

Dilatometer & Cone Penetration Tests on Peat Soil in Carver, Massachusetts

A well planned program of in-situ testing can lead to greater certainty of below-ground conditions.

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In 2005, U.S. Route 44 was relocated in southeastern Massachusetts. Parts of the new highway alignment spanned across existing ponds and cranberry bogs in the town of Carver, located about 64 kilometers (40 miles) southeast of Boston. The realigned roadway is a four-lane divided highway with a typical median width of approximately 20 meters (66 feet). Environmental concerns dictated that sheet piles needed to be placed at roadway sections that crossed, or were adjacent to, bogs in order to excavate and replace the underlying organic soils (peat) and construct the embankments of the roadway.

Peat consists of organic residues formed through the decomposition of plant and ani-

mal constituents under aerobic and anaerobic conditions associated with low temperatures and geologic effects such as glacial ice. Peat exhibits poor strength and undergoes large deformations over a long period of time. As a result, peat and organic soils are characterized as being among the worst kinds of foundation material and are associated with low bearing capacity, high compressibility and long-term settlement.

In some places at the project site, organic soils were found immediately below the surface of the cranberry bogs. These wetland areas consisted primarily of fibrous peat, and many areas at the site required embankment fills to accommodate the realigned highway. A flat plate dilatometer testing program was one of the field-testing techniques that were employed on this project to provide a preliminary investigation of the site before removing the organic soils. Since the site was not dewatered during backfilling operations (after the removal of the organic soils), it was suspected that these backfill materials would be in a loose state, making them susceptible to liquefaction.



FIGURE 1. A view of a length of the sheet piling.

Mechanically stabilized earth (MSE) walls supporting the raised highway had also to be placed above the sheet piling. Accordingly, deep dynamic compaction (DDC) was planned for these areas. Thus, in order to verify the effectiveness of the DDC, cone penetration testing was designated (before and after the DDC) for these areas. Figure 1 shows a picture of the sheet-piling wall, which supported the backfill material of Route 44 where it crossed through a cranberry bog. Figure 2 shows the MSE wall above the sheet piling.

Subsurface Conditions

Extensive subsurface investigation of the project area showed that soil type and density were relatively consistent throughout the site and the associated wetland areas. The soil profile consisted primarily of a thin surface sand layer underlain by fibrous peat over a fine to coarse sand layer. The thickness of the peat deposits ranged from non-existing to 8 meters

(26 feet) and the groundwater table was near the ground surface. Figures 3 and 4 show a plan view of part of Route 44 and a longitudinal section for a 427-meter (1,400-foot) long segment of the constructed route, respectively. The section presented in Figure 4 includes ten borings and numerous probes that outlined the contour of the peat layer. For instance, the blow counts from the standard penetration test (SPT) at Station 151 suggested that the internal friction angle, ϕ , ranged approximately from 29 degrees near the ground surface to 33 degrees at a depth 4 meters (13 feet) below the ground surface.

Flat Dilatometer Testing & Results

The flat plate dilatometer (DMT) is an in-situ test device with specific advantages that can be useful on certain projects. However, it is not yet used as frequently as the cone penetration test in engineering practice, especially in the United States. The test was developed in Italy

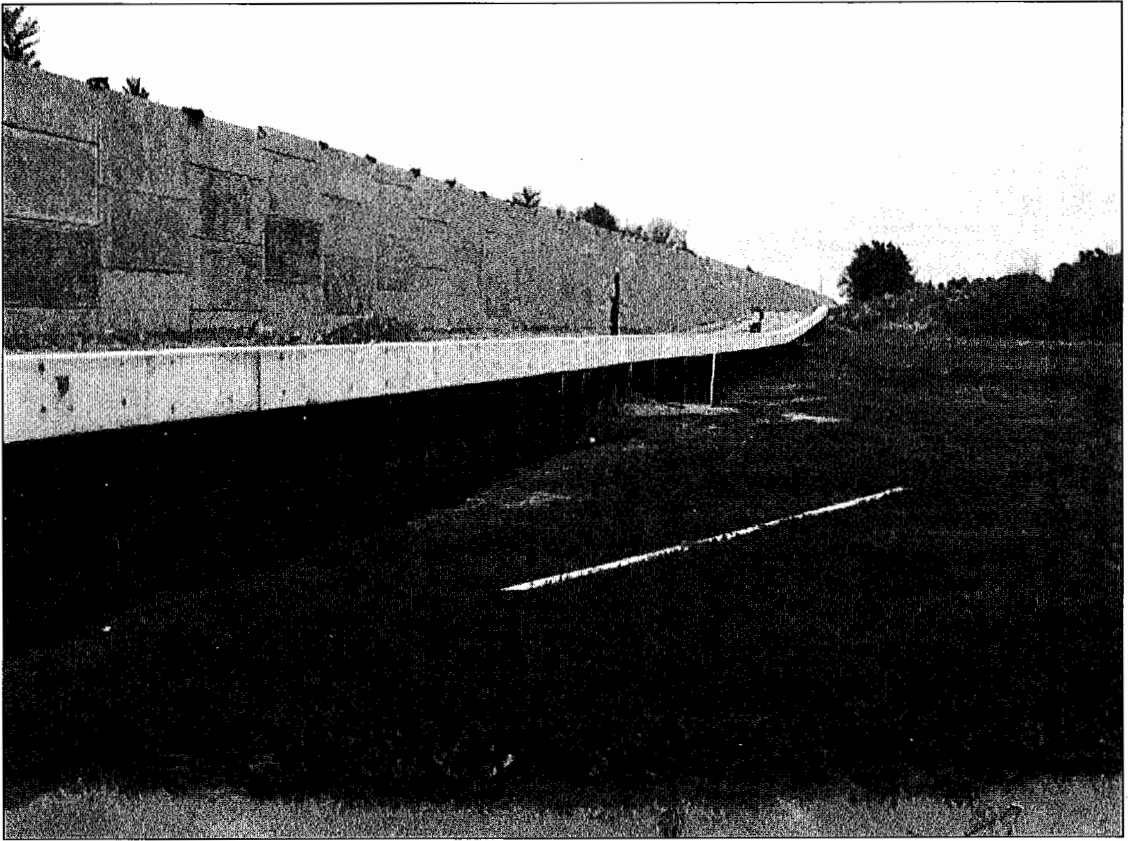


FIGURE 2. The MSE wall above the sheet piling.

by Marchetti.¹ The front view of a DMT consists of 60-millimeter (2.4-inch) diameter circular membrane mounted on a flat face of 95-millimeter (3.7-inch) wide by 220-millimeter (8.7-inch) long and with a thickness of 14 millimeters (0.5 inches). A DMT is pushed into the soil, often using the same rig that is used to push a cone penetrometer. However, a DMT cannot be pushed through very dense, hard or gravelly soils that might damage the membrane or buckle the blade. The membrane is expanded after the blade has been pushed to the desired depth. Dual gauges provide measurements of the pressure required to expand the DMT membrane. The first measurement is of the pressure required to lift the membrane off its seat (otherwise known as "lift off"). The second measurement is of the pressure required to expand the membrane by 10 millimeters (0.4 inches). These pressures are used for soil profiling, and for estimating the in-situ lateral stress and lateral soil stiffness. Results from

Station 151EI were selected because it was conducted in granular soil, while at Station 155EI tests were performed on both organic and inorganic soils. Figure 5 shows the dilatometer profiles of Stations 151EI and 155EI. Also, Figure 6 and 7 show the correlation indices from the DMT data from Stations 151EI and 155EI, respectively. Using the tentative classification chart by Boghrat,² the soil profile of Station 151EI was determined to consist of silty sand and sand throughout the entire depth, while the soil profile of Station 155EI was determined to consist of silty sand up to a depth of 4 meters (13 feet) below the ground surface, which is underlain by a 0.5-meter (1.6-foot) thick layer of peat. The preceding results were in agreement with the subsurface conditions depicted in Figures 3 and 4.

DMT Results

The results of DMT testing can be interpreted using the following equations:^{3,4}

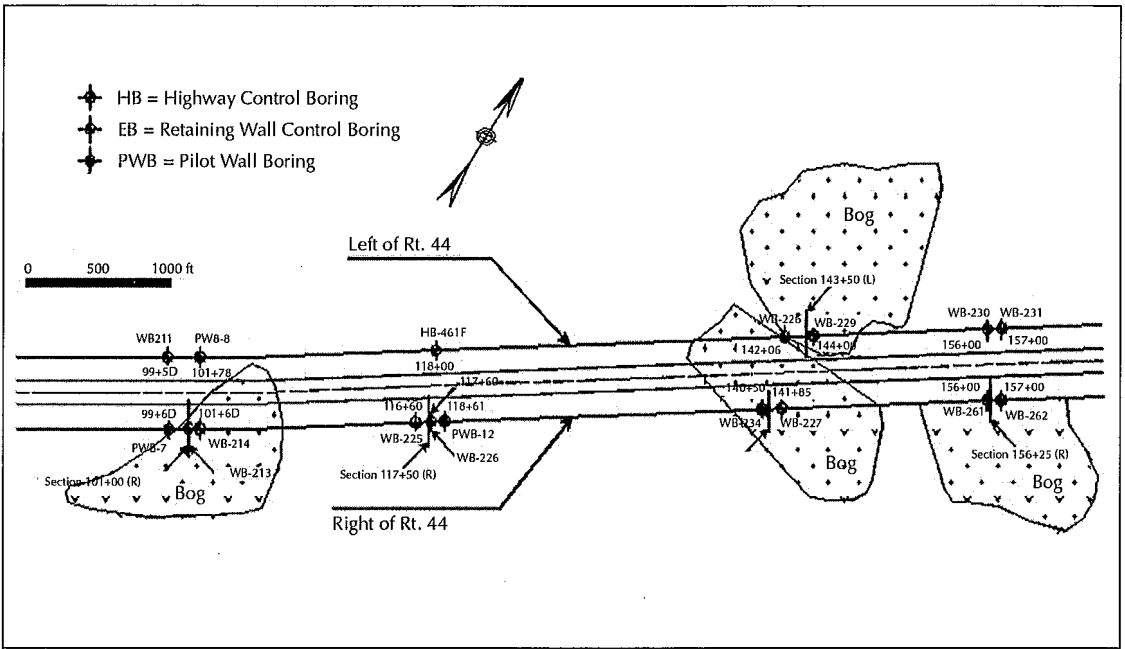


FIGURE 3. Plan view of Rt. 44 in Carver, including instrumented sections and related borings.

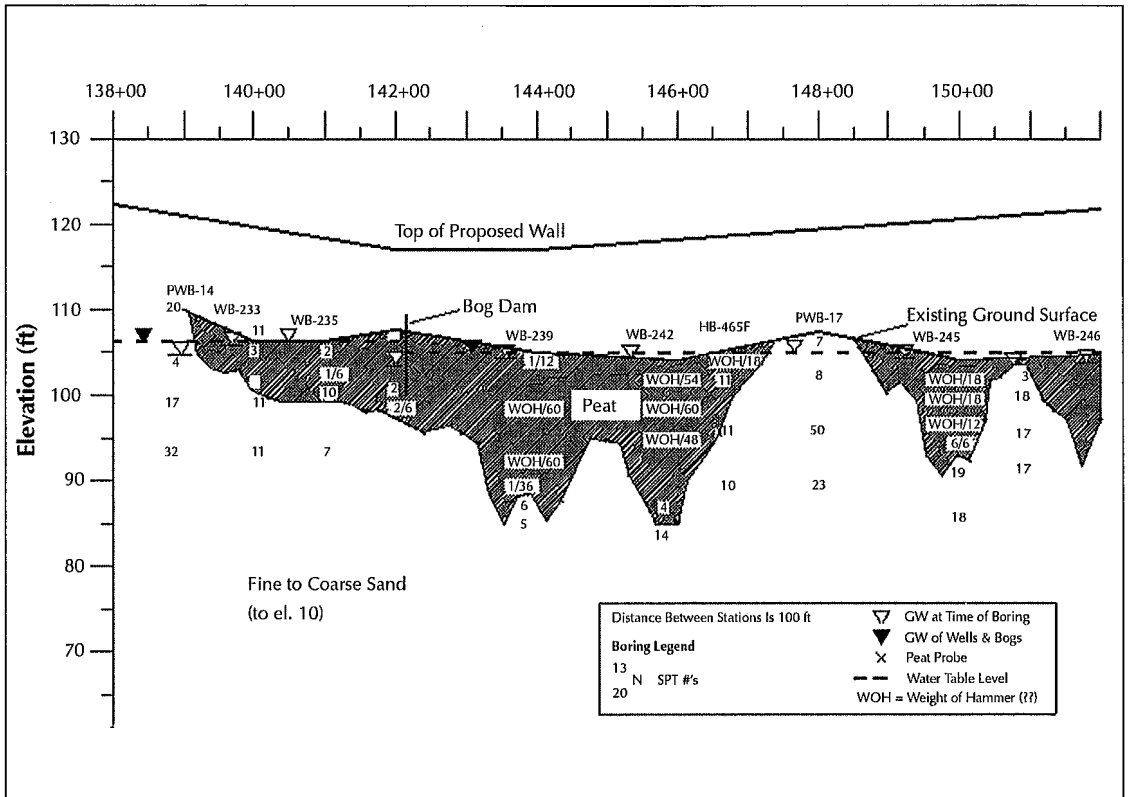


FIGURE 4. Subsurface conditions.

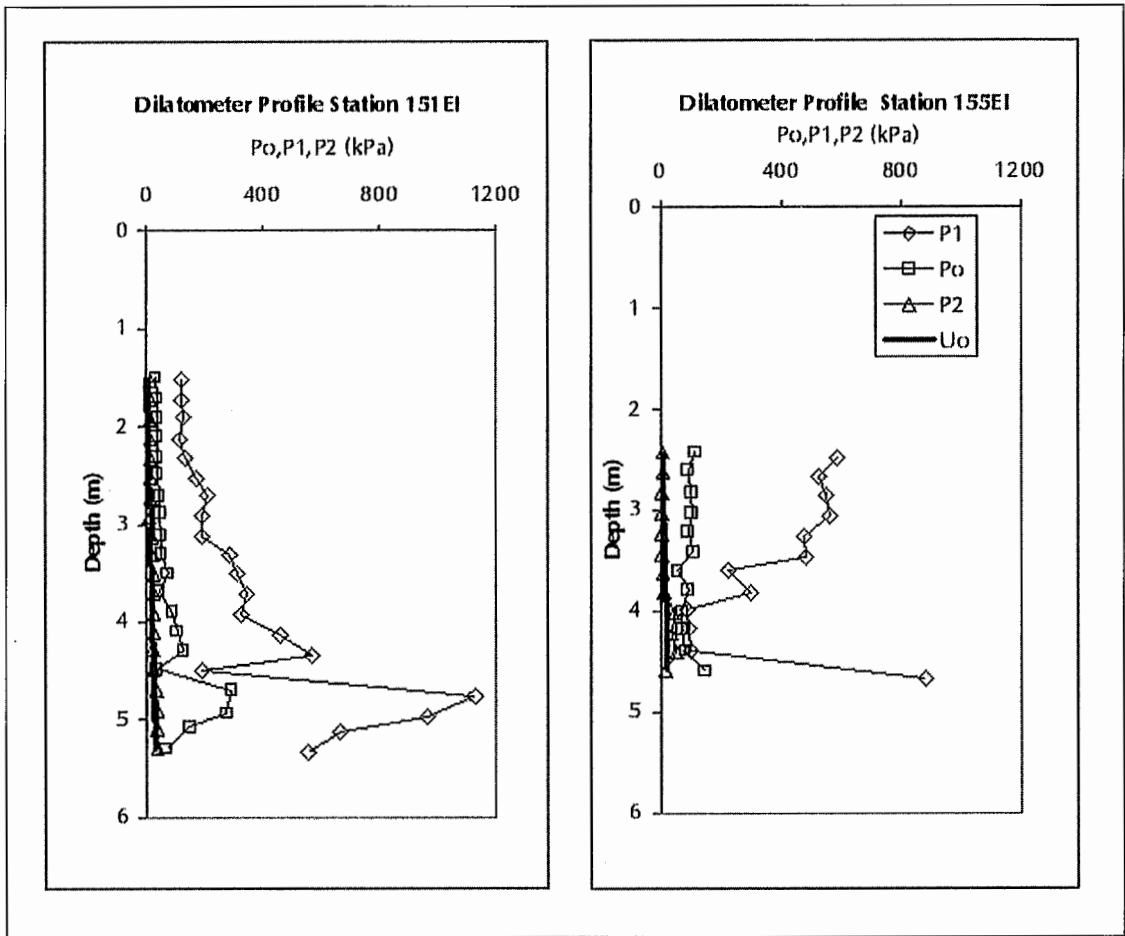


FIGURE 5. Dilatometer profiles for Stations 151EI and 155EI.

$$k_0 = (k_D/2)^{0.47} - 0.6 \quad (1)$$

(For k_0 greater than or equal to 0.3, which is valid for soils that are not subjected to cementation.)

$$k_0 = 1 - \sin \phi \quad (2)$$

Where:

- k_0 is the coefficient of lateral earth pressure at rest;
- k_D is the lateral stress index from DMT correlations; and,
- ϕ is the internal friction angle.

For Station 151EI at a depth 4 meters (13 feet) below the ground surface, the internal friction angle was approximately 36 degrees, which came in close agreement to the friction

angle interpreted from the SPT results at the same location.

For Station 155EI, since Equation 1 is only valid for k_0 greater than or equal to 0.3, this condition eliminated the evaluation of k_0 within the peat layer where k_0 ranged from 0.2 to 0.25. The internal friction angles of the sandy soil above the peat ranged from 6 degrees at the ground surface to 36 degrees at a depth of 4 meters (13 feet), which also came in close agreement to the interpretation from SPT results at the same location.

The small difference in the results between the DMT and SPT tests is acceptable for most practical purposes and may be related to the fact that k_D is mainly controlled by two factors: relative density, D_r , and horizontal stress, σ_{ho} . Therefore, in order to evaluate in a reliable way k_0 as a function of k_D , it is necessary to

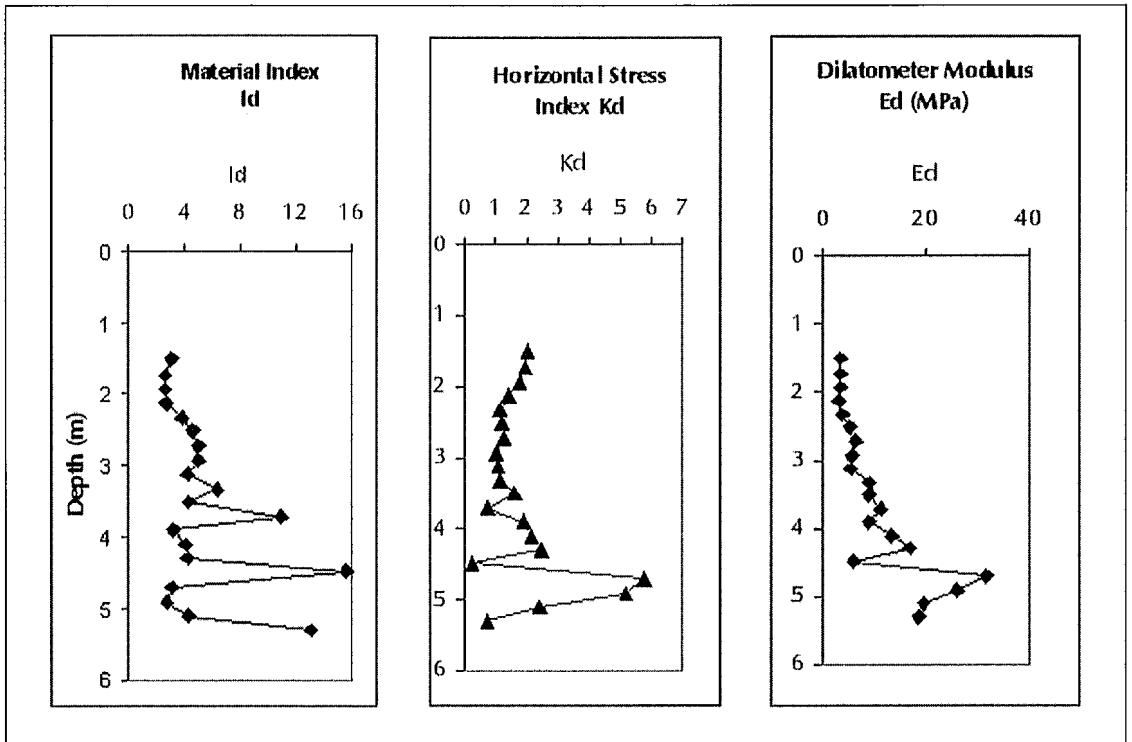


FIGURE 6. Correlation indices for DMT data for Station 151EI.

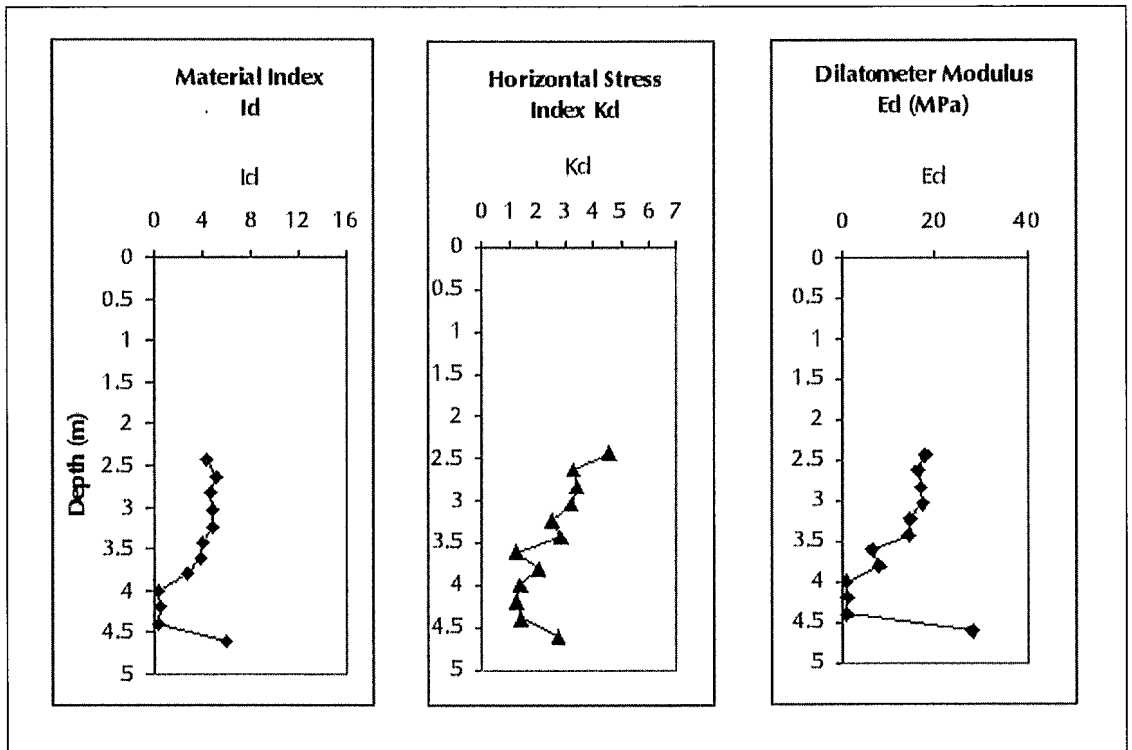


FIGURE 7. Correlation indices for DMT data for Station 155EI.

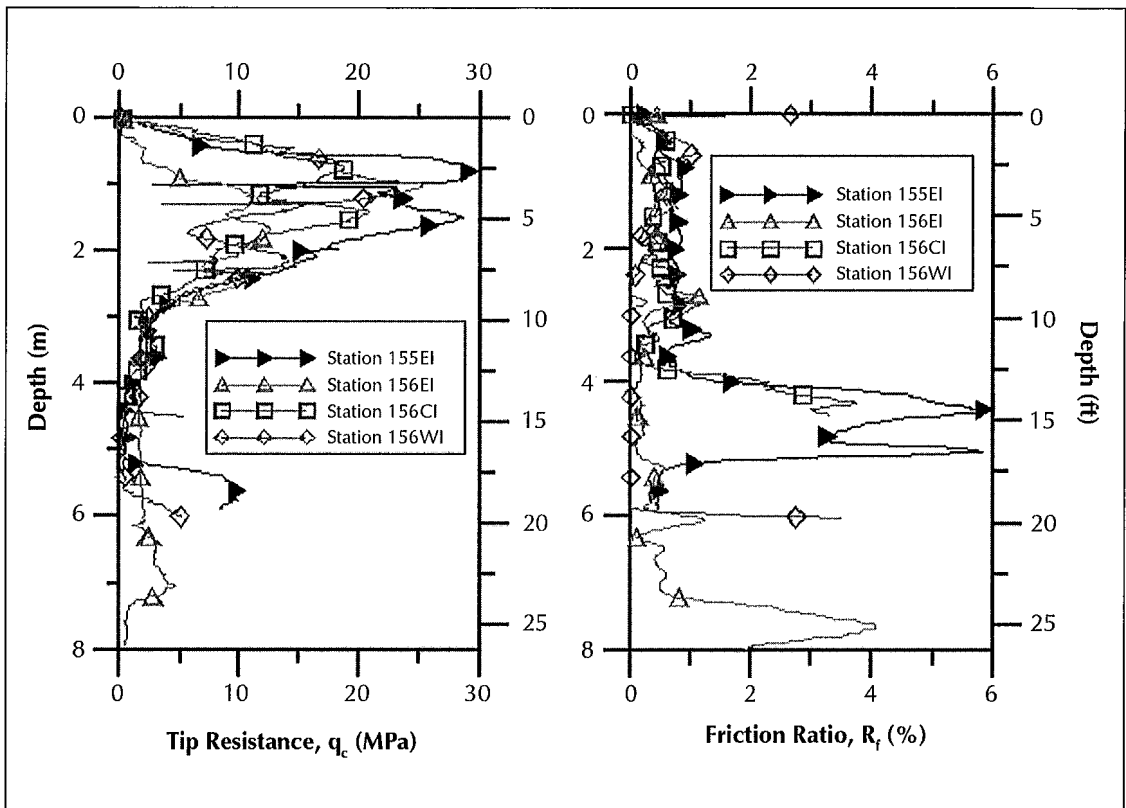


FIGURE 8. A CPT profile of backfill material prior to compaction.

separate the influence of D_r on k_D from that of σ_{ho} . The dilatometer modulus, E_D , can be calculated as a function of Young's modulus of elasticity, E , and Poisson's ratio, μ .³

$$E_D = E/(1 - \mu^2) \quad (3)$$

Plugging the data from Station 155EI at a depth 4 meters (13 feet), which was from the peat layer, into Equation 3 yields a value for Young's modulus of 650 kPa (94 psi) when Poisson's ratio is 0.25. The value of Young's modulus reported in a previous study for an undisturbed sample that was oriented horizontally of Carver peat (at an effective confining pressure of 35 kPa [5 psi] from triaxial isotropic consolidated undrained tests) was 800 kPa (116 psi).⁷ It seems that DMT correlations underestimate the Young's modulus by 18 percent. Some investigators (such as Marchetti and Jamiolkowski *et al.*) concluded that, for normally consolidated specimens of sand, the DMT correlations have a tendency to

underestimate the constrained modulus, M , by an amount of 7 percent (± 26 percent). The constrained modulus, M , is related to Young's modulus, E :

$$M = \frac{E(1 - \mu)}{(1 + \mu)(1 - 2\mu)} \quad (4)$$

Cone Penetration Testing & Results

Piezocone penetration tests (PCPTs) were conducted and performed in accordance with ASTM D5778 (the Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils). In this test, an instrumented cone is pushed hydraulically through the soil while continuous readings are recorded on a portable computer. The instrumented cone has a cross-sectional area of 10 square centimeters (1.6 square inches), with a 60-degree conical tip. The cone is advanced through the ground at a constant rate of 2 centimeters per second (0.8 inches per second), and pore water pressure measure-

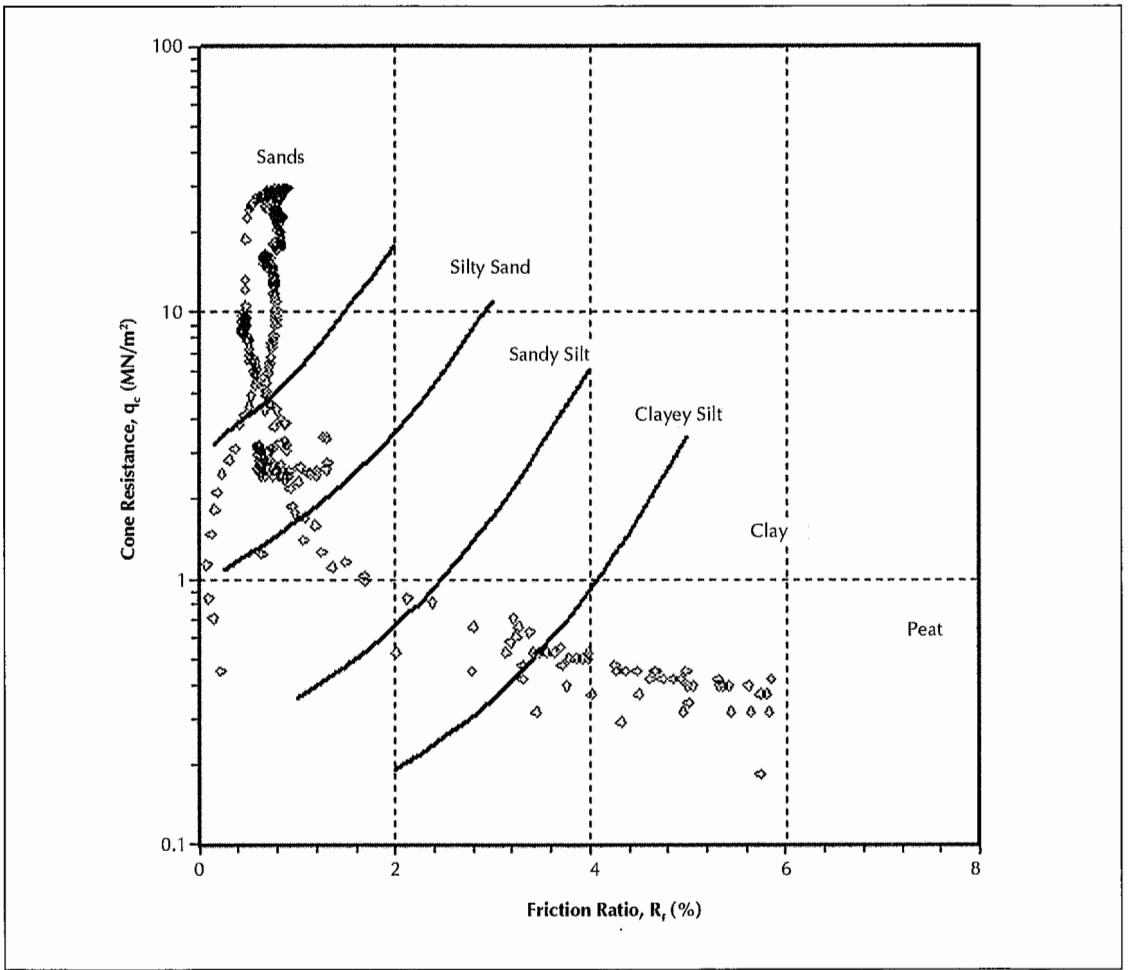


FIGURE 9. Backfill classification for Station 155EI.

ments are taken directly behind the tip. No soil samples for this location were gathered using the CPT subsurface investigation technique. However, SPT tests were performed for comparison and validation purposes. PCPTs were conducted using a self-propelled, self-anchoring, ATV-style testing rig. The reaction force for the test was achieved by using helical anchors that were hydraulically inserted approximately 2 meters (6.5 feet) into the ground. PCPTs were conducted at approximately 30-meter (100-foot) intervals following the fill placement. A PCPT was used to verify the quality and completeness of the backfill material and its strength prior to compaction. Figure 8 depicts the CPT profile of the backfill material prior to the deep dynamic compaction for some stations of the project. Using

the classification guidelines suggested by Robertson and Campanella, the type of soils that were encountered at the backfill material can be predicted.⁹ For instance, Figures 9, 10, 11 and 12 represent the backfill classifications for Stations 155EI, 156EI, 156CI, and 156WI, respectively.

PCPT Results

The relative density is correlated to cone resistance as follows:¹⁰

$$D_r = -98 + 66 \log^{10} \frac{q_c}{(\sigma'_{vo})^{0.5}} \quad (5)$$

Where:

q_c is the cone resistance; and,
 σ'_{vo} is the effective vertical overburden stress.

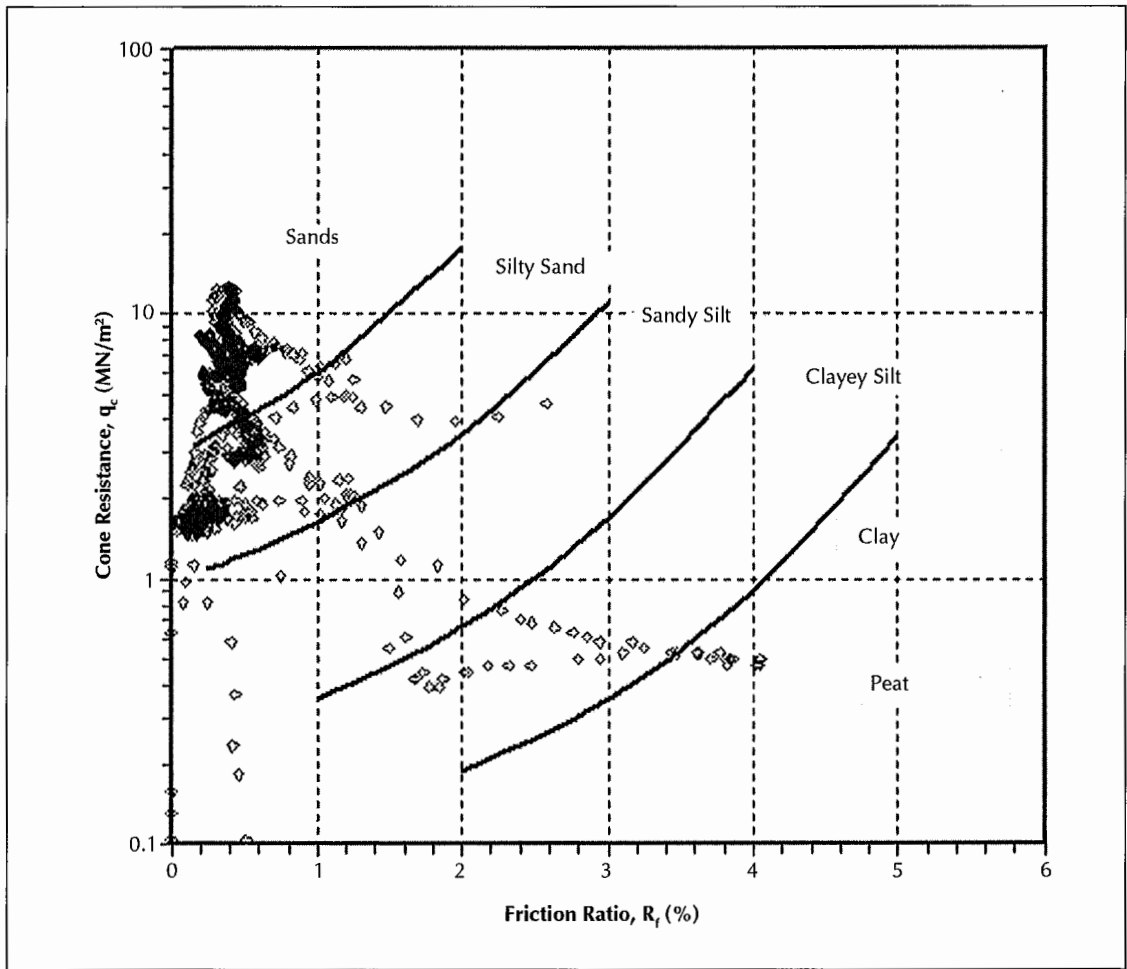


FIGURE 10. Backfill classification for Station 156EI.

Plugging the data from Figure 8 into Equation 5 yields results that indicate that the granular soils that were encountered in the logs are in loose to medium condition. These conditions were expected during this stage of construction.

Using the results shown in Figures 9 through 12, the combination of the values of the cone resistance and the friction ratio indicated the existence of very soft soils in some areas. It was expected from the PCPT data profiles after the backfilling that whole layers would be classified as sand and sandy silt in a loose state. However, it is surprising to note that some of the backfill material was classified as a soft soil and was, in fact, peat. These results are not those that would be expected for suitable backfill material. Accordingly, an SPT was employed to collect samples of the

backfill material throughout the site. The PCPT test data and the SPT results suggested that some existing peat layers were not completely removed prior to the backfilling, or that the backfilling materials were mixed with some pockets of peat during the replacement operations. At some locations it had been observed that if the distance between the stations were 15 meters (50 feet), different layers will be encountered, including peat. For example, at Station 159+00WI at a depth 5.8 to 6.5 meters (19 to 21 feet) there was black organic silt and trace fine to coarse sand gravel; however, at the same depth for Station 159+50WI there was brown coarse to fine sand, some coarse to fine gravel (which indicates that some of the backfilling material was mixed with organic soils).

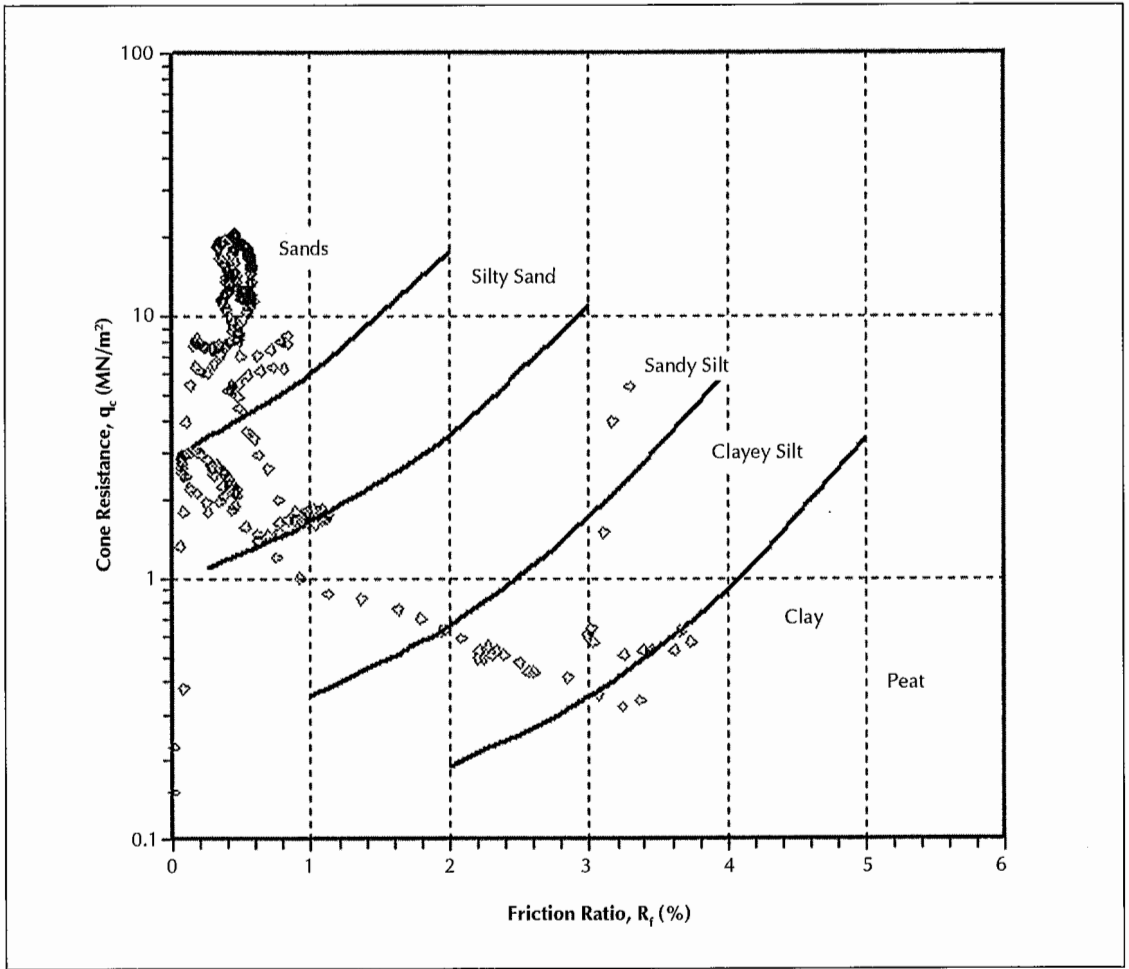


FIGURE 11. Backfill classification for Station 156CI.

Figures 13 and 14 show examples of the locations where unsuitable material was detected in the cross-sections of Stations 155+00 and 156+00, respectively. The soil classifications based on the PCPTs were found to be in agreement with the actual sampling obtained via SPT investigations. Subsequent to these investigations, the peat layers were removed from the designated area and replaced with suitable material prior to the DDC.

Undrained Shear Strength of Peat

Since the data of PCPTs on peat were available, it was possible to evaluate the undrained shear strength, taking into consideration that the remaining in-place layers of peat had been subjected to disturbance during the excavation and backfilling operations. The

undrained shear strength of the peat layers was evaluated from total cone resistance as:

$$S_u = (q_t - \sigma_{vo})/N_{kt} \quad (6)$$

Where:

- S_u is the undrained shear strength;
- q_t is the total cone resistance;
- σ_{vo} is the vertical stress; and,
- N_{kt} is an empirical cone factor.

N_{kt} values have been reported to vary between 4 and 30 in actual practice. Several factors — such as plasticity, stress history, stiffness, sensitivity and fabric — are known to be the cause for such wide variation. A value between 15 and 20 was chosen for this study as a preliminary estimation. Figure 15 shows the

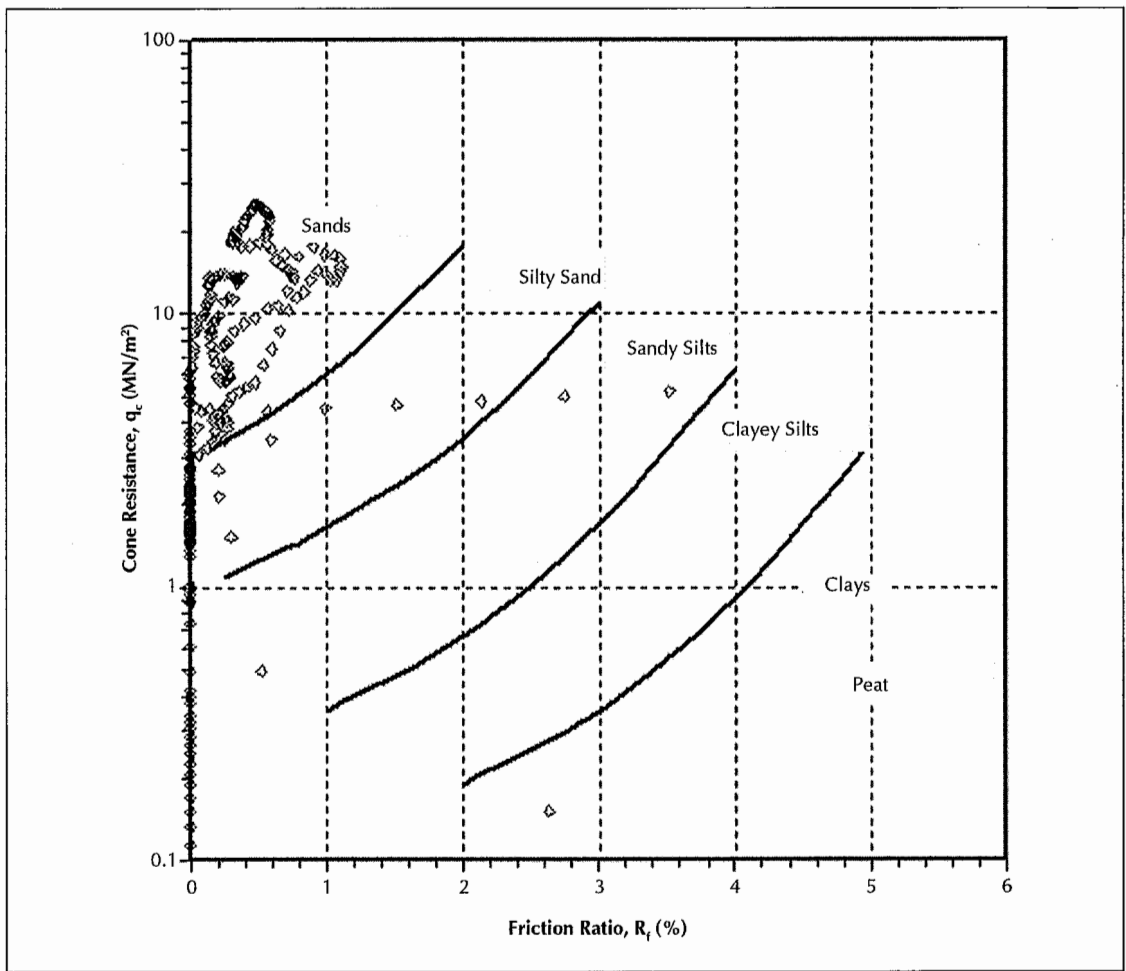


FIGURE 12. Backfill classification for Station 156WI.

undrained shear strength of Carver peat versus depth for Stations 155EI and 156CI, respectively. The values of the undrained shear strength obtained experimentally from triaxial tests were plotted on the same graph for comparison.^{7,11} The undrained shear strength of peat obtained from triaxial tests on high-quality undisturbed samples was in good agreement with the values predicted from the CPT results.

Conclusion

On this project, the flat plate DMT has predicted soil profiles with results that agreed very well with those obtained via common field tests such as SPT and CPT.

Using a DMT permitted the detection of the existence of a thin layer of soft soil (peat) and

predicted the value of Young's modulus at 18 percent less than the actual value obtained via laboratory test on an undisturbed high-quality sample. The evaluation of the coefficient of lateral earth pressure at rest was not valid for peat since Equation 1 is restricted to materials not subjected to cementation. The variations in the results between laboratory tests and field tests may refer to the degree of composition of peat (humification), which is different from one sample to another and from one place to another, and which results in different degrees of cementation. Also, the fibrous effects might alter the results.

A PCPT is an effective tool in obtaining a continuous soil profile. The decision that was made on this project to perform a PCPT on a before-and-after basis was successful. Before

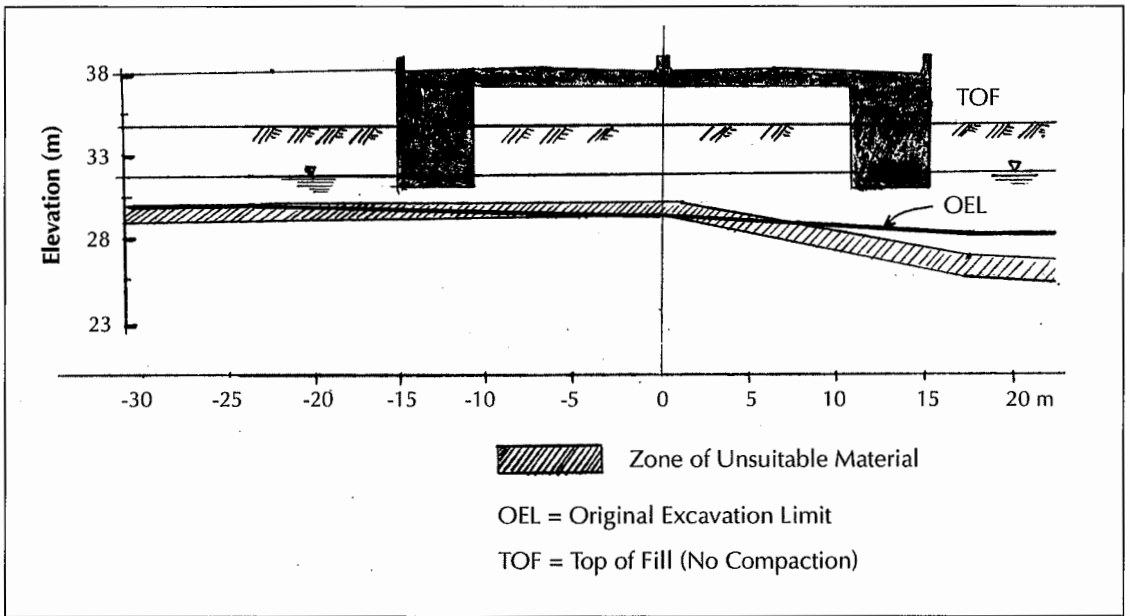


FIGURE 13. Location of unsuitable material for Station 155+00.

the DDC, the soil profile was expected to be loose granular soil suitable for backfilling. However, after the DDC, the soil profile was expected to be very dense backfill material. These before and after test decisions led to the surprising conclusion that the backfill material was not of pure granular soil but mixed with peat pockets.

A PCPT identified the existence of a thin soft soil (peat) in some areas. The undrained shear strength of peat, which had been obtained via laboratory test, was in good agreement with those results obtained via in-situ testing.

Generally, in-situ tests and their associated charts only provide a guide to soil behavior

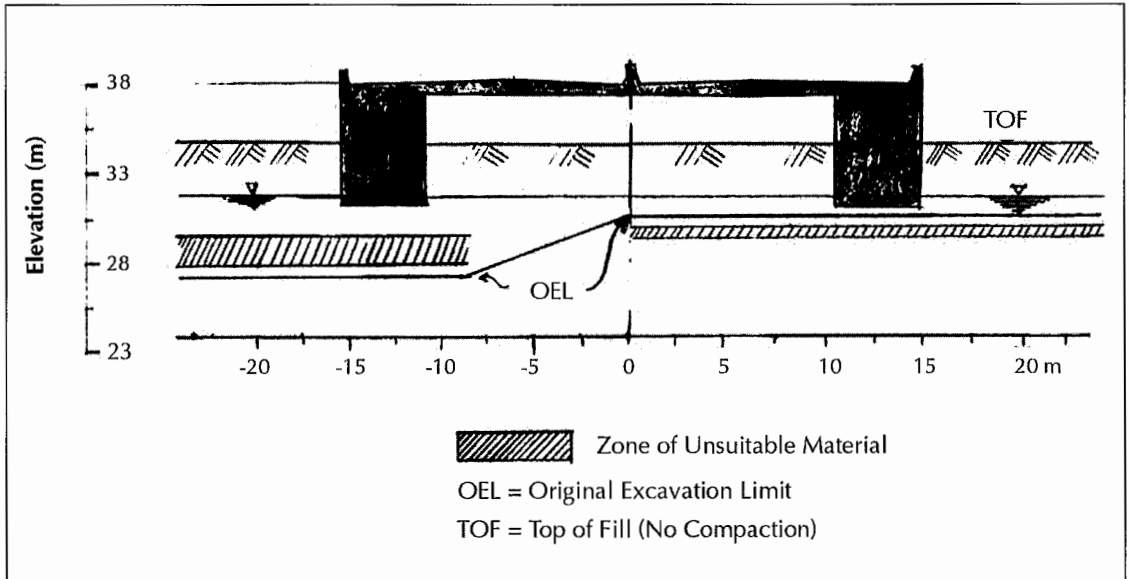


FIGURE 14. Location of unsuitable material for Station 156+00.

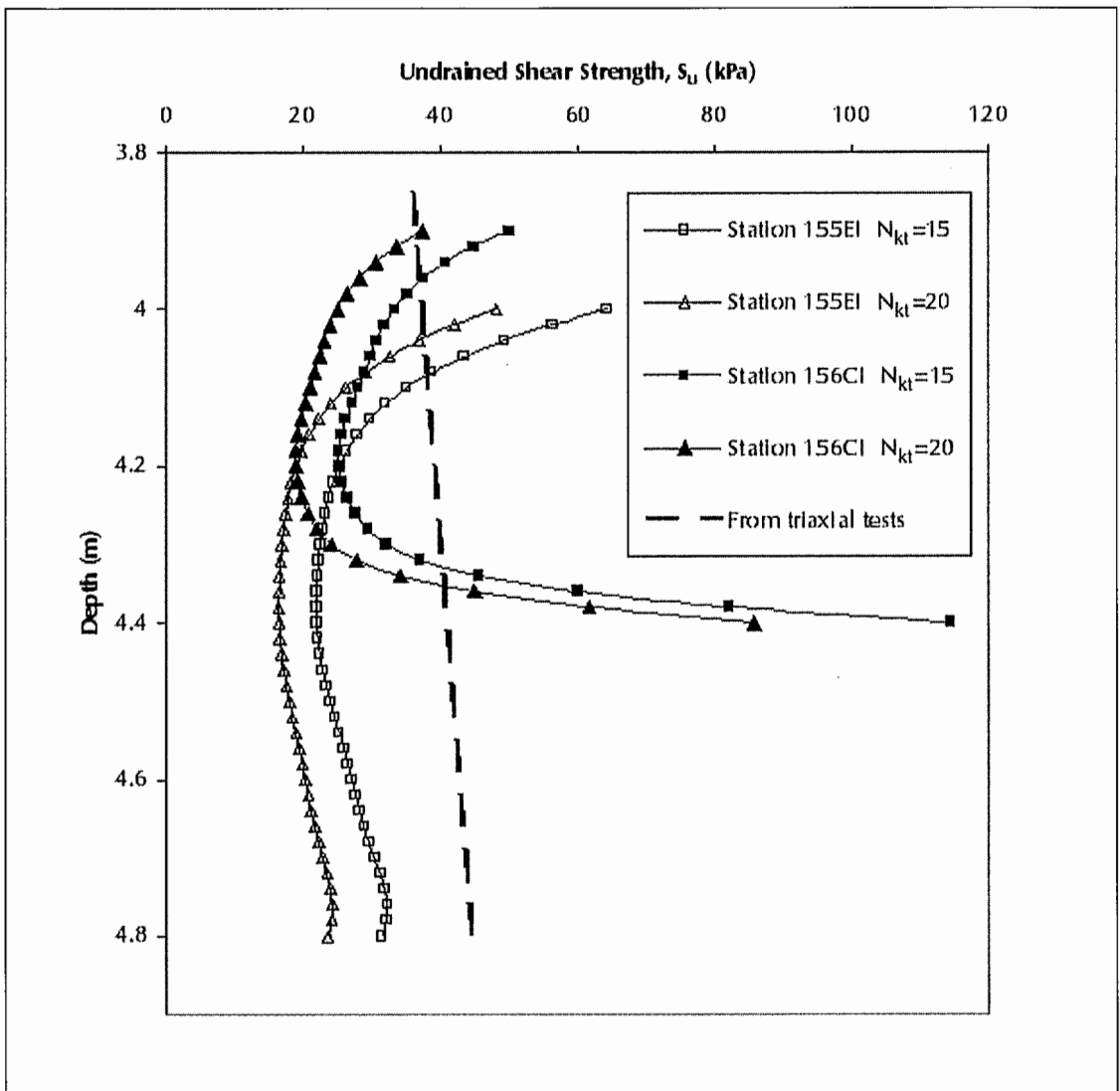


FIGURE 15. Undrained shear strength of the peat versus depth for Stations 155EI and 156CI.

and identification. Artificial or mechanical stabilization may change the mechanical properties of a soil, and the data should be interpreted based on local experience.

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