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Current status of the Marchetti dilatometer test

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ABSTRACT: The use of the Marchetti Dilatometer Test is rapidly expanding worldwide such that the test is becoming one of the premier in situ tests available to the geotechnical profession. The current state-of-the-practice of the test is described and a summary of current usage is presented. A review of the procedures for determining conventional design parameters of soils and an assessment of the quality of predicted field performance is presented and discussed.

1 INTRODUCTION

The Marchetti Dilatometer Test (DMT) has rapidly grown in the past decade to one of the more popular in situ tests available to researchers and practicing engineers. This rapid growth seems to reflect a need in the geotechnical profession for simple, rapid, and cost-effective tools to characterize sites for geotechnical projects. The DMT seems to possess most of the preferred qualitites of in situ tests, i.e., it is simple to operate, rugged, non-electronic, can be used with a wide variety of practical insertion equipment and appears to give very reproducible results.

In situ tests fulfill a very real need of many practicing engineers; namely, they provide rapid estimates of soil parameters for predicting field performance of real structures. This is true of both long-standing tests, such as the Standard Penetration Test and Vane Shear Test, and relatively newer tests, such as the Dilatometer Test. However, even tests with a long history of use by the profession are not without problems, e.g., the high variability of the SPT and the need for a shear strength correction obtained from the field vane.

Like all soil tests, the DMT may have limitations and may not apply to all geotechnical materials and problems. The purpose of this paper is to provide a review of the current status of the DMT with respect to both predicting conventional geotechnical parameters and predicting field performance of geotechnical works. In providing such a review, it is appropriate to briefly look at where the test has been, what changes have occurred and finally what new developments have recently taken place.

At the preparation of this paper, it is evident that the worldwide use of the DMT has expanded significantly in the past 10 years. This is made obvious by the wide range of materials wherein the test has seen use. Using predominantly information available in the open literature, Table 1 summarizes reported usage by the profession not including the original work presented by Marchetti (1980). Quite obviously, practicing engineers, who are by nature keen to expand any technique to new areas, probably have encompassed an even wider range of materials.

Even so, Table 1 clearly demonstrates that the DMT has been used in a broad range of materials: cohesive and cohesionless; saturated and partially saturated; normally consolidated and overconsolidated; "quick" and very stiff; natural and artificial. There appear to be only minor limitations in regard to use of the DMT in natural geologic materials, namely, bouldery glacial sediments or gravelly deposits, both of which resist penetration and may damage the blade and/or diaphragm. Offshore use of the DMT has been reported by Marchetti (1980), Burgess (1983), Sonnenfeld et al. (1985) and Lacasse and Lunne (1988). A specially designed DMT for offshore use is currently under development at NGI.

Table 1. DMT tested materials.

Material	Reference
Sensitive marine clay	Lacasse and Lunne (1983) Fabius (1985) Bechai et al. (1986) Hayes (1986) Lutenegger and Timian (1986) Lutenegger (1987)
Soft non-sensitive clays	Minkov et al. (1984) Ming-Fang (1986) Saye and Lutenegger (1988a)
Lacustrine clay	Chan and Morgenstern (1986)
Glacial tills and/ or very stiff over-consolidated clays	Davidson and Boghrat (1983) Schmertmann and Crapps (1983) Powell and Uglow (1986)
Sand	Schmertmann (1982) Baldi et al. (1986) Clough and Goeke (1986) Lacasse and Lunne (1986) Schmertmann et al. (1986)
Deltaic silt	Campanella and Robertson (1983) Konrad et al. (1985)
Loess	Lutenegger and Donchev (1983) Hammandshiev and Lutenegger (1985) Lutenegger (1986)
Peat	Hayes (1983) Kaderabek et al. (1986)
Compacted fill	Borden et al. (1985)
Industrial slime	GPE (1984)
Soft/medium rock	Sonnenfeld et al. (1985)

2 CONDUCTING THE TEST

The DMT consists of a flat-plate penetrometer which is instrumented with a flexible, circular diaphragm mounted on one face of the blade and a console to provide operational control. The dimensions and geometry of the blade are shown in Fig. 1. A detailed recommended procedure for conducting the test has been presented by ASTM Subcommittee D18.02 (Schmertmann, 1986a)and will only briefly be summarized here.

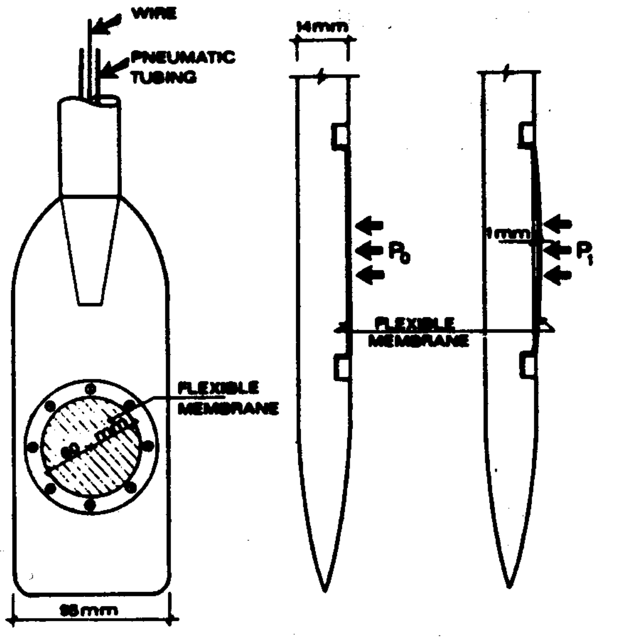

Figure 1. Marchetti Dilatometer

Immediately after the blade is forced into the ground to a desired test depth, preferably by quasi-static penetration, the flexible diaphragm is expanded by compressed gas. As gas pressure is slowly increased and the membrane moves outward against the soil, an electric signal identifies the pressure required for the diaphragm to lift off the plane of the blade. As diaphragm expansion continues, a second electric signal denotes when a central diaphragm expansion of about 1 mm is reached. The two pressures are denoted as the A- and B-Reading, respectively. These pressures are corrected for diaphragm inertial resistance via a simple calibration procedure such that:

$$p_o = A + A \text{ correction} \quad (1)$$

$$p_1 = B - B \text{ correction} \quad (2)$$

The pressures p_o and p_1 are used along with an estimate of the vertical effective stress, σ'_{vo}, and in situ pore pressure, u_o, at the elevation of the test, to provide three indices denoted as:

$$\text{Material Index; } I_D = \frac{p_1-p_o}{p_o-u_o} \quad (3)$$

$$\text{Horizontal Stress Index; } K_D = \frac{p_o-u_o}{\sigma'_{vo}} \quad (4)$$

$$\text{Dilatometer Modulus; } E_D = 34.7\ (p_1-p_o) \quad (5)$$

The use of these indices for predicting a variety of soil parameters was proposed by Marchetti (1980) who presented a series of empirical correlations based on more conventional laboratory and field test data. The interrelationships between these index values and soil parameters proposed by Marchetti (1980) and others are summarized in Table 2.

A third pressure reading designated as C may also be obtained by controlled gas deflation after obtaining the B-Reading and denotes the pressure at which the diaphragm recontacts the plane of the blade. The pressure p_2 is obtained from the C-Reading as:

$$p_2 = C - A \text{ correction} \tag{6}$$

This pressure reading has only recently been introduced and its use has not been fully investigated. An additional DMT index has been proposed (Lutenegger and Kabir, 1988) which has the form:

$$U_D = \frac{p_2 - u_o}{p_o - u_o} \tag{7}$$

The use of U_D will be discussed further in a later section.

3 ESTIMATING SOIL PARAMETERS

One of the main uses of the DMT is to provide estimates of a number of conventional soil parameters, e.g., undrained strength, K_o, compressibility, etc. Since the test may be conducted at intervals of about 20 cm, for many projects a large amount of data is obtained. This allows the use of statistical analyses for probabilistic designs. Since tests may be initiated at depths as shallow as 20 cm, this may be particularly advantageous for pavement subgrades, shallow foundations and laterally loaded pile problems. As with most in situ penetration tests, the values obtained are estimates; the use of which is often to give the range of actual values. Quite often the use of penetration tests is in the form of preliminary site investigations and the tool is used for rapid identification of problem layers where more detailed in situ or laboratory tests may be required. However, the DMT has often provided accurate estimates of soil properties and therefore it is appropriate to review these predictions.

Table 2. Interrelationships between soil parameters and DMT Indices.

Soil Parameter	DMT Index	Reference
s_u (clays)	I_D, K_D	Marchetti (1980)
ϕ' (sands)	I_D, K_D, thrust or adjacent q_c	Schmertmann (1982) Marchetti (1985)
K_o (clays)	I_D, K_D	Marchetti (1980) Marchetti (1986)
K_o (sands)	K_D, thrust	Schmertmann (1982)
OCR (clays)	I_D, K_D	Marchetti (1980)
OCR (sands)	K_D, thrust	GPE (1983)
M	I_D, E_D, K_D	Marchetti (1980)
E_i	I_D, E_D	Robertson et al. (1988)
E_{25}	E_D	Campanella and Robertson (1983) Baldi et al. (1986)
Cyclic stress ratio to cause liquefaction	K_D	Robertson and Campanella (1986)
k_h (subgrade reaction modulus)	p_o, K_D	Schmertmann and Crapps (1983) Robertson et al. (1988)
CBR	E_D	Borden et al. (1985)

3.1 Site Stratigraphy

The Material Index, I_D, given in Eq. [3] was used by Marchetti (1980) as a simple means of identifying soil type. Although no specific values could be related to grain-size distribution, the value of I_D appeared to be reproducible in similar materials and had a range of only about 2 magnitudes roughly 0.1 to 10.0. Marchetti proposed a simple classification scheme, shown in Table 3, so that the engineer would have some means of identifying soil type. This is particularly appealing to engineers since no sample is obtained. Similar schemes are used in other penetration tests such as the CPT and CUPT.

I_D is a measure of the relative change from p_o to p_1 with respect to the original

Table 3. Proposed Soil Classification Based on I_D.

Peat or Sensitive clays	CLAY		SILT			SAND	
		Silty	Clayey		Sandy	Silty	
I_D values	0.10	0.35	0.6	0.9	1.2	1.8	3.3

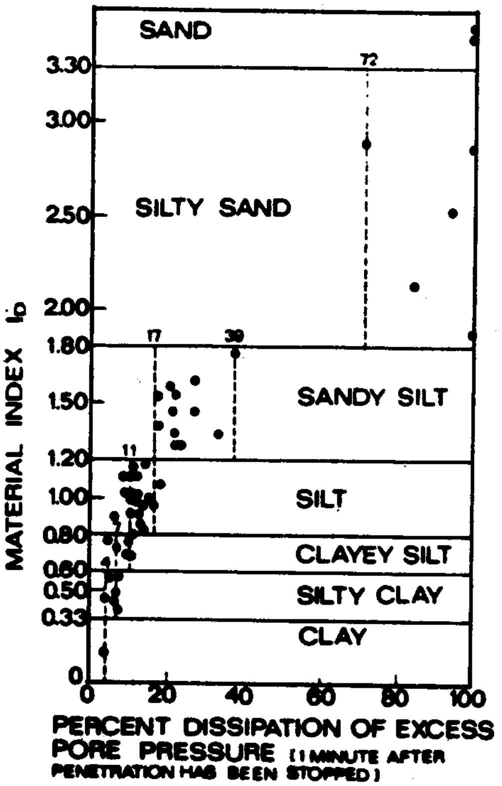

Figure 2. Degree of Dissipation of Excess Pore Water Pressure 1 min. After Penetration as a Function of I_D (Davidson and Boghrat, 1983)

p_o, corrected for static pore water pressures. Using an instrumented DMT, Campanella et al. (1985) showed that in soft clays, the increase in pressure during the expansion from p_o to p_1 was equally matched by an increase in pore water pressure; while in sands, the pressure increase from p_o to p_1 occurs without generating pore water pressures, and is a real measure of soil response. Therefore, one might logically conclude that I_D is a measure of the response of the soil to the increased cavity expansion. As will be shown later, in soft clays, p_o should closely predict P_L from a pressuremeter, while in overconsolidated clays and sands, P_L is closer to p_1. I_D also reflects the degree of drainage which takes place in a particular soil and therefore should be related to permeability or the coefficient of consolidation, Fig. 2.

Is I_D a material property or is it dependent on other parameters, such as degree of saturation, etc? The writer conducted a series of tests at a site in which partially saturated silts were artificially wetted. After each successive wetting, a DMT profile was conducted. Results are shown in Fig. 3. With increasing saturation there is a systematic decrease in K_D and E_D as the material becomes softer, however, the Material Index I_D, remains essentially constant, indicating the same material type. Similar results have been reported by Schmertmann (1982) and Lacasse and Lunne (1986) in compacted sands.

It is the writer's opinion that I_D may be potentially useful as a measure of other soil properties such as sensitivity in cohesive soils. However, at the current time it is used predominantly as an approximate indication of soil type and as a qualifier for application of empirical correlations.

3.2 Undrained Shear Strength

Marchetti (1980) suggested estimating s_u based on:

$$s_u = f\,(K_D) \qquad [8]$$

This relationship is based on the suggestion by Ladd et al. (1977) that:

$$\frac{\left(\frac{s_u}{\sigma'_{vo}}\right)_{OC}}{\left(\frac{s_u}{\sigma'_{vo}}\right)_{NC}} = OCR^m \qquad [9]$$

i.e., many clays exhibit normalized undrained shear strength behavior. Thus, the estimate of undrained strength provided by the DMT is via OCR through the empirical relation to K_D, which is in turn linked directly to p_o as given by Eq. [4]. However, as reviewed herein it can be shown that s_u also correlates to the DMT via limit pressure or pore pressure behavior concepts.

Several investigations (e.g., Lacasse and Lunne, 1983; Fabius, 1985; Greig et al., 1986; Lutenegger and Timian, 1986; Ming-Fang, 1986) have shown that the DMT prediction of undrained shear strength in soft saturated clays compares very well with <u>uncorrected</u> field vane results. In stiff overconsolidated clays, the current correlation appears less accurate. Why does the DMT provide accurate results of s_u in soft clays and how accurate is it?

The quality of the correlation between $s_{u(DMT)}$ and $s_{u(reference)}$ of course depends

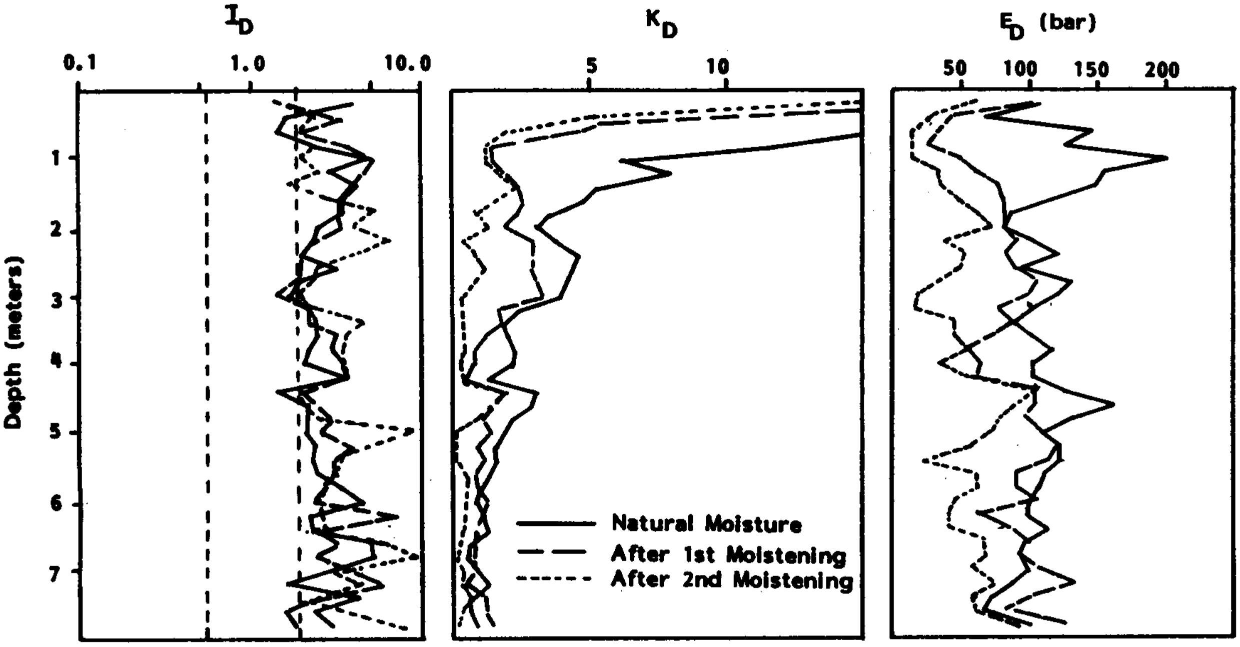

Figure 3. Insensitivity of I_D to Changes in Saturation in a Silt

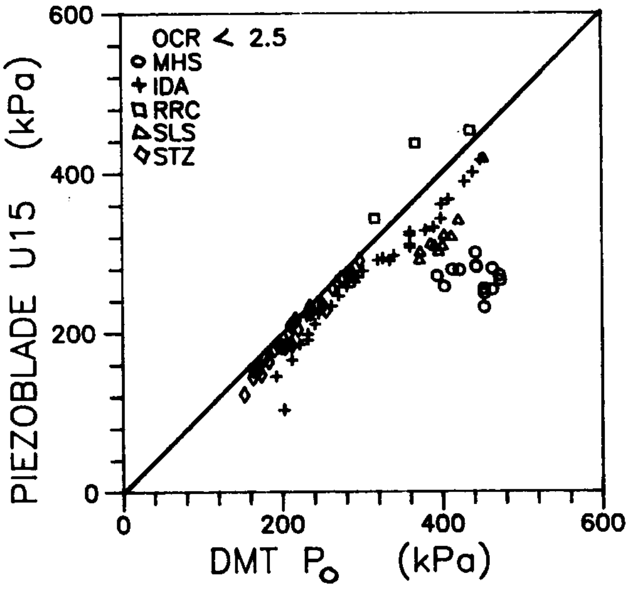

Figure 4. Comparison Between DMT p_o and Penetration Pore Pressures from Piezoblade in Normally Consolidated and Lightly Overconsolidated Clays

on the quality of the reference measurement. This discussion will assume that the uncorrected s_u from the field vane represents the "most likely" value of undrained shear strength. Problems associated with misuse of the vane in materials which are too stiff or do not remain undrained during testing or problems of soil variability will be neglected. Additionally, it should be left up to the engineer's discretion whether or not to apply a vane correction depending on the design problem.

As noted, the current estimate of s_u indirectly makes use of the p_o pressure reading, which is normally obtained within 15 to 30 sec. after penetration. Several studies (Davidson and Boghrat, 1983; Campanella et al., 1985; Lutenegger and Kabir, 1988) have shown that in soft saturated clays (OCR<2) the DMT is essentially an undrained test and p_o is dominated by penetration pore water pressures. Figure 4 shows a comparison between p_o and u_{excess} measured with a DMT Piezoblade 15 sec. after penetration. It can be seen that in these materials the p_o reading primarily reflects penetration pore water pressures.

Cavity expansion theory (Vesic, 1972; Ladanyi, 1963) and strain path analysis (Baligh, 1985) for cylindrical penetrometers accurately predict undrained shear strength from penetration pore water pressures for normally consolidated and lightly overconsolidated (non-dilating) clays. Therefore, it should not be too surprising that the DMT is able to accurately predict s_u in these materials provided that substantial pore water pressure dissipation does not occur in the time required to obtain p_o. Mayne (1987) has shown that despite the differences in geometry, the DMT

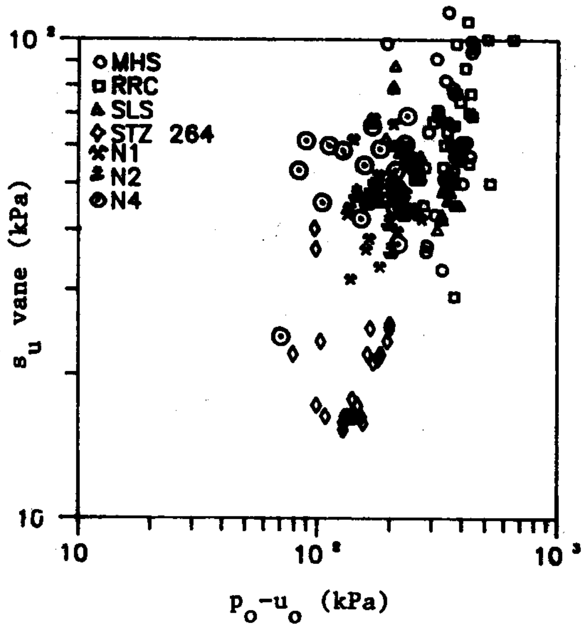

Figure 5. Comparison Between s_u (uncorrected field vane) and Corrected DMT p_o

p_o is nearly identical to pore water pressures obtained behind the base of a piezocone. In the case of a cylindrical piezocone Robertson and Campanella (1983) suggest that, based on cavity expansion theory the undrained shear strength for normally consolidated clays can be estimated as:

$$3 < \frac{\Delta u}{s_u} < 5 \qquad [10]$$

for a cylindrical cavity with Δu = $(u_{measured}-u_{equilibrium})$ measured behind the cone. Since the penetration of the DMT represents some form of cavity expansion and p_o in soft saturated clays is predominantly excess pore water presures, one might expect a relationship similar to [10] to exist for the DMT.

A preliminary check on this approach is shown in Fig. 5, which presents data obtained in normally consolidated clays by the writer. It appears that this approach indeed has some merit with the values generally ranging from:

$$3 < \frac{p_o-u_o}{s_u} < 9 \qquad [11]$$

The range in values no doubt reflects differences in rigidity index and other factors relating to cavity expansion.

The accuracy of the prediction of s_u appears to be strongly linked to proper soil identification through the use of the Material Index, I_D. This is illustrated in Fig. 6 which compares the measured error in s_u between DMT and field vane and I_D for a number of clay sites investigated by the writer. As the material index increases, it appears that the accuracy in predicting s_u is reduced. This no doubt reflects the fact that with increasing I_D more pore water pressure dissipates as the test in conducted. If one takes 20% to be the acceptable error, Fig. 6 suggests that s_u estimates should be restricted to I_D values less than about 0.33. This value roughly coincides with the limit shown by Davidson and Boghrat (1983) for approximately 5% penetration pore pressure dissipation after 1 min. One encouraging point noted in Fig. 6 is that the DMT consistently underpredicts s_u.

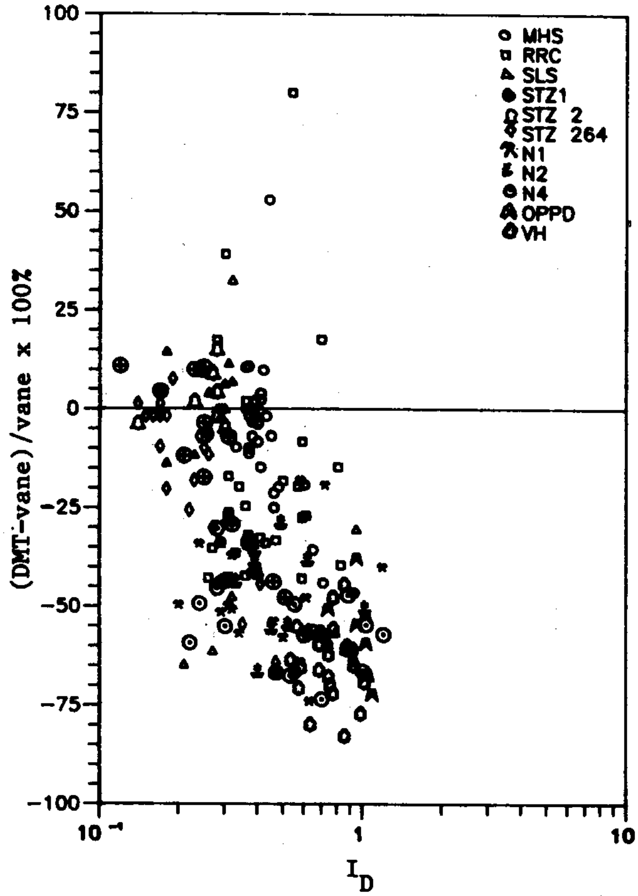

Figure 6. Accuracy in Estimate of DMT s_u (as proposed by Marchetti, 1980) as a Function of I_D

The data shown in Fig. 6 suggest that for various values of I_D, a simple correction may be applied to the current DMT empirical correlation to bring the estimate of s_u to within acceptable limits. However this correction should be used with caution considering the degree of scatter indicated in Fig. 6.

Results obtained in stiff overconsolidated clays (e.g., Powell and Uglow, 1986) seem to indicate that the current relationship for estimating s_u from the DMT is not as accurate, sometimes overpredicting and sometimes underpredicting s_u. However, the comparisons have not been with field vane, but with other field tests (e.g., PMT, Plate Load) or laboratory tests (e.g., UU triaxial).

Powell and Uglow suggested that in highly overconsolidated clays and tills the normalized undrained strength could be expressed as a function of K_D. This is essentially the same as comparing s_u directly to p_o as previously shown in Fig. 5 since both are normalized with respect to σ'_{vo}. Figure 7 shows pore pressure measurements obtained from Piezoblade and DMT p_o values in overconsolidated (2.5 < OCR < 10) soils. In contrast to results obtained in more normally consolidated clays and shown in Fig. 4, data from overconsolidated clays clearly show that p_o is predominantly greater than u_{excess}. Therefore, in overconsolidated clays there is a component of p_o which is attributed to soil resistance, which may be regarded as the "overstress" created by penetration of the blade. In light of these data, it is not surprising that current empirical correlations do not predict s_u as accurately as in normally consolidated materials.

3.3 Stress History

The estimation of soil stress history using the DMT was proposed by Marchetti (1980) by correlating K_D with OCR from oedometer tests. Marchetti restricted the use of the correlation to "materials free of cementation, attraction etc. in simple unloading" and for I_D < 1.2. Unfortunately, many natural soils do not fit this description for one reason or another and therefore attempts have been made to apply the correlation outside the original scope. This is only natural, since engineers deal with a multitude of materials and wish to extend the use of any tool to its practical limits.

Recently, Mayne (1987) has reviewed the existing literature and proposed a simple relationship between the "effective" DMT p_o (p_o-u_o) and vertical preconsolidation stress obtained from stress controlled incremental oedometer tests, p'_c. These data are shown in Fig. 8 and include an additional 32 data points obtained by the writer in a variety of materials. The simple relationship between p'_c and p_o indicated in Fig. 8 is similar to that presented by Marchetti (1980) since both OCR and K_D are normalized with respect to the effective overburden stress.

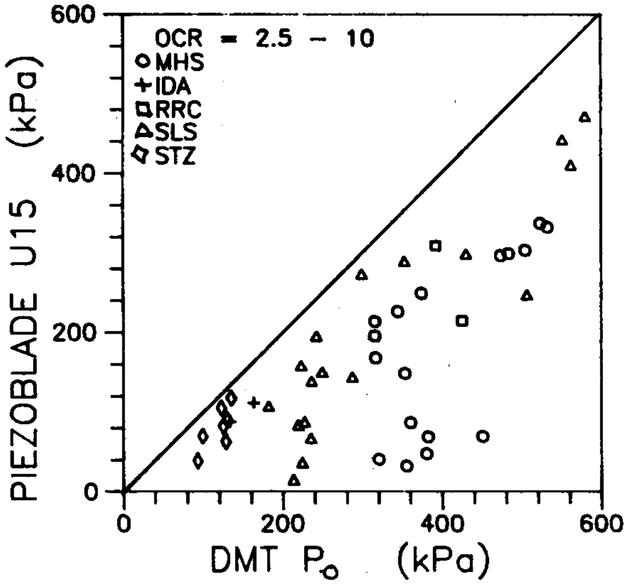

Figure 7. Comparison Between DMT p_o and Penetration Pore Pressures from Piezoblade in Overconsolidated Clays

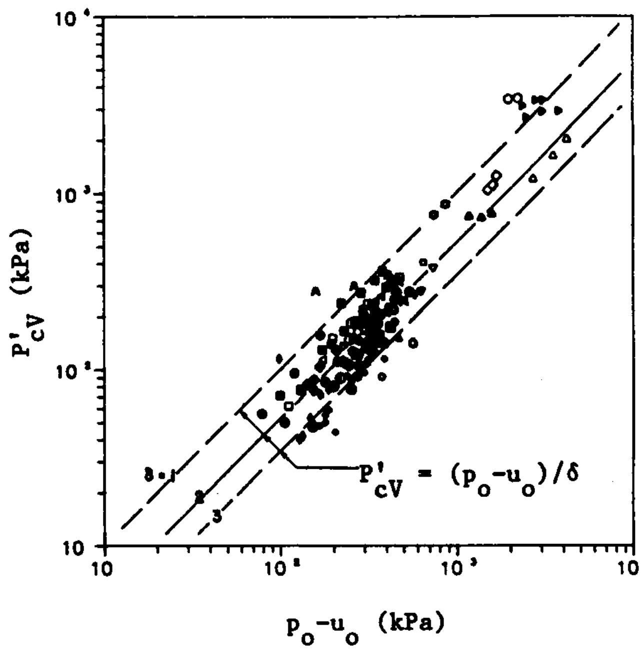

Figure 8. Relationship Between Corrected p_o and Vertical Preconsolidation Stress from Oedometer (modified from Mayne, 1987)

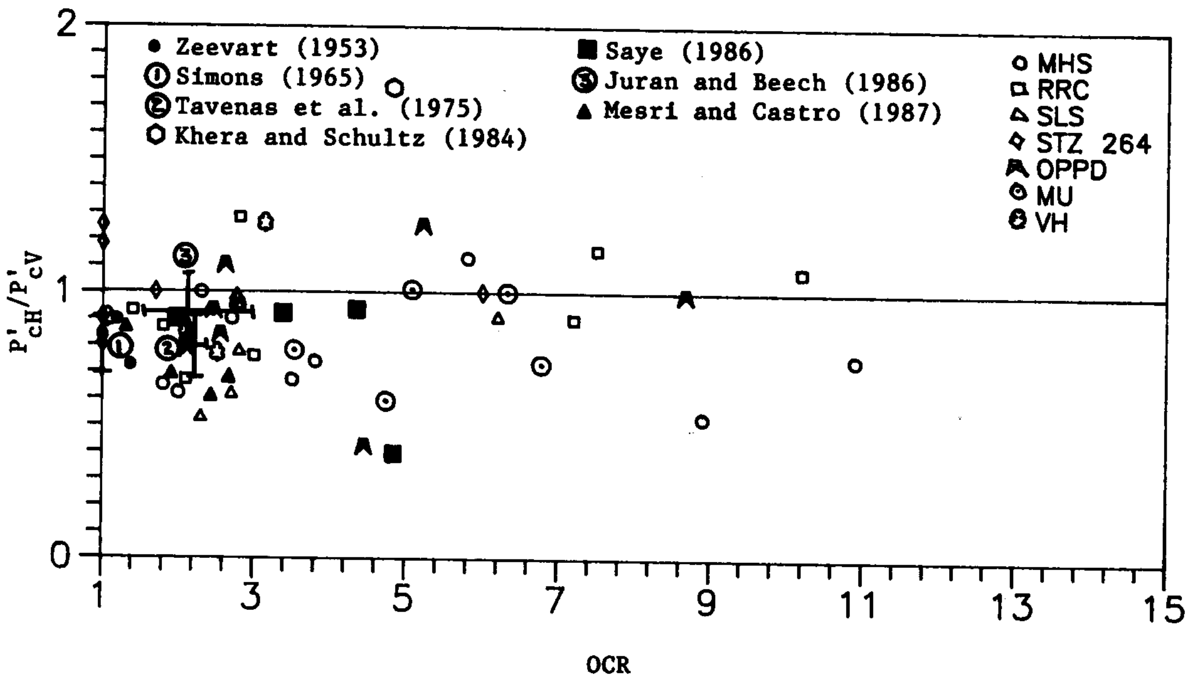

Figure 9. Variation in p'_{cH}/p'_{cV} with OCR

An alternative approach to determining stress history has been proposed utilizing the p_2 reading (Lutenegger and Kabir, 1988), in which the OCR is directly related to the parameter U_D. This approach is attractive since it makes use of the approximate value of the penetration pore pressures via p_2 as will be discussed.

In normally consolidated clays in which $p'_c = \sigma'_{vo}$ it should not be surprising that $p_o - u_o$ accurately predicts p'_c since p_o is dominated by penetration pore water pressures. Thus if p_o is accurate in predicting s_u; and s_u/σ'_{vo} is approximately constant, a test which accurately predicts one will automatically predict the other. However, the data of Fig. 8 suggest that p_o provides reasonable values of p'_c over a wide range of OCR.

One of the most often asked questions in critique of the DMT is "how can a test which presumably measures soil response in the lateral direction predict vertical soil properties"? The answer to this question lies partially in the fact that in softer more normally consolidated soils, the p_o reading is dominated by pore water pressures, as previously discussed. In these materials, if s_u/p'_c is approximately constant, then the p_o value reflects both s_u and p'_c. However, it may also be that a large percentage of naturally occurring sediments are not highly anisotropic with respect to oedometric yield stress and compressibility, both of which are controlled in large part by soil fabric.

During the course of research work connected with the DMT and other projects, the writer has conducted a number of oedometer tests on undisturbed Shelby tube and piston samples with samples oriented in the horizontal direction, such that the preconsolidation stress normal to the ground surface and therefore parallel to the DMT may be determined. A comparison between the ratio of horizontal to vertical preconsolidation stress and vertical OCR for all tests is shown in Fig. 9. Also shown are available tests from the literature.

These results indicate (quite unexpectedly to the writer) that there is no significant trend in p'_{cH}/p'_{cV} with changing OCR. This is probably due to the fact that the majority of the samples represent geologic materials which <u>have not</u> been physically preloaded, and thus the apparent preconsolidation pressure obtained from oedometer tests is a result of some other phenomenon, such as, freeze-thaw, shrink-swell, fluctuations in water level, etc. There is not sufficient evidence to indicate that such materials should be anisotropic with respect to yield stress in K_o loading. Some of the scatter indicated in Fig. 9 may also derive from sample disturbance and other problems associated with determining p'_c, e.g., graphical

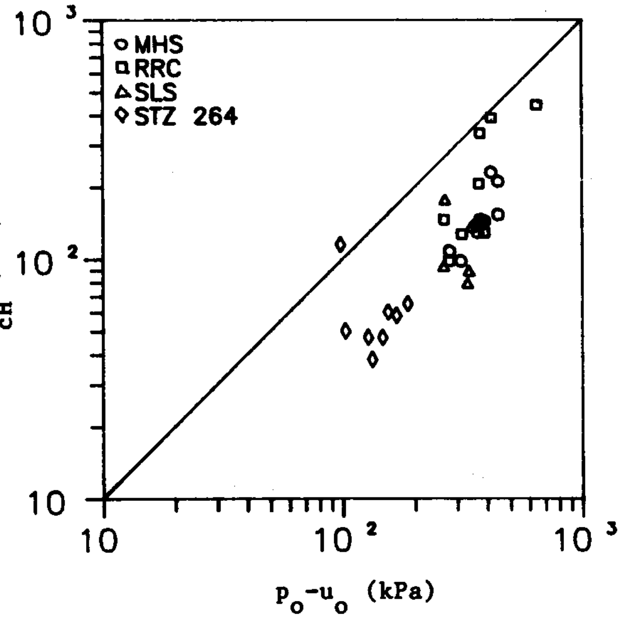

Figure 10. Comparison Between p_o and p'_{cH} from Oedometer Tests.

interpretation.

In light of these data, the relationship between p_o and p'_c presented in Fig. 8 may be reasonable, irrespective of OCR. In order to further investigate the response of p_o to horizontal yield stress, Fig. 10 presents a comparison between corrected p_o and p'_{cH} for tests conducted by the writer. These data fall within the correlation range suggested by Mayne (1987) for vertical p'_c as previously shown in Fig. 8.

3.4 Lateral Stress Ratio, K_o

Because of its geometry, the DMT largely records the horizontal response to penetration when placed vertically. Thus, a measure of horizontal total stress is obtained, which Marchetti used to define a horizontal stress index K_D, Eq. 4. In recent years, engineers have become increasingly aware of the influence that horizontal ground stresses have on engineering behavior. Schmertmann (1985) recently summarized a number of engineering problems wherein the lateral stress invokes a significant influence, Table 4.

The value of K_D was directly related to K_o by Marchetti (1980) primarily using the empirical relation to OCR presented by Brooker and Ireland (1965). The initial correlation appeared to be independent of soil type (excluding sands) and stress history and therefore has been used by a number of investigators.

Table 4. Qualitative Effects of Changing K on Engineering Behavior (Schmertmann, 1985). (arrows show direction of usual less conservative behavior)

Engineering Behavior	In Situ: Low K — High K
Bearing Capacity	safety decreases
Slope Stability	safety decreases
Fracture of Dams	safety decreases
Pressure on Walls	increases
Pile Friction	decreases
Settlement/Deformation	increases
Liquefaction	safety decreases
Ground Treatment Improvement	more difficult

One of the difficulties in establishing a direct relationship between K_D and K_o is that a reference value of K_o is difficult to obtain. Unlike other soil parameters such as undrained strength or compressibility, which may be reasonably determined by acceptable methods, there is no specific technique which is agreed upon by the profession as the preferred method for determining K_o. Several recent investigations have made use of other field or laboratory tests to compare the K_o value obtained using the DMT: e.g., push-in spade cells (Chan and Morgenstern, 1986); K_o-Stepped Blade (Lutenegger and Timian, 1986); prebored pressuremeter (Powell and Uglow, 1986); hydraulic fracturing and laboratory correlations (Lacasse and Lunne, 1983); and self-boring pressuremeter (Clough and Goeke, 1986).

These and other studies indicate that K_o values derived from the DMT K_D correlation are nearly all within a factor of about 1.5 for a wide range of materials. Many geotechnical engineers are now comfortable with horizontal stress measurements obtained with the self-boring pressuremeter as a reference test. The writer suspects that within the next few years, a number of studies will be available to combine SBPMT and DMT data to help refine the K_D prediction of K_o.

Based on calibration chamber tests in sands, Baldi et al. (1986) have suggested that K_o be determined using both DMT K_D values and adjacent CPT penetration data. The proposed equation for predicting K_o has the form:

$$K_o = 0.376+0.095K_D-0.00172\ q_c/\sigma'_{vo} \qquad [12]$$

Of course the drawback is in having to assess the CPT value q_c.

One area in which the DMT appears particularly useful is in the assessment of changes in horizontal stress which have occurred as a result of induced loading, excavation, landslides, or other changes. In this case, the DMT offers the ability to rapidly and inexpensively obtain data to compare the relative changes in horizontal stress, e.g., as a result of preloading under known stress history (Saye and Lutenegger, 1988a).

3.5 Deformation Characteristics

3.5.1 Constrained Modulus

Expansion of the DMT diaphragm from p_o to p_1 produces a known displacement which was used by Marchetti to define the Dilatometer Modulus, E_D, as given in Eq. [5]. Marchetti (1975) had previously suggested that the DMT expansion could be used to define a lateral subgrade reaction modulus value, however engineers are more often in need of a deformation parameter for settlement estimates. Marchetti thus proposed a correlation between E_D, K_D, and M, the local oedometric constrained modulus at the effective in situ overburden stress, σ'_{vo} . More specifically, the correlation is for the local reload modulus. Therefore, in normally consolidated soil, the DMT should not be expected to provide information about the reload modulus and in overconsolidated soil the DMT will provide no direct measurement of the virgin compression modulus.

Schmertmann (1981) had shown that the values of M estimated by the DMT were generally within a factor of about 2 when compared with laboratory oedometer tests. Other studies have generally indicated about the same accuracy in both clays and sands (e.g., Lacasse and Lunne, 1982; Lutenegger and Timian, 1986; Ming Fang, 1986; Lacasse and Lunne, 1986; Borden et al., 1986). Baldi et al. (1986) note that, in the case of calibration chamber tests on overconsolidated sands, the Marchetti correlation results in a pronounced underestimate of M.

There is a distinct advantage in estimating M, which is particularly appealing to practitioners, since the Janbu (1963, 1967) technique for estimating settlement may be used. The relative accuracy of the estimate of M was clearly demonstrated by Schmertmann (1986a) who compared DMT settlement estimates with measured settlements over a wide range in magnitude.

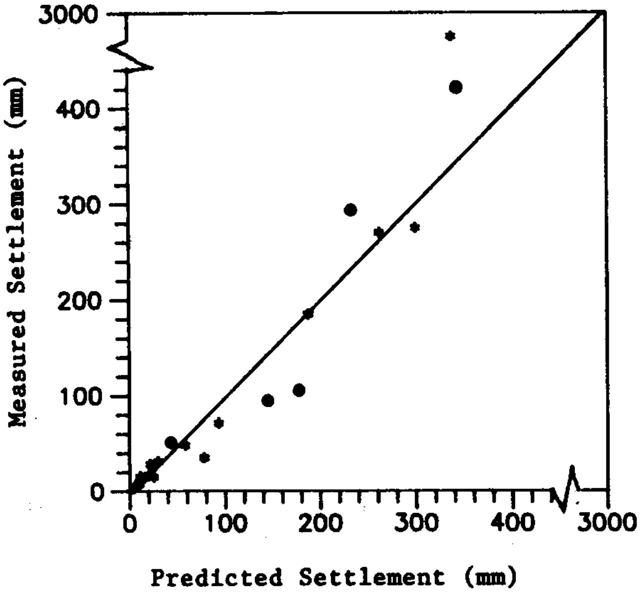

Figure 11. Comparison Between Predicted and Measured Settlements Using M and DMT. (modified from Schmertmann, (1986b)

The real excitement in the data is that if one is interested in a rapid estimate of settlement for preliminary design the DMT appears particularly accurate! The writer has replotted settlement values obtained by Schmertmann (1986b) and included six additional case histories which have been independently investigated using this procedure, Fig. 11.

It is obvious to the writer that constrained modulus values for comparison with the DMT should be obtained from back calculated settlement records on projects wherein an exact determination of footing or other loading stress may be made. It should also be kept in mind that the accuracy of predictions with reppect to field performance is also dependent upon the variability associated with the use of the mathematical model involved. This applies to any test.

3.5.2 Elastic Modulus

In many design situations, the engineer may need an estimate of elastic modulus, E, e.g., for use in drilled shaft design or immediate settlement estimates. Depending on the design problem, a different value of E may be required, i.e., E_1, E_{25}, etc.

Davidson and Boghrat (1983) suggested that in highly overconsolidated clays, the value of E_1, obtained from unconsolidated undrained triaxial compression tests could

Table 5. Suggested Correction Factor F For Use in Eq. [13].

Soil Type	Modulus	F	Ref.
Cohesive soils	E_i	10	Robertson et al. (1988)
Sand	E_i	2	Robertson et al. (1988)
Sand	E_{25}	1	Campanella et al. (1985)
NC Sand	E_{25}	0.85	Baldi et al. (1986)
OC Sand	E_{25}	3.5	Baldi et al. (1986)

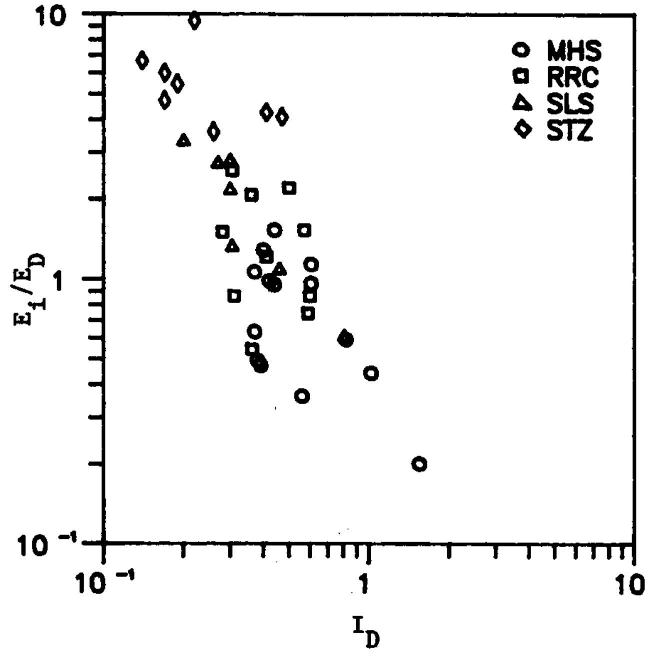

Figure 12. E_i/E_D in Cohesive Soils as Related to I_D (E_i from UU triaxial compression)

be related directly to the DMT E_D using a factor of about 1.4. Recently, Robertson et al. (1988) have suggested factors of 10 and 2 for clays and sands, respectively, for use in laterally loaded pile design. All of these results suggest a simple correction factor of the form:

$$E = F\,E_D \qquad [13]$$

Table 5 presents suggested factors of F for different materials.

Baldi et al. (1986) presented results of calibration chamber tests on pluviated sands over a wide range of relative densities and compared E_D to E_{25} obtained from CK_oD triaxial compression tests for both the normally consolidated and over-consolidated condition. Their results follow the form of Eq. [13] and are included in Table 5. For normally consolidated sands, a value of F=1 was also suggested by Campanella, et al. (1985) to estimate E_{25}.

Borden et al. (1985) suggested a relationship between the initial tangent modulus from unconfined compression tests on partially saturated compacted A-6 soil and E_D which had the form:

$$E_i = 0.142\,E_D^{1.298} \qquad [14]$$

The writer reanalyzed these data to fit into the form of Eq. [13] and found that the value of F ranged from about 0.4 to 1.1.

The writer was curious about the variation in factor F for estimating the initial tangent modulus. If I_D relates to soil stiffness, then it should be suspected that the value of F is related to I_D. Figure 12 presents a comparison of E_i/E_D as a function of I_D. E_i values were obtained from UU triaxial compression tests at confining stresses equal to total horizontal stress estimated from the DMT. The trend of decreasing F with increasing I_D is in agreement with Table 5 and again indicates the importance of I_D.

3.6 Drained Friction Angle in Cohesionless Soils

The penetration of the DMT blade in sands and other freely draining soils represents a drained bearing capacity failure approximating a plane-strain condition. Since the failure condition is controlled by the frictional strength component and in situ state of stress in granular materials, it is reasonable to expect that the results from the DMT may be used to determine ϕ'.

A procedure for obtaining the effective axi-symmetric friction angle in sands was presented by Schmertmann (1982) using the wedge penetration theory of Durgunoglu and Mitchell (1975) and is attractive since it incorporates the horizontal effective stress which is estimated during the DMT. This technique requires measurement of the pushing thrust to advance the DMT, such that an estimate of the tip resistance may be obtained. This measurement may be made using an appropriate load cell which ideally should be located immediately behind the blade.

An alternative approach was recently proposed by Marchetti (1985), but this procedure requires parallel electric CPT data. The technique essentially makes use of the stress ratio estimated from the DMT and the tip resistance requires more effort and is subject to discrepancy associated with reproducing adjacent test results.

Comparisons between calculated values of ϕ' and reference values are relatively scarce in the literature. Using the Schmertmann (1982) technique, Clough and Goeke (1986) obtained values within an accuracy of about 15% for gravelly sand compared with laboratory triaxial compression tests.

4 PREDICTING FIELD PERFORMANCE

The real value of any soil test is in its ability to accurately predict field performance. In addition to obvious uses as a site profiling tool and in obtaining predictions of conventional soil properties, several more direct applications of the DMT to specific engineering problems have been reported. Table 6 presents a current summary. The list is no doubt larger since the application of the DMT is rapidly expanding and new uses are continually being investigated.

Most of the current applications of the DMT to real engineering design problems make use of conventional soil parameters predicted by the DMT. Therefore the accuracy of the predictions generally indicate the ability of the DMT to accurately predict properties.

In the case of quality control wherein the DMT has been used to investigate <u>changes</u> in parameters, the DMT may find a new use in investigating the effect that changing in situ lateral stress has on other in situ tests, e.g., SPT, CPT, VST, etc.

The success of the DMT to accurately predict performance has generally been linked to a design approach based on an accepted engineering practice. Thus, at the current time, conventional design procedures using DMT derived conventional soil engineering properties are generally being used. This is in contrast to a hybrid design approach which is often used with other in situ tests; for example, the pressuremeter approach to foundation design based on E_m or P_L. The writer considers this a distinct advantage of the DMT and an attribute which should make the test more appealing to practicing engineers.

Table 6. Current Reported Application of DMT for Design.

Application	Reference
Settlement Prediction	Schmertmann(1986b) Hayes (1986) Saye and Lutenegger(1988b)
Laterally Loaded Driven Pile Design	Schmertmann and Crapps (1983) Robertson et al. (1988)
Skin Friction of Axial Loaded Piles	Marchetti et al. (1986)
Load on Buried Pipe	Schmertmann
Liquefaction Potential of Sands	Marchetti (1982) Robertson and Campanella(1986)
Compaction Control/ Verification	Schmertmann (1982) Schmertmann et al. (1986) Lutenegger (1986) Lacasse and Lunne (1986)
Ultimate Uplift of Anchor Foundations	Lutenegger et al. (1988)
Transmission Tower Foundation Design	Bechai et al. (1986)
End Bearing, Side Friction and Settlement of Drilled Shafts	Schmertmann and Crapps (1983)
Earth Pressures on Existing Retaining Walls in Distress	Schmertmann (pers. comm. 1987)

5 RECENT DEVELOPMENTS

5.1 Penetration Pore Pressure

Davidson and Boghrat (1983) had clearly shown that in some cases large penetration pore water pressures could be generated when a flat plate with identical geometry to the DMT was forced into saturated soils. Campanella et al. (1985) demonstrated that the corrected DMT closure pressure, p_2, obtained by deflation after obtaining the p_1 pressure could be used to estimate such

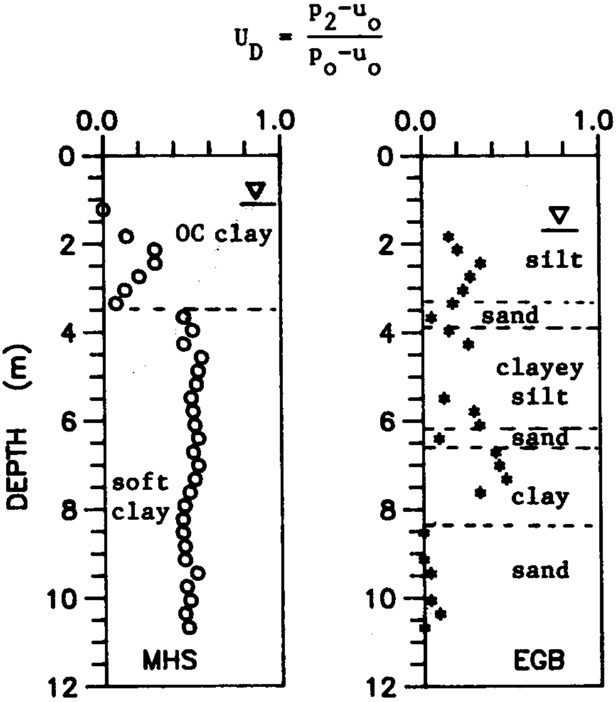

Figure 13. Use of U_D for Site Stratigraphy

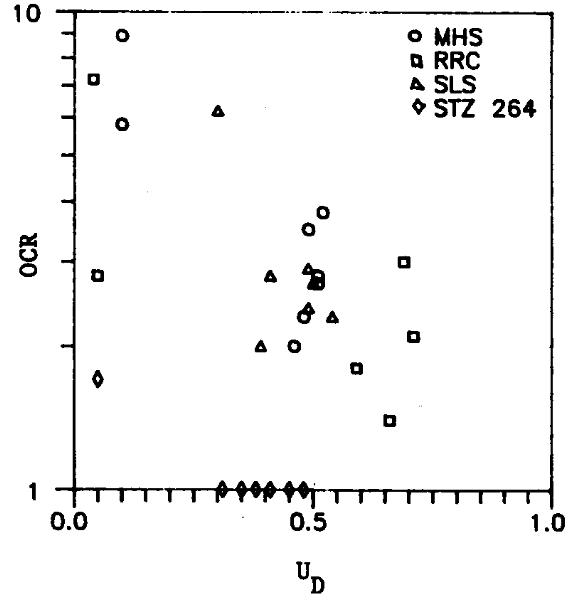

Figure 14. Variation in U_D with OCR

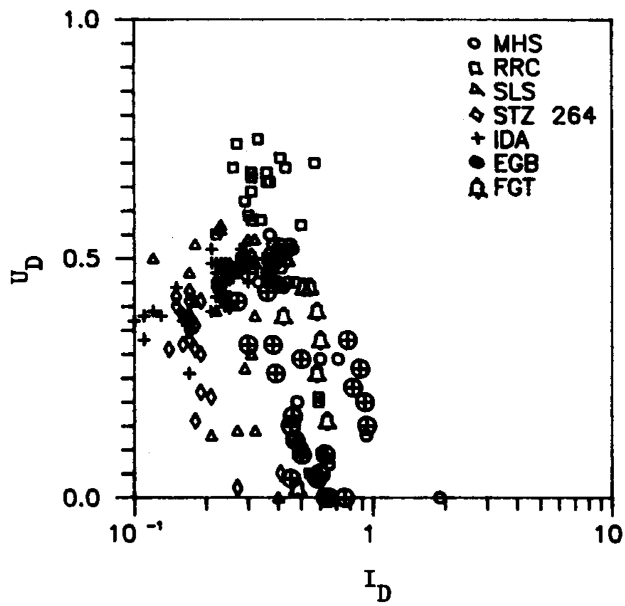

Figure 15. Relationship Between U_D and I_D

pore water pressures. In sands, where the test measures predominantly drained behavior, the p_2 reading reflects the initial in situ water pressure u_o. In saturated clays, in which little drainage occurs, the pressure p_2 reflects both initial and excess pore water pressure generated during penetration. The generated pore water pressures may be as high as seven times the initial pore water pressure in soft clays (Lutenegger and Kabir, 1987).

The parameter U_D, defined by Eq. [7] has shown to be useful in determining site stratigraphy, as shown in Fig. 13. Variations in U_D reflect drainage conditions and, for a given soil, the tendency for generating positive pore pressures, which one can expect to vary with the stress history of the soil. Therefore, one should logically expect a relationship between U_D and OCR. Available data from oedometer tests for a number of sites are shown in Fig. 14. Additionally, if both U_D and I_D are an indication of soil type, one should expect a strong relation between these two parameters. Combined data from several sites are shown in Fig. 15 and verify this.

It is clear to the writer that the p_2 pressure and U_D provide additional insight into the drainage conditions and pore pressure surrounding the DMT. At this time, the full implications of p_2 are not known, however, the writer recommends that the deflation C-Reading should be obtained as a routine part of every DMT.

5.2 Pore Pressure Dissipation

The dissipation record of excess pore water pressures following penetration of a cylindrical probe, i.e., piezocone, into saturated soils may be used to estimate the horizontal coefficient of consolidation, c_h (Torstenssen, 1975; Baligh and Levadeaux, 1986, Gupta and Davidson, 1986). Similarly, the rate of pore water pressure dissipation from the face of a flat-plate may also provide an estimate of c_h. As previously discussed, it appears that the DMT closure pressure p_2 closely approximates initial plus excess generated pore water pressures; therefore one might suspect that a timed sequence of p_2 measurements may provide the necessary information to estimate the time-rate of pore water pressure dissipation.

Marchetti et al. (1986) have shown that a time-dependent decay in the p_o pressure may be used to estimate the reconsolidated horizontal effective stress after penetration, which would be useful for effective stress analysis for vertical driven pile design. However, this decay is a measure of the rate of total horizontal stress dissipation, which would be expected to be similar to that obtained by others using total stress spade cells (e.g., Massarsch, 1975; Tedd and Charles, 1981).

Baligh et al. (1985) have shown that for a cylindrical probe, the dissipation rates of excess pore water pressure and horizontal total stress acting at the face of the probe are not the same and therefore one would not be able to estimate c_h from a total stress dissipation record. A comparison between the change in p_o and the pore pressure dissipation from the face of a DMT blade instrumented to measure pore pressures give similar results, as shown in Fig. 16. It is important to note that the DMT dissipation test was conducted without performing an expansion test; i.e., after the first and all subsequent p_o was obtained, the pressure was released so that no further expansion of the diaphragm would occur.

obv. different curves are to be used

If a full DMT expansion cycle is obtained, and then timed measurements of the p_2 closing pressure are recorded, the pressure dissipation curve more closely matches the Piezoblade dissipation dissipation curve, Fig. 17. In this case it is of interest to note that the reexpansion p_o values are nearly identical to the p_2 closure pressures. This may indicate that the p_1 expansion to 1 mm has created a cavity between the blade and the soil.

At the present time, no theoretical solution is available to calculate the time-rate of consolidation using pore water pressure dissipation rates from flat plates. However, based on the theoretical time factors for a cylindrical probe, Robertson et al. (1988) have proposed a technique to estimate c_h from the DMT.

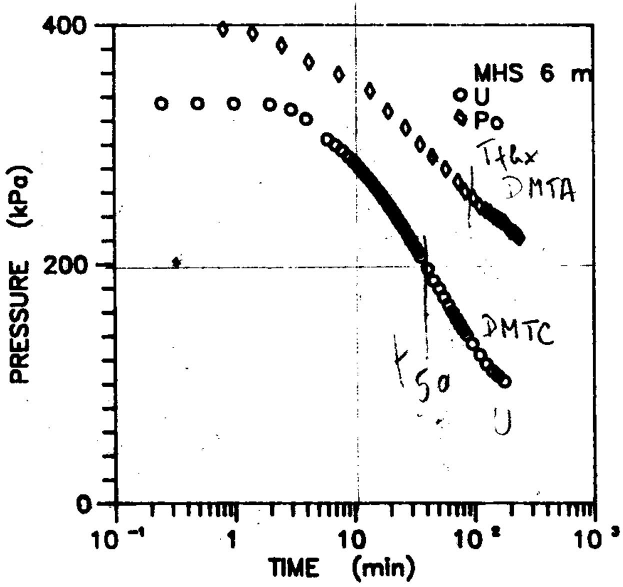

Figure 16. Dissipation of p_o and u (MHS 6m)

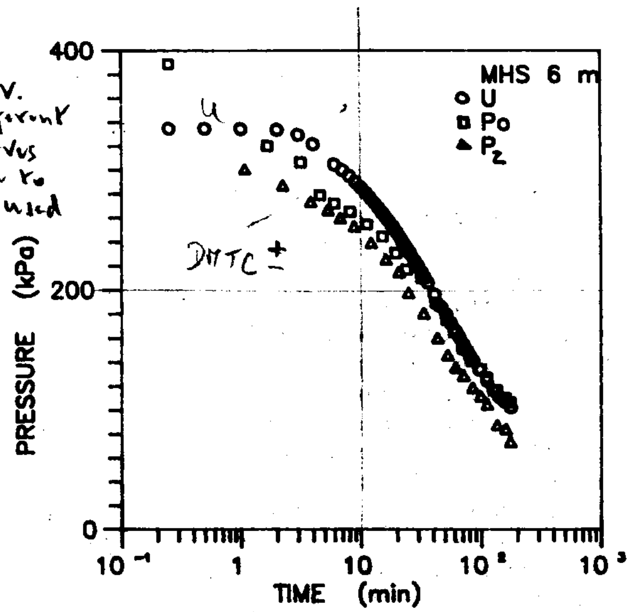

Figure 17. Dissipation of p_o, p_2, and u (MHS 6m)

5.3 Comparison with Other In Situ Tests-PMT

Campanella et. al (1985) noted that the DMT penetration might be sufficient to produce a limit pressure in soft clays, keeping in line with cavity expansion theory. However, because of time-rate effects, location of the membrane in relation to the tip, etc., the p_o may actually be less than the limit pressure, P_L. In a prebored pressuremeter (PMT) test engineers normally define the limit pressure at two times the initial cavity volume for practical reasons, in contrast to the infinite volume ratio which occurs in the DMT which expands from zero thickness to the thickness of the blade. This would make P_L less than p_o. Additionally, pore pressure dissipation may occur during the PMT expansion test which normally takes about 20 minutes compared with the approximately 30 seconds required to obtain p_o. This would also reduce P_L vs. p_o. As noted by Campanella et al. (1985) the DMT expansion to obtain p_1 may reestablish P_L.

Powell and Uglow (1986) compared the limit pressures obtained from pressuremeter tests (Menard, self-boring and push-in types) to the p_1 DMT pressures at three stiff clay sites in the U.K. The bulk of their data fell within the range of $p_1 = P_L$ to $p_1 = 1.4\ P_L$.

Field tests conducted by the writer using a prebored Menard PMT to obtain limit pressures in soils ranging from very soft clays to dense silts and sands indicate a similar relationship to p_1 as shown in Fig. 18. These data are generally within the ranges indicated by Powell and Uglow (1986). If one compares the DMT p_o with P_L, the data indicate a trend slightly skewed to $p_o < P_L$. Thus, as an approximation for a wide range of conditions it appears that one could use the average pressure $((p_o+p_1)/2)$ to estimate P_L.

This may be justified if we consider the following. In very soft clays, the increase from p_o to p_1 is small and consists mainly of pore water pressure, hence the low values of I_D. As stiffness of the material increases, the difference between p_o and p_1 likewise increases which indicates a component of soil stiffness, and thus I_D increases. Therefore, there is a tendency for p_1 to underpredict P_L in soft soils where limit pressures have already been reached and for p_o to overpredict P_L in stiff soils. Hence, the average value for a wide range of materials may be appropriate.

A comparison between p_o and the limit pressure obtained from a full-displacement (Pencel) pressuremeter, which creates a plane strain cylindrical cavity expansion from zero to infinity, is shown in Fig. 19. In softer materials (OCR < 2.5) the values of p_o and P_L are nearly identical, while in stiffer materials (OCR 2.5-10) P_L is significantly underpredicted by p_o. While some of the difference relates to obvious probe geometric differences, rates of testing, creep, drainage etc., these results help explain why the p_o prediction of undrained strength is accurate in

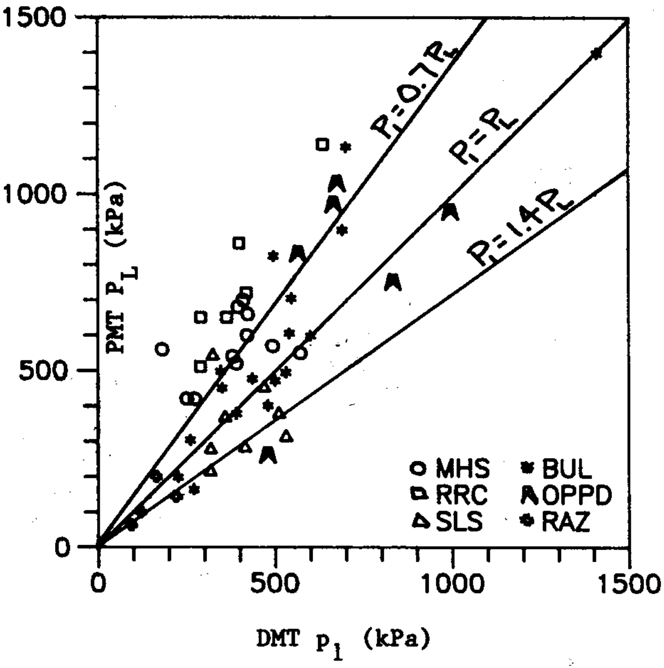

Figure 18. Comparison Between P_L (prebored PMT) and p_1

softer materials, i.e., the penetration of the blade creates a cavity expansion failure. In stiffer materials no limit pressure failure has taken place and further membrane expansion is required. The data are consistent with Figs. 4 and 7.

Elastic modulus values obtained from a prebored DMT may be compared with E_D from the DMT as shown in Fig. 20. These data indicate considerable scatter which may be in part related to variations in borehole drilling techniques used for PMT testing (e.g., augered vs. thin-walled tube sampler). However, because of the displacement created by the DMT, one should suspect the E_D is really a reload modulus and therefore it might be more appropriate to compare the PMT reload modulus with E_D. Unfortunately, these results are not available for the tests shown in Fig. 20. However, since the PMT reload modulus is higher than the initial modulus, usually by a factor of about 2 to 5, the tendency for E_D to overpredict E_M in many of the tests indicates the correct trend.

6 SUMMARY AND CONCLUSIONS

The DMT is quickly earning a place in the geotechnical profession as a cost-effective tool for conducting routine site investigations. The device has developed to a

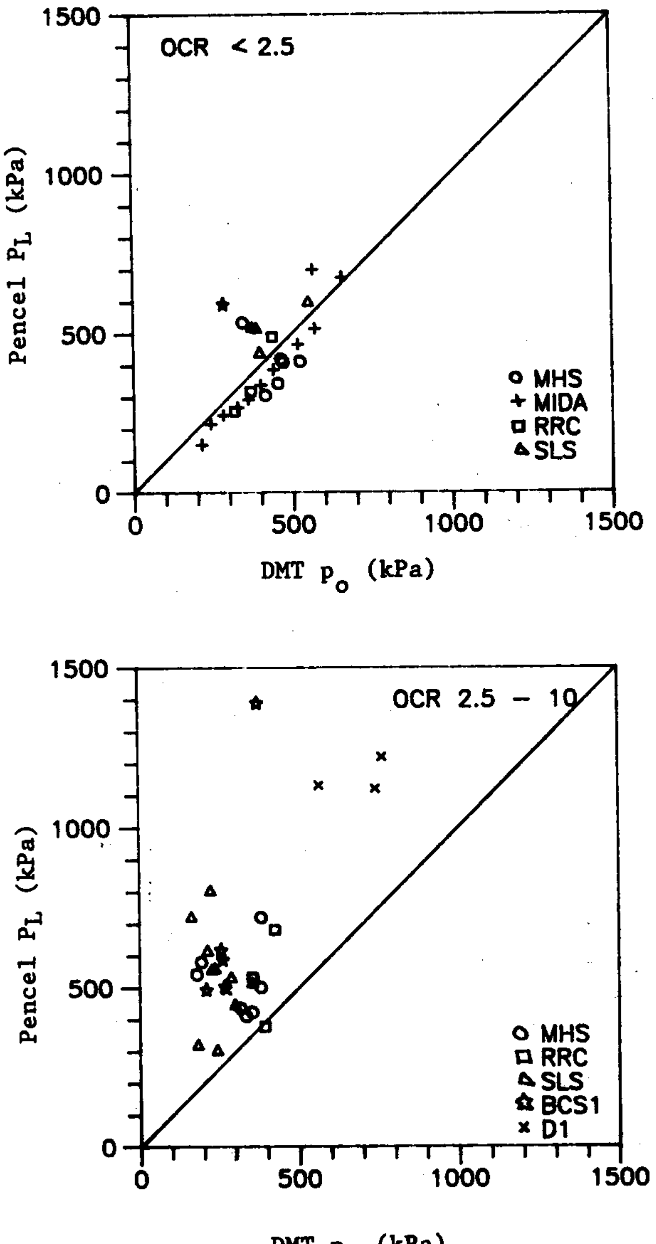

Figure 19. Comparison Between P_L(full displacement PMT) and p_o

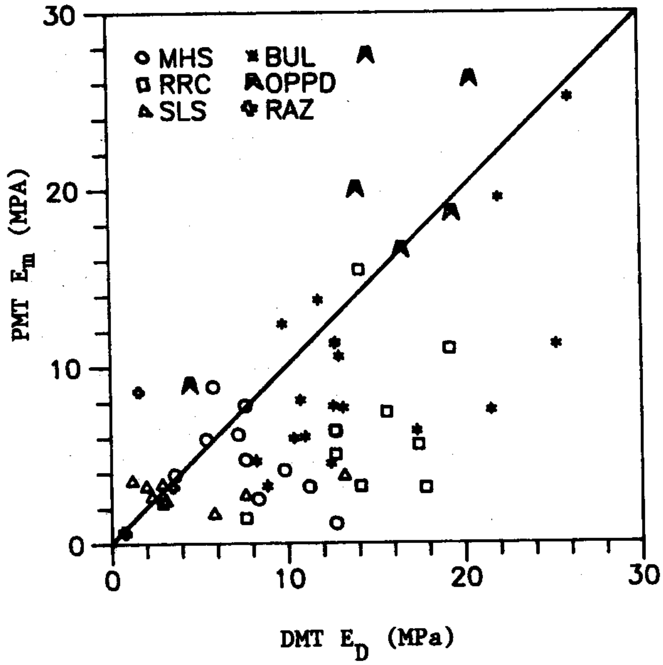

Figure 20. Comparison Between E_M(prebored PMT) and E_D

point where there now appears to be substantial theoretical justification for many of the empirical correlations proposed by Marchetti. Certainly, the use of the instrument will continue to expand into new areas with more general use. The standardization of the test by ASTM, which is currently underway, comes at a timely point and will help strengthen the development and accuracy of correlations by reducing test variability.

The writer would like to offer the following concluding remarks about the test and interpretation:

(1) The DMT is a simple and efficient test for estimating a number of soil engineering properties, and may also be used as an extremely useful logging device since it allows closely spaced vertical test points. In specific applications the test may be even more efficient if the engineer chooses to test at a few preselected depths.

(2) Use of the DMT in a wide range of earth materials has been demonstrated with nearly all materials providing reasonable test results.

(3) In soft, saturated, cohesive materials, the penetration of the blade creates a cavity expansion failure condition leading to the generation of large excess positive pore water pressures. Therefore, estimates of undrained shear strength in these materials using p_o are very accurate. In stiffer overconsolidated materials further expansion of the DMT diaphragm is required to create a failure, therefore estimates of undrained strength, based on p_o, are somewhat lower than predicted by other tests.

(4) Prediction of soil stress history relies on the p_o measurement and appears to be more accurate in normally consolidated and lightly overconsolidated materials for similar reasons given in (3) above.

(5) Estimates of other soil parameters e.g., deformation modulus, K_o and drained friction angle of sands, are generally very good for most materials.

(6) The use of DMT results for design is well established and encompasses a wide range of common geotechnical problems. Predictions for both deformation and limit equilibrium problems, which rely not only on the quality of input parameters but also on a mathematical model for a given problem are excellent.

(7) The recent addition of a controlled deflection C-Reading following the conventional A and B pressure readings allows for determination of an additional index parameter , U_D. This parameter which approximates total pore pressure around the DMT blade appears to have significant merit in determining site stratigraphy and stress history and may have other uses, e.g., determining undrained strength. Therefore, U_D may provide an alternative assessment of soil engineering properties to compare with current procedures. Because of it's apparent usefulness, the writer recommends that it become a routine measurement.

(8) It appears that by using different testing procedures, it is possible to conduct both total stress and pore pressure dissipation tests using the DMT. The results of these tests are useful for practical problems such as estimating effective stress axial pile capacity and designs involving horizontal drainage.

7 ACKNOWLEDGEMENTS

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