Measuring and comparing soil parameters for a large bridge on East coast of United States

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ABSTRACT: At a bridge site on the East coast of the United States, the subsurface soils varied from very soft clays to very dense sands. Seismic (p and s) dilatometer tests (SDMT), cone penetrometer tests with pore pressure measurements (CPTU), pressuremeter tests (PMT), vane shear tests (VST), and standard penetration tests (SPT) measured the geotechnical properties of these soils. A large crane lowered the direct push "sea-floor" system to the mudline, which pushed the DMT and CPTU soundings. Underwater hammers generated either the compression wave or shear wave needed for the seismic tests. At the main span, the exploration holes were clustered together and those results are compared.

1 INTRODUCTION

In 1939 a U.S. Department of Transportation performed subsurface explorations by dynamically driving a 1 inch (25 mm) diameter sampling pipe with a 150 pound (68 kgf) hammer into those soils. This testing method predates the standard penetration test and as a result, we had little understanding of the existing soil properties.

In contrast, the new explorations included numerous high quality in-situ tests, such as pressuremeter, vane shear, cone penetrometer, dilatometer, and downhole seismic tests. While we carefully planned our methods to conduct these tests, the geology and shear strengths of these soils differed from what we anticipated. The upper clays were much softer than we anticipated and the lower clays were significantly stronger than anticipated. We modified our test procedures to efficiently perform the explorations.

At the main channel, where the new bridge will have its largest foundations, all of the above tests were performed. The soils' shear strengths and deformation properties are compared for the different tests.

2 PERFORMING THE EXPLORATIONS

A direct push seafloor system, weighing 13,600 kgf (15 tons), pushed the dilatometer and cone penetrometer test probes until penetration refusal occurred. Figure 1 shows a large crane lowering the seafloor system into the river.

A Central Mine Equipment (CME) Model 75 truck drill rig completed pushing the dilatometer and cone penetrometer tests to the contract depth requirements. Its leveling jacks were welded to the barge keeping it in the same exact location as well as giving it maximum pushing power. This drill rig also performed the standard penetration tests, undisturbed sampling, and prepared the holes for the vane shear tests and pressuremeter tests.

Figure 1: Lowering the seafloor direct push system into the river with a large crane

2.1 *Standard penetration testing*

The drill crews made their drilling fluid by adding bentonite and polymer to the river water that they pumped into a mud tub. They added soda ash to lower its ph. The client had concerns about drilling fluid getting into the river, so the drill crew used 5-inch (125 mm) inner diameter casing telescoping inside 8-inch (200 mm) ID casing to make the hole. The larger casing contained any drilling fluid that may have escaped from the smaller casing from entering the river. After the drill crew completed each hole, they pumped the remaining drilling fluid into large steel drums that were later removed from the site.

The 8-inch (200 mm) casing weighed 110 pounds (50 kgf) for each 5 foot (1.5 m) long section and the 5-inch (125 mm) casing weighed 70 pounds (32 kgf) for each 5 foot (1.5 m) long section. The drill crew found this heavy casing cumbersome to handle and thread on/off. They wisely used the large crane that could lift items 120 feet (37 m) above the barge's deck to lower the casing into the soil when starting the hole and remove it as one long piece when they completed the hole.

Often, when drilling deep holes, the driller can spend a significant amount of time threading and unthreading drill rods. For this project, the drill crews used NWJ rods, which had high strength and enabled the drilling fluid to easily flow through them. The crane operator also picked-up long lengths of rods each time, avoiding numerous threading of rods. The driller would lower the front leveling jacks of the drill rig each time so that the crane's hoist would not strike the drill rig tower. This process significantly reduced the testing time.

The heavy casing penetrated the upper very soft clays as much as 40 feet (12 m). Unfortunately, the required two sets of large casing eliminated testing and sampling of these upper soils. The driller performed standard penetration tests according to ASTM D-1586). In the softer clays, the driller used a fixed piston sampler that he pushed into the clay, while in the harder clays he used a Denison piston sampler that he drilled into those clays.

2.2 *Pressuremeter Testing*

Our engineers carefully monitored the driller's preparation of the borehole for the pressuremeter tests. The drill rig turned the rods at a rate of about 1 turn per second and pumped the drilling mud at a flow rate of 10 gallons/minute (40 liters/minute). For the cohesive soil, a 2.93 inch (74.4 mm) diameter three-winged bit with down discharge made the test zone, while for the cohesionless soil, a 3.06 inch (77.8 mm) diameter tri-cone bit also with down discharge made the test zone. Above the test zone, the driller used a 4.88 inch (124 mm) tri-cone bit to create a large hole so that the mud flow for the test zone would not be impeded.

A Texam pressuremeter using a 74 mm diameter monocell probe performed strain-controlled pressuremeter tests. We calibrated each probe for membrane resistance in air and for system compressibility inside a heavy walled steel pipe. The raw test values were corrected for membrane resistance, system compressibility and hydrostatic pressure head. The membrane expanded into the soil and its resisting pressure was measured at each 40 cubic centimeter interval. After the pressuremeter failed the soil past its elastic behavior, the pressuremeter performed an unload-reload cycle. Often, after this unload-reload cycle the pressuremeter held its pressure for 10 minutes by slowly inflating and measured the soil's creep properties. Additionally, the pressuremeter performed up to two more unload-reload cycles at higher radial strains. Figure 2 shows us performing the pressuremeter test.

Figure 2: Performing a pressuremeter test

2.3 *Vane shear tests*

A computer controlled the rotation rate of the drive motor that turned the vane and measured the torque resistance of the vane with a calibrated electronic torque cell. The drive motor, positioned about 30 cm above the vane, eliminated the parasitic rod friction common to many other types of vane shear equipment. For each test, the computer turned the vane at 0.1 degrees per second for the first 90 degrees to obtain the peak shear strength, 6 degrees per second for ten revolutions and 0.1 degrees per second for the last 90 degrees to obtain the remolded shear strength. Figure 3 shows the computer taking the vane shear test readings of torque and rotation angle.

Figure 3: Performing the VST

The upper clays from the mudline to approximately 15 meters below it had low shear strengths and a 75 mm diameter and 150 mm long vanes performed the tests. The lower clays had much higher shear strengths and either 40 or initially 50 mm diameter and 80 and initially 100 mm long vanes attempted to measure their strengths. Unfortunately these clays had strengths that exceeded the maximum vane equipment's torque and as a result did not fail. Twice, when we pressed the vane into these strong clays, their shafts bent.

2.4 *Cone penetrometer tests*

The direct push seafloor system has the following significant advantages over pushing probes from the barge deck into the river deposits:

1. Testing starts at the mudline rather than the depth below the mudline where the casing stops settling. The engineer can measure the strength and deformation properties of these very soft deposits with the seafloor system.
2. Casing attached to the top of the seafloor system and extended to the barge deck serves to measure the tests depths. This depth reference does not move with either waves or the tide and provides accurate measurements.
3. When pushing using a drill rig, the rods can move laterally between the push point and the mudline and rely on the casing for lateral support. The casing can also buckle requiring several different sizes of casing telescoped inside each other. The seafloor system avoids this zone of parasitic rod buckling.

The crane operator carefully set our direct push seafloor system on the river's mudline. Because of its large base area (192 square feet or 17.8 square meters) the seafloor system generally settled less than 20 cm. We estimated the amount of settlement by observing the thickness of mud on the base plates after the completing the sounding and lifting the seafloor out of the river. The upper clays had very low shear strengths and we initially lowered instead of pushed the CPT probe at a controlled penetration rate of 2 cm/second in the clays. Often we did not start pushing the probe into the clays until we had penetrated them about 12 meters. Their low strengths also caused unfortunately low lateral support for the push rods. For each sounding we pushed the CPT probe until we exceeded the lateral capacity of these soft clays and rod buckling began to occur. Many of the soundings could be pushed more than 30 meters below the mudline and the deepest penetration went about 41 meters.

The scheduled depths for the CPT soundings were about 180 feet (55 m) below mudline and the seafloor system had penetration refusal above those depths. Initially we continued these soundings by drilling a 3 inch (75 mm) diameter hole using casing and lowering the CPT and its rods into that hole. The drill rig pushed the CPT until penetration refusal occurred, withdrew the probe and unthreaded its push rods, drilled through the penetration zone and continued the push process. This procedure simply took too much time to do. We discovered that the lower clays had high shear strengths and high lateral earth pressures. They would adhere to the sides of the push rods and cause penetration refusal even though we used a friction reducer to try to eliminate this parasitic rod resistance.

Because we could only push about 5 meters or less each try, we decided to use the torpedo method with the crane to lower and remove the CPT probe and push rods. The CPT cable was threaded through the bottom 5 to 6 meters of AWJ rods (1.75 inch or 44 mm OD) and then exited to the outside of the NWJ rods. The driller taped the CPT cable to the outside of the NWJ rods at approximately 6 meter intervals to try to prevent it from being damaged. To prevent the cable from getting tangled, we rolled or unrolled it onto or off a 30 cm diameter reel.

A data acquisition computer recorded the tip resistance, sleeve friction, and pore water pressure at 5 cm depth intervals as the probe advanced into the soil. At many of the soundings, pore pressure dissipation tests were performed.

2.5 *Dilatometer tests*

At two locations at the two ends of the main span, we performed dilatometer test soundings. Like with the CPT soundings, the seafloor system pushed the DMT blade into the soils. Figure 4 shows the seafloor direct push system pushing the DMT blade into the soft clays.

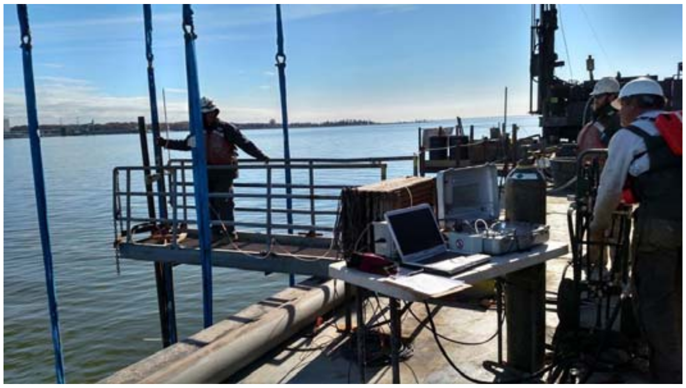

Figure 4: Pushing the DMT with the seafloor system

Unfortunately at the two locations, a dense sand and gravel layer that caused penetration refusal existed below the very soft clays, which had thicknesses of 18 meters. While the seafloor system measured the soil properties of these very soft clays, we continued these soundings using the torpedo method after drilling through the sand and gravel formation.

For each DMT test, we measured the "A", "B", "C" and penetration thrust values. In the very soft clays the difference between the "A" and "B" reading were just slightly more than their calibration values. In these clays we slowly and carefully inflated and deflated the membrane to obtain accurate measurements.

2.6 *Seismic downhole tests*

This project required both shear and compression seismic wave velocities at two test locations to depths of 58 meters. We knew that we could not push the seismic probe to those depths using the seafloor direct push system and decided to drill to 58 meters with the drill rig and lower the seismic DMT probe to that depth. Fine well-graded washed gravel was placed into the annulus between the seismic DMT probe and the borehole sidewalls to achieve good coupling.

We also knew that we would need a lot of energy to successfully make measurements to those depths. A large shear plate and heavy shear and compression hammers mounted to the seafloor system and positioned about 3 meters away from the sounding created the energy waves. After lowering the probe to its bottom depth and orienting its shear sensors parallel to the direction of the seismic strike, the shear and compression tests started. After performing about 5 compression and shear strikes and recording those waves with the computer acquisition system, we raised the seismic probe 1 meter to perform the next set of seismic tests. We successfully continued this process until we raised the probe and performed tests 3 to 4 meter below the mudline, where the driller could no longer place the gravel and successfully couple the probe and the borehole sidewalls.

The SDMT combines the flat plate dilatometer with a seismic module for the measurement of the shear wave velocity. The seismic module instruments a rod placed above the DMT blade, equipped with two shear receivers located at 0.5 m distance apart and two compression receivers located 0.6 m apart. The shear wave, generated at the mudline, travels downward and arrives first to the upper receiver, then, after a delay Δt, to the lower receiver (Fig. 5). At each test depth, the seismic module amplifies and digitizes the seismograms acquired by the two receivers, and then transmits that data, using the single wire from the standard DMT cable, to a computer that determines the delay of the wave arrival. Vs equals the ratio between the difference in the distances of the shear wave travel paths from the source to each receiver (S2 - S1) and the time delay, Δt, in the wave arrival. The compression wave, also generated at the mudline by striking the plate vertically, travels to the compression receivers and its speed computes similarly to the shear wave. Performing compression waves below the water table offers little significance to the designer as the virtually incompressible water transmits the wave.

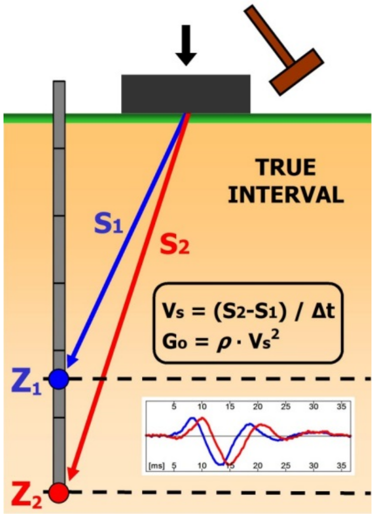

Figure 5: Typical shear wave set-up for SDMT

3 COMPARISON OF THE TEST RESULTS

Figures 6 and 8 show the undrained shear strengths versus depth for the DMT, VST, CPT, and PMT for the north and south ends of the proposed main span of the bridge, respectively. Because the upper clays had strengths much less than the lower clays, Figures 7 and 9 show the strength for the upper clays (0 to 15 meters) with an enlarged scale for the north and south ends, respectively.

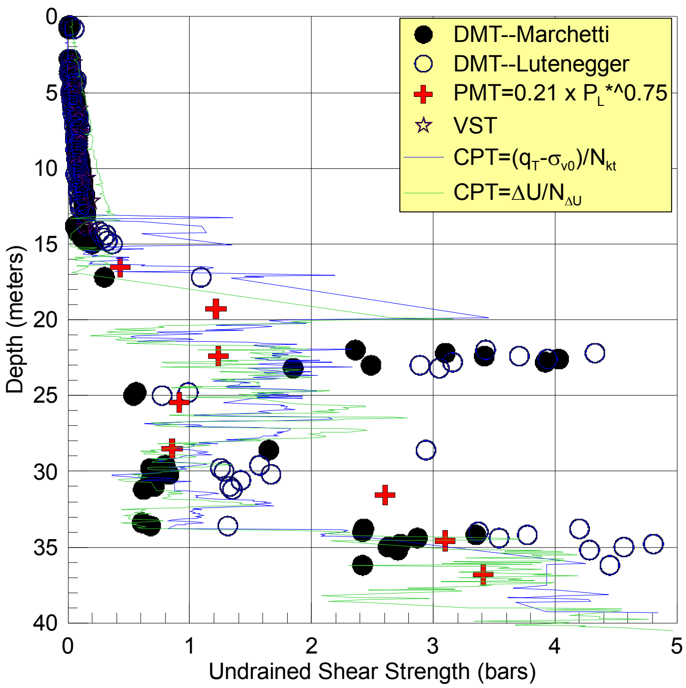

Figure 6: Undrained shear strength for north end

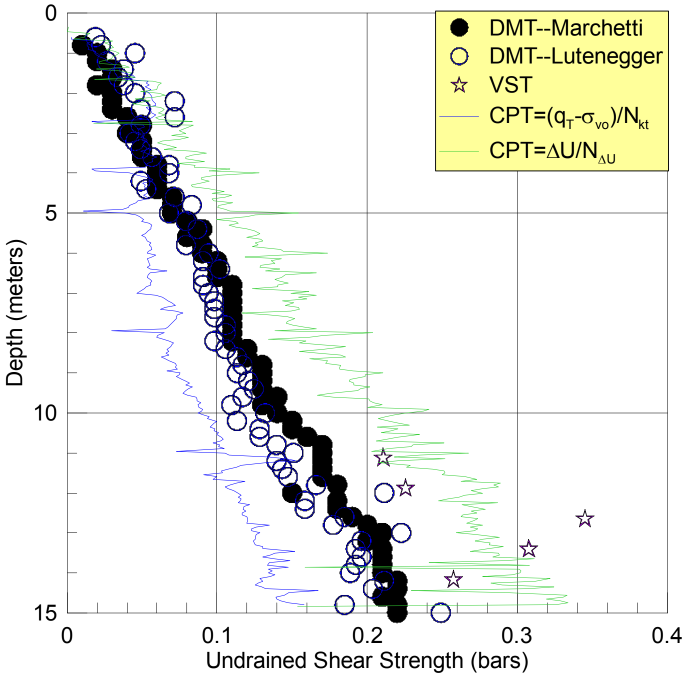

Figure 7: Undrained shear strength for north end for soft clays (0 to 15 m)

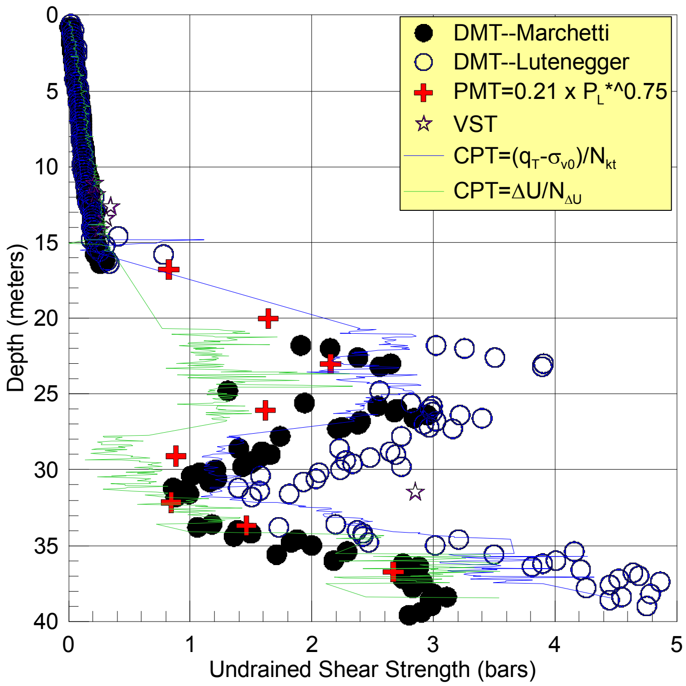

Figure 8: Undrained shear strength for south end

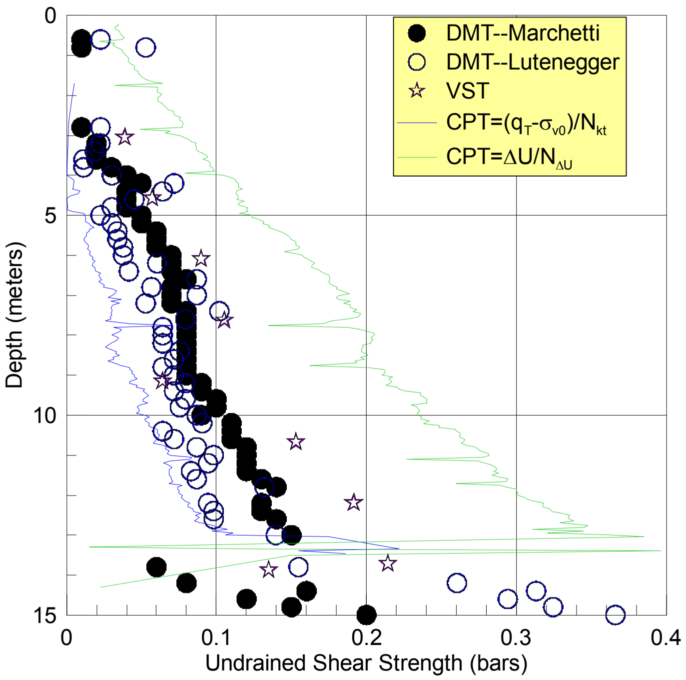

Figure 9: Undrained shear strength for south end for soft clays (0 to 15 m)

3.1 *Shear tests*

The DMT data computed the undrained shear strength of the clays using Marchetti (1980) and Lutenegger (2006) methods. Marchetti used an empirical relationship based on excellent comparisons of shear strength at eight well documented research sites. Many other researchers have since then shown that Marchetti's method accurately predicts the undrained strength at numerous sites, world-wide.

$Su = 0.22\sigma_{v0}'(0.5K_D)^{1.25}$ --Marchetti

Lutenegger used cylindrical cavity expansion theory to develop his method and showed how this method predicts shear strength in soft clays at several sites. For the hard clays at this site Lutenegger's method shows higher values than the other methods, but with similar trending patterns as the other methods.

$Su = (P_0-P_2)/2.65$ --Lutenegger

The CPTU data computed the undrained shear strength based on the corrected tip resistance and on the excess pore water pressure as follows:

$Su = (q_T-\sigma_{v0})/N_{kT}$, and

$Su = \Delta U/N_{\Delta U}$.

We selected $N_{kT} = 15$ and $N_{\Delta U} = 6$ for the shear strength computations at this site. While these values predict the undrained shear strength for the stronger clays fairly well, different correlation factors would better predict the strengths in the very soft clays from 0 to 15 meters, demonstrating why engineers should choose these correlation factors based on site or geologic specific correlations.

The pressuremeter data predicted the undrained shear strength equal to $0.21\ (P_L^*)^{0.75}$, where P_L^* = the net limit pressure. The vane data computed the shear strength equal to $6T/(7\pi D^3)$, where T equals the torque and D equals the vane's diameter.

3.2 *Deformation tests*

Figures 10 and 11 show the predicted constrained deformation moduli from correlations with DMT, CPT and PMT data. The DMT data computed the modulus using Marchetti (1980) equation. For the PMT data, Young's modulus equals E_0/α, where E_0 is the initial pressuremeter modulus and α is the rheological factor obtained from the Pressiorama chart (Baud, 2013). Based on a Poisson's ratio, ν, of 0.33, the constrained deformation modulus = 1.482 * the Young's modulus. The CPT data computed the constrained modulus equal to $\alpha * q_T$, where α was assumed as 8.

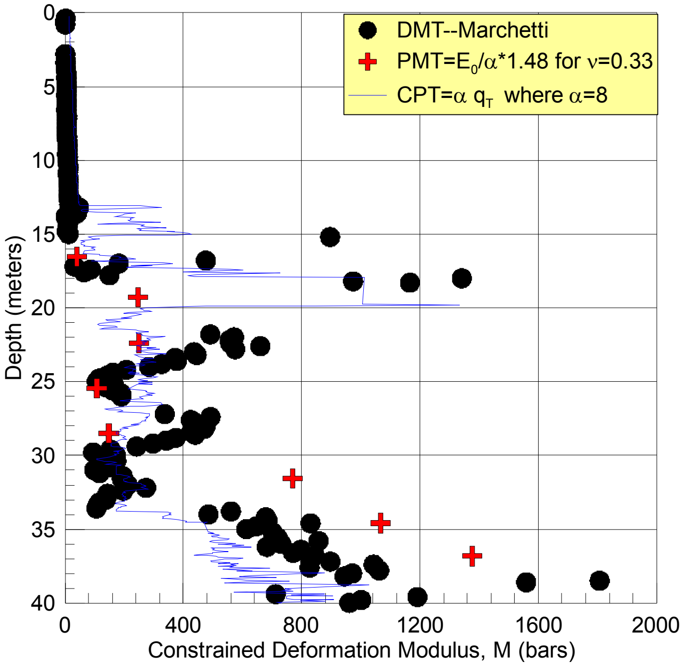

Figure 10: Constrained deformation modulus for the North end

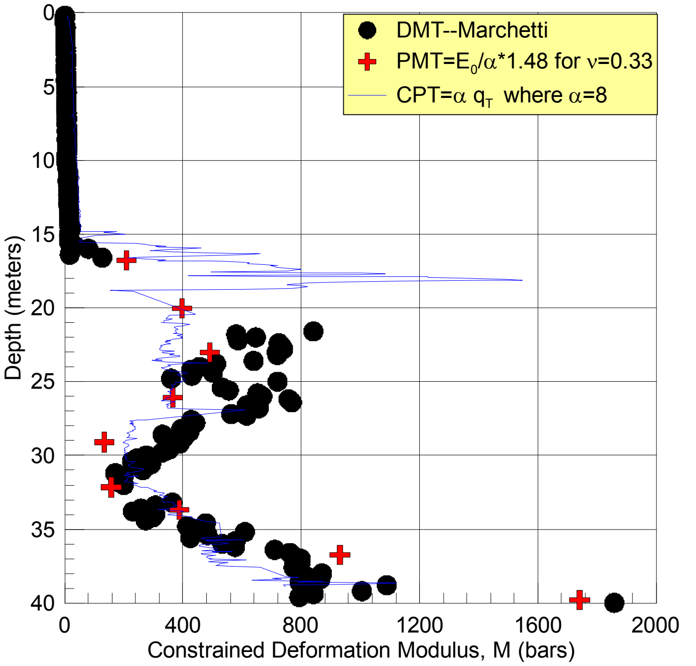

Figure 11: Constrained deformation modulus for the South end

3.3 *Seismic shear wave tests*

The large scale shear wave source provided high quality signals to large depths. As an example, Fig 12a shows the seismograms recorded 30 m below the river bed. The delay of the shear wave arriving to the two receivers, vertically spaced 0.50 m, is clear and consistent. Fig 12b shows the same seismograms after the red trace, corresponding to the lower receiver, has mathematically shifted to the left by a delta time, Δt, until it superimposes on the blue trace, the upper receiver. The shear wave velocity simply computes as the difference in wave travel distance divided by Δt. Figure 13 shows the full profile of Vs in one of the test locations.

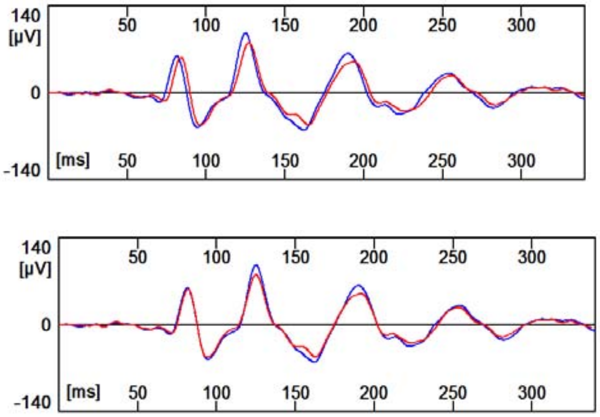

Figure 12: Seismic shear wave recorded at 30 m

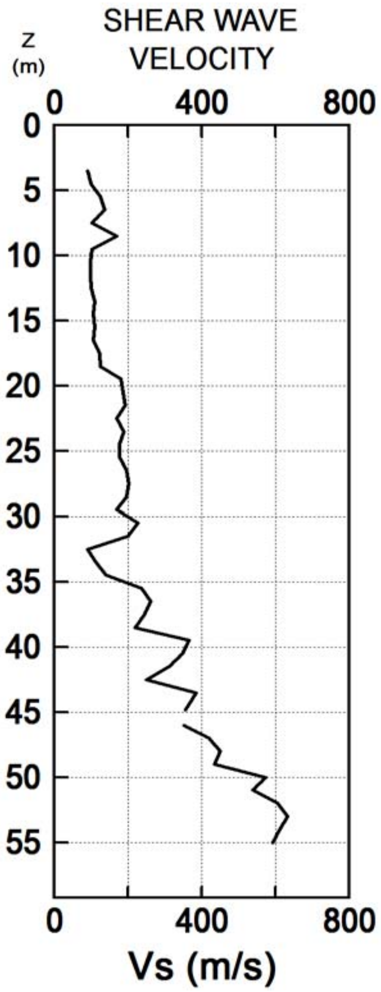

Figure 13: Shear wave profile

4 CONCLUSIONS

Direct pushing of the DMT and CPT probes from the mudline eliminates obstacles that occur when pushing those probes from the barge deck. Those obstacles include missing the tests from the mudline to the depth that casing stops settling, buckling of rods between the barge deck and mudline, and having inaccuracies in depth measurements from waves and tides changes.

The true interval seismic DMT accurately measured the shear and primary waves generated by heavy hammers striking a plate embedded at the seafloor.

The undrained shear strengths from the VST, CPT, DMT and PMT compared favorably with each other. The CPT correlation factors depend on the geological formation and its stress history and one should use different factors for different formations.

The constrained deformation moduli from the DMT, CPT and PMT compared favorably with each other.

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