Flat Plate Dilatometer and Ko-Blade Correlations in the Coastal Plain in Delaware

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ABSTRACT: To design retaining walls for new interchange ramps connecting SR1/SR7/I-95 in northern Delaware several CPT, DMT and Ko-blade probes and Shelby tube samples were obtained. Construction of this wall will require cutting about 22-ft (6.7-m) into the Potomac Formation: an overconsolidated silt and clay formation. To determine the subsurface conditions including stress history, several UU and CIU triaxial compression tests and one-dimensional consolidation tests were performed. This paper discusses experience gained using laboratory test results and already published correlations for CPT and DMT tests for this geologic formation of the Atlantic Coastal Plain.

1 INTRODUCTION

1.1 *Project Description*

Traffic in the project area often experiences significant delays during peak hour and holiday travel. As part of the program to improve traffic flow the interchange connecting SR1, SR7 and I-95 will be improved. The existing ramp that connects north bound SR1 to northbound I-95 is in a cut section and it is proposed to relocate the ramp as much as 150-ft (45.7 m) to the east. To avoid encroaching excessively into the mall parking lot, retaining walls will be used to support the mall parking lot. The retaining wall to the right of the ramp will be about 2610-ft (796 m) long and will be about 18-ft (5.49 m) high. Also, to provide room to widen the south bound lanes of SR-1 another retaining wall will be built on the west side of the interchange. This wall will be 970-ft (295 m) long and 22-ft (6.7 m) high. A new flyover ramp is proposed to connect south bound I-95 with south bound SR1/7. The exit ramp from I-95 will require widening the interstate roadway to the northwest. To reduce the foot-print of the ramp retaining walls will be cut into the existing side slopes. Most of the new flyover will be structure, but a portion of it will be supported on an embankment. The embankment will be as high as 45-ft (13.7 m)

1.2 *Geologic Setting*

According to Woodruff and Thompson (1972) the project site is located in the Atlantic Coastal Plain Physiographic Province. The coastal plain consists of a wedge of sedimentary deposits that thickens to the southeast from the edge of the Piedmont. The top of crystalline rock is mapped at a depth of about 150-ft (24 m) below sea level, and dips to the southeast at about 90-ft/mile (17 m/km).

The Potomac Formation consists mostly of silts and clays with interbedded seams and lenses of sands and gravels. The Potomac Formation consists of the dark gray, maroon, and varicolored clays with micaceous sand deposited during the Cretaceous Period. This stratum consisted predominately of CL and CH with some seams of SC. The moisture content typically ranged from 16 to 26 percent, averaging 21 percent; the liquid limit typically ranged from 29 to 57, averaging 42; and the plasticity index typically ranged from 17 to 27, averaging 21. The lower portion of this formation is mostly coarse grained, but it is difficult to develop correlations across large areas. Typically, the highest elevation of this deposit is near El 100 (El 30.5 m), but about 6-miles (9.6 km) to the west of the project site deposits at El 270 (El 82.3 m) are mapped.

The Columbia Formation typically consists of varicolored silty sand and gravel deposited unconformably over the underlying Cretaceous age deposits during the Pleistocene Epoch. It is believed that this formation was deposited during the late Wisconsin or early Sangamon ages by straight to meandering, shallow but wide streams. It is not mapped in the southern portion of the interchange and is mapped as being as thick as 40-ft (12.2 m) in the northern portion of the interchange. The borings generally tended to confirm this general stratigra-

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phy. This material consisted mostly of SM and SC with some GM noted in road cuts. There were various thicknesses of fill that were typically associated with construction of the existing I-95 ramps and the nearby mall.

2 SUBSURFACE EXPLORATION

2.1 *Soil Borings and Laboratory Testing*

The field work consisted of drilling 206 Standard Penetration Test (SPT) borings, twenty-seven Cone Penetration Test (CPT) probes, twenty-five flat plate dilatometer (DMT) probes, two Ko-blade probes, and thirty-one groundwater monitoring wells. The subsurface exploration work was performed from October 2004 to March 2005. Typically, soil samples were obtained using the SPT method, but in addition several Shelby tube samples were obtained to conduct laboratory testing.

The laboratory testing consisted of consolidation tests, direct shear tests, CU-triaxial compression tests with pore pressure measurement, unconfined compression tests, and UU-triaxial compression tests. In addition, several index and classification tests were performed on Shelby tube and split spoon samples DelDOT (2005A).

2.2 *DMT Probes*

The DMT testing was performed in accordance with ASTM subcommittee 18.02 "Suggested Method for Performing the Flat Plat Dilatometer Tests". The test consisted of pushing the dilatometer blade into the soil with the hydraulic ram of a truck mounted rig. During penetration the operator measured the thrust needed to advance the blade. At the desired test depth, the operator used gas pressure to expand the membrane located on one side of the blade. The operator measured and recorded the pressure required to expand the membrane into the soil at two preset deflections. The membrane was then deflated, advanced to the next test depth and the process repeated.

Where the DMT blade could not be advanced, the DMT hole was pre-augered using hollow stem augers of a drill rig to advance through the hard zones. After pre-augering, the DMT was performed at regular intervals of about 30-cm or 1-ft to the final sounding depth.

The equipment used was purchased from GPE, Inc. and included a standard control unit having 40-bar (580-psi) capacity pressure gage and Marchetti dilatometer tip with a "hard" membrane.

2.3 *CPT Probes*

The CPT soundings were performed using a 20-ton truck mounted CPT rig. The piezocone, a 10-ton subtraction cone was pushed by twin hydraulic rams capable of developing 45-kips of down feed force and 60-kips of pullout force. Where the CPT probe could not be advanced the CPT hole was pre-augered by a drill rig.

2.4 *Ko-Blade Probes*

The Ko-Blade soundings were continuously pushed using a 20-ton truck mounted CPT rig. The Ko-blade consists of a steel blade with four thicknesses or steps of 7.5, 6, 4.5 and 3 mm. At each step is a membrane that can be inflated and it is connected to a direct reading gauge. At the test depth system the thinnest portion of the blade is inserted and the horizontal stress measured. The blade is then advanced and the horizontal stress is measured at the same depth using the next thickest step. The process is repeated for each of the four steps at a given test depth. The log of pressure is plotted against the blade thickness and the plot is then extrapolated to zero thickness. This pressure is the in situ horizontal stress.

3 TEST RESULTS

3.1 Summary of Results

Figure 1 and Table 1 compares the results from the two Ko-blade and the two closest DMT probes IDMT-9 and 10. Below a depth of about 15-ft (4.57 m) the Ko values from all four probes are in very close agreement and seem to converge on a value of about 1.0 below a depth of 20-ft (6.1 m). Assuming a φ-angle of about 15^{o} and an average OCR of about 3 this is not unreasonable based on the Jaky equation. At depths shallower than 15-ft (4.57 m) the Ko blade results indicate the Ko value is as much as twice the Ko values obtained from the DMT probes. The OCR of the soils at depths less than 15-ft (4.57 m) generally ranges from about 9 to over 100 except in IDMT-10 where there seems to be a softer zone with an OCR of about 4 near a depth of 10-ft (3.05 m). The OCR below a depth of 15-ft (4.57 m) generally declined smoothly from about 10 to about 3 or 4 with depth. In this area, the Columbia Formation was absent and the soils encountered in these four probes are thought to be the Potomac Formation.

The large OCR values near the surface can probably be accounted for by erosion, desiccation, the impact of previous construction equipment, and the effects of animals and plant roots as well as secondary effects of ageing. Figure 2b illustrates the relationship between depth below ground surface and

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the lateral stress as obtained by both the K_o blade and the DMT. As with the K_o value there is fair agreement below depths of about 15-ft. If the lateral stress is extrapolated to zero, then the estimated depth of erosion is about 40-ft (12.2 m). Using the estimated OCR values from the lower 20-ft (6.1 m) of the probes, the estimated overburden eroded away ranged from about 50 to 70-ft (15.7 to 21.3 m).

Figure 2 illustrates the relationship of undrained shear strength with elevation. The separate graphs are based on the proximity the each boring and CPT/DMT probe to each other. Figure 3 relates the Stress history with elevation and compares the results of the laboratory testing, CPT correlations and DMT correlations. Figure 4 compares the E_i elastic modulus obtained from the DMT with that obtained from the UU and CU triaxial tests.

Table 1. Ratio of Horizontal Stresses as measured by Ko-blade and DMT

Depth (ft)	Ko-9/DMT-9	Ko-10/DMT-10
1.3	1.4	10.9
1.6	2.1	3.8
2.0	4.2	3.2
4.6	3.6	1.5
4.9	3.8	2.2
11.5	3.5	2.1
11.8	1.7	2.2
14.4	2.4	1.9
14.8	2.6	2.6
15.1	3.3	No DMT
15.4	3.3	No DMT
17.4	1.1	No DMT
17.7	1.3	2.1
21.0	1.2	2.8
21.3	1.1	1.9
21.7	1.1	2.0
24.3	0.6	1.2
24.6	1.1	0.7
24.9	0.8	0.9
27.6	0.9	0.7
27.9	0.9	0.8
28.2		0.9

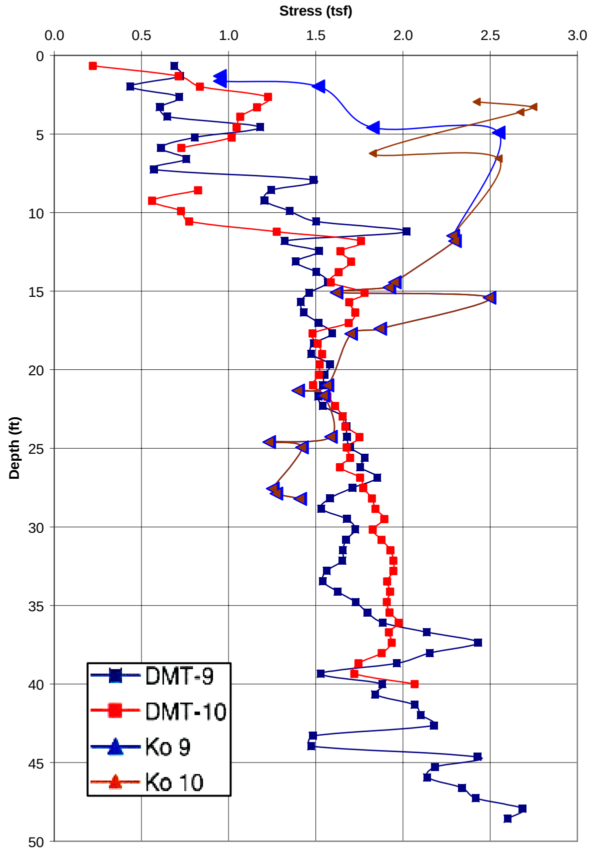

Figure 1a. IDMT – 9&10 In Situ Lateral Stress Coefficient

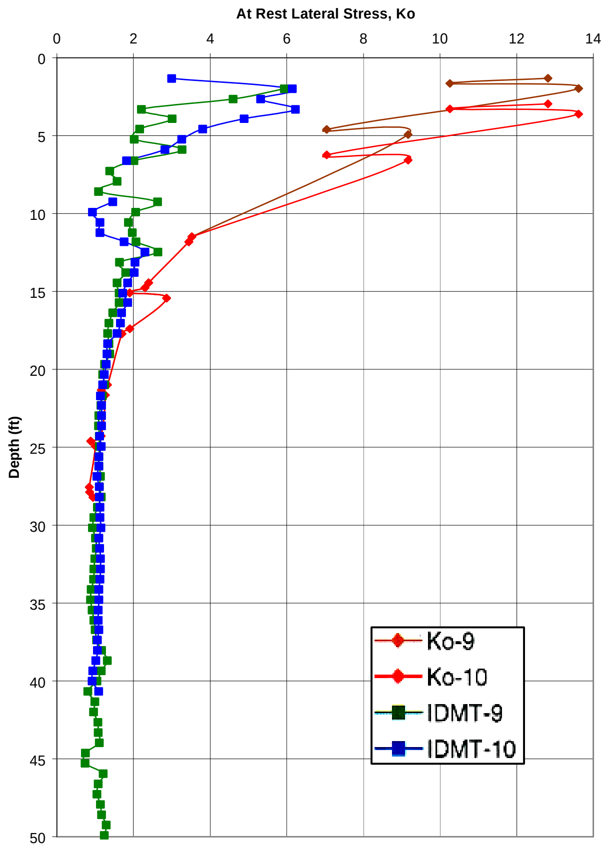

Figure1b. IDMT 9 & 10 Lateral Stresses

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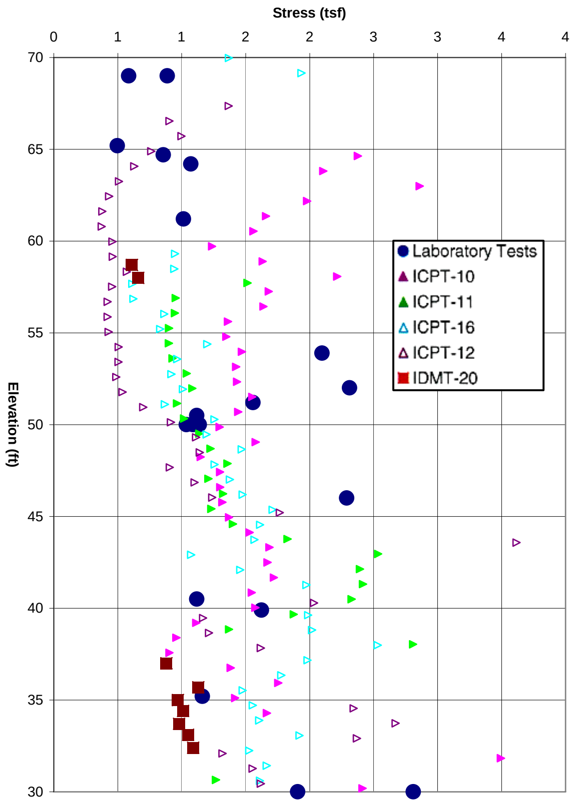

Figure 2a. IDMT-17 Undrained Shear Strength

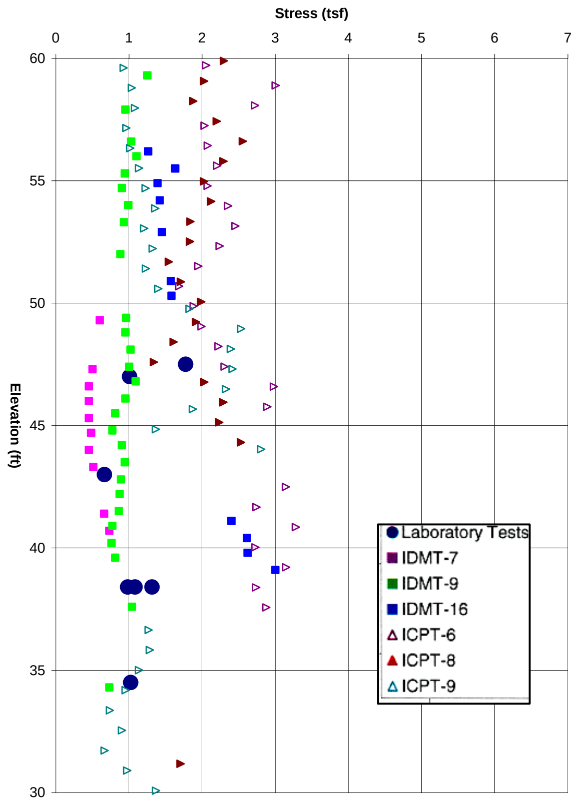

Figure 2c. IDMT-7, 9 & 16 - Undrained Shear Strength

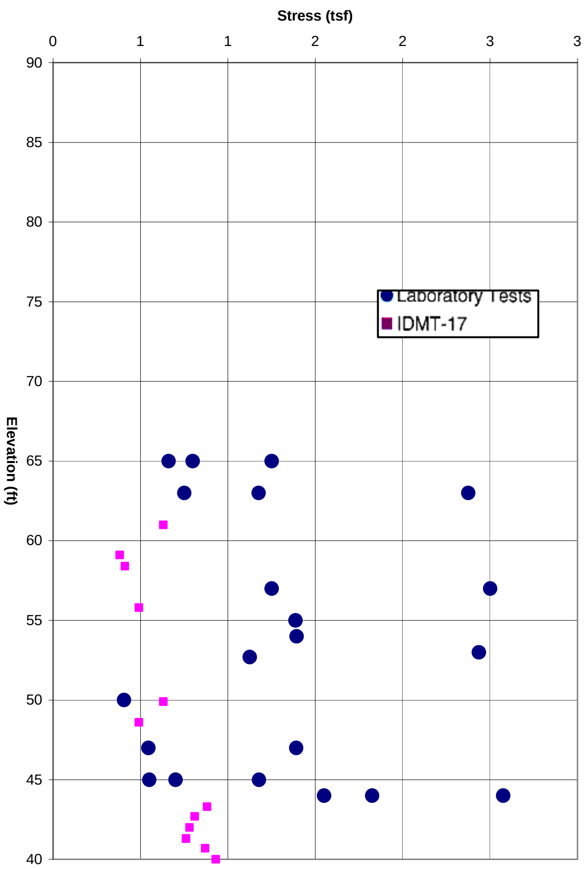

Figure 2b. IDMT-20 Undrained Shear Strength

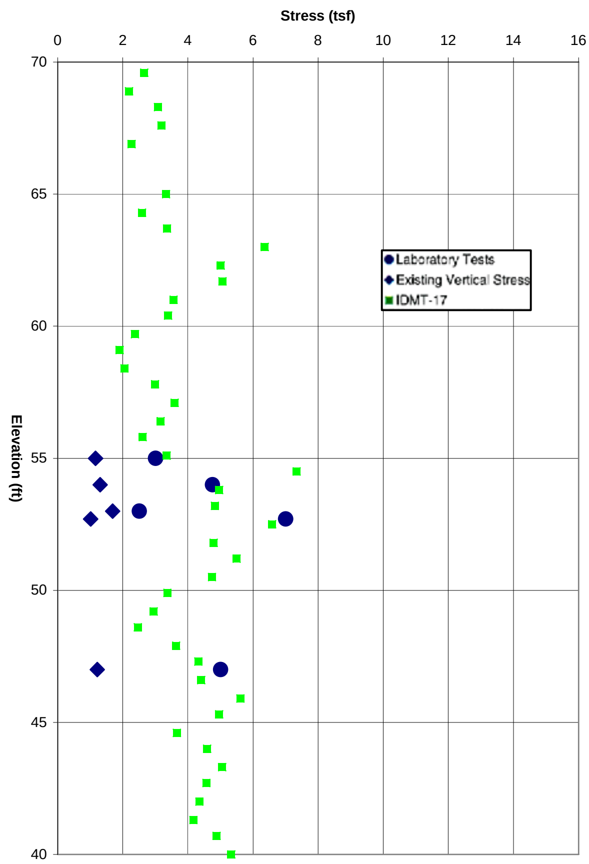

Figure 3a. IDMT-17 Stress History

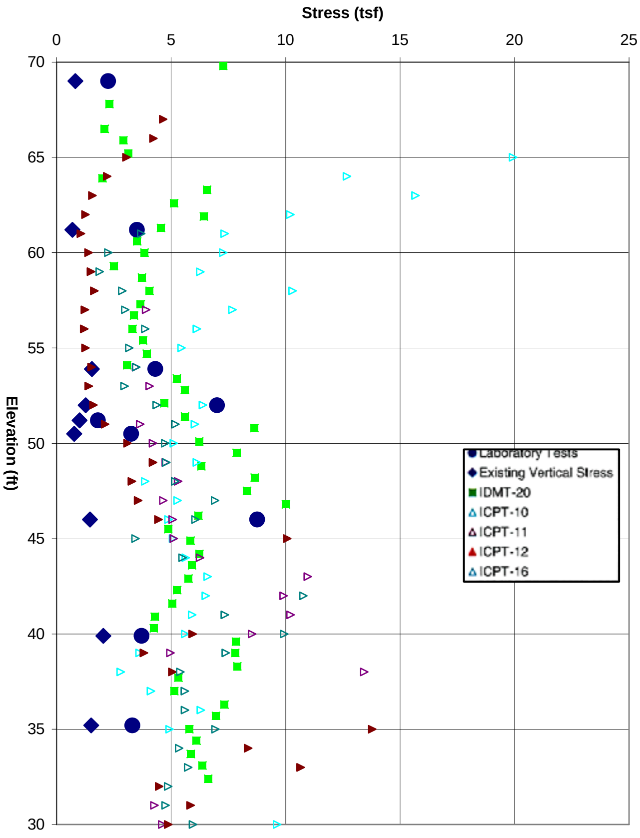

Figure 3b. IDMT-20 Stress History

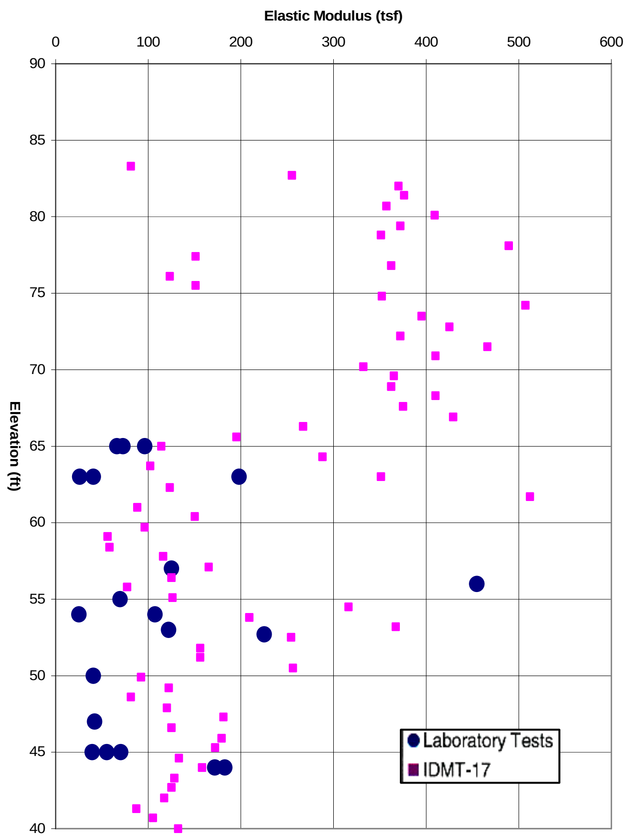

Figure 4a. IDMT-17 Tangent Modulus, E_i and DMT Modulus E_D

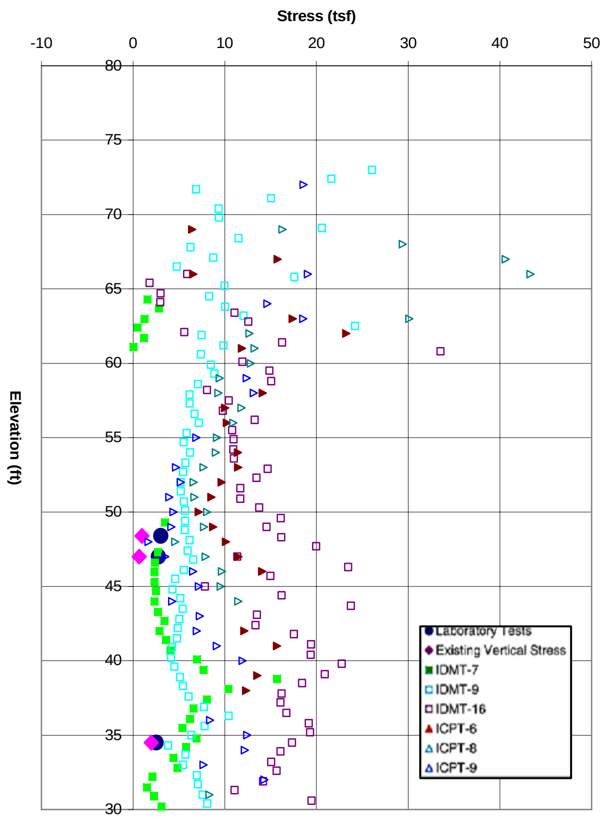

Figure 3c. IDMT-IDMT 7, 9 & 16 Stress History

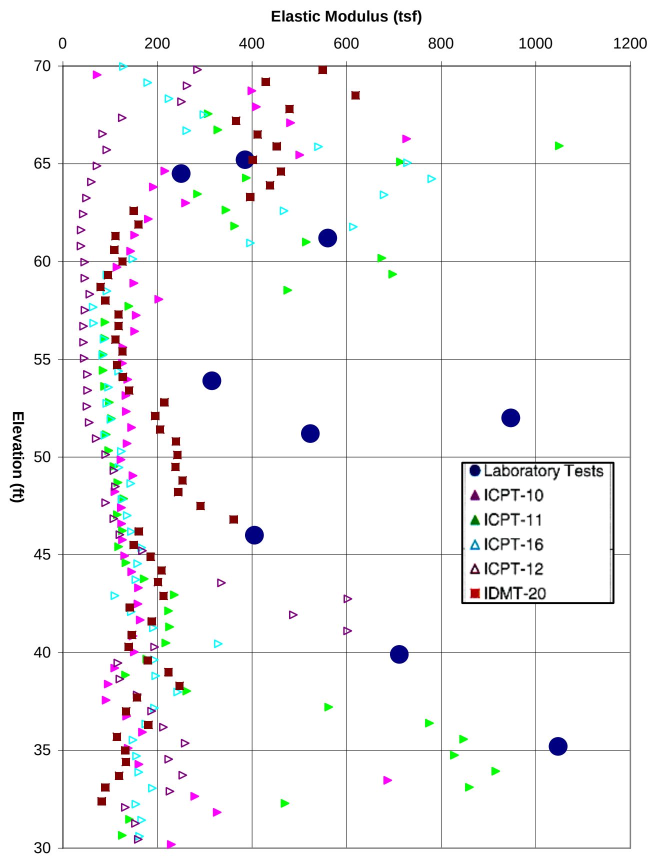

Figure 4b. IDMT-20 Tangent Modulus, E_i and DMT Modulus E_D

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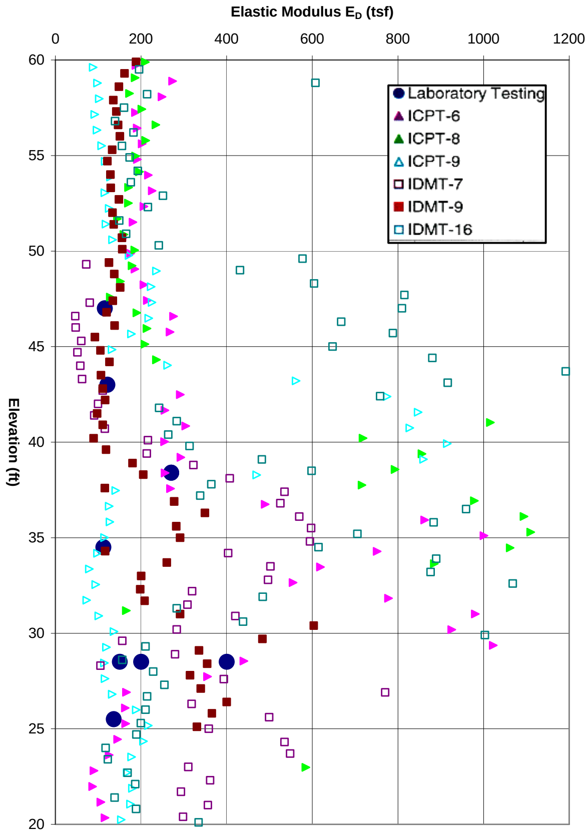

Figure 4b. IDMT-7, 9 & 16
Tangent Modulus, E_i and DMT Modulus E_D

3.2 DMT Correlations

FHWA (1992) recommends that the at rest lateral stress coefficient, K_o, for fine-grained soils be estimated from the DMT by:

$$K_0 = 0.68\ K_D^{0.54}\ \textit{for}\ \ s_u/\sigma'_{vo} > 0.8 \qquad (1)$$

$$\text{or}\quad K_0 = 0.34\ K_D^{0.54}\ \textit{for}\ \ s_u/\sigma'_{vo} < 0.5 \qquad (2)$$

The K_o on the other hand is more nearly directly measured and can be used in granular materials and not just fine-grained soils. Below depths of about 15-ft there seems to be little difference between the two methods, but at shallower depths the DMT correlations result in much smaller estimates of the horizontal stress as compared to the K_o-Blade.

Marchetti proposed the original correlation for deriving OCR from the horizontal stress index K_D from the observation of the similarity between the K_D profile and the OCR profile.

$$OCR_{DMT} = (0.5\ K_D)^{1.56} \qquad (3)$$

The above equation is in correspondence that $K_D = 2$ for OCR = 1 and has been confirmed in non cemented aging clay deposits. The Horizontal Stress Index K_D is a function of the vertical effective stress, σ'_{vo}; pore pressure, u_o and corrected A-pressure, p_o.

$$K_D = \frac{p_0 - u_0}{\sigma'_{vo}} \qquad (4)$$

The preconsolidation stress is then estimated by multiplying the OCR by the effective vertical stress.

The original correlation developed by Marchetti for determining the undrained shear strength, s_u, from DMT,

$$s_u = 0.22\ \sigma'_{vo}\,(0.5\ K_D)^{1.25} \qquad (5)$$

These correlations were found to provide consistent results for soils as shown in Figure 1, and are consistent with the laboratory test results and the results obtained from the CPT.

Two different values of elastic modulus are used, the initial tangent modulus, E_i, and the modulus at 25% of strength, E_{25}. Either E is obtained by applying a correction factor F to E_D according to the following expression:

$$E = (F)E_D \qquad (6)$$

F is a function of both I_D and K_D. Table 6.2 in FHWA (1992) presents values of F. This is not a unique proportionality constant and mostly ranges from 1 to 3, but for cohesive soils is reported to be 10 to derive E_i. Figure 4 illustrates the relationship between E_D as obtained from the DMT and the initial tangent modulus, E_i, obtained from UU and CU testing. In the figures E_i, was compared to E_D because it compared more favorably to the laboratory tests than M_{DMT}, E_{25} or other relationships as presented in FHWA (1992). There was some difficulty is obtaining an accurate initial tangent modulus from some of the laboratory tests due to some sample disturbance and settling in of the test apparatus, so some engineering judgment was used in establishing E_i. For the overconsolidated clay soils encountered an F value of 1 to less that 1 seemed to be the best fit.

3.3 CPT Correlations

The Young's modulus for clay can be estimated by using figures in FHWA (1992) which shows the variation of E_u / s_u as a function of stress level. The

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undrained shear strength must first be determined. It is often estimated using the tip resistance, q_c and the effective vertical stress σ'_{vo}.

$$s_u = \frac{(q_c - \sigma_{vo})}{N_k} \quad (7)$$

The cone factor, Nk, is empirical and it should be correlated for each project. There are also other methods to estimate s_u using the pore pressure measurements. For this project several values of Nk ranging from 10 to 18 were used estimate he undrained shear strength. For both fine-grained strata, Nk = 16 seemed to best fit the data. To estimate the OCR, the s_u must first be determined and the s_u/σ_{vo} determined. Several charts are presented in FHWA (1992).

4 CONCLUSIONS

When using in situ testing techniques such as the DMT and CPT it is very important to understand how the correlations with soil parameters are obtained. For example, nearly all the correlations depend on knowing the vertical effective stress. Although a rough guess of 125-pcf (7.8 kg/ m^3) is usually close to the actual unit weight, once laboratory testing is obtained, however, significantly different in situ test results may be obtained. It is often instructive to use a range of values of unit weights as well as other constants to establish a potential range of parameters. One of the most important factors affecting the effective vertical stress is the location of the groundwater level. The operator in the field should measure the depth to water or at least cave in at the time of testing. Groundwater levels typically change with time, so obtaining a water reading from a nearby boring or well a few days before or later is usually not sufficient, unless, of course, it is all that is available. The engineer should also be aware of the entire groundwater regime or regimes to accurately determine the existing vertical effective stress at each point of a test. Perched water can often lead to an error in estimating the vertical effective stress.

Several constants such as the cone factor for the CPT are empirical, and can be varied from site to site and even for different geologic formations on the same site. Several values should be experimented with and compared to the laboratory test data to obtain a good fit with the data.

Often using both DMT and CPT will provide a range of values that can be compared to each other. This can be beneficial in situations where good laboratory testing is unavailable or a wide range of values are obtained. One of the often overlooked benefits of using CPT and DMT is the large number of data points available. This allows the engineer to evaluate likely ranges of soil parameters and select a Factor of Safety (FS) or β-value of a risk based analysis is being used that will result in a cost effective design. The results of these tests at this site tend to support the correlations as presented, but care should be exercised by the engineer designing with in situ testing. In situ testing should not be considered a black box; it is recommended that in addition to hard copy test results, the electronic results be submitted to the engineer by the field operator. This way the engineer can plot results of different test methods and develop site specific correlations or constants using the published correlations as well as adjust the vertical effective stress to be consistent with laboratory test results.

Additional research is still required for in situ testing. Specifically, the unloading characteristics of soils are poorly understood and correlated with either the DMT or the CPT. Since a common use of either method of in situ testing is excavation support structures and retaining walls a better understanding of the relationship of the unloading characteristics would lead to more economical and safer designs for support of excavations. In urban areas and with increasing frequency in suburban area such designs are of increasing importance.

In heavily overconsolidated soils the Ko-Blade tends to provide estimates that are much larger than the DMT. At lower elevations, however, there seemed to be very good agreement with the DMT, the Ko-Blade and the Jaky equation.

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