Cavity expansion model to estimate undrained shear strength in soft clay from Dilatometer

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ABSTRACT: The Dilatometer has rapidly become a common in situ test for evaluating geotechnical properties of clays. In general, current empirical correlations for most engineering properties are in part site specific and considerable scatter between estimated and measured values of soil properties has been reported. At the present time there are at least seven different empirical methods available for estimating undrained shear strength in clays from Dilatometer results. In this paper, a technique based on a simple cylindrical cavity expansion theory is proposed for predicting the undrained shear strength of soft and medium stiff saturated clays using the results of flat Dilatometer tests. The method uses an estimate of the excess pore water pressures generated by an advancing full-displacement probe to predict the penetration effective stress at the probe face. An estimate of the penetration effective stress on the face of the blade after penetration is obtained from (P_o - P_2). A comparison between values estimated using this approach and undrained strength obtained by field vane tests at a several clay sites are presented and show excellent results. The proposed method appears to be superior to existing empirical methods for evaluating undrained strength from the DMT and is generally independent of the site.

1 INTRODUCTION

The Flat Dilatometer has become a common in situ test used by a growing number of geotechnical engineers throughout the world for routine site investigations. The test is also seeing increased usage in a variety of soils and applications (Marchetti, 1980; Lutenegger, 1988). Apart from its use as a profiling tool in which individual pressure measurements may be used to indicate relative changes in stratigraphy, the test has excellent potential for use in estimating several specific soil properties; provided proper interpretation techniques are employed. As suggested by Wroth (1984), such techniques should be well founded in soil mechanics and should be checked against other well established data and/or well documented case histories in which soil behavior can be reliably deduced.

One of the specific uses for the DMT has been to provide an estimate of the undrained shear strength of saturated clays. Generally, comparisons of the predicted strength have been reasonably accurate and generally on the conservative side in softer soils but are less accurate in stiffer soils which exhibit "overconsolidated" behavior. The current procedure for predicting undrained shear strength of clays as proposed by Marchetti (1981) has been shown to be unreliable in some cases and as a result may often require extensive local correlation to develop site specific correlations and a sense of reliability.

This paper presents the results of a field investigation performed to compare the results of the DMT with undrained shear strength in clay obtained with the field vane test. A simple cylindrical cavity expansion model is presented and is proposed as an initial theoretical basis to serve as a framework for interpreting the DMT for undrained shear strength. Issues relating to values of undrained strength obtained from either laboratory tests or other in situ tests are not addressed.

2 BACKGROUND – EVALUATING UNDRAINED STRENGTH FROM DMT

The DMT represents an in situ soil test which has seen rapid growth in use, partly because of its robust construction, simple deployment and operation, and general applicability in a wide range of materials. In fine-grained soil deposits, the DMT is particularly attractive over other in situ tests that might be used; it is faster than a field vane, easier to deploy than a piezocone; and generally makes more sense than a Standard Penetration Test. A specific application of the DMT in these materials is in the evaluation of the undrained shear strength. A number of methods have been suggested for evaluating undrained shear strength from DMT measurements.

2.1 *Marchetti(1980)*

Marchetti (1980) had suggested that a simple empirical relationship could be used to predict the normalized undrained strength of cohesive soils from the DMT lift-off pressure, P_o, according to the expression:

$$s_u/\sigma'_{vo} = 0.22\ (0.5\ K_D)^{1.25} \qquad (1)$$

where: s_u = undrained shear strength, σ'_{vo} = initial vertical effective stress, K_D = DMT Lateral Stress Index = $(P_o - u_o)/\sigma'_{vo}$, and u_o = in situ pore water pressure. This correlation was developed based on the observed comparison between soil overconsolidation ratio (OCR) determined from oedometer tests and K_D and the SHANSEP concept presented by Ladd et al. (1977) in which:

$$(s_u/\sigma'_{vo})_{OC} = (s_u/\sigma'_{vo})_{NC} OCR^m \qquad (2)$$

Using a value of $(s_u/\sigma'_{vo})_{NC}$ equal to 0.22 as suggested by Mesri (1975) based on his observations of Bjerrum's (1972) field vane correction chart and a value of m = 0.8 as suggested by Ladd et al. (1977), Marchetti obtained Eq.1. Marchetti (1980) presented a comparison between Eq.1 and the results of undrained shear strength measurements obtained from laboratory unconfined compression tests, triaxial compression tests, and in situ field vane tests which provided reasonable accuracy for the soils investigated. This technique has been used by a number of investigators to compare with a local data base for individual soil types and it appears from more recent investigations that there is a need for site specific verification (e.g., Chang 1988; Lacasse and Lunne 1988; Powell and Uglow 1988). In some cases, Eq.1 tends to overpredict strength obtained by other lab or field techniques, but more generally, it tends to underpredict strength which would be on the conservative side of design.

It may be useful to consider several points about the application of Eq.1 which may contribute to errors in its use:

(1) The normally consolidated value of normalized strength $(s_u/\sigma'_{vo})_{NC}$ = 0.22 was obtained by Mesri (1975) by combining the results of the variation in field shear strength for "young" and "aged" clays with Bjerrum's (1972) field vane correction, and therefore the strength predicted by eq.1 is apparently a "corrected" field vane shear strength. Recall that this correction factor was obtained from back-calculated embankment failures and was developed to force the factors of safety to 1.0 and then applied to the field vane strength. Bjerrum's correction factor may be considered inappropriate in certain design situations by some engineers since variations in vane testing techniques, determination of plasticity index, analytical procedures, etc., are unknown. It may be more appropriate to obtain a measure of the "uncorrected" strength and let the engineer decide if corrections are appropriate to the given design situation, e.g., embankment stability vs. pile skin friction.

(2) The normalized undrained shear strength parameter of 0.22 σ'_{vo} for normally consolidated clays may provide an appropriate initial approximation but does not appear to accurately depict the laboratory derived strength of all clay soils. Available strength data from direct simple shear tests and reported in the open literature, suggest that normalized undrained strength of NC clays increases slightly with increasing plasticity index. Values of $(s_u/\sigma'_{vo})_{NC}$ range from about 0.19 to 0.50 over the range in P.I. from 5 to 90. Some of this variation may be because of difference in test procedures and equipment used even within the same type of test however the results suggest a significant source of error when applying Eq.1. Similar observations have been suggested by other investigators (e.g., Larrsson 1982).

(3) Some engineers may argue that the use of Eq.2 is not generally appropriate for describing the relationship between normalized undrained strength and OCR in other than artificially sedimented soils prepared in the laboratory or very soft young deposits which have not developed any substantial structure. Natural soil deposits which have developed an overconsolidated crust from mechanisms other than simple unloading may have a shear strength relationship which deviates considerably from that described by Eq.2.

(4) In a summary of a large number of available test results, Mayne (1980) showed that the value of m in Eq.2 varied considerably for different clays, ranging from 0.20 to 0.95. The value of m = 0.8 presented by Ladd et al. (1977) was for direct simple shear results, and there is evidence (Mayne 1980) that the value of m varies depending on test conditions for the same soil, e.g., simple shear vs. triaxial CK_oUE vs. triaxial CK_oUC. Additionally, m may

vary with strain rate and other factors which are as yet unknown.

(5) The reference data which were used as the basis for comparison for the results given by Eq.1 were obtained from a number of different laboratory and field tests yet P_o is obviously obtained from the same technique. More appropriately, since undrained shear strength in clays is a function of test technique and other factors, a single test procedure would be desirable for developing a correlation. It should be recognized that even within a single reference test, such as the field vane test, variations in test equipment such as vane length-to-diameter ratio, vane geometry, blade thickness, torque measurement technique, etc. and test procedures such as strain rate, waiting time, etc., may produce different results.

As indicated, comparisons between Eq.1 and measurements of undrained strength using some reference value show a wide variation. Several investigators have presented comparisons with field vane strength and laboratory or other field strength tests. Naturally one would suspect variations because of the reasons previously described. Additionally, it should be remembered that the correlation presented by Marchetti (1980) was developed on a relatively small database and as the base has expanded to other soils variations in accuracy should be expected. Figure 1 shows a comparison of a number of reported correlations between K_D and normalized undrained shear strength illustrating this variation.

The writer (Lutenegger 1988) previously had shown that the accuracy of Eq.1 in predicting the uncorrected field vane strength in clays was related to the DMT material index, I_D, (= $(P_1 - P_o)/(P_o-U_o)$) which generally describes the drainage characteristics of the test; i.e., low I_D indicates undrained while high I_D indicated drained. As I_D increases, it appears that the error in the estimated strength increases. These results may help explain some of the variations obtained by other investigators.

2.2 *Roque et al. (1988)*

An alternative approach to estimating the undrained shear strength was presented by Roque et al. (1988) using a simple bearing capacity approach as:

$$s_u = (P_1 - \sigma_{HO})/N_c \qquad (3)$$

where: P_1 = DMT 1 mm expansion pressure; σ_{HO} = in situ total horizontal stress = $K_o\sigma'_{vo} + u_o$; N_c = bearing capacity factor. Values of N_c varying from 5 to 9 were suggested by Roque et al. (1988) as:

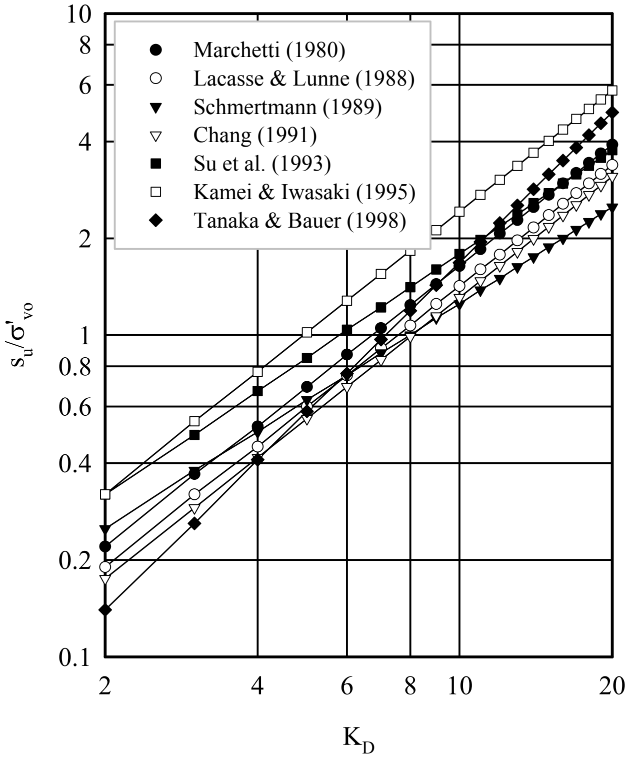

Figure 1. Comparison of several proposed DMT undrained strength correlations.

Soil	N_c
Brittle clay & silt	5
Medium clay	7
Nonsensitive plastic clay	9

This procedure is similar to the semi-empirical approach used to predict undrained shear strength from a prebored (Menard type) pressuremeter using the limit pressure, P_L, where:

$$s_u = (P_L - \sigma_{HO})/N_p \qquad (4)$$

In Eqs. 3 and 4, it is assumed that a limit pressure is obtained during the expansion phase of the test such that $P_1 = P_L$. For the pressuremeter, values of N_p from the literature are often in the range of 5 to 7 which compares well with values of N_c suggested by Roque et al. (1988). This technique requires a value of the in situ horizontal stress and some assumption of the soil type to estimate the bearing capacity factor, N_C. One could estimate K_o from the DMT K_D, however this may introduce an additional source of unknown error.

2.3 *Schmertmann (1989)*

Schmertmann (1989) presented an explanation for an expected trend between K_D and the undrained strength based on the limit pressure from cylindrical cavity expansion. For an ideal elastic-plastic, cylindrical expansion in saturated clay with Poisson's ratio = 0.5, the undrained strength may be obtained from:

$$s_u = P_L^*/[1+\ln(E/3s_u)] \qquad (5)$$

where: P_L^* = net limit pressure = $P_L - (K_o\sigma'v_o + u_o)$. The denominator of Eq.5 may be replaced with:

$$\lambda = 1 + \ln(E/3\, s_u) = 5.2 \text{ to } 7.5 \qquad (6)$$

for $200 < E/s_u < 2000$

The normalized undrained strength may then be written as:

$$s_u/\sigma'_{vo} = [(P_L-u_o)/(\sigma'_{vo} - K_o)]/\lambda \qquad (7)$$

In soft clays, (i.e., OCR < 2.5) it has been noted that the DMT lift-off pressure, P_o, is approximately equal to the limit pressure obtained from a pressuremeter (Lutenegger 1988), therefore one can reasonably substitute the value of P_o for P_L in Eq.7. Noting that by definition:

$$K_D = (P_o-u_o)/\sigma'_{vo} \qquad (8)$$

gives:

$$s_u/\sigma'_{vo} = (K_D - K_o)/\lambda \qquad (9)$$

Schmertmann (1989) suggested that since K_o may be expressed in terms of K_D using the empirical equation presented by Marchetti (1980) and using a reasonable value of $\lambda = 6$ from pressuremeter tests, that a good approximation for predicting the normalized undrained strength would be:

$$s_u/\sigma'_{vo} = K_D/8 = (P_o-u_o)/(8\, \sigma'_{vo}) \qquad (10)$$

While this technique derives from initially sound theoretical basis from cylindrical cavity expansion, it may suffer from at least two potential sources of error:

(1) Experimental data presented by Lutenegger and Blanchard (1990) have shown that the limit pressure from a full-displacement pressuremeter, which is installed in a manner similar to the DMT, is more accurately predicted by the DMT 1 mm expansion pressure, P_1, for a wide range of clays. This means that it may be more appropriate to substitute P_1 for P_L in Eq.7. Dividing through by the vertical effective stress, this expression becomes identical to Eq.3. Use of Eq.10 then would result in a conservative estimate of undrained strength since $P_o < P_1$. The error will be least for soft clays since P_1 will be close to P_o and greatest for stiff clays where P_1 is much greater than P_o.

(2) The use of Eq.10 indirectly uses an empirical correlation between K_D and K_o, which may also introduce an unknown error.

2.4 Yu et al. (1993)

Yu et al. (1993) performed a numerical study of the undrained penetration mechanics of the DMT by modeling the penetration of the blade as the expansion of a flat cavity. An elastoplastic soil model was used and a plane strain condition was assumed so that no strain was permitted in the vertical direction. The results of this study indicated that the lift-off pressure is a function of the initial horizontal stress, the undrained shear strength, and the rigidity index of the soil. It was found that the normalized lift-off pressure, defined as:

$$N_{po} = (P_o - \sigma_{HO})/s_u \qquad (11)$$

N_{po} was not a constant, but increases with the rigidity index of the soil as:

$$N_{po} = -1.75 + 1.57 \ln(G/s_u) \qquad (12)$$

For typical values of rigidity index for clays, the normalized lift-off pressure would range from about 3.6 to 8.3. Rearranging Eq. 12 and solving for s_u would give:

$$s_u = (P_o - \sigma_{HO})/N_{po} \qquad (13)$$

2.5 Kamei and Iwasaki (1995)

A suggestion was made by Kamei and Iwasaki (1995) that for soft clays and peat, a correlation could be established between the undrained shear strength obtained from laboratory UU triaxial compression tests and unconfined compression tests and the DMT elastic modulus, E_D, as:

$$s_u = 0.018\, E_D \qquad (14)$$

The correlation was based on results of tests conducted in Holocene deposits, all of which have undrained strengths less than 100 kPa. It may be reasonable to expect such a correlation in very soft soils since the value of P_1 is only slightly higher than P_o, giving very low values of I_D. Since E_D reflects the difference in going from P_o to P_1 it is reasonable to expect that as strength increases E_D also increases.

3 PROPOSED MODEL FOR ESTIMATING UNDRAINED STRENGTH

It may be possible to use a different approach to predicting the undrained strength in saturated soft clays from the DMT by evaluating the installation effective stress acting on the face of a full-displacement (closed-end) probe. Soil movements during the installation of a full-displacement driven cylindrical pile have been described by Carter et al. (1979) as involving purely radial straining. The use of undrained cavity expansion theory provides analytical and numerical methods to predict the installation stresses in the soil adjacent to the pile face. These studies have been summarized by Randolph et al. (1979), Wroth et al.(1979), and Carter et al. (1979).

From cylindrical cavity expansion theory, the installation radial effective stress acting at the face of a cylindrical probe or pile may be given as:

$$\sigma'_r = [1 + (3/M)^{0.5}]\, s_u \qquad (15)$$

where: s_u = initial (in situ) undrained shear strength prior to installation; M = critical state line gradient. This prediction of effective radial stress resulting from full-displacement installation assumes that the soil adjacent to the shaft of the pile is at critical state under plane strain conditions with a radial major principal stress. The plane strain value of the critical state line gradient, M, may be obtained from:

$$M = 3 \sin \varphi'_{ps} \qquad (16)$$

where: φ'_{ps} = plane strain friction angle. By rearranging terms, eq.15 may be rewritten in terms of the undrained strength as:

$$s_u = \sigma'_r/\alpha \qquad (17)$$

where: $\alpha = [1 + (3/M)^{0.5}]$. For most clays, reasonable values of φ'_{ps} range from about 20^o to 30^o, and from Eq.17, it follows that α only varies from 2.56 to 2.72. This represents a maximum difference of only about 6%. Therefore, a reasonable estimate of the undrained strength from the initial installation effective stress for a cylindrical cavity expansion may be obtained as:

$$s_u = \sigma'_r/2.65 \qquad (18)$$

Eq.18 suggests that an estimate of the in situ undrained shear strength may be obtained from full-displacement probes provided that an evaluation of the installation radial effective stress at the soil/probe interface may be made. In most situations this would require a measurement of both the installation radial total stress and total (excess + in situ) pore water pressure at the face of the probe. For most in situ tests, this is not done. Usually, one or the other is measured, but not both. A comparison between predicted and measured installation stresses on a small diameter model pile using this theory was presented by Coop and Wroth (1989) and showed very good results.

4 INSTALLATION EFFECTIVE STRESS ON DMT

The DMT is an instrument which is designed to provide measurements of total stress and has only been equipped to measure pore water pressures as a research tool (Robertson et al., 1988; Campanella and Robertson, 1991). A tool designed to investigate pore water pressures generated by the DMT blade has also been described as the Piezoblade (Boghrat and Davidson, 1983; Lutenegger and Kabir, 1988). It has been shown by several investigators that the total stress value obtained from the DMT lift-off pressure, P_o, is nearly identical to the initial penetration stress from a cylindrical probe (e.g., Full-Displacement Pressuremeter or Lateral Stress Cone).

Robertson et al. (1988) and Lutenegger and Kabir (1988) have shown that the recontact pressure, P_2, obtained from the DMT, is essentially a pore water pressure measurement. Since the P_2 reading is obtained about 1 min after penetration because of the time to inflate the probe to obtain P_o and P_1 and then deflate to obtain in P_2, one would expect this value to be slightly lower than the pore pressure obtained from the Piezoblade which is obtained on installation. It appears that during penetration, at least in soft and medium stiff clays, the effective stress conditions around a cylindrical probe and the DMT do not differ that much. This is probably related to the fact that the aspect ratio of the DMT blade (width/thickness) is not all that far removed from an axisymetric condition and is far from plane strain conditions. In terms of the measurements taken with the DMT, Eq.18 may be rewritten as:

$$s_u = (P_o - P_2)/2.65 \qquad (19)$$

Therefore, it may be that a simple cavity expansion approach may be used to obtain an estimate of the undrained shear strength from the DMT using two pressure readings. The author recommends that the P_2 measurement be taken routinely as a part of the test and therefore this approach does not require any significant modification to the equipment or procedure. The pressure must be released from the blade after the P_1 reading is obtained before the blade can be advanced to the next test depth anyway; the only difference being that the C-Reading requires slow controlled rather than rapid deflation. Unlike the method presented by Marchetti (1980) the proposed technique does not require estimates of the vertical effective stress or the in situ pore water pressure, both of which may introduce errors.

5 RESULTS

In order to evaluate the accuracy of applying Eq.19 to predict the undrained shear strength of natural clays, a field testing program was conducted at several test sites using both the DMT and field vane test. The approach is illustrated herein using results obtained at four test sites. Table 1 presents a summary of the sites presented. In most of the cases, the sites have a weathered surficial crust which exhibits stiffer overconsolidated behavior.

Table 1. Sites Used to Illustrate Method.

Site	Soil
UMass	Lacustrine soft clay with stiff clay crust
IDA	Marine clay - moderately sensitive
St. Albans	Marine clay - highly sensitive
Bothkennar	Marine clay - sensitive

Dilatometer tests were performed using a standard DMT blade. At each test depth (generally intervals of 0.3 m) the three pressure readings corresponding to P_o, P_1, and P_2 were obtained. The DMT and vane profiles were generally performed within a distance of about 1.5 m. At sites investigated by the author, field vane tests were conducted using a Nilcon Vane Borer with a self-recording torque head. Tests were performed using a 65 mm diameter rectangular vane with a height to diameter ratio of 2 and a blade thickness of 1.5 mm. Tests were performed within one minute of the vane insertion.

The first two test sites (UMass and IDA) were tested by the author. Field vane results from St. Albans were taken from the literature (LaRochelle et al. 1974). Dilatometer and field vane results from Bothkennar were taken from the literature (Nash et al. 1992). These four sites were selected to illustrate the accuracy of the proposed method. To date, the method has been applied to 18 different sites with similar results.

5.1 UMass

Figure 2 shows test results obtained in the Connecticut Valley Varved clay at the UMass site in western Massachusetts.

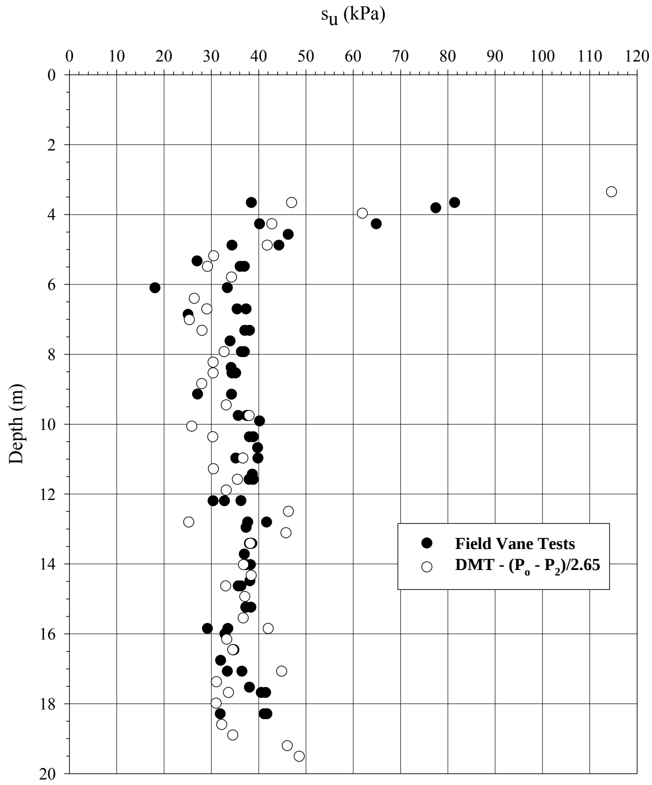

Figure 2. DMT Results at UMass.

5.2 IDA

Figure 3 shows test results obtained in the marine clay at the IDA site in northern New York.

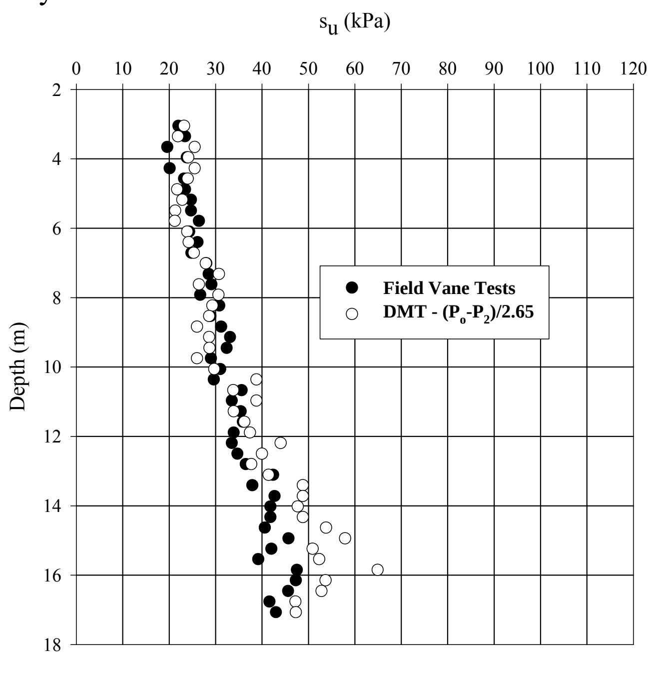

Figure 3. DMT Results at IDA.

5.3 St. Albans

Figure 4 shows test results obtained in the marine clay at the St. Albans site in southern Ontario.

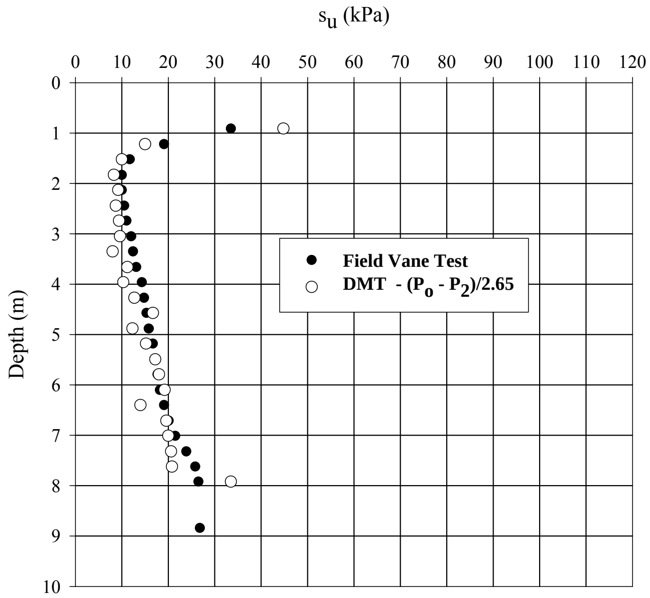

Figure 4. DMT Results at St. Albans.

5.4 Bothkennar

Figure 5 shows test results obtained in the marine clay at the Bothkennar site in Scotland.

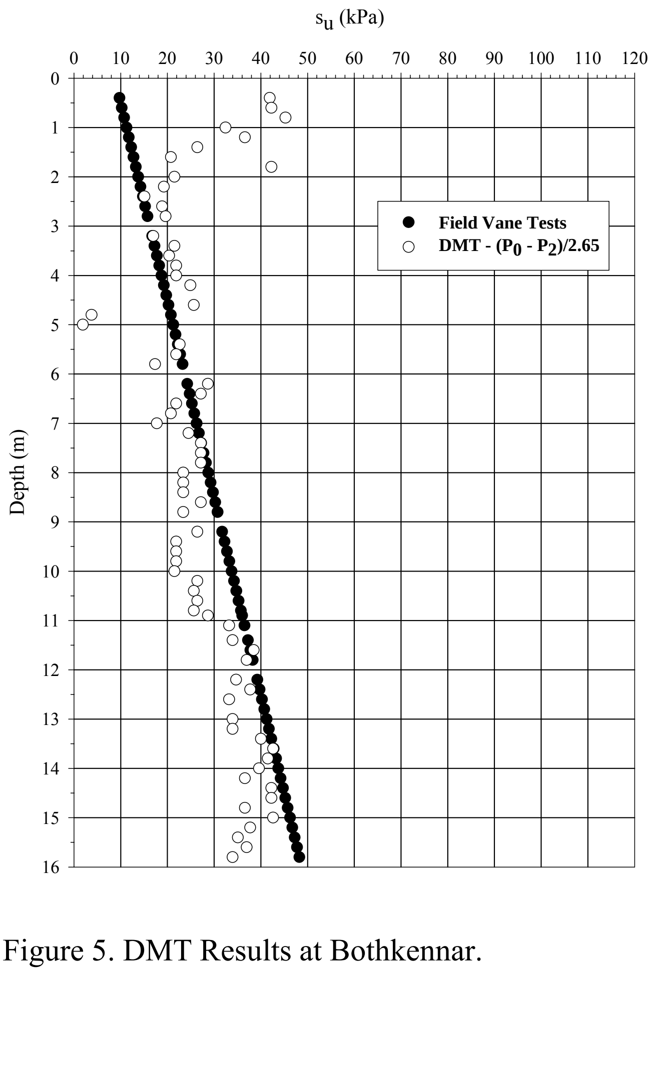

Figure 5. DMT Results at Bothkennar.

A comparison using the method proposed in this paper and expressed by Eq. 19, for all of the results obtained by the author from the field vane and DMT tests shows the results to be grouped between α = 2.0 to 3.0 which fits well with Eq.18. The correlation does not appear to be site specific. Additional examination of the test results is needed to investigate the dependence of α on other specific soil characteristics, such as Plasticity Index (P.I.) and the stress history (OCR) as data become available.

6 DISCUSSION

There are both advantages and disadvantages to the method presented in this paper. These may also be considered in regard to the correct application and potential limitations of the method.

6.1 Disadvantages/Limitations

1. The proposed method often requires the subtraction of two numbers which are relatively close to each other; i.e., the difference between two large numbers. This means that there may be some question about the precision of the resulting number. In order to obtain reliable values for the lift-off (A) and recontact (C) pressure readings operators should be instructed to be careful in performing the test.
2. The method requires an additional pressure reading to be obtained over the two pressure readings originally presented by Marchetti (1980). The author considers this pressure reading of significant importance to the test; some engineers may consider this an unnecessary complication of the test and one which just can lead to confusion for the operator.
3. In order to accurately obtain the recontact pressure reading, a modification to the control console may be necessary by incorporating a flow control needle valve in the deflation pressure circuit.
4. The method is limited by the applicability of Eq. 15. The interpretation assumes that the soil adjacent to the blade is at critical state which may not always be true, especially for overconsolidated soils.
5. It is assumed that the recontact pressure is an accurate representation of the total pore water pressure acting on the face of the blade. As previously shown, this assumption appears to be adequately justified in softer materials (lightly overconsolidated to near normally consolidated) but will certainly be incorrect in the case that negative shear induced pore water pressures are generated. This is because it is not possible to measure a value less than zero on the control console.
6. The test procedure may adversely influence the results. Data presented by Powell and Uglow (1986)

have shown that the recontact pressure may increase if the diaphragm is inflated past the 1 mm pressure (B-reading). Therefore it is important that the operator shut off the inflation valve and begin deflation immediately when the B-Reading is obtained.

6.2 Advantages

1. The proposed method makes use of two pressure measurements obtained from the test to make a prediction of a single soil behavioral property. This means that the correlation should be stronger than methods which use only a single measurement to predict a property.
2. The method makes use of a theory which provides a direct connection from the measurements to the predicted property. There is no required assumption of normalized behavior or normally consolidated behavior or consolidated state.
3. Unlike the method of Marchetti (1980) in which the in situ total stress and in situ pore water pressure at the test depth must be known in order to evaluate the strength, the proposed method does not require input of either total stress or in situ pore pressures. This may be especially advantageous in situations where the in situ pore water pressures are not known or are not hydrostatic and in situations where the vertical stress is difficult to evaluate, such as below fills or adjacent to structures.
4. The method does not appear to be site specific, requiring a new correlation to be developed with each new geologic material or area tested and appears to be reasonably successful in a number of different materials representing a wide range of geologies, plasticity, OCR, sensitivity, etc. Since a single concept based on soil behavior and single reference strength is used, this may be expected.

7 CONCLUSIONS

The results presented in this paper have shown that there is a sound theoretical basis by which the results of Dilatometer Tests may be used to estimate the undrained field vane strength of soft clays. The method requires the measurement of the recontact pressure, P_2. On the basis of comparisons with field vane strengths obtained at several sites, the test results suggest that the approach is sound. It is suggested however, that since the data base presented was obtained using a field vane as the basis for comparison, any precautions which an engineer might normally take when using field vane data because of uncertainties in its application to design should still be applied.

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