by solving M_c = slope of the frictional envelope at failure under triaxial compression, where $M_c = (6\sin\phi')/(3-\sin\phi')$. Such presentation of deviator stress q versus the mean effective stress p' is called the Cambridge q - p' space (Schofield & Wroth 1968; Mayne et al. 2009). Figure 2 shows series of both undrained and drained stress paths from triaxial compression tests on normally-consolidated Korean soft clays. It is quite evident that the effective stress friction angle ϕ' is interpreted as ϕ'= 27.7° with c'=0, thereby establishing the friction angle as a fundamental soil property.

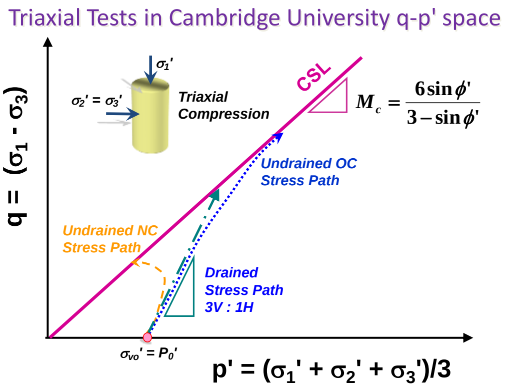

Figure 1 Schematic plot showing stress path from triaxial compression test

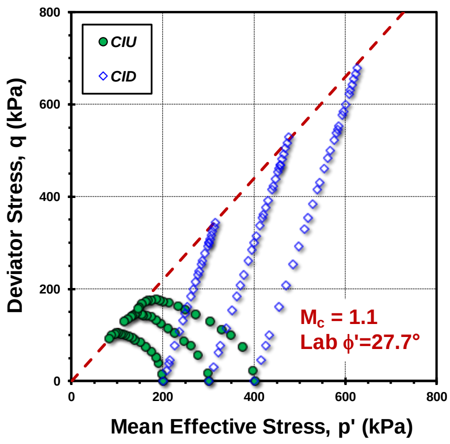

Figure 2 Triaxial compression tests on Korean soft clay- drained and undrained series (data from Lee et al. 2000)

For triaxial tests on soils, the interpretation of ϕ' can be evaluated on the basis of different criteria, including (a) maximum deviator stress (q_{max}); (b) maximum obliquity $(\sigma_1'/\sigma_3')_{max}$, where σ_1' and σ_3' are the major and minor principal effective stresses, respectively; or (c) value taken at large strains (≈ 20%). For instance, representative triaxial test results including effective stress path and stress-strain curve for an Atlantic offshore sediment are illustrated in Figure 3 and the effective stress friction angle defined at different criteria are presented. The value of friction angle using the three criteria above show: (i) ϕ' = 32.2° at peak shear stress; (ii) maximum obliquity gave ϕ' = 32.1°, and (iii) ϕ' = 32°at large strains. For this soil, all three criteria output very comparable value of effective stress friction angle.

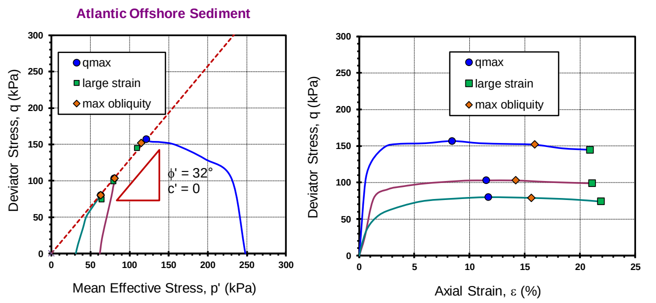

Figure 3 Comparison of three criteria for defining magnitude of ϕ" for an Atlantic offshore sediment from triaxial test results (data from Olsen et al. 1986)

The parameter c' indicates the existence of tensile strength, which is characteristic of cemented soils, including those of certain mineralogical constituency (e.g. carbonates, calcareous components). More often, however, many soils are uncemented and it is found that c' = 0 applies to a large majority of young soils, including normally consolidated (NC) to lightly overconsoldiated (LOC) soft clays, silts, and loose sands of Holocene age (Leroueil & Hight 2003; Mesri & Abdel-Ghaffar 1993; Lade 2016). Often, these uncemented sediments are composed of common minerals such as kaolin, illite, quartz, silica, feldspar, and chlorite. Thus, for soft to firm clays that are normally-consolidated (NC) to lightly-overconsolidated (LOC), the adoption of c' = 0 is a reasonable assumption for many sedimentary deposits.

1.2 Triaxial versus Piezocone

In most circumstances, a total stress analysis (TSA) is adopted for the interpretation of CPTu data in clays. Yet, an effective stress analysis (ESA) is preferred in order to permit a more fundamental assessment of soil behavior, including the quantification of ϕ' in clays. TSA only requires one measurement, and this is used to assess the undrained shear strength, su. In the laboratory, the evaluation of · ' in clays requires either drained triaxial tests (Δu = 0), or undrained shear tests with porewater pressure measurements. During CPTu, soft to firm to stiff intact clays will exhibit excess porewater pressures during penetration tests (Δu > 0, where $\Delta u = u_2 - u_0$).

A conceptual comparison between the triaxial compression test and the piezocone penetration test is depicted in Figure 4. Notably, the same equipment and instrumentation are used for both tests, including: axial load cell, filter elements, pore pressure transducers, and depth/displacement sensors. During a triaxial test, an extracted soil specimen is chosen to represent a single point in the field. An all-around confining stress (σ_3) is applied to the specimen by means of chamber fluid and a back pressure (u) is applied to simulate groundwater conditions. An added deviator stress ($\Delta\sigma = q = \sigma_1 - \sigma_3$) is then applied to the specimen in the axial direction to impart shearing. Drainage from the specimen can either be allowed or prevented, depending on the desired test conditions. The results from the triaxial test provide information for a sole point in subsurface space from the clay deposit. Boundary conditions are well established.

In contrast, a typical piezocone test sounding taken to 30 meters depth results in the testing of billions of soil particles within the clay deposit. Axial load is converted to cone resistance over the cross-sectional area of the penetrometer. Via the cylindrical porous filter element, porewater pressures are recorded by a transducer. However, boundary conditions within the soil matrix for the CPTu are not exactly well-established. Regardless, the similarities in triaxial apparatus and cone penetrometer are quite apparent, with benefits and drawbacks to both tests detailed above.

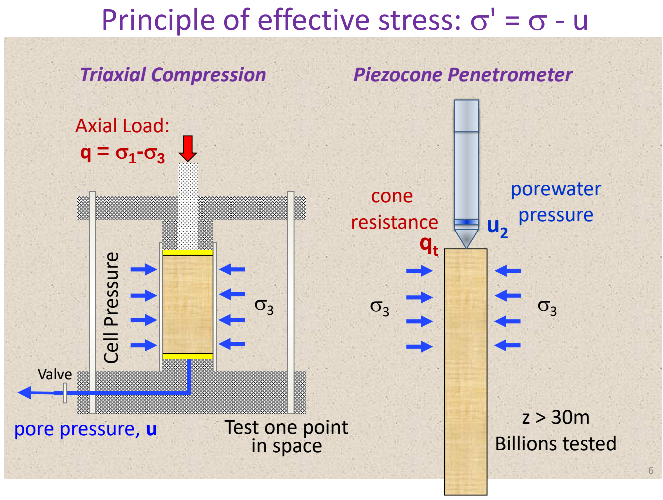

Figure 4 Conceptual comparison between triaxial apparatus and the piezocone penetrometer

2. CPTU EFFECTIVE STRESS SOLUTION FOR CLAYS

An effective stress limit plasticity solution that utilizes the total cone resistance (qt) and measured porewater pressure (u2) has been developed by the Norwegian Institute of Technology (NTH), as detailed by Janbu & Senneset (1974), Senneset et al. (1982), Senneset & Janbu (1985), Senneset et al. (1989), Sandven (1990), and Sandven et al. (2016). The general NTH solution for normally consolidated clay where the cohesion intercept c' = 0 is given by:

$$N_m = \frac{\tan^2(45°+\varphi'/2)\cdot \exp(\pi\cdot \tan\varphi')-1}{1+6\cdot \tan\varphi'\cdot(1+\tan\varphi')\cdot B_q} \tag{3}$$

For convenience, the CPTu results can be presented in terms of two dimensionless parameters:

Cone resistance number: $N_m = q_{net}/\sigma_{v0}'$ (4)

Normalized porewater pressure: $B_q = \Delta u_2/q_{net}$ (5)

Where $q_{net} = (q_t - \sigma_{v0})$ is the net cone tip resistance and $\Delta u_2 = (u_2 - u_0)$ is the excess porewater pressure.

For overconsolidated clays, a modification to the original NTH solution is necessary in order to account for stress history effects on the measured CPTu data (Sandven et al. 2016). A revised definition of Equation (4) gives the modified cone resistance number (N_{mc}):

$$N_{mc} = \frac{q_t - \sigma_{vo}}{\sigma_e'} \tag{6}$$

in which σ_e' is called the equivalent stress and determined from:

$$\sigma_e' = \sigma_{v0}' \cdot OCR^{\Lambda} \tag{7}$$

where OCR = σ_p'/σ_{vo}' = overconsolidation ratio, σ_p'= effective preconsolidation stress, Λ = 1 – Cs/Cc = plastic volumetric strain potential, Cs = swelling index, and Cc = virgin compression index.

The equivalent stress concept was originally detailed by Hvorslev (1960) based on direct shear box tests and later explained in terms of triaxial shear tests and critical state soil mechanics (e.g., Schofield & Wroth 1968; Mayne et al. 2009).

While Λ can be theoretically calculated from consolidation test data, operational values of Λ are best determined as the slope of $\log(s_u/\sigma_{vo}')$ vs. $\log$ (OCR) plots obtained from the specific mode of study, such as triaxial compression tests (Mayne 1980). If the OCR is unknown, a plot of $\log(s_u/\sigma_{vo}')$ vs. $\log(1/\sigma_{vc}')$ can be used as long as the effective confining stresses (σ_{vc}') are below the natural σ_p' value (Mayne & Swanson 1981). For natural clays that are insensitive and inorganic, these values have been shown to be somewhat dependent on shear mode, including $\Lambda \approx 0.72$ (triaxial compression), $\Lambda \approx 0.80$ (simple shear), and $\Lambda \approx 0.88$ (triaxial extension), as shown by Kulhawy & Mayne (1990).The profile of overconsolidation ratio (OCR) is best assessed from a series of laboratory consolidation tests on samples taken from various depths (Sandven 1990; Mayne & Pearce 2005). Alternatively, the OCR be found from an interpretation of triaxial data (Mayne 1988), vane shear or flat dilatometer tests (Kulhawy & Mayne 1990), or even from the CPTu results themselves (Lunne et al. 1997; Mayne 2007; Robertson 2009).

For both NC and OC clays, equation (7) can be used directly to determine ϕ' by substituting $N_m = N_{mc}$ from CPTu soundings.

3. CASE STUDIES

Two case studies are presented to show the use of NTH solution for evaluating ϕ' in clays.

3.1 Mini-CPTu in Calibration Chambers of NC Kaolin:

Miniature piezocone penetration tests were carried out in normally consolidated (NC) deposits of Speswhite kaolin using a calibration chamber (Hird & Sangtian 2003). The miniature piezocone has a shaft diameter of 11.3mm and an apex angle of 60°. The instrument can measure vertical force on the end of the cone and the porewater pressure at the shoulder position. The Speswhite kaolin layer is about 160mm high with the following index properties: liquid limit=61%, plastic limit=31%, specific gravity=2.60, and clay fraction=80%. Information on the engineering properties of Speswhite kaolin and kaolin-sand mixes is documented in Rossato et al. (1992). The clay was mixed to a slurry at an initial moisture content of 120 % and consolidated one-dimensionally under a vertical stress of 200 kPa. Figure 5a shows the measured q_t and u_2 profiles. Figure 5b gives the corresponding interpreted profiles of N_m and B_q. It is shown that the average cone resistance $N_m = 2.0$ and the average $B_q = 0.74$.

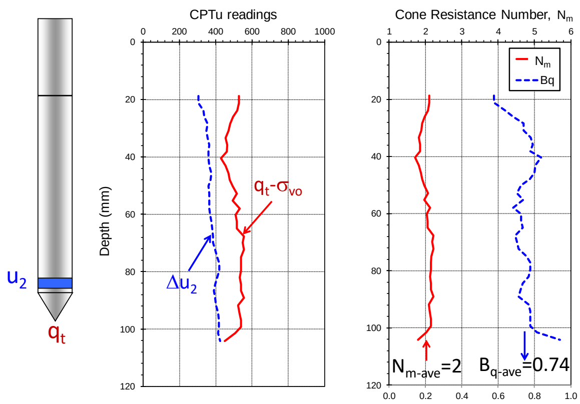

Figure 5 Miniature CPTu in calibration chamber of NC kaolin (data from Hird & Sangtian 2003)

Applying the paired values of N_m and B_q into Equation (3), the effective stress friction angle of the normally consolidated Speswhite kaolin is calculated as ϕ' = 23° with c' = 0, as suggested by Figure 6.

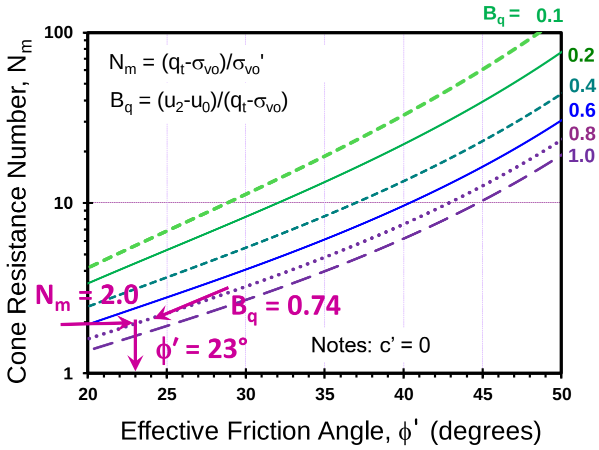

Figure 6 Evaluation of ϕ' for Speswhite kaolin in chamber test

Figure 7 shows series of triaxial compression tests on Speswhite kaolin reported by Vall-Marquez (2009). A mean effective friction angle value ϕ' = 22.6° with c' = 0 can be interpreted, which agree well with the value of friction angle interpreted from CPTu data using the NTH solution.

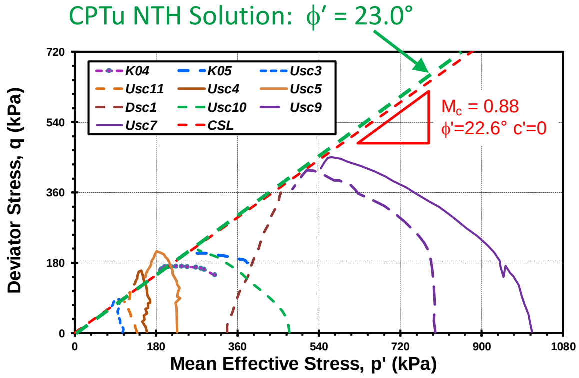

Figure 7 Triaxial tests on Speswhite kaolin clay (data from Valls-Marquez 2009)

An approximate NTH friction angle equation for intact clays with c' = 0 can be approximated when the following ranges are met: 20°≤ ϕ'≤ 45° and $0.05 \leq B_q \leq 1.0$.

$$\phi' = 29.5 \cdot B_q^{0.121}\,[0.256 + 0.336 \cdot B_q + \log N_{mc}] \qquad (8)$$

A comparison of the exact NTH solution and approximation is shown in Figure 8.

3.2 CPTu in OC marine clay from Anchorage

Results of piezocone tests in overconsolidated clay at the nearshore Port of Anchorage in Alaska are utilized to illustrate the application of the modified NTH method for evaluating ϕ'. Extensive in-situ and laboratory testing programs have been conducted for the Port of Anchorage Expansion involving use of a SeaCore jack-up platform for conducting soil test borings, laboratory triaxial testing, one-dimensional consolidation, piezocone penetration tests, vane shear, and downhole shear wave velocities to characterize the subsurface profiles (Pearce & Hale, 2004, Mayne & Pearce 2005). Basic mean index parameters on this overconsolidated clay include: natural water content wn = 20 to 31%, liquid limit LL = 45%, and plasticity index PI = 24%. From the consolidation tests, the mean value of virgin compression index Cc = 0.242, swelling index Cs = 0.060, giving = 0.75, which is reasonable for low to medium sensitive clays (Kulhawy & Mayne, 1990). Figure 9 shows a representative piezocone sounding and a corresponding profile of the overconsolidation ratio (OCR) with depth below mean water line.

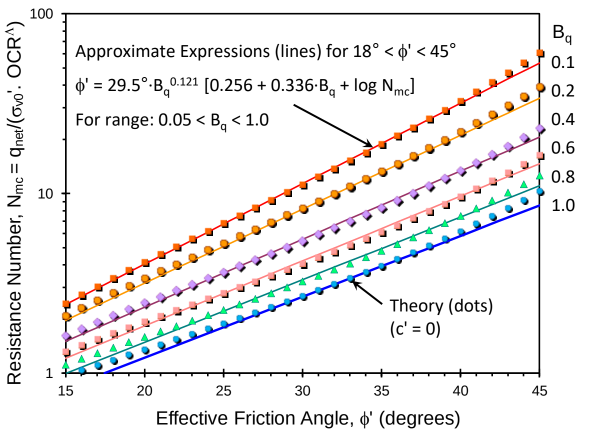

Figure 8 Approximate modified NTH method for evaluating ϕ' in intact clays

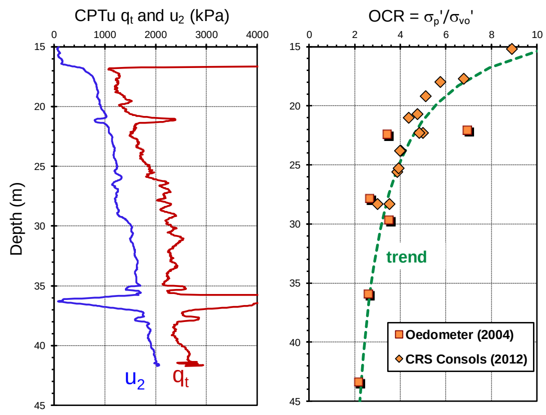

Figure 9 Representative profiles at Port of Anchorage: (a) CPTu results; (b) overconsolidation ratio from lab consolidation tests (data from Mayne & Pearce 2005)

A series of 19 lab specimens were subjected to isotropically - consolidated type triaxial compression tests with porewater pressure measurements (CIUC) and the effective friction angle ϕ' can be interpreted as ϕ' = 28° with c' =0, as shown by the effective stress paths in Figure 10.

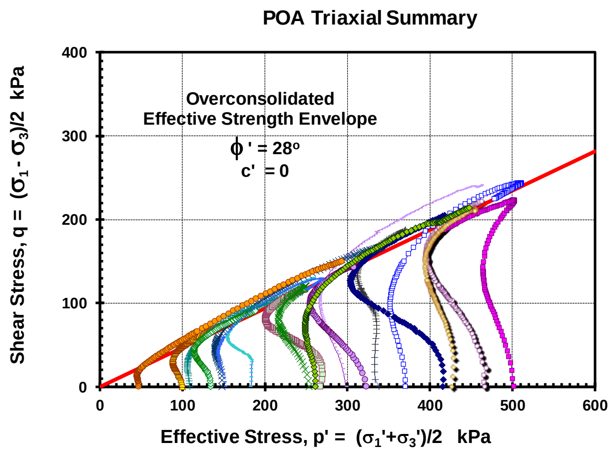

Figure 10 Summary of CIUC triaxials on overconsolidated clay at Port of Anchorage (data from Mayne & Pearce 2005)

Following the procedure for determining both the original cone resistance number Nm and the modified cone resistance Nmc, along with calculation of the normalized porewater pressure Bq, the effective stress friction angle ϕ' profiles calculated using the CPTu data are presented by Figure 11. It can be observed that the original NTH solution generates an unrealistically high friction angle profile (> 42°). The friction angle interpreted using modified NTH solution gave excellent agreement with the corresponding friction angle determined from the laboratory triaxial compression at corresponding elevation: ϕ' ≈ 28°.

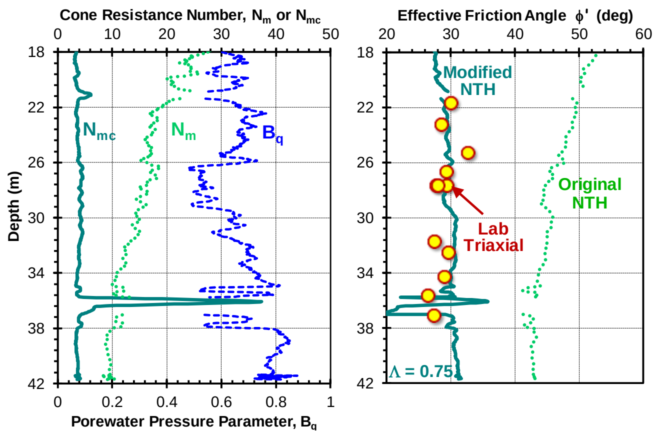

Figure 11 Profiles at Anchorage: (a) normalized CPTu parameters; (b) interpreted ϕ' from triaxial tests and CPTu using NTH solution

3.3 Triaxial – Piezocone database for clays

Ouyang and Mayne (2018) verified the limit plasticity NTH solution by calibrating the theory with data from global field sites underlain by mainly normally consolidated natural clays and clayey silts that were subjected to both field CPTu and laboratory triaxial testing. Later Ouyang & Mayne (2019) carried out calibration of the modified NTH solution applied to overconsolidated clays where both intact clays and fissured clays were investigated using piezocone penetrometers. A compiled database from 132 piezocone soundings in intact clays performed at field sites, 1-g chamber tests, and centrifuge series are compared with triaxial tests to show the full range of 18° < ϕ' < 45° of natural and artificial clays are achieved, as shown by Figure 12.

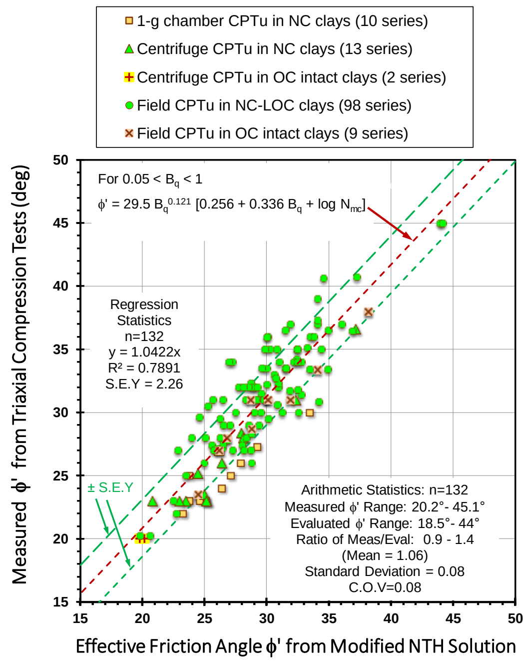

Figure 12 Summary plot of laboratory triaxial measured ϕ' versus CPTu ϕ' from different test series using modified NTH method

Overall, the measured laboratory values of ϕ' covers the range from 20.2° to 45.1° and the CPTu-evaluated ϕ' range extends similarly, from 18.5° to 44°. Both the statistical regression and arithmetic statistic of the measured/interpreted ratios generally support that the modified NTH solution yield a comparable evaluation of the effective stress friction angle of normally consolidated and overconsolidated intact clays when referenced to the benchmark triaxial value.

4. CONCLUSIONS

A modified effective stress limit plasticity solution is presented in this paper for the evaluation of the effective stress friction angle (ϕ') of normally-consolidated to overconsolidated intact clays from piezocone penetrometer tests (CPTu). The solution takes account of stress history effects by introducing an equivalent stress concept derived from critical state soil mechanics (CSSM). Case studies involving mini-CPTu in 10 series of laboratory chamber tests, 15 series centrifuge test series, and 107 field CPTu soundings in natural NC and OC clay deposits are used to validate the applicability of the NTH solution. For clays that are intact, insensitive, and inorganic, the NTH solution for evaluation ϕ' from CPTU is found to be in good agreement and comparable in values with the laboratory benchmark values obtained from triaxial compression tests.

5. ACKNOWLEDGMENTS

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