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Preconsolidation pressure from piezocone tests in marine clays

J.-M. KONRAD* and K. T. LAW†

Piezocone penetration tests were carried out at five sites in marine clay with overconsolidation ratios ranging from 1.3 to 4.8. A simple model for the behaviour of these clays indicates that the effective vertical yield stress mobilized along the cone axis during penetration is equal to the difference between the actual induced total vertical stress in the soil and the total porewater pressure. The actual induced total vertical stress can be derived from piezocone test data. Theoretically the ratio of the vertical yield stress inferred from piezocone results and the preconsolidation pressure should correlate with the overconsolidation ratio. This hypothesis is supported by the field and laboratory investigations presented in this study.

KEYWORDS: laboratory testing; piezocone; preconsolidation pressure; yield envelope.

INTRODUCTION

The preconsolidation pressure of marine clays is an important characteristic for the design of foundations on these soils, which are widespread in eastern Canada. Casagrande (1936) defined the preconsolidation pressure σ_p' as the 'largest overburden beneath which the soil had once been consolidated'. The preconsolidation pressure that can be determined in the laboratory from oedometer tests on undisturbed samples is also dependent on the type and the procedure of testing, however (Crawford, 1964). Therefore, the preconsolidation pressure σ_p' actually refers to the break in the oedometer curve and represents a yield stress that separates small strains from large strains. σ_p' is also affected by sample disturbance, and the determination of a complete profile of preconsolidation pressure with depth may require a substantial amount of time and testing.

These disadvantages may be overcome by using an in situ testing device such as the piezocone penetrometer, which produces continuous profiles of cone resistance and porewater

Des essais de pénétration avec un piezocone ont été effectués sur cinq sites d'argile marine ayant des rapports de surconsolidation variant de 1,3 à 4,8. Un modèle simple de comportement de ces argiles indique que la contrainte effective verticale à l'état limite mobilisée durant la pénétration est égale à la différence entre la contrainte totale verticale et la pression interstitielle générées dans le sol. La contrainte totale verticale peut être déterminée à partir des résultats des essais de piezocone. En théorie, le rapport de la contrainte verticale à l'état limite, calculée à partir des résultats du piezocone, et de la pression de surconsolidation devrait être une fonction du rapport du surconsolidation. Cette hypothèse est vérifiée par les essais in situ et de laboratoire présentés dans cet article.

pressure induced during the penetration. These results or derived parameters may then be correlated with the stress history of the deposit. Senneker, Janbu & Svanø (1982) suggested the use of the ratio $\Delta u/(q_c - \sigma_v)$, denoted by B_q , as the standard parameter for interpretation of piezocone data, where Δu represents the induced excess porewater pressure, q_c is the corrected total cone resistance and σ_v is the initial total vertical stress, i.e. the overburden pressure. Limited data, available in the literature (Lacasse & Lunne, 1982; Jamiolkowski, Ladd, Germaine & Lancellotta, 1985), indicate that B_q can be correlated with reliable data of overconsolidation ratios OCR.

The aim of this Paper is to investigate the potential use of piezocone test data for determining the stress history of marine clay deposits. It reviews available interpretation methods, using piezocone test results carried out on five different sites of low to moderate overconsolidated clays ($\text{OCR} \leq 4.5$), and also presents a simple model based on elasto-plastic behaviour of the soil, which relates the effective vertical yield stress induced during penetration to quantities that can be derived from piezocone test data. The proposed interpretation yields a profile of effective vertical yield stress that is compared with the preconsolidation pressure obtained from laboratory oedometer tests on samples of different size. An empirical correlation relating these stresses to the

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Table 1. Typical values of the properties of the clay studied

Soil property	Gloucester	St Marcel	Varenes	NRCC	STP
Water content: %	80	80	60	80	70
Plastic limit: %	27	25	22	25	25
Liquid limit: %	55	60	62	60	55
Plasticity index: %	28	35	40	35	30
Clay content: %	80	80	—	70	66
Sensitivity	100	30	20	10–500	40–500
σ_p'/σ_v'	1.5	1.3–1.8	2.4–4.9	1.7–3.7	2.5–4.5
Ground elevation: m	80	—	14	61	53

overconsolidation ratio summarizes the experimental data obtained at the sites reported in this study.

DESCRIPTION OF SITES

For this study, five sites were selected in the St Lawrence Lowlands in eastern Canada. These marine clay deposits are located at Gloucester (Ontario), Saint-Marcel (Québec), Varennes (Québec), National Research Council of Canada Ottawa Campus (denoted NRCC) (Ontario) and the Ottawa sewage treatment plant (denoted STP) (Ontario).

Gloucester site

The Gloucester site is 21 km south-east of Ottawa. It is a deposit of highly sensitive, soft Champlain Sea clay formed in different stages of erosion and deposition (Bozozuk & Leonards, 1972). The clay is about 21 m thick and is underlain by glacial till. It is slightly overconsolidated, with an OCR of about 1.5. The plasticity index ranges from 20% to 30%, and the average clay content is about 80%.

Saint-Marcel site

The Saint-Marcel site is approximately 100 km north-east of Montreal, on the south shore of the St Lawrence River. A detailed geotechnical investigation reported by Lefebvre & Langlois (1982) shows that the soil profile consists of 1.5 m of weathered crust of silty clay underlain by a thick deposit of light greyish clay. The natural water content in the clay is about 80%, which is 15–20% higher than the liquid limit. The OCR ratio is 1.8 at 2 m depth and decreases slightly with increasing depth to reach a constant value of 1.3 at 7 m below ground surface.

Varenes site

The Varennes site is about 20 km east of Montreal on the property of N.L. Chem Canada Inc.

The original ground surface is at an elevation of approximately 14 m. The stratigraphic profile has been studied by Casagrande, Firing, Schoof & Turcke (1965) and Samson, Leroueil, Morin & Le Bihan (1981). Beneath a layer of dense fill 90 cm thick is a stratum 1–2.8 m thick of loose silty fine sand and clayey silt. Below this stratum lies a deposit of grey clay 14 m thick.

The natural water content ranges from 55% to 65% and the liquid limit from 64% to 80%. Oedometer tests reveal that the preconsolidation pressure σ_p' is about 200 kPa for depths between 4 m and 8 m and increases steadily below 8 m. The OCR varies thus between 4.9 at 4 m and 2.5 near the bottom of the clay layer.

National Research Council of Canada site

The NRCC site is in an open field within the Montreal Road campus of the NRCC. The subsoil profile consists of Champlain Sea clay underlying a fissured crust 7 m thick. The natural water content is about 80% at a depth of 7 m and 60% 17 m below ground surface. It is overconsolidated, with OCR values ranging between 1.7 and 3.7 (Eden & Hamilton, 1956). For the upper 10 m the sensitivity is relatively low but is extremely high (greater than or equal to 200) below a depth of 12 m.

Sewage treatment plant site

The STP site is along the Ottawa River at the eastern outskirts of the city of Ottawa, in an extensive clay plain with a surface elevation of 53 m. The soil profile down to the underlying glacial till consists entirely of Leda clay (Eden, 1970). From the surface down to 2.5 m the clay is friable and oxidized. From 2.5 m to about 14 m the clay is fairly stiff, with a sensitivity of about 20. Below 14 m the clay becomes slightly coarser, the natural water content is 50% and the sensitivity increases dramatically, being of the order of 1000 at 17 m. The clay is overconsolidated by about 400 kPa. Table 1 presents a summary of typical values of the properties of the clay studied.

Fig. 1. Piezocone profiles at the Gloucester site

Fig. 2. Piezocone profiles at the Saint-Marcel site

PIEZOCONES TEST RESULTS

Electric GMF 25 kN 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 porewater pressures u were measured through a cylindrical porous stone 4 mm thick located immediately above the conical tip. Water pressures can act over a part of the base area of the cone in a direction opposite to the resisting forces developed during pushing; thus the measured total stress q_c will be reduced. To obtain the actual total stress q_t , a correction must be applied to account for the design of the cone. This correction is done directly by the computer during data processing for each set of readings, using the relationship

$$q_t = q_c = 0.34u$$

In soft soils the tip loads are only about 2–4% of the design capacity of the cone, and considerable care in the testing procedure is required to obtain meaningful data, as discussed by Konrad (1986). The rate of penetration for all tests in this study was 1 cm/s. At this rate one set of readings could be obtained for every 2 cm of penetration.

The profiles of q_t and u for the five sites are presented in Figs 1–5. The water-table is also indicated in these figures. From these profiles soil layers and silt and/or sand lenses can be identified. For the Gloucester site (Fig. 1), the q_t and u profiles increase uniformly with depth and clearly define the boundary between the clays of different water contents at 7 m and 14 m, as reported by Bozozuk & Leonards (1972). The piezocone test results from the Saint-Marcel site (Fig. 2) indicate

clearly the homogeneity of the clay deposit. At the Varennes site (Fig. 3), the piezocone profiles show a relatively uniform clay deposit with some stronger layers at 4.4 m, 5.0 m and 7.3 m. The pore pressure profile does not indicate draining layers, suggesting that these layers may be siltier clayey soils, which is in agreement with the results of Casagrande *et al.* (1965).

The profiles at the NRCC site (Fig. 4) show a surface crust to a depth of about 8 m in which the pore pressure generated during driving decreases slightly with depth. Two distinct layers are

Fig. 3. Piezocone profiles at the Varennes site

Fig. 4. Piezocone profiles at the NRCC site

detected below the crust: the first, relatively thin, extends to a depth of 10.5 m and corresponds to a water content of about 70%. Below this layer, the average water content is only about 60%.

Finally, both q_t and u profiles at the STP site (Fig. 5) increase linearly with depth between 5 m and 12 m. Below 12 m, q_t and u decrease slightly with depth. According to Eden (1970) the boundary between moderately sensitive and extremely sensitive clay at this site is found approximately at 13 m, which is in close agreement with the boundary obtained from piezocone data.

Fig. 5. Piezocone profiles at the STP site

EVALUATION OF AVAILABLE INTERPRETATION TO DETERMINE OCR

When the use of the static cone test was introduced for investigating clay deposits only the cone resistance q_c was measured and correlated with OCR. Schmertmann (1978) recommended that the approximately linear increase in q_c with depth observed in many natural homogeneous clay deposits could be extrapolated back to the depth axis to obtain an estimate of the position of the original ground surface and, consequently, of OCR with depth.

The suggested thickness of removed earth by erosion is about 3 m for the Gloucester site, 4.5 m at Saint-Marcel, 9 m at Varennes, 3 m at NRCC and 9 m for the STP site. There is no systematic correspondence between the value of the estimated OCR and the OCR obtained from undisturbed samples. This lack of correlation is probably due to the fact that the processes leading to overconsolidation, i.e. erosion, ageing, chemical and dynamic actions, lowering of water-table, freeze-thaw cycles, are not adequately considered in this method. Tavenas & Leroueil (1979) argued that the cone resistance is a measure of limit state stress conditions or of limit state related to deformation characteristics and is related to the preconsolidation pressure σ'_p . In Champlain clays they suggested that the ratio q_c/σ'_p is about 3. The analysis of the results obtained in this study indicates that the value of q_t/σ'_p varies between 3.8 and 5.3 at Gloucester, between 5 and 5.3 at Saint-Marcel, between 4.0 and 4.8 at Varennes, between 4.3 and 4.6 at NRCC, and between 3.5 and 3.8 at the STP site. While there is merit in using the framework of the limit state theory to derive OCR, the experimental data suggest that additional soil characteristics are required to link the ratio q_t/σ'_p to OCR.

With the advent of piezocones a considerable variety of empirical relationships based on the measured cone resistance with or without correction for pore pressure effects and total or excess pore pressure have been proposed to obtain OCR. It is accepted that the pore pressure at failure is related to a change in octahedral normal stresses $\Delta\sigma_{oct}$, shear stresses, $\Delta\tau_{oct}$ (Henkel, 1960) and also to OCR. Since the actual $\Delta\sigma_{oct}$ and $\Delta\tau_{oct}$ around the piezocone are not known at present, Senneset *et al.* (1982) proposed to use the ratio $\Delta u/(q_c - \sigma_v)$ denoted by B_q to correlate with the overconsolidation ratio.

Values of B_q derived from information available in the literature have been plotted against the relevant value of OCR (Fig. 6). Some sites such as Ønsey, Pontida and Boston show a tendency for the dimensionless ratio B_q to decrease with increasing OCR. However, the value of B_q is dependent on soil type at least for these three

Fig. 6. Pore pressure coefficient B_q versus OCR from available data

sites. Furthermore, even within the same deposit B_q may vary, as indicated by the tests at Porto Tolle. Jamiolkowski *et al.* (1985) suggested that the irregular trend in B_q versus depth is probably due to the presence of more pervious lenses and layers. At the Saint-Alban site (Fig. 6), for an almost constant OCR with depth, the value of B_q increases continuously from 0.5 to 0.6.

The data obtained during cone testing are stored on tape, which enables efficient and flexible processing after completion of a sounding. Figs 7 and 8 present the value of B_q for each sounding at the various sites of this study. Over the first 6–8 m of penetration, there is a general tendency of B_q to increase with depth and to remain almost constant thereafter. As shown in Fig. 9, the values of B_q for these different clays range between 0.4 and 0.82. At some sites an apparent relationship with OCR can be observed, but at the Gloucester and STP sites the trend is not clearly defined.

The review of existing data and the interpretations of the test results of the present investigation indicate that no unique relationship between B_q and changes in OCR should be expected. Even within a given clay deposit, changes in B_q may not necessarily reflect changes in OCR, which indicates that B_q is not sufficiently sensitive to correlate successfully with OCR.

THEORETICAL CONSIDERATIONS

Baligh (1975) pointed out that undrained penetration of a cone in saturated clays is essentially strain controlled and causes significant shearing

Fig. 7. Pore pressure coefficient B_q versus depth at Gloucester and Saint-Marcel

and distortion of the surrounding soils. Moreover, the strain field around the cone varies with distance from the cone. As a result, the total and effective stresses, and hence the pore pressures, also vary, depending on their location. Furthermore, except for soil elements close to the vertical cone axis where the problem is axisymmetric, the principal axes of stress rotate within the soil. Thus, to solve the penetration problem from sim-

Fig. 8. Pore pressure coefficient B_q versus depth at Varennes, NRCC and STP sites

Fig. 9. Pore pressure coefficient B_q versus OCR from data from this study

plified strain fields, a realistic soil model including rotation of principal stresses must be used. Such a model is not readily available.

Consequently, engineers still seek alternative methods of cone test interpretation based on empirical correlations with data from other laboratory or field tests. Wroth (1984) correctly stated that any successful relationship should ideally be

- (a) based on a physical appreciation of why the properties can be expected to be related
- (b) set against a background of theory, however idealized this may be
- (c) expressed in terms of dimensionless variables.

To apply this approach to the very complex penetration problem in natural soils may prove impossible. However, simple relationships of the processes involved during penetration can be obtained if the soil behaviour is idealized as perfectly elasto-plastic. In the next section the pore pressures induced during cone penetration are analysed within the framework of the yield envelope concept.

Porewater pressure response in idealized soil during cone penetration

The initial conditions in a soil element located under the cone axis before being subjected to any influence of the penetrating cone is represented in Fig. 10 by point A. As the cone moves towards the soil element, changes in total stresses are induced, leading to an immediate pore pressure increase for fully saturated conditions. Fig. 10(a) shows the stress path directions obtained for one-dimensional consolidation conditions, i.e. drained (path Aa), and for undrained axisymmetric loading conditions for a soil with a Skempton pore pressure parameter equal to unity (path Ab). Since rapid cone penetration corresponds to practically undrained conditions, the stress path

Fig. 10. Ideal soil behaviour during cone penetration: (a) schematic effective stress path under cone axis; (b) pore-water pressure response

in a soil element under the cone axis as the cone penetrates is expected to be to the left of path Aa. In overconsolidated soils, the Skempton parameter is generally less than unity, and the stress path induced during cone penetration is then expected to be to the right of path Ab. The actual effective stress path during cone insertion depends on the amount of stress change and on the actual pore pressure parameter which, in turn, is a function of the overconsolidation ratio. The stress path AB shown in Fig. 10(a) is thought to be a typical path in a soil element under the cone axis during penetration. Work by Baligh (1985) and Acar & Tumay (1986) on the strain field induced by cone penetration suggests stress paths below the cone axis that are consistent with path AB. The increase in total stresses as the cone approaches the soil element leads to an increase in vertical effective stress up to σ'_{vc} where yielding occurs. This condition is represented by point B. Since the soil element is on the cone axis, the stress state is axisymmetric and the excess pore pressure Δu_B induced during this phase can be related to the change in deviatoric stress ($\Delta \sigma_v - \Delta \sigma_h$) and to the change in mean total stress (Fig. 10(b)).

Observations from laboratory compression tests in triaxial and plane strain conditions

(Skempton, 1954; Henkel, 1960; Law & Bozozuk, 1979) indicate that the pore pressure response before yield in soils ranging from heavily to lightly overconsolidated is equal to or less than that of an elastic medium, i.e. $A_r \leq 1/3$. If the excess pore pressure at B, Δu_B , is related to the increase in total vertical stress, which exceeds the increase in total horizontal stress before yielding, it can be shown that the slope of the line AB in Fig. 10(b) is less than unity in overconsolidated soils. Its magnitude depends mainly on the actual stress path which, in turn, is a function of soil stiffness and OCR.

For post-yield conditions, Δq is zero, since there is no further strength change; thus $\Delta \sigma_v$ is equal to $\Delta \sigma_h$. Any further increase in total vertical stress from B to C is accompanied by an equal increase in total horizontal stress. The increase in excess porewater pressure from B to C is equal to the increase in total vertical stress. Consequently, point C is situated on a line at 45° to the horizontal axis which passes through point B (Fig. 10(b)).

The stress conditions at point A representing the initial state can be estimated with fair accuracy. The conditions q_v and u_m at point C, corresponding to the final state when the cone reaches the soil element, can be inferred from the measured total cone resistance q_t and the measured pore pressure at the base of the cone, u . Only the conditions at B that are related to the yield condition are unknown. Geometrical considerations from Fig. 10(b) indicate that

$$IF = ID + DF \quad (1)$$

Since the slope of the line passing through B and C is equal to unity

$$DF = CF = u_m - u_0 \quad (2)$$

Rewriting equations (1) and (2) with $IF = q_v$ yields

$$\begin{aligned} q_v - u_m &= ID - u_0 \\ &= IE - ED - u_0 \end{aligned} \quad (3)$$

which gives

$$q_v - u_m = \sigma_{yc} - \Delta u_B - u_0 = \sigma_{yc}' \quad (4)$$

When a cone reaches a soil element adjacent to the tip, equation (4) establishes that the difference between the induced total vertical stress q_v and the total porewater pressure u_m is directly related to the vertical effective yield stress σ_{yc}' which is mobilized along the effective stress path developed during cone penetration.

Relevant stress conditions from piezocone measurements

The theory of cavity expansion has been applied by Gibson (1950) to the problem of

Fig. 11. Simplified stress conditions around a cone

bearing capacity of deep foundations. The cavity expansion model approximately represents the soil behaviour under a deep circular pile only at a large distance from the pile. Several assumptions are necessary to take into account the change in direction of stresses and strains near the cone tip. As illustrated in Fig. 11, it can be considered that the cone is acted on by a uniformly distributed normal soil pressure q_n 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, which is taken as a hemisphere of radius that of the pile. The unit shear stress is taken as the undrained shear strength of the soil, s_u . Ladanyi & Eden (1969) assumed 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.

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 & Roy (1986) 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

$$\tau = (q_n - u_m)M \tan \phi'$$

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 porewater pressure is measured above the base of the cone, the pore pressure in

Fig. 12. Comparison between the effective vertical yield stress inferred from piezocone data and the preconsolidation pressure from laboratory tests—Gloucester site

the failure zone, u_m , can be approximated by

$$u_m = \alpha u \quad (5)$$

where α is a factor usually greater than unity. In soft soils, available data for measured pore pressures at both locations (Roy, Tremblay, Tavenas & La Rochelle, 1982) indicate that α is in the range 1–1.1. Considerations of force equilibrium at failure give the following expression for a cone with apex angle θ

$$q_t = q_n + M \cot \theta \tan \phi' (q_n - \alpha u) \quad (6)$$

and therefore

$$q_n - u_m = \frac{q_t - \alpha u}{1 + M \tan \phi' \cot \theta} \quad (7)$$

q_n is equal to the total vertical stress q_v induced by the cone penetration process only on the cone axis, see Fig. 11. Here $q_n - u_m = q_v - u_m = \sigma_{yc}'$ (see equation (4)).

The measured cone resistance q_t represents the sum of the integration of the resistance of the soil surrounding the cone and of the integration of the shear stresses developed along the cone–soil interface during penetration. Therefore, any correlation between cone data with soil properties from other tests should be attempted with q_n rather than with the measured cone resistance q_t .

INTERPRETATION OF PIEZOCONE TEST RESULTS

The results obtained from the piezocone tests at the five sites will be interpreted using equations (4) and (7) and will provide a quantitative estimate of the vertical effective yield stress mobilized during cone penetration, σ_{yc}' . Several parameters are required for the analysis summarized in equation (7). Only q_t and u can be determined from piezocone tests. The other parameters, α and $M \tan \phi'$, have to be estimated independently.

As discussed in the previous section α is close to unity in soft soils, and to date very little data are available in other types of soils. An upper bound value is 1.33 if it is assumed that spherical cavity expansion applies below the cone tip, while cylindrical cavity expansion holds around the shaft, and hence at the base of the cone. α will be less than 1.33, since the actual cone penetration produces an expansion that is intermediate between a sphere and a cylinder. $M \tan \phi'$ represents the coefficient of friction between the soil and the cone face. According to measurements made by Potyondy (1961) and Bozozuk, Keenan & Pheeney (1979), M can be taken between 0.5 and 0.75 for smooth steel. For normal roughness, M can be taken to be 1.0.

In this analysis, α and M are assumed to equal unity for the five sites studied. Since this assumption results in an overestimation of both the numerator and the denominator of equation (7) by about 10–20%, depending on the values of q_t and u , the calculated difference $q_n - u_m$ is expected to be fairly close to the real value.

Analysis of the piezocone tests using equation (7) is readily performed with a computer. Results are presented in Figs 12–15 for four sites. Unfortunately, the cassette with the data from the STP site was damaged, and the data had to be processed by hand. Fig. 16 shows the average value of the vertical effective yield stress along the cone axis versus depth for the STP site.

To evaluate the performance of this conceptual model, the preconsolidation pressures σ_p' determined from laboratory consolidation tests on samples obtained from either 5 cm or 12.7 cm dia. samplers or block samples were also plotted in Figs 12–16. These stresses, i.e. σ_{yc}' and σ_p' , are obtained from tests with different stress paths as well as different strain rates and should not be expected to be equal. Nevertheless, since these two stresses relate to a yield condition of the same soil it is expected that the features displayed by the σ_p' profile should also be detected in the σ_{yc}' profile.

At four sites, σ_{yc}' is slightly less than σ_p' , whereas for the Saint-Marcel clay deposit σ_{yc}' is slightly greater than σ_p' in the upper 8 m and is then equal to σ_p' between 8 m and 10.5 m depth.

Fig. 13. Comparison between the effective vertical yield stress inferred from piezocone data and the preconsolidation pressure from laboratory tests—Saint-Marcel site

Fig. 15. Comparison between the effective vertical yield stress inferred from piezocone data and the preconsolidation pressure from laboratory tests—NRCC site

The most striking feature is that the shape of the effective yield stress profile inferred from piezocone tests is similar to that of the preconsolidation stress obtained from oedometer tests. If the geological history of a clay deposit is relatively simple, such that the profile increases linearly with depth, the proposed interpretation should show that the σ'_{yc} profile is also increasing linearly with depth. This is well illustrated at the

Saint-Marcel and the Varennes sites. If the past history of the deposit is more complex, such as at the Gloucester, STP and NRCC sites, the profile of σ'_p versus depth will reflect the past events and may present significant changes. Despite the complex geological actions at these sites, the proposed interpretation is successful in showing the layers corresponding to the various phases of the geological processes as well as in quantifying the

Fig. 14. Comparison between the effective vertical yield stress inferred from piezocone data and the preconsolidation pressure from laboratory tests—Varennes site

Fig. 16. Comparison between the effective vertical yield stress inferred from piezocone data and the preconsolidation pressure from laboratory tests—STP Ottawa site

effect of these phases on the stress state of the deposit.

Using this procedure a continuous profile of σ_{yc}' can be obtained from the raw data of a cone test in about 5 min. In contrast a σ_p' profile is not continuous and requires considerable time to obtain, since each point corresponds to about 10 days of testing. The use of data obtained from oedometer tests may also lead to incomplete characterization of a clay deposit. As shown in Fig. 15, the most probable preconsolidation pressure profile extrapolated from laboratory data reported by Eden & Law (1980) for the NRCC site did not identify a distinct layer below a depth of 10.5 m. Oedometer tests carried out in 1985 for another project, before cone testing and interpretation, show that a lower value of σ_p' was obtained at a depth of 10.5 m, which indicates that a second layer may exist. The preconsolidation pressure profile derived from the piezocone tests shows that a second layer does exist.

Similar observations are made for the site at the Ottawa STP. Earlier interpretations (Eden, 1970) from oedometer tests on block samples considered that the most probable preconsolidation pressure profile was linear between 10 m and 23 m (Fig. 16). The interpreted cone data do not follow the same trend at these depths. A compilation of all the consolidation tests performed by the Division of Building Research (now the Institute for Research in Construction), NRCC, on samples obtained either with 5 cm or 12.7 m dia. piston samplers, as well as all tests carried out on samples from blocks, shows that the σ_{yc}' profile obtained from piezocone tests is approximately parallel to all the σ_p' data. Moreover, at a depth of 19 m, a much lower value of σ_p' was obtained from a block sample, which was consistent with the σ_p' profile determined using samples from a piston sampler.

ESTIMATION OF IN SITU PRECONSOLIDATION PRESSURE FROM PIEZOCONES TESTS

Before seeking a correlation between the vertical effective yield stress inferred from piezocone tests and the in situ preconsolidation pressure that can be derived from the results of laboratory consolidation tests, as suggested by Leroueil, Tavenas, Samson & Morin (1983), several factors must be evaluated. The factors known to influence the laboratory preconsolidation pressure, the relationship between field and laboratory preconsolidation pressures and the vertical effective yield stress inferred from piezocone tests are the sample quality, the stress paths, the strain rate effects, the strain softening and the magnitude of the overconsolidation ratio that influences the initial in situ stress conditions.

Effect of strain rate

The time or strain rate dependence of the undrained shear strength of clays is well established. Higher shear strengths are observed at higher strain rates. An increase of 5–10% in strength for a tenfold increase in strain rate is common in many clays. This effect is related to the fact that the size but not the shape of the field envelope is strain rate dependent. Tavenas & Leroueil (1977) and Graham, Crooks & Bell (1983) have presented evidence that shows that the size of the yield envelope increases approximately proportionally to the logarithm of the strain rate. Since the apparent preconsolidation pressure is one point on the yield envelope, the lower the rate of loading in an oedometer test the lower the apparent preconsolidation pressure. Leroueil *et al.* (1983) showed that to a first approximation the relative effect of strain rate on σ_p' is unique for all Champlain clays, irrespective of their plasticity, degree of overconsolidation or the testing technique. The data presented by these researchers show that a tenfold increase in the strain rate results in an increase of about 10% in σ_p' . Multiple-stage loading tests with load increment durations of 24 h (conventional) correspond to a strain rate that is typically $10^{-7}/s$ for Champlain clays.

Since there is no complete theory of deep penetration, only an approximate relationship between the cone penetration rate $\dot{s}$ and a representative strain rate $\dot{\epsilon}$ can be deduced at present. On the basis of spherical cavity expansion theory, Ladanyi (1982) showed that the strain rate at the elastic-plastic interface around an expanding hemispherical cavity coinciding with the cone can be expressed as

$$\dot{\epsilon} \approx 6\epsilon_{1f}\dot{s}/d \quad (8)$$

where ϵ_{1f} is the strain at failure in triaxial compression tests and d is the cone diameter.

Assuming a value of 1% for ϵ_{1f} , $\dot{\epsilon}$ is then about $1.7 \times 10^{-2}/s$ for a penetration rate of 1 cm/s. The strain field around cones in steady penetration obtained by Acar & Tumay (1986) can be used to determine the strain rate profile below the cone tip. The calculated average strain rate decreases with increasing distance from the cone tip and is about $8 \times 10^{-3}/s$ for a soil element at a distance of $1.5d$ and $7 \times 10^{-4}/s$ at a distance of $5d$. Thus, the actual strain rate along the stress path AB (Fig. 17(a)) mobilized during cone penetration is not constant. The average strain rate along path AB during cone penetration is about 10^{-3} – $10^{-4}/s$, which is approximately three or four orders of magnitude larger than the strain rate in a conventional oedometer test. The size of the yield envelope corresponding to the field condition

Fig. 17. Real soil behaviour during core penetration: (a) schematic effective stress path under the cone axis in natural soil; (b) porewater pressure response

during cone penetration is then much larger than the size of the yield envelope mobilized in the laboratory. Therefore, assuming that all other conditions remain the same in both cases, it is expected that σ_{yc}' would be overestimated by considering strain rate effects only.

Effect of strain softening

The behaviour of natural clays is that of a strain softening material characterized by a drop in strength after failure. A schematic effective stress path for an element located on the cone axis in these soils is shown in Fig. 17(b). When the effective stresses reach the yield envelope corresponding to the field strain rate, the shear strength decreases from the peak value along the stress path BC. The amount of the drop in strength is a function of many factors, such as stress path, overconsolidation ratio and strain rate. It has been shown previously that for an elastic-plastic material the pore pressure change from B to C was equal to the total vertical stress change.

In a strain softening material, the reduction in shear strength from B to C means that the increase in horizontal total stress is larger than the increase in vertical total stress. Hence, the change in pore pressure from B to C, which is intermediate between the horizontal and vertical total stress increases, is slightly higher than the total vertical stress change. This implies that the slope of the line BC in Fig. 17(b) exceeds 45° for

Fig. 18. Typical stress paths in an oedometer test and in a soil element on the cone axis during penetration

most natural clays. For a given depth, the conditions of the initial point A are known, the stresses at C can be obtained from piezocone data using equations (5) and (6), and the stress conditions prevailing at B are inferred from equation (7), which assumes that the porewater pressure change after yielding is equal to the vertical stress change. This assumption results in an underestimate of the actual effective yield stress σ_{yc}' , as shown in Fig. 17(b).

Effect of stress path and OCR

It is considered in this section that the yield envelope is independent of the strain rate. Furthermore, it is assumed that its shape is also independent of OCR so that a unique normalized yield envelope with respect to the preconsolidation pressure can be used.

Data from one-dimensional consolidation tests reported by Sällfors (1975) and Lefebvre & Philibert (1979) suggest that the initial stiffness of the intact overconsolidated clay leads to a stress path such as OE for a lightly overconsolidated clay and OF for a heavily overconsolidated soil within the yield envelope (Fig. 18). When the limit state is reached, at points E and F, the stress paths move towards and then follow the K_0 line in the normally consolidated stress range.

A probable stress path imposed on a soil element along the cone axis during penetration is also shown in Fig. 18 for soils with different values of OCR. AB represents the stress path for lightly overconsolidated soils and CD is typical for heavily overconsolidated clays. With increasing OCR, it is expected that the slope of the stress path induced during cone penetration will decrease, as a result of a smaller induced pore pressure during shear. Fig. 18 indicates clearly that the stress paths in the field and in the laboratory are different. The amount of preconsolidation significantly affects the initial stress state in

Fig. 19. Proposed empirical correlation for the determination of OCR from piezocone test data

the soil and also the stress paths in the field.

For lightly overconsolidated soil, the initial stress state is close to the yield envelope and the stress path generated during cone penetration is relatively steep. The values of σ_{yc}' and σ_p' are very close, as shown schematically in Fig. 18. For clays with high OCR, the initial stress state is closer to the origin and relatively far from the isotropic stress state on the yield envelope. Although the slope of the field stress path is not as steep as for slightly overconsolidated soils, because of less porewater pressure generation during shear, the values of σ_{yc}' are much lower than the preconsolidation pressure obtained from oedometer tests.

If it is assumed that the strain rate does not influence the yield envelope, Fig. 18 clearly indicates that σ_{yc}'/σ_p' decreases with increasing OCR.

The influence of the remaining factors is restricted to the laboratory determination of σ_p' and has adequately been discussed by Leroueil *et al.* (1983).

EMPIRICAL CORRELATION TO DETERMINE IN SITU σ_p' FROM PIEZOCONE TESTS

Figure 19 shows the relationship between the ratio σ_{yc}'/σ_p' and the overconsolidation ratio for all sites studied in this investigation. This relationship is based on conventional oedometer tests on samples from 70–80 mm dia. piston samplers. Fig. 19 indicates that for an OCR close to unity, i.e. lightly overconsolidated clays, the stress ratio σ_{yc}'/σ_p' is about 1.2 and for an OCR of 4.5 it is 0.7. The trend shown in Fig. 19 is consistent with the expected behaviour based on Fig. 18, namely that the stress ratio σ_{yc}'/σ_p' decreases with increasing OCR. Furthermore, the proposed interpretation of piezocone data produces less

scatter at any given overconsolidation ratio compared with the scatter obtained when the same data are interpreted in terms of the B_q parameter as shown in Fig. 9.

The scatter in results can be attributed to several factors. For example, tiny draining layers may reduce the measured pore pressure leading to an overestimation of the value of σ_{yc}' , and thus of the ratio σ_{yc}'/σ_p' . The large variation in pore pressure profiles such as at the NRCC site, and especially at the Ottawa STP site, may indicate such a condition. Therefore, the points for the STP site may be slightly lower than shown in Fig. 19. Another consideration is the accuracy of measuring the cone resistance in very weak soils, using cones with 25–50 kN capacity. It is strongly recommended that the cones be calibrated for each expected range of loads and that a proper testing procedure be adopted (Konrad, 1986) to minimize the errors. Despite these efforts, the cone resistance may still be underestimated in weak soils, which may result in underestimates of σ_{yc}' .

To determine OCR from piezocone test results without prior knowledge of the stress history, an iterative procedure may be used, provided that a reasonable estimate of σ_{vo}' can be made. The ratio $\sigma_{yc}'/\sigma_{vo}'$ provides the first value of OCR, which in turn yields the ratio σ_{yc}'/σ_p' using the mean relationship given in Fig. 19. A new value of OCR is then obtained by dividing $\sigma_{yc}'/\sigma_{vo}'$ by σ_{yc}'/σ_p' . This procedure is repeated until there is satisfactory convergence of the value of OCR. A few iterations suffice to reach convergence.

The in situ preconsolidation pressure depends on the specific engineering situation, i.e. the stress path dependence, as pointed out by Leroueil *et al.* (1983). Since the σ_p' value obtained from laboratory tests is also affected by sample disturbance, the relationship between the preconsolidation pressure operative in the field and inferred from laboratory testing will also be empirical. Leroueil *et al.* (1983) proposed to obtain the in situ preconsolidation pressure for embankment design from σ_p' determined from conventional oedometer tests, by using a correction factor that is a function of OCR and sampler size. This relationship can be combined with the empirical correlation proposed in Fig. 19 to determine the in situ σ_p' value from piezocone tests.

CONCLUSIONS

The purpose of this study was to develop a method for interpreting piezocone penetrometer test data to obtain the preconsolidation pressure in cohesive soils. Theoretical relationships have been developed here for the vertical effective yield stress along the cone axis during cone penetration

in an elasto-plastic material. Although this relationship can only approximate real soil behaviour, it indicates a pattern of behaviour that can be compared with a pattern obtained from other tests in the laboratory or in the field and can lead to useful empirical correlations. An experimental study was conducted to investigate the potential of using the proposed piezocone data interpretation for evaluating the geological history of five sites in marine clay deposits.

The study has led to the following conclusions.

- (a) The ratio of the excess pore pressure measured during piezocone penetration and the total net cone resistance ($q_t - \sigma_{v0}$) is not sufficiently sensitive to reflect OCR changes, at least for OCR ranging from 1.3 to 4.5.
- (b) The effective vertical yield stress mobilized during cone penetration is equal to the difference between the induced total vertical stress and the total porewater pressure.
- (c) The induced total vertical stress can be derived from the measured cone resistance, assuming that the unit shear stress at the cone-soil interface is computed using effective stress parameters combined with the measured pore pressure generated during penetration.
- (d) The continuous profiles of cone resistance and pore pressure with depth result in a continuous profile of the vertical effective yield stress that is approximately parallel to the profile of the preconsolidation pressure obtained from laboratory oedometer tests.
- (e) Both profiles are affected by the strain rate, the stress path, the overconsolidation ratio and the stress-strain characteristics.
- (f) It is suggested that these effects are included in an empirical correlation between the overconsolidation ratio and the ratio of the vertical effective yield stress derived from the proposed model, for the preconsolidation pressure obtained from conventional oedometer tests on samples from 70 mm dia. samplers. This correlation shows that the stress ratio decreases with increasing OCR. For the five sites studied the proposed empirical correlation exhibits less scatter than the relationship based on the B_q parameter.
- (g) The in situ preconsolidation pressure may be estimated from the preconsolidation pressure derived from the previous correlation by applying a correction factor, which is also dependent on OCR.

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