Comparative Study of Different In-Situ Tests For Site Investigation

MD Sahadat Hossain, Assistant Professor, Department of Civil and Environmental Engineering, University of Texas at Arlington, TX 76019, email: hossain@uta.edu.
Bill Khouri, Principal, Schnabel Engineering North, Suite 700, Gaithersburg, MD 20878, email: khouri@schnabel-eng.com.
Mohamed A. Haque, Graduate Student, Department of Civil and Environmental Engineering, University of Texas at Arlington, TX 76019, email: msa_889@yahoo.com.

Abstract: In situ testing is rapidly emerging as a viable alternative to the traditional approach of obtaining geotechnical parameters required for prediction of soil bearing capacity and settlement. The diversity of the data obtained during in situ testing enables engineers to obtain a better sense of site conditions and variability, leading to more reliable geotechnical solutions. This paper presents the results of site investigation using in situ tests for a building in northern Virginia. The site investigation included pressuremeter tests, dilatometer tests, Standard Penetration Tests (SPT), cone penetration tests (CPT), and a plate load test. The objective of the current paper is to compare the bearing capacity and settlement predictions based on the different in-situ tests used for the building.

1 INTRODUCTION

The interpretations of initial geostatic stress state and stress-strain-strength-flow characteristics can be obtained with laboratory test data on high-quality samples (Mayne, 2004). However these are often done at high costs, and also the accuracy of geotechnical parameters measured from laboratory testing had been debated extensively over the last three decades. A growing awareness of this fact led to an increasing interest in all forms of in situ testing, where the disturbance of the soil structure is minimal. In situ testing is rapidly emerging as a viable alternative to the traditional approach of obtaining geotechnical parameters for design and analysis (Crawford and Campanella, 1990, Bergado et al., 1991). In recent years, some researchers have indicated the existence of a strong correlation between the predicted results from some of the insitu test methods and the observed results from the field. Bergado et al., (1991) investigated the usefulness of the screw plate and pressuremeter tests to provide meaningful results for the prediction of embankment settlement on soft clays. The settlement predictions were generally in good agreement with the observed field settlement. LeClair et al. (1999) utilized flat dilatometer, piezocone, and screw plate tests to predict consolidation settlements of embankments at Vancouver International Airport. The authors concluded that settlement magnitudes can be predicted with reasonable confidence based on the parameters interpreted from in situ tests. In this paper the results obtained from four insitu tests namely standard penetration test (SPT), cone penetration test (CPT), dilatometer (DMT), and pressuremeter (PMT) on a site for the regional jail located in Fort A. P. Hill, Virginia are presented. The objective of this paper is to compare the bearing capacity and settlement values predicted from the in-situ tests with those of observed from plate load test.

2 SITE AND PROJECT DESCRIPTION

The site for the regional jail is located in Fort A. P. Hill on the west side of U.S. Route 301, midway between the towns of Bowling Green and Port Royal in Caroline County, Virginia. Existing grades vary between EL 216 Ft. (66 m) in the northeast corner of the site to about EL 170 Ft. (52 m) along the south side. Several tributaries are located along the northern and western boundaries of the property. These ravines have relatively step slopes up to about 2.5H: 1V. Most of the site is wooded except in some areas, which were recently cleared and along the existing dirt roads. The proposed construction consisted of three housing facilities, an industries building, a food service building, a recreation center, special housing units, and an employee administration building. These buildings would consist of one to two stories with no below grade levels. In the areas where the upper portion of the natural soils is loose, the footings would be lowered. The estimated highest footing sub-grade elevations for the footings supported on natural soils at the locations of some of the borings are given in Table 1. The lowest levels of these buildings are planned at about EL 203 Ft. (62 m). The column and wall loads are not expected to exceed 30 kips (133 kN) and 6 kips (27 kN) per linear foot, respectively. Spread footings founded on natural soils of Stratum A are to be designed for a maximum allowable soil bearing pressure of 2000 psf. (96kPa)

Table 1 Estimated highest footing sub-grade elevation

Boring No.	Highest footing sub-grade elevation, ft (m)
B-7	197 (60.0)
B-22	196 (59.7)
B-24	206 (62.8)

3 FIELD INVESTIGATIONS

The field investigations were conducted by using four different in-situ tests at various locations. In order to investigate the surface conditions for the proposed development, 40 Standard Penetration Tests (SPT) were conducted. Based on the test borings and laboratory test results, the following generalized soil profile was developed for the site to the maximum depths of investigation:

Stratum A: (Chesapeake Group)	Below the topsoil to depths of 10 to 50 feet (3 to 15 m), which is the maximum depth of the borings.	Brown clayey sand (SC), silty sand (SM), and poorly graded sand (SP, SP-SM, SP-SC) with silt, clay, and clay layers, trace wood fragments, cemented sand and roots; generally very loose in the upper 6 feet and loose to firm below this depth (N = 1 to 21).
Stratum B: (Chesapeake Group)	Below Stratum A in borings B-7 and B-106 to the maximum depth of these borings.	Brown and gray elastic silt (MH) and lean clay (CL), with sand layers; generally stiff (N = 8 to 11).

In addition to the above strata, the site also contained topsoil depths of 0.1 to 0.4 feet (0.03 to 0.12 m). The soils of Strata A and B are marine deposits from the Chesapeake group of the upper Pliocene to the lower Miocene geologic ages. The site investigation in Stratum A indicated between 7.1 and 29.2 percent fines passing the No. 200 sieve. The samples were classified as clayey sand (SC) and poorly graded sand (SP-SM) per ASTM D-2487. The clayey sand material had liquid limits of 26 and 40, and plasticity indices of 8 and 25. On the basis of available information the poorly graded sand is considered to have an average moist unit weight of 115 lb/ft^3 (18.1 kN/m^3). The natural moisture content of the samples varied between 6.7 and 17.7 percent. Most of the borings indicated dry conditions except for a few borings where the ground water level varied between 3.0 to 33.5 feet (0.9 to 10.2 m) below the existing grades. High ground water was observed only in the low lying areas of the site.

However, only three boring locations were selected for this study, since all the in-situ tests were performed in close proximity to these three borings. Figure 1 shows the site plan and the locations of borings B-7, B-22, and B-24 at which all the four in-situ tests were done, and also B-16 where the plate load test was done. The results from each of these in-situ test methods at each boring location are discussed in the following sections.

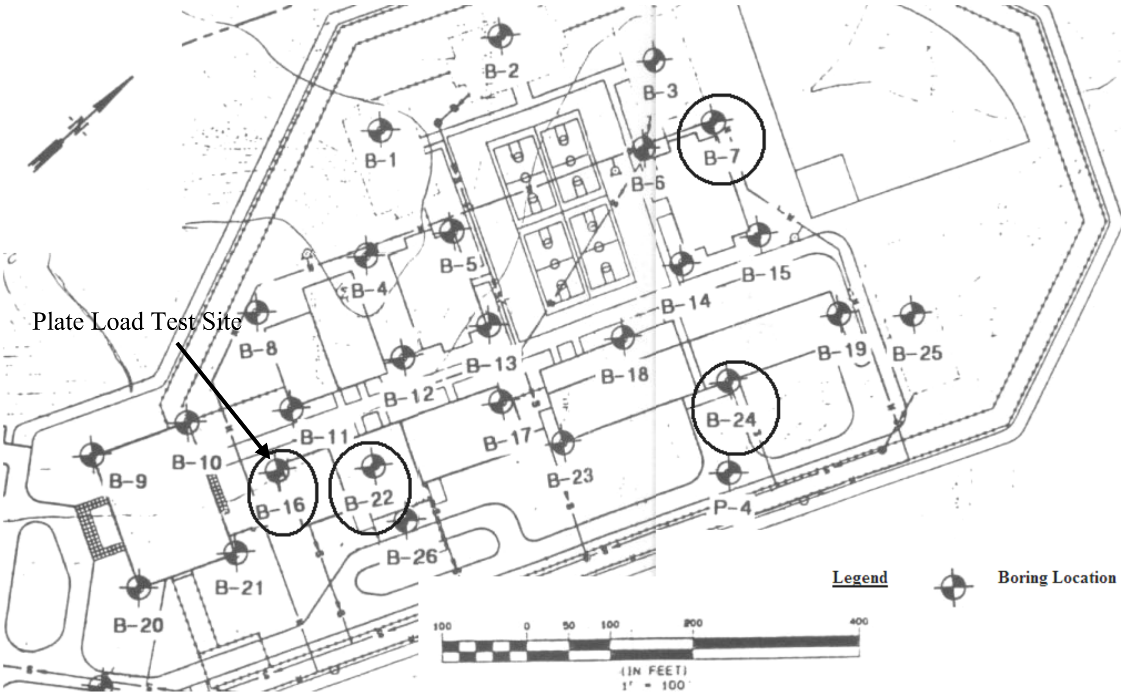

Figure 1 Plan view of A.P.Hill regional jail site, Virginia.

3.1 Standard Penetration Test (SPT)

The SPT -*N* values were obtained using a standard 2-inch (50.8 mm) O.D., 1-3/8-inch (34.9 mm) I.D. sampling spoon driven with a 140 pound (63.5 kg) hammer falling 30 inches (762 mm) as per ASTM D-1586. The soil profile for borings B-7, B-22, and B-24 is shown in Figure 2. Borings B-7 and B-22 were at the same elevation and had almost the same soil profile, whereas B-24 was at a higher elevation, and had an 8 feet (2.4 m) thick layer of clayey sand. The results indicate that the upper surfacial soils in the top six feet are very loose and are underlain by generally firmer soils. The average corrected SPT –*N* values from B-7 and B-22 for the top 8 feet (2.4 m) of the poorly graded sand layer were almost the same; however B-7 indicated a higher *N* value below 8 feet (2.4 m). The poorly graded sand layer in boring B-24 showed a higher *N* value than the other two borings. Friction angles for the different layers at each of these borings were calculated using the Hatanaka and Uchida (1996) relationship by using the corrected SPT –*N* values (Table 2). The SPT –*N* values are corrected using Liao and Whitman's (1986) relationship.

Table 2 SPT –N values and the Computed Average Friction Angles

Boring No.	Depth, ft (m)	SPT-N	Corrected $(N_1)_{60}$	Friction Angle (ϕ^o)
B-7	2 (0.6)	2	6	30
	4.5 (1.4)	2	4	
	7 (2.1)	3	5	
	9.5 (2.9)	12	16	38
	14.5 (4.4)	17	19	
B-22	2 (0.6)	2	6	31
	4.5 (1.4)	3	6	
	7 (2.1)	4	6	
	9.5 (2.9)	5	7	
	14.5 (4.4)	13	14	
B-24	7 (2.1)	6	10	33
	9.5 (2.9)	10	14	
	14.5 (4.4)	5	6	

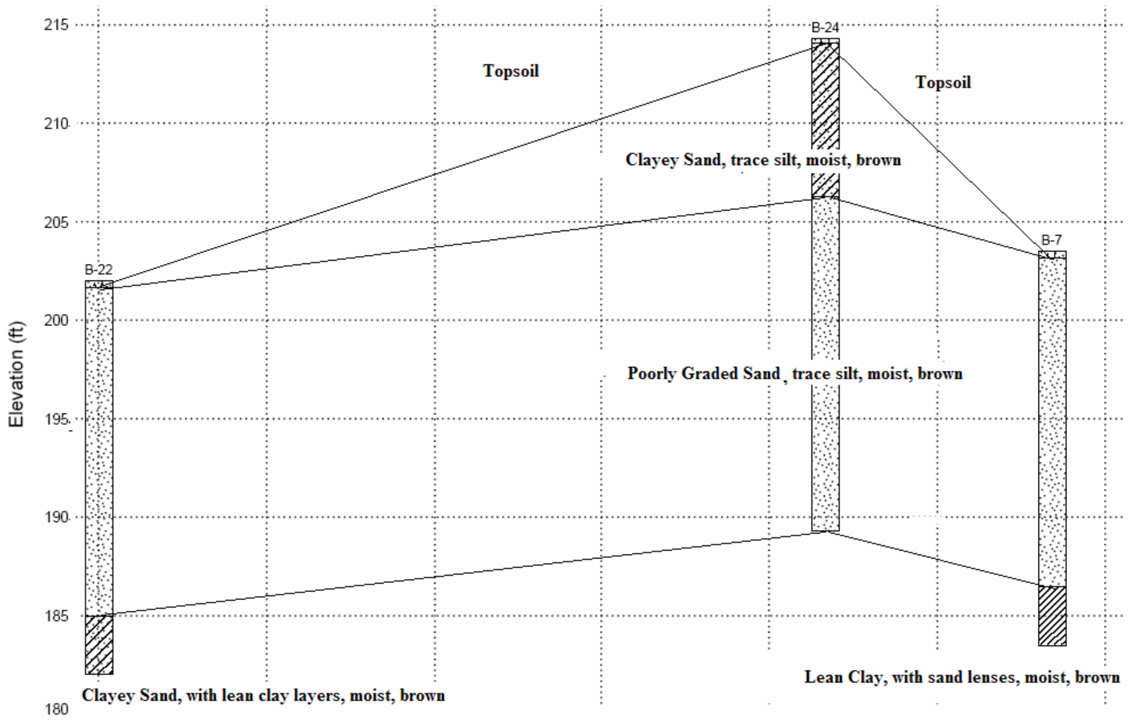

Figure 2 Soil profile for borings B-7, B-22, and B-24

3.2 Cone Penetration Test (CPT)

In-situ cone penetrometer *testing* was performed at seven boring locations to aid in evaluating soil bearing capacity and settlement characteristics. The soil interpreted from the CPT data was similar to that observed from the SPT data in some borings. However, interpretation of CPT data indicated thin clay seams in between the sandy silt layer. Since the SPT was performed only in layers of 18-inch (457 mm) increments, these thin seams may have been missed. The test results for borings B-7, B-22, and B-24 are shown in Figure 3. The results indicate the presence of clayey silt in the upper layers underlain by generally firmer silty sand to sandy silt. Also, interpretations of results from B-22 indicated the presence of sensitive fine grained soils up to a depth of 7 feet. The friction angle (ϕ^o) was calculated using the Robertson and Campanella (1983) charts and is presented in Table 3. Also, the CPT data gave higher strength parameters than those estimated by using SPT.

Table 3 Computed strength parameters from CPT data

Boring No.	Depth, ft (m)	Cohesion (C), tsf (kPa)	Friction Angle (ϕ^o)
B-7	1 – 5 (0.3-1.6)	0.88 (84)	0
	5–13 (1.6-4.0)	0	40
B-22	1 – 7 (0.3-2.1)	0.8 (77)	0
	7–16 (2.1-4.9)	0	41
B-24	1 – 4 (0.3-1.2)	1.1 (105)	0
	4–16 (1.2-4.9)	0	38

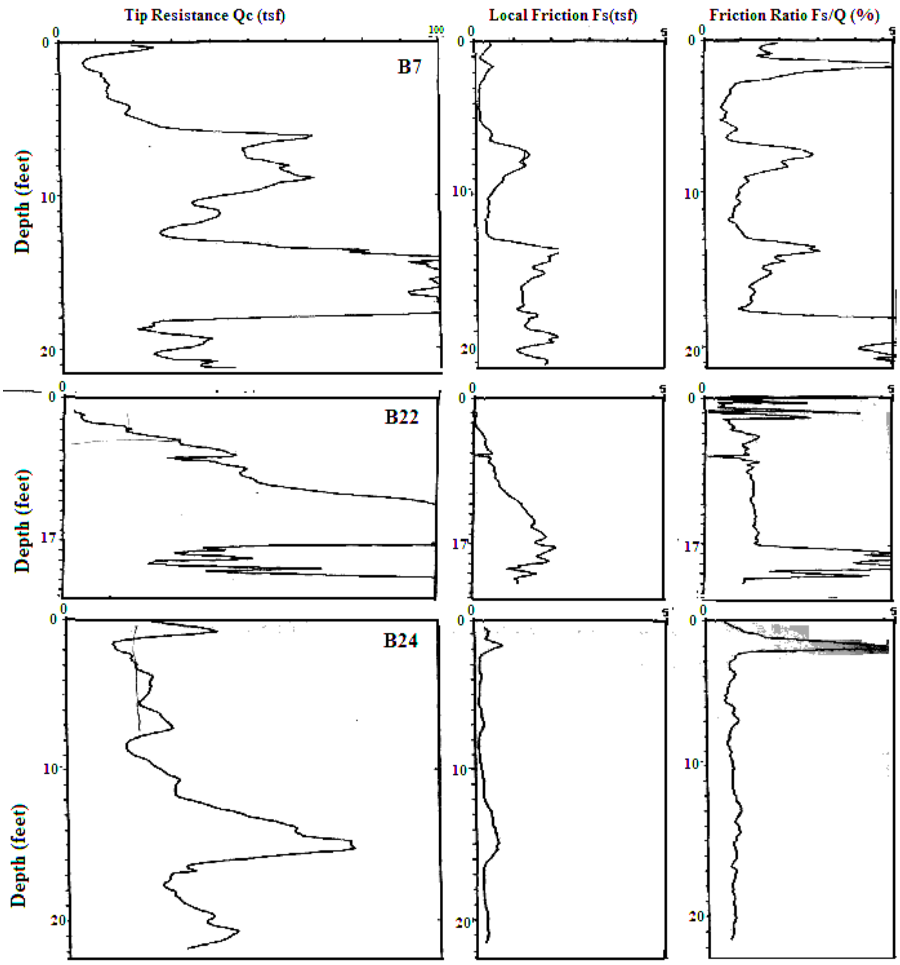

Figure 3 CPT data from borings B-7, B-22, and B-24

3.3 Menard Pressuremeter Test (PMT)

A total of seven in-situ pressuremeter tests were performed at borings B-4A, B-7A, B-11A, B-22A, and B-24A. The pertinent design values obtained from the tests are summarized in Table 4. The limit pressure (P_L) determined using the correlations from the PMT data is the pressure at which failure occurs and the pressuremeter modulus (E_M) estimated from this test is a representation of stiffness of the soil. The PMT produces much more direct measurements of soil compressibility and lateral stresses than the SPT and CPT (Coduto, 2001). The results indicate an increase in limit pressure with depth, demonstrating the presence of stiffer soils below 5 feet (1.6 m). A lowest pressuremeter modulus of 52 tsf (5.0 MPa) was obtained in B-24A, indicating the presence of a weaker sandy clay layer.

Table 4 Results from Pressuremeter Test

Boring Number	Depth, ft (m)	N value	Pressuremeter Modulus, tsf (MPa)	Limit Pressure, tsf (MPa)
4 A	5.0 (1.6)	5	118 (11.3)	9.5 (0.91)
7 A	4.0 (1.2)	4	82 (7.8)	7.5 (0.72)
11 A	6.5 (2.0)	4	118 (11.3)	11.8 (1.13)
22 A	5.0 (1.6)	4	127 (12.2)	11.3 (1.08)
22 A	9.0 (2.7)	5	115 (11.0)	12.4 (1.19)
24 A	5.0 (1.6)	7	52 (5.0)	8.2 (0.79)
24 A	9.5 (2.9)	10	112 (10.7)	13.8 (1.32)

3.3 Dilatometer Test (DMT)

Seven dilatometer tests were performed to evaluate soil bearing capacity and settlement characteristics. The soil resistance measured during insertion of the dilatometer blade is correlated to the strength of granular soils, while the soil modulus, undrained strength and other parameters are determined during dilation of the blade against the soil. The strength parameters from the DMT test results are computed using Schmertmann (1986) method and the results are shown in Figure 4. The test results predicted a lower strength and stiffness parameter for surfacial soils up to six feet, and generally uniform higher values below this depth.

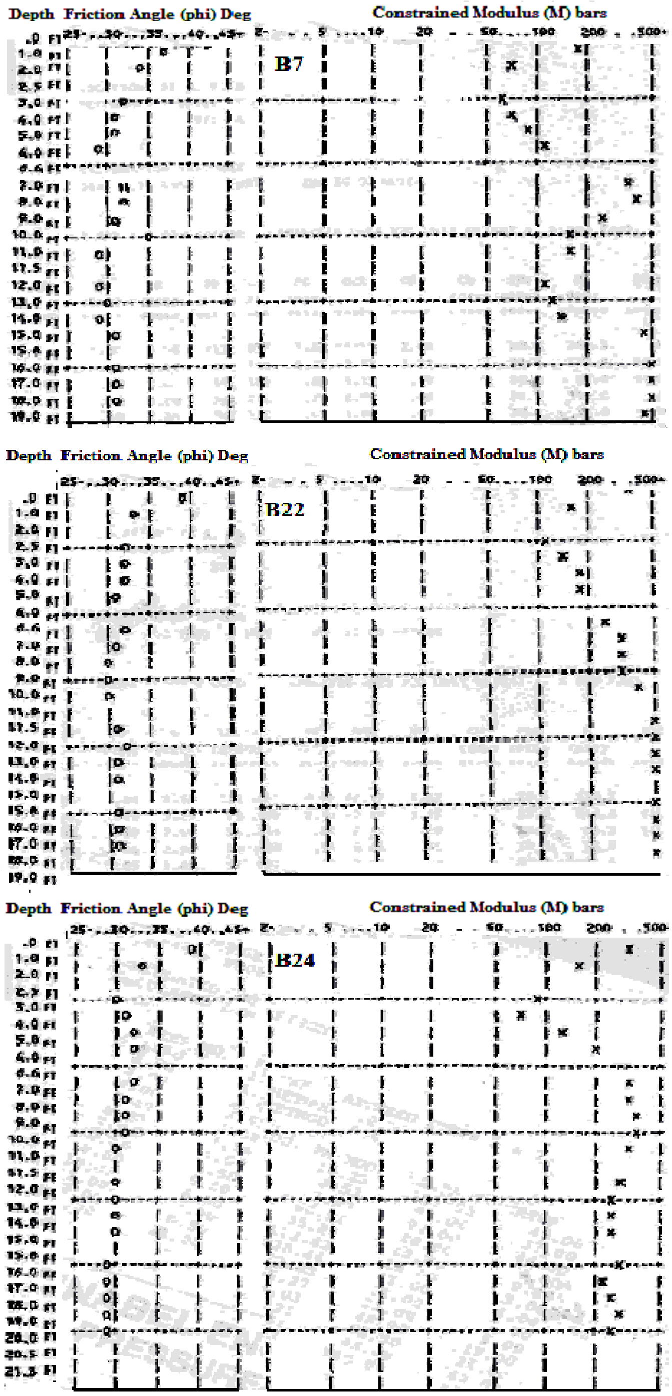

Figure 4 Results from DMT tests.

3.4 Plate Load Test

A plate load test was performed using a 1’ × 1’ square plate in the area of test boring B-16. Subsoil encountered around this vicinity was considered to be the least favorable for direct support of the footings. The plate load test results shown in Figure 5 are typical of a dense cohesionless soil which does not show any marked sign of shear failure under the loading intensities of the test. The observed cumulative settlement using this method for a bearing pressure of 2000 psf (96 kPa) was 0.21 inches (5.3 mm).

4 SOIL BEARING CAPACITY AND SETTLEMENT FROM IN-SITU TESTS

Bearing capacity and settlement were estimated at three boring locations (B-7, B-22, and B-24) using the data from SPT, CPT, DMT, and PMT. The footings at B-7 and B-22 should be founded six feet below the ground surface, and the footing at B-24 should be eight feet below the ground surface. All three footings would be resting on the sand layer. Meyerhof’s (1963) bearing capacity equation was used to estimate the ultimate bearing capacity of the soil by using the data obtained from SPT, CPT, and DMT. Bearing capacity from the PMT data was estimated using the pressuremeter limit pressure (P_L) in the Menards (1975) correlation.

The estimated allowable bearing capacities and settlements at borings B-7, B-22, and B-24 are presented in Figure 6. A factor of safety of 3 was used to estimate the allowable bearing capacity from ultimate bearing capacities. The bearing capacity of the soil varied with each boring, boring B-22 returned higher values of bearing pressure. SPT always underestimated the bearing capacity in comparison to CPT and PMT, regardless of the borings. CPT predicted higher bearing capacities than SPT and DMT, but less than PMT. The pressuremeter test predicted higher values of bearing capacity out of all the methods. It should be noted that the PMT produces much more direct measurements of soil compressibility and lateral stresses than do SPT and CPT (Coduto, 2001).

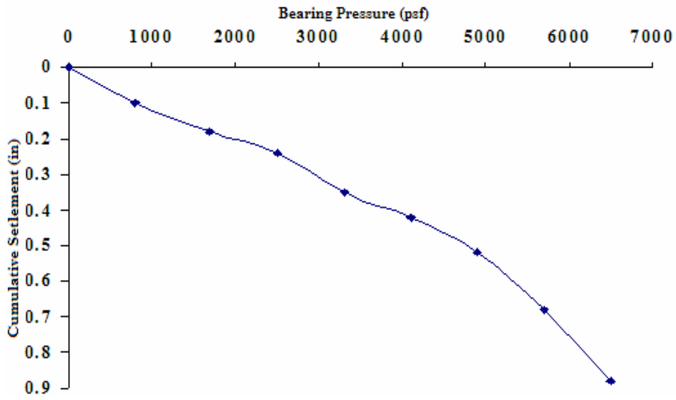

Figure 5 Plate load test results

Schnabel (1990) indicated that the bearing capacity calculations from PMT would generally yield high values of bearing pressure and must be used with an adequate factor of safety. The dilatometer test creates a bearing capacity, or cavity expansion, failure and allows for direct determination of ultimate strength values. Two methods are currently used for estimating ϕ from DMT (Marchetti, 1997). The first method provides simultaneous estimates of ϕ and K_0 derived from the pair K_D and q_D or from the pair K_D and q_c. The second method provides a lower bound estimate of ϕ based only on K_D. Marchetti et al. (2001) indicated that the underestimation of ϕ would be between 2° to 4°. The authors have also suggested that higher values of ϕ could be used if those values are more accurate. In this study the second method is used to estimate the ϕ value, this is the reason for DMT results predicting lower bearing capacity than the other three methods.

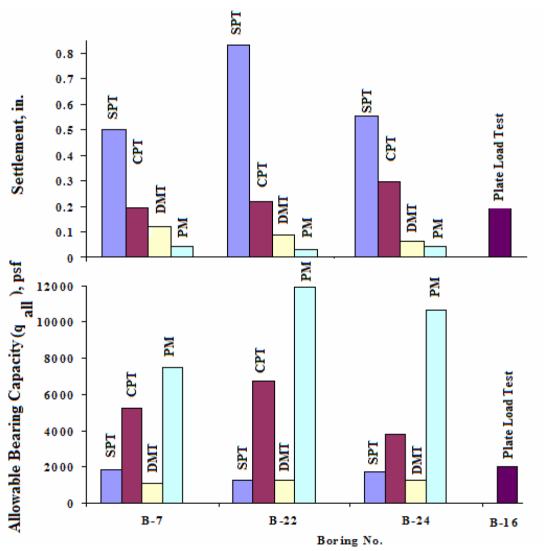

Figure 6 Predicted and measured bearing capacity and settlement

The settlement calculations for settlement in sandy soils from SPT and CPT data were performed using Bowles (1977) and Schmertmann (1978) formulations respectively. A foundation pressure of 2000 psf (96 kPa) was used in all the settlement analysis. DMT modulus (M) was used to predict the settlement from DMT data. The SPT and CPT data predicted a settlement higher than PMT and DMT. The pressuremeter modulus (E_M) estimated from PMT is a representation of stiffness of the soil, and hence used to evaluate the settlement of foundations directly. Generally settlement calculations based on the Menard method indicate low values that may be more accurate than other evaluations, but at the same time represent a lower safety margin and should be handled accordingly (Schnabel, 1990). The settlement calculated using DMT was generally higher than that calculated with the PMT method. The same phenomenon was also noted on a silty sandy soil in Quebec by Geopac (1992).

Borings B-16 and B-22 were closer to each other, therefore it is quite reasonable to compare the predicted and measured settlement from the in-situ tests in those two borings. SPT and CPT predicted a higher settlement than the plate load test in all three borings, whereas the other two methods predicted lower values. These results indicate that SPT and CPT are overestimating the actual settlement. However, the settlement predicted by DMT and PMT in boring B-22 was less than 0.1 in (2.5 mm). The possible reason for the difference in predicted settlement from DMT and PMT, and the measured values from plate load test, might be due to the small size of the plate used in the plate load test. Due to the small size of the plate, the test reflected only the properties of the uppermost soils and thus could be misleading. This is of great concern especially when the soil properties vary with depth (Coduto, 2001). In the case presented here the soil properties varied with depth, the soil profile showed generally weaker soils in the top 6 foot (1.8 m)followed by firmer soils. This might be the reason for the plate load test showing higher settlement values than the DMT and PMT. Though the PMT predicted slightly lower value than DMT, the absolute difference between the two did not exceed more than 2 mm of settlement. From these results it can be concluded that the settlement predicted by DMT and PMT is almost equal, and could possibly represent actual settlement.

5 SUMMARY AND CONCLUSION

In situ testing is rapidly emerging as a viable alternative to the traditional approach of obtaining geotechnical parameters required in prediction of bearing capacity and settlement. The site investigation for building in northern Virginia included pressuremeter tests (PMT), dilatometer tests (DMT), Standard Penetration Tests (SPT), cone penetration tests (CPT), and plate load test. The bearing capacity and settlement predicted by the four in-situ methods at three boring locations was compared with the observed settlement from the plate load test and summarized as follows:

- The CPT and SPT methods predicated lower bearing capacity and higher settlement than PMT method.
- DMT method predicted bearing capacities of less than 2000 psf (96 kPa), due to underestimation of strength parameters.
- The settlements predicted by DMT and PMT were 0.1 in (2.5 mm). Whereas, CPT and SPT predicted a settlement of more than 0.3 in (7.6 mm). The settlement observed in the field using the plate load test for a bearing pressure of 2000 psf (96 kPa) was 0.21 in (5.3 mm).
- SPT and CPT over estimated the settlement, while DMT and PMT predicted settlements less than those observed in the field by the plate load test.
- The soil profile showed generally weaker soils in the top 6 foot (1.8 m) followed by firmer soils and the plate load test was performed at the least favorable soil conditions for footing. Therefore, it is expected that plate load test would show higher settlement than actual field settlement. This might be the reason for the plate load test showing higher settlement values than the in-situ DMT and PMT.
- From these results it can be concluded that the settlement predicted by DMT and PMT could possibly represent actual settlement.

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