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Settlement Predictions in Residual Soils by Dilatometer, Pressuremeter and One-Dimensional Compression Tests: Comparison with Measured Field Response

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SYNOPSIS: A case study investigating settlement predictions based on data from one dimensional compression, pressuremeter (PMT) and dilatometer (DMT) tests is presented. A relationship is established between PMT and DMT evaluated moduli and the standard penetration N values. These relationships are utilized in the settlement computations. The predictions obtained by each method are compared to the actual measured settlement. The column location at which settlement observations were made was instrumented with strain gages to measure the actual applied loads. A comparison between actual and design loads is made. Settlement predictions using PMT were performed utilizing two different existing approaches. A distinction is made between the rheological factors, both termed α, used in each of the methods.

INTRODUCTION

It is generally believed that settlement predictions based on one-dimensional compression test data often overestimate the observed settlement of structures constructed on piedmont residual soils. Overestimation of shallow foundation settlements could unnecessarily result in the choice of a more costly deep foundation system. Less traditional insitu tests such as the pressuremeter and Marchetti dilatometer have been successfully used to more accurately predict settlement. On a recent project by Brookhollow Corporation in Greensboro, North Carolina, for which Trigon Engineering Consultants (TEC) was the geotechnical consultant, TEC performed one dimensional compression tests, pressuremeter tests, and in conjunction with North Carolina State University (NCSU), dilatometer tests. Settlement estimates were then made based on the data from each test. This paper compares these estimates with measured field response.

PROJECT DESCRIPTION

The project consists of a split-level building with four levels in the front and five levels in the rear of an office building core area. A single story section wraps around this taller core area. The building was constructed using a steel frame with composite decking and stub girder system. According to Guinnin-Cambell, the structural engineers, the maximum column loads occur at four column locations in the building core area. Total design column loads within this core area range from 180 kips to a maximum of 730 kips. The total design column loads outside the core area, around the single story section, range from 10 to 20 kips.

FIELD INVESTIGATION

Initially, five widely spaced soil borings were performed as part of a preliminary subsurface exploration at the site. Subsequently, an additional eleven soil test borings, five pressuremeter (PMT) tests, and three dilatometer (DMT) profiles were performed. The boring locations and plan view of the building are shown in Figure 1.

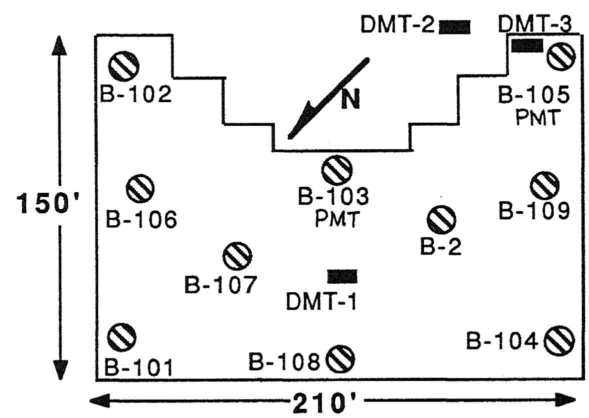

Figure 1. Boring and Test Location Plan

The soil test borings were performed to depths ranging from 15 feet to approximately 75 feet below the ground surface. Standard Penetration Tests (SPT) were performed and Shelby tube samples were recovered from the borings at designated intervals. A generalized soil profile is shown in Figure 2.

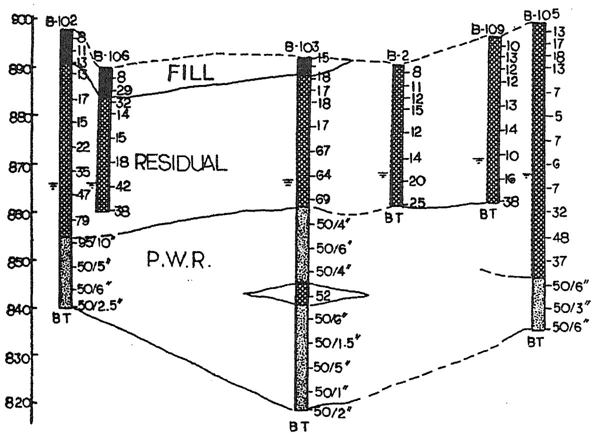

Figure 2. Generalized Subsurface Profile

Based on the SPT profiles, a series of five pressuremeter tests were performed by TEC in borings B-103 and B-105. NCSU and TEC personnel performed a total of three DMT profiles (DMT-1 through DMT-3) adjacent to the previous PMT borings, as shown in Figure 1.

In an attempt to better understand the actual loads transferred to the footings, strain gages were mounted on two columns within the taller building core area. These gages were mounted on the column steel after erection of the first level of steel and placement of the first floor concrete slab.

LABORATORY INVESTIGATION

The supporting laboratory testing program consisted of moisture content determinations, liquid and plastic limit tests, sieve analyses and one-dimensional compression tests. Table 1 shows a summary of the laboratory test results; one dimensional compression curves are shown in Figure 3.

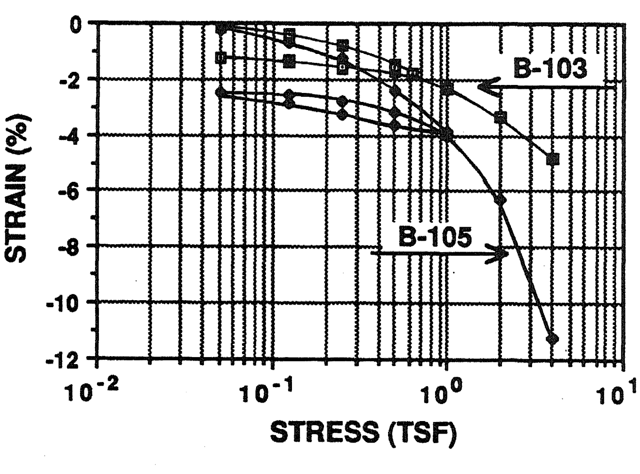

Figure 3. One-Dimensional Compression Tests

Table 1. Summary of Laboratory Test Data

	B-103A	B-105A
Depth	10.0 - 12.0'	18.0 - 20.0'
Natural ωc	20.9%	59.5%
LL / PL / PI	-----	62 / 45 / 17
Classification	Light gray fine sandy silt	Tan fine sandy clayey silt
Cc	0.09	0.43
γ wet	104.3 pcf	102.1 pcf
γ dry	86.7 pcf	64.1 pcf
Saturation	61.5%	99.4%
e_o	0.88	1.51

ANALYSIS OF DATA

The heaviest loaded column is located in the building core area, in the vicinity of B-103. This column was chosen as the focus of this study.

Loads

The loading information noted previously refers to the design loads for the project and those used in determining the footing sizes. For estimating settlement, Guinnin-Campbell initially suggested that these loads be reduced by a factor of 0.64. This resulted in a total column load composed of 87% dead and 13% live load.

Settlement predictions in this study were based on the projected actual dead loads derived from detailed engineering calculations and collaborated by strain gages mounted on the column of interest. The strain gages were monitored during construction as the steel framing and concrete for the second and third floors were completed. The strain gage readings are shown in Table 2. In interpreting the strain measurements, Poissons ratio and Young's Modulus for the 38.8 in^2 column were taken to be 0.27 and 29,000 ksi, respectively.

Table 2. Measured Column Loads

Date	Reading=2ε_h	ε_v	σ (ksi)	P (kips)
09/07/86	0	0	0	0
09/14/86	20x10 E -6	37x10 E -6	1.07	41.6
09/21/86	41x10 E -6	75.9x10 E -6	2.20	85.4
09/28/86	51x10 E -6	One gage loose		
10/05/86	Both gages destroyed			

Because the strain gages were destroyed prior to completion of construction, Guinnin-Campbell was asked to re-evaluate the loading conditions for this study without design live loads or factors of safety. The calculated dead loads for each floor are shown in Table 3.

As noted previously, the strain gages were mounted at the base of the column after the first floor pour. Therefore the readings taken on September 14, 1986, represent the response due to the estimated second floor load of 40.9 kips, as shown in Table 3.

Table 3. Calculated Construction Load for Column C.9-5

ITEM	LOAD (kips)
Load from fourth floor*	40.9
Load from third floor*	40.9
Load from second floor*	40.9
Load from first floor*	18.4

* Due to metal deck, concrete and steel framing

The second reading on September 21 was made after the third floor pour, which brought the estimated load, after gage activation, to 81.8 kips. These values compare quite favorably with the measured values of 41.6 kips and 85.4 kips, respectively. This provided the desired collaboration of the Guinnin-Campbell calculated loads.

Table 4 shows the calculated total column load. The weight of the 5-inch slab-on-grade and soil above the footing was not considered in evaluating settlements since a net stress increase for this load component would be approximately zero. Nor were items 2 and 3, because they were not in place during our settlement readings. For item 5, the difference between the weight of the soil and the weight of concrete was used.

Due to a construction problem, the foundation for column C.9-5 was overexcavated, resulting in a footing area approximately 25% larger than that originally planned. Utilizing this larger footing area with the modified calculated loads shown in Table 4 results in a net bearing pressure of approximately 1.0 ksf. Stress increases in the soil profile were calculated using the Boussinesq theory.

Pressuremeter

A summary of the five PMT tests is shown in Table 5. A ratio of the pressuremeter modulus, E_m, to the "N" value obtained directly below the PMT test elevation was used to interpret pressuremeter moduli at elevations other than the test locations. The highest and lowest E_m/"N" values were excluded, in our calculations and an average of the remaining ratios was calculated to be 9.7 . Table 6 shows the interpretation of the PMT test results. Settlement was calculated using the modulus profile shown in Table 6 and the settlement equation developed by Menard (1975).

The empirical soil factor or rheologic coefficient, α , used in these equations relates the volumetric compression modulus, obtained from the one-dimensional consolidation test, and the shear modulus, obtained from the pressuremeter test. The α coefficient is dependent on the grain size and stress history of the soil. Consistent with the range of E_m/P_1 ratios obtained, $\alpha = 2/3$ was used in the analysis.

Table 4. Foundation Dimensions and Calculated Total Loads

ITEM	COLUMN GRIDS C.9-5
1. Total Weight of structural steel, metal deck, concrete and roofing material (kips)	262.3*
2. Weight of elevator equipment (kips)	14+
3. Weight of ceiling, mechanical, shaft walls and fireproofing (kips)	33.4*+
4. Weight of 5" slab-on-grade and soil above footing (kips)	213.7+
5. Weight of footing (kips)	108.6+
6. As built Footing Size: width	11'-6"
length	22'6"
depth	3'6"
7. Bottom of footing (below top of slab-on-grade)	-8'-6"
8. Load to base of footing (kips)	650+

* Accuracy estimated to be +/- 5%
+ Not used in settlement analysis.

Table 5. Summary of Menard Pressuremeter Tests

Boring	Depth (Ft.)	PMT Modulus, E_m (TSF)	Limit Pressure, P_1 (TSF)	$\frac{E_m}{P_1}$	SPT "N"	$\frac{E_m}{\text{"N"}}$
105B	19.0	96.5	6.0	16.0	4*	24.0
105C	8.5	120.9	7.25	16.7	13	9.3
105D	20.75	72.6	6.75	10.8	9	8.1
103C	11.0	257.8	15.25	16.9	22	11.7
103	22.5	291.1	22.5	12.9	60	4.8

* SPT performed from 20' to 21.5'

Table 6. Interpretation of E_m Value by Using PMT Data of B-105

Layer/Depth Below Footing	N (Blows/ft)	N ave (Blows/ft)	E_m=9.7 N ave (TSF)
1R=5.75 ft	13		
2R=11.5 ft	18	15	145.5
3R=17.25 ft	6		
4R=23.0 ft	7		
5R=28.75 ft	6		
6R=34.5 ft.	7	6.5	96.5
7R=40.25 ft	32		
8R=46.0 ft	48	39	260
BELOW 46 ft	>100	>100	300

A second method, introduced and subsequently revised by Martin (1977, 1987), was also used for predicting settlements with the pressuremeter data. In this second method Schmertmann's strain influence factor distribution (1970, 1978) was used with the soil deformation modulus, E_s. Martin uses a rheological factor, which he also calls α, to relate the pressuremeter modulus, E_m to E_s. He suggests that a value equal to 1 be used for piedmont residual soils along with a regional correction factor equal to 0.6. This correction factor is suggested to compensate for the discrepancy between calculated and measured results.

Martin has also developed a relationship between SPT and E_{pm} for piedmont residual soils. This relationship is shown in Figure 4. A correlation coefficient equal to 0.788 for line 1, 0.795 for line 2 and 0.790 for line 3 was calculated. The difference resulted from more data points being progressively added for each line. The SPT and E_{pm} values developed for this study were used in conjunction with Figure 4 to develop E_{pm} values at the depths of interest.

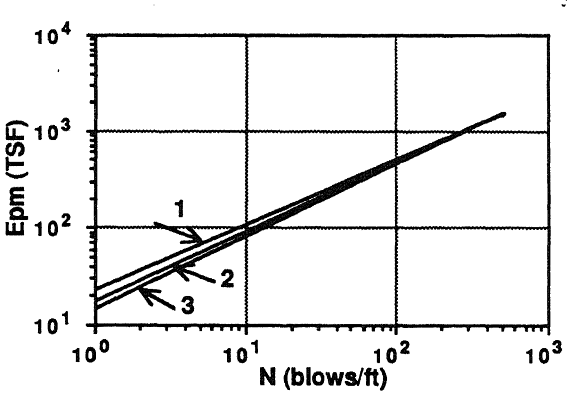

Figure 4. Relationship Between N Values and E_{pm} (After Martin, 1987)

Dilatometer

The computer program "DILLY 4", Schmertmann and Crapps (1986), was used for reduction of the DMT field data. The output of interest includes the dilatometer modulus, E_d, the material index, I_d, and a horizontal stress index, K_d. Using this DMT data, a constrained modulus, M, was calculated as suggested by Marchetti (1980) by the equation $M=R_mE_d$, where R_m is a function of the soil type (I_d) and K_d. This modulus was then used to estimate settlements. The pertinent results of DMT-1 are shown in Figure 5.

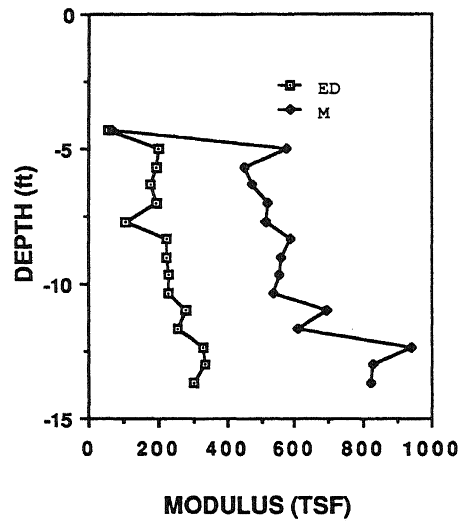

Figure 5. DMT E_d and 1-D Modulus vs Depth

A relationship was established between the DMT M values and SPT N values obtained from adjacent borings, to estimate M values at elevations other than those tested. Figure 6 shows the M/N ratio as a function of depth for locations DMT-1, DMT-2, and DMT-3. It is evident that M/N values are generally in the range of 30-70 for DMT-3 and 45-80 for DMT-1; M/N values for both locations show quite comparable results. The M/N ratio between 23 to 33 ft. in Boring DMT-2 is on the order of 10. This layer was identified as a silty clay, and the ratio of 10 indicates that the M/N ratio will be soil type dependent. An evaluation of the split spoon samples obtained below Col. C9-5 (Boring 103) shows the profile in this area to be sandy silt to a depth of 19 ft., below which the soil is a silty fine to coarse sand with some fine gravel size quartz (rock) fragments. Based on these observations, it was deemed reasonable to use an M/N ratio of 45 in the subsequent analysis.

Although Marchetti determined the R_m value which relates Ed to M to be a function of soil type and the horizontal stress index, Borden et al (1986), in a study on laboratory compacted and field samples, suggested the use of E_d as an upper bound to the anticipated insitu constrained modulus for piedmont residual soils. This amounts to choosing $R_m = 1$. Both methods for determining the constrained modulus were used in this study.

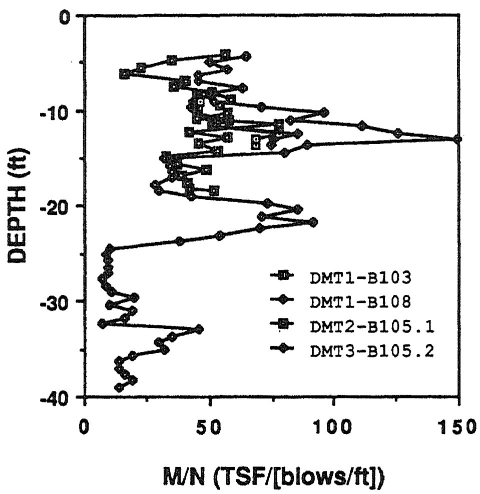

Figure 6. DMT-M/N Ratio Vs. Depth

One-Dimensional Compression

Based on the $\sigma-\varepsilon$ curve from the 1-D compression test of B-103A (Figure 3), the best fit equation for the test data was found to be: $\varepsilon = 0.0114 + 0.006\,\sigma + 0.000108\,\sigma^2 - 0.000034\,\sigma^3$. By differentiating the above equation, the M value is defined by $M = 1/m_v$, where m_v is the coefficient of volume change. The $M - \sigma$ curve is shown in Figure 7. The one-dimensional settlement of each sublayer was then made using the following equation:

settlement = (Δ stress)(thickness)/modulus (M).

The total settlement is obtained by adding the contribution of each sublayer.

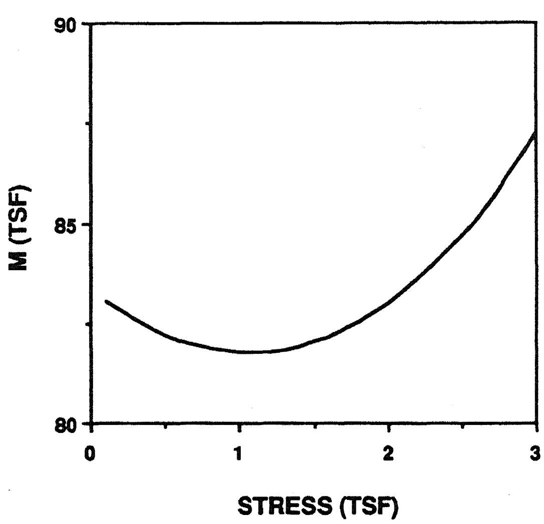

Figure 7. Constrained Modulus vs Vertical Stress

Settlement Comparison

Table 7 shows a summary of the calculated and measured settlements. Figure 8 presents a bar graph comparison of the settlement predicted by the various methods. These results are shown in conjunction with the measured settlement of 0.3 inches.

Table 7. Summary of Calculated and Measured Settlements (inches)

		DMT		PMT	
Measured	1-D Compression	M	$M=E_D$	$\alpha = 2/3$	Martin's $\alpha = 1$ (x0.6)
0.3	0.8	0.11	0.29	0.12	0.22 (0.13)

The predicted PMT settlement using Menard's formula and $\alpha = 2/3$ is 0.12 in. Using Schmertmann's strain influence factor method and Martin's $\alpha = 1$, a settlement of 0.22 in. is predicted. Applying Martin's regional correction factor, results in a settlement of 0.13 in., which further underpredicts the observed settlement.

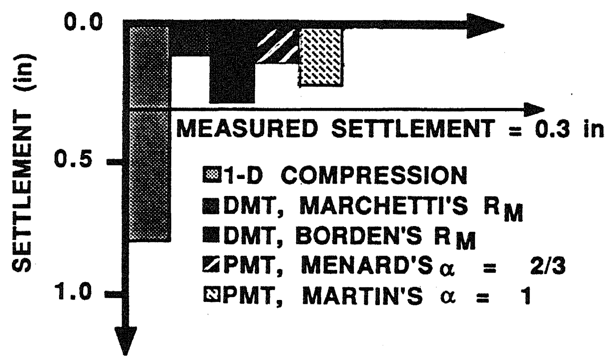

Figure 8. Predicted Vs Measured Settlement

From Figure 8 it can be seen that the predicted DMT settlement using constrained modulus (M) as suggested by Marchetti underpredicts the observed settlement. Utilizing $R_m = 1$, therefore choosing E_d as the upper bound for M, results in a prediction of 0.29 in., which is in good agreement with the 0.3 in. measured.

In contrast to the DMT and PMT prediction methods, using the interpreted M values obtained from the 1-D compression data, resulted in a predicted settlement of 0.8 in. As this overprediction is somewhat typical, it is local practice to multiply this value by 2/3, which would reduce the prediction to 0.53 in., or nearly 1.8 times the measured settlement.

Summary and Conclusions

As a preface to our conclusions, it should be noted that this study is for only one project. The measured settlement was less than one half inch and the accuracy of our measurements is estimated to be ∓ 0.1 inch. Therefore, additional studies are needed to substantiate the findings. At the time this paper was prepared, additional studies with the same scope of work were being planned. These studies will be reported as they are completed.

The following points can be made concerning the findings in this study:

1. The α factor used in the PMT analyses significantly influences settlement predictions. The α factor used by Martin in conjunction with Schmertmann's strain influence factor method is not the same as that suggested by Menard. The fact that these two factors are both called α could undoubtedly lead to confusion. Further examination of the settlement predictions shows that for Martin's method, applying a regional correction factor of 0.6 is essentially equivalent to using α equal to 2/3 with Menard's formula.

2. The settlement predictions made by using constrained modulus (M) profile obtained from Marchetti's correlation underpredicted the settlement. The prediction made using M equal to Ed shows a much better result. This indicates that Marchetti's M value correlation might overpredict the stiffness of residual soils.

3. Settlement predictions made using the 1-D compression test data overestimated the measured settlement.

4. In evaluating the building loads utilized in the settlement analysis, it was observed that the calculated dead loads were very close to those measured by the strain gages. In contrast, the initial design loads provided for settlement estimates were 165% of the actual loads. The use of the more conservative design loads would have resulted in much more conservative settlement predictions. When using design versus actual loads to predict settlement, one may have the impression that a particular analysis method is conservative or unconservative, when in fact it is not the method which is being evaluated as much as the appropriateness of the assumed loads.

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