DMT testing for redesign using shallow foundations

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ABSTRACT: Yes, the United States has far too many lawyers, and geotechnical engineers worry about their liability. But, when geotechnical engineers recommend costly foundation solutions because they don't have accurate enough data, we are making inexcusable errors and are not serving the owner's needs. Dilatometer tests provide engineers with high quality data so that they can make good foundation design decisions. Presented in this paper are several case studies showing how dilatometer tests and analyses resulted in much more economical foundation design solutions than in the originally proposed solutions.

1 INTRODUCTION

Engineers in the U.S. often use standard penetration testing as the only method of investigating a project site. Laboratory consolidation testing is routinely omitted either due to too small of a testing fee or sands that are difficult to sample. Because of the high uncertainty in defining and understanding the deformation characteristics of the soil, the engineer becomes overly conservative with his design. Unfortunately, many engineers are often reluctant to ask the owner to pay for additional investigations after they know that they need them to do good design. Faced with expensive foundation recommendations that the owner is not sure he needs, the owner will lose confidence in the first engineer and often ask another engineer to redesign the foundation. As the second engineers, we performed subsurface investigations using dilatometer tests to characterize the deformation characteristics of the soils better and provide much more economical yet safe designs.

2 REVIEW OF SPT SETTLEMENT PREDICTION

2.1 *SPT Procedure*

The standard penetration test (SPT) is a dynamic penetration test that strains the soil to much higher levels than what structures impose on the underlying soil (Figure 1). Correlations between the dynamic penetration response of the soil and the soil's static deformation modulus are poor. There is further uncertainty in correlation coefficients when trying to extrapolate the deformation modulus from a high strain test to a medium strain loading condition.

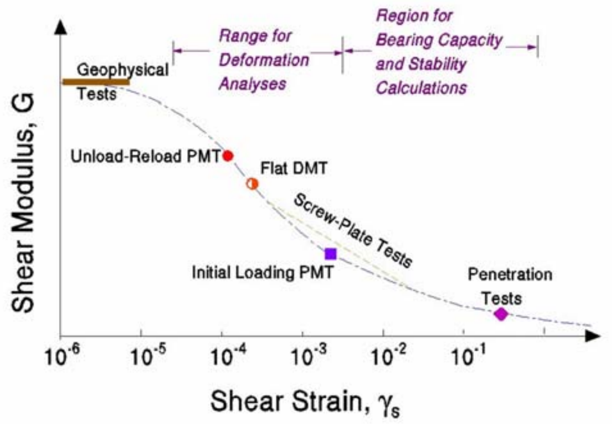


Figure 1: Strain levels imposed by DMT and other in-situ tests (Mayne, 2001)

While the applied hammer energy of the SPT can vary from 30 to 95% of the potential energy of 4200 in-lbf [48260 kgf-mm] (30-inch drop times 140 lbf hammer), it is rarely calibrated. The test is operator dependent. Higher quality operators provide more repeatable results. The uncertainty from measurement noise (test repeatability) can be as high as 45 to 100% (Schmertmann, 1978; Kuhawy, 1996).

Much research for the SPT was performed in the 1940s-1960s using mud rotary drilling methods and donut and safety hammers. Instrumentation had not been developed then to measure the applied hammer energy. Researchers believe the applied hammer energies were about 55 to 60% of the potential energy. Skempton (1986) proposed correcting the SPT N-value to an N_{60}-value, representing a 60% applied hammer energy. However, even today it has been rare to find N_{60} values shown on boring logs in the U.S.

Many newer SPT drill rigs use automatic hammers. Many of these hammers, provided that they are well maintained, consistently deliver 90 to 95% of the SPT potential energy. Without making the N_{60} correction, the N-value from the automatic hammer will be about 2/3 of the N-value from a safety hammer.

In the 1940s-1960s the inner diameter of the barrel of the SPT spoon was the same as the tip. Today, the inner barrel has an inside diameter that is larger than the tip inside diameter, which allows liners to be inserted in the barrel. Without liners, the frictional resistance along the inside of the spoon is greatly reduced. While the reduction in resistance depends on soil conditions, Skempton (1986) suggests that an average reduction of 20% occurs.

When a borehole is made using hollow-stem augers, the pre-existing geostatic stresses are removed. When a borehole is made using mud rotary drilling, about half of the pre-existing geostatic stresses are removed. Reductions in the pre-existing geostatic stresses soften or loosen the soils and result in lower N-values.

With today's methods and without the N_{60} correction, the uncorrected N-values can be ½ of the N-value measured during the 1940s-1960s. Yet, geotechnical engineers will often use their uncorrected N-values with the design methods from that era. As a result, they are misled into believing the soils are much weaker than they actually are.

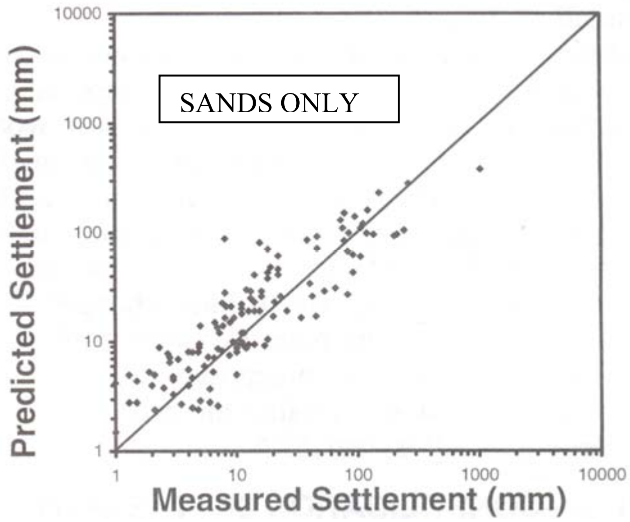
2.2 *SPT Design Methods for Settlement*

In sands Burland and Burbridge (1985) developed the following equation to predict settlement using the SPT:

$$S = B^{0.75}\,\{1.7/(N_{60AVG})^{1.4}\}(q\text{-}2/3\sigma_{vo}')$$

where S= predicted settlement (mm),
B= footing width (m),
q = applied bearing pressure (kPa),
σ_{vo}' = initial effective vertical stress at the base of the footing level (kPa),
and N_{60AVG} = average SPT blow count within a depth of $B^{0.75}$ meters beneath the footing.

Their case study database revealed the following graph (Figure 2) of predicted and measured settlement.



Fiure 2: Predicted vs. Measured Settlement from SPT in Sands Only (Burland and Burbridge (1985).

Based on the Burland and Burbridge (1985) equation, Duncan (2000) presented a settlement example that showed that an average settlement of 0.3 inches [7.6 mm] was required for the structure to have less than 1.0 inch [25 mm] of settlement. Duncan (2000) showed that the coefficient of variation (standard deviation/average value) was 0.67 for the Burland and Burbridge (1985) method. Failmezger (2001) showed that when measurement noise (test repeatability) and spatial (site subsurface variability) are considered in addition to the method error, the average settlement such that settlement would not likely exceed 1.0 inch [25 mm] is less than 0.3 inches [7.6 mm].

Engineers may use other design charts or correlations to predict settlement in sands and even other soil types. SPT tests in clay and residual soils destroy the soil structure and will often result in low "N" values that may only be representative of remolded properties instead of intact properties. The accuracy with these methods will be even less than the Burland and Burbridge (1985) method.

In summary, settlement predictions based on SPT are too inaccurate to be used for design.

3 REVIEW OF DMT SETTLEMENT PREDICTION

Schmertmann (1986) developed his ordinary and special methods for computing settlement of a structure or embankment. The ordinary method is simply the increase stress multiplied by the layer thickness divided by the constrained deformation modulus. In his special method the modulus is adjusted to account for whether the increase stress occurs below the preconsolidation pressure (highly overconsolidated soil), above the preconsolidation pressure (normally consolidated soil) or starts below the preconsolidation pressure and then exceeds it (lightly overconsolidated soil). Generally, settlement prediction from the ordinary method is within 10% of the special method. Using his 16 case studies, Schmertmann (1986) had an average predicted to measured ratio of 1.18 with a standard deviation of 0.38. If the predictions where the dilatometer blade was driven and where tests were performed in quick clayey silts are excluded from the data set, the average predicted to measured ratio reduces to 1.07 with a standard deviation of 0.22.

From dilatometer test data, Hayes (1986) computed settlement at 5 sites using Schmertmann's (1986) methods. From his case studies with the ordinary method, the average predicted to measured ratio was 1.02 with a standard deviation of 0.14 and for the special method, the average predicted to measured ratio was 1.06 with a standard deviation of 0.25. If we use all the case study data and exclude the data for the quick clayey silts and driven DMT data, the average predicted to measured ratio is 1.06 and its standard deviation is 0.18. A summary graph (Figure 3) from these researchers is shown below:

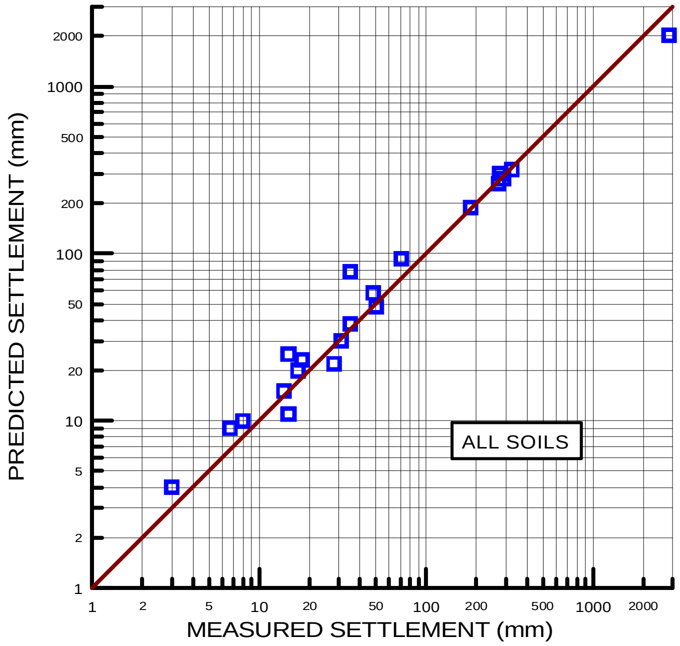


Figure 3: Predicted vs. Measured Settlement from DMT in All Soils (adapted from Schmertmann, 1986) and Hayes, 1986)

4 CASE STUDIES

Five case studies are presented below that demonstrate the value of using dilatometer test data for design. In each case the redesign saved the owners between US $200,000 and US $800,000. Each building is performing to the satisfaction of the owners. A summary of the original design and the redesign based on dilatometer testing is shown in Table 1.

Site	Column Load (kips/kN)	First Engineer's Recommended		DMT Redesign Recommendation	
		Bearing Capacity (psf/kPa)	Predicted Settlement (inches/mm)	Bearing Capacity (psf/kPa)	Predicted Settlement (inches/mm)
Westminister Village	90 [400]	1000 [48]	1 to 4 [25-100]	1500 [72]	< 1.0 [<25]
Walmart Store at Ocean Landing Shopping Center	60 to 160 [267 to 712]	1000 [48]	2.5 [64]	2000 [96]	0.5 [13]
Old Town Crescent	250 [1112]	3000 [144]	3 [75]	5000 [239]	< 0.5 [<13]
Fox Run Village - Novi, Michigan	300 [67]	2000 [96]	3 to 5 [75-125]	4000 [192]	< 1.0 [<25}
Monarch Landing - Naperville, Illinois	200 [45]	2000 [96]	> 1 [>25]	6000 [287]	< 1.0 [<25]

Table 1: Summary of Foundation Redesign Case Studies

4.1 *Westminister Village*

In the first geotechnical investigation program, soil test borings showed 7 to 13 feet [2.1 to 4.0 m] of sand underlain by a soft to medium stiff clay. One laboratory consolidation test was performed on an "undisturbed" clay sample. The stress-strain curve from that test was rather flat indicating that the sample was disturbed. The geotechnical engineer predicted settlements between 1 and 7 inches [25 and 178 mm] for shallow spread footings and recommended pile foundations.

We performed dilatometer tests near the two boring locations where the clay was the softest and thickest. The results of the dilatometer tests are presented in Figure 4. We redesigned the building to be supported on shallow spread footings and conventional ground supported floor slabs. We predicted settlements of about 0.5 inches [12.7 mm].

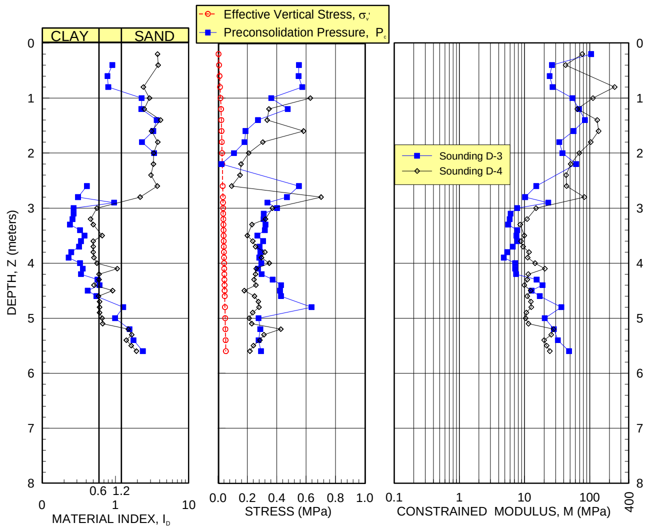


Figure 4: Summary of dilatometer results from Westminister Village

4.2 *Ocean Landing Shopping Center--Walmart Store*

For the Walmart Store site, the first geotechnical engineer performed soil test borings that showed sand with an underlying near surface organic silt and clay layer. Based on a consolidation test from an undisturbed Shelby tube sample, the engineer predicted 2.5 inches [64 mm] of settlement. The engineer recommended pile foundations to support the column and slab loads.

We performed 13 dilatometer test soundings within the footprint of the building. Representative results are presented on Figure 5. We predicted settlement to be between 0.25 and 0.75 inches [6.4 and 19.1 mm].

To verify our settlement predictions, an embankment load test was performed (Figure 6). The fill height was 8 feet [2.4 m], which imposed the same stress on the organic layer that the proposed footings would impose. Piezometers and settlement points were installed within the embankment. Under the load, a settlement of 0.5 inches [12.7 mm] occurred rapidly and excess pore pressures dissipated quickly.

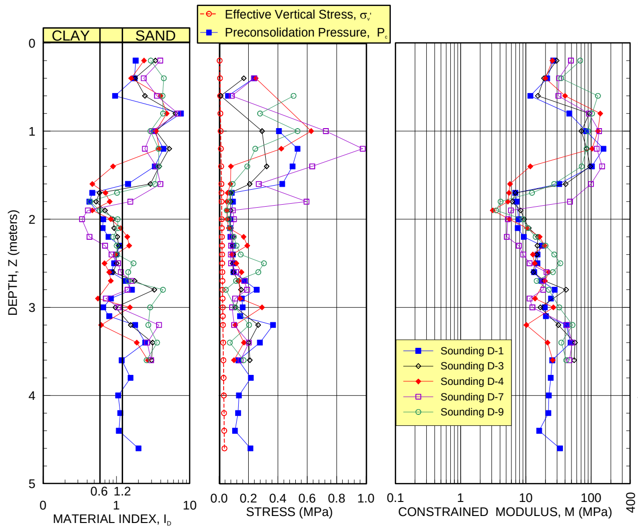


Figure 5: Summary of dilatometer tests from Ocean Landing Shopping Center

Figure 6: Embankment load test setup

At an adjacent site, without the benefit of dilatometer test data, the geotechnical engineer recommended using stone columns to support a similarly loaded structure. We investigated the adjacent parcel on the other side to this center parcel with dilatometer tests. The boring logs show that all three sites have similar geologic conditions. The two sites where dilatometer tests were performed were successfully designed using conventional spread footings, while we believe the center site was overdesigned at an additional cost of US $750,000.

4.3 *Old Town Crescent*

Based on standard penetration tests, the first geotechnical engineers found a loose silty fine sand between 12 and 22 feet [3.7 and 6.7 m]. Groundwater was about 5 feet [1.6 m] deep. They recommended using shallow spread footings with an allowable bearing pressure of 1500 psf [72 kPa].

Settlement predictions based on SPT are very inaccurate even in sands (Failmezger, 2001). As the second geotechnical engineer, we performed dilatometer test soundings at the corners and center of the proposed building. Those DMT results are summarized on Figure 7. Because the structure also had a 1-level underground garage, we considered the removal of 960 psf [46 kPa] of overburden as well as no overburden removal in our settlement analyses. The design column load was 250 kips [1110 kN]. With the overburnen removal and with a design bearing pressure of 5000 psf [240 kPa], our settlement predictions were less than 0.25 inches [6.4 mm]. Without the overburden removal, our settlement predictions were between 0.2 and 1.1 inches [5.1 and 27.9 mm].

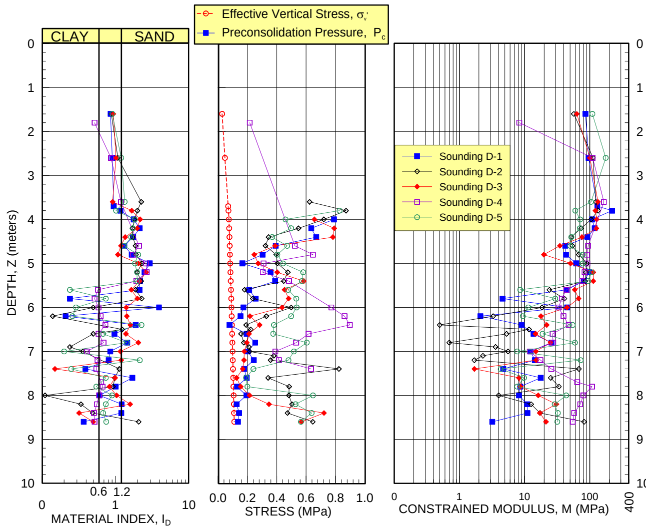


Figure 7: Summary of dilatometer tests from Old Town Crescent

4.4 *Fox Run Village*

The first geotechnical engineer recommended a mat foundation for the proposed 3 to 4 story residential retirement buildings. From the standard penetration test results, the first engineer concluded that the clays at the site were soft. One building was under construction and the two other buildings (Nos. 2.3 and 3.1) had their building pads graded when we were hired to reevaluate the first engineer's recommendations.

We performed dilatometer test soundings for Buildings 2.3 and 3.1 and one dilatometer sounding adjacent to the constructed mat foundation. For Buildings 2.3 and 3.1, we predicted settlements of less than 1.0 inch [25 mm] for the design column load of 300 kips [1334 kN] using an applied bearing stress of 4 ksf [191 kPa]. For the building with an existing mat foundation, we found that the clays were softer there. Here the foundations needed an applied bearing pressure of 1.7 ksf [81 kPa] to keep settlements less than 1.0 inch [25 mm].

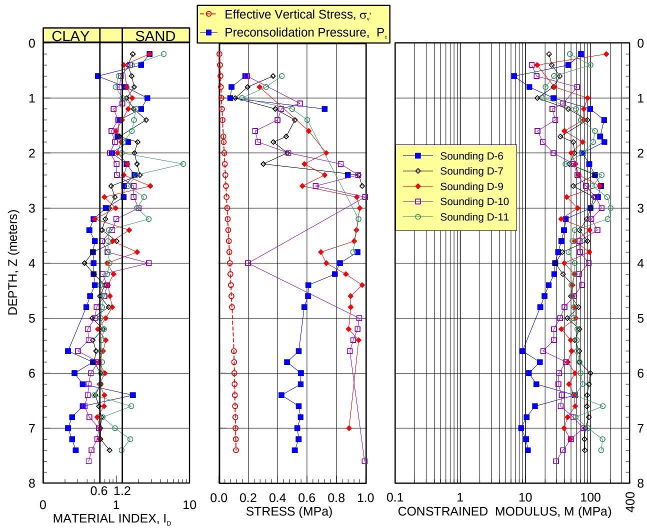


Figure 8: Summary of dilatometer tests from Fox Run Village

4.5 *Monarch Landing*

The first geotechnical engineer performed 62 soil test borings and 21 test pits as their subsurface exploration plan. They recommended supporting the building, which had design interior column loads of 1500 kips [6672 kN] on spread footing using an allowable bearing pressure of 3000 psf [144 kPa].

We performed 15 dilatometer test soundings at the site to reevaluate their design. While the depth intervals for the dilatometer tests were generally 20 cm, in areas where softer clays were found we used depth intervals of 10 cm to define those clays better. Where the clays were too soft to provide adequate support, the close interval test spacing helped us to determine how deep to undercut those clays and replace them with compacted structural fill. We found that the allowable bearing pressure could be 6000 psf [287 kPa] and the resulting settlements would be less than 1.0 inch [25 mm].

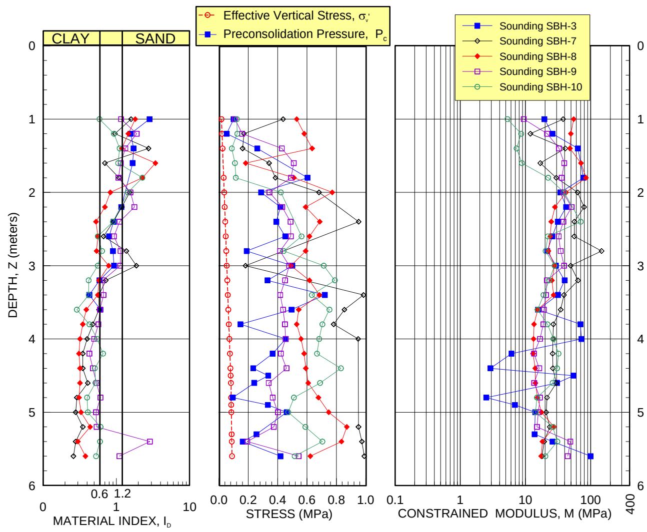


Figure 9: Summary of dilatometer tests from Monarch Landing

5 CONCLUSIONS

1. Today engineers' biggest mistakes are recommending a costly foundation solution without adequate data to prove that this solution is necessary.
2. Standard penetration test data should never be used to predict foundation settlements for any soil.
3. Accurate settlement predictions can be made using dilatometer test data.
4. The dilatometer is not an expensive in-situ test, and the appropriate interpretation of the testing data can save quite a lot of money in the foundation design, as presented in the five case studies.

6 REFERENCES

Burland, J. B. and Burbridge, M. C., (1985), "Settlement of Foundations on Sand and Gravel", Proc., Inst. of Civ. Engrs, Part 1, 78, 1325-1381.

Duncan, J. M. (2000), "Factors of Safety and Reliability in Geotechnical Engineering", ASCE Journal of Geotechnical and Geoenvironmental Engineering, Vol 126, No. 4, pp. 307-316.

Failmezger, R. A., (2001), Discussion of "Factors of Safety and Reliability in Geotechnical Engineering", ASCE Journal of Geotechnical and Geoenvironmental Engineering, Vol 127, No. 8, pp. 703-704.

Failmezger, R. A. and Bullock, P. J., (2004), "Individual foundation design for column loads", International Site Characterization '02, Porto, Portugal, pp. 1439-1442.

Hayes, J.A., (1986), "Comparison of flat dilatometer in-situ test results with observed settlement of structures and earthwork", Proceedings 39th Geotechnical Conference, Ottawa, Ontario, Canada.

Marchetti, S., (1980), "In situ tests by flat dilatometer, Journal of the Geotechnical Engineering Division, ASCE, Vol. 106, No. GT3, pages 299-321.

Mayne, P. W. (2001). Keynote: "Stress-Strain-Strength-Flow Parameters from Enhanced In-Situ Tests", Proceedings, International Conference on In-Situ Measurement of Soil Properties & Case Histories (In-Situ 2001), Bali, Indonesia, 27-47.

Schmertman, J. H. (1978), "Use the SPT to Measure Dynamic Soil Properties? – Yes, But..!", Dynamic Geotechnical Testing, ASTM STP 654, American Society for Testing and Materials, pp. 341-355.

Schmertmann, J. H., (1986), "Dilatometer to compute foundation settlement", Proceedings, ASCE Specialty Conference, In-Situ '86, VPI, Blacksburg, Virginia, pages 303-321.

Skempton, A. W. (1986), "Standard penetration test procedures and the effects in sands of overburden pressure, relative density, particle size, ageing, and overconsolidation", Geotechnique 36, No.3, pp. 425-447.