Comparison of DMT and CPTU testing on a deep dynamic compaction project

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ABSTRACT: This paper describes Marchetti flat dilatometer testing (DMT) and piezocone penetration testing (CPTU) conducted for site characterization and quality assurance and control QA/QC on a major highway relocation project in Carver, Massachusetts (USA). Stretches of the new highway span existing cranberry bogs with thick peat deposits. Sheet piling was installed along both sides of the new highway alignment, and organic material was dredged from between the sheet pile walls. The area was then backfilled with sands. Since most of the sand was placed in a fairly loose state underwater, subsidence and liquefaction were potential problems. Therefore, deep dynamic compaction (DDC) was used to densify the fill.

An extensive in situ testing program was instituted to characterize site conditions prior to densification, and to assess the sufficiency of the DDC after treatment. The results of this study suggest that both the DMT and the CPTU are excellent tools for providing stratigraphic profiles. Both devices were particularly helpful in identifying pockets of organic soils (i.e., peat) that were not completely removed during the initial dredging operations. In terms of compaction QA/QC, comparisons were conducted between the dilatometer modulus (E_D), DMT horizontal stress index (K_D), DMT constrained modulus (M) and the corrected tip resistance (q_t) values from CPTU testing. The DMT and CPTU parameters showed similar trends regarding the zone of maximum soil improvement. The constrained modulus values determined from the DMT appeared to be the most sensitive indicators of densification effects.

1 INTRODUCTION

The Massachusetts Highway Department (MassHighway) is in the process of relocating a section of US Route 44 from the existing Route 44 in Carver, MA to US Route 3 in Plymouth, MA. The new roadway section will be a four-lane divided highway which will replace the current two-lane highway. The layout of the new highway extends across several existing cranberry bogs with underlying peat deposits. The peat deposits, which extended up to 9.1m (30ft) deep from the existing ground surface, required removal and replacement with on-site soils. Underlying the peat, the in situ soils are glacial outwash deposits consisting of loose to dense, coarse to fine sands with lenses of silt, clay and gravel and occasional cobbles and boulders.

Due to right-of-way considerations which severely restricted the space available for the roadway and environmental concerns regarding the remaining cranberry bog sections, traditional sloped earth embankments could not be used. Therefore, an innovative design incorporating sheet piling and mechanically stabilized earth (MSE) walls was used at the cranberry bog crossings. A typical cross-section of this design is presented in Figure 1.

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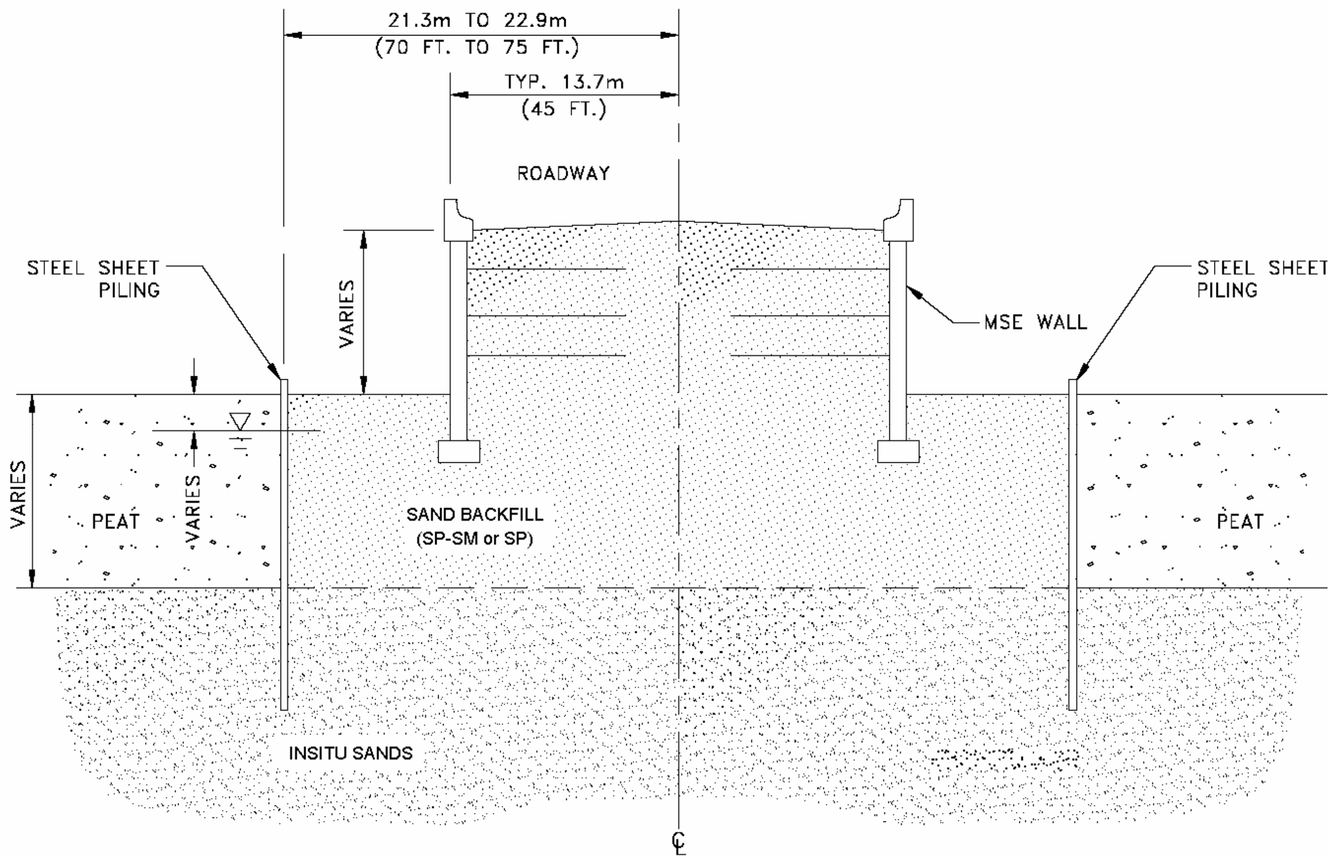

Figure 1. Typical Highway Cross-Section over Peat (Hajduk et al. 2004).

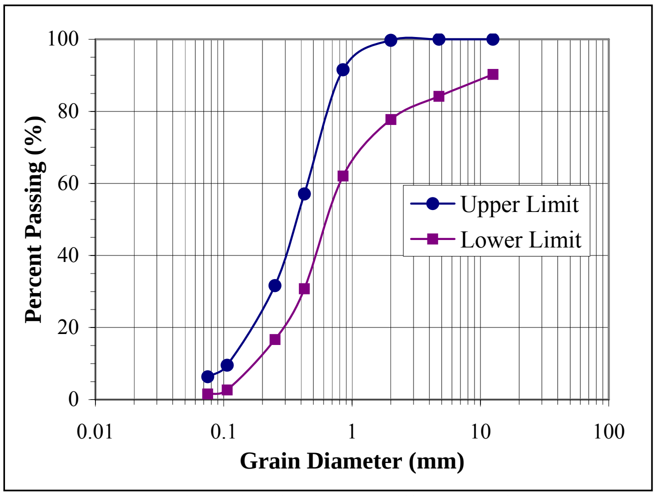

Figure 2. Grain Size Distribution of Fill Material

The construction project started with the installation of steel sheet piling through the pond/bog sections. The sheeting was located about 21.3 to 22.9m (70 to 75ft) off the proposed highway centerline. The removal of the peat between the steel sheeting was accomplished without dewatering using a crane outfitted with a dragline bucket. The thickness of the peat deposits ranged from about 1.52m (5ft) to about 9.1m (30ft). After removal of the peat deposits from within the sheet pile walls, granular fill was placed between the sheet piling by pushing the material forward (from the "land side") with a dozer. Fill was placed from the dredged mudline (which varied widely in elevation) to approximately Elevation 34.5 m (113 ft), which was roughly 1.6 m (5 ft) above the static groundwater table. A typical grain size distribution curve, as well as upper and lower limits of the range of grain size distribution of the fill material is provided in Figure 2. The fill is generally classified as poorly-graded sand (SP or SP-SM) according to the USCS classification system. The mean D_{50} is approximately 0.4 mm.

Since most of the sand was placed in a fairly loose state underwater, the potential for liquefaction was a concern. Therefore, deep dynamic compaction (DDC) was used to densify the fill. In situ testing was conducted before and after compaction to obtain baseline soil parameters and to assess the sufficiency of the DDC treatment.

2 DEEP DYNAMIC COMPACTION PROGRAM

Deep Dynamic Compaction is a process whereby soil is densified by repeatedly dropping a massive weight from a crane to impact the ground. Dynamic energy is applied on a grid pattern over the site, typically using multiple passes with offset grid patterns. The DDC process, described in detail by Lukas (1995), is generally very effective in densifying loose granular deposits. The degree of improvement is a function of the applied energy per unit cross-

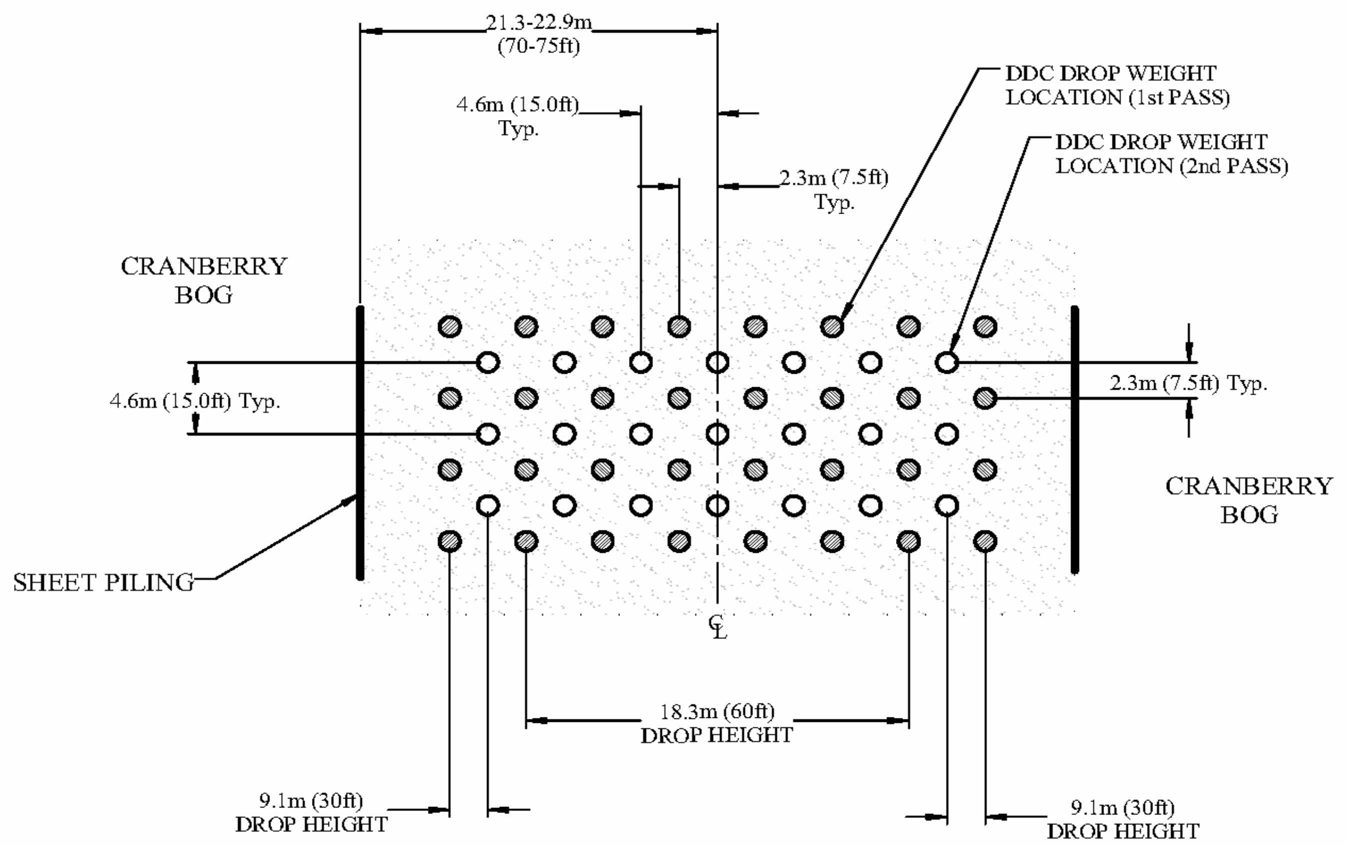

Figure 3. Typical DDC Layout Pattern (Hajduk et al. 2004).

sectional area, which is related to the tamper mass, the drop height, the number of drops and number of passes applied. The depth of improvement can be estimated using an empirical equation developed by Lukas (1995):

$$D = n(WH)^{0.5} \tag{1}$$

where D = depth of improvement in meters; W = mass of tamper in megagrams; H = drop height in meters; and n = empirical coefficient (for pervious soil deposits with a high degree of saturation, a value of 0.5 is recommended; for semi-pervious soils with a high degree of saturation, a value of 0.35 to 0.4 is recommended). The maximum improvement resulting from DDC is likely to occur within the zone from about 1/3 to 1/2 of the depth, D, calculated using equation (1).

The DDC planned for this project consisted of two passes over the site. In situ testing was conducted after the initial two passes of DDC, and additional compaction was applied to any areas where the initial compaction was not deemed sufficient. The layout for each pass consisted of a square pattern with a spacing of 4.6m (15ft). The second pass was offset within the center spacing of the 1st pass. A typical DDC layout for the project is presented in Figure 3. At each grid point location, a maximum of 9 drops were applied, with less drops applied if the depth of the crater exceeded approximately 1.52m (5ft). In some instances, the number of drops applied and/or the drop heights were reduced in response to lateral movement of the sheet pile walls and/or sand boils that occurred over portions of the site.

The DDC was conducted using a tamper weight of 13.15 Mg (14.5 tons). The tamper was a six sided lead weight with an approximate diameter of 1.52m (5ft) and a height of 0.90m (35 inches). Drop heights varied with distance from the roadway centerline. From the roadway centerline to 11.4m (37.5ft) from the roadway centerline, the DDC drop height was 18.3m (60ft). From a distance of 13.7m (45ft) from the roadway centerline and beyond, the drop height was reduced to 9.1m (30ft). The decrease in drop height was implemented to reduce the lateral stresses on the sheet piling from the DDC.

3 IN SITU TESTING PROGRAM

An extensive in situ testing program was carried out to provide baseline conditions of the fill and to assess the degree of compaction resulting from the deep dynamic compaction. The MassHighway construction specifications required an initial round of cone penetration testing to be conducted prior to the DDC, and a verification phase of CPTU after two passes of DDC. WPC conducted the QC testing under the construction contract with P.A. Landers. Refer to Hajduk et al. (2004) for additional details concerning the initial and verification cone penetration testing for the project. Supplemental in situ testing conducted before and after DDC under a research contract between MassHighway and UMass Dartmouth (UMD) included standard penetration testing (SPT), drive cone penetration testing (DCPT), dilatometer testing (DMT) and instrumented dilatometer testing (IDMT). The University of New Hampshire conducted the DMT and IDMT testing, and Applied Research Associates (ARA) conducted additional cone penetration testing after DDC for the MassHighway/UMD research project. This paper will focus on the results of the DMT and CPTU tests.

3.1 *Cone penetration testing*

The cone penetrometer consists of a steel probe with a conical tip that is pushed at a rate of 2 cm/sec into the soil in accordance with ASTM D5778. Cone penetrometers with a 15cm^2 projected tip area and a 225cm^2 friction sleeve were used throughout the testing. A porous piezo-element saturated in silicon oil is located behind the tip (type 2 for u_2) and detects in-situ penetration pore pressure during cone advancement,

The CPTU data acquisition system records the cone penetration resistance (q_c) and the local sleeve friction (f_s). Typically, CPTU tip resistance values are adjusted to account for porewater pressure effects due to unequal end areas, and the "corrected" values are expressed as q_t. From that information, the friction ratio (FR) can be calculated as equal to the local sleeve friction divided by the corrected tip resistance (f_s/q_t), typically expressed as a percentage.

The CPTU is beneficial in obtaining continuous profiles that provide information concerning soil stratification and variation in soil properties. Under the MassHighway construction specifications, the criterion for ground improvement was based on corrected CPTU tip resistance (q_t) values. The increase of CPTU tip resistance has been widely used to monitor the densification effect of various ground improvement techniques (Dove et al., 2000). Although the use of shear wave velocities measured during seismic testing (SCPT) has gained increased use for determining the degree of ground improvement, specifically the resistance to liquefaction (Andrus and Stokoe, 2000), it was not used for this project.

The ground improvement criterion was set as the minimum q_t value that would prevent liquefaction from the design earthquake. These minimum q_t values were established by WPC using the procedures developed at the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils and outlined by Youd and Idriss (2001). The design earthquake for the project has a 2% probability of exceedance in 50 years (i.e. 2,475 return period). According to the WPC liquefaction analysis, the minimum required corrected tip resistance ranged between 5.75 MPa to 7.66 MPa (60 tsf to 80 tsf).

3.2 *Dilatometer testing*

The DMT, introduced by Marchetti in 1975, consists of a stainless steel blade 95 mm wide, 15 mm thick with a 20-degree apex that is statically pushed into the ground for testing. On one face of the blade is a circular flexible steel membrane 60 mm in diameter. At typical test intervals of 15 to 30 cm the penetration is stopped and the membrane is expanded against the soil. Three pressure readings are generally recorded during a test and corrected for membrane stiffness: P_0 the pressure corresponding to the initial movement of the membrane, P_1 the pressure at a displacement of 1.1 mm into the soil and P_2 the pressure at which the membrane recontacts the body of the probe upon deflation. From the corrected pressures, Marchetti introduced the dilatometer indices I_D (material index), K_D (horizontal stress index), and E_D (dilatometer modulus), which can be used to empirically obtain various soil properties. For this project, the tests were carried out according to the ASTM procedure D-6635-01.

The dilatometer has been previously used in monitoring ground improvement by various means including deep dynamic compaction. Schmertmann et al. (1986) and Marchetti et al. (2001) suggest that since most densification work is aimed at reducing settlement, the constrained modulus from the DMT is a better indicator of improvement than relative density. The constrained modulus, M_{DMT} is empirically calculated using the DMT indices I_D, K_D and E_D. Consequently, this modulus inherently takes into account stress history and the state of stress. Their studies have also shown that increases in M_{DMT} are often twice that observed using q_c from the cone penetration test. In addition, settlement calculations based on the M_{DMT} have been in good agreement with observed settlements. The horizontal stress index, K_D is also a good indicator of improvement as densification translates into an increase in the lateral stress coefficient.

3.3 *In situ test results*

As part of the research contract between MassHighway and UMass Dartmouth, extensive testing was performed between stations 156+00 and 159+00 to enable comparison of different in situ test results. Figure 4 shows typical profiles of corrected pressures P_0, P_1 and P_2 for DMT-102 and DMT-104, located near the sheeting at station 156+00 and near the highway centerline at station

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159+00, respectively. It should be noted that for most of the DMT soundings, the fill from the ground surface to a depth of about 1.52m (5ft) was pre-bored with a hollow stem auger and then the DMT soundings were initiated at a depth of about 1.83m (6ft). This was done to avoid damage to the DMT blade, since the upper fill material was fairly dense as a result of construction traffic through the area and it also contained some gravel.

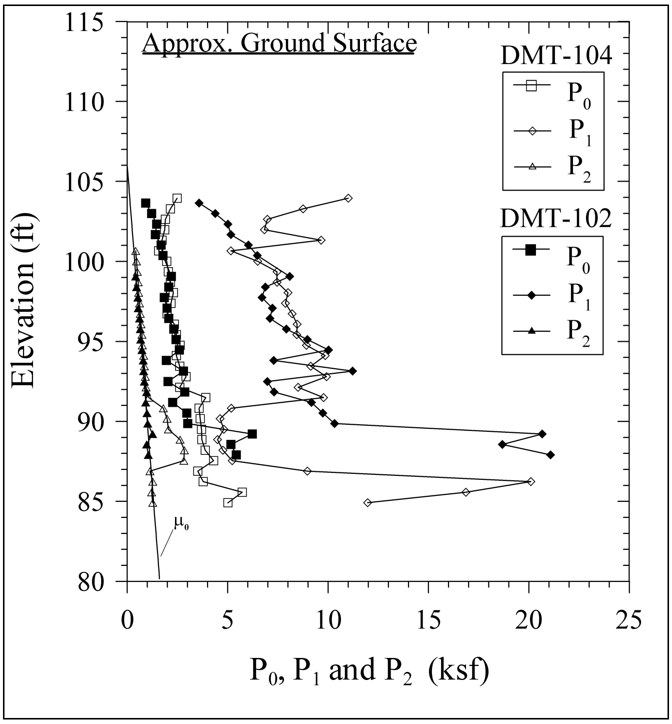

Figure 4. Corrected pressures P_0, P_1 and P_2 for DMT-102 and DMT-104

Below Elevation 92, profile DMT-104 shows a dramatic decrease in P_1 and an increase in P_2 (above hydrostatic conditions) indicating that a 1.22 to 1.52m (4 to 5-foot) layer of soft organic material was left in place prior to filling that area. The material above Elevation 101 appears to be stiffer at DMT-104 than at DMT-102. This is likely due to heavy construction traffic that occurred along the centerline during and after the filling operations. Although the two profiles are approximately 300 feet apart, the results (excluding the deeper soft layer and the upper compacted zone) seem to show that the filling process was minimally variable, especially with respect to P_0.

Profiles of CPTU data from approximately the same location as DMT-104 (station 159+00, centerline) are shown in Figures 5 and 6. The influence of the heavy construction traffic along the centerline is clearly reflected in the high CPTU q_t values within the upper 3.05m (10ft) of fill. Just below Elevation 92, the drop in tip resistance, and increases in pore pressure and friction ratio also suggest that a 1.22 to 1.52m (4 to 5-foot) layer of soft organic material was present below that elevation, just as in DMT-104.

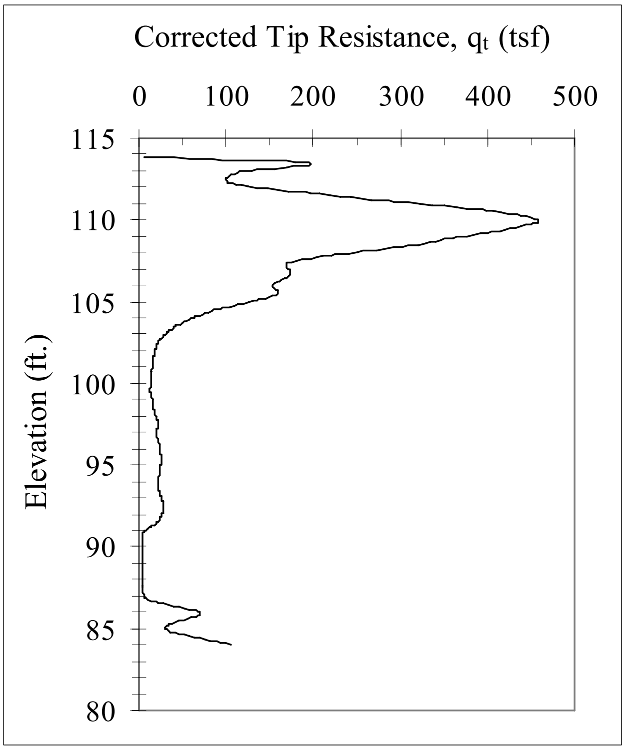

Figure 5. CPTU tip resistance profile at station 159+00

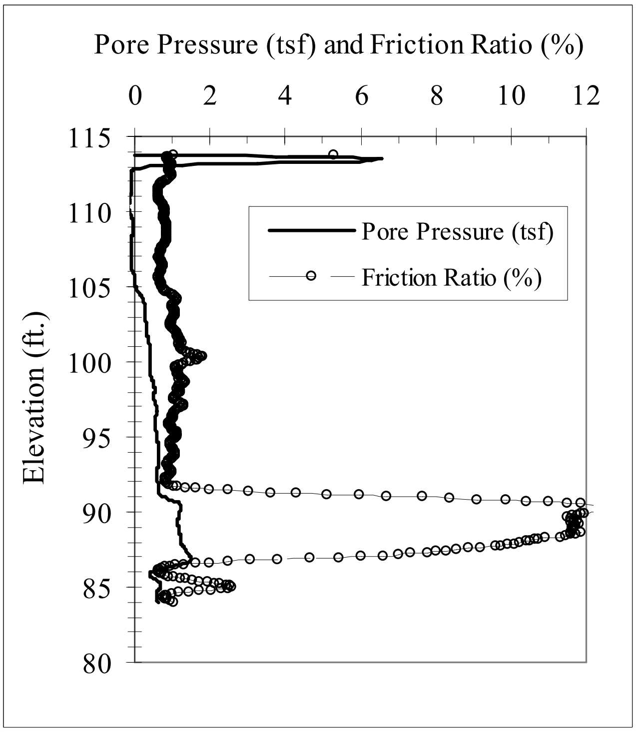

Figure 6. CPTU pore pressure and friction ratio profiles at station 159+00

The locations of DMT and CPTU tests near the sheeting (eastbound lane) at station 156+00 are shown in Figure 7. Figures 8 and 9 show profiles

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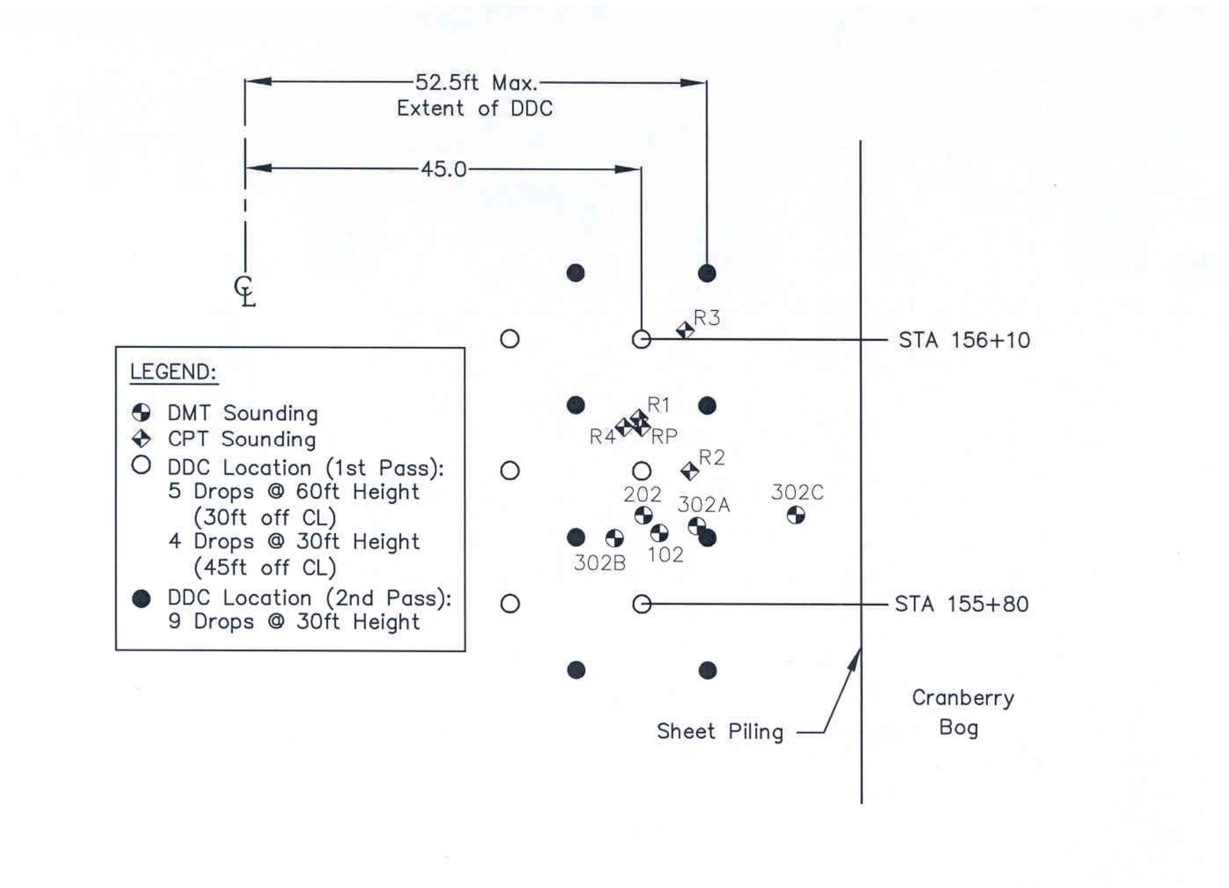

Figure 7. Locations of DMT and CPTU tests near station 156+00

of the horizontal stress index, K_D and the constrained modulus, M_{DMT} for the five DMT soundings; DMT-102 was conducted prior to compaction and the remaining four were conducted after compaction with DDC.

Figure 8 shows expected increases in lateral stress due to compaction with the most significant increases between Elevations 104 and 98. The maximum improvement appears to be approximately between Elevations 100 and 102. Below that depth, the horizontal stress increase attenuates, but still remains higher than the pre-compaction stage except at profile DMT-302C. At that location, it is possible that the lack of increase in horizontal stress resulted from two factors: (1) DMT-302C was located outside of the DDC limits and (2) significant lateral movement of the sheet pile wall occurred during compaction, which likely reduced the horizontal stresses closer to the wall. Inclinometer data obtained at station 156+25 indicated that the sheet pile wall deflected outward about 76 cm (30 inches) near the top of the wall. Outward deflections decreased linearly to about 23 cm (9 inches) at a depth of 8.54m (28ft).

Figure 9 also indicates that the constrained modulus increased substantially between Elevations 104 and 98, especially at DMT-202 and DMT-302B. At those locations, the maximum improvement also appears to be approximately between Elevations 100 and 102, where post-DDC values of constrained modulus are about 15 to 20 times larger than the pre-compaction values. Another trend noted in Figure 9 is that the increase in constrained modulus values is less as one moves farther away from centerline towards the sheet piling.

As illustrated in Figure 7, DMT-302C was located outside of the DDC limits, so the smaller increases in modulus may be due to little direct energy from the DDC being delivered to that area. At profile DMT-302A, however, the applied energy was roughly equivalent to that applied at DMT-302B, and greater than that applied in the vicinity of DMT-202. Since the M_{DMT} values in DMT-302A were lower than those in DMT-202, it is likely that the lateral movement of the sheet pile wall that occurred during compaction had a pronounced effect on the DMT constrained modulus values.

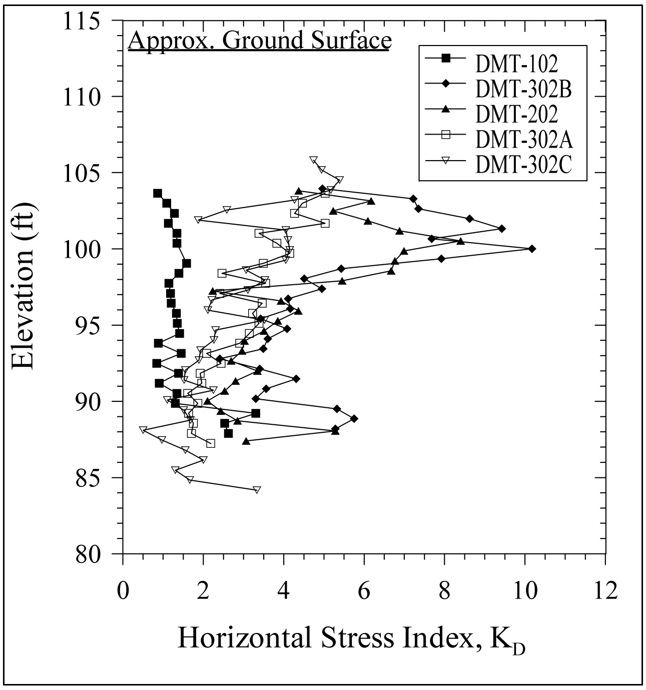

Figure 8. Profiles of horizontal stress index, K_D, near station 156+00

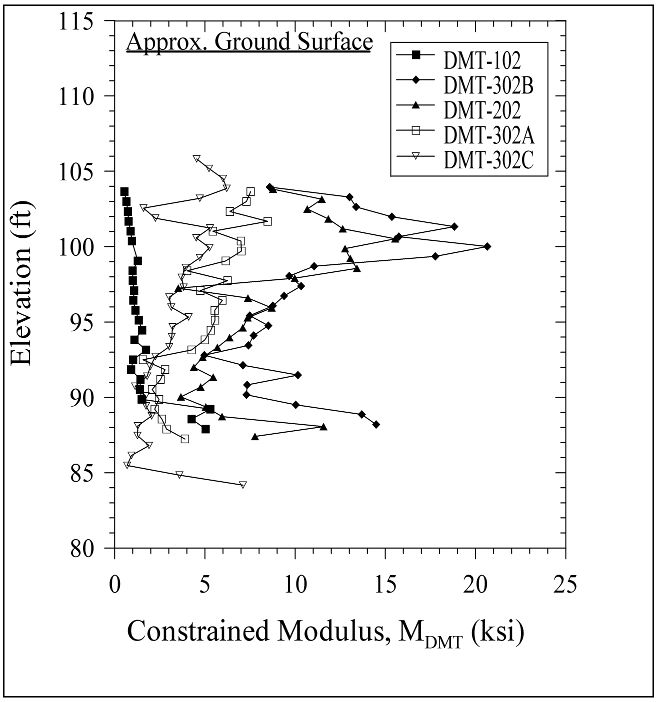

Figure 9. Profiles of constrained modulus, M_{DMT}, near station 156+00

Profiles of tip resistance values in CPTU soundings conducted near the sheeting (eastbound lane) at station 156+00 are shown in Figure 10. Sounding RP was conducted prior to compaction, and the remaining four soundings were conducted after compaction with DDC. It is interesting to note that, within the upper 0.9 to 1.2m (3 to 4 feet) of fill, the post-DDC q_t values shown in Figure 10 are actually less than the pre-DDC values. Ground improvement in this zone was not expected, since DDC severely affects near surface soils, resulting in a looser surface after the process is completed.

Below Elevation 110, Figure 10 shows expected increases in tip resistance due to compaction. The most significant increases are approximately between Elevation 108 and Elevation 98, which is consistent with the DMT data. The maximum improvement zone appears to be approximately between Elevations 103 and 105.5, which is slightly higher than the maximum improvement zone indicated by the DMT data. Within the zone of maximum improvement, the post-DDC q_t values are about 5 to 8 times larger than the pre-compaction values.

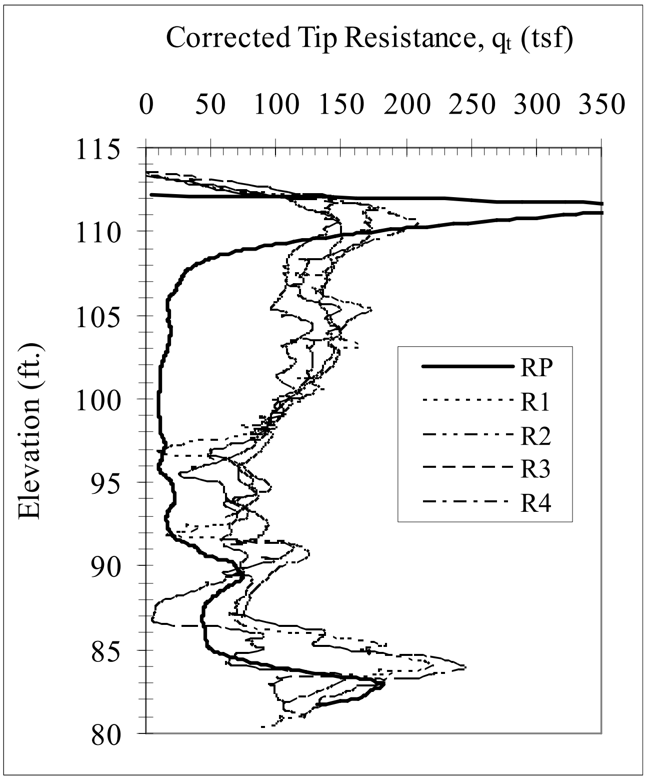

Figure 10. Profiles of CPTU tip resistance, q_t, near station 156+00

Based upon a 13.15 Mg tamper and a 9.1m (30ft) drop height, the depth of improvement computed from equation (1) using a coefficient, n, of 0.5 is 5.5m (18ft). The corresponding maximum improvement would then be predicted to occur within a zone between 1.8 and 2.7m (5.9 and 9.0 feet) below ground surface (i.e., Elevation 107 to 104, respectively). Both the CPTU and DMT data indicate that the depth of improvement extended slightly below that predicted by equation (1), and that the zone of maximum improvement may also be slightly deeper than that predicted using equation (1).

In contrast to the DMT horizontal stress index and constrained modulus data, the CPTU q_t data does not clearly indicate decreases in tip resistance as one moves farther away from centerline towards the sheet piling. This suggests that the DMT horizontal stress index and constrained modulus values

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are more directly related to lateral stress conditions than the q_t values measured in CPTU testing. Given the direction of the measurements, it would stand to reason that the DMT readings would be more sensitive than the CPTU to changes in horizontal stresses such as those caused by lateral wall movements.

4 CONCLUSIONS

The results of this study suggest that both the DMT and the CPTU are very useful tools for providing stratigraphic profiles as well as parameters for QA/QC on in situ densification projects. During preliminary site investigations, the DMT and CPTU were particularly helpful in identifying pockets of organic soils (i.e., peat) that were not completely removed during the initial dredging operations. After compaction, the CPTU tip resistance values and the horizontal stress index and constrained modulus values obtained from the DMT were all good indicators of densification effects. The DMT constrained modulus values appeared to be the most sensitive indicators of densification effects.

Data from both the DMT and the CPTU indicate that the depth of improvement resulting from DDC extended slightly beyond the depth predicted using equation (1), and that the zone of maximum improvement may also be slightly deeper than that predicted using equation (1).

And finally, the trends observed in the DMT data presented herein illustrate another interesting phenomenon. This site was somewhat unusual in that DDC was conducted between rows of sheet piling spaced about 46m (150ft) apart, parallel to the highway centerline. Based upon the profiles of constrained modulus shown in Figure 9, it appears that lateral movement of the sheet piling that occurred during compaction reduced effectiveness of the DDC in areas adjacent to the sheet piling.

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APPENDIX: UNIT CONVERSIONS

1 foot (ft) = 0.3048 m
1 kip/in^2 (ksi) = 6.895 MPa
1 kip/ft^2 (ksf) = 47.88 kPa
1 British ton-force/ft^2 (tsf) = 95.76 kPa