In Situ Testing in Overconsolidated Clays

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The usefulness of several in situ test methods in the determination of engineering properties of overconsolidated clays is described. The majority of overconsolidated clays have a stiff to very stiff consistency, which is favorable for most in situ test methods. Four popular test methods are considered: cone penetration, pressuremeter, vane shear, and Marchetti dilatometer. The test equipment for these methods is readily available, and they are likely to yield useful data if used properly. For each test method, details of the apparatus, test procedure, and interpretation of the test data are given.

Except in highly overconsolidated, very hard clays, there is little physical limitation in conducting in situ testing in overconsolidated (OC) clays. It is often possible to bore relatively deep holes in OC clays using a solid stem auger without the need for either steel casing or drilling fluid. The soil conditions are often favorable for methods such as the pre-bore pressuremeter test, where a borehole is needed. There are many in situ test methods, and most of them can be used in OC clays. A few commonly used in situ test methods that are known to be effective in characterizing OC clays are described.

Essentially all in situ tests involve complicated and often unknown boundary conditions caused by the installation of the test device. The situation is further complicated by the anisotropic and strain rate-dependent nature of clays. As a result, the test data are valid only if standard equipment and test procedures are followed. It is rarely feasible to interpret the in situ test data rigorously. Laboratory tests performed on relatively undisturbed specimens have been used to establish empirical correlations between in situ tests and soil properties. The validity of an empirical or semiempirical interpretation method should be considered as being site-specific and test-method-specific.

The in situ test methods described in this paper are cone penetration, pressuremeter, vane shear, and Marchetti dilatometer tests. One of the most common tests, the standard penetration test (SPT) is not included because the SPT blow count is known to be sensitive to hammer efficiency and to not have a consistent relationship with cohesive soil properties.

The aim of this paper is to provide brief but sufficient information to highway engineers and to assist in their selection of an in situ test method. For each method, the apparatus, test procedure, and a means of interpretation of test data are presented.

CONE PENETRATION TEST

Apparatus and Test Procedure

According to ASTM 3441 (*1*), a cone penetrometer should have a point angle of 60 degrees and a base diameter of 35.7 mm, resulting in a projected area of 10 cm^2. The friction sleeve has a surface area of 150 cm^2. The cone tip (q_c) and sleeve (f_s) resistance are measured by means of force transducers located within the cone tip. In an older design, generally referred to as the mechanical cone, the q_c and f_s readings are taken at the ground surface with a pressure gauge or a load cell. A thrust machine such as a drill rig or cone truck is used to push the cone. A penetration rate of 20 mm/sec should be maintained when obtaining resistance data.

Electric cone penetrometers may include other transducer measurements as well as, or instead of, the friction sleeve measurement. A common one is a piezometer to provide pore pressure measurements (u) during penetration. The cone penetration test (CPT) using a cone equipped with a piezometer is referred to as a piezocone test (CPTU). There is no standard for the location of the piezometer with respect to the cone tip. Unfortunately, the piezometer location can have a significant effect on the magnitude of pore pressure measurement (*2*). Hence, it is necessary to indicate the position of the piezometer in reporting the CPTU data.

Interpretation

An important advantage of a piezocone is to account for the unbalanced water forces acting on the cone tip and sleeve because of unequal end areas in cone design (see Figure 1). Correction of q_c should be carried out using the following relationship (*3,4*):

$$q_T = q_c + u(1 - a) \tag{1}$$

where

q_T = corrected total tip resistance,
u = measured pore pressure using a filter located at the shoulder point behind the cone tip, and
a = net area ratio.

The difference between q_T and q_c can be very significant, especially in a soft clay. A similar correction is required for sleeve friction data.

The major application of cone penetration has been for soil profiling. Earlier classification charts have been based on q_T and friction ratio $FR = (f_s/q_T) \times 100$ percent. In general, sandy soils have higher q_T and lower FR values, whereas clayey soils have lower q_T and higher FR values. The soil classification can be further refined using the pore pressure readings from CPTU. Earlier studies have suggested the possibility of revealing OCR using the pore pressure readings from CPTU (*5*). Figure 2 shows a classification chart according to Robertson (*6*) based on q_T, FR, and pore pressure parameter ratio B_q, defined as

$$B_q = \frac{\Delta u}{q_T - \sigma_{vo}} \tag{2}$$

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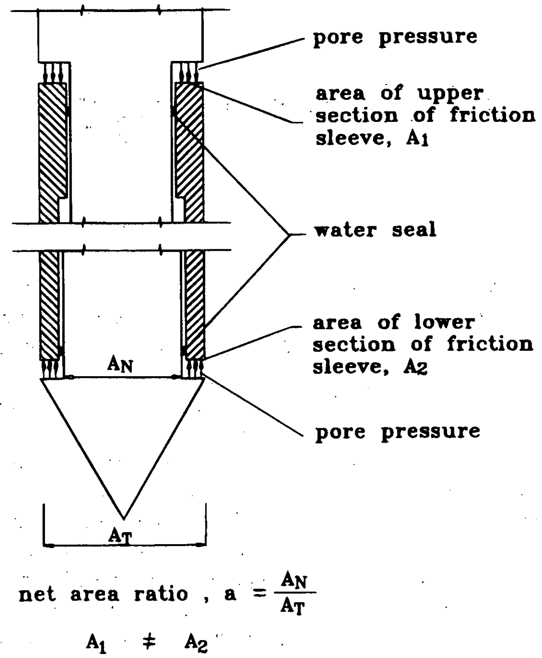

FIGURE 1 Unequal end area correction (*2*).

where Δu is the excess pore pressure ($u - u_0$), and σ_{vo} is the total overburden stress.

The classification chart by Robertson (*6*) should be used with caution as researchers (*7–9*) have indicated that the relationship between B_q and OCR may be very site-specific.

Mayne (*10*) combined the cavity expansion and modified Cam–Clay theories and proposed equations that relate CPTU data with OCR. For CPTU with pore pressure measurements at the cone tip (u_t),

$$OCR = 2\left[\frac{1}{1.95M}\left(\frac{q_T - u_t}{\sigma'_{vo}} - 1\right)\right]^{1.33} \tag{3a}$$

where

σ'_{vo} = effective overburden stress,
$M = 6 \sin \phi'/(3 - \sin \phi')$, and
ϕ' = effective friction angle.

For CPTU with pore pressure measurements behind the cone tip (u_{bt}),

$$OCR = 2\left[\frac{1}{1.95M + 1}\left(\frac{q_T - u_{bt}}{\sigma'_{vo}} - 1\right)\right]^{1.33} \tag{3b}$$

Mayne (*10*) reviewed data with OCR ranging from 1 to over 60 and ϕ' from 20 degrees to 38 degrees, and concluded that Equations 3a and 3b provide reasonable first-order estimates of in situ OCR for a variety of clay deposits.

Undrained cone penetration in clay is a very complex problem, and there is no generally accepted theory for the determination of the undrained shear strength s_u from CPT or CPTU. A common procedure in estimating s_u is to use the bearing capacity equation as follows (*11*):

$$s_u = \frac{q_T - \sigma_{vo}}{N_{KT}} \tag{4}$$

where N_{KT} is the empirical cone factor.

The value of N_{KT} unfortunately could vary between 4 and 30. Factors that may influence N_{KT} include sensitivity, stress history, stiff-

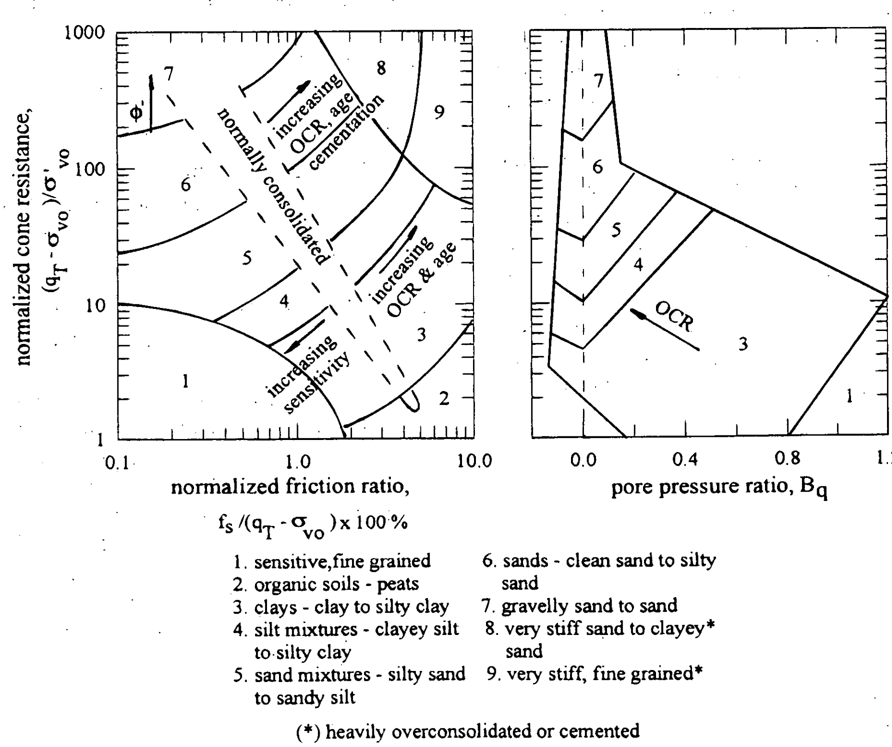

FIGURE 2 Soil behavior type chart from CPTU data (*6*).

ness, clay macrofabric, and definition of s_u. Often, a representative value of $N_{KT} = 15$ is adopted for obtaining the average s_u in intact clays (*12*). There is no general agreement on what s_u should refer to. Campanella and Robertson (2) suggest that the s_u from field vane shear test should be used as the reference s_u value and should be stated when reporting the interpretation of cone penetration data.

Methods of evaluating the clay coefficient of consolidation (c_h) from the rate of pore pressure dissipation around the cone tip have been proposed. However, these methods are suitable only for normally or lightly overconsolidated (OCR < 4) clays because of difficulties in estimating the initial pore pressure distribution around the cone in a stiff overconsolidated (OCR > 4) clay. Readers interested in the dissipation test are referred to the paper by Levadoux and Baligh (*13*).

PRESSUREMETER TEST

Apparatus and Test Procedure

The basic concept of the pressuremeter test (PMT) is to lower an inflatable cylindrical probe into a borehole and expand it to measure the pressure-deformation properties of soil. The pressuremeter as it was originally developed by Menard (*14*) consists of three independent, water-inflated chambers (tricell) stacked one above the other. The purpose of the top and bottom chambers (guard cells) is to protect the middle chamber (measuring cell) from the end effects caused by the finite length of the apparatus. All the test results are based on the measurements in the middle chamber.

A number of variants of the pressuremeter have been introduced since the late 1960s. Figure 3 shows five of the new alternatives along with the original Menard pressuremeter. The Menard, TEXAM, and OYO pressuremeters are designed to be used in prebored holes. They are referred to collectively as the prebore pressuremeter. The full displacement and stressprobe are introduced into the ground by pushing and therefore displacing soil during insertion. The self-boring pressuremeter is hollow and cylindrical in shape and has its own cutter to make the borehole and remove cuttings through the internal opening. All of the newer designs shown in Figure 3 use a single-cell probe (i.e., it has no guard cells). The single-cell pressuremeters are generally easier to operate than the tricell Menard pressuremeter. The use of non-prebore pressuremeters is not common for general geotechnical engineering exploration work and thus will not be discussed further.

A key to the success of performing a prebore PMT is the preparation of the borehole. According to ASTM D4719-87 (*1*), the borehole diameter should be within 1.03 to 1.2 times the pressuremeter probe diameter. For OC clays, the pressuremeter cavity may be prepared using an auger or a thin-wall Shelby tube sampler. For borehole preparation in highly overconsolidated, very hard clays, Lukas and Seiler (*16*) indicated that a rock bit or shaver along with drill and mud may be used. The shaver is a device that has a rock bit attached at the lower end of a cylindrical tube.

Upon borehole preparation, the PMT probe should be inserted as soon as possible. The probe expansion may be stress or strain-controlled. For the stress-controlled test, readings are taken at 30 sec and 60 sec after the pressure increments have been applied. The volume difference between the 30-sec and 60-sec readings is associated with soil creep and is referred to as the creep volume. There should be sufficient increments to yield data points that can properly define a volume-pressure curve (Figure 4).

Interpretation

The pressuremeter curve has characteristics as shown in Figure 5. Because of soil disturbance and the oversize condition of the bore-

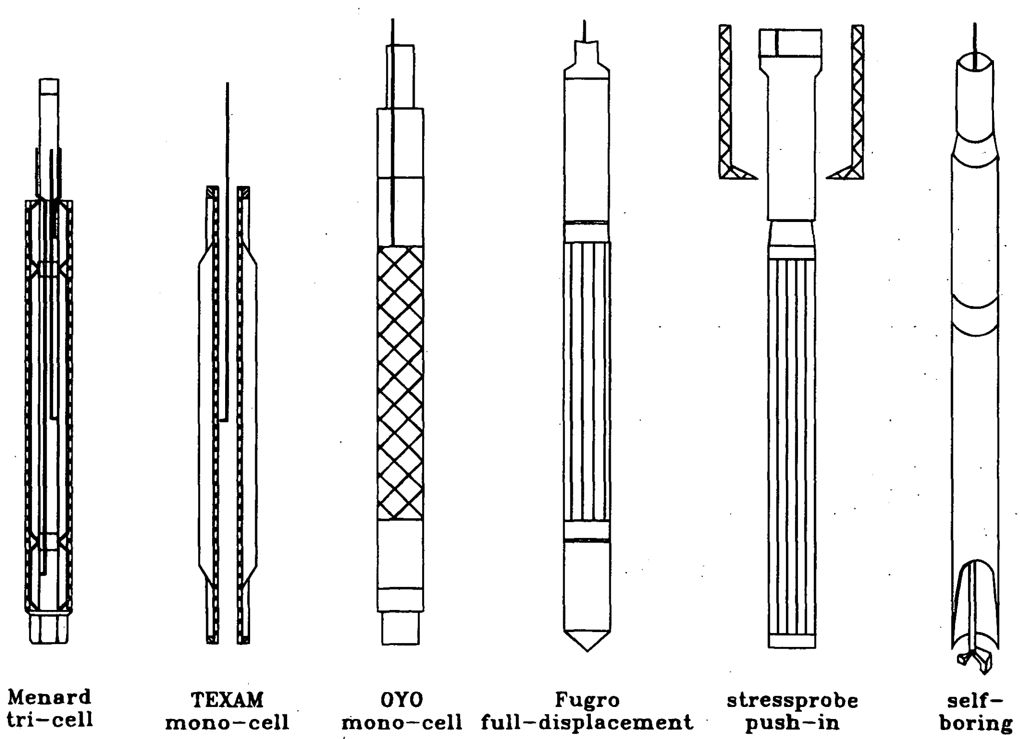

FIGURE 3 Menard and other pressuremeter probes (*17*).

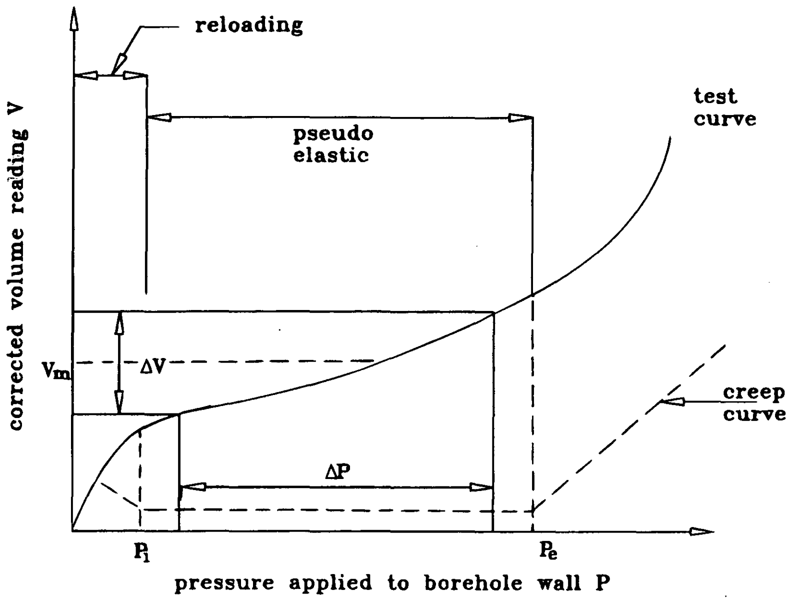

FIGURE 4 Typical pressuremeter expansion curve.

hole, some probe expansion occurs before the probe pressure reaches the lateral stress present in the soil mass ("reloading" in Figure 4). The inflection point corresponds to a point where creep volume reduces to a minimum, defined as the break point in the expansion curve where reloading ends and loading starts. The probe pressure at this inflection point (P_i) is considered by many researchers (*17–19*) to be an estimate of the in situ lateral stress.

For a PMT in OC clay, there is usually a pseudoelastic part of the pressuremeter expansion curve following the P_i point. Within the pseudoelastic zone, the creep volume remains relatively constant

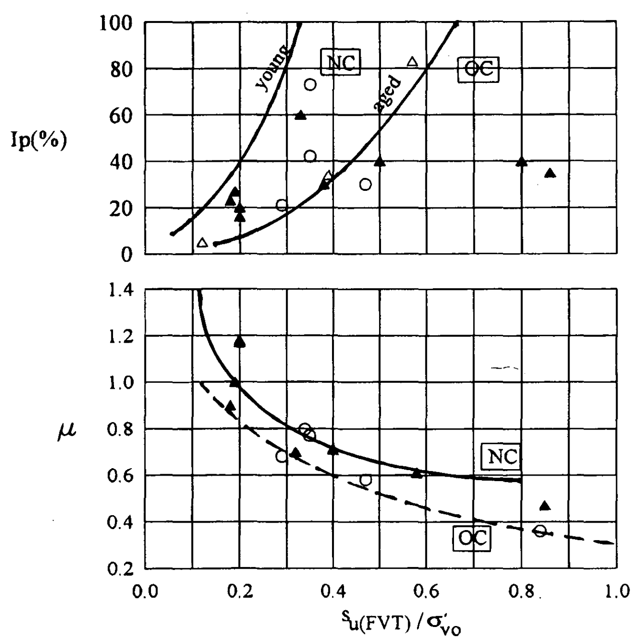

FIGURE 5 Determination of stress history and field vane correction factor (*41*).

and the expansion curve is close to a linear condition. Taking derivative within the linear part of the expansion curve, the pressuremeter modulus (E_p) is defined as

$$E_p = 2(1 + \nu)(V_o + V_m)\frac{\Delta P}{\Delta V} \tag{5}$$

where

ν = Poisson's ratio,
V_o = initial volume of the pressuremeter probe, and
V_m = volume reading in the center portion of the ΔV volume increase.

The break point in the expansion curve where creep volume starts increasing is referred to as the creep pressure (P_e). The limit pressure (P_l) is defined as the pressure where the probe volume reaches twice the original cavity volume and is usually obtained as an extrapolated value (*17*).

Baguelin et al. (*20*) presented empirical procedures that use PMT results directly in foundation designs. For axially loaded foundations the net ultimate bearing capacity q_{net} may be calculated as

$$q_{net} = k\,(P_l - P_i) \tag{6}$$

where k is the bearing capacity factor.

For OC clays, k values range from 0.8 to approximately 3.6. The foundation settlement w is related to PMT results as follows (*20*):

$$w = \frac{1.33}{3E}pR_0\left(\lambda_2\frac{R}{R_0}\right)^{\alpha} + \frac{\alpha}{4.5E}p\lambda_3 R \tag{7}$$

where

p = net bearing pressure,
R_0 = reference length equal to 30 cm,
R = radius or half-width of the foundation,
λ_2,λ_3 = shape factors (see Table 1),

TABLE 1 Shape Factors (*20*)

$L/2R$	Circle	Square	2	3	5	20
λ_2	1	1.12	1.53	1.78	2.14	2.65
λ_3	1	1.10	1.20	1.30	1.40	1.50

α = structure coefficient, equal to 1 for OC clays, and
E = essentially the harmonic mean of E_p values for soils below the foundation level.

For the analysis of a long, laterally loaded pile, Gambin (*21*) proposed that the lateral soil reaction modulus k be calculated as

$$\frac{1}{k} = \frac{1.33}{3E} R_0 \left(2.65 \frac{R}{R_0}\right)^{\alpha} + \frac{\alpha}{3E} R \tag{8}$$

Methods of establishing p-y curves for the analysis of laterally loaded piles have been proposed (*21–23*). Briaud and Cosentino (*24*) suggested the use of PMT results in pavement designs.

The PMT can be reasonably considered as a cylindrical cavity expansion. That simplification enables interpretation of the test results in a more rigorous manner. If the PMT expansion curve is shifted so that P_i corresponds to zero radial strain, a stress-strain relationship may be derived by taking derivatives of the expansion curve (*25–27*) as follows:

$$\sigma_r - \sigma_\theta = 2\epsilon_{ro} \frac{dP}{d\epsilon_{ro}} \tag{9}$$

where σ_r is the radial stress, and σ_θ is the circumferential stress.

The shear modulus G can be computed as

$$G = 2 \frac{dP}{d\epsilon_{ro}} \tag{10}$$

Other graphical or curve-fitting techniques (*28*) have also been proposed to obtain stress-strain relationships from PMT results. Because of the soil disturbance and relaxation during borehole preparation, the stress-strain relationships obtained from these rigorous procedures are not always reliable (*29*). As an alternative, it is more desirable to empirically relate soil parameters to PMT results. Consider the PMT in OC clay as an undrained test ($\nu = 0.5$), and according to Hill (*30*),

$$P_l - P_i = \beta s_u$$

$$\beta = 1 + \ln \frac{E_u}{3s_u} \tag{11}$$

where E_u is the undrained modulus.

Data collected by Holtz and Kovacs (*31*) show that E_u/s_u values range from 200 to 1,800, which would give β values of 5.2 to 7.4. Baguelin et al. (*20*) indicated that for stiff to very stiff clays, β has an average value of 9. Lukas and De Bussy (*32*) reported a β value of 5.1 for cohesive tills and hardpan in Chicago.

The ratio of E_p/P_l relates to soil properties and may be used to classify soils. Baguelin et al. (*20*) and Gambin (*21*) showed that for OC clay, E_p/P_l is greater than 16. Lukas and Seiler (*16*) showed that E_p/P_l varies from 4 to 11 for low-plasticity clays. For high-plasticity clays, E_p/P_l ranges from 8 to well over 25.

FIELD VANE SHEAR TEST

Apparatus and Test Procedure

The field vane test (FVT) has been used extensively for the in situ determination of the undrained strength of soft clays. For practical purposes, the FVT may be used in clays with OCRs less than 10. Beyond that, the excessive torque may cause distortion or even breakage to the vane blades, unless an unusually small vane is used.

A standard field vane (ASTM D2573-72) has four blades. The height of the vane (*H*) should be twice the diameter (*D*). The rod friction should be accounted for with the use of sleeved rods or a slip coupling. The vane should be inserted to a depth that is at least five times the diameter of the borehole or that of the vane housing before testing. There is usually a "rest period" of not more than 5 min following vane insertion. The vane should be rotated at a rate not exceeding 0.1 degree/sec (ASTM D2573-72). Following the determination of the maximum torque, the vane is rotated rapidly for 10 revolutions. The test is then repeated to determine the remolded strength. The ratio of peak to remolded strength is referred to as sensitivity.

Vanes with different dimensions are allowed by the ASTM standard. An advantage of allowing different vane dimensions is that the accuracy of the torque measuring system maybe optimized. However, for a given rate of rotation, the strain rate at the tip of the vane blade is proportional to the vane diameter. Studies (*33–35*) have shown that the strain-rate effects are important for the FVT. To minimize the strain rate effects, it is beneficial to restrict the vane dimensions. Chandler (*36*) suggests that since the most widely used dimensions are H = 130 mm and D = 65 mm, these would seem to be the most appropriate for standardization.

Although not specified in the ASTM standard, there seems to be a general agreement that the vane blade thickness should be approximately 2 mm, and the area ratio (the ratio of the volume of soil displaced by the vane to the soil volume swept by the rotated vane) should be less than 12 percent.

Interpretation

Assuming that the clay is isotropic and shear stress is uniformly distributed along the edge of the vane blades, then for $H/D = 2$, the undrained shear strength is

$$s_u = 0.86T/\pi D^3 \tag{12}$$

where T is the maximum recorded torque. Wroth (*37*) concluded that the shear stress distribution at the top and bottom of the vane blades should be described by a polynomial. In that case

$$s_u = 0.94T/\pi D^3 \tag{13}$$

Using Wroth's approach, the vertical surfaces contribute 94 percent of the resistance to the total torque, not 86 percent according to Equation 12, and the shear strength will be dominantly that exhibited by the vertical planes. Consequently, Equation 12 would underestimate s_u and the FVT is not likely to reveal the strength anisotropy by changing H/D ratios.

Due to strain rate effects and soil anisotropy, Bjerrum (*38,39*) pointed out that there is a discrepancy between the shear strength from the FVT ($s_{u(FVT)}$) and that backcalculated from embankment and excavation failures and proposed that

$$s_{u\,\text{corrected}} = \mu s_{u(FVT)} \quad (14)$$

where μ is a correction factor based on the plasticity index, I_p. The validity of Bjerrum's approach was seriously questioned by many researchers [e.g. Schmertmann (*40*)] because of significant scatter of the data that Bjerrum used to establish μ values. Aas et al. (*41*) attributed that scatter to the lack of consideration of soil stress history and aging. A set of modified or renewed correction curves as shown in Figure 5 was proposed by Aas et al. (*41*). These curves consider both aging and stress history of clays. To use Figure 5, the $s_{u(FVT)}$ values should be calculated using Equation 12.

MARCHETTI DILATOMETER TEST

Apparatus and Test Procedure

The Marchetti dilatometer (*42*) consists of a stainless steel blade with a circular, expandable diaphragm on one side. The dimensions and geometry of the blade are shown in Figure 6. The Marchetti dilatometer test (DMT) involves the penetration of the blade followed by expansion of the diaphragm. A recommended DMT test procedure has been proposed by Schmertmann (*43*). Upon penetration, the diaphragm is expanded slowly by air pressure. A pressure gauge in the control console monitors the air pressure being applied behind the diaphragm. The console gives an electric signal when the diaphragm moves 0.05 mm horizontally off the vertical blade and when the central diaphragm expansion reaches 1.1 mm. The two corresponding pressures are referred to as the A and B reading, respectively. These pressures are corrected for diaphragm stiffness such that

$$P_0 = 1.05(A - Z_M + \Delta A) - 0.05(B - Z_M - \Delta B) \quad (15)$$

$$P_1 = B - Z_M - \Delta B \quad (16)$$

where

P_0 = net soil pressure against the membrane immediately before its expansion into the soil,
P_1 = net soil pressure at 1.1 mm membrane expansion, and
Z_M = gauge pressure deviation from zero when vented at atmospheric pressure.

The tests are repeated at intervals of approximately 20 cm, thus resulting in a large number of data for a given location.

Interpretation

The P_0 and P_1 pressures along with an estimate of the effective vertical stress σ'_{vo} and hydrostatic pressure u_0 at the test level are used to provide three indices:

Material index:

$$I_D = \frac{P_1 - P_0}{P_0 - u_0} \quad (17)$$

Horizontal stress index:

$$K_D = \frac{P_0 - u_0}{\sigma'_{vo}} \quad (18)$$

Dilatometer modulus:

$$E_D = 34.7(P_1 - P_0) \quad (19)$$

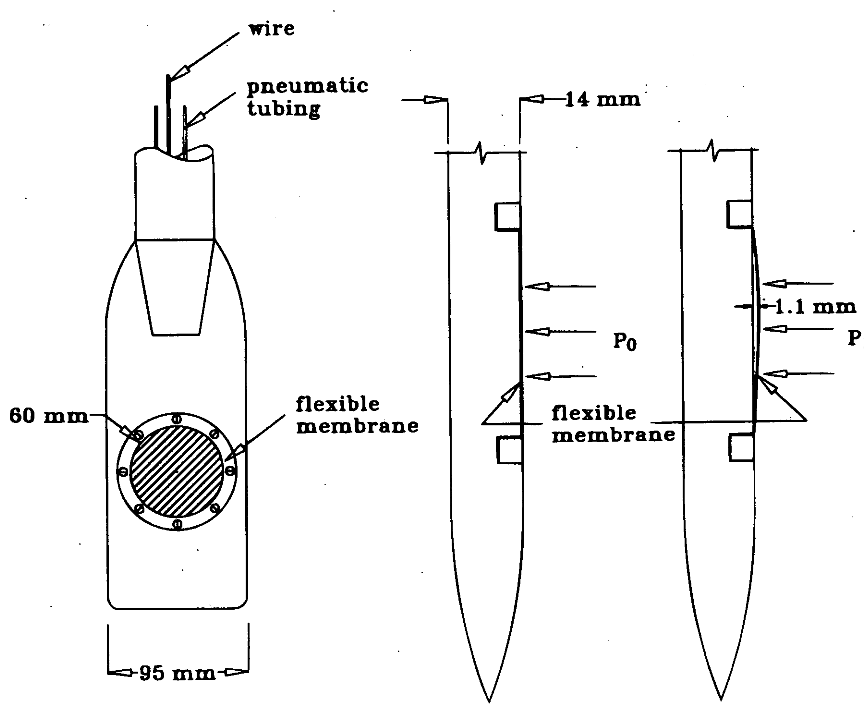

FIGURE 6 Marchetti dilatometer (*47*).

TABLE 2 Soil Classification Based on I_D Values (*48*)

	Soil Classification					
	Clay		Silt		Sand	
	Clay	Silty clay	Clayey silt	Silt	Silty sand	Sand
I_D value	0.10	0.35	0.6	0.9	1.8	3.3

Table 2 shows the soil classification according to I_D (*42*). Marchetti and many other researchers have proposed a series of empirical equations to relate soil parameters to the DMT indices. These soil properties include OCR, s_u, at-rest lateral earth pressure coefficient (K_0), constrained modulus (M), and initial modulus (E_i).

For uncemented OC clays ($I_D < 1.2$), resulted from simple unloading, Marchetti (*42*) proposed that

$$K_0 = (K_D/1.5)^{0.47} - 0.6 \tag{20}$$

and

$$OCR = (0.5\ K_D)^{1.56} \tag{21}$$

Following the concept of Ladd et al. (*44*), Marchetti (*42*) suggested estimating s_u based on

$$\frac{s_u}{\sigma'_{vo}} = 0.22(0.5K_D)^{1.25} \tag{22}$$

There is no unique relationship between M and E_D. If $R_M = M/E_D$ is considered, Marchetti (*42*) suggested that R_M increases with K_D, and proposed a series of empirical equations that relate these two parameters. However, because of the scatter of data, the validity of those equations is questionable.

More recent studies have proposed a linear relationship between E_i and E_D:

$$E_i = FE_D \tag{23}$$

For highly overconsolidated clays, Davidson and Boghrat (*45*) suggested that $F = 1.4$. For laterally loaded pile design, Robertson et al. (*46*) recommended that $F = 10$ for cohesive soils ($I_D < 1.0$). Lutenegger (*47*) suggested that F should decrease with I_D for clays.

Concluding Remarks

The CPT or CPTU is an efficient tool in establishing soil profiles and stratigraphy. Soil layers as thin as 5 mm could be identified with the help of pore pressure measurements in the CPTU (*2*).

The PMT is one of the few, if not the only, in situ testing method that measures a soil stress-strain curve. It has the potential of being very useful in predicting the performance of both the axially or laterally loaded foundations. The results of the PMT are sensitive to the quality of the borehole and the skills of the operator. It is thus imperative to follow the standard procedure as closely as possible and report details of the test method.

The FVT is a very useful tool in establishing the undrained shear strength profile of a clay deposit. Experience has indicated (*35,36*) that design of both the vane and test procedure can have significant effects on the test results. For the FVT results to be interpreted with meaning, it is important to report details of the test equipment and procedure utilized in the field.

Cases of foundation design or predicting field performance using DMT results have been reported (*46*). Most of these cases used conventional soil parameters derived from DMT data. Because of its efficiency and unique capability of measuring stress/stiffness in lateral direction, the DMT can be a very useful quality assurance tool for soil improvement operations (*48*).

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