Use of the Flat Dilatometer Test (DMT) in geotechnical design

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ABSTRACT: This paper presents an overview of the DMT and of its design applications, in the light of the experience accumulated over the last 20 years. The following applications are discussed: determining common soil parameters, predicting settlements of shallow foundations, compaction control, detecting slip surfaces in clay slopes, predicting the behavior of laterally loaded piles, evaluating sand liquefiability, estimating consolidation/flow coefficients, selecting soil parameters for FEM analyses. The basic differences of the DMT compared to other penetration tests are also discussed.

1 INTRODUCTION

The Flat Dilatometer Test (DMT), developed in Italy in 1980, is currently used in over 40 countries both for research and practical applications. The wide diffusion of the DMT lies on the following reasons (Lutenegger 1988): (a) Simple equipment and operation. (b) High reproducibility. (c) Cost effectiveness. (d) Variety of penetration equipment.

The test procedure and the original correlations were described by Marchetti (1980). Subsequently, the DMT has been extensively used and calibrated in soil deposits all over the world. In addition to some 300 research publications, various standards (ASTM Suggested Method 1986), regulations (Eurocode 7 1997) and manuals (US DOT 1992) are available today. Design applications, recent findings and new developments are described by Marchetti (1997) in a state-of-the-art report.

2 DESCRIPTION OF THE TEST

The dilatometer consists of a steel blade having a thin, expandable, circular steel membrane mounted on one face. When at rest, the membrane is flush with the surrounding flat surface of the blade. The blade is connected, by an electro-pneumatic tube running through the insertion rods, to a control unit on the surface (Fig. 1). The control unit is equipped with pressure gauges, an audio-visual signal, a valve for regulating gas flow (provided by a tank) and vent valves. The blade is advanced into the ground using common field equipment, i.e. push rigs normally used for the cone penetration test (CPT) or drill rigs. (The DMT can also be driven, e.g. using the SPT hammer and rods, but statical push is by far preferable). Pushing the blade with a 20 ton penetrometer truck is most effective (up to 100 m of profile per day).

The test starts by inserting the dilatometer into the ground. Soon after penetration, the operator inflates the membrane and takes, in about 1 min, two readings: the *A* pressure, required to just begin to move the membrane ("lift-off"), and the *B* pressure, required to move the center of the membrane 1.1 mm against the soil. A third reading *C* ("closing pressure") can also optionally be taken by slowly deflating the membrane soon after *B* is reached. The blade is then advanced into the ground of one depth increment (typically 20 cm).

The pressure readings *A*, *B* are corrected by the values ΔA, ΔB determined by calibration to take into account the membrane stiffness (Marchetti 1999b) and converted into p_0, p_1 as indicated in Table 1. The data p_0 and p_1 are generally interpreted (in "normal" soils) using the correlations reported in Table 1.

The DMT can test from extremely soft to very stiff soils (clays with c_u from 2 - 4 to 1000 kPa, moduli *M* from 0.5 to 400 MPa).

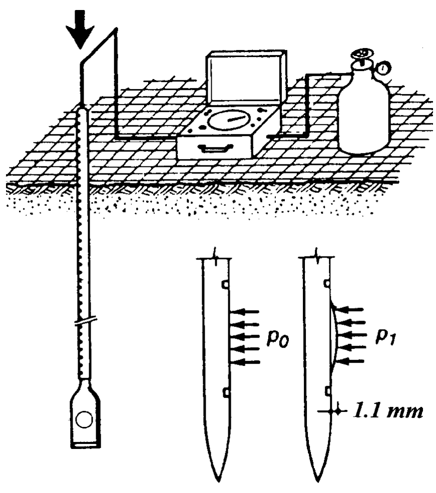

Figure 1. General layout of the dilatometer test

Table 1. Basic DMT reduction formulae

SYMBOL	DESCRIPTION	BASIC DMT REDUCTION FORMULAE	
p_0	Corrected First Reading	$p_0 = 1.05\ (A - Z_M + \Delta A) - 0.05\ (B - Z_M - \Delta B)$	Z_M = Gage reading when vented to atm. If ΔA & ΔB are measured with the same gage used for current readings A & B, set $Z_M = 0$ (Z_M is compensated)
p_1	Corrected Second Reading	$p_1 = B - Z_M - \Delta B$	
I_D	Material Index	$I_D = (p_1 - p_0) / (p_0 - u_0)$	u_0 = pre-insertion pore pressure
K_D	Horizontal Stress Index	$K_D = (p_0 - u_0) / \sigma'_{v0}$	σ'_{V0} = pre-insertion overburden stress
E_D	Dilatometer Modulus	$E_D = 34.7\ (p_1 - p_0)$	E_D is NOT a Young's modulus E. E_D should be used only AFTER combining it with K_D (Stress History). First obtain $M_{DMT} = R_M E_D$, then e.g. $E \approx 0.8\ M_{DMT}$
K_0	Coefficient Earth Pressure in Situ	$K_{0,DMT} = (K_D / 1.5)^{0.47} - 0.6$	for $I_D < 1.2$
OCR	Overconsolidation Ratio	$OCR_{DMT} = (0.5\ K_D)^{1.56}$	for $I_D < 1.2$
c_u	Undrained Shear Strength	$c_{u,DMT} = 0.22\ \sigma'_{V0}\ (0.5\ K_D)^{1.25}$	for $I_D < 1.2$
j	Friction Angle	$\varphi_{safe,DMT} = 28° + 14.6 \log K_D - 2.1 \log^2 K_D$	for $I_D > 1.8$
c_h	Coefficient of Consolidation	$c_{h,DMTA} \approx 7\ cm^2 / T_{flex}$	T_{flex} from A-log t DMTA-decay curve
k_h	Coefficient of Permeability	$k_h = c_h\ \gamma_w / M_h\ \ (M_h \approx K_0\ M_{DMT})$	
g	Unit Weight and Description	(see Marchetti 1999a)	
M	Vertical Drained Constrained Modulus	$M_{DMT} = R_M E_D$ if $I_D \le 0.6$ $\quad R_M = 0.14 + 2.36 \log K_D$ if $I_D \ge 3$ $\quad R_M = 0.5 + 2 \log K_D$ if $0.6 < I_D < 3$ $\quad R_M = R_{M,0} + (2.5 - R_{M,0}) \log K_D$ with $R_{M,0} = 0.14 + 0.15\ (I_D - 0.6)$ if $K_D > 10$ $\quad R_M = 0.32 + 2.18 \log K_D$ if $R_M < 0.85$ $\quad$ set $R_M = 0.85$	
u_0	Equilibrium Pore Pressure	$u_0 \approx p_2 \approx C - Z_M + \Delta A$	In freely-draining soils

An example of DMT results is shown in Figure 2. The results are used as follows:

- I_D (Material Index) gives information on soil type (sand, silt, clay).
- M (vertical drained constrained modulus) and c_u (undrained shear strength) in the usual ways.
- The profile of K_D (Horizontal Stress Index) is similar in shape to the profile of the overconsolidation ratio OCR. $K_D \approx 2$ indicates in clays $OCR = 1$, $K_D > 2$ indicates overconsolidation. A first glance at the K_D profile is helpful to "understand" the deposit.

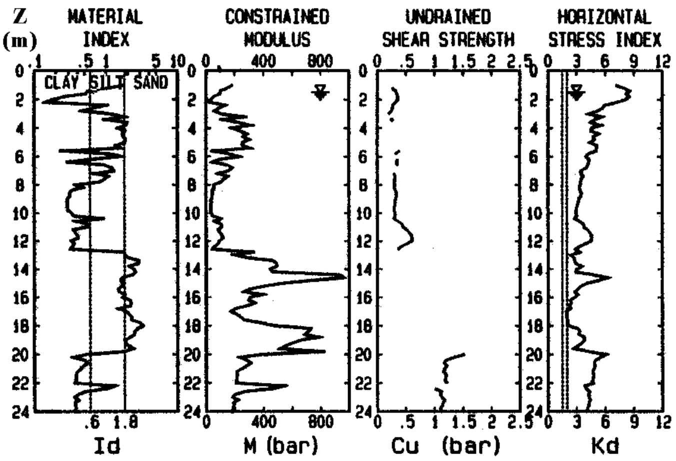

Figure 2. Example of DMT results

3 DMT vs OTHER PENETRATION TESTS

(a) Comparative studies have indicated that the DMT results (in particular K_D) are noticeably reactive to factors that are scarcely felt (especially in sands) by other tests, such as stress state/history, aging, cementation, structure. Such factors are scarcely reflected e.g. by q_c (cone penetration resistance from CPT) and by N_{SPT}, and in general, also due to the arching phenomenon, by cylindrical-conical probes (Marchetti 1999a). Yet such factors are of primary importance in determining some basic soil properties, e.g. deformability and (in sands) resistance to liquefaction.

(b) The DMT provides two independent parameters, while most of the penetration tests just provide one "primary" parameter (the penetration resistance) for the interpretation. It is known that in situ tests represent an "inverse boundary conditions problem", since such tests measure soil responses instead of soil properties. According to the theory, an in situ test should be able to measure 36 responses, being 36 the (variable) coefficients linking the 6 stress components to the 6 strain components. One measurement is a very small fraction of 36. Two measurements are also a very small fraction, yet 100% more than one measurement.

4 DESIGN APPLICATIONS

4.1 *Design using soil parameters*

In most cases the DMT is used to determine "commonly used" geotechnical design parameters, notably the undrained shear strength c_u and the constrained modulus M. Comparisons carried out at various National Research Sites by international research groups (see Figs 3-4) have shown quite good agreement between the profiles of $c_{u\ DMT}$ and M_{DMT} and the profiles determined by other tests.

Figure 5 illustrates the good correlation between K_D and OCR (note that $K_D \approx 2$ for $OCR = 1$). Such correlation has also been confirmed theoretically by Finno (1993). The ability to estimate OCR is important, since OCR governs many soil properties, while, on the other hand, OCR profiles are usually hard and costly to obtain. A comparison between profiles of OCR estimated by DMT and by other tests at the Bothkennar Research Site is presented by Marchetti (1997).

4.2 *Settlement prediction*

Predicting settlements of shallow foundations is probably the application No. 1 of the DMT, especially in sands, where undisturbed sampling and estimating deformability parameters are particularly difficult. Settlements are generally calculated by means of the one-dimensional formula:

$$S = \sum \frac{\Delta s_v}{M_{DMT}} \Delta z \tag{1}$$

with $\Delta\sigma_v$ generally calculated according to Boussinesq and M_{DMT} constrained modulus estimated by DMT using the correlation (see Table 1) $M_{DMT} = R_M \, E_D$, where R_M is a function primarily of K_D. Since K_D incorporates the effects of the horizontal stresses σ_h and stress history, then also M_{DMT} incorporates, through K_D, such effects. The capability of taking into account σ_h is important, since high σ_h dramatically reduce settlements (as observed e.g. by Massarsch 1994). For this reason E_D, in general, should not be used as such, because it lacks the stress history information contained in K_D, but should first be combined with K_D to obtain M. Note that E_D (despite the symbol) should not be confused with the Young's modulus E. If required, E can be derived from M via theory of elasticity ($E \approx 0.8\, M$ for $\nu = 0.20 - 0.30$).

Several studies have indicated that the DMT reduces the uncertainty in settlement predictions by a factor of over 3 compared with predictions based on penetration resistance q_c from CPT. This can be observed e.g. by comparing the datapoints band amplitude (ratio between maximum and minimum) in Figure 6a (Hayes 1990) and in Figure 6b (Baldi et al. 1988). Among the reasons: (a) Wedge shaped tips deform the soil considerably less than conical tips (Baligh & Scott 1975). (b) The modulus obtained by expanding the DMT membrane (a "mini" load test) is physically more related to deformability than is the penetration resistance. (c) The availability of a second independent parameter K_D, reflecting σ_h / stress history, leads to more realistic values of M.

The accuracy of settlements predicted by DMT has been confirmed by many investigators. Schmertmann (1986a) reported 16 comparisons of observed vs DMT-calculated settlements at different sites and for various soil types. The ratio calculated/observed settlement was 1.18 on average, with a narrow variation range (from 0.75 to 1.3). Similar agreement was observed, among many others, by Lacasse & Lunne (1986) and Sallfors (1988).

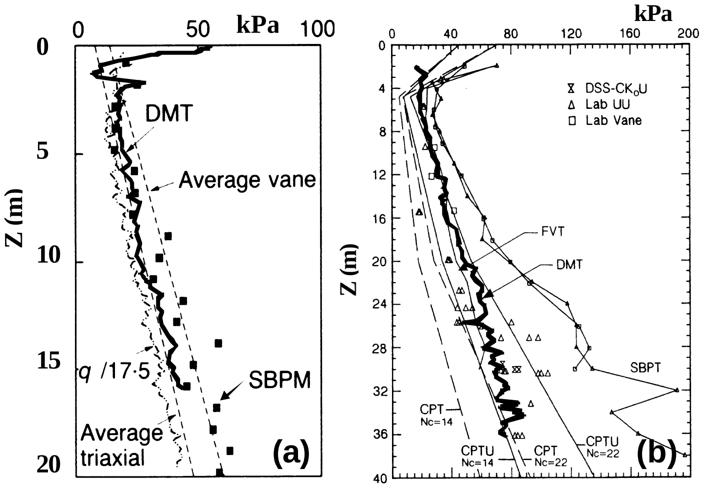

Figure 3. Comparison between c_u determined by DMT and by other tests. (a) National Research Site of Bothkennar, UK (Nash et al. 1992). (b) National Research Site of Fucino, Italy (AGI 1991).

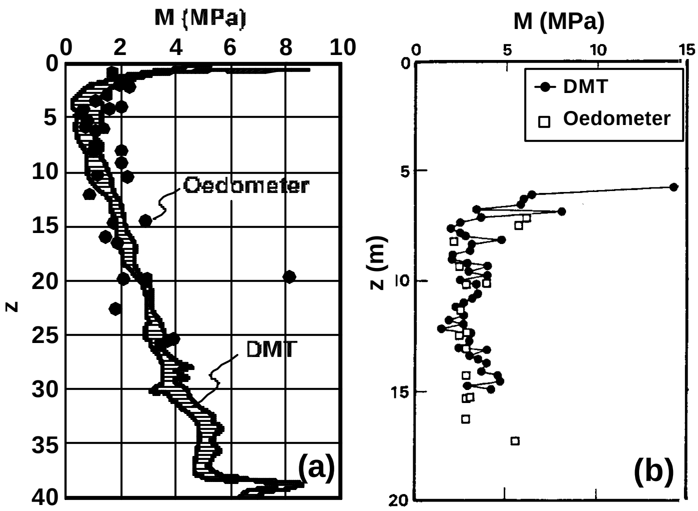

Figure 4. Comparison between M determined by DMT and by oedometer tests. (a) Onsøy (Norway). Tests performed by Norwegian Geotechnical Institute. (b) Komatsugawa (Tokyo, Japan). Tests performed by Kiso-Jiban Geotechnical Research Center.

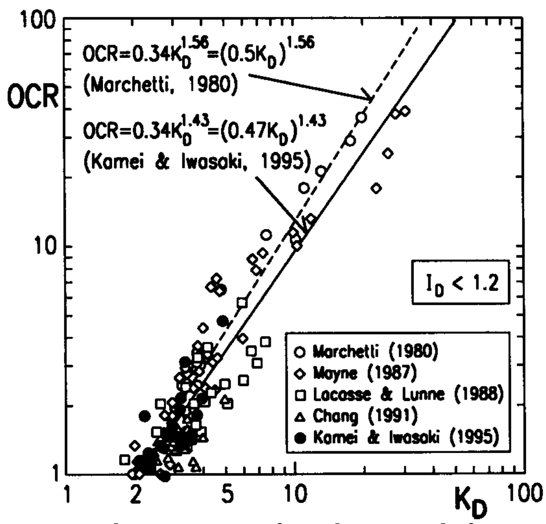

Figure 5. Correlation K_D-OCR for cohesive soils from various geographical areas (Kamei & Iwasaki 1995)

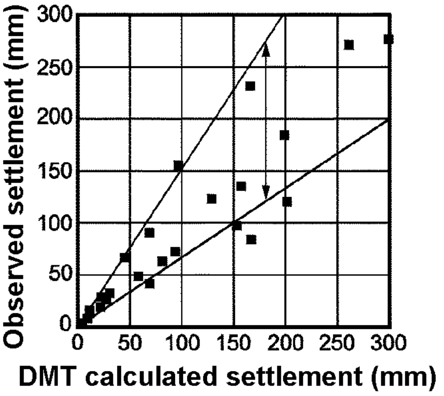

Figure 6a. Comparison between observed and DMT-calculated settlements (data by Hayes 1990)

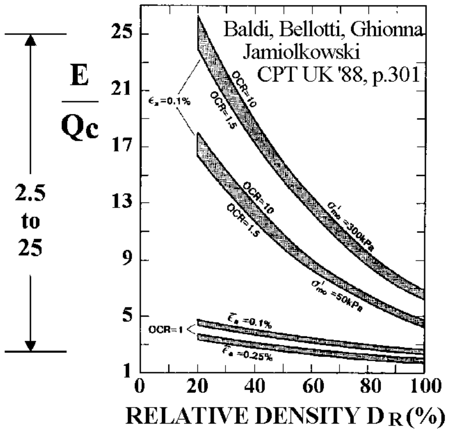

Figure 6b. Ratio E/q_c as a function of D_r and OCR - Ticino Sand (Baldi et al. 1988)

4.3 *Compaction control*

The DMT, due to its sensitivity to σ_h, is particularly suitable to monitor soil improvement. Several studies present comparisons of results of CPTs and DMTs performed before/after a compaction treatment. Schmertmann (1986b) observed that the increase in M_{DMT} after dynamic compaction of a sandy soil was approximately twice the increase in q_c (CPT). Similar results were observed by Jendeby (1992) in monitoring deep compaction of a loose sand fill by "vibrowing" (Fig. 7). The higher "sensitivity" of M_{DMT} compared to q_c was also observed, in a vibroflotation case, by Pasqualini & Rosi (1993).

The DMT has also been used to check the effects induced in the soil by the installation of various types of piles. De Cock et al. (1993) described the use of DMT performed before/after the installation of Atlas piles, and concluded that the DMT detects more clearly than the CPT the effects of the installation.

All the above observations indicate that the DMT results are noticeably reactive even to slight changes of σ_h (or relative density) in the soil. Therefore, the DMT is particularly suitable in cases where the expected stress variations are very small (e.g. relaxation upon "microboring").

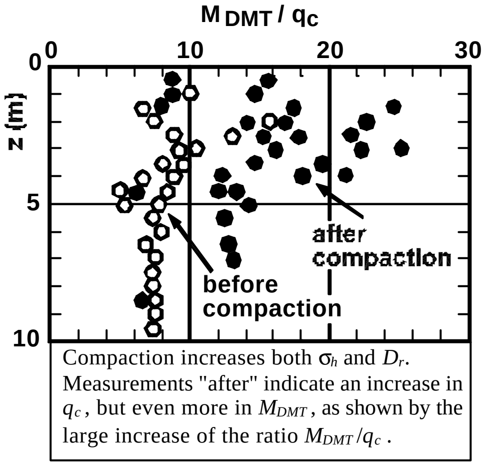

Figure 7. Ratio M_{DMT}/q_c before and after compaction of a loose sand fill (Jendeby 1992)

4.4 *Detecting slip surfaces in clay slopes*

The DMT permits to verify quickly if a slope in overconsolidated (OC) clays contains active or old slip surfaces. The method proposed by Totani et al. (1997), based on inspection of the K_D profiles, is founded on the following basis (Fig. 8):

– the process of sliding and reconsolidation generally creates a remolded zone of nearly normally consolidated (NC) clay, with loss of structure, aging or cementation;
– since in NC clays $K_D \approx 2$, if an OC clay slope contains layers where $K_D \approx 2$, these layers are likely part of a slip surface (active or quiescent).

Note that the method involves searching for a specific numerical value ($K_D = 2$) rather than for simply "weak zones", which could be detected just as easily also by other in situ tests.

The "K_D method" provides a faster response than inclinometers in detecting slip surfaces (no need to wait for movements to occur). Moreover, the method enables to detect even possible quiescent surfaces (not revealed by inclinometers), which could reactivate e.g. after an excavation. On the other hand, the method itself, unlike inclinometers, does not permit to establish if the slope is moving at present and what the movements are. In many cases, DMT and inclinometers can be used in combination (e.g. use K_D profiles to optimize location/depth of inclinometers).

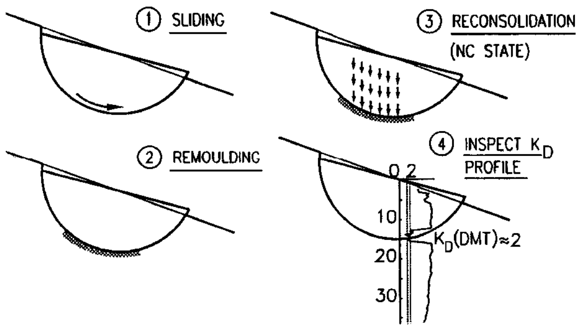

Figure 8. K_D method for detecting slip surfaces in OC clay slopes

4.5 *Laterally loaded piles*

Of the various methods developed for deriving *P-y* curves from DMT results, the ones recommended by the authors are those by Robertson et al. (1987) and Marchetti et al. (1991). A number of independent validations (NGI, Georgia Tech and tests in Virginia sediments) have indicated that the two methods provide similar predictions, in quite good agreement with the observed behavior.

4.6 *Liquefaction*

Figure 9 summarizes the available knowledge for evaluating sand liquefiability by DMT. The curve recommended to estimate the cyclic resistance ratio (*CRR*) from the parameter K_D is the curve by Reyna & Chameau (1991), based in part on their curve K_D - D_r (relative to NC sands) in Figure 10. This correlation has recently been confirmed by additional datapoints K_D - D_r obtained by Tanaka & Tanaka (1998) at the sites of Ohgishima and Kemigawa, where D_r was determined on high quality frozen samples. Once *CRR* has been evaluated from Figure 9, it is used in liquefaction analysis with the methods developed by Seed.

Figure 9 permits to estimate *CRR* as an alternative to the methods which derive *CRR* from N_{SPT} or q_c. The possibility of obtaining independent evaluations of *CRR* is of great interest, since it has been recently demonstrated (Sladen 1989; Yu et al. 1997) that the relation q_c - *SP* (*SP* = state parameter) is not unique, but strongly dependent on the stress level. Sladen (1989) has shown that ignoring the non-unicity of the correlation q_c - *SP* in design can lead to catastrophic consequences. The non-unicity of the correlation q_c - *SP*, due to the strong link *SP* - *CRR* (*SP* governs the attitude of a sand to increase or decrease in volume when sheared) involves large scatter in the correlation q_c - *CRR*, hence large errors in *CRR* estimated from q_c. In fact, Robertson (1998) warns that the correlation q_c - *CRR* may be adequate for low risk, small scale projects, while for medium to high risk projects he recommends to estimate *CRR* by more than one method. Moreover, experimental work over the last 20 years (an overview is presented by Marchetti 1999c) has shown that, while K_D is fairly sensitive to the past stress-strain history, q_c is scarcely reactive to this factor, which, on the other hand, greatly increases resistance to liquefaction.

Figure 9, in combination with the available experience, suggests that a clean sand (natural or sandfill) is adequately safe against liquefaction (M = 7.5 earthquakes) for the following K_D values:

- non seismic areas: $K_D > 1.7$
- low seismicity areas ($a_{max}/g = 0.15$): $K_D > 4.2$
- medium seismicity areas($a_{max}/g = 0.25$): $K_D > 5.0$

high seismicity areas ($a_{max}/g = 0.35$): $K_D > 5.5$

4.7 *Consolidation and flow parameters*

The DMT permits to estimate the horizontal

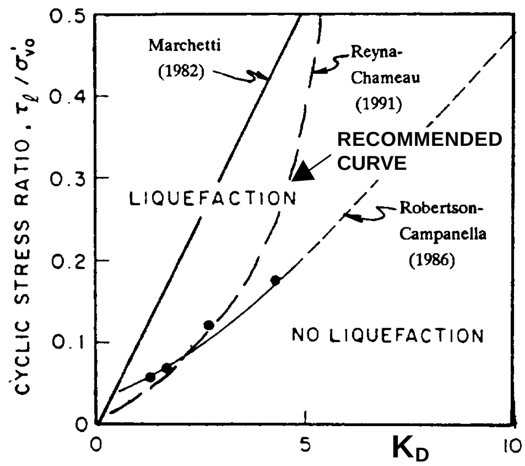

Figure 9. Curves for estimating the cyclic resistance ratio *CRR* from K_D (Reyna & Chameau 1991)

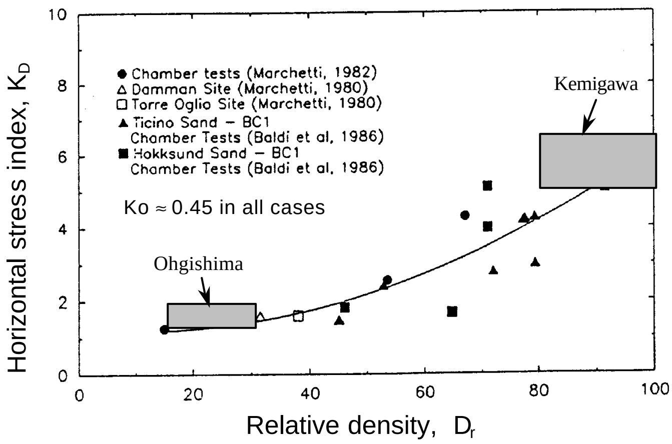

Figure 10. Correlation K_D - D_r for NC sands (Reyna & Chameau 1991). The shaded areas represent datapoints obtained by Tanaka & Tanaka (1998) on frozen samples.

coefficient of consolidation c_h and permeability k_h in clay by means of dissipation tests. Various procedures have been formulated (DMTC, Robertson et al. 1988; DMTA, Marchetti & Totani 1989). All methods are based on the decay with time of $\sigma_{h\ total}$ against the membrane after stopping the blade at a given depth.

The DMTA method (probably more used than DMTC) consists of taking a timed sequence of *A* readings until stabilization. (The DMTA dissipation is perfectly analogous to the "holding test" by pressuremeter). c_h is estimated from the time T_{flex} at which the S-shaped decay curves *A - log t* exhibit a contraflexure point. The coefficient of permeability k_h is then determined from c_h and M_{DMT} (see formulae in Table 1). Case histories presented by Totani et al. (1998) have indicated that c_h values from DMTA are generally 1 to 3 times smaller than c_h back-calculated from "real life" observations.

Determining c_h and k_h from DMT dissipations presents various advantages over the piezocone: (a) lower distorsion induced in the soil by the penetration of the blade; (b) absence of problems of saturation, filter clogging, smearing; (c) "integral" - rather than "punctual" - measurement.

4.8 *Use of DMT for FEM input parameters*

Two approaches have been considered so far.
(a) Model the dilatometer test by a FEM computer program by adjusting the input parameters until the DMT results are correctly "predicted". This approach has the shortcoming of involving, at the same time, many additional (unknown) parameters.
(b) Based on the soil information available, give an initial "tentative" set of input FEM parameters. Then simulate by FEM a simple laboratory test (e.g. oedometer), adjusting FEM input parameters to improve the matching of M_{FEM} vs M_{DMT}. This approach is less ambitious, yet it could help avoid gross mistakes in the FEM analysis.

An example of use of deformation parameters determined by DMT in design of underground structures (Cairo metro tunnels) is illustrated by Hamza & Richards (1995). Their numerical analyses adopted the simplest possible model (linear elastic), with $E \approx 0.8\ M_{DMT}$. The model is elementary, but often even simple models, with a judicious choice of the parameters, can provide fairly accurate solutions.

5 CONCLUSIONS

Based on the available experience, the DMT best applications are: (a) M and c_u profiles. (b) Settlement prediction. (c) Monitoring soil improvement. (d) Recognizing soil type. (e) Distinguish freely-draining from non freely-draining layers. (f) Verify if a clay slope contains active/old slip surfaces.

The DMT also gives useful information on: (a) *OCR* and K_0 in clay. (b) Coefficient of consolidation and permeability. (c) *P-y* curves for laterally loaded piles. (d) Sand liquefiability. (e) Friction angle in sand. (f) (*OCR* and K_0 in sand).

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