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Undrained Shear Strength From Piezocone Tests

by J.-M. Konrad and K.T. Law

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Undrained shear strength from piezocone tests

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With the advent of piezocones — penetrometers measuring both the mechanical resistance and induced pore pressures near the tip during penetration into the soil — a new interpretation of penetrometer test data is possible. This paper presents a review of available interpretation methods for obtaining the undrained shear strength of soft soils, and introduces a new interpretation taking into account measured pore pressures. The undrained shear strength is considered to be solely related to the ultimate cavity expansion pressure, which is one of the components of the tip resistance. The other component is calculated assuming that effective friction is developed at the cone-soil interface.

Parametric studies on the parameters required for strength determination based on the proposed method are also presented. Special self-boring pressuremeter tests to obtain relevant values of soil rigidity index, which is a key parameter for cavity expansion modelling, are described. Tests were conducted at three sites having the characteristics of soft sensitive clay, stiff sensitive clay, and clayey silt. The operational undrained strength mobilized during the cone insertion derived from the proposed model is equal to or lower than the CK_0U triaxial strength, which depends on soil brittleness. The proposed approach yields results consistent with known soil behaviour at all three sites.

Key words: piezocone penetrometer, pressuremeter, excess pore-water pressure, undrained shear strength, rigidity index, *in situ* tests, laboratory test.

L'utilisation du piézocône permet une nouvelle interprétation des résultats du test de pénétration grâce aux mesures combinées de la résistance en pointe et de la pression interstitielle générée près de la pointe. Cet article revoit les principales méthodes d'interprétation en vue d'obtenir la résistance au cisaillement non drainé et présente une nouvelle méthode qui tient compte des pressions interstitielles. La résistance au cisaillement non drainé est calculée à partir de la pression ultime d'expansion de cavité qui est une des deux composantes de la résistance à la pénétration. Le frottement développé le long de l'interface cône-sol, qui est déterminé en fonction des contraintes effectives, est la deuxième composante.

L'article présente également les résultats d'études paramétriques et décrit des essais spéciaux de pressiomètre autoforeur permettant la détermination de l'indice de rigidité du sol correspondant aux conditions d'expansion de cavité non drainées. Des essais ont été effectués à trois sites présentant les caractéristiques d'argile molle et sensible, d'argile raide et sensible et de limon argileux. La résistance au cisaillement globale mobilisée pendant la pénétration calculée en utilisant les équations de la méthode proposée est égale ou inférieure à la résistance d'essais triaxiaux CK_0U , qui est dépendant de la fragilité du sol. La méthode proposée produit des résultats en accord avec le comportement observé dans d'autres études pour les sols des trois sites.

Mots clés: pénétrömètre piézocône, pressiomètre, pression interstitielle, résistance au cisaillement non drainé, indice de rigidité, essais *in situ*, essais au laboratoire.

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Introduction

The continuing research for oil and gas resources has led to increased exploration activity in the Arctic offshore regions. The drilling of deep wells has been carried out to date from man-made artificial islands. The presence of very soft soils in several offshore Arctic and subarctic areas causes a major concern in the design of these gravity structures, particularly in view of the very large ice loads exerted on the structures.

The geotechnical characteristics of these soft deposits are usually obtained from site investigations and laboratory testing. Cone penetrometer tests in soft soils, particularly offshore, where the use of other *in situ* tests is severely limited, are mainly conducted to determine the subsurface stratigraphy. Indeed, the cone penetrometer can provide continuous profiles of tip resistance, sleeve friction, and, more recently, pore pressures generated during pushing. Furthermore, the relative rapidity of testing and mobility of equipment render it quite cost effective. While recent electronic developments have greatly enhanced the accuracy of the results, the actual interpretation of the test with respect to undrained shear strength, overconsolidation ratio, and deformation and consolidation characteristics is still an area of active research.

Since the foundation design for these structures must consider the stability during set down and during their lifetime under the environmental loads, this paper will focus on the determination of undrained shear strength from piezocone penetrometer tests in fine-grained soils. The paper first presents a critical review of available interpretation methods, followed by a theoretical model that estimates the total soil pressure normal to the cone face from the measured total cone resistance and induced pore pressure. Assuming that this pressure is equal to the ultimate pressure of a spherical cavity expansion process, the undrained shear strength mobilized during cone insertion can be derived from known cavity expansion theory in elastoplastic medium. Since this approach requires the soil's rigidity index, the paper also devotes attention to factors that influence determination of the rigidity index from laboratory and self-boring pressuremeter tests.

In another section an experimental study deals with the application of the proposed interpretation to soils from three different sites ranging from soft to stiff clays to silty soils. The undrained shear strength derived from piezocone tests is compared with the strength obtained from field vane and laboratory tests on undisturbed samples. The results of existing interpretations are also shown for one penetration data from each site.

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TABLE 1. Theoretical bearing capacity factor for deep circular foundations

$N_c (\phi = 0)$	σ_i	Solution class	Remarks	Reference
7.41	σ_{vo}	1		Terzaghi (1943)
7.0	σ_{vo}	1		Caquot and Kerisel (1956)
9.34	σ_{vo}	1	Smooth base	Meyerhof (1951)
9.74	σ_{vo}	1	Rough base	
9.94	σ_{vo}	1		de Beer (1977)
$\frac{4}{3} \left[1 + \ln \frac{E_t}{3S_u} \right] + 1$	σ_{vo}	2	SCE E_t : Initial tangent modulus	Meyerhof (1951)
$\frac{4}{3} \left[1 + \ln \frac{E_s}{S_u} \right] + 1$	σ_{vo}	2	SCE E_s : secant modulus at 50% failure stress	Skempton (1951)
$\frac{4}{3} \left[1 + \ln \frac{E_s}{3S_u} \right] + \cot \theta$	σ_{vo}	2	SCE	Gibson (1950)
$\frac{4}{3} \left[1 + \ln \frac{E_s}{S_u} \right] + \cot \theta$	σ_{vo}	2	SCE, finite strain theory	Gibson (1950)
$\frac{4}{3} [1 + \ln I_R]$	σ_{vo}	2	SCE	Vesic (1972)
$\frac{4}{3} [1 + \ln I_R] + 2.57$	σ_o	3	SCE	Vesic (1975)
$[1 + \ln I_R] + 11$	σ_{ho}	3	CCE	Baligh (1975)
$\frac{S_a}{S_u} + \frac{4}{3} \frac{S_r}{S_u} \left[1 + \ln \frac{E_r}{3S_{ur}} \right] + \frac{4}{3}$	σ_{vo}	4	Trilinear stress-strain relationship	Ladanyi (1967)
$\left[\frac{E_u/S_u - E_r/S_{ur} \cdot S_{ur}/S_u}{E_u/S_u - E_r/S_{ur}} \right] \ln \frac{E_u S_{ur}}{S_u E_r}$				

NOTE: SCE: spherical cavity expansion; CCE: cylindrical cavity expansion; I_R : rigidity index = $G_u/S_u = E_u/3S_u$; σ_o : mean normal total stress = $(\sigma_{vo} + 2\sigma_{ho})/3$; θ : semiapex angle.

Review of existing theories on cone resistance

This section reviews theories seeking to relate the unit cone resistance (q_c) to the undrained shear strength (S_u) for incompressible and fully saturated soils. Most studies into the interpretation of cone penetration tests have fallen into one of the following classes:

- (1) The classical plasticity approach to bearing capacity of deep foundations;
- (2) Cavity expansion theories combined with classical plasticity;
- (3) The principle of conservation of energy with cavity expansion;
- (4) Analytical and numerical approaches using linear and nonlinear stress-strain relationships;
- (5) Empirical studies attempting to relate cone resistance to undrained shear strength derived from different tests and, in some cases, to additional soil index parameters.

All known theories present the solutions for q_c using the well-known relationship

$$[1] \quad q_c = N_c S_u + \sigma_i N_q$$

where N_c and N_q are dimensionless bearing capacity factors and σ_i is the initial *in situ* total pressure. With the condition that $\phi = 0$, the factor N_q is equal to unity and [1] reduces to

$$[2] \quad q_c = N_c S_u + \sigma_i$$

Owing to the complexity of the penetration problem, the methods of interpretation refer to approximate theoretical solutions based on simplified assumptions with respect to soil behaviour, shape of slip lines in the plastic zone, and boundary conditions. Class 1 solutions, based on the original work by Prandtl (1921), consider incipient failure of a rigid and plastic material and are strongly dependent on the assumed shape of the plastic zone. This is illustrated in Table 1, which presents values of N_c between 7.0 and 9.94 obtained by different investigators. In class 2 solutions it is assumed that the penetration of a pile or a cone is equivalent to expanding a spherical cavity from zero radius to a radius equal to the radius of the pile or cone in an infinite elastoplastic medium. Bishop *et al.* (1945) first suggested that the end-bearing capacity of a foundation at any given depth was of the same order as the maximum internal pressure in the spherical cavity. For fully saturated clays for which $\phi = 0$, the ultimate internal pressure (P_{us}) was expressed as

$$[3] \quad P_{us} = \frac{4}{3} \left(\ln \frac{E_u}{3S_u} + 1 \right) S_u + \sigma_{vo}$$

where E_u denoted the undrained Young's modulus of the clay and σ_{vo} the *in situ* total vertical stress.

FIG. 1. Assumed failure pattern under pile point: (a) after Vesic (1975); (b) after Baligh (1975).

Meyerhof (1951) extended their approach by considering that full cohesion was mobilized along the surfaces of the pile point and that E_u might be defined as the initial tangent to the triaxial stress-strain curves. Gibson (1950), using the large-strain theory of Swainger and considering full mobilization of cohesion along the surface of the pile point, proposed a modified equation (shown in Table 1) where E_u was conveniently defined as the secant modulus to one-half of the ultimate compressive strength. Skempton (1951) proposed using Gibson's equation with a 90° pile point ($\theta = 45^\circ$) for the bearing capacity of foundations.

Vesic (1975) suggested that the bearing capacity can be estimated by determining the ultimate pressure needed to expand a spherical cavity associated with a realistic failure pattern. Furthermore, the point resistance was assumed to be governed by the total octahedral normal soil stress (σ_o) and not by σ_{vo} . Based on the geometry and boundary conditions shown in Fig. 1a, an approximate value of N_c (Table 1) can be found if the average normal stress along the ring BD is assumed to be equal to P_{us} .

As pointed out by Vesic, in relatively dense soils the wedge associated with the pile tip pushes the radial shear zone II sideways into the plastic zone III; however, in relatively loose soil this wedge forces its way through the medium without producing other visible slip surfaces. Thus, the extension of Vesic's equation to soft soils should be done with caution. Class 2 solutions, like class 1 solutions, are extremely dependent on the shape of assumed slip surfaces.

An alternative approach has been proposed by Baligh (1975), who analysed the problem of continuous penetration as a steady-state situation. Considering that the work done by the

FIG. 2. Cone factor N_K vs. plasticity index of clays.

external force per unit area to push the cone a unit distance is equal to the sum of the work done to push the cone tip alone (wedge) at a constant velocity V over a unit distance and the work done to expand a cylindrical cavity as shown in Fig. 1b, Baligh arrives at the equation for a 60° cone shown in Table 1. This equation assumes that the deformable material surrounding the wedge is isotropic, massless, rigid — perfectly plastic, and has a Tresca yield criterion, whereas the expansion of a cylindrical cavity assumes an elastoplastic medium, where the initial stress state is equal to the total horizontal stresses at a given depth.

Ladanyi (1963) proposed an approach using actual stress-strain curves obtained from laboratory triaxial tests idealized as a series of straight segments. He showed that the strain field around the expanded cavity was independent of the soil properties. Levadoux and Baligh (1980) applied this condition to calculate stress paths around the cone. They obtained a strain field from steady-state flow of a nonviscous fluid around the penetrometer, from which strain paths were derived. Using a constitutive relationship for the soil and finite element analysis, stress paths can be calculated. The predicted values of cone resistance were corrected to account for cone roughness by adding the vertical component of the residual shear stress acting on the face of the cone. Fair agreement between prediction and observations was obtained in Boston blue clay. However, other analyses using finite elements and constitutive relations for soils (de Borst and Vermeer 1982; Sloan and Randolph 1982) showed that the predicted q_c is very sensitive to the type of elements used and the boundary conditions considered in the analyses.

In order to account for strain-softening behaviour of soils such as sensitive clays, Ladanyi (1967) proposed that the measured stress-strain relationship be idealized by a trilinear model. Based on this soil model, the expression proposed by Ladanyi (Table 1) relates cone resistance to the peak undrained shear strength, Young's modulus at small and large strain (E_u and E_r), as well as the residual or large-strain undrained shear strength (S_{ur}). Roy *et al.* (1974) suggested that the approach by Ladanyi may be used as a first approximation in sensitive clay but also pointed out that the determination of the residual characteristics by laboratory tests presents some difficulties.

FIG. 3. Assumptions for proposed model: (a) Proposed stress field acting on cone; (b) expansion of spherical cavity.

Finally, owing to the above-mentioned shortcomings of theoretical analysis of the cone, empirical correlations have been proposed in a form similar to that given by [2], namely,

$$[4] \quad q_c = N_K S_{u_{ref}} + \sigma_{vo}$$

where N_K is the empirical cone factor obtained from [4] using the measured q_c and a reference value of S_u . Reference tests currently used are mainly field vane tests, with or without Bjerrum's empirical correction, and laboratory triaxial compression tests. As reported by de Ruiter (1982), N_K is mainly affected by the type of penetrometer, the plasticity index (I_p), overconsolidation ratio (OCR), and the macrofabric of the clay. Typical values of N_K are 10–15 for normally consolidated and 15–20 for overconsolidated clays. Many investigators have chosen to relate N_K to I_p as shown in Fig. 2. The data show significant scatter and suggest that further analysis should be made before utilizing the empirical relationships.

Proposed method to determine S_u from piezocone tests

Assumptions and derivation of S_u

As discussed in the previous section, the predicted values of S_u differ substantially, depending on the assumed shape and location of the plastic zone. It is also essential that the computation of S_u should be based on a realistic failure pattern. Thus, the development of any model requires a set of assumptions, which, in turn, results only in an approximate solution.

The theory of cavity expansion has been applied by Gibson (1950) to the problem of bearing capacity of deep foundations and can, therefore, be applied to cone penetration. However, several assumptions are necessary to take into account the change in direction of stresses and strains in the vicinity of the cone tip. As illustrated on Fig. 3a, it can be considered that the cone is acted upon by a uniformly distributed normal soil pressure (σ_n), normal to the cone, and by a shear stress along the cone (τ), mobilized during penetration. The shear stress along the cone has been introduced by Gibson (1950), who assumed that full cohesion could be mobilized along the failure surface, taken as a hemisphere of pile radius. The unit shear stress is taken as the undrained shear strength of the soil (S_u). Ladanyi and Eden (1969) assume that the value of the shear stress is equal to the remoulded adhesion between the cone surface and the clay, i.e., typically about half the undrained shear strength.

FIG. 4. Influence of K_0 on strength using proposed model.

Recent studies on pile foundations have indicated that the mobilized shaft friction in clays could be interpreted in terms of effective stress parameters (Bozozuk 1972; Burland 1973). Furthermore, Konrad and Roy (1987) showed that this concept is also valid at various stages during the reconsolidation phase. Hence, it can be assumed that the unit shear stress at the cone–soil interface can be expressed as

$$[5] \quad \tau = (\sigma_n - u_m) M \tan \phi'$$

where ϕ' is the effective friction angle of the normally consolidated soil and M is the friction factor for the soil acting on the cone surface.

Since the pore-water pressure is measured above the base of the cone, the pore pressure in the failure zone (u_m) can be approximated by

$$[6] \quad u_m = \alpha u$$

where α is a factor usually greater than 1. In soft soils, available data for measured pore pressures at both locations (Roy *et al.* 1982) indicate that α is in the range of 1–1.1. Considerations of force equilibrium at failure give the following expression for a cone with apex angle θ (Fig. 3a):

$$[7] \quad q_t = \sigma_n + M \tan \phi' (\sigma_n - \alpha u) \cot \theta$$

Introducing $F = M \tan \phi' \cot \theta$, which can be viewed as an overall soil–cone parameter, [7] results in

FIG. 5. Operational undrained shear strength mobilized by the cone in different soil types: (a) normally consolidated; (b) work softening; (c) work hardening.

$$[8] \quad \sigma_n = \frac{q_t + F\alpha u}{1 + F}$$

As suggested by the many researchers mentioned in the previous section, σ_n can be assumed to be equal to the maximum internal pressure required to expand a spherical cavity from zero radius to the cone radius R as illustrated in Fig. 3b.

The problem of the expansion of cavities in ideal soils has been widely studied and the solution is based on the following assumptions for the $\phi = 0$ case:

- rigid-plastic incompressible soil in a plastic zone,
- no volume change in the plastic region,
- linearly deformable soil in an elastic zone characterized by constant values of Young's modulus (E) and Poisson's ratio (ν),
- homogeneous, isotropic material.

It is further assumed that prior to cavity expansion the entire soil mass has an isotropic initial total stress (σ_i) and that the soil is weightless. Considering a Tresca yield criterion, *i.e.*, independent of the hydrostatic pressure, the maximum internal pressure (P_{us}) is then given as

$$[9] \quad P_{us} = \frac{4}{3} (\ln I_R + 1) S_{uc} + \sigma_i$$

where S_{uc} is the undrained shear strength mobilized by the cone, and I_R is the soil's rigidity index (Hill 1950; Vesic 1972).

Combining [8] and [9] yields the undrained shear strength mobilized during the penetration process:

$$[10] \quad S_{uc} = \frac{q_t - (1 + F) \sigma_i + F\alpha u}{\frac{4}{3}(1 + F) (\ln I_R + 1)}$$

Evaluation of parameters

Several parameters are required for strength determination based on the proposed method summarized by [10]. Only q_t and u can be determined from piezocone tests, while the other parameters, *i.e.*, σ_i , M , α , $\tan \phi'$, and I_R , have to be evaluated independently.

The porous element in the piezocone used in this investigation is located above the conical tip, where water pressure can act over a part of the base area of the cone in a

direction opposite to the resisting forces developed during pushing. Hence, the measured total stress (q_c) will be reduced. To obtain the actual total stress (q_t), a correction taking into account the design of the cone must be applied. The actual cone resistance is obtained as follows (Campanella and Robertson 1981):

$$[11] \quad q_t = q_c + (1 - a)u$$

where a is a bearing net area ratio.

The initial total stress σ_i has been assumed to be spherical and should therefore be approximated by the *in situ* total octahedral stress (σ_o). This stress requires the determination of both vertical and horizontal total stress. While σ_{vo} may be readily determined, σ_{ho} may present some difficulty. Taking σ_{vo} as σ_i in [10] instead of σ_o will introduce an error that is mainly dependent on the coefficient of earth pressure at rest (K_o). It is clear that for $K_o = 1.0$, the use of σ_{vo} would be correct. However, for K_o less than 1.0, as for many soft clays, the error is generally small and is on the safe side. This is best illustrated by considering a typical soft clay for which $q_c = 600$ kPa, $u = 400$ kPa, $\sigma_{vo} = 160$ kPa, $\phi' = 27^\circ$, $M = 1.0$, and $I_R = 120$. As shown in Fig. 4, the ratio S_{uo}/S_{uc} varies only from 1.0 for $K_o = 1$ to 1.06 for $K_o = 0.5$, where S_{uo} is the shear strength calculated using [10] considering σ_o and S_{uc} corresponding to the case $\sigma_i = \sigma_{vo}$. Therefore, if K_o is not known, it is justified to use the total vertical stress as the initial isotropic total stress required in [10].

In [7], $M \tan \phi'$ represents the coefficient of friction between the soil and the cone face. According to measurements made by Potyondy (1961) and Bozozuk *et al.* (1979), M can be taken between 0.5 and 0.75 for smooth steel surfaces. For usual roughness, M can be taken equal to unity. Within this range, the variation of S_u according to [10] is again small, higher values of M resulting in lower values of S_u . Parametric studies have also shown that a variation of 2° in the effective friction angle has little influence on S_u . Hence for general application, $\phi' = 35^\circ$ is suggested as an average value of effective friction angle for silty soils and $\phi' = 27^\circ$ for clays.

The rigidity index (I_R) is one of the parameters that influence the undrained shear strength. Different methods used to obtain relevant values of I_R are described later.

FIG. 6. Geotechnical profile at Gloucester.

Operational strength

Owing to the assumptions of the model, the relationship of [10] yields the maximum undrained shear strength mobilized around the cone in an ideal elastoplastic soil, which is adequate for normally consolidated clays (Fig. 5a). However, if the actual soil behaviour is either work softening or work hardening, the proposed interpretation yields an operational shear strength. It can be viewed as an intermediate value between peak and residual strength for work-softening soils (Fig. 5b), and for work-hardening media the operational strength is intermediate between yield and maximum strength (Fig. 5c). Vesic's and Baligh's interpretations also result in an operational strength, since elastoplastic behaviour is considered in their models. As discussed earlier, Ladanyi's approach is the only one yielding the peak strength in work-softening soils (Ladanyi and Eden 1969).

The calculated peak strength, however, is a function of the assumed ratio of residual to peak strength (S_{ur}/S_{up}), which in turn is stress path dependent (Law 1981b). It must be stressed that in Ladanyi's method this ratio is determined from laboratory tests under stress paths different from that mobilized during cone insertion. Thus, the strength determined using Ladanyi's approach may not truly reflect the peak value.

Experimental study

Site description

Three sites were chosen to study the use of the proposed method for strength determination: Gloucester, Ontario; the Ottawa Sewage Treatment Plant (STP) in Ottawa, Ontario; and Fredericton, New Brunswick. The Gloucester site (Fig. 6) is located 21 km southeast of Ottawa. It is a deposit of highly sensitive, soft Champlain Sea clay formed in different stages of erosion and deposition (Bozozuk and Leonards 1972). The clay is about 21 m thick, underlain by glacial till. It is slightly overconsolidated with a ratio (OCR) of 1.5. The plasticity

index ranges from 20 to 30% and the average clay content is about 80%. The Ottawa STP site (Fig. 7) is located at the eastern outskirts of Ottawa, on a firm to stiff Champlain Sea clay about 23 m thick. The top 3 m is a surface crust and the underlying clay has an average OCR of 3.3, average plasticity index of 20%, and clay content of 66%. The Fredericton site (Fig. 8) is located in downtown Fredericton. The site consists of fill of about 3 m underlain by a sand layer to a depth of 5.5 m and then 30 m of clayey silt, which has an average plasticity index of about 10%. The clay size content is about 50% and includes some fine particles of quartz. The OCR of the clayey silt is uncertain because of the difficulty of obtaining high quality undisturbed samples of the determination of representative *in situ* compressibility.

A GMF piezocone, a Cambridge self-boring pressuremeter, and a Geonor vane were used at each site. Test holes were located within 5 m of one another. At the Gloucester site, 127 mm diameter Osterberg samples were taken for laboratory tests. At the Fredericton site, a hollow-stem auger was used in the fill and provided a casing through which the tests were carried out.

Data acquisition system

A computer system was used in conjunction with the data acquisition system for the piezocone test and the pressuremeter test. Details of this system were described by Law (1985). Briefly, it comprises a personal computer, a data acquisition unit (DAU), and a plotter housed in a mobile laboratory (Fig. 9). For the cone penetrometer tests, readings are taken with the DAU and processed on site, taking into account the appropriate corrections to yield the tip resistance, sleeve friction, pore-water pressure, and depth of penetration. The undrained strength was calculated using [10]. All results can be plotted on site following the completion of the penetration.

An additional feature of the computer system is its ability to automatically control pressuremeter tests. Five test types have

FIG. 7. Geotechnical profile at Ottawa Sewage Treatment Plant (after Eden 1971).

been programmed to date. Control of pressuremeter tests is achieved by means of an electric-to-pneumatic (E/P) transducer, which receives an electrical signal from the computer and regulates a pressure output to the pressuremeter in proportion to that signal. The computer continuously takes readings, converts them to engineering quantities, and checks the current test conditions against the given ones. An appropriate signal to the E/P transducer is issued if required to modify the pressure to the pressuremeter so that the prescribed test conditions are followed. A constant strain rate test was chosen for the present study. At the South Gloucester and Ottawa STP sites, the strain rate was 0.16%/min, while at the Fredericton site a higher rate of 0.5%/min was used because the soil is more permeable. Other details of the pressuremeter have been described by Eden and Law (1980).

Piezocone penetrometer (CPT)

Electric GMF 25 and 50 kN cone penetrometers fitted with 60° tapered, 10 cm² area cones were used to measure tip resistance (q_c) and sleeve friction (f_s). The pore-water pressures were measured through a 4 mm thick cylindrical porous stone located immediately above the conical tip. The rate of penetration for all tests of this study was 1 cm/s, which permits an averaging of 10 measurements for every 2 cm depth recorded. Tests conducted at a standard rate of penetration, i.e., 2 cm/s, did not indicate any change in either q_t or u profile.

As pointed out by de Ruiter (1982), one major problem with cone design has been the difficulty in combining ruggedness with adequate sensitivity for penetration testing in soft fine-grained soils. Thus, if a tip with a capacity of 25 or 50 kN is used in a 2 kN range typical for most soft soils, considerable care in the testing procedure is required to obtain meaningful

data. The testing procedure adopted in the present investigation is described by Konrad (1986).

Pore pressures generated around the advancing cone penetrometer are sensed with a pressure transducer installed within the penetrometer about 3 cm above the base of the cone. As pointed out by Zuidberg *et al.* (1982) the transducer should be as rigid as possible, it should have a minimum fluid-filled space, and the design should be optimized for deairing. Proper probe saturation is of paramount importance for reliable dynamic pore pressure measurements. The filter elements should be deaired under vacuum and saturated with glycerin in the laboratory. Furthermore, each filter should be kept immersed in glycerin until used. The cavity inside the penetrometer is also filled with glycerin and a rubber membrane is pulled over the tip while the tip is totally immersed in glycerin. Loss of saturation during the installation phase or when traversing dilative dense sand layers is avoided (Campanella *et al.* 1982) owing to the improved surface tension properties of the glycerin. In contrast to the tip loads, which are only about 2–4% of the design capacity, the pore pressure developed in soft soils is generally recorded with transducers close to their design capacity, resulting in relatively accurate measurements.

Determination of rigidity index

Two methods have been used to determine the rigidity index in this study. The first method was applied to laboratory test data on Gloucester clay using the undrained triaxial test (CK_0U) with anisotropic consolidation stresses equal to those in the field as measured by Bozozuk (1972). Stress-strain relationships were determined; these yielded an undrained shear strength and any secant modulus, from which the rigidity index was evaluated. The second method was used on data

FIG. 8. Soil profile at Fredericton (after Alkins 1983).

TABLE 2. Results of triaxial tests on anisotropically consolidated undrained Gloucester clay

Depth (m)	Undrained shear strength (kPa)	Secant modulus (MPa)		Rigidity index (2)†
		(1)*	(2)†	
2.36	23.8	2.11	12.54	176
3.10	26.7	3.08	11.75	147
3.96	24.9	2.15	6.25	84
4.72	29.3	2.78	11.75	134
5.30	31.0	2.39	14.97	161
6.15	34.9	3.59	11.76	112
7.67	37.9	8.17	17.62	155
9.19	39.6	5.83	17.46	147
10.72	46.4	6.96	18.61	134
12.19	50.9	7.06	17.56	115
13.41	51.6	7.82	21.93	142

*(1) At failure.

†(2) At midpoint between failure and beginning of undrained shear

from self-boring pressuremeter tests conducted at each of the three sites.

The undrained shear strength and the secant modulus at two shear stress levels from laboratory tests are shown in Table 2. The secant modulus is the slope of the line, on a stress-strain plot, joining the point at the beginning of undrained shear to the appropriate stress level. The modulus at the midpoint between failure and the beginning of undrained shear (E_{50}) is about three to six times higher than that at failure (E_f), indicating that the stress-strain behaviour is nonlinear.

FIG. 9. Computer system mounted inside mobile laboratory (after Law 1985). (1) personal computer, (2) plotter, (3) signal conditioner, (4) strip chart recorder, (5) switching board, (6) data acquisition unit, (7) DC power supply.

Most theories for the cone penetration test interpretation, including the one proposed herein, assume linear behaviour, whereas nonlinearity prevails in real soils. Hence, the theories require a single modulus value that must be determined from measurements. As mentioned earlier, the secant modulus at half the failure stress (E_{50}) and the initial tangent modulus (E_i) have been used in the past. The use of E_{50} is considered more reasonable because the modulus mobilized outside the plastic zone ranges from E_f at the plastic-elastic interface to E_i at infinity or a considerable distance away. The average mobilized value lies between these limiting values and is probably better approximated by E_{50} . The corresponding rigidity index derived from E_{50} is shown in Table 2.

A graphical method was introduced to obtain a pressuremeter rigidity index, I_{RP} , from the pressure-strain curve of the pressuremeter test. The index I_{RP} will be equal to I_R for some given conditions to be discussed later. Based on Palmer (1972), the shear stress (τ) at the pressuremeter-soil interface is given approximately by

$$[12] \quad \tau \approx \epsilon \frac{d\sigma_r}{d\epsilon}$$

where σ_r = total radial pressure, ϵ = strain = $\Delta a/a_0$, Δa = radial displacement at the interface, and a_0 = radius of pressuremeter before expansion.

In Fig. 10, σ_r is plotted against ϵ . Point A corresponds to the total *in situ* horizontal pressure and B is any point on the curve beyond A. A tangent to the curve is drawn through B to intersect the σ_r axis at C. The difference in ordinate between B and

FIG. 10. Graphical determination of rigidity index from pressuremeter test data.

FIG. 11. Typical pressuremeter test result and variation of rigidity index with strain level.

C is equal to $\epsilon (\partial \sigma_r / \partial \epsilon)$, or τ . CD is drawn parallel to the ϵ -axis, with D directly below B, so that $BD = \tau$. If an unloading-reloading cycle is carried out at B, the shear modulus (G) is equal to half of the gradient of the cycle (Wroth 1984). This cycle cuts CS at E, thus,

$$[13] \quad G = BD / (2DE) = \tau / (2DE)$$

or

$$[14] \quad I_{RP} = \frac{G}{\tau} = \frac{1}{2DE}$$

Therefore the pressuremeter rigidity index is equal to half of the reciprocal of DE . This graphical method was applied to all the pressuremeter results. The I_{RP} values were found to depend on the strain level (Fig. 11). To ascertain the condition for which $I_{RP} = I_R$, a controlled stress path plane strain test was conducted on an undisturbed sample of Gloucester clay. A computerized servo system described by Law (1981a) was used to simulate in the laboratory the effective stress path in the pressuremeter test. The result shown in Fig. 12 indicates that the laboratory test satisfactorily reproduces the field stress path.

The effective stress path from the pressuremeter test is significantly different from that of a conventional consolidated undrained triaxial test on a sample from the same depth. As shown in Fig. 12, the triaxial stress path bends towards the

FIG. 12. Effective stress paths for various tests.

FIG. 13. Shear stress - axial strain and volumetric strain - axial strain relations during the controlled stress path plane strain test.

origin after reaching a peak value, while the pressuremeter stress path bends to the right, reaching higher p' values and hence higher shear resistance. The stress-strain-volume change during the plane strain test for simulating the pressuremeter test is shown in Fig. 13. At axial strains less than 1% (equivalent to 1.33% strain in pressuremeter tests), the undrained condition is satisfied. Beyond this point, significant volume change takes place and this leads to straining at higher p' with the resulting shear resistance, corresponding to a partially drained strength. This strength is normally taken as the pressuremeter strength and is therefore higher than the undrained strength. The I_{RP} for this range will then be lower than I_R for the undrained condition. On the other hand, I_{RP} determined at the strain level around the point when drainage starts will correspond to the undrained condition and may yield a closer value to I_R . It is understood that since I_{RP} corresponds to

TABLE 3. Results of pressuremeter tests

Site	Depth (m)	Rigidity index
Gloucester	3.19	124
	4.69	133
	6.52	133
Ottawa Sewage Treatment Plant	7.29	113
	8.79	143
	10.29	120
Fredericton	9.57	155
	10.93	133
	12.91	123
	16.03	175
	17.23	156
	18.7	124

τ and I_R to S_u , I_{RP} may still be different from I_R . Values of I_{RP} from pressuremeter tests from the three sites were evaluated at different strain levels. The resulting values were used in [10] to obtain the undrained shear strength (S_{uc}). It was found that S_{uc} corresponding to I_{RP} at 10% pressuremeter strain is about 8–10% higher than that from I_{RP} at 1.5% pressuremeter strain. Furthermore, for the present study I_{RP} determined at 1.5% is compared with I_R from the high quality samples tested in the triaxial cell (Table 2). For soils at the same depth, the I_{RP} and I_R from both test series are practically equal. Therefore I_R is taken equal to I_{RP} at 1.5% for all the sites and the results are shown in Table 3.

The use of the pressuremeter to obtain I_R may appear complicated in using the piezocone for measuring S_u . The data from the present study however indicate a possibility that I_R may lie within a narrow range for a given deposit. In future when more test data are available, some generalization and simplification may be possible so that the pressuremeter test will no longer be needed. This generalization may cause a strength variation of about 10% and is definitely superior to the empirical approach.

Results of piezocone tests

The profiles of q_t and u for the three sites are plotted on the same scale in Fig. 14. There is a general increase in amplitude of variation in q_t and u with respect to silt content. The Gloucester clay has the least amount of silt and the q_t and u profiles are relatively smooth, while the widest variations are found in the Fredericton clayey silt. All these profiles provide a good means for identifying soil layers. Three layers can be observed at Gloucester, with demarcations at about 7 and 14 m. This is in good agreement with the extensive test results of Bozozuk and Leonards (1972). The profiles at the Ottawa Sewage Treatment Plant (STP) show a surface crust to a depth of 3 m. The next layer extends to 14 m, below which a third layer is found. This layering is consistent with what Eden (1971) found at the same site. The profiles at the Fredericton site give an excellent means of precisely locating silt or sand bands in the soil. Such bands are indicated by a sharp rise in q_t and an abrupt drop in u because of the free-draining and dilative nature of dense sands. These bands are quite frequent at depths above 20 m. Below this point, a more homogenous layer is found, with q_t roughly increasing linearly with depth.

Evaluation of operational strength from piezocone data

The evaluation of the operational undrained strength (S_{uc}) was made using the above q_t and u profiles, along with I_R

FIG. 14. Piezocone profiles: (a) Gloucester; (b) Ottawa Sewage Treatment Plant; (c) Fredericton.

determined from the pressuremeter tests. The results are compared with strengths determined using the methods of Vesic (1975) and Baligh (1975) in Figs. 15–17.

The profiles of undrained strength from the piezocone (S_{uc}) are in general agreement with those from the vane test (S_{uv}). At the Ottawa STP and Fredericton sites, S_{uc} agrees well with S_{uv} , while at the Gloucester site, S_{uc} is consistently higher than S_{uv} by about 10%. The peak strength from CK_0U triaxial tests on Osterberg samples and on a block sample (Law 1975) is in good agreement with S_{uc} at the Gloucester site.

At the STP site no CK_0U tests were performed. Instead, comparison was made with unconsolidated undrained (UU) tests carried out by Eden (1971) on block samples. The peak strength from these tests exceeds S_{uc} at depths above 10 m and is about equal to S_{uc} at lower elevations. Based on experience with other stiff Champlain Sea clays (Law 1975), the peak strength from CK_0U test is higher than that from a UU test. The S_{uc} at this site, therefore, would be lower than the peak CK_0U strength.

Results of strength interpretations based on other theories are also shown in Figs. 15–17. For Gloucester, using Ladanyi's model (1967) and parameters from Ladanyi and Eden (1969), with the cone resistance shown in Fig. 10, gives a peak strength 60% higher than the peak strength from the CK_0U tests. Such high peak strength cannot be attributed solely to rate effect. Baligh's method (1975), on the other hand, yields the lowest strength of all the sites. At the STP site, for instance, this amounts to only about 35% of the vane strength. Vesic's method (1975) provides an undrained strength that is reasonably close to that of the present proposed model. At the Gloucester site these two strengths are almost identical with Vesic's strength, slightly lower than that of the proposed interpretation. At the STP site, Vesic's method yields a slightly higher value in the crust, where $u = 0$, and a slightly lower value below the crust. At the Fredericton site, strength determined from Vesic's method slightly exceeds that from the present model for the top 20 m and the strengths are equal

FIG. 15. Strength profiles at Gloucester site.

FIG. 16. Strength profiles at Ottawa Sewage Treatment Plant.

FIG. 17. Strength profiles at Fredericton site.

below 20 m depth, where the relatively homogeneous layer is located.

Discussion

It was emphasized above that the present proposed model yields an operational undrained strength (S_{uc}) that reflects the combined effects of the peak and the postpeak strength. It is to be expected, therefore, that S_{uc} would be lower than the peak strength from a CK_0U test for the stiff and brittle clay at the Ottawa Sewage Treatment Plant. It is also not surprising that S_{uc} is close to the peak strength at Gloucester, because the clay there is much softer and hence less brittle. Moreover, the strain rate during piezocone penetration is about two orders of magnitude faster than in the triaxial test. The higher rate will raise S_{uc} from an intermediate value to one that is much closer to, or may be even slightly higher than, the peak value.

It is noteworthy that for these three sites, Vesic's method and the present proposed model yield more or less the same results despite the fact that different failure surfaces are assumed in the models. As shown in Fig. 1a the assumed shape of slip lines in Vesic's approach results in the addition of a constant equal to 2.57 to the cavity expression term in Vesic's (1975) bearing capacity equation. The undrained shear strength derived from the cone resistance is therefore reduced by an amount dependent upon the soil's rigidity. The proposed model considers that the cavity expansion pressure is equal to the total stress (σ_n) normal to the cone face obtained from the cone resistance assuming that the friction at the cone-soil interface is calculated using the effective stress concept. Obviously, σ_n is less than q_1 depending upon the magnitude of induced pore pressure. Hence the proposed method also reduces the undrained shear strength by an amount dependent upon the generated pore

FIG. 18. Cone factor N_K obtained in present investigation.

pressures. Therefore, although for different reasons, it is expected that the strength derived from Vesic's and the proposed equation can be approximately the same. It seems that this is the case in soils where the ratios of u/q_1 is between 0.65 and 0.73. The strength values produced by Baligh's method appear to be too low, as illustrated by comparison with the vane strength at the STP site, where Bjerrum's (1972) correction factor for vane strength is 1.0. In other words, the vane strength realistically reflects the available strength for first-time loading failure. The low strength from Baligh's model for these sites may be due to the fact that the assumed failure surface is not compatible with what actually happened.

The Geonor vane measures mostly the strength on a vertical surface upon which the effective horizontal stress (σ'_{ho}) acts. For normally consolidated or lightly overconsolidated clays, σ'_{ho} is smaller than the effective vertical stress. The measured vane strength will therefore be lower than the peak strength of a CK_0U triaxial test on samples trimmed in the vertical (conventional) direction. This view is consistent with the comparison of these two strengths for the Gloucester and the Ottawa STP sites. Consequently, the strength from piezocone (S_{uc}) is higher than the vane strength at Gloucester. That S_{uc} is equal to the vane strength at the STP and Fredericton sites is encouraging but may be coincidental and should be treated with caution. More study is required to establish a clear relationship between these two strengths.

Figure 18 illustrates clearly the previous point. The empirical cone factor (N_K) obtained from the present study is compared with the lower and upper bounds of published data summarized earlier. In siltier soils the lower bound of the empirical relationships N_K vs. I_P appears to result in values of N_K that are too high by about 50%. This, in turn, would give an appreciably lower shear strength, which could seriously affect the economics of a project.

Summary and conclusions

The main purpose of this paper is to develop an interpretation method for piezocone penetrometer test results in order to obtain the undrained shear strength in cohesive soils. To do so,

many assumptions were necessary, and thus any approach could result in only an approximate value of the soil strength. The assumption concerning the slip surfaces when the cone is pushed into a media is the most significant with respect to the theoretical bearing capacity factor, and thus with respect to the soil strength itself. The proposed method considers the soil-cone interface to be a slip surface in soft soils. The analysis of piezocone results in terms of undrained strength is based on cavity expansion in an elastoplastic medium coupled with the determination of the vertical component of friction over the cone mantle, calculated using effective friction parameters and measured pore pressures generated during penetration. Several parameters that cannot be obtained from piezocone tests are also required for strength determination. The paper also discusses their influence on the strength.

This study shows the following:

- (1) It is possible to use simple assumptions with regard to the mechanics involved in the penetration process in soft soils for the evaluation of undrained shear strength based on piezocone data.
- (2) Owing to the assumptions of the soil's constitutive relationship, the undrained shear strength mobilized around the cone corresponds to the maximum strength for an ideal elastoplastic medium. For actual soil behaviour, either work softening or work hardening, the proposed interpretation results in an operational shear strength that is intermediate between that mobilized at small and at large strains.
- (3) Using representative parameters the proposed approach yields an operational strength dependent on induced pore pressures.
- (4) The values obtained from existing models that do not explicitly consider the pore pressures are within the range of undrained shear strength obtained using the proposed interpretation.
- (5) The rigidity index can be obtained with a graphical method applied to the pressure-strain curve from the self-boring pressuremeter test.
- (6) The rigidity index from the pressuremeter is a function of strain level. At about 1.5% strain for sensitive clays, this value corresponds to an undrained condition and is equal to that obtained from CK_0U tests, based on a secant modulus at a stress level midway between the beginning of shear and failure.
- (7) The operational undrained strength (S_{uc}) determined using the proposed model is equal to the CK_0U triaxial strength for the Gloucester clay and lower than that for the Ottawa STP clay. This is consistent with the rate effect and brittleness associated with these two clays.
- (8) S_{uc} is higher than the vane strength at the Gloucester site but equal to the vane strength at the other two sites.
- (9) At sites studied, Vesic's method yields more or less the same strength as the proposed model, whereas Baligh's method yields too low a strength for the three sites.

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