Dilatometer Experience in Washington, D.C., and Vicinity

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For 3 years now, the flat dilatometer has been used to supplement standard soil borings and cone penetration tests during routine geotechnical investigations at project sites located within a 300-km radius of Washington, D.C. Although criticized for its mostly empirical nature, the dilatometer test (DMT) appears to provide very reasonable interpretations of soil properties in a diversity of geologic formations including residuum, marine sediments, and alluvium. Moreover, the DMT is an expedient and cost-effective method of profiling subsurface conditions, except in very dense or gravelly deposits where insertion is difficult. Several case histories are presented in support of these conclusions.

Over 160 dilatometer soundings have been completed in the vicinity of Washington, D.C., Northern Virginia, and Maryland during the past 3 years. These dilatometer tests (DMTs) were performed in conjunction with routine geotechnical studies using soil borings and cone penetration tests. Several sites are located as far north as Baltimore, Maryland, and Philadelphia, Pennsylvania, and as far south as Newport News, Virginia, and Wilmington, North Carolina. In addition to simplicity and economy of operation, the primary advantage of the DMT is a direct interpretation of engineering parameters for use in analysis.

The test involves thrusting a flat steel blade into the ground and, at predetermined depth intervals, measuring the lift-off and expansion pressures of a flexible membrane located on one face of the blade. Test procedures are described by Marchetti (*1*) and Schmertmann (*2*). The hydraulic unit on a standard drill rig is used to advance the dilatometer attached to the end of cone rods. No borehole is required, unless very dense, hard, or gravelly layers are encountered. Each test takes approximately 1 min, including measurement of the thrust, contact pressure (P_o or A reading), and expansion pressure (P_1 or B reading). Recently, the recording of the closing pressure (C-reading) of the membrane has been included, which adds another 15 sec duration to each test. For production, tests are normally taken at 0.3 m intervals. An example of DMT records taken in soft organic alluvial clay is presented in Figure 1. The fact that these soundings are about 100 m apart and were taken by three different operators indicates the repeatability of the test, as well as the uniformity of the deposit.

Much of the interpretation centers around three indices, which are calculated from P_o, P_1, the effective overburden stress (σ_{vo}'), and the ambient hydrostatic pressure (u_o). Actually, in sands, the C-reading has been shown to be a measure of u_o (*2*). The DMT indices are

$$I_D = \text{classification index} = (P_1 - P_o)/(P_o - u_o) \quad (1)$$

$$K_D = \text{horizontal stress index} = (P_o - u_o)/(\sigma v_o') \quad (2)$$

$$E_D = \text{dilatometer modulus} = 34.7\ (P_1 - P_o) \quad (3)$$

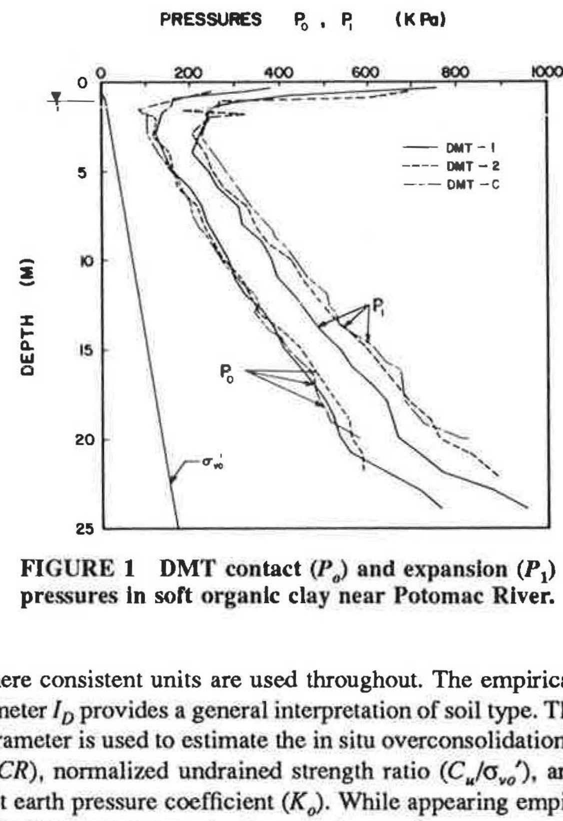

FIGURE 1 DMT contact (P_o) and expansion (P_1) pressures in soft organic clay near Potomac River.

where consistent units are used throughout. The empirical parameter I_D provides a general interpretation of soil type. The K_D parameter is used to estimate the in situ overconsolidation ratio (OCR), normalized undrained strength ratio (C_u/σ_{vo}'), and at-rest earth pressure coefficient (K_o). While appearing empirical, it has been shown that in clays, K_D is actually a measure of the normalized excess pore pressure ($\Delta u/\sigma_{vo}'$) caused during blade penetration (*3*). The modulus E_D is derived from elastic theory considerations and serves as the basis for estimating the constrained modulus ($M = 1/m_v$). In sands, a theoretical development has allowed the calculation of the effective stress friction angle (ϕ') from the thrust measurements (*4*). Details concerning DMT data reduction are given by Bullock (*5*). An example of the reduced computer output is presented as Figure 2.

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LAW ENGINEERING                                                                  TEST NO. DMT-17
FILE NAME:     CEBAF
FILE NUMBER:   NK5-1182

RECORD OF DILATOMETER TEST NO. DMT-17
USING DATA REDUCTION PROCEDURES IN MARCHETTI (ASCE,J-GED,MARCH 80)
KO IN SANDS DETERMINED USING SCHMERTMANN METHOD (1983)
PHI ANGLE CALCULATION BASED ON DURGUNOGLU AND MITCHELL (ASCE,RALEIGH CONF,JUNE 75)
PHI ANGLE NORMALIZED TO 2.72 BARS USING BALIGH'S EXPRESSION (ASCE,J-GED,NOV 76)
MODIFIED MAYNE AND KULHAWY FORMULA USED FOR OCR IN SANDS (ASCE,J-GED,JUNE 82)

    LOCATION: NEWPORT NEWS, VIRGINIA
    PERFORMED - DATE:  9-26-86
                 BY:   FROST/TROUT


    CALIBRATION INFORMATION:
    DA=   .13 BARS      DB=   .30 BARS      ZM=   .00 BARS      ZW=  1.93 METERS
    ROD DIA.=  3.70 CM    FRICTION RED. DIA.=  4.80 CM    ROD WEIGHT=  6.50 KG/M    DELTA/PHI=   .50

    1 BAR = 1.019 KG/CM2 = 1.044 TSF = 14.51 PSI         ANALYSIS USES H20 UNIT WEIGHT =  1.000 T/M3
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Z (M)	THRUST (KG)	A (BAR)	B (BAR)	ED (BAR)	ID	KD	UO (BAR)	GAMMA (T/M3)	SV (BAR)	PC (BAR)	OCR	KO	CU (BAR)	PHI (DEG)	M (BAR)	SOIL TYPE
*****	******	*****	*****	*****	*****	*****	******	******	******	*****	*****	*****	*****	*****	******	************
.61	600.	5.50	12.60	243.	1.32	48.15	.000	1.950	.110	65.43	*****	6.19		23.2	969.2	SANDY SILT
.91	600.	5.50	13.80	287.	1.58	31.28	.000	1.950	.167	38.07	*****	4.17		23.6	1026.4	SANDY SILT
1.22	600.	5.60	18.80	465.	2.63	22.38	.000	2.000	.227	24.41	*****	3.11		24.0	1518.1	SILTY SAND
1.52	600.	4.50	15.40	381.	2.68	14.34	.000	2.000	.286	10.22	35.69	2.10		26.5	1083.9	SILTY SAND
1.83	600.	2.00	3.40	35.	.49	6.10	.000	1.600	.341	1.94	5.70	1.33	.303		70.5	SILTY CLAY
2.13	600.	2.00	6.50	148.	2.24	5.13	.020	1.800	.372	1.76	4.74	.89		31.6	279.9	SILTY SAND
2.44	600.	1.20	2.00	13.	.31	3.21	.050	1.600	.393	.82	2.09	.83	.156		18.0	CLAY
2.74	600.	1.00	2.20	28.	.80	2.47	.079	1.600	.411	.57	1.39	.66	.117		30.4	CLAYEY SILT
3.05	800.	1.50	4.20	83.	1.69	3.27	.110	1.700	.430	.92	2.15	.60		34.9	118.6	SANDY SILT
3.35	1500.	5.20	15.80	371.	2.28	10.28	.139	2.000	.455	6.47	14.20	1.44		35.2	936.2	SILTY SAND
3.66	2150.	7.40	22.60	538.	2.34	13.63	.170	2.000	.486	11.35	23.36	1.81		36.3	1503.2	SILTY SAND
3.96	2700.	6.60	21.60	531.	2.64	11.26	.199	2.000	.515	7.68	14.91	1.46		38.9	1386.9	SILTY SAND
4.27	2400.	8.80	21.40	443.	1.58	14.85	.230	1.950	.545	15.30	28.08	1.98		35.9	1274.6	SANDY SILT
4.57	2200.	4.80	9.80	167.	1.08	7.79	.259	1.800	.571	4.75	8.33	1.57			374.9	SILT
4.88	1800.	5.40	7.20	50.	.28	8.69	.289	1.800	.595	5.89	9.90	1.68	.821		117.6	CLAY

END OF SOUNDING

FIGURE 2 Reduced DMT data for Norfolk and Yorktown sediments at Newport News, Virginia.

In practice, the DMT results are used to predict the behavior of foundations and embankments. Some of the common types of problems and suggested methods of analysis for these situations are summarized in Table 1. The dilatometer can provide estimates of the various required parameters. Problems involving elastic settlements, however, may require an assumed value of Poisson's ratio ($\overline{\mu}$), which fortunately does not greatly affect the calculated results. The elastic modulus $E = (1 - \overline{\mu}^2)E_D$ for undrained conditions may be obtained using $\overline{\mu} = 0.5$. In order to illustrate the applicability of the DMT, several case studies are presented in subsequent sections.

HANGAR 91, WASHINGTON, D.C.

At the confluence of the Anacostia and Potomac rivers, a thick alluvial deposit of soft organic silty clay (highly organic or "OH") extends to typical depths of 18 to 25 m. The land was reclaimed about 1910 when 1 to 2 m of sandy fill were placed to form Potomac Park as well as two military reservations. In 1941, an aircraft hangar was constructed, which added about 1 m of new fill, a 0.45-m-thick at-grade slab, and about 10 KPa floor loading. The structural columns were supported on timber piles. The floor slab has exhibited significant settlements of up to 1 m since that time (as shown by Figure 3), requiring a major reconstruction of the floor and finally, the design of a new replacement structure to be built soon near the area.

The properties of the clay have been previously characterized for several subway crossings (*14*). Typical index properties include $LL = 83$, $PI = 37$, $w_n = 68$, and $e = 1.68$. Typical standard penetration test (SPT) resistances of about 0 to 3 blows/30 cm are indicated in Figure 4. Piezocone data reported by Mayne (*3*) show cone resistances (corrected for pore pressure effects on unequal areas) increasing with depth from about 450 to 900 KPa. The stress history profile shown in Figure 4 is believed typical of undeveloped areas beneath the original fill. Apparently, based on time rate of consolidation calculated from oedometer data, the soft clay directly beneath Hangar 91 may actually still be undergoing the process of primary consolidation as a result of the imposed floor loading.

The undisturbed clay appears to be lightly overconsolidated, probably as a result of a combination of some erosion of the overburden and aging effects. The portion resulting from aging may be approximately estimated as

$$OCR = (t/t_p)\ C_\alpha/C_c \tag{4}$$

where

t = time of consideration,
t_p = time for primary consolidation,
C_α = coefficient of secondary consolidation, and
C_c = virgin compression index.

TABLE 1 APPLICATIONS OF DMT DATA

Geotechnical Problem	Required Parameters	Method of Analysis
Settlement		
Embankment	M, P_c'	Schmertmann (2)
Spread footing	E	Poulos and Davis (6)
	M	Schmertmann (2)
Structural mat	$E, \bar{\mu}$	Fraser and Wardle (7)
Pile foundation	$E, \bar{\mu}$	Poulos and Davis (8)
Undrained distortion	E_u, OCR, K_o, C_u	Foott and Ladd (9)
Stability analyses	C_u, ϕ', γ	Siegel (10)
Pile capacity	C_u, ϕ'	Poulos and Davis (8)
	K_o	Marchetti et al. (11)
Time rate of settlement	c_v	Lutenegger and Kabir (12)
Permeability	k	Robertson et al. (13)

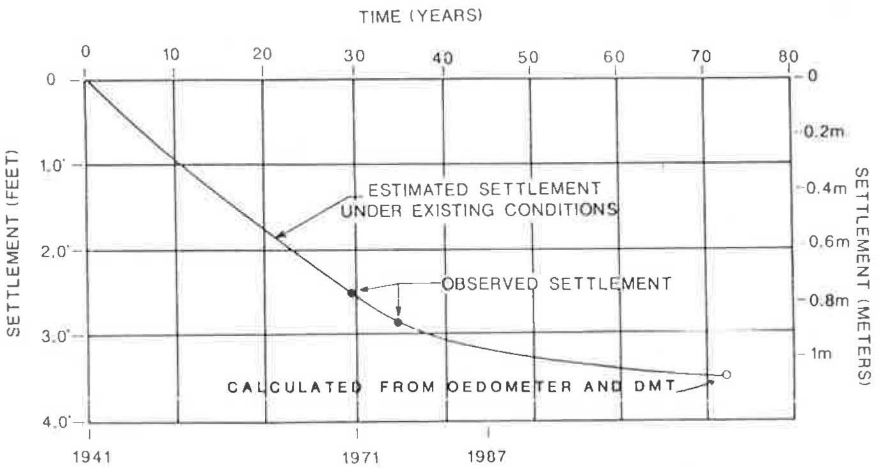


FIGURE 3 Observed settlement of Hangar 91 floor over 30-year period.

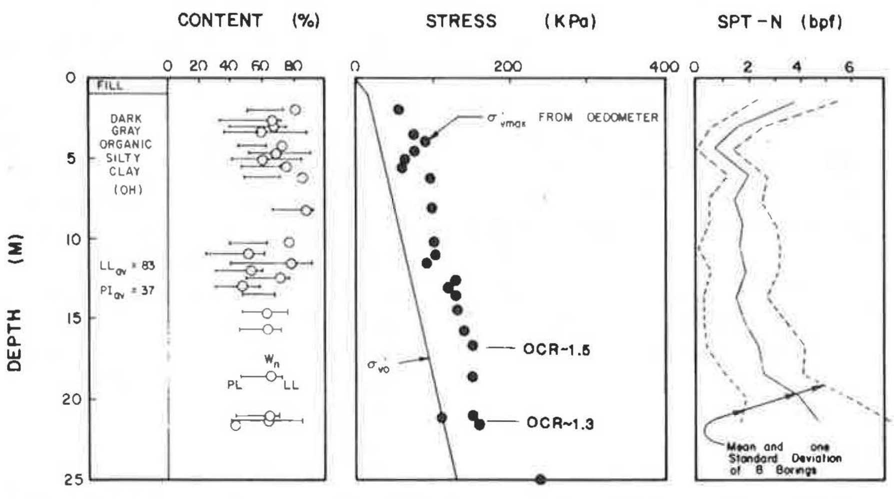


FIGURE 4 Stress history profile and index properties of Potomac River alluvium.

As shown by Mesri and Castro (15), the ratio of C_α/C_c may be assumed to be 0.05 for organic plastic clays. For a 50-year period and $t_p = 0.01$, the calculated OCR as a result of creep is about 1.5.

The profiles of OCR from 5 DMT soundings are shown to be in good agreement with standard oedometer tests conducted on thin-walled tube specimens of the clay (Figure 5). The Casagrande method of determining σ_p' was used for oedometer data. The values of constrained moduli from the DMTs are also shown to compare well from those determined from consolidation tests.

The long-term performance of Hangar 91 allows a backcalculation of soil parameters from measured settlements. An analysis of the induced stresses under the 48-by-73-m building reveals that the upper 5 to 6 m of the clay are forced into a normally consolidated state corresponding to virgin compression. In this regard, the "special case" of settlement calculation described by Schmertmann (2) is warranted. In

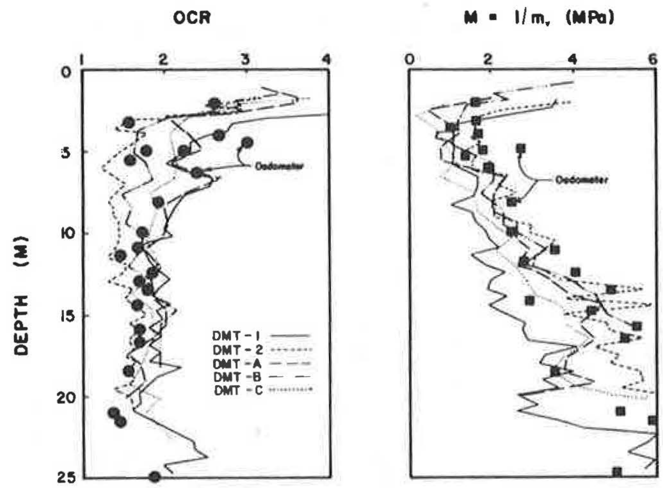


FIGURE 5 Profiles of overconsolidation ratio (*OCR*) and constrained modulus (*M*) from DMT soundings and laboratory oedometer tests.

addition, significant settlements caused by undrained distortion would be expected for loadings beyond the maximum past pressure (*9, 16*).

By definition, the constrained modulus ($M = 1/m_v$) equals the change in vertical stress ($\Delta\sigma_v$) divided by change in vertical strain ($\Delta\Sigma_v$) under conditions of one-dimensional consolidation ($\Delta\Sigma_h = 0$):

$$M = \Delta\sigma_v'/\Delta\Sigma_v \tag{5}$$

The virgin compression index (C_c) is commonly used to characterize settlements in the normally consolidated range:

$$C_c = \Delta e/\Delta \log \sigma_v' \tag{6}$$

and since strain is related to void ratio (e) by the expression $\Delta\Sigma = \Delta e/(1 + e)$, it may be derived that

$$M = (1 + e) \ln 10\ \sigma_v'/C_c \quad \text{for } \sigma_v' > \sigma_p' \tag{7}$$

The results of some 28 consolidation tests indicate a mean value $C_c = 0.83 \pm 0.25$ for $\sigma_v' > \sigma_p'$, corresponding to normally consolidated states. Consequently, the constrained modulus for the "special case" was taken as $M = 7.4\ \sigma_v'$ for calculations involving stresses that exceeded the in situ preconsolidation pressures.

Based on the DMT data, the calculations suggest an undrained distortional settlement of 0.34 m and drained settlement under primary consolidation of 0.64 m. The predicted total settlement of 0.98 m is comparable to the observed value of 0.86 m after 35 years from construction (Figure 3).

Although no dissipation tests were taken at the Potomac River site, methods of evaluating the time rate of consolidation and coefficient of consolidation (c_v) from DMT readings are now available (*12, 13*).

ACCELERATOR FACILITY, NEWPORT NEWS, VIRGINIA

The design of a new continuous-wave electron beam accelerator required stringent tolerances for differential settlement. For protective shielding, the beam tunnels would require 6-m-high earth berms constructed along the 1700-m-long facility. The project site is underlain by approximately 7 m of variable sands, silts, and clays of the Norfolk formation over preconsolidated sandy (low plasticity) clays (CL) and silts (ML) of the Yorktown formation.

The stress history profile of the site as determined from one-dimensional oedometer tests and triaxial shear tests is shown in Figure 6. In order to better define the apparent yield stress corresponding to the preconsolidation pressure, a dead weight consolidometer was modified to permit loading as high as 5000 kN/m². This ensured compression into the normally consolidated range and reduced the uncertainty of interpreting σ_p'.

The results of isotropically consolidated undrained triaxial compression tests (CIUC) were also used to determine the in situ *OCR*s. This method involves a stress history and

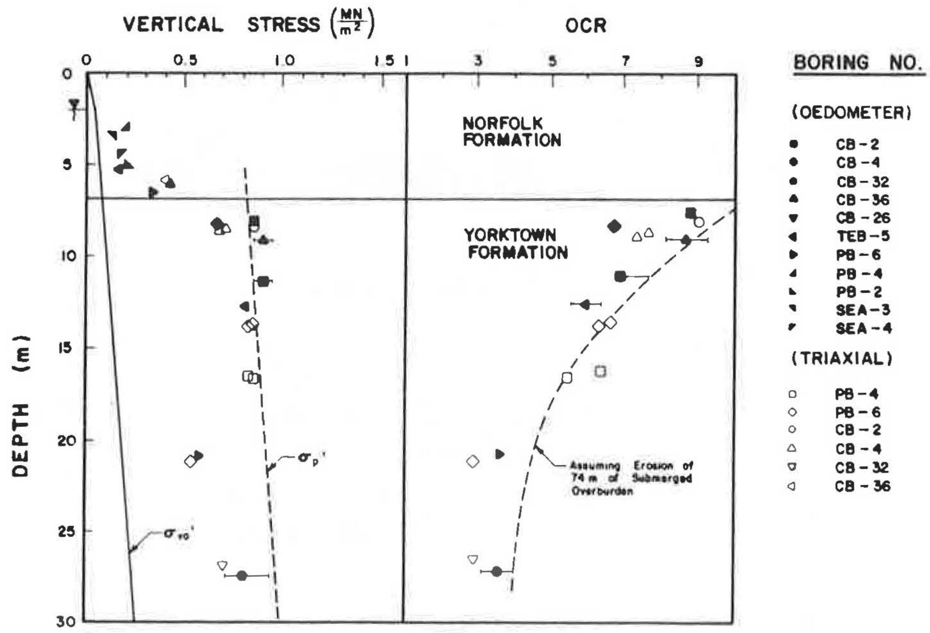


FIGURE 6 Stress history of Norfolk and Yorktown formations.

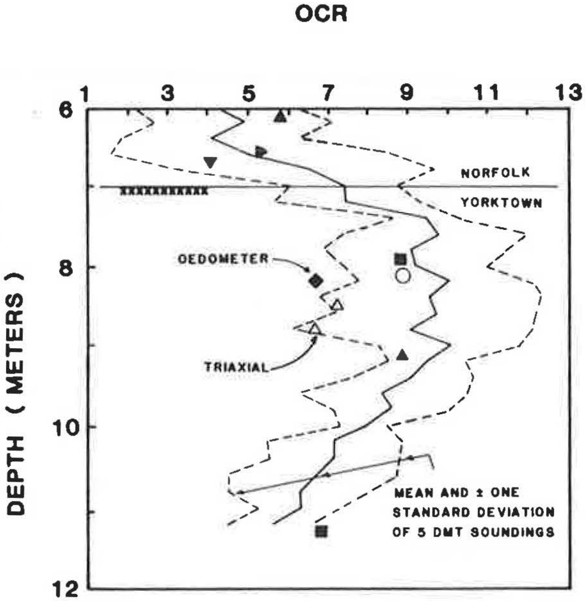


FIGURE 7 OCR profile of Yorktown from DMT data.

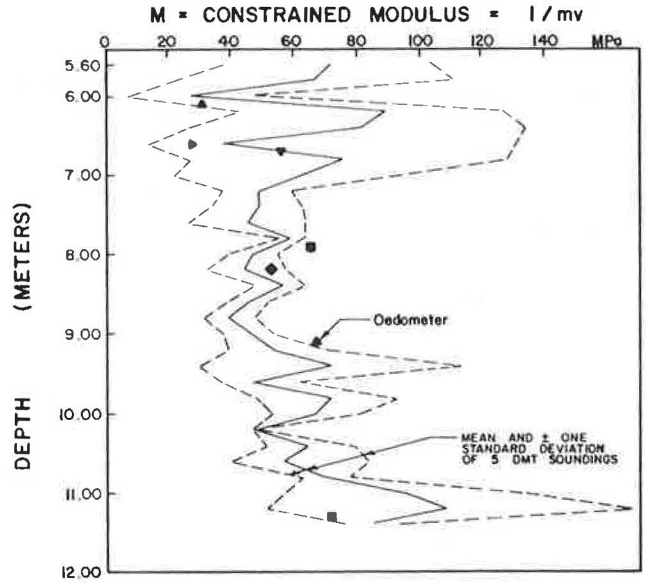


FIGURE 8 Constrained moduli of Yorktown from DMT and oedometer data.

normalized engineering parameter (SHANSEP) (*9*) data base and normalized strength parameters to determine the degree of overconsolidation (*17*). These laboratory test series imply a mean σ_p' of about 830 ± 140 KPa in the Yorktown formation. The results of DMTs show comparable profiles of *OCR* at the Newport News site (Figure 7). Although 17 DMT soundings were advanced at this site, only 5 were able to penetrate a very dense shell layer that separates the Norfolk and Yorktown formations.

The expected magnitudes of primary settlement under the proposed berm loading were evaluated using four separate approaches: oedometer, DMT, finite element method (FEM), and full-scale instrumented test berm. Conventional analyses using oedometer data suggested total settlements on the order of 5.0 cm for the area of the site designated for construction of the test embankment. The DMT sounding from this area (DMT-17 shown in Figure 2) predicted 3.7 cm for primary compression based on the established *M* values (Figure 8). An FEM study was initiated using a plane-strain model developed by Duncan et al. (*18*), which indicated a maximum settlement of 4.3 cm under the center of the embankment. Actually, values of σ_p' and *M* from oedometer tests on the sands and clays of the

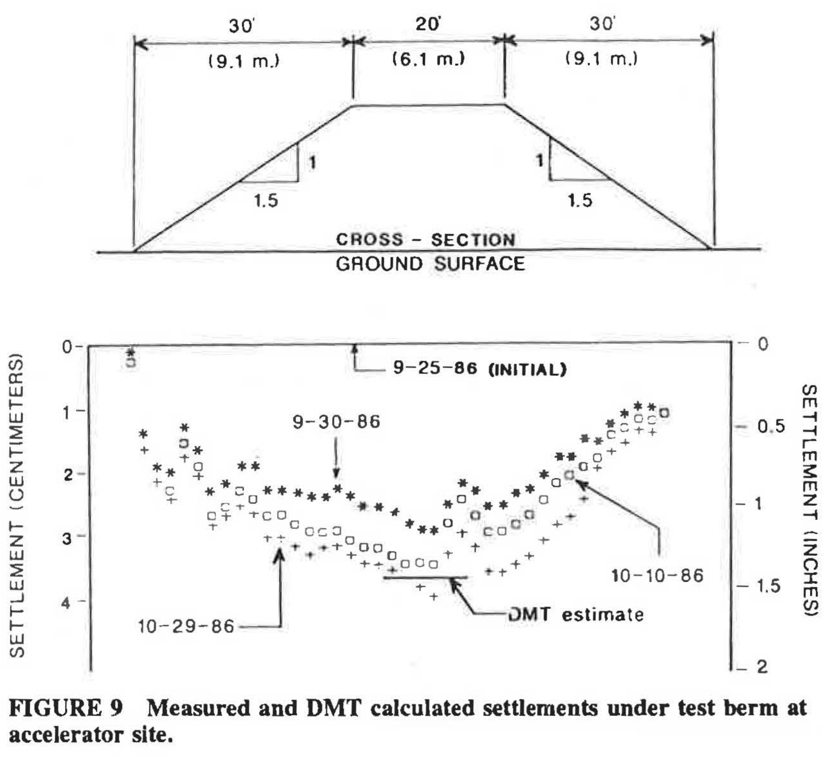


FIGURE 9 Measured and DMT calculated settlements under test berm at accelerator site.

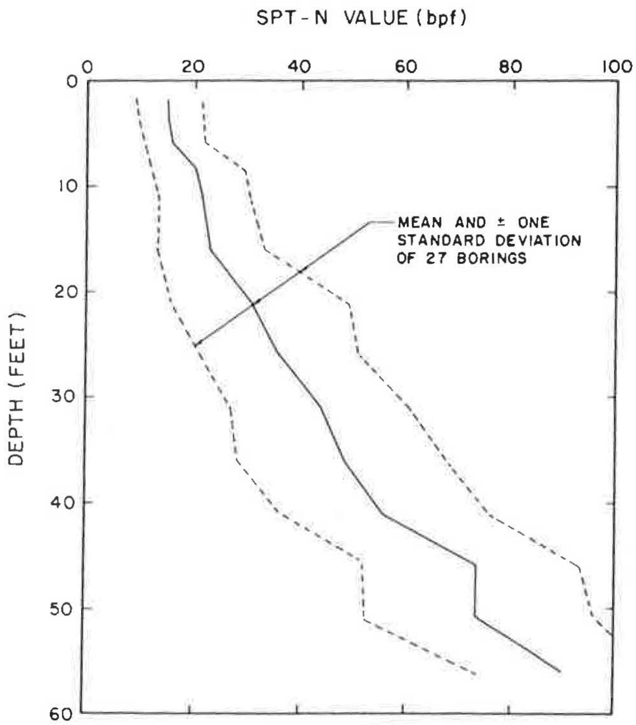


FIGURE 10 Statistical variation of SPT resistance with depth in Piedmont residuum at Fairfax Hospital.

upper Norfolk formation may have been slightly underestimated because of sample disturbance effects. Furthermore, most of these samples were not tested with the high-stress-level oedometer.

The test berm was constructed in September 1986 and instrumented with horizontal inclinometer pipe, settlement plates, borros points, pneumatic piezometers, and open standpipes. Maximum recorded settlements of 3.5 cm were observed in May 1987, although as much as 85–95 percent of all settlements had apparently occurred as recently as October 1986. The settlement profile recorded by the horizontal slope indicator along the test embankment's long axis is shown in Figure 9.

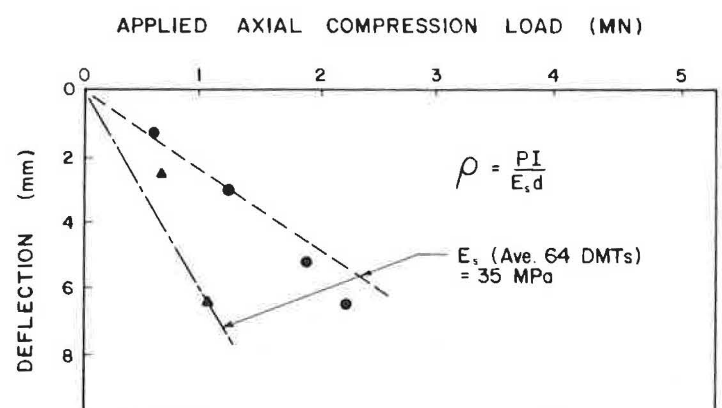
FAIRFAX HOSPITAL, FALLS CHURCH, VIRGINIA

A geotechnical exploration for a new 13-story hospital wing included dilatometer soundings, soil borings, and cone penetration tests. With design loads in axial compression of up to 9 MN, drilled shafts were selected as the appropriate foundation system, having been used for the existing main hospital wing, as well as providing minimal noise and vibration during installation.

The site lies within the Piedmont Geologic Province, which extends from Philadelphia to beyond Atlanta. The overburden soils are residuum, having been derived from the in-place weathering of the underlying metamorphic and igneous bedrock (primarily schist and phyllite at this site). In this regard, penetration test data typically increase with depth, reflecting the lower degree of weathering and saprolitic structure of the formation (Figure 10). The residuum typically classifies as a fine sandy silt (ML) and contains varying amounts of mica.

In addition to geotechnical data being collected for the new hospital wing, the results of two previous load tests were reviewed. The tests included a drilled shaft foundation (d = 0.91 m and L = 19.8 m) tested for the main wing construction in 1967 and a bored, or augered, concrete pile (d = 0.36 m and L = 12.8 m) tested for the ambulatory wing in 1977.

The results of two soundings advanced at the site indicated statistical mean values from 64 individual DMTs: I_D = 1.8, E_D = 35 MPa, and M = 74 MPa. Using the method of Poulos and Davis (*8*), the E_D modulus may be used to provide a prediction of the load-deflection response of deep foundations subjected to axial compression loading. As shown by Figure 11, reasonable predictions of the pier and pile performances are obtained with the DMT results.



SYSTEM	D	L	I_0	R_k	R_b	R_μ	I
● Drilled Pier	0.91^M	19.8^M	0.083	1.15	0.85	0.95	0.077
▲ Bored Pile	0.36^M	12.8^M	0.059	1.30	0.93	0.95	0.065

FIGURE 11 Measured and predicted load deflections of drilled shaft and bored pile at Fairfax Hospital.

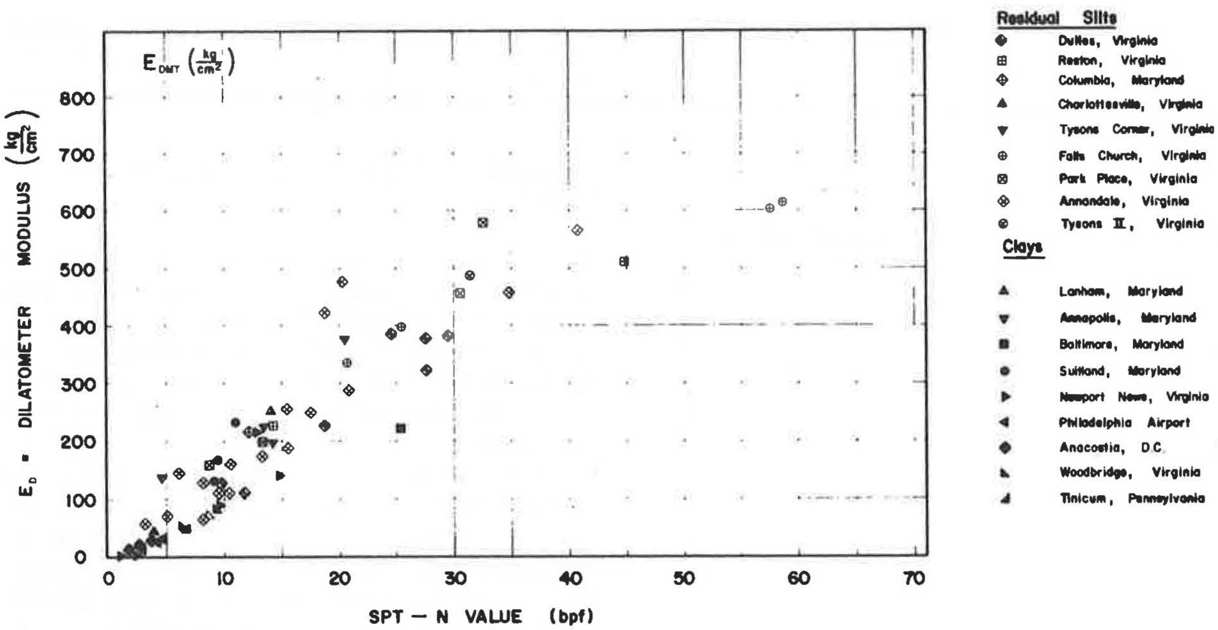


FIGURE 12 Observed correlation between dilatometer modulus (E_D) and SPT resistance in Piedmont residuum near Washington, D.C.

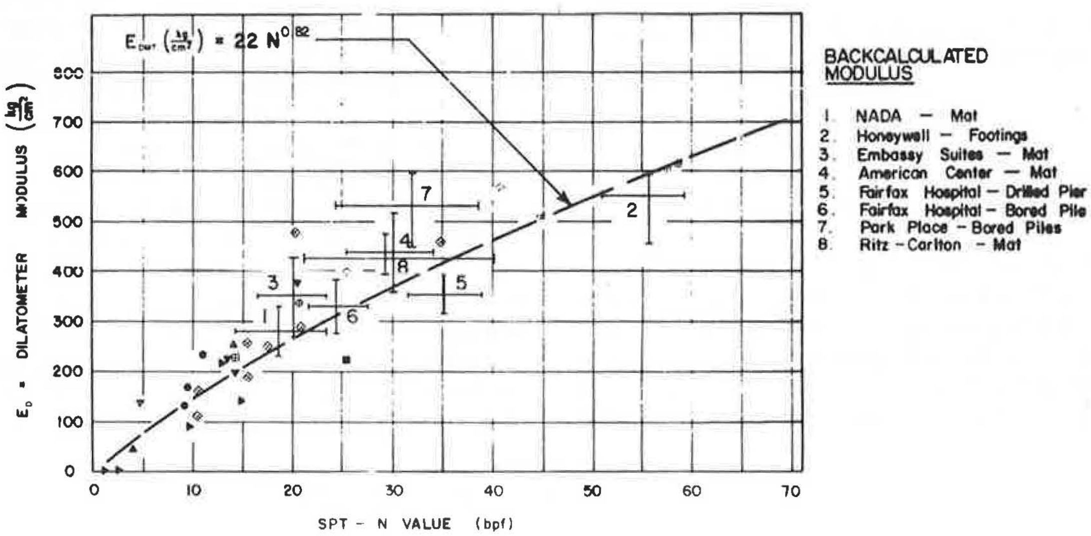


FIGURE 13 Backcalculated moduli from foundation performance data in Piedmont residuum.

ADDITIONAL SITES

Field performance data from several other projects located in Piedmont residuum have been collected by the authors and from published sources (*19, 20*). For routine use on small projects in the residual silts of the Washington, D.C., area, a correlation was developed between the dilatometer modulus (E_D) and SPT resistance because of the widespread use of the latter. Data points represent averages of E_D from DMTs and *N*-values from SPT. The observed correlation (Figure 12) bears a remarkable similarity to the mean relationship determined by Martin (*20*) between pressuremeter modulus and standard penetration tests. The DMTs suggest that for this sandy silt residuum, the local correlation is

$$E_D = 22 P_a N^{0.82} \tag{8}$$

where P_a is referenced equal to atmospheric pressure (1 bar = 1 tsf = 100 kPa). Measured settlement data from four mat foundations were reviewed and used to backcalculate soil moduli using elastic theory (*6, 7*). Although DMT data were not directly available for those particular sites, these properties are located adjacent to sites from which the E_D correlation with *SPT* was developed. Figure 13 includes data from the performance of spread footing foundations (*20*) and two additional bored pile load tests performed in Tysons Corner, Virginia. DMT data from the latter site gave good predictions of pile performance under axial compression loading, similar to those described for the Fairfax Hospital project.

CONCLUSIONS

Over 160 dilatometer test soundings have been used to supplement routine soil boring and cone penetrometer soundings in the vicinity of Washington, D.C., during the past 3 years. The DMT estimates of overconsolidation ratio and moduli compare reasonably with values obtained from laboratory oedometer tests as well as backcalculated values from field performance data.

ACKNOWLEDGMENTS

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