Flat Plate Dilatometer Correlations in the Coastal Plain in Maryland

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ABSTRACT: To design the retaining wall for widening the outer loop of the Capital Beltway (I-495) several CPT and DMT probes and Shelby tube samples were obtained. Construction of this wall will require cutting about 35-ft (10.7 m) into the Monmouth and Potomac Formations: two over consolidated silt and clay formations. 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 two geologic formations of the Atlantic Coastal Plain and recommends areas for future research.

1 INTRODUCTION

1.1 *Project Description*

The traffic on the existing six-lane Woodrow Wilson Bridge has exceeded the traffic planned when the bridge was designed, so the bridge will have to be replaced. The replacement bridge will be a twelve-lane structure that will carry both loops of the Capital Beltway (I-495/95) over the Potomac River. As part of this work several interchanges need to be improved and the Capital Beltway (I-495/95) approaching the new bridge needs to be widened. The outer loop of I-496/95 near the MD 210 interchange will be widened requiring about 70-ft (21.3 m) outside the existing roadway. The roadway in this area is a cut area with side slopes of 2(H):1(V). Roughly parallel to and south of the beltway are two ramps connecting southbound I-295 with southbound MD 210 and northbound MD 210 with northbound I-295. These ramps are supported by a 15-ft (4.57 m) high Mechanically Stabilized Embankment (MSE) that is situated on top of a 2(h):1(v) slope that slopes down to the outer loop of the beltway.

To provide space to add more lanes to the outer loop, the proposed construction will consist of replacing this slope with a new retaining wall: Structure 6B. This wall will be about 1880-ft (573 m) long and will typically be about 25-ft (7.62 m) high, but the portion of the wall closest to the existing MSE will be about 33-ft (10.06 m) high. Two bridges will span over Structure 6B. Structure 1 will be a multi-span bridge that will connect northbound MD 210 with the inner loop of the beltway and Structure 2 that will be a two span bridge to provide local access to a nearby national park.

To build Structure 6B it will be necessary to use top down construction to avoid undermining the existing MSE wall supporting the two ramps of I-295. The ramps can not be closed during construction, so all construction will need to be from below the existing slope. Excavation will extend below the groundwater level; therefore, ground water will need to be controlled.

At the eastern end of the project it is proposed to replace the bridge that carries the beltway over Livingstone Road, a local road. The new bridge, Structure 4, will be wider to support the additional lanes and longer to provide better pedestrian passage under the bridge. In this area, the beltway is supported on an embankment and it is proposed to widen the embankment using a retaining wall, since there is no additional space for a wider slope.

1.2 *Geologic Setting*

According to USGS (1964) 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. The top of crystalline rock is mapped at a depth of about 600-ft (180 m) below sea level, and dips gradually. The overlying sedimentary formations dip progressively less. The formations described below are based the mapping units described in USGS (1964) and the symbols are the Washington Metropolitan Area Transit Administration (WMATA) generalized strata descriptions.

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The Sunderland Formation [T] typically consists of varicolored boulders, cobbles, gravels and silty sands deposited in stream valley and estuarine deposits that were placed during an interglacial period in the Pleistocene Epoch. Typically, the silty T1 material overlies the more granular deposits of the T2 layer. This stratum overlies the C stratum or where the C is not present the M stratumThe SPT N-values ranged from 4 to 100/3-inches, but most of the larger SPT N-values were exaggerated due to gravel and cobbles.

The Chesapeake Group [C] typically consists of dark gray to light gray, olive diatomaceous silt and clay and fine yellow sand deposited during the Miocene Epoch. In this area, it is relative thin and was not observed in all the borings. This formation consisted of CL and ML with some samples of SM and CH.

The Monmouth Formation [M] consists of very fine black sand with mica and glauconite with weathering rust-brown. This was deposited during the Upper Cretaceous Period and unconformably overlies the Potomac Group. The M material consisted predominately of CL and ML with occasional CH and SM samples encountered. In this area little C stratum was encountered and it was difficult to differentiate between the C and the M. The SPT N-values in the C/M stratum ranged from 3 to 38 bpf and averaged 13-bpf. The moisture content ranged from 12 to 43-percent and averaged 30-percent. The liquid limit ranged from 23 to 52 and the PI ranged from 4 to 25.

The Patapsco Formation and Arundel Clay [P1] is the uppermost formations of the Potomac Group. The Patapsco Formation consists of the dark gray, maroon, and varicolored clays with micaceous sand deposited during the Upper Cretaceous Period. The Arundel Clay consists of red and brown clay, and these two units are often mapped togther. The P1 stratum consisted predominately of CL and CH with some seams of SC.

There were various thicknesses of fill that were typically associated with construction of the existing I-295 ramps.

For the most part, the T-1 and T-2 were too dense for either the CPT or DMT to penetrate, so these materials were pre-augered and no in situ testing was obtained from these strata. The CPT and DMT could penetrate a fair distance into the P1, but would often encounter refusal on a dense sand layer.

2 SUBSURFACE EXPLORATION

2.1 *Soil Borings and Laboratory Testing*

The field work used to design Structure 6B consisted of drilling twenty-nine Standard Penetration Test (SPT) borings, four Cone Penetration Test (CPT) probes, five flat plate dilatometer (DMT) probes, and three groundwater monitoring wells. The SPT borings were drilled in four phases in September 2001, November 2001, April 2002 and August 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 for Structure 6B consisted of consolidation tests, CIUC-triaxial compression tests with pore pressure measurement, and UU-triaxial compression tests. In addition, several index and classification tests were performed on Shelby tube and split spoon samples PCC (2002 B and 2005A).

For Structure 4 the subsurface exploration program consisted of drilling four SPT borings. Two of the SPT borings were drilled in January 2002, and two of the SPT borings were drilled in August 2005 PCC (2002B) and PCC (2005B).

2.2 *DMT Soundings*

The DMT soundings for Structure 6B were performed in February to March 2002. The DMT probes nearby Structure 4 were performed January 2001, PCC (2002A and 2002B).

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 Soundings*

The CPT soundings for Structure 6B were obtained in two phases in October 2001 for Bridge No. 1 and again in December 2001, PCC (2002A and 2002B).The two CPT probes for Structure 4 were obtained in January 2002, PCC(2002B).

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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.

3 TEST RESULTS

3.1 *Summary of Results*

Tables 1 and 2 summarize undrained shear strength, S_u, and initial elastic modulus, E_i, as determined using the CU and UU-triaxial tests and the preconsolidation stress Pc as determined from the one-dimensional consolidation test from Structures 6 and 4 at the MD 210 interchange, respectively.

Figure 1a relates the stress history at Structure 6 with elevation and compares the results of the laboratory testing and DMT correlations. Figure 1b relates the stress history at Structure 6 with elevation and compares the laboratory test results with the CPT soundings. Figure 1c compares the stress history at Structure 4 using the laboratory test results and the CPT soundings. Figures 2a to 2c illustrate the relationship of undrained shear strength with elevation. The separate graphs are based on the proximity the each boring and CPT/DMT sounding to each other. Figure 3 compares the E_i elastic modulus obtained from the DMT with that obtained from the UU and CU triaxial tests.

Table 1. Summary of Laboratory Test Results Structure 006

Boring	Depth (ft)	USCS	S_u(tsf)	Ei (tsf)	Pc (tsf)
2-S-006-18	32	CL	1.22	235	5.5
	33	CL	1.43	400	-
	34	CL	1.79	375	-
2-S-006-19	40	CL	0.73	150	-
	41	CL	2.41	227	-
	42	CL	3.17	850	-
2-S-006-A1	29	CL	0.95	107	10
	39	ML	2	425	11
2-S-006-A3	49	CL	0.66	500	7
	69[1]	CL	2.76	135	16
2-S-006-A4	54	CL	0.96	133	5
	66	CL	3.06	345	-
	67	CL	3.62	340	-
	68	CL	4.36	350	-
	74[1]	CH	2.61	574	10

Note 1: These two samples are P1 stratum, all others are M stratum.

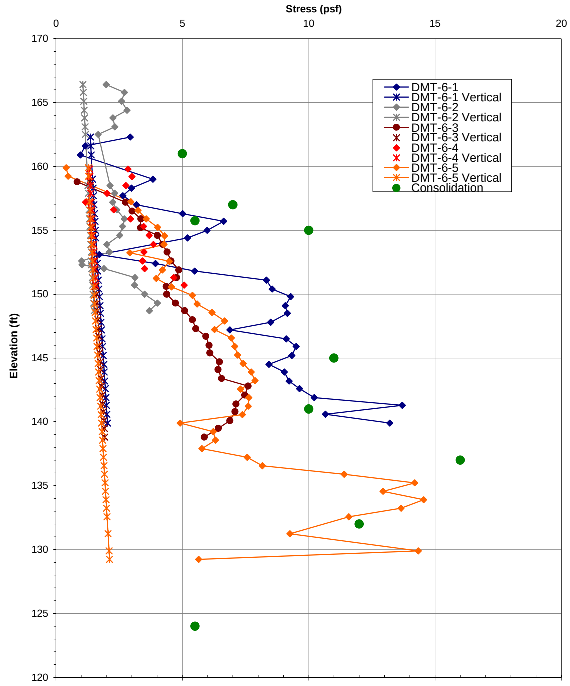

Figure 1a – Stress History (Structure 006B)

Table 2. Summary of Laboratory Test Results Structure 004

Table 2 - Summary of Laboratory Test Results Structure 4

Boring	Depth (ft)	USCS	S_u(tsf)	E_i (tsf)	Pc (tsf)
2-S-030-2	42	CL	3.54	469	
2-S-004-3	47	ML	1.55	219	
	48	ML	2.77	589	
	49	ML	3.11	539	
	53	SM	1.77	174	12
	61	SM	2.29	251	5.5
2-S-004-4	30	CL	2.43	360	
	31		2.51	485	

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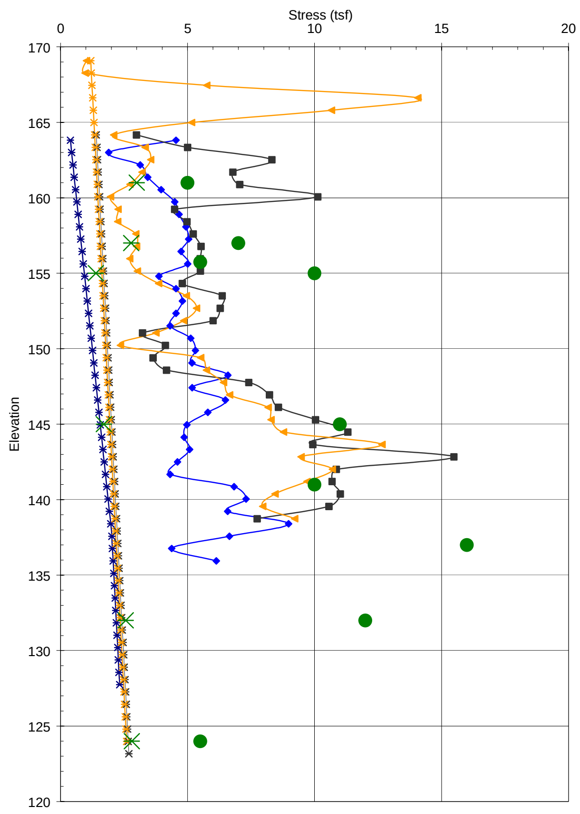

Figure 1b. Stress History CPT Results (Structure 006B)

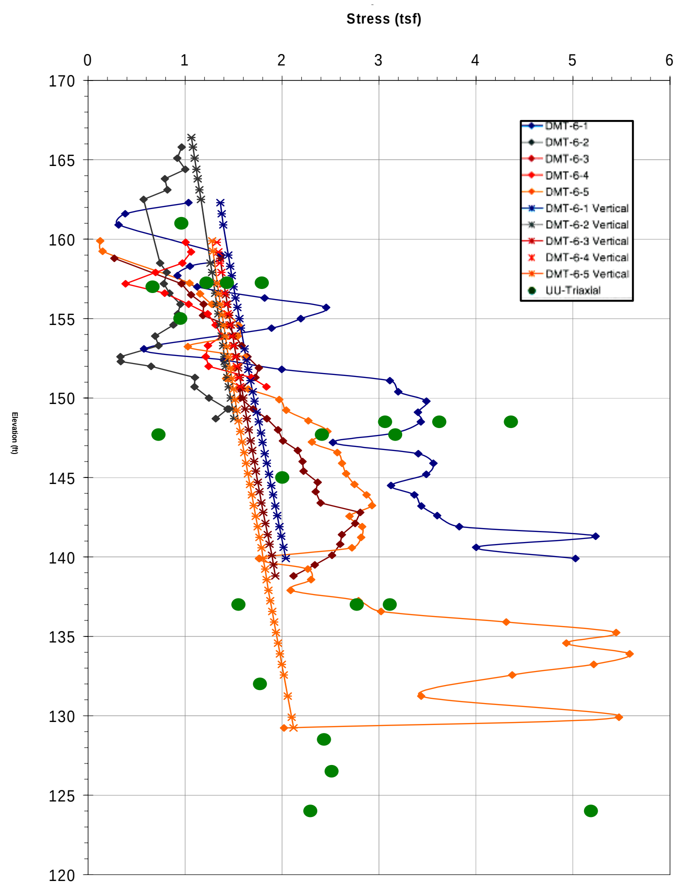

Figure 2a. Undrained Shear Strength DMT Results
STR 006B M Layer

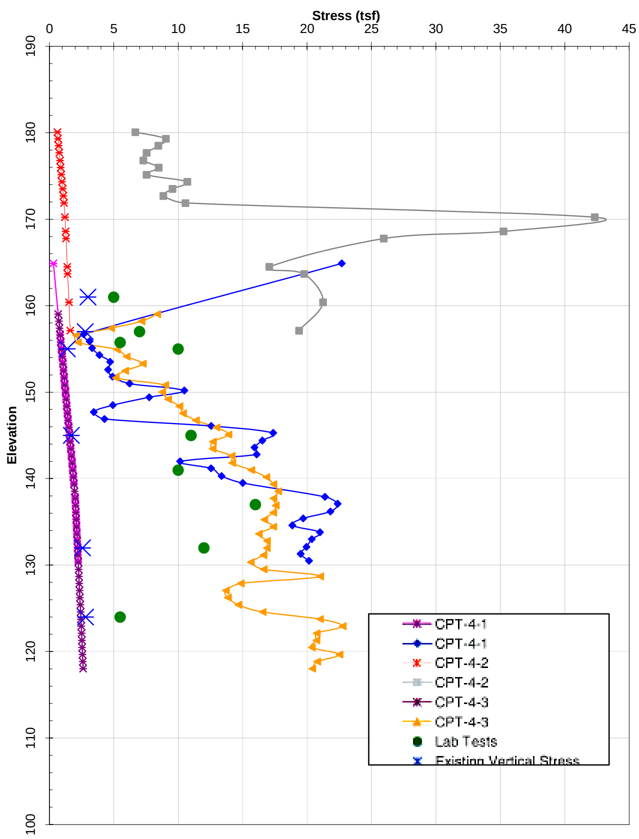

Figure 1c. Stress History CPT Results (Structure 004)

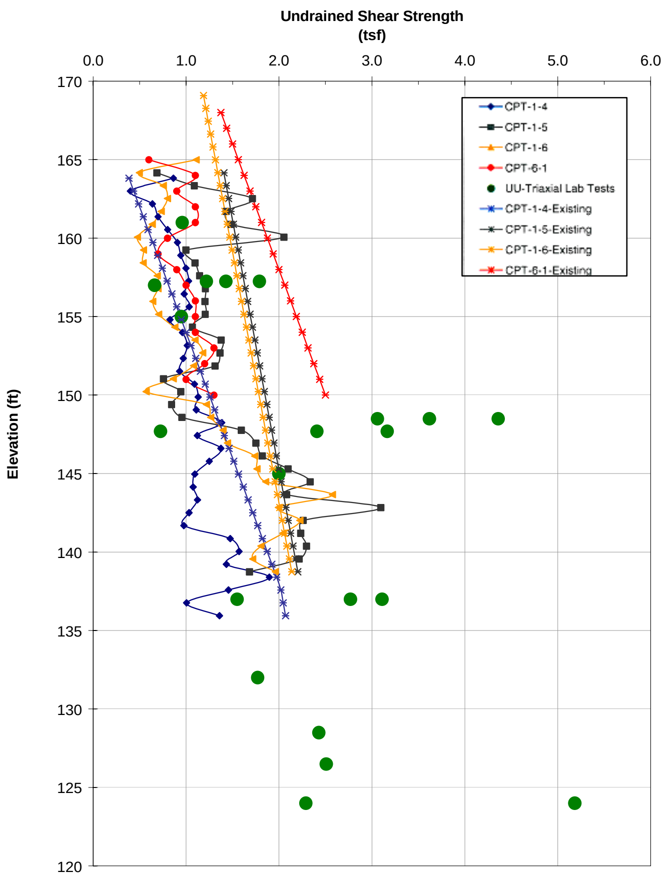

Figure 2b. Undrained Shear Strength CPT Results (Str 006)

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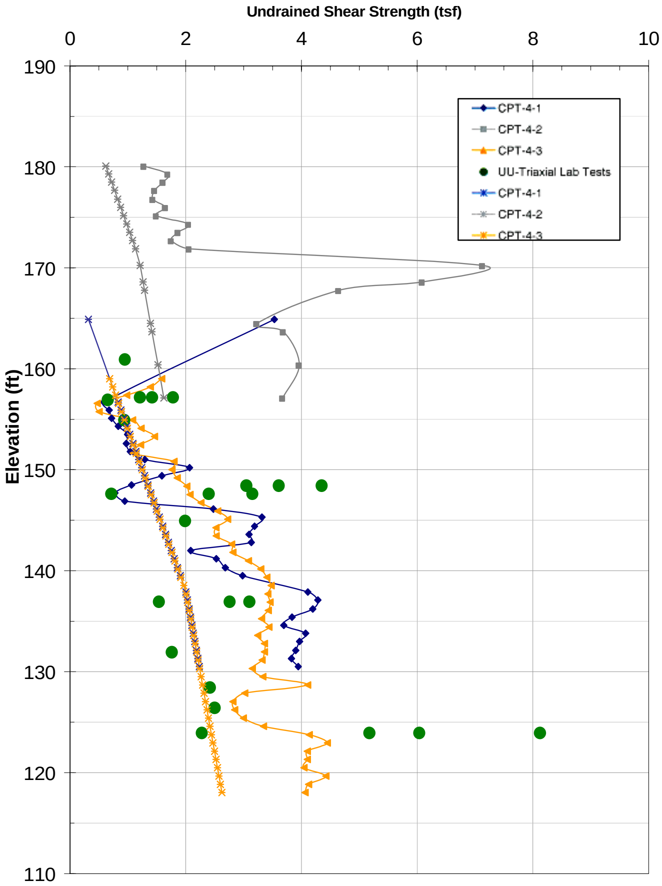

Figure 2c. Undrained Shear Strength CPT
Str 004 M Layer

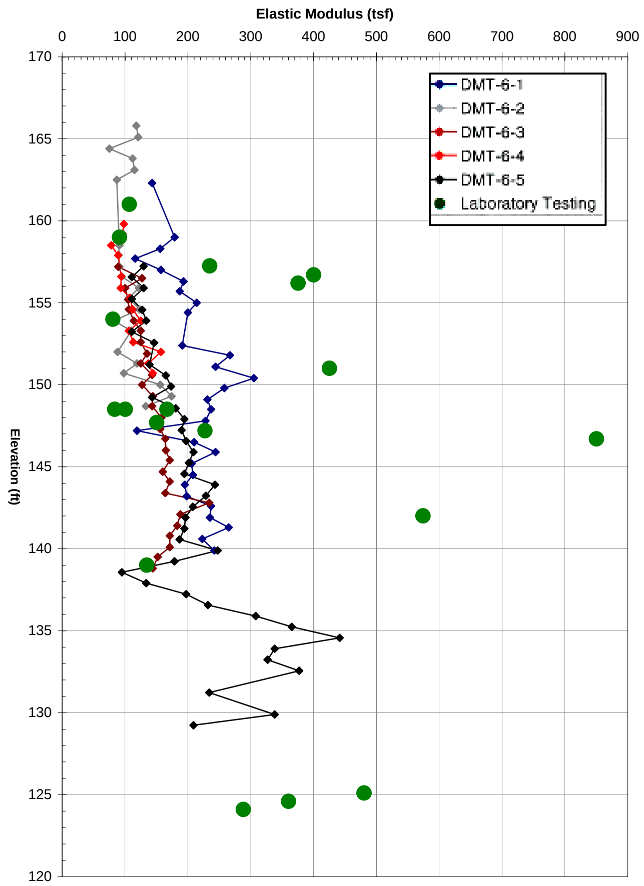

Figure 3a. Tangent Modulus, E_1 and DMT Modulus, E_D
Structure 006B

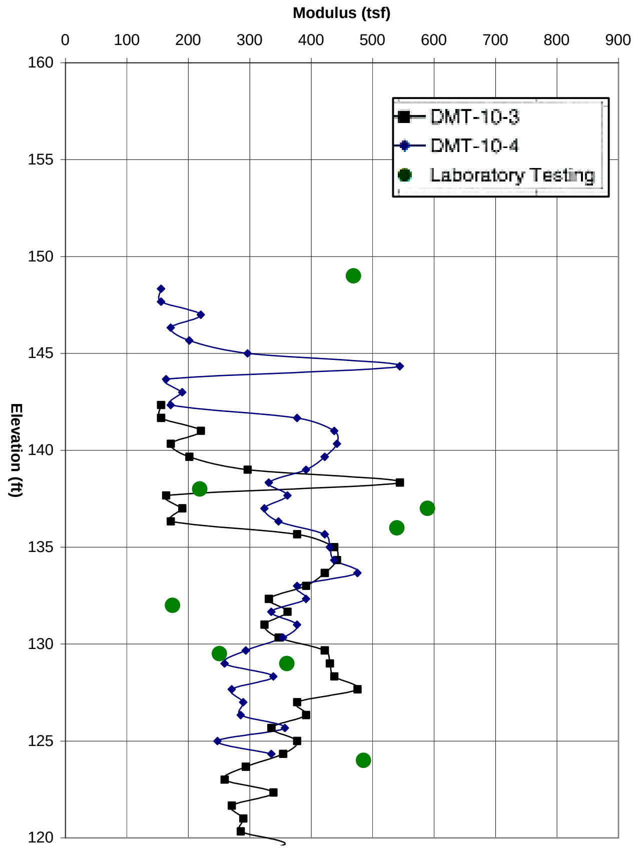

Figure 3b. Tangent Modulus, E_1 and DMT Modulus, E_D
Structure 004

In general the results of the DMT and CPT were consistent with the laboratory testing and with each other. The results were significantly improved when the CPT and DMT data were modified based on laboratory test results and more accurate groundwater readings to more accurately determine the vertical effective stress. Initially, the in situ testing operator made an estimate concerning the unit weights of the soils the groundwater regime. Once the laboratory tests were completed, the in situ parameters were re-evaluated with the updated soils information. In general, this seemed to improve the agreement between the laboratory test results and the in situ testing. In several cases, even after the in situ test results were revised, the preconsolidation estimated by the in situ tests was underestimated, but not enough to effect any engineering recommendations significantly. To estimate the preconsolidation stress from the laboratory test results, both the conventional, Casagrande method and the work-energy method (FHWA 2002) were used along with engineering judgment to reconcile the two methods (note that the axes in Figure 50 of FWHA 2002 are reversed). Several of the soil samples were disturbed slightly, and it is possible that the interpreted preconsolidation stresses from the laboratory testing might not be representative of the actual in situ conditions.

In Figure 3a, some of the modulus values are significantly larger than the in situ tests and some of the

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other laboratory tests. These results are from CIUC-triaxial tests and the results with excessively large values are from specimen with large confining stresses.

3.2 DMT Correlations

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.

$$OCR_{DMT} = (0.5\ K_D)^{1.56} \tag{1}$$

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.

$$K_D = \frac{p_0 - u_0}{\sigma'_{vo}} \tag{2}$$

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,

$$s_u = 0.22\ \sigma'_{vo}\,(0.5\ K_D)^{1.25} \tag{3}$$

These correlations were found to provide consistent results for both the M and the P1 strata as shown in Figure 1, and are consistent with the results obtained from the CPT as shown in Figure 2.

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:

$$E = (F)E_D \tag{4}$$

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

$$s_u = \frac{(q_c - \sigma_{vo})}{N_k} \tag{9}$$

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 often 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. An item 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

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point of a test. Perched water can significantly affect the estimated vertical effective stress.

Several constants such as the cone factor 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 in situ testing consultant. This way the engineer can compare and plot results of different test methods and develop site specific correlations or constants using the published correlation relationships as well as adjust the vertical effective stress to be consistent with laboratory test results.

In addition to foundation design, in situ testing is often used in the design of top down retaining walls and cut slopes. The stress paths of the soils in these conditions are significantly different from that used in the traditional and standardized UU and CU triaxial test methods. Additional correlations should be developed for such unloading conditions particularly to estimate shear strength and elastic modulus parameters. This could improve the results from numerical modeling, retaining wall design and slope stability evaluations.

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