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EVALUATION OF K_0 AND ITS INFLUENCE ON THE FIELD VANE STRENGTH OF OVERCONSOLIDATED SOILS

EVALUATION DE K_0 ET SON INFLUENCE SUR L'INTENSITE DU POUR LES SOLS SURCONSOLIDES

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SYNOPSIS: This paper presents a new laboratory method for the determination of *in situ* horizontal stress of heavily overconsolidated clays using a stress path triaxial apparatus. The proposed method is based on the concept that if an undisturbed overconsolidated soil sample is consolidated isotropically to a stress equivalent to the effective *in situ* vertical stress, and subsequently the radial stress is increased while maintaining the *in situ* vertical stress constant, the sample will experience significant axial strain only when the radial stress exceeds the *in situ* horizontal stress. In the second part of this study, the influence of K_0 on the field vane strength of an overconsolidated soil has been investigated. It has been observed that there is a good correlation between K_0 and field vane strength where field vane strength was found to increase with the increase in K_0 values. This trend was observed for field vane strengths obtained by both the conventional method of interpretation where isotropic soil behaviour has been assumed and also by the more accurate non-conventional method where strength anisotropy, non-uniform stress-distribution around the vane and effect of shearing rate have been taken into account.

The evaluation of *in situ* stresses in overconsolidated soils is important for problems related to foundations, excavations, and numerical analyses. The determination of horizontal stress in overconsolidated soils is however not easy, and more problematic than its evaluation in a normally consolidated clay. Direct measurements of *in situ* horizontal stresses by field testing methods (e.g., Kenney 1967; Bjerrum and Andersen 1972; Massarsch et al. 1975; Marchetti 1980; Handy et al. 1982; Jefferies 1988;) are difficult and introduce uncertainty as a result of changes to initial stress state during installation of the various devices (Tavenas et al. 1975). The hydraulic fracturing method (Bjerrum and Andersen 1972) is applicable only to normally consolidated and lightly overconsolidated clays (Jamiolkowski et al. 1985). The only known laboratory test method, for the measurement of *in situ* stresses, is the use of oedometer developed by Skempton (1961). Similarly, while there are a number of laboratory methods available for the estimation of K_0 for normally consolidated or slightly overconsolidated clays (Bishop and Henkel 1962; Bishop 1958; Campanella and Vaid 1972; Poulos and Davis 1972; Tavenas et al. 1975; Chang et al. 1977), i.e., for clays with $K_0 < 1$, the techniques for evaluation of K_0 in overconsolidated clays are non-existent. There are some empirical or indirect methods (e.g., Brooker and Ireland 1965; Mayne and Kulhawy 1982) available for the determination of *in situ* stresses of heavily overconsolidated clays, but all of these methods are dependent on a combination of laboratory tests, such as oedometer consolidation, Atterberg limits determination or triaxial tests. Jaky's (1948) well known equation ($K_0 = 1 - \sin\phi'$) is not applicable to overconsolidated clays.

Stress history of overconsolidated clays

A typical stress history of an overconsolidated clay in the field is shown in Fig. 1(a). The line AB indicates the stress path due to sedimentation process, during which the soil is normally consolidated under K_0 condition. The stress path BC indicates unloading due to some erosional process. The soil is overconsolidated along the stress path BC. Point C is the *in situ* condition with present stresses σ_{v0}' and σ_{h0}' . After subsequent sampling and preparation for laboratory testing, both vertical and horizontal stresses become equal since the total stress as well as the pore water pressure in the sample is isotropic. In the ideal case of perfectly undisturbed sampling, this isotropic stress state lies at point D', located above point D.

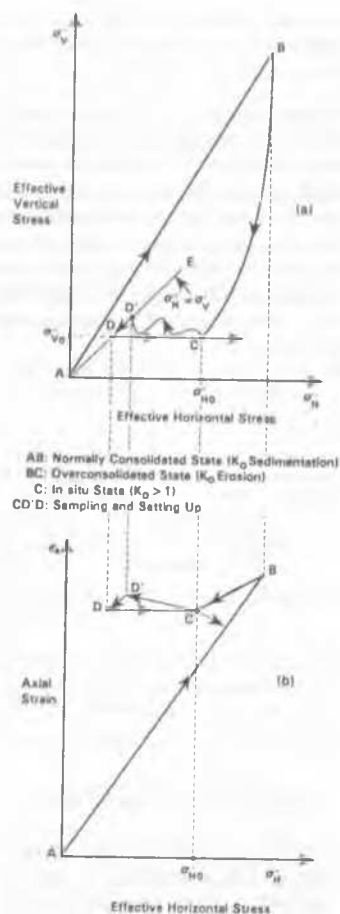


Fig. 1. Typical field stress history of overconsolidated clays.



The proposed method for evaluation of K_0

The proposed method is based on the concept that if an undisturbed overconsolidated soil sample is consolidated isotropically to a stress equivalent to the effective *in situ* vertical stress (σ_{v0}'), and subsequently the radial stress is increased while maintaining the *in situ* vertical stress constant, the sample will experience significant axial strain only when the radial stress exceeds the *in situ* horizontal stress (σ_{h0}'). The concept is illustrated in Fig. 1. During the first stage, the sample is consolidated isotropically to the point D where the consolidation pressure is equal to σ_{v0}' . During the second stage, the radial stress (σ_r') is increased while maintaining the axial effective stress constant equal to σ_{v0}' . The sample will experience appreciable axial strain only when σ_r' exceeds σ_{h0}' , i.e., when the stress path along DC moves towards right of point C. The corresponding axial strains, which may not necessarily be linear, associated with the stress path are shown in Fig. 1b.

Testing procedure and interpretation of test data

The testing procedure requires a triaxial apparatus, preferably computer controlled, with the capability of applying vertical and radial stresses independently to simulate various stress paths. The sample is initially consolidated to an effective cell pressure equal to the *in situ* effective vertical stress using a back pressure equal to *in situ* pore water pressure. After full consolidation, the test is continued fully drained by increasing radial stress (cell pressure) while keeping the vertical effective stress constant. For the overconsolidated clays tested in this study, the rate of application of the radial stress was 1.0 kPa/hour to ensure full dissipation of excess pore water pressure. The test was stopped automatically when the axial strain exceeded 0.20%.

Among the various data recorded during the test, only the axial strain, cell pressure and back pressure measurements are required for the evaluation of σ_{h0}' using the proposed technique. Test results are plotted on a graph with the σ_h' along the ordinate and the axial extension strain along the abscissa. An ideal shape of this plot for a truly undisturbed sample is shown in Fig. 2, where an abrupt transition must occur when σ_h' exceeds σ_{h0}' . In practice, this location of σ_{h0}' is not immediately obvious, since experimental data points show a gradual transition as indicated by the curved line in Fig. 2. In such cases, the value of σ_{h0}' can be determined by projecting the straight line portion of the inclined arm on the σ_h' axis.

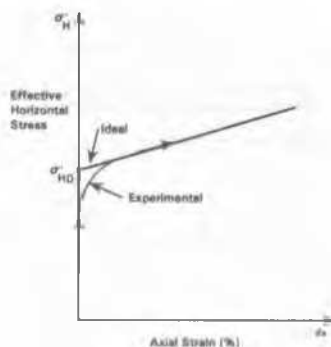


Fig. 2. Ideal and experimental σ_h - ϵ_a plot.

Verification of the method using samples of known stress histories

In order to verify the suitability of the proposed method, slurried clay samples were prepared to induce a typical stress history of an overconsolidated soil in the field. After simulating a known stress history, the sample was tested to verify whether the proposed method would indicate the same horizontal stress as the one imposed artificially. Fig. 3 indicates result of one such verification. Clearly the correspondence between the imposed and estimated values of K_0 is

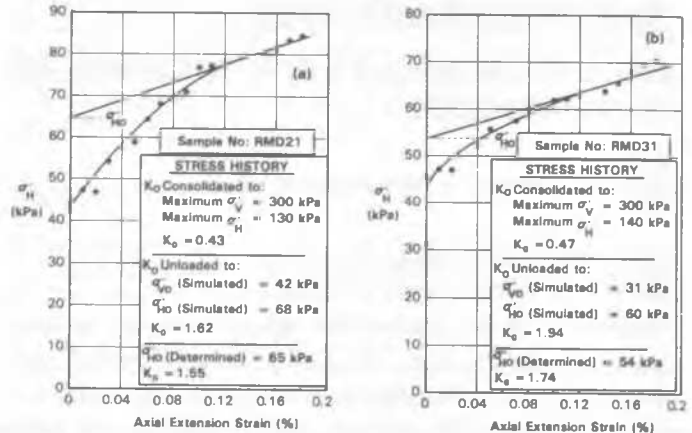


Fig. 3. K_0 determination for artificially prepared samples.

remarkable. A complete discussion of the test method, verification and comparison with other *in situ* tests has been presented by Garga and Khan (1991).

Effect of K_0 on Field Vane Shear Strength

A commonly used method of evaluation of undrained shear strength, for the analysis of embankments and shallow foundations, is the field vane (FV) shear test, and has been widely applied because of its simple and economic nature despite a number of controversial corrections (Schmertmann 1975; Menzies 1976; Azzouz et al. 1983) related to the interpretation of test data. An important concern with the field vane (FV) shear test is whether S_u obtained from this test for overconsolidated soils can be used for design. A number of researchers (Dascal et al. 1972; La Rochelle et al. 1974; Graham 1979; Lefebvre et al. 1987; for example) have observed significantly higher values of S_u from field vane tests than those obtained from conventional laboratory tests and back analyses of field failures. Dascal et al. (1972) conducted a full-scale embankment failure test, and the S_u obtained from the back analysis of this artificially failed embankment was compared with S_u (FV). The factor of safety determined from FV test was 1.6 instead of 1.0. At the same site, Lefebvre et al. (1987) found that the FV test indicated much higher strength when compared with those from field plate loading and large shear box tests, and laboratory triaxial compression and extension tests. They have suggested that the FV test should not be used to determine the S_u of a clay crust, but only its thickness. These inconsistent observations require a re-examination of the analysis of this test method in overconsolidated soils.

Interpretation of field vane test data

In order to obtain a rational interpretation of the field vane test data in overconsolidated soils, it is necessary to examine the key factors influencing the test results. These are discussed below.

(1) *Shear stress distribution around the field vane*: The conventional method of interpretation assumes that the shear stress distribution on both the cylindrical (vertical), and top and bottom (horizontal) surfaces is rectangular, i.e. uniform. The validity of this assumption has been critically re-examined by a number of researchers (Menzies and Mailey 1976; Donald et al. 1977; Menzies and Merrifield 1980; Wroth 1984; Griffiths and Lane 1990). From these studies the assumption of rectangular stress distribution on the cylindrical vertical surface has been confirmed as reasonable, but the stress distributions on the horizontal (top and bottom) shear faces are far from the rectangular distribution as assumed in conventional interpretation. From these observations, Wroth (1984) obtained a relationship of shear stress distribution on the horizontal surfaces described by an expression of the form

$$\frac{S_{u(r)}}{S_{uh}} = \left(\frac{r}{D/2}\right)^n \quad (1)$$

where: r = radial distance from the central axis on the horizontal surface
 D = diameter of the cylinder circumscribed by the vane blades
 $S_{u(r)}$ = undrained shear stress mobilized at a distance r
 S_{uh} = maximum value of $S_{u(r)}$ on the horizontal surface
 n = a coefficient that depends on the shape of the shear stress distribution, as shown in Table 1 in Silvestri and Aubertin (1988).

Thus the portion of the torque M_h carried by the horizontal surfaces is

$$M_h = 2 \int_0^{D/2} S_{u(r)} 2\pi r^2 dr = 4\pi \int_0^{D/2} \frac{S_{uh}}{(D/2)^n} r^{n+2} dr = \frac{\pi D^3 S_{uh}}{2(n+3)}$$

$$\text{or, } M_h = \frac{\pi D^3 S_{uh}}{p_n} \quad (2)$$

where, $p_n = 2(n+3)$ = shear stress distribution factor for horizontal shear planes as shown in Table 1 in Silvestri and Aubertin (1988).

The portion of the torque M_v provided by the vertical cylindrical surface is same as in the conventional analysis, i.e. $M_v = \pi D H (D/2) S_{uv}$, where, H = height of the vane blades. For a standard vane shape, $H/D = 2$, and,

$$M_v = \pi D^3 S_{uv} \quad (3)$$

The ratio of the torque carried by horizontal and vertical shear planes, from Eqs. 2 and 3, is:

$$\frac{M_h}{M_v} = \frac{S_{uh}}{S_{uv}} \frac{1}{p_n} \quad \text{or,} \quad \frac{M_h}{M_v} = \frac{K_s}{p_n} \quad (4)$$

where, $K_s = S_{uh}/S_{uv}$, the coefficient of horizontal undrained strength.

In conventional analysis, when the soil is considered isotropic (i.e. $K_s = 1$), and the shear stress distribution is considered uniform (i.e. $n = 0$ and $p_n = 6$)

$$\frac{M_h}{M_v} = \frac{1}{6} \quad (5)$$

From Menzies and Merrifield's data obtained from instrumented vane test, $n \approx 5$ and $p_n = 16$, giving

$$\frac{M_h}{M_v} = \frac{1}{16} \quad (6)$$

(2) Undrained strength anisotropy: Although the conventional analysis assumes that the shear strength of the soil being measured is isotropic (i.e. $S_{uv} = S_{uh}$), natural clays are generally anisotropic. Aas (1967) has conducted vane shear tests using rectangular vanes with different H/D ratio and diamond shaped vanes to determine the anisotropy of undrained shear strength along horizontal (top and bottom) and vertical (cylindrical side) shear planes in normally consolidated and overconsolidated clays. The value of K_s for normally consolidated clays was between 1.5 and 2.0; for overconsolidated clays K_s was less than 1, and that for slightly overconsolidated clays was close to 1. Aas concluded that in an overconsolidated clay where the *in-situ* effective horizontal stress is greater than the *in-situ* effective vertical stress, the coefficient of undrained horizontal strength should accordingly be less. Aas also concluded that the $S_u(FV)$ is dependent on the *in-situ* effective normal stress on the failure plane.

Silvestri and Aubertin (1988) suggested that there are mainly two reasons which explain strength anisotropy in a natural clay: (i) soil structure which results from particle orientation during sedimentation, and (ii) anisotropic *in-situ* state of stress which is related to stress history of the soil. In a natural soil, perfectly isotropic condition is not generally found. Therefore, for a

realistic analysis, strength anisotropy must be incorporated in the interpretation of field vane test data where a major portion (greater than 86%) of torque is carried by the cylindrical vertical surface of a field vane.

The total torque carried by the vane, as obtained from Eqs. 2 and 3, is given by:

$$M = M_v + M_h = \pi D^3 S_{uv} + \frac{\pi D^3 S_{uh}}{p_n} = \pi D^3 S_{uv} \left(\frac{p_n + K_s}{p_n} \right)$$

$$\text{or, } S_{uv} = \frac{M}{\pi D^3 (p_n + K_s)} \quad (7)$$

$$\text{Similarly, } S_{uh} = \frac{M}{\pi D^3} \left(\frac{p_n K_s}{p_n + K_s} \right) \quad (8)$$

(3) Rate effect: It is well recognized that the rate of loading or rate of straining has a significant influence on the evaluation of undrained shear strength in the field as well as in the laboratory (e.g. Bjerrum 1972; Chandler 1988; Roy and Leblanc 1988 among others). In a standard field vane test, rate of rotation during shearing is 6°/minute where the failure occurs in less than 1 minute. This time to failure (t_f) is small in comparison with that from various laboratory triaxial (UU, CIUC, CK₀UC, etc.) and undrained direct shear tests where t_f may vary from few minutes to several hours. The t_f for full-scale field failure may be much higher. Therefore, a correction for rate effect is essential in order to obtain strength from field vane tests which is comparable to laboratory tests or field failures. Bjerrum (1972) correlated this rate effect, and the less pronounced effect of anisotropy and progressive failure, with plasticity index when he compared field vane test data with field failures.

(4) Disturbance due to vane insertion: There are mainly two consequences of soil displacement due to vane insertion (Chandler 1988): (i) a local destruction of interparticle bonds within the soil, which in the case of more sensitive clays may significantly reduce the available undrained shear strength in the vicinity of the vane; and, (ii) a local displacement of soil particles together with a corresponding increase in the pore pressure around the vane, which on dissipation will result in an increase in effective stress.

Chandler concluded that the undrained shear strength could be underestimated for structured sensitive clays, by as much as 25% if the effect of disturbance (generation of pore water pressure and destruction of soil structure) due to vane insertion is not taken into account. Roy and Leblanc (1988) reported that the insertion of the vane creates disturbance which is linked to blade thickness and to the type of clay under study.

When the FV shear strengths are to be corrected for the rate effect and disturbance due to vane insertion, Eq. 7 and Eq. 8 become

$$S_{uv} = \frac{M}{\pi D^3} \left(\frac{p_n}{p_n + K_s} \right) \mu_R \cdot \mu_D \quad (9)$$

and,

$$S_{uh} = \frac{M}{\pi D^3} \left(\frac{p_n K_s}{p_n + K_s} \right) \mu_R \cdot \mu_D \quad (10)$$

where, μ_R and μ_D are correction factors for rate effect and disturbance due to vane insertion, respectively.

The undrained shear strengths on horizontal and vertical shear faces of a field vane can now be obtained directly from the generalized Eqs. 9 and 10 for anisotropic soils with non-uniform shear stress distribution on horizontal surfaces.

Analysis of Test Results

The re-interpretation of the vane test in overconsolidated soils is demonstrated using test data in the weathered clay crust overlying soft Champlain Sea Clay in Ottawa. The soil shows overconsolidation ratio (OCR) of 16 at 1 m, decreasing with depth to OCR of 3.1 at a depth of 3.7 m. FV tests were conducted at the standard rate of rotation of 6°/minute, at an interval of 40 cm within the crust and 1 m below the crust, using the NGI vane, 43 mm diameter

by 86 mm high. Fig.4 shows the strength profile obtained from these tests by conventional analysis (i.e. $S_u = 0.86M/\pi D^3$ considering uniform stress distributions and isotropic strength with no corrections). From the field vane test profile, the depth of crust may visually be determined at approximately 3 m. There is a significant increase in sensitivity below the crust.

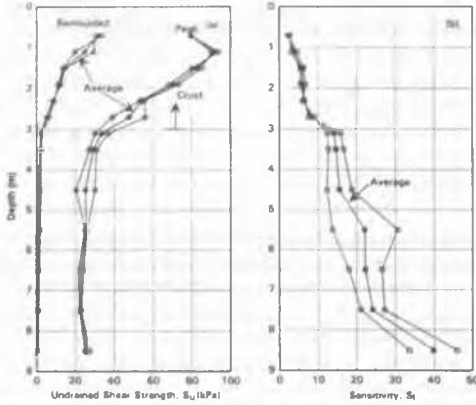


Fig. 4. Results of conventionally analyzed field vane test data.

As a part of this study, conventional direct shear tests, on samples 60 mm square in plan and 20.5 mm in height, were carried out on horizontally and vertically trimmed undisturbed specimens to evaluate the undrained strengths on these two orthogonal planes, and to evaluate the coefficient of horizontal strength, which is assumed to be the same *in-situ*. Two types of triaxial compression tests were also conducted: isotropically consolidated to σ_{vo} (CIUC) (conventional type) and anisotropically consolidated to *in-situ* stress state (CK₀UC). All these laboratory tests were performed for five depth levels: three (1.00, 1.80 and 2.60 m) within the crust and two (3.33 and 3.60 m) immediately below the overconsolidated crust.

Stress distribution used in this analysis: For strain-softening type of clays, the peak strength would be reached first at the vane corners, and by the time peak torque would be attained, the shear stress distribution would be as shown in Fig. 5b (Menzijs and Mailey 1976; Silvestri and Aubertin 1988), and is a consequence of progressive failure (Flaate 1966; Donald et al. 1975). The end shear stress distribution assumed in the present analysis is as shown in Fig.5b since the soil showed strain-softening behaviour in triaxial (CK₀UC) and direct shear tests. The value of p_a in Eqs. 9 and 10, used in this analysis for strain-softening soil, was therefore taken as 7.27 from Table 1 in Silvestri and Aubertin (1988).

Estimation of strength anisotropy: The coefficients of horizontal strength (K_s), at various depths were estimated from direct shear tests. Direct shear (DS) tests were performed on two types of samples. The first type were conventional samples with the shear plane in the horizontal direction, where the applied normal stress was equal to the *in-situ* vertical stress σ_{vo} ' (Fig. 6a). The second type of tests were conducted on samples with the shear plane in the vertical direction where the normal stress during shear was equal to σ_{ho} ' (Fig. 6b). The strength anisotropy obtained from direct shear tests is assumed to be equal to that obtained from field vane tests i.e. $S_{uh}(FV)/S_{uv}(FV) = S_{uh}(DS)/S_{uv}(DS) = K_s$. The variation of K_s with depth (Z) is shown in Fig. 7 which indicates a good correlation. This correlation can be expressed by the following equation:

$$K_s = 0.49(Z)^{0.38} \quad (\text{where } 1 \text{ m} < Z < 4 \text{ m}) \quad (11)$$

Strength anisotropy was accounted in the present analysis incorporating this value of K_s in Equations 9 and 10.

Adjustment for rate effect: Direct shear tests at various strain rates (0.004%-2.0%/minute) were conducted to determine the rate effect on the undrained shear strength of the crust, as shown in Fig.8. These tests were conducted along horizontal shear plane (Fig. 6a) for samples obtained from a depth of 3.6

m. The results indicate a decrease of strength by approximately 9% for each log cycle increase in time to failure (t_f).

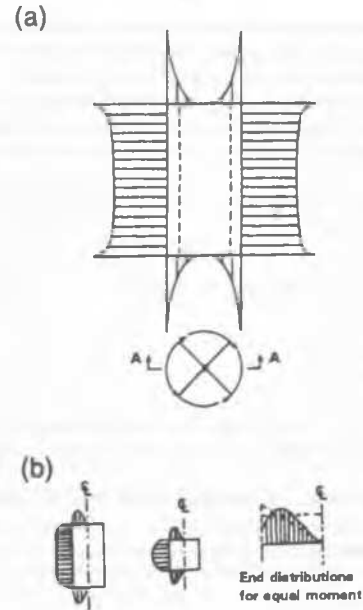


Fig. 5. Stress distribution (Donald et al. 1977). Elastic (a) and plastic (b) shear stress distributions.

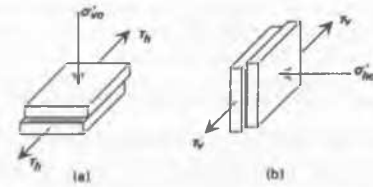


Fig. 6. Orientation of samples for direct shear tests: (a) shear plane in horizontal direction, and (b) shear plane in vertical direction.

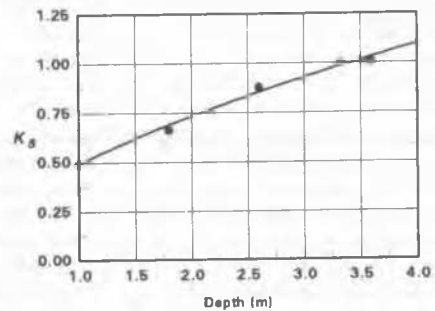


Fig. 7. Correlation between coefficient of horizontal strength (K_s) and depth.

The time to failure in the laboratory undrained tests (triaxial and direct shear) varied between 30 minutes at 3.60 m depth and 160 minutes at 1.0 m depth although the same rate of strain of 0.04%/minute was used for all the laboratory triaxial and direct shear tests. The samples from more shallow depths failed at higher strain level than the samples from greater depths. Time to failure and consequently strain rate were determined using the equation $t_f = 0.51.t_{100}$ minutes, as suggested by Head (1986), where t_{100} is the required time for 100% primary consolidation obtained from triaxial consolidation test.

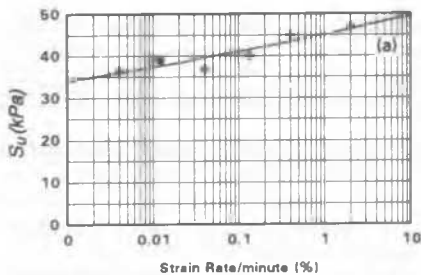


Fig. 8. Rate effect on undrained shear strength.

The average t_f for laboratory undrained shear strength tests can be taken as 100 minutes. The t_f for FV tests was approximately 1 minute. Therefore, for the purpose of comparison between FV and laboratory tests, the S_u (FV) was corrected multiply by 0.83 i.e. $\mu_R = 0.83$ in Eqs. 9 and 10.

Correction for disturbance due to vane insertion: The weathered crust is permeable due to the presence of random micro-fissures. The sensitivity lies between 2 and 8. Hence, the pore-pressure build-up and destruction of structured bonds due to vane insertion in this soil can be assumed to be small and negligible. According to Chandler (1988), this disturbance is unimportant for a soil with sensitivity of less than 15. A μ_D value of 1.0 in Eqs. 9 and 10 has therefore been used in this analysis.

The FV test data, re-interpreted considering the above four factors, are shown in Fig.9. Undrained shear strengths from triaxial compression tests (CIUC and CK₀UC) and direct shear tests (horizontal and vertical samples) are also plotted in Fig.9.

Comparison and discussion of test results

It is evident from Fig.9 that the undrained shear strengths on the vertical plane S_{uv} (FV), within the crust depth, obtained from field vane tests are significantly higher than the undrained shear strengths on horizontal planes S_{uh} (FV). This is to be expected for a soil with $K_0 > 1$ where the normal stress (σ_{ho}) on the vertical shearing face is greater than the normal stress (σ_{vo}) on the horizontal shearing face. It is also evident that the values of S_u (FV) obtained from conventional analysis (Fig. 4a) are close to the values of S_{uv} (FV) but are considerably higher than S_{uh} (FV) as well as the values of S_u obtained from laboratory triaxial compression and direct shear tests (Fig. 9). Moreover, the mode of failure in a vane shear test, where the strength is mobilized on the vertical and horizontal faces, is different from that in the field. Although the values of S_u (FV) obtained from conventional analysis or S_{uv} (FV) obtained from the proposed interpretation technique give considerably higher values of undrained shear strengths within the crust, S_{uh} (FV) corresponds well with S_u (CK₀UC). The comparison between S_{uh} (FV) and S_u (CIUC) or S_{uh} (DS) is also reasonable. Also shown in Fig. 9 is the conventionally interpreted FV strength which has not been corrected for strain rate effect.

The values of S_{uv} obtained from field vane tests and from direct shear tests on samples with vertical shear plane are expected to be similar since the mode of failure on the vertical plane for both of these tests is also similar. However, the match between the two values is not as close as that between S_{uh} (FV) and S_{uh} (DS).

Undrained shear strengths, obtained from field vane tests, for any soil, by conventional analysis, will be close to the undrained shear strength on the cylindrical vertical face (S_{uv}) of the FV since the vertical surface carries major portion (86% or more) of the torque mobilized by the vane. Therefore, in any overconsolidated weathered crust with $\sigma_{ho} > \sigma_{vo}$ (i.e., $K_0 > 1$), it is anticipated that the undrained strength determined from conventional interpretation of the vane test will be higher than the shear strength back calculated from field failure, or from other laboratory tests. However, for normally consolidated clays where $\sigma_{ho} < \sigma_{vo}$, S_u (FV) would underestimate strength since, in this case, the normal stress on the vertical plane (σ_{ho}) is less than that on the horizontal plane (σ_{vo}). Therefore, for realistic analysis, strength anisotropy should be

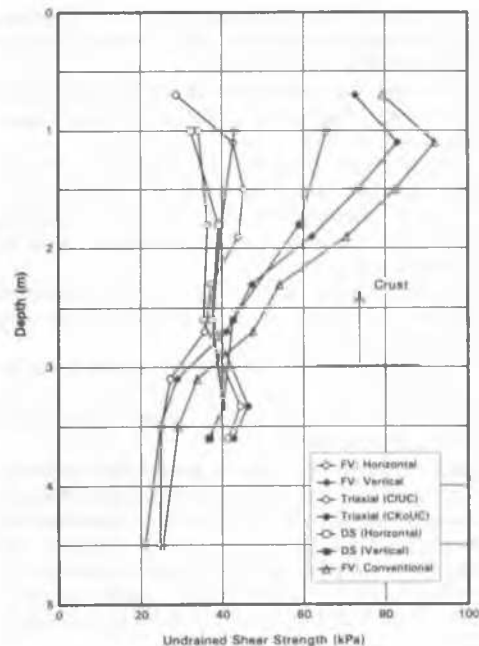


Fig. 9. Undrained shear strengths from field vane, triaxial compression and direct shear tests.

incorporated in the field vane interpretation procedure and the use of a single correction factor for strength anisotropy is unsatisfactory. Assumption of isotropic behaviour of an overconsolidated soil may result in significant error in the estimation of the shear strength. The field vane would provide reasonable estimate of the undrained shear strength, from conventional analysis, only when the soil is isotropic. A detailed discussion of these results has been presented by Garga and Khan (1992).

CONCLUSIONS

1. A simple practical method to evaluate K_0 in overconsolidated soils has been proposed.
2. S_u obtained from FV tests, using conventional analysis, is considerably higher than that obtained from laboratory undrained shear strength tests, and should not be used for overconsolidated soils. This is attributed to the fact that the major portion of the torque mobilized by the field vane is developed on the cylindrical vertical face which is normal to the *in-situ* horizontal stress (σ_{ho}) where $\sigma_{ho} > \sigma_{vo}$. S_{uh} (FV) compares well with S_u obtained from K_0 consolidated laboratory triaxial tests. Therefore, it is suggested that S_{uh} (FV) should be used instead of S_u (FV) from conventional analysis. An interpretation technique which takes the strength anisotropy into account, to obtain the values of S_{uh} (FV), has been discussed.
3. The undrained strength on horizontal plane in direct shear test, S_{uh} (DS), is similar to that obtained from conventional triaxial shear test, S_u (CIUC). These strengths are, however, smaller than S_{uh} (FV) or S_u (CK₀UC).
4. Strength anisotropy varies with depth. This variation should be incorporated in the interpretation of the field vane test data. The use of a single correction factor (Bjerrum 1972) is unsatisfactory.

REFERENCES

- Aas, G. 1965. A study of the effect of vane shape and rate of strain on the measured values of *in-situ* shear strengths of clays. Proceedings, Sixth International Conference on Soil Mechanics and Foundation Engineering, Montreal, Canada, Vol.1, pp. 141-145.
- Aas, G. 1967. Vane tests for investigation of anisotropy of undrained shear strength of clays. Proceedings of the Geotechnical Conference, Oslo, Vol.1, pp. 3-8.

- Azzouz, A.S., Baligh, M.M. and Ladd, C.C. 1983. Corrected field vane strength for embankment design. *ASCE Journal of the Geotechnical Engineering Division*, 109: 730-734.
- Bishop, A.W. 1958. Test requirements for measuring the coefficient of earth pressure at rest. *Proceedings, Conference on Earth Pressure Problems, Brussels*, Vol. 1, pp. 2-14.
- Bishop, A.W. and Henkel, D.J. 1962. *The measurement of soil properties in the triaxial test*. 2nd edition, Edward Arnold, London, U.K., p. 140.
- Bjerrum, L. 1972. Embankments on soft ground. *Proceedings, Specialty Conference on Performance of Earth and Earth-Supported Structures*, Purdue University, pp. 1-54.
- Bjerrum, L. and Andersen, K.H. 1972. *In situ* measurement of lateral pressures in clay. Publication No.91, Norwegian Geotechnical Institute, Oslo, pp. 29-38.
- Brooker, E.W. and Ireland, H.O. 1965. Earth pressures at rest related to stress history. *Canadian Geotechnical Journal*, 2: 1-15.
- Campanella, R.G. and Vaid, Y.P. 1972. A simple K_0 triaxial cell. *Canadian Geotechnical Journal*, 9: 249-260.
- Chandler, R.J. 1988. The *in-situ* measurement of the undrained shear strength of clays using the field vane. *In Vane shear strength testing in soils: field and laboratory studies*, ASTM STP 1014. Edited by A.F. Richards, pp. 13-44.
- Chang, M.F., Moh, Z.C., Liu, H.H. and Viranuvut, S. 1977. A method for determining the *in situ* K_0 coefficient. *Proceedings, 9th International Conference on Soil Mechanics and Foundation Engineering, Tokyo, Vol.1*, pp. 61-64.
- Dascal, O., Tournier, J.P., Tavenas, F. and La rochelle, P. 1972. Failure of a test embankment on sensitive clay. *Proceedings, ASCE Specialty Conference on Performance of Earth and Earth-Supported Structures*, Purdue University, Lafayette, Indiana, Vol.1, pp. 129-158.
- Donald, I.B., Jordan, D.O. and Parker, R.J. 1975. Interpretation of vane testing in clays. *Fifth Symposium on In-Situ Testing for Design Parameters*, Melbourne, Australia, pp. 1-25.
- Donald, I.B., Jordan, D.O., Parker, R.J. and Toh, C.T. 1977. The vane test - a critical appraisal. *Ninth International Conference on Soil Mechanics and Foundation Engineering, Tokyo, Vol.1*, pp. 81-88.
- Flaate, K. 1966. Factors influencing the results of vane tests. *Canadian Geotechnical Journal*, 3: 18-31.
- Garga, V.K. and Khan, M.A. 1991. Laboratory evaluation of K_0 for overconsolidated clays. *Canadian Geotechnical Journal*, 28: 650-659.
- Garga, V.K. and Khan, M.A. 1992. Interpretation of field vane strength of an anisotropic soil. *Canadian Geotechnical Journal*, 29: 627-637.
- Graham, J. 1979. Embankment stability on anisotropic soft clays. *Canadian Geotechnical Journal*, 16: 295-308.
- Griffiths, D.V. and Lane, P.A. 1990. Finite element analysis of the shear vane test. *Computers and Structures*, 37(6): 1105-1116.
- Handy, R.L., Remmes, B., Moldt, S., Lutenegger, A.J. and Trott, G. 1982. *In situ* stress determination by Iowa stepped blade. *ASCE Journal of Geotechnical Engineering Division*, 108: 1405-1422.
- Head, K.H. 1986. *Manual of Soil Laboratory Testing*, Vol.3, John Wiley & Sons, Toronto.
- Jaky, J. 1948. Pressure in soils. *Proceedings, 2nd International Conference on Soil Mechanics and Foundation Engineering, Rotterdam, Vol. 1*, pp. 103.
- Jefferies, M.G. 1988. Determination of horizontal geostatic stress in clay with self-bored pressuremeter. *Canadian Geotechnical Journal*, 25: 559-573.
- Kenney, T.C. 1967. Field measurement of *in situ* stresses in quick clays. *Proceedings of the Geotechnical Conference, Oslo, Vol. 1*, pp. 49-55.
- La Rochelle, P., Roy, M. and Tavenas, S. 1973. Field measurements of cohesion in Champlain clays. *Eighth International Conference on Soil Mechanics and Foundation Engineering, Moscow, Vol.1*, pp. 229-236.
- Lefebvre, G., Pare, J.J., and Dascal, O. 1987. Undrained shear strength in the surficial weathered crust. *Canadian Geotechnical Journal*, 24: 23-34.
- Marrchetti, S. 1980. *In situ* tests by flat dilatometer. *ASCE Journal of Geotechnical Engineering Division*, 106: 299-321.
- Massarsch, K.R. 1975. New method for measurement of lateral earth pressure in cohesive soils. *Canadian Geotechnical Journal*, 12: 142-146.
- Massarsch, K.R., Holtz, R.D., Holm, B.G. and Fredriksson, A. 1975. Measurement of horizontal *in situ* stresses. *Proceedings, Conference on In Situ Measurement of Soil Properties. Specialty Conference of ASCE Geotechnical Engineering Division, Raleigh, North Carolina, Vol. 1*, pp. 266-286.
- Mayne, P.W. and Kulhawy, F.H. 1982. K_0 -OCR relationships in soil. *ASCE Journal of Geotechnical Engineering Division*, 108: 851-872.
- Menzies, B.K. 1976. An approximate correction for the influence of strength anisotropy on conventional shear vane measurements used to predict field bearing capacity. *Geotechnique*, 26: 631-634.
- Menzies, B.K. and Mailey, L.K. 1976. Some measurements of strength anisotropy in soft clays using diamond-shaped shear vanes. *Geotechnique*, 26: 535-538.
- Menzies, B.K. and Merrifield, C.M. 1980. Measurements of shear stress distribution on the edges of a shear vane blade. *Geotechnique*, 30: 314-318.
- Poulos, H.G. and Davis, E.H. 1972. Laboratory determination of *in situ* horizontal stress in soil masses. *Geotechnique*, 22: 177-182.
- Roy, M. and Leblanc, A. 1988. Factors affecting the measurements and interpretation of the vane strength in soft sensitive clays. *In Vane shear strength testing in soils: field and laboratory studies*, ASTM STP 1014. Edited by A.F. Richards, pp. 117-128.
- Schmertmann, J.H. 1975. Measurement of *in situ* shear strength. *Proceedings, ASCE Specialty Conference on In Situ Measurement of Soil Properties. Vol.II, Raleigh, NC*, pp. 57-138.
- Silvestri, V. and Aubertin, M. 1988. Anisotropy and *in-situ* vane tests. *In Vane shear strength testing in soils: field and laboratory studies*, ASTM STP 1014. Edited by A.F. Richards, pp. 88-103.
- Skempton, A.W. 1961. Horizontal stresses in an overconsolidated Eocene clay. *Proceedings, 5th International Conference of Soil Mechanics and Foundation Engineering, Paris, Vol. 1*, pp. 351-357.
- Tavenas, F.A., Blanchette, G., Leroueil, L., Roy, M. and La Rochelle, P. 1975. Difficulties in the *in situ* determination of K_0 in soft sensitive clays. *Proceedings, Conference on In Situ Measurement of Soil Properties. Specialty Conference of ASCE Geotechnical Engineering Division., Raleigh, North Carolina, Vol. 1*, pp. 266-286.
- Wroth, C.P. 1984. The interpretation of *in-situ* soil tests, 24th Rankine Lecture, *Geotechnique*, 34: 449-480.