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In Situ Tests by Flat Dilatometer

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Introduction

This paper describes the flat dilatometer, a recently developed device (18) for in situ investigation of soil properties, and presents a series of empirical correlations between test results and some geotechnical parameters used in design.

The information presented is based on the experience gained in performing dilatometer tests (DMT) at over 40 sites. The correlations have been established based on DMT performed at selected sites reasonably homogeneous and geotechnically well documented.

Description of Test

The flat dilatometer [Figs. 1(*a*) and 1(*b*)] consists of a stainless steel blade with a thin flat circular expandable steel membrane on one side. When at rest, the external surface of the membrane is flush with the surrounding flat surface of the blade. The blade is jacked into the ground using a penetrometer rig [Fig. 1(*c*)] or a ballasted drilling rig. The blade is connected to a control unit on the surface by a nylon tube containing an electrical wire. The tube runs through the penetrometer rods. At 20-cm depth intervals jacking is stopped and, without delay, the membrane is inflated by means of pressurized gas. Readings are taken of the A pressure required to just begin to move the membrane and of the B pressure required to move its center 1.00 mm into the soil. The rate of pressure increase is set so that the expansion occurs in 15 sec–30 sec. The total time needed for obtaining a 30-m profile, if no obstructions are encountered, is about 2 h.

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FEATURES OF TEST

Dimensions.—The membrane diameter, *D*, (60 mm) and the blade width (95 mm) were chosen as large as possible consistent with the ability to carry out

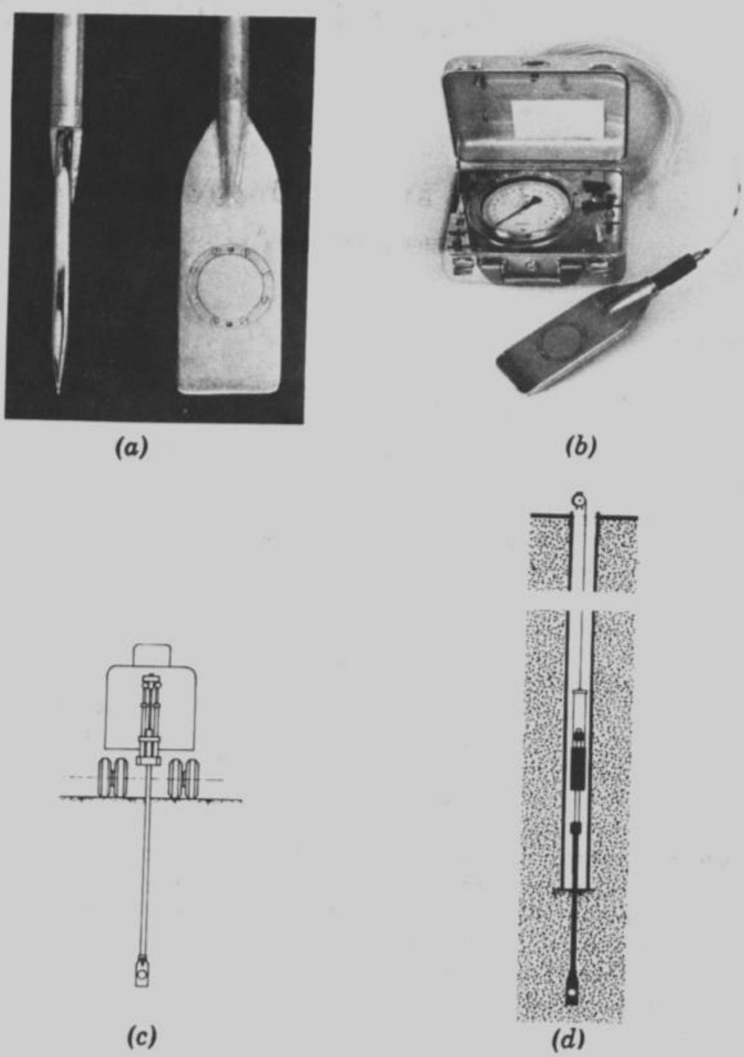

FIG. 1.—Flat Dilatometer: (*a*) Side View and Front View; (*b*) Blade, Control Unit and Cable; (*c*) Dilatometer Being Jacked into Ground; (*d*) Dilatometer Being Driven by Down-The-Hole Wireline Hammer

tests from the bottom of cased holes of usual diameters. The thickness of the blade (14 mm) was chosen as small as possible consistent with the requirement that it must not be easily damaged or bent. The specified deflection, s_o, was chosen as small as possible (1 mm) in order to keep soil strains in the expansion stage as small as possible.

Penetration Rate.—Penetration rates in the range of 2 cm/s–4 cm/s have generally been adopted. No specific investigations have been carried out to explore the influence of penetration rate on A and B. However the continuity of the profiles at the sites described later in the paper, where penetration rates varied randomly, suggests it to be fairly small.

Expansion Rate.—It has been observed in various soils that a variation by a factor of 2 in the expansion rate does not significantly alter the increment of pressure required for the expansion.

Alternative Methods of Advancing Dilatometer.—Tests have been performed in which penetration was accomplished using a down-the-hole wire line sliding

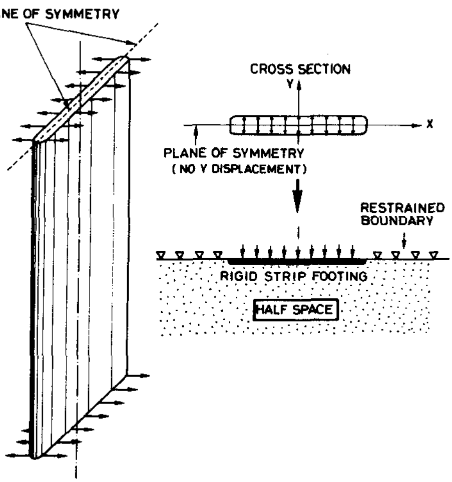

FIG. 2.—Modeling Dilatometer Penetration as Expansion of Flat Cavity

hammer [Fig. 1(d)]. A limited number of comparative tests performed in the Montalto overconsolidated clays indicated that for these clays the difference in the results, compared with those obtained by jacking the blade, were quite small.

Test Mechanism

Penetration Stage.—The penetration of the dilatometer can be regarded as a complex loading test on the soil. A possible way of analyzing the penetration process is to model it as the expansion of a flat cavity (Fig. 2), tractable as the enforcement of two vertical rigid strip footings into the soil. An analysis of this kind is outside the scope of the present paper. However such analysis would indicate that the measured horizontal total soil pressure against the blade increases with the horizontal in situ stress, soil strength parameters, soil stiffness.

The penetration of the dilatometer causes a horizontal displacement of the soil elements originally on the vertical axis of 7 mm (half thickness of the dilatometer), displacement considerably lower than that induced by currently used conical tips [18 mm for cone penetration test (CPT)]. Fig. 3 which, according to a theoretical solution by Baligh (4), shows the different strains caused by wedges having an apex angle of 20° (angle of the dilatometer) and 60° (angle of many conical tips), may give an idea of the different magnitudes of the strains induced by DMT and CPT.

During the penetration of the dilatometer there is a concentration of shear strain near the edges of the blade, so that the volume of soil facing the membrane

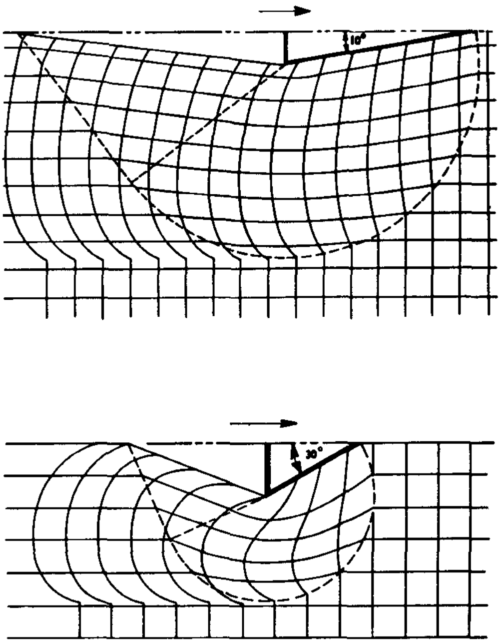

FIG. 3.—Deformation of Square Grid Due to Wedge Penetration (4)

undergoes a shear strain lower than the average (20).

Expansion Stage.—In this stage the increments of strain in the soil are relatively small. The theory of elasticity may be used to infer a modulus. Such modulus competes primarily to the volume of soil facing the membrane. However this soil has been prestrained during the penetration. As already noted, shear strains in this volume are low (compared with the strains induced by other presently used penetrating devices). However soil stiffness is sensitive to prestrains. Thus correction factors are necessary for evaluating the stiffness of the original soil. In sensitive soils, properties alterations due to the penetration are generally large and undefinable, so that the properties of the original soil cannot be traced back.

Reduction of Data

Readings A and B are first corrected for membrane stiffness in order to determine pressures p_o and p_1 which are applied to the soil at the start and at the end of the expansion. The following expressions are used:

$$p_o = A + \Delta A \quad (1)$$

$$p_1 = B - \Delta B \quad (2)$$

in which ΔA = the external pressure which must be applied to the membrane in free air to keep it in contact with its seating. The membrane once used, acquires a permanent outward curvature. The value ΔB is the internal pressure which, in free air, lifts the membrane center 1.00 mm from its seating.

The values ΔA and ΔB can be measured, by a simple field procedure, by applying to the dilatometer inlet vacuum and pressure, respectively, and reading the values at which the membrane is seated or deflected by 1.00 mm. Measurements taken before and after many tests indicate the following ranges of these corrections: $\Delta A = 0.15 \pm 0.05$ kg/cm^2, $\Delta B = 0.5 \pm 0.2$ kg/cm^2 (1 kg/cm^2 = 2.048 ksf = 98.1 kPa). In practice average values of ΔA and ΔB can be used for all except very soft cohesive soils. In these soils it may be worthwhile to measure ΔA and ΔB directly.

Once having determined p_o and p_1, the difference $\Delta p = p_1 - p_o$ can be computed. The value Δp can be converted into a modulus of elasticity of the soil using the theory of elasticity. For this problem a solution is available if the space surrounding the dilatometer is admited to be formed by two elastic half spaces in contact along the plane of symmetry of the blade. For an elastic half space, having Young's modulus E and Poisson's ratio μ, subject to the condition of zero settlement external to the loaded area, it is (13):

$$s_o = \frac{2\,D\,\Delta p}{\pi}\,\frac{(1-\mu^2)}{E} \quad (3)$$

For a membrane diameter D = 60 mm and s_o = 1 mm, Eq. 3 becomes:

$$\frac{E}{1-\mu^2} = 38.2\,\Delta p \quad (4)$$

The ratio $E/(1-\mu^2)$ calculated using Eq. 4 is termed hereafter dilatometer modulus E_D.

Test Results

For each site, DMT results are presented along with the available geotechnical information. The DMT results are presented as diagrams of p_o, p_1 versus depth, and as diagrams of the calculated parameters I_D, K_D, E_D defined as follows:

$$I_D = \frac{\Delta p}{p_o - u_o}; \quad \text{Material Index} \quad (5)$$

$$K_D = \frac{p_o - u_o}{\bar{\sigma}_v}; \quad \text{Horizontal Stress Index} \quad (6)$$

$E_D = 38.2\,\Delta p$; Dilatometer Modulus . . . . (7)

in which u_o and $\bar{\sigma}_v$ are the pore water pressure and the vertical effective stress, respectively, prior to blade insertion and have to be known (at least approximately).

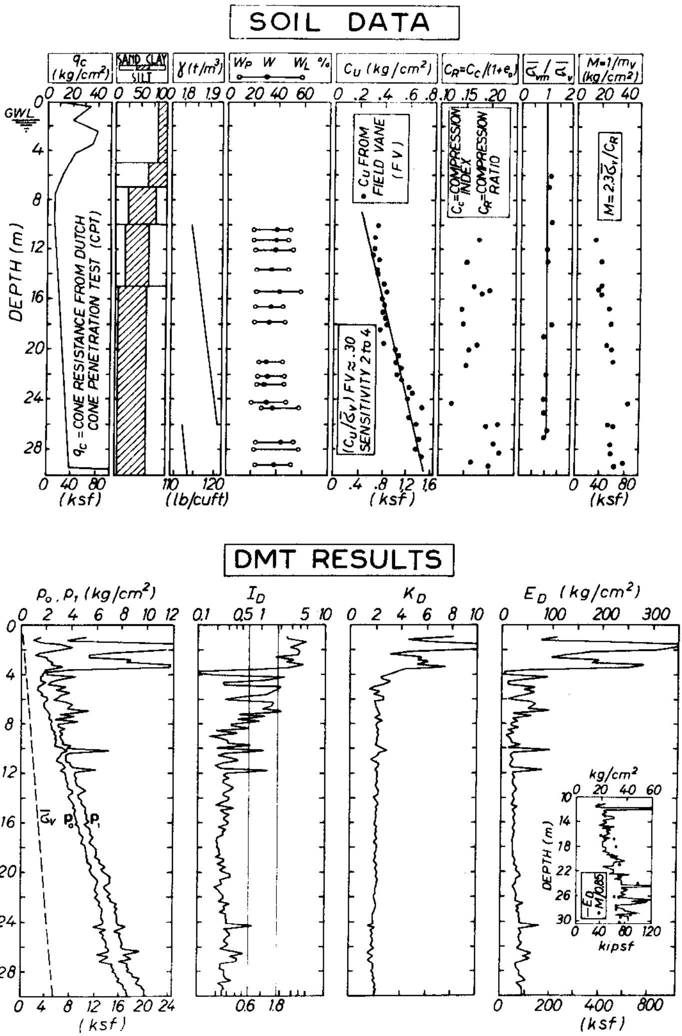

FIG. 4.—Soil Data and Dilatometer Test Results at Porto Tolle (1 kg/cm² = 98.1 kPa)

The following comment clarifies the use of the difference $p_o - u_o$ (rather than of p_o) in Eqs. 5 and 6. In two underwater soil deposits, identical in all except for the head of water above ground surface, at identical depth below ground surface, the difference $p_o - u_o$ for both deposits is equal while the value of p_o will be different.

Test at Porto Tolle, Northern Italy

The soil type is recent, normally consolidated delta deposits of the river Po. Geotechnical properties of this site have been investigated in detail (e.g., 1, 2). A thick layer of "young" silty clay is found below 10-m depth (Fig. 4). Other geotechnical parameters of the silty clay layer are $\bar{c} = 0$, $\phi = 28°$, $K_o \simeq 0.53$.

Comments on DMT Results.—The following comments may be made:

1. In the silty clay layer p_o and p_1 increase approximately linearly with depth and are closed to each other. The value I_D which (see Eq. 5) is an index of the "relative spacing" between p_o and p_1, is relatively small (0.2 to 0.3).
2. In the silty clay layer the horizontal stress index, K_D, is almost constant ($K_D \simeq 2$).
3. In the silty clay layer E_D increases almost linearly with depth, the rate of increase being quite low.
4. Moving upwards in the top 10 m of the deposit the index, I_D, gradually increases. The same trend is observed in the percentage of coarse material.
5. Near the surface, where K_o values were expected to be higher than the normally consolidated values, due to desiccation effects and to the compaction performed on the made-sandfill, K_D is considerably higher than 2. In this sand layer E_D is far higher than in the silty clay layer.
6. For the silty clay layer the average value of R_M (defined as the ratio M/E_D, in which $M = 1/m_v$ is the local tangent constrained modulus) was selected by trial as 0.85 (see the small diagram included within the E_D versus depth graph in Fig. 4).

Test at Torre Oglio, Northern Italy

The soil type is normally consolidated (14) medium fine loose sand, of alluvial origin, in the Po river vally. Typical CPT and SPT results (26) are shown in Figs. 5(*a*) and 5(*b*). Relative densities determined from N_{SPT} using Ref. 12 are shown in Fig. 5(*d*); $\gamma \simeq 1.85$ tons/m^3 (115.5 lb/cu ft) (23). The in situ K_o is estimated in the range 0.40 ± 0.05 according to Ladd, et al. (16). Fig. 5(*e*) shows profiles of: (1) The value M determined by using the correlation $M = 2.5\ q_c$ (22); and (2) $E/(1 - \mu^2)$ determined from N_{SPT} using the correlation given by D'Appolonia, et al. (9) for NC sands.

Comments on DMT Results.—The following comments may be made:

1. The spacing between the p_o and p_1 profiles is relatively large, with I_D values (5 to 5.5) considerably higher than in cohesive materials.
2. The nearly constant value $K_D = 1.5$ to 1.6 is lower than the K_D found for the Porto Tolle NC silty clay layer.
3. The value E_D increases with depth at a faster rate than it does in the NC silty clay layer at Porto Tolle.
4. The two most satisfactory proportionality constants relating dilatometer's

E_D to the moduli plotted in Fig. 5(*e*), determined by trial, are $R_M \simeq 1$ using SPT data, $R_M \simeq 0.65$ using CPT data. Note that, for the purpose of determining R_M, both profiles in Fig. 5(*e*) have been considered as profiles of M, i.e.,

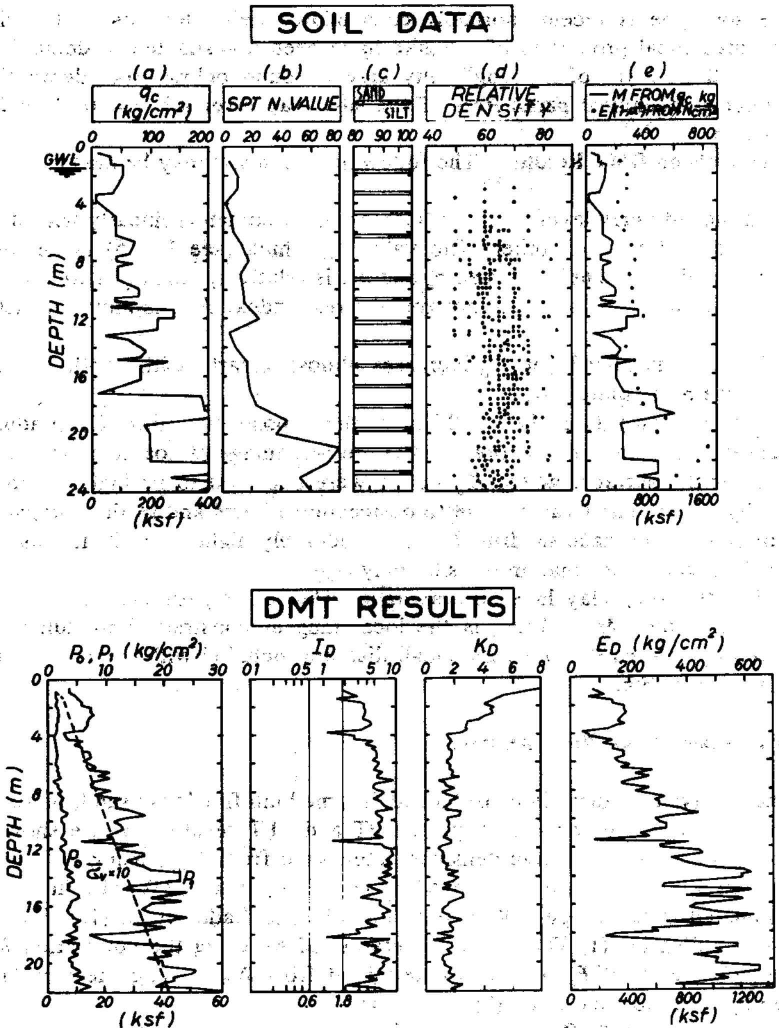

FIG. 5.—Soil Data and Dilatometer Test Results at Torre Oglio (1 kg/cm² = 98.1 kPa)

no distinction has been made between M and $E/(1 - \mu^2)$. This approximation appears reasonable, considering the margin of uncertainty of the used correlations and that, for $\mu = 0.25$, the theory of elasticity assigns a value of 1.11 to the ratio M to $E/(1 - \mu^2)$.

Tests at Damman, Saudi Arabia

The soil type is medium-fine loose sand (hydraulic fill, placed by sluicing) including some silty pockets. Several CPT, performed at some distance from the DMT location, indicated that in the top few meters q_c varies erratically in the range 10 kg/cm^2–50 kg/cm^2 (20 ksf–102 ksf). Below the surface layer: $q_c \simeq 50$ kg/cm^2. These were the only soil data available. The compaction treatment consisted of a triangular array of 2-m spaced, 13-m deep vibroflotation probes. The first DMT was performed prior to the compaction and the second DMT was performed several weeks after the compaction, at the centroid of a vibroflotation array. This DMT had to stop at about 8-m depth, as the rig pushing capacity was reached. (See Fig. 6.)

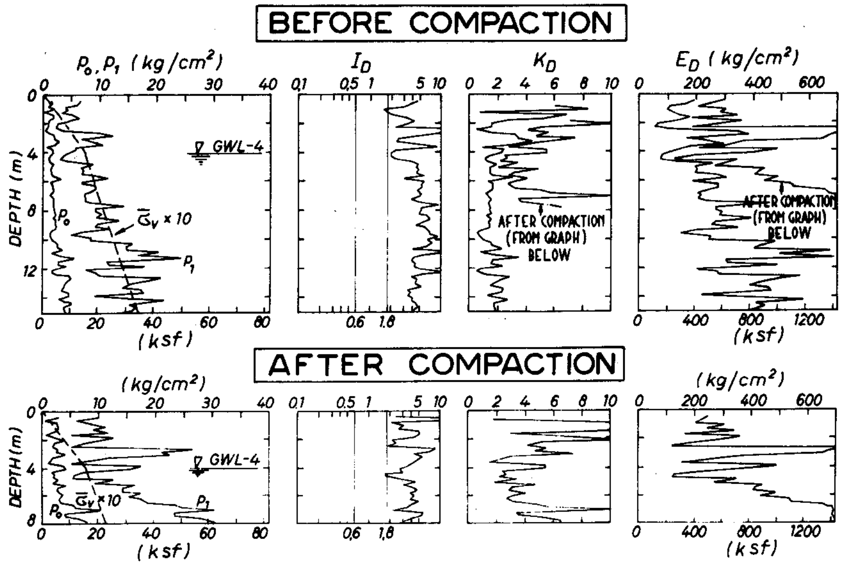

FIG. 6.—Damman-Saudi Arabia: Dilatometer Test Results Before and After Vibroflotation Treatment (1 kg/cm^2 = 98.1 kPa)

Comments on DMT Results.—The following comments may be made:

1. The profiles of p_o and p_1 in the sand prior to the compaction show an overall similarity with the Torre Oglio p_o and p_1 profiles (Fig. 5). Note that at both sites the soil consisted of loose NC sands.
2. The compaction significantly increased both the dilatometer modulus E_D and the horizontal stress index K_D (especially in those sands where $I_D > 5$).

Tests at Montalto, Central Italy

The soil type is overconsolidated marine Pliocenic clay (17). Fig. 7 shows test results on high quality piston samples (25). Note:

1. Values of the maximum past pressure, $\bar{\sigma}_{vm}$, have been determined from the oedometer compression curves using the Casagrande construction. The break in the laboratory curves is very well defined [a typical compression curve is shown in Fig. 7(*k*)].

2. The c_u to $\bar{\sigma}_v$ ratios plotted in Fig. 7(*f*) are based on the best fit straight line values of c_u [see Fig. 7(*d*)].

3. The overconsolidation ratios (OCR) plotted in Fig. 7(*h*) are based on the assumption that overconsolidation has been caused by a unique value of q

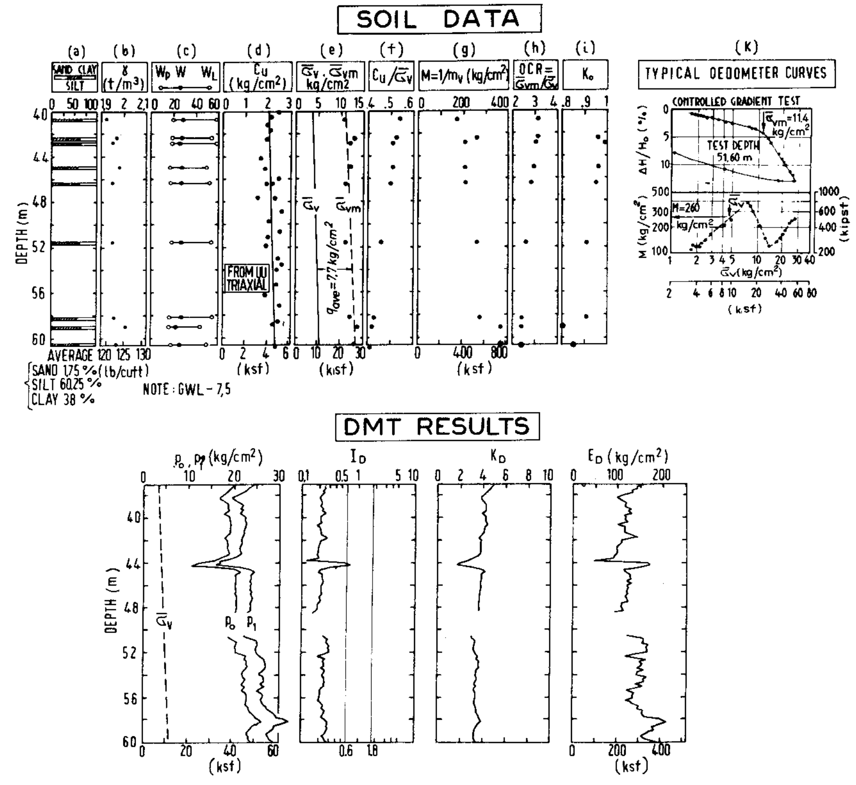

FIG. 7.—Soil Data and Dilatometer Test Results at Montalto (1 kg/cm² = 98.1 kPa)

(q = maximum past overburden on present ground surface; q is estimated as $\bar{\sigma}_{vm} - \bar{\sigma}_v$). The average value q = 7.7 kg/cm² (15.8 ksf) has been used to derive all OCR values (note: OCR = $1 + q/\bar{\sigma}_v$).

4. The tangent moduli M plotted in Fig. 7(*g*) have been derived directly from the oedometer (of a continuous loading type) consolidation curves, in correspondance to the in situ $\bar{\sigma}_v$. Considering the effects of sample disturbance the M values plotted should represent conservative estimates of in situ M.

5. The values of K_o shown in Fig. 7(*i*) have been calculated from the OCR and the plasticity index (PI) using Brooker and Ireland's correlations (6).

Note that all the overconsolidated clay deposits considered in this paper for which K_o has been estimated using (6) are marine Plio-Pleistocenic formations essentially in conditions of simple unloading (8,11), which are the conditions considered by Brooker and Ireland (6). The following circumstances indicate that these clays are not appreciably cemented: (1) The value $\bar{\sigma}_{vm}$ determined by oedometer tests is in reasonable agreement with the maximum past pressure estimated based on the geological history (8,11,17); and (2) the values of $q = \bar{\sigma}_{vm} - \bar{\sigma}_v$ calculated at different depths are reasonably constant.

Comments on DMT Results.—The following comments may be made: (1) The p_o and p_1 diagrams are close to each other (in relation to their distance from

SOIL DATA

DEPTH (m)	γ (t/m³)	W	W_L	PI	% SAND	% SILT	% CLAY	$\bar{\sigma}_v$ kg/cm²	Cu kg/cm²	$Cu/\bar{\sigma}_v$	$\bar{\sigma}_{vm}$ kg/cm²	q kg/cm²	OCR	K_o
6.8	2.01	21	425	28.4				0.65	1.29(U)	1.98			10.2	1.7
9.1	1.97	29.8	545	35.8				0.88			7.3	6.4	10.7	1.65
11.7	1.93	19.9			59	24	17	1.12					8.6	1.25
17.4	1.99	25.8	482	30.4	14	41	45	1.65	1.97 (UU)	1.19	10.2	8.5	6.2	1.2

NOTE: GWL ≃ GROUND SURFACE LEVEL

CONVERSION FACTORS
1 t/m³ = 62.43 lb/cuft
1 kg/cm² = 2.048 kipsf

DMT RESULTS

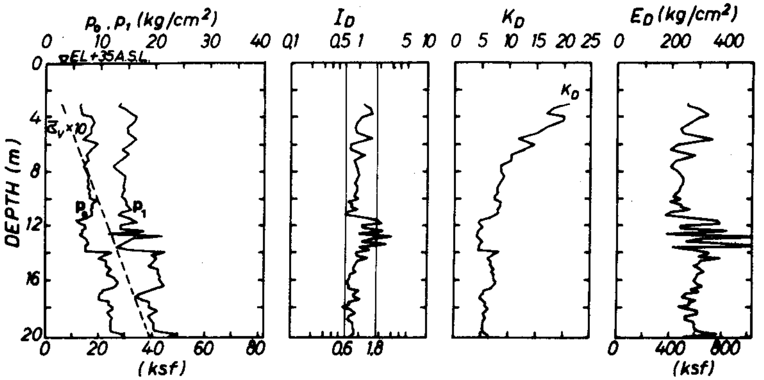

FIG. 8.—Soil Data and Dilatometer Test Results at Sciacca (1 kg/cm² = 98.1 kPa)

the vertical axis); (2) the I_D values are quite low (0.2 to 0.3); (3) the K_D decreases slightly with depth; and (4) the E_D increases slightly with depth.

Tests at Sciacca, Southern Sicily

The soil type is overconsolidated silty and sandy clay. Layers containing a considerable sand fraction are present between 11.5-m and 14-m depth. The soil data given in Fig. 8 summarize laboratory test results (24). Note: (1) The tabulated OCR values have been calculated at the four elevations assuming a unique prestress value q = 8.5 kg/cm² (17.4 ksf) (24); (2) the tabulated K_o

values were estimated using Ref. 6; and (3) the M values derived from oedometer tests have not been included in the soil data, since these values are believed

SOIL DATA

DEPTH (m)	γ (t/m³)	W	W_L	PI	% SAND	% SILT	% CLAY	C_u, (kg/cm²) (a)	$\bar{\sigma}_v$ kg/cm²	$C_u/\bar{\sigma}_v$	OCR ASSUMING q=35 kg/cm²	K_o (b)
7	2.03	25	50 to 60	30	10	55	35	6	1.05	5.7	34.3	2.7
10	AVERAGE VALUES								1.33	4.3	27.3	2.3
15	(a) FROM UNCONFINED COMPRESSION TESTS (U TESTS)								1.82	3.1	20.2	2.2
18.6	(b) THROUGH BROOKER AND IRELAND(6) FIG. 11 ENTERED WITH APPROPRIATE PI AND OCR								2.17	2.6	17.1	2.05

NOTE: GWL − 4

CONVERSION FACTORS
1 t/m³ = 62.43 lb/cu ft
1 kg/cm² = 2.048 ksf

DMT RESULTS

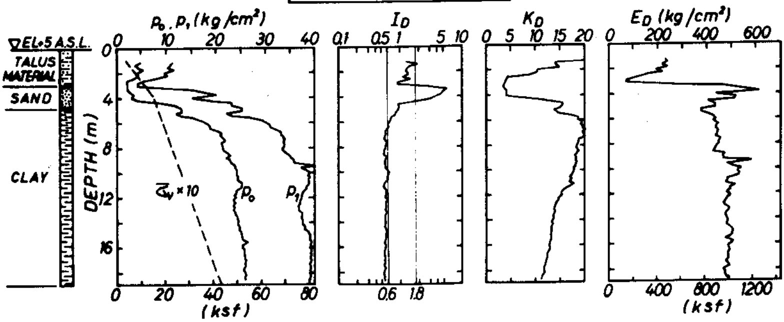

FIG. 9.—Soil Data and Dilatometer Test Results at Numana (1 kg/cm² = 98.1 kPa)

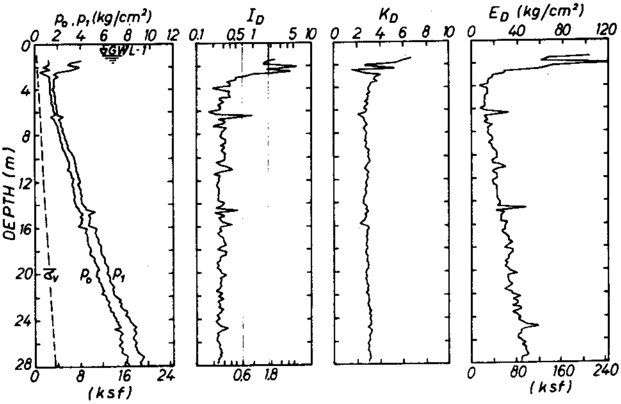

FIG. 10.—Dilatometer Test Results at Conca del Fucino (1 kg/cm² = 98.1 kPa)

to underestimate significantly in situ M, considering the relatively high silt and sand fractions.

Comments on DMT Results.—The following comments may be made:

1. The spacing between the p_o and p_1 profiles is not as small as the spacing

typical for clayey materials, nor as large as the spacing typical for sandy materials. Correspondingly, the I_D values (in the range 0.8 to 1) are intermediate between I_D values for clays and sands.

2. The value K_D is relatively high (over 20) near the surface. Then K_D decreases with depth, at a decreasing rate. A significant reduction in K_D is noted at the level of the sandy layer (OCR in this layer is presumably the same as that of the adjoining cohesive layers).

3. The increase of E_D with depth is small. Higher values of E_D are found in the sandy layer.

Test at Numana, Central Italy

The soil type is heavily OC stiff fissured marine Plio-Pleistocenic blue silty clays (7,8,11). Available quantitative geotechnical information (7) is summarized in the soil data in Fig. 9. The values of OCR given are calculated assuming a common prestress value q = 35 kg/cm^2 (72 ksf) (7). This is the most highly precompressed deposit tested. The K_o values have been estimated using Ref. 6.

Comments on DMT Results.—The following comments can be made: (1) Values of K_D at this site are the highest among all K_D values appearing in this paper; (2) in the main formation K_D decreases with depth; and (3) despite more than twofold OCR variation with depth, no noticeable I_D variation is observed in the main clay formation.

Test at Conca del Fucino, Central Italy

The soil type is normally consolidated soft lacustrine slightly organic silty clay (Fig. 10). Typical soil parameters are: γ = 1.5 tons/m^3–1.6 tons/m^3 (94 lb/cu ft–100 lb/cu ft); PI = 50–60; $(c_u/\bar{\sigma}_v)_{UU,\text{triaxial}}$ = 0.32; $(c_u/\bar{\sigma}_v)_{FV}$ = 0.48–0.55; sensitivity$_{FV}$ = 1.6–2.3; C_c = 1–1.3. These data suggest that the soil is quasi-preconsolidated. Using Bjerrum's data (3), a $(c_u/\bar{\sigma}_v)_{FV}$ = 0.48–0.55 suggests OCR = 1.7–1.8. For OCR = 1 the correlations (6) indicate $K_o \simeq$ 0.7, for OCR = 1.7–1.8 the correlations (6) indicate $K_o \simeq 0.9$.

Comments on DMT Results.—The following comments can be made: (1) The K_D values found at this site (2.8–3) are significantly higher than K_D values obtained for other NC cohesive deposits; and (2) the M values given in Ref. 10 derived from the slope of the virgin portion of the oedometer compression curve, are three to five times lower than dilatometer's E_D, in contrast to $R_M (R_M = M/E_D)$ values of almost unity for other NC cohesive deposits. These differences are believed to be due to the quasi-preconsolidation. The subject will be considered again later in the paper.

Other DMT Results

Besides the tests described previously, many others have been performed. Some indications derived from these tests are:

1. In NC cohesive deposits K_D is nearly constant with depth, in the range 1.8–2.3.

2. In OC cohesive deposits where overconsolidation has been caused by surface

erosion, K_D decreases with depth according to a characteristic profile similar to the profiles found for the overconsolidated sites described.

3. All tests have shown fairly consistently that: (a) The value $I_D > 1.8$ for sandy materials; (b) the value $I_D < 0.6$ for clayey materials; and $0.6 < I_D < 1.8$ for silty materials.

4. High reproducibility of the results and independence from operator effects have been observed whenever more than one DMT was performed at one site.

5. Tests performed in various soil types with dilatometers having different thickness (in the range 12 mm– 20 mm) and different specified deflections (s_o in the range 0.80 mm–1.30 mm) yielded values of I_D, K_D, E_D not significantly different (differences in general within 10%–20%).

6. Tests performed in two NC quasi-preconsolidated sensitive Scandinavian clays (at Ønsoy and Drammen, unpublished) have shown that: (a) The quasi-preconsolidation of the clay is reflected by relatively high K_D values (2.5–3); and (b) the index I_D is very low (I_D = 0.15–0.20 at Ønsoy and less than 0.10 at Drammen). The low value of I_D tells (see Eq. 5) that the strains induced by the penetration have almost fluidified the soil facing the membrane. In these conditions the test cannot be expected to yield meaningful results.

TABLE 1.—Relation between Grain Size Distribution and Dilatometer Material Index I_D

Site and depth, in meters (1)	Sand, as a percentage (2)	Silt, as a percentage (3)	Clay, as a percentage (4)	I_D (5)
Montalto, 40–60	2	60	38	0.20 to 0.26
Porto Tolle, 15–30	3	61	36	0.21 to 0.29
Fermo Lido, 29–35	5	55	40	0.35 to 0.40
Numana, 6–18	10	55	35	0.36
Sciacca, 17.4	14	41	45	0.85
Sciacca, 11.7	59	24	17	2
Torre Oglio, 6–10	93	7	0	5 to 5.5
Torre Oglio, 13	97.5	2.5	0	7 to 8

7. Tests performed using a large calibration chamber on large triaxial pluvially deposited dry sand specimens are described in Ref. 5. Chamber test results will be quoted specifically when relevant.

Correlations

The data used for establishing the correlations, the majority of which taken from the sites described previously, are tabulated in Table 1 and Table 2. Data relative to sensitive clays have not been considered.

Grain Size Distribution Versus I_D

The data indicate that the material index, I_D, is closely related to the prevailing grain size fraction. Table 1 shows that I_D increases rapidly as the amount of soil fines decreases. This appears true irrespective of soil stress history. For instance in the overconsolidated cohesive deposits described earlier I_D does not vary appreciably with depth, despite the marked OCR variation (a clear example is the I_D profile in Fig. 9). On the other hand, a change in soil formation is generally reflected by a discontinuity of the I_D profile.

Table 3 gives the proposed I_D based classification chart. Note that the single parameter I_D cannot provide *detailed* information on grain size distribution.

TABLE 2.—Soil Data and Dilatometer Test Results Used in Correlations

Site and symbol (1)	Depth, in meters (2)	OCR (3)	K_o (4)	$c_u/\bar{\sigma}_v$ FV (5)	U (6)	UU (7)	R_M (8)	K_D (9)	I_D (10)
Porto Tolle ○	15 to 30	1	0.51	0.30			0.85	2	0.25
Torre Oglio +	6 to 22	1	0.4				0.65 to 1 (average 0.82)	1.5 to 1.6 (average 1.55)	5 to 5.5 (average 5.2)
Montalto ●	40.7	3.1	1			0.54	1.6	4.27	0.19
	42.4	3	0.96			0.52	2.5	3.84	0.19
	42.8	3	0.99			0.51	2.1	3.78	0.18
	45.0	2.9	0.96			0.51	1.6	3.88	0.23
	46.4	2.8	0.95			0.50	1.8	3.76	0.21
	51.6	2.6	0.91			0.46	1.6	3.26	0.29
	58.2	2.4	0.91			0.43	1.3	3.80	0.28
	59	2.4	0.80			0.42	2.2	3.25	0.26
Sciacca ▼	6.8	10.2	1.7		1.98			10.3	1.19
	9.1	10.7	1.65					7.7	0.94
	11.7	8.6	1.25					4.6	2.0
	17.4	6.2	1.20			1.19		4.8	0.9
Numana ■	7	34.3	2.7					19.8	0.59
	10	27.3	2.3		4.29			17.7	0.64
	15	20.2	2.2		3.13			13.2	0.58
	18.6	17.1	2.05		2.63			11.2	0.55
Conca del Fucino ✱	4 to 28	1 to 1.8	0.7 to 0.9	0.48 to 0.55 (average 0.51)		0.32		2.8 to 3 (average 2.9)	0.25
San Salvo ◆ (*PI* = 25)	8.3					1.08		5.1	0.29
	12.2					1.02		4.7	0.59
	16.2					1.03		6.4	0.36
Cesena △ (Sensitivity = 1.5 to 2)	3			3.24				10.6	0.85
	5			1.25				4.6	0.64
	9			0.74				3.1	0.60
	10			0.80				6.1	0.50
	11			1.22				7	0.65
	12			1.12				4.1	0.65
	14			0.69				3.9	0.55
Priolo ⊖ (*PI* = 50)	11.2					1.72		11.1	0.33
	21.2					1.22		8.9	0.28
Livoro ◇ (*PI* = 30, Sensitivity = 2 to 3)	20			0.66				4.4	.21
	24			0.52				3.2	.32
	25.3	3.3	0.93			0.54	1.9	3.8	.19
	28.3	2.91	0.82			0.62	1.57	3.8	.21
Damman ×	7 to 9	1	0.4					1.4	5 to 6 (average 5.5)
Chamber Tests* ▽									

*See Table I in Ref. 5.

For instance similar I_D values have been found for 100% silts and for clays containing a small sand fraction. However I_D is a function of the mechanical consequences of the *whole* grain size distribution. The value I_D will be used as a parameter in the empirical correlations considered later in the paper.

The value I_D may also be regarded as a ratio between soil stiffness (as measured by Δp) and soil strength (as measured by $p_o - u_o$). Its wide variability is in agreement with the well known fact that soil stiffness and soil strength are relatively independent properties. No direct correlation has been found to exist between I_D and PI. Similar I_D values have been obtained in soils of different PI.

In Situ K_o Versus K_D

The horizontal pressure p_o against the side of a penetrating dilatometer must be a function, besides of the in situ σ_h, of many other parameters. No unique relationship can be expected to exist between p_o (or K_D) and σ_h (or K_o). However it was considered of interest to investigate the scatter of the correlation K_o versus K_D possibly making use of the parameter I_D also available from the test. The available pairs of values of K_o and K_D (see Table 2) have been plotted in Fig. 11(a). This figure shows that: (1) A single curve fits well all the available experimental data referring to *uncemented clays*; and (2) the material type or I_D have no discernable effect on the correlation. Note:

1. The correlation in Fig. 11(a) is based mostly on data relative to uncemented clays. This correlation is not relevant for clays that have experienced aging, thixotropic hardening, cementation, etc. In these clays K_D probably reflects, besides σ_h, the additional strength contributed by the mentioned phenomena.

TABLE 3.—Proposed Soil Classification Based on I_D Value

Peat or sensitive clays (1)	Clay		Silt			Sand	
	Clay (2)	Silty clay (3)	Clayey silt (4)	Silt (5)	Sandy silt (6)	Silty sand (7)	Sand (8)
I_D Values	0.10	0.35	0.6	0.9	1.2	1.8	3.3

2. Chamber tests on sands possessing attraction (15) indicate (5) that, for such sands, the continuous line in Fig. 11(a), entered with K_D, significantly overpredicts K_o. The value K_D seems to reflect, besides the radial stress, σ_h, actually applied to the sample, the additional strength contributed by the attraction term.

OCR Versus K_D

Having observed in many deposits a marked similarity between the OCR and the K_D profiles, the correlation OCR versus K_D has been investigated [see Table 2 and Fig. 11(b)]. It is noted:

1. The experimental points relative to *uncemented cohesive* soils, characterized by $0.2 < I_D < 2$, in conditions of simple unloading, fall within a rather narrow band, which is fairly well defined by the expression:

$$\mathrm{OCR} = (0.5\, K_D)^{1.56} \quad (8)$$

2. For cohesionless natural soils, characterized by $I_D < 2$ only three experi-

mental points are available. These data strongly suggest that a different correlation exists for cohesionless soils. Provisionally, a straight line through the available points has been drawn (dash-line) so as to provide an indication of its possible position.

3. Fig. 11(b) is not relevant with: (a) Deposits which have experienced a

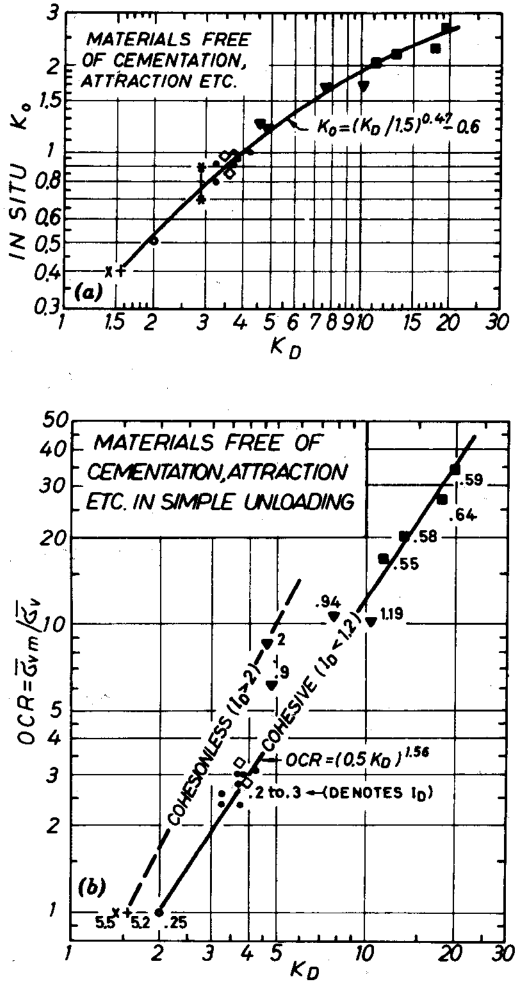

FIG. 11.—Correlation between (a) K_o and K_D; (b) OCR and K_D (See Table 2 for Site Identification)

complex stress history (so that the horizontal stresses are not those corresponding to simple unloading); (b) cemented clays (see comment 1 to the correlation K_o versus K_D); and (c) sands exhibiting attraction, where Eq. 8 considerably overpredicts the maximum past pressure really experienced by the soil (chamber tests, Ref. 5).

K_D Profile

Fig. 12 summarizes, in an idealized form, the K_D profiles found for various uncemented cohesive deposits in conditions of simple unloading. Fig. 12 evidences: (1) For NC unaged clays K_D = 1.8–2.3; (2) for OC clays uncemented and in conditions of simple unloading K_D decreases with depth; the decrease is fast near the surface, slow at greater depths; (3) the heavier is the removed overburden q, the more the profiles move to the right; and (4) as the depth increases and the clay tends to be NC, K_D tends to its typical NC values.

The shape of the K_D profile of a clay deposit, examined within the framework

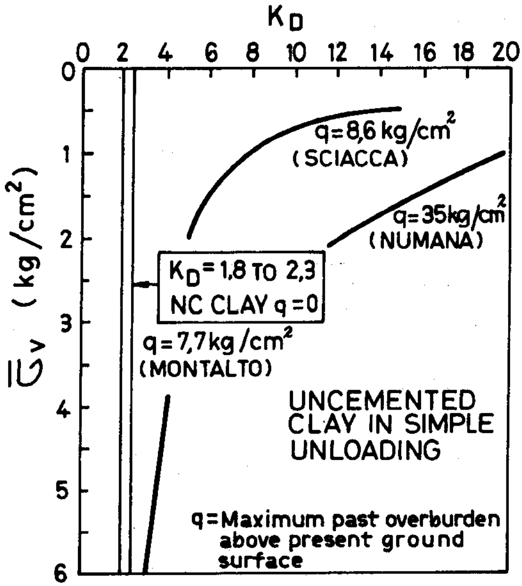

FIG. 12.—Profiles of K_d Versus $\bar{\sigma}_v$ for Uncemented Clays in Simple Unloading (1 kg/cm² = 2.048 ksf = 98.1 kPa)

provided by the profiles in Fig. 12, yields the following information concerning its stress history:

1. If K_D is almost constant with depth, and in the range 1.8–2.3, the clay is NC. If K_D is constant but higher, the clay is NC but probably aged or naturally cemented.
2. If the K_D profile is similar to the profiles labeled with values of q in Fig. 12, then: (a) The deposit is in conditions of simple unloading; and (b) an approximate value of the maximum past overburden q above present ground surface can be estimated using Fig. 12.
3. If the K_D profile does not fit with the profiles shown in Fig. 12, this indicates that either the horizontal stresses are not those corresponding to simple unloading or the clay is cemented, or both.

M Versus E_D

Having observed general agreement for a variety of soils between dilatometer's E_D and the modulus M (M = local tangent constrained modulus = $1/m_v$), the correlation M versus E_D has been investigated. The relationship M versus

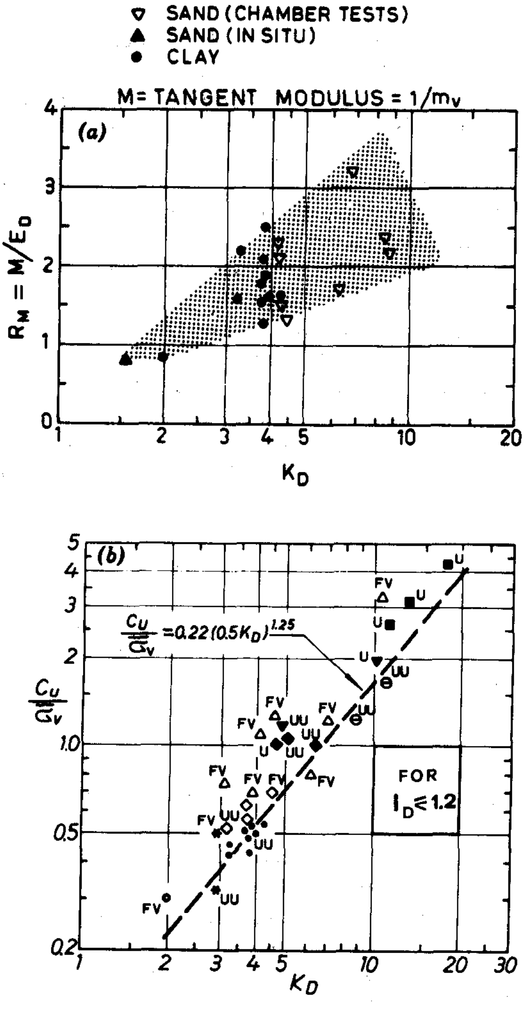

FIG. 13.—Correlation between (a) R_M and K_D; (b) $c_u/\bar{\sigma}_v$ and K_D (See Table 2 for Site Identification)

E_D must depend upon a large number of parameters, among which material type, anisotropy, pore pressure parameters, drainage characteristics, etc., and therefore no unique relationship can be expected to exist between M and E_D. On the other hand, the DMT provides, in addition to E_D, also the parameters I_D and K_D, containing at least some information on material type and stress history, respectively. The correlation M versus E_D with I_D and K_D as parameters has been investigated. The available data are plotted, in nondimensional form, in Fig. 13(a).

Comments on Fig. 13(a).—The following comments may be made:

1. There is no unique proportionality constant relating M to E_D, i.e., $R_M = M/E_D$ is not a constant.
2. The value R_M increases with K_D. The additional parameter K_D appears necessary for a proper estimation of M based on E_D.
3. The scatter in the correlation is considerable. However at least part of this scatter is probably originated by the margin of uncertainty of the M values used as reference values. The margin of uncertainty associated with the use of a correlation averaging the data points in Fig. 13(a) is probably acceptable in many practical cases, in view of the reliability of alternative methods and the accuracy currently required for M.
4. The material index, I_D, has no discernable influence on the correlation (except perhaps at low K_D values, where R_M seems to be higher for cohesionless soils). Thus within the approximation expressed by the scatter of the data points, data relative both to cohesive and cohesionless soils can be displayed in the same plot. It is possible that the present margin of uncertainty of the data points obscures the influence of I_D. The situation might change when a sufficient number of values of M of superior accuracy will be available for calibration.
5. The following equations, based on Fig. 13(a), are presently used by the writer for deriving analytically R_M from K_D

If $I_D \le 0.6 \quad R_M = 0.14 + 2.36 \log K_D$;

If $I_D \ge 3.0 \quad R_M = 0.5 + 2 \log K_D$;

If $0.6 < I_D < 3 \quad R_M = R_{M,o} + (2.5 - R_{M,o}) \log K_D$ with $R_{M,o} = 0.14 + 0.15\,(I_D - 0.6)$;

If $K_D > 10 \quad R_M = 0.32 + 2.18 \log K_D$;

Always $R_M \ge 0.85$ . . . . (9)

6. Results of chamber tests on sands exhibiting attraction fit well with the other data points (relative to deposits free of attraction or cementation). This seems to imply that to a high K_D corresponds a high R_M no matter what the origin of K_D (σ_h or attraction).
7. The reference values of M used for establishing Fig. 13(a) are local tangent values. Therefore this correlation must be expected to yield estimates of the local tangent value of M. On the other hand, it is well known that the local tangent value of M is the appropriate deformation parameter in settlement analysis only if the stress increment is small or, more in general, when the variation of M over the stress increment is small. This remark is particularly relevant for aged clays, where M suffers a substantial drop once the preconsolidation pressure p_c is exceeded. In such soils Fig. 13(a) can provide estimates of M for stresses up to p_c, but not of the much lower M values beyond p_c. An example is the Conca del Fucino site, where the M values predicted using Fig. 13(a) were several times lower than M values calculated from the oedometer virgin compression curve.

Ratio $c_u/\bar{\sigma}_v$ Versus K_D (Cohesive Soils)

The dependence of $c_u/\bar{\sigma}_v$ on OCR is well recognized. The indication of a relationship between K_D and OCR [Fig. 11(*b*)] prompted an investigation on the correlation $c_u/\bar{\sigma}_v$ versus K_D.

The available pairs of values for $c_u/\bar{\sigma}_v$ and K_D (see Table 2) are plotted in Fig. 13(*b*). The trend defined by the experimental points was compared with the relationship

$$\left(\frac{c_u}{\bar{\sigma}_v}\right)_{OC} = \left(\frac{c_u}{\bar{\sigma}_v}\right)_{NC} \mathrm{OCR}^m \quad \text{(10)}$$

in which $m \simeq 0.8$ according to Ladd, et al. (16). By combining Eqs. 8 and 9:

$$\left(\frac{c_u}{\bar{\sigma}_v}\right)_{OC} = \left(\frac{c_u}{\bar{\sigma}_v}\right)_{NC} (0.5\,K_D)^{1.25} \quad \text{(11)}$$

If, as suggested by Mesri (21), a value of 0.22 is assigned to $(c_u/\bar{\sigma}_v)_{NC}$ the dashed line in Fig. 13(*b*) is obtained.

Comments.—The following comments may be made:

1. Almost all experimental data fall within a band with a vertical amplitude corresponding to a factor of 2. Part of this scatter is probably due to the scatter of the reference values of c_u caused by different methods of testing and different amount of sample disturbance.
2. The available data do not allow grouping the points in Fig. 15 according to I_D or to test type.
3. The slope of the band occupied by the data points is in good agreement with the slope of the dash-line.
4. The dash-line gives a lower strength than the average of the experimental data and should therefore represent a fairly conservative estimate of the in situ c_u.
5. There is indication (19) that the correlation in Fig. 13(*b*) applies even to clays overconsolidated by reasons other than removal of an overburden (aging, thixotropic hardening, cementation, etc.). This would imply that to a high K_D corresponds a high $c_u/\bar{\sigma}_v$ no matter what the origin of K_D (σ_h or attraction).

Conclusions

The following conclusions may be drawn:

1. The dilatometer in situ testing device (DMT) described in the paper can be used, with the empirical correlations presented, to provide relatively inexpensive quick estimates of a number of parameters used in geotechnical engineering.
2. Results from the DMT are expressed in terms of the three index parameters which have the following significance: I_D, material index, is related to the prevailing grain size fraction; K_D, horizontal stress index, is related to in situ K_o; and E_D, dilatometer modulus, is a parameter related to soil stiffness.
3. Quantitative estimates of K_o, OCR, $M = 1/m_v$ and c_u can be obtained

from the empirical correlations with dilatometer's I_D, K_D, E_D.

4. The correlations shown in Figs. 11, 13, and Table 2 can be used with a reasonable degree of confidence in a variety of insensitive soils, within the limitations specified for each correlation.

5. It must be emphasized that Fig. 13(*a*) provides estimates of the tangent value of the modulus M. It is important to appreciate this limitation especially when significant M variations are expected over the stress increment.

6. The shape of the K_D profile of a clay deposit, examined within the framework provided by the profiles in Fig. 12, yields useful information concerning the stress history of the deposit.

Acknowledgments

The writer gratefully acknowledges the assistance and the helpful suggestions offered by the firms Icels Pali, Geotest, Germani, Rodio, and Telespazio during the in situ tests described in this paper. The writer wishes to express his gratitude to M. Jamiolkowski, who promoted the dilatometer tests at Porto Tolle, Torre Oglio, Montalto, Sciacca and made available the soil data. Conversations with A. Burghignoli, G. Calabresi, P. Colosimo, J. C. P. Dalton, F. Esu, M. Jamiolkowski, M. Ottaviani, C. P. Wroth, and many others provided valuable input to this paper. The expenses of this research were partly covered by contributions from the National Council for Scientific Research.

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IN SITU TEST BY FLAT DILATOMETER[a]

Discussion by John H. Schmertmann,[2] F. ASCE

This paper makes the rare contribution of telling the profession about an original and useful insitu testing device together with well documented examples of its use. The dilatometer test seems to offer a great deal of important soil property information using relatively simple and practical equipment and methods. On first reading this seems almost too good to be ture. However, this writer has had some personal experience using this equipment in Florida and can add a note of optimism regarding its practicality and potential usefulness.

After about 6 months of experience our company has found this equipment very practical to use in conjunction with the Dutch static cone penetration test (CPT). The same 10-ton thrust hydraulic equipment and rods used to insert the static cone easily adapt to the dilatometer. Instead of using inner rods the operator has to prethread the push rods with the plastic tubing from the dilatometer that contains the gas pressure and electrical signal lines from the dilatometer. Typically, we have found that we can make a dilatometer sounding at a rate of about 10 min/m, when taking the A and B dilatometer reading at 0.2-m. (8-in.) depth intervals. We have also found that with the 10-ton CPT equipment we can push the dilatometer from the surface through soils with standard penetration (SPT) resistance blowcount of about $N = 30$ or less to reach a layer of interest. Stronger surface soils would probably require preboring to permit dilatometer access.

The dilatometer test also offers the unique potential of permitting a rapid determination of a soil's consolidation behavior, essentially within minutes rather than the days or weeks required for undisturbed sampling and laboratory consolidation testing. Furthermore, the engineer can just as easily perform the test in soils such as silts and clayey silts as in clays—soils that often defy effective undisturbed sampling and when so sampled often produce laboratory data indicating too high compressibility. The writer has had a few opportunities to check dilatometer compressibility predictions, based on the formulas presented by the author in his paper, against laboratory measured values. Woodward-Clyde Consultants from Overland Park, Kans., Universal Engineering Testing Co. from Orlando, Fla., and the University of Florida provided the oedometer test results. Fig. 14 herein shows the comparisons. The data "points" include any uncertainty involved with either the comparative dilatometer data (primarily because of abrupt changes in test results between adjacent tests near the elevation of parallel oedometer test samples) or the laboratory data (primarily because of uncertainty as to whether to consider the soil normally consolidated or over consolidated). The reader can see that the dilatometer produced reasonable values of compres-

[a]March, 1980, by Silvano Marchetti (Proc. Paper 15290).

[2]Principal, Schmertmann & Crapps, Inc., Consulting Geotechnical Engrs., 4509 N.W. 23rd Ave., Suite 19, Gainesville, Fla. 32601.

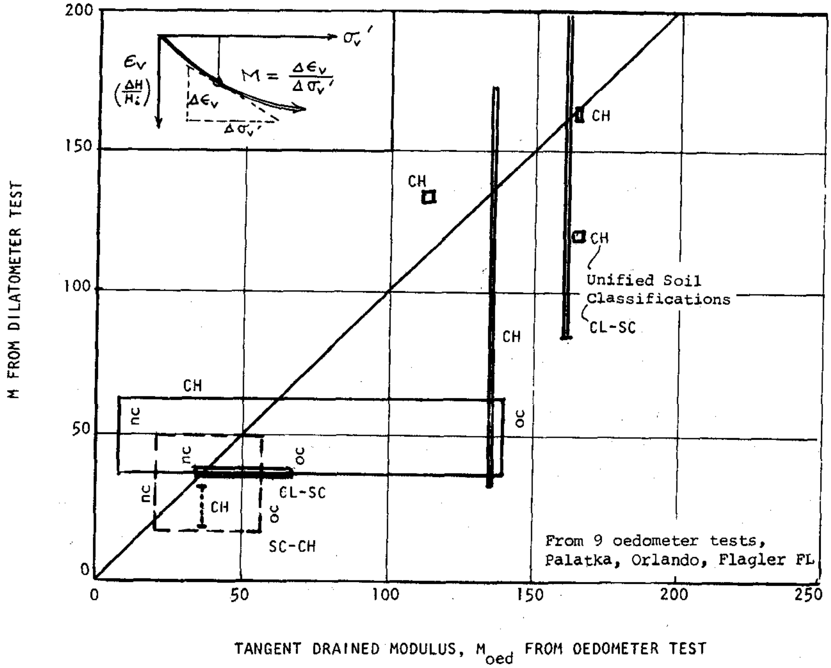

FIG. 14.—Comparisons between One-Dimensional Compressibility Predictions by Dilatometer and by Oedometer Methods

sibility that match laboratory predictions within a factor of about 2 and that on the average tend to fall on the conservative (dilatometer prediction high) side.

Based on this writer's experience to data, it appears that the author's dilatometer test may, at least in some circumstances, provide a rapid and relatively inexpensive substitute for undisturbed sampling and laboratory oedometer testing. It can test potentially compressible soils otherwise very difficult to sample properly, and provides approximately continuous data along any single dilatometer sounding.

Closure by Silvano Marchetti[3]

The writer thanks Schmertmann for the additional information he presented concerning the convenience and limitations of using the Dutch static cone penetration test (CPT) equipment for performing a dilatometer test (DMT) sounding. The writer supports Schmertmann's statements concerning the practicality of using static cone insertion equipment for dilatometer testing. Schmert-

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mann suggested an SPT blowcount penetration limit of about 30 for 10-ton CPT equipment. However, the SPT drill rig and hammer itself can drive the dilatometer into and through even higher blowcount soils. The writer has successfully tested and driven through soils with SPT N values as high as 50–70. Also, in very weak woils that will not easily support the CPT and SPT equipment, the writer has successfully used handpushed or lightweight ram-driven equipment to advance the dilatometer to depths of 6 m (20 ft). On the other extreme, the maximum test depth reached so far has been 140 m (460 ft) using the system shown in Fig. 1(*d*).

Schmertmann added to the correlation information between the dilatometer predictions of the drained oedometer modulus, M, compared to oedometer test results. He noted, for some Florida soils, conservative agreement within a factor of about 2. This factor also represents the approximate data band width in Fig. 13(*a*) in the paper. [However, the maximum deviation from a prediction line through the data points of Fig. 13(*a*) is only about ±35%.] It is encouraging to see this agreement between Italian and Florida soils and the writer expects that the data base for all the correlations discussed in the paper will expand with the expanding use of the dilatometer test.

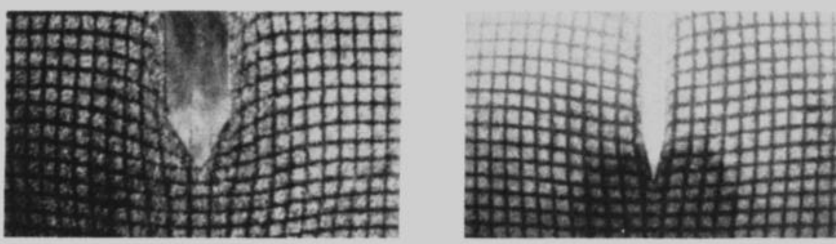

FIG. 15.—Model Test Results in Sand Showing Distorsions Induced by 36-mm Diam Conical Tip and by 14-mm Thick Blade of Dilatometer (from Ref. 28)

The writer agrees with Schmertmann that the rapid prediction of the magnitude of expected consolidation settlements within a factor of 2, compared to the more expensive and much longer times involved in obtaining oedometer test results, should in many practical cases provide an adequate substitute for at least some consolidation testing that would otherwise be required. It should prove particularly useful for preliminary settlement estimates and also offers the advantage of providing many more data points than an engineer can normally obtain by oedometer testing.

The writer would also like to respond here to several questions of general interest concerning the paper brought to the writer's attention either verbally or by mail:

1. The DMT as a penetration test.—The dilatometer test belongs conceptually and practically to the class of penetration tests and not to the class of pressuremeter tests. Penetration tests distort the insitu soil by displacement. The enforced distortion depends to a considerable extent on the geometry (and only marginally on the operator). The dilatometer, with its flat-plate shape, and with its sharp

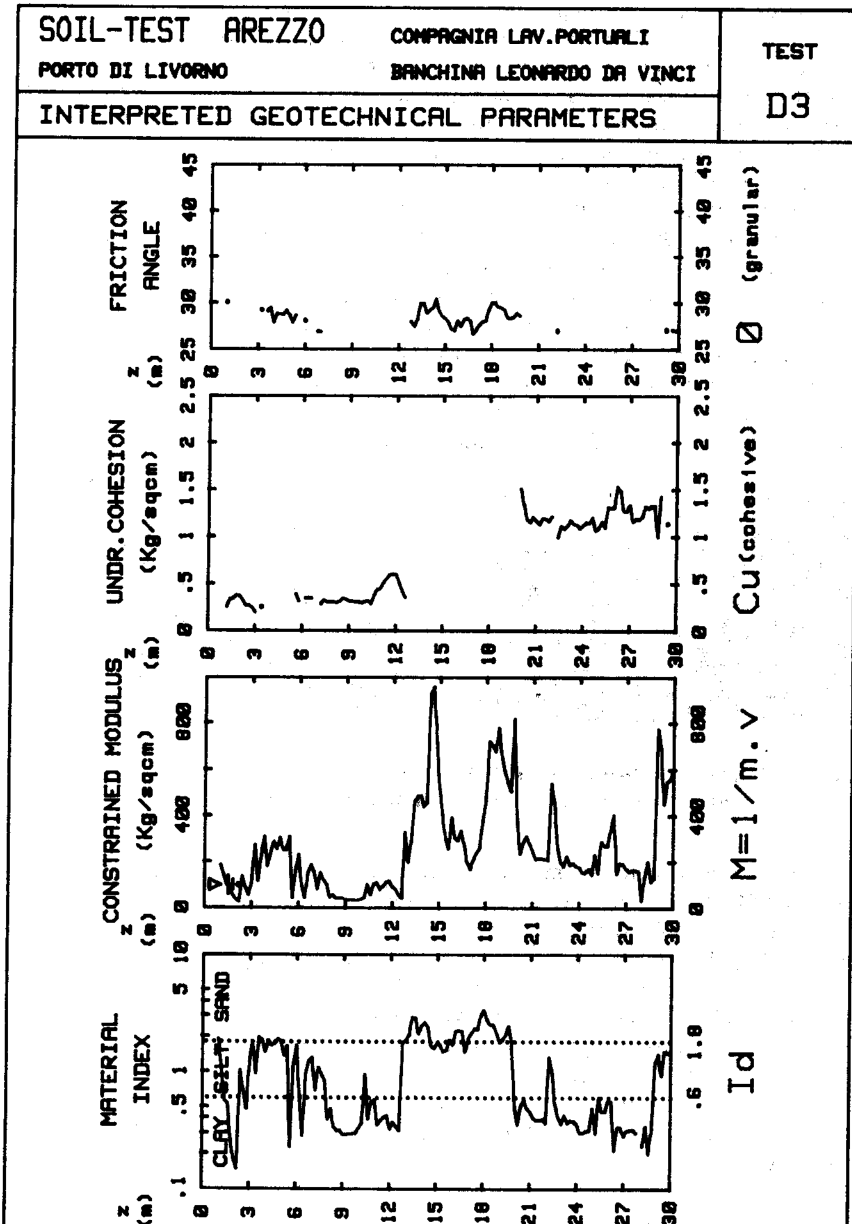

FIG. 16(*a*).—Example of DMT Results: Profiles Plotted by Computer cutting edge, should, as Fig. 3 in the paper and the added Fig. 15 of model tests in sand demonstrate, modify the original stress-strain state of the in situ soil to a lesser degree than penetration tests such as the CPT and SPT. This

SOIL-TEST AREZZO — DILATOMETER TEST N.D3 — TEST PERFORMED 4 Ottobre 1979

COMPAGNIA LAV.PORTUALI
BANCHINA LEONARDO DA VINCI

PORTO DI LIVORNO
CANALE INDUSTRIALE

Test depth: m 1 to m 30
Water table m 1.5

Po = Corrected A reading Kg/sqcm
P1 = Corrected B reading Kg/sqcm
Gamma = Bulk unit weight t/cu m
Sigma' = Effective overb. stress Kg/sqcm
U = Pore pressure Kg/sqcm
Id = Material Index (-)
Kd = Horizontal stress index (-)
Ed = Dilatometer modulus Kg/sqcm

INTERPRETED GEOTECHNICAL PARAMETERS

Ko = In situ earth press. coeff. (-)
Ocr = Overconsolidation ratio (-)
q = Eroded overburden above g.s. Kg/sqcm
M = Constrained modulus (Sigma') Kg/sqcm
Cu = Undrained cohesion (cohesive) Kg/sqcm
Fi = Friction angle (cohesionless) (-)

m	Po	P1	Gamma	Sigma'	U	Id	Kd	Ed	Ko	Ocr	q	M	Cu	Fi	DESCRIPTION
1.0	1.2	3.5	1.80	.18	0.0	1.89	7.0	85	1.5	16.6	2.7	185		30	SILTY SAND,LOW RIGIDITY
1.2	1.6	3.4	1.70	.21	0.0	1.09	7.8	66	1.6	8.4	1.6	149	.25		SILT,LOW DENSITY
1.4	2.1	3.2	1.70	.25	0.0	.56	8.5	43	1.7	9.5	2.1	100	.33		SILTY CLAY LOW CONSIST
1.6	2.2	3.3	1.70	.27	.0	.53	8.0	43	1.6	8.8	2.1	97	.34		SILTY CLAY LOW CONSIST
1.8	2.4	3.0	1.60	.28	.0	.27	8.4	23	1.6	9.3	2.4	54	.37		CLAY,SOFT
2.0	2.4	2.8	1.60	.30	.1	.18	8.0	16	1.6	8.7	2.3	35	.37		CLAY,SOFT
2.2	2.2	2.5	1.50	.31	.1	.15	7.0	12	1.5	7.0	1.9	25	.32		MUD
2.4	1.9	3.8	1.70	.32	.1	1.02	5.8	70	1.3	5.3	1.4	137	.27		SILT,LOW DENSITY
2.6	2.0	3.2	1.70	.33	.1	.68	5.6	47	1.3	5.0	1.3	89	.26		CLAYEY SILT,LOW DENSITY
2.8	1.9	2.7	1.60	.35	.1	.48	5.1	31	1.2	4.3	1.1	56	.24		SILTY CLAY,SOFT
3.0	1.6	3.4	1.70	.36	.2	1.19	4.2	66	1.0	3.2	.8	108	.20		SILT,LOW DENSITY
3.2	2.3	6.0	1.80	.37	.2	1.82	5.6	140	1.3	10.1	3.4	273		29	SILTY SAND,LOW RIGIDITY
3.4	2.0	3.8	1.70	.39	.2	.96	4.8	66	1.1	3.9	1.1	116	.25		SILT,LOW DENSITY
3.6	2.0	5.4	1.80	.40	.2	1.97	4.4	128	1.1	7.6	2.6	221		29	SILTY SAND,LOW RIGIDITY
3.8	2.5	6.8	1.90	.42	.2	1.88	5.5	159	1.2	10.5	3.9	309		29	SILTY SAND,MED RIGIDITY
4.0	2.2	5.0	1.70	.44	.3	1.46	4.5	105	1.1	4.6	1.6	180		28	SANDY SILT,LOW DENSITY
4.2	2.3	5.9	1.80	.45	.3	1.85	4.4	136	1.1	6.8	2.6	235		29	SILTY SAND,LOW RIGIDITY
4.4	2.7	6.8	1.80	.47	.3	1.67	5.3	152	1.2	7.6	3.1	286		29	SANDY SILT,MED DENSITY
4.6	2.5	6.4	1.70	.48	.3	1.74	4.6	144	1.1	6.6	2.7	255		29	SANDY SILT,LOW DENSITY
4.8	2.7	7.2	1.90	.50	.3	1.89	4.8	167	1.1	8.3	3.6	303		29	SILTY SAND,MED RIGIDITY
5.0	2.5	6.5	1.80	.51	.4	1.82	4.3	148	1.0	6.2	2.6	250		29	SILTY SAND,LOW RIGIDITY
5.2	3.1	6.7	1.80	.53	.4	1.37	5.1	136	1.2	5.1	2.2	250		28	SANDY SILT,MED DENSITY
5.4	3.1	7.6	1.80	.55	.4	1.66	5.0	167	1.2	6.9	3.2	307		29	SANDY SILT,MED DENSITY
5.6	3.2	3.8	1.70	.56	.4	.23	5.0	23	1.2	4.1	1.8	42	.39		CLAY LOW CONSISTENCY
5.8	2.8	5.4	1.70	.58	.4	1.11	4.1	97	1.0	3.1	1.2	158	.31		SILT,LOW DENSITY
6.0	2.7	6.6	1.80	.59	.5	1.69	3.9	144	1.0	4.6	2.1	230		28	SANDY SILT,MED DENSITY
6.2	3.1	4.6	1.70	.61	.5	.61	4.3	58	1.0	3.3	1.4	95	.34		CLAYEY SILT,LOW DENSITY
6.4	3.1	3.8	1.70	.62	.5	.28	4.2	27	1.0	3.2	1.4	44	.35		CLAY LOW CONSISTENCY
6.6	3.1	5.6	1.70	.63	.5	.97	4.1	93	1.0	3.1	1.3	150	.34		SILT,LOW DENSITY
6.8	3.1	6.2	1.70	.65	.5	1.24	3.9	117	1.0	3.0	1.3	185		27	SANDY SILT,LOW DENSITY
7.0	2.8	5.7	1.70	.66	.6	1.31	3.4	109	.9	2.5	1.0	157		27	SANDY SILT,LOW DENSITY
7.2	2.9	4.4	1.70	.68	.6	.69	3.4	58	.9	2.3	.9	81	.29		CLAYEY SILT,LOW DENSITY
7.4	3.1	5.8	1.70	.69	.6	1.09	3.6	101	.9	2.5	1.1	151	.32		SILT,LOW DENSITY
7.6	3.0	5.3	1.70	.70	.6	.96	3.4	85	.9	2.3	.9	122	.30		SILT,LOW DENSITY
7.8	3.0	5.0	1.70	.72	.6	.83	3.4	74	.9	2.2	.9	103	.30		SILT,LOW DENSITY
8.0	3.1	4.0	1.70	.73	.7	.39	3.3	35	.9	2.2	.9	48	.30		SILTY CLAY LOW CONSIST
8.2	3.1	4.2	1.70	.75	.7	.48	3.2	43	.8	2.1	.8	57	.30		SILTY CLAY LOW CONSIST
8.4	3.2	4.0	1.70	.76	.7	.34	3.3	31	.8	2.2	.9	42	.31		SILTY CLAY LOW CONSIST
8.6	3.4	4.2	1.70	.77	.7	.31	3.5	31	.9	2.4	1.1	44	.34		CLAY LOW CONSISTENCY
8.8	3.4	4.2	1.70	.79	.7	.32	3.4	31	.9	2.3	1.0	43	.33		CLAY LOW CONSISTENCY
9.0	3.3	4.0	1.70	.80	.8	.29	3.2	27	.8	2.1	.8	36	.31		CLAY LOW CONSISTENCY
9.2	3.3	4.0	1.70	.82	.8	.29	3.1	27	.8	2.0	.8	35	.31		CLAY LOW CONSISTENCY
9.4	3.3	4.0	1.70	.83	.8	.29	3.0	27	.8	1.9	.7	35	.31		CLAY LOW CONSISTENCY
9.6	3.3	4.0	1.70	.84	.8	.30	2.9	27	.8	1.8	.7	34	.30		CLAY LOW CONSISTENCY
9.8	3.3	4.0	1.70	.86	.8	.30	2.9	27	.8	1.8	.7	33	.30		CLAY LOW CONSISTENCY
10.0	3.4	4.2	1.70	.87	.9	.33	2.9	31	.8	1.8	.7	38	.31		SILTY CLAY LOW CONSIST
10.2	3.5	4.4	1.70	.89	.9	.36	3.0	35	.8	1.8	.7	44	.32		SILTY CLAY LOW CONSIST
10.4	3.3	5.6	1.70	.90	.9	.95	2.7	85	.7	1.6	.5	102	.29		SILT,LOW DENSITY
10.6	3.9	5.0	1.70	.91	.9	.39	3.2	43	.8	2.1	1.0	58	.37		SILTY CLAY LOW CONSIST
10.8	4.3	6.2	1.70	.93	.9	.55	3.7	70	.9	2.6	1.5	103	.44		SILTY CLAY LOW CONSIST
11.0	4.4	6.4	1.80	.94	1.0	.57	3.7	74	.9	2.6	1.5	109	.45		SILTY CLAY MED CONSIST
11.2	4.9	6.2	1.70	.96	1.0	.35	4.1	51	1.0	3.0	1.9	80	.51		SILTY CLAY LOW CONSIST
11.4	5.2	6.7	1.80	.97	1.0	.38	4.3	58	1.0	3.3	2.2	95	.55		SILTY CLAY MED CONSIST
11.6	5.4	7.2	1.80	.99	1.0	.40	4.5	66	1.1	3.5	2.5	111	.60		SILTY CLAY MED CONSIST
11.8	5.5	7.4	1.80	1.00	1.0	.42	4.5	70	1.1	3.5	2.5	118	.61		SILTY CLAY MED CONSIST
12.0	5.6	7.0	1.80	1.02	1.1	.33	4.4	54	1.1	3.5	2.5	91	.61		CLAY MEDIUM CONSISTENCY
12.2	5.1	6.5	1.70	1.04	1.1	.37	3.9	54	1.0	2.8	1.8	83	.52		SILTY CLAY LOW CONSIST
12.4	4.5	5.6	1.70	1.05	1.1	.34	3.2	43	.8	2.1	1.2	57	.42		SILTY CLAY LOW CONSIST
12.6	4.1	5.0	1.70	1.06	1.1	.32	2.8	35	.7	1.7	.7	42	.36		CLAY LOW CONSISTENCY
12.8	4.6	10.8	1.80	1.08	1.1	1.77	3.3	229	.8	3.6	2.8	329		28	SANDY SILT,MED DENSITY
m	Po	P1	Gamma	Sigma'	U	Id	Kd	Ed	Ko	Ocr	q	M	Cu	Fi	DESCRIPTION

FIG. 16(*b*).—Example of DMT Results: Numerical Output

reduces the amount of extrapolation required to estimate the undisturbed in situ behavior and thereby should improve the quality of the correlations with engineering design parameters.

2. Form of displaying results—Figs. 4–10 of the original paper express the DMT results in terms of the "intermediate parameters" I_D, K_D, E_D, because the scope in the paper was to study the correlations between these three parameters and conventional geotechnical parameters. Presently (January, 1981) the usual form in which DMT results are displayed is the one in Fig. 16. This figure shows an example of DMT results analyzed and displayed using a computer program having built in the correlations described in the paper. The reader may notice that Fig. 16 includes friction angle predictions for the cohesionless layers. However the correlations used for such predictions result from only a limited number of cohesionless deposits, not adequate at present (January, 1981) to make any general recommendations.

3. Reproducibility of results—An example of results documenting the reproducibility of the DMT results, rated as "high" in the paper, is given in Fig. 17. The DMT readings were taken in the course of two soundings, a few meters

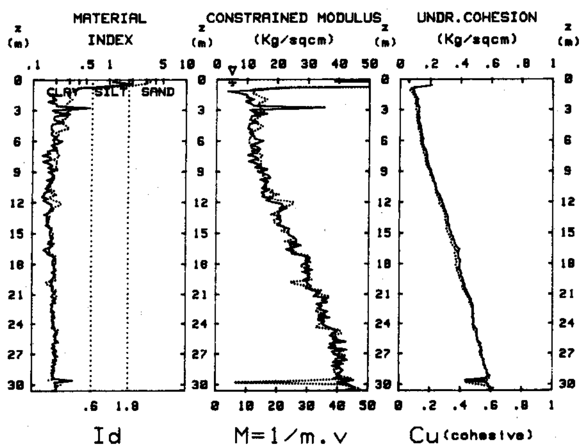

FIG. 17.—Results of Two DMT Soundings Carried out in NC Norwegian Marine Clay (Onsøy site)

apart, by 4 different operators (F. Cestari, S. Lacasse, T. Lunne and the writer) alternating with each other. The agreement appears very satisfactory.

4. In aged clays DMT overpredicts K_o and OCR—Various questions received by the writer seem to indicate that the paper did not emphasize enough that the DMT overpredicts both K_o and OCR in clays exhibiting marked aging or cementation effects. Possible reasons were discussed on pp. 314 and 315 of the paper. For instance in markedly aged NC clays OCR values predicted by the DMT are well above 1 (but, interestingly, approximately constant with depth). Conversely, DMT evaluations of OCR appreciably above 1 in a NC clay deposit may indicate possibly important aging/cementation effects.

5. Use of c_u determined by DMT—As noted in the paper, the c_u values predicted by DMT are generally lower than field vane values, with their ratio usually in the range of 0.7–1.0. The c_u values determined by DMT are comparable with the field vane c_u after reduction using the Bjerrum correction. Based on

this it has been the writer's practice when selecting the strength profile for design purposes in nonsensitive clays to use the c_u predicted by the DMT without any modification. Similar conclusions were reached by D'Antonio (27) in a recent presentation of a case history comparing profiles of c_u determined in situ using the field vane test, the CPT and DMT, and laboratory triaxial UU tests. The design profile of c_u for evaluating the stability of a 3-km long quaywall, in 13.5 m of water, was finally selected by D'Antonio as the "conservative average" of the unmodified c_u determined by the DMT.

Appendix.—References

27. D'Antonia, V., "Soil Investigations and Stability Analysis of Gravity Quaywalls," (in Italian), to be published in *Giornale del Genio Civile*, Consiglio Superiore dei Lavori Pubblici, Roma, 1981.
28. Marchetti, S., and Panone, C., "Distorsions Induced in Sands by Probes of Different Shapes," (in Italian), to be published in *Rivista Italiana di Geotechnica*, Vol. 15, 1981.

Errata.—The following correction should be made to the original paper:

Page 314, Table 3: Should be replaced by the following table

TABLE 3.—Proposed Soil Classification Based on I_D Value

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