TERRANOTES
A Ground Improvement Update from TerraSystems

SOIL MODULUS AFTER GROUND IMPROVEMENT

Evaluation of ground improvement is accomplished using a variety of methods, from simple elevation surveys to document the amount of compression caused, to geotechnical in-situ and laboratory testing, to geophysical testing. The most common types of geotechnical testing used to evaluate the modulus of the soil after ground improvement are penetration tests such as the Standard Penetration Test (SPT) or the Static Cone Penetration Test (CPT). Engineers occasionally ignore two significant factors when evaluating these test results: (1) continued increase of CPT or SPT values over time, and (2) an increased correlation between modulus and penetration resistance for overconsolidated soils, as compared to normally consolidated soils.

During performance of ground improvement, ground surface settlements of one to three feet are not uncommon at many sites. Calculated settlements under foundation loads are generally much less than the observed settlement. If the post-improvement geotechnical testing indicates little improvement has occurred, the geotechnical engineer is faced with the following question. *"Was my observation that the soils have been prestrained incorrect or are the correlations between compressibility and the geotechnical testing parameter wrong?"* Since the site settlement (i.e., prestraining) is an easily verifiable phenomenon, the answer is obvious. The next step is to determine how best to reconcile the known site settlement with the geotechnical testing.

It has long been recognized that both strength and compressibility properties of soils improve with time in both natural soil deposits and soils improved by dynamic compaction and vibrocompaction or vibroreplacement. Improvement has been observed in sand, clay, and silt. Numerous articles have been published on this phenomenon, including the Twenty-Fifth Karl Terzaghi Lecture (Schmertmann, 1991). This strength gain and modulus increase occur well after excess pore water pressures dissipate, and thus can not be attributed merely to this occurrence. Various investigators have attributed this aging improvement to thixotropy, secondary compression, cementation, dispersive particle movements and internal stress arching, as well as other explanations.

Ground improvement practitioners, as well as geotechnical consultants involved in the ground improvement industry, are constantly faced with the question of how to evaluate the degree of improvement of a densified soil deposit. Quite often geotechnical testing by SPT or CPT falsely indicates a decrease in strength and compressibility shortly after ground improvement. However, after a short time period the strength and compressibility properties improve dramatically. Schmertmann (1986) presented data on a 10m layer of silty sand in Jacksonville, Florida that was improved by dynamic compaction. The static cone bearing capacity q_c over time was compared to q_{co} immediately after dynamic compaction. This data is presented as Figure 1.

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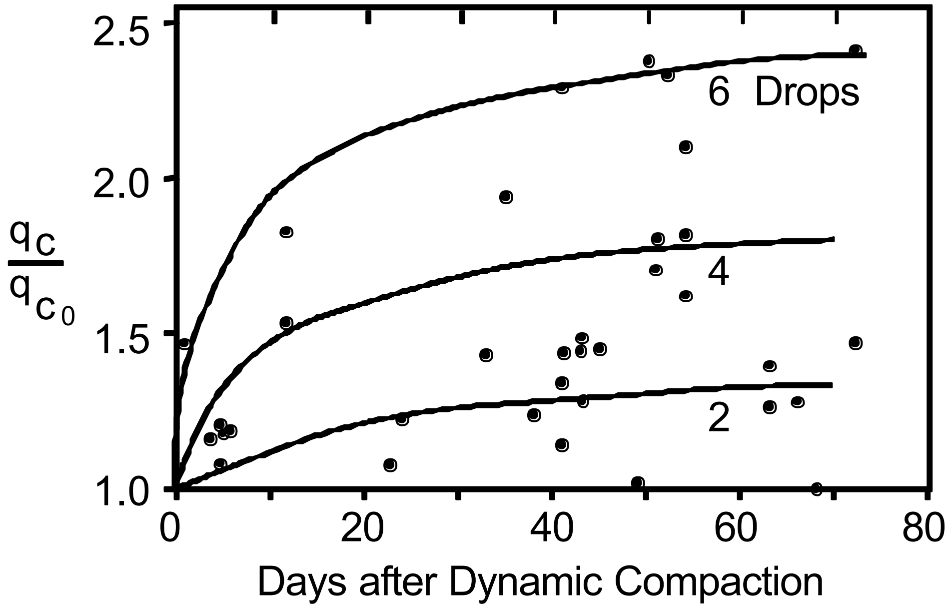

Figure 1 – Increase in Static Cone Resistance with Time Following Dynamic Compaction (Ref. 9)

Jones (1988) reported on a vibroreplacement project at the Treasure Island Dental Clinic in San Francisco, California. During treatment of the silty/clayey sands overlying the Bay Mud, it was observed that an equivalent ground surface subsidence of about 2 feet (10% strain) occurred, yet the static cone resistances experienced little to no increase, and in many cases, were less than before compaction. In one portion of the site, extreme difficulty was observed in penetration of the vibrators. It was discovered that about 8 years before, that portion of the site had been treated by vibroreplacement, with similar soil behavior. During that earlier time period, due to the inability to increase the CPT resistances of the silty/clayey sand, even with a 10% volumetric reduction, it was decided to use the volumetric reduction as an acceptance criterion. When testing was performed in 1988 on the soils treated 8 years earlier, it was found that there was a *five hundred* percent increase in the CPT values.

Lukas (1997) reported on results of dynamic compaction on a 4 ft. thick layer of cinders overlying 5 ft. of virgin clayey silt, where the water table was located at the top of the clayey silt. The ground water level rose approximately 4 feet during dynamic compaction, causing the ground to behave in a very spongy manner. Pressuremeter tests performed in the clayey silt 15 days after compaction were found to be lower than initial values. Excess pore pressures had dissipated after 35 days. After 50 days the pressuremeter modulus was slightly greater than the initial values. Seventy days after compaction the pressuremeter modulus had doubled from the 50 day values. Pressuremeter tests were also conducted after 14 years, showing a 300 percent increase over the 50 day value.

Penetration test results are most commonly used to estimate the settlement behavior of the soils. Calculated settlement is inversely proportional to the soil modulus, either the elastic modulus, E, or the constrained modulus, M. The constrained modulus is the more commonly used parameter. Based on published studies from various sources, the general expression for constrained modulus from CPT data is:

$$M = \alpha q_c$$

where ? depends upon stress state, soil type, and degree of preconsolidation.

A review of calibration chamber tests on normally consolidated sands from Norway, Italy, UK and the US (Mitchell and Gardner, 1975, Lunne and Christoffersen, 1985) and presented as Figure 2 indicates that:

$3 < \alpha_{n.c.} < 8$

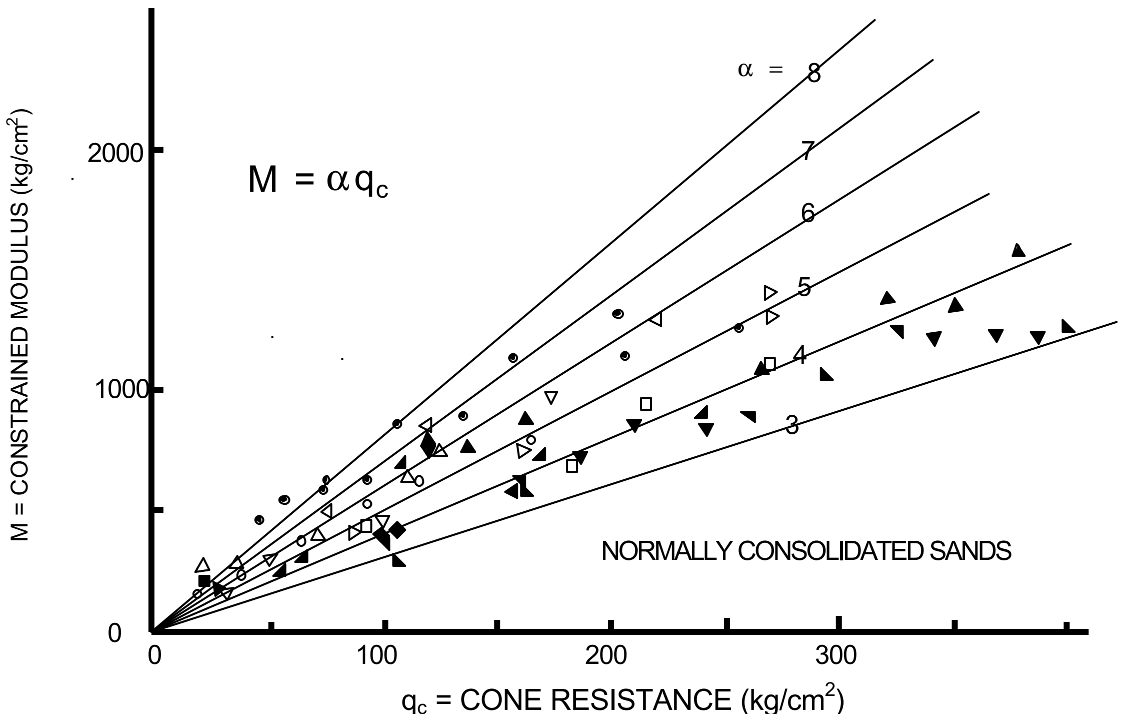

Figure 2 – Constrained Modulus vs. Cone Resistance for Normally Consolidated Sands (Ref. 4)

The application of dynamic compaction or vibrocompaction results in overconsolidated soil. Consequently, the ratio of $\alpha = M/q_c$ for dynamically compacted or vibrocompacted sand is much higher than for normally consolidated sand (Robertson and Campanella, 1983). Figure 3 summarizes data from overconsolidated sands indicating the range to be $7 < \alpha_{o.c.} < 36$, significantly greater than for normally consolidated sand.

The use of SPT data may be similarly applied to calculation of foundation settlements. The SPT resistance may be converted into an equivalent CPT-q_c resistance and the foregoing approach applied to the particular foundation situation. Robertson and Campanella (1983) have developed a conversion diagram, which is included as Figure 4. In evaluating a soil modulus based on the equivalent CPT resistance, the same trends discussed previously apply.

Schmertmann (1970) presented an approximate method for calculating foundation settlements in sand based on strain distributions from elastic theory, and presented revisions to his suggested calculation method in 1978. The 1978 work includes an acknowledgment that for the same CPT resistance, the

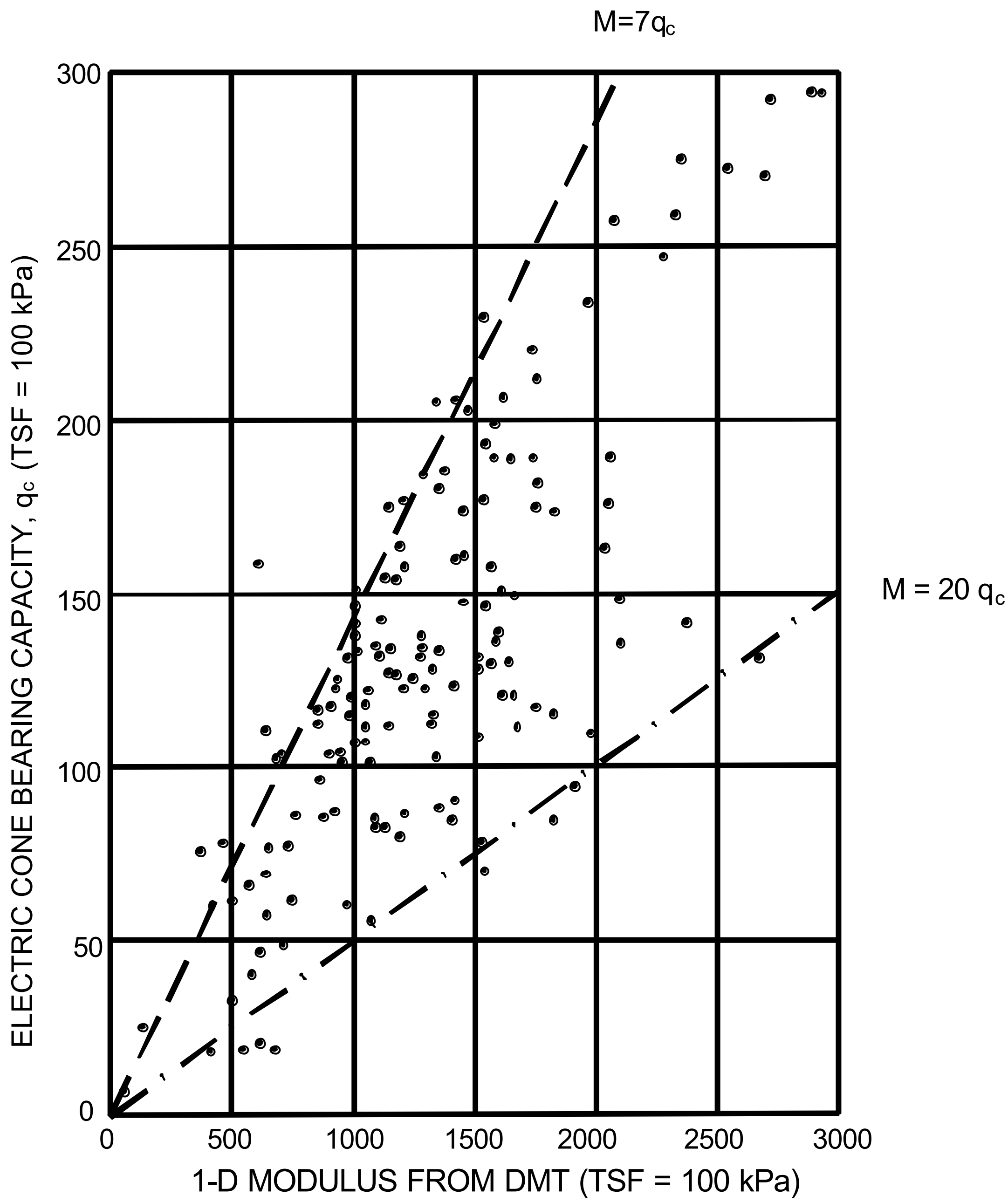

Figure 3 – Correlation of Static Cone Resistance and DMT Modulus for Overconsolidated Sands (Ref. 4)

modulus in an overconsolidated sand will be at least double that expected in a normally consolidated sand, resulting in half the settlement.

As a lower bound estimate for clean to silty/clayey sands treated by dynamic compaction, the results of site correlations by Schmertmann et. al. (1986) indicate a conservative relationship:

$M = 7\ q_c$

This is significant in that for normally consolidated sands, the conservative bound for α is only 3. Thus, following soil improvement, even if the CPT or SPT test values do not show an increase, the literature indicates that the calculated settlement values would be less than half of what would be expected before ground improvement for those same CPT or SPT values.

In summary, two factors may lead engineers to underestimate the effectiveness of ground improvement procedures when reviewing post-construction test data. First, test values generally increase significantly for weeks, months, or even years after the ground improvement is completed. Second, test results may overestimate settlements unless care is taken to use the proper correlations for over-consolidated soils and to calibrate the correlations to the observed degree of soil prestraining from site observations.

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TERRASYSTEMS INCORPORATED

39565 Cottage Grove Lane
Lovettsville, VA 20180
540-882-4130
FAX: 540-882-3866