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Measurement of Strength Gain in Presumpscot Formation Clay using In Situ Vane Shear Testing

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ABSTRACT: A large regional landfill in Southern Maine, USA, is founded on soft, sensitive Presumpscot Formation marine clay soils up to 24 m thick. The original landfill phase is unlined and relies on the 24 m of clay for its liner. Landfilling at this phase was completed in 2009 and capped while the foundation soils were undergoing consolidation and strength gain. This increase in strength of the soft clay is an essential part of future landfill expansions which might involve filling over this portion of the site via piggybacking. Geotechnical instrumentation at the site includes vibrating wire piezometers, settlement platforms, and inclinometers. An additional important component of the monitoring has included periodic measurement of the undrained shear strength via in situ vane shear testing (VST). This paper presents the results of 3 episodes of VST, performed in 1994 (initial conditions), 2002, and between 2009 and 2013, at several locations across the site that have undergone as much as 1.2 to 1.8 m of settlement. The results are evaluated using laboratory direct simple shear testing and consolidation testing and compared to strengths predicted by the SHANSEP method.

Keywords: vane shear; marine clay; Presumpscot Formation; landfill; in situ testing

1. Introduction

The Ecomaine landfill facility is located in the towns of South Portland, Westbrook and Scarborough, Maine, USA. The landfill has been operating for more than four decades and includes several phases reflecting the transition from municipal solid waste (MSW) only disposal in the late 1970's to late 1980's to primarily ash only disposal from 1980 to the present time. The footprint of the facility is about 38.6 hectares and is spread over four areas as shown on Fig. 1 and described as follows:

Balefill 1-6: Approximately 12.9 hectares with baled MSW only, closed in 1997.

Balefill 7 & 8: Approximately 5.5 hectares, with Baled MSW only, closed in 1995.

Original Ashfill/Balefill Facility: Approximately 12 hectares , 2 MSW cells, with the remaining ash disposal.

Southern Expansion: Approximately 8 hectares, Phase I constructed in 2006, Phase II West Constructed in 2012. Phase II West is the active cell.

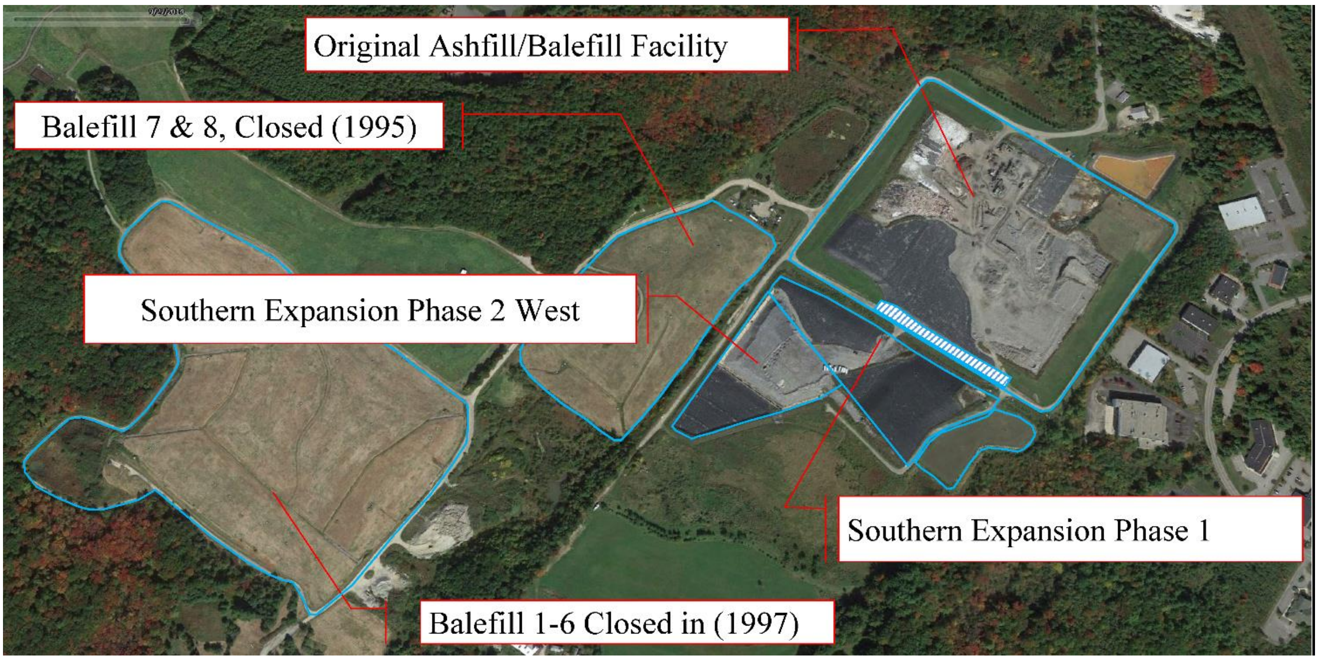

Figure 1. Ecomaine Landfill Site Overview Map

The Original Ashfill/Balefill Facility and Phase I of the Southern Expansion are founded entirely or partially on a soft clay foundation stratum. The investigations summarized in this paper are from these two areas. Figure 2 shows the area of investigation, site instrumentation and the locations of two of the many explorations discussed herein.

The clay deposit is known as the Presumpscot Formation. It extends along the coasts of Maine, New Hampshire and part of Massachusetts where it is known as the Boston Blue clay. It is a late Pleistocene deposit derived from glacial abrasion of parent bedrock, deposited in meltwater below the sea level. The deposit at the site consists of 40 to 60 percent silt sized particles and the remainder clay size. The saturated unit weight ranges from 1680 to 1840 kg/m^3.

The foundation clay soils are soft, compressible and sensitive. As waste is added at the surface, the clay soils undergo settlement at a slow rate allowing the clay to gain shear strength during the consolidation process. A major factor in the development of these two facilities is the impact that waste loading has on the stability of the underlying soft clay foundation soils. A cross section of the site, showing the waste loading at the time of the VST program is shown on Figure 3

This paper presents the results from an extensive site characterization and monitoring efforts to ensure the global stability of the Ecomaine landfill which is largely based on the undrained shear strength of the clay.

2. Site Characterization

2.1. Laboratory Testing

Laboratory testing at the site included 1D incremental consolidation Testing, (IL CON), Direct Simple Shear Testing, (DSS), moisture contents and Atterberg Limits. Typical Consolidation tests from elevation 11.1 m, are shown on Figure 4. The red consolidation curve was obtained from DSS testing test on a 3-inch Shelby tube obtained prior to site loading. The maximum previous stress was 98 kPa determined from the strain energy method. The green curve was from a sample retrieved at the same elevation in 2009, after 15 years of consolidation. The maximum previous consolidation stress was estimated as about 110 kPa. It is noted that the excess pore pressure at this location was about 50 kPa at the time of sampling, therefore the sample is actually under consolidated.

The initial characterization of the undrained shear strength of the soft marine clay at the site was evaluated using laboratory direct simple shear (DSS) testing and incremental consolidation testing to develop an undrained shear strength profile using SHANSEP. This initial strength profile was compared to the initial strength measured via the vane shear test (VST). The SHANSEP method, developed by Ladd and Foott (1974) suggested the following well-known relationship:

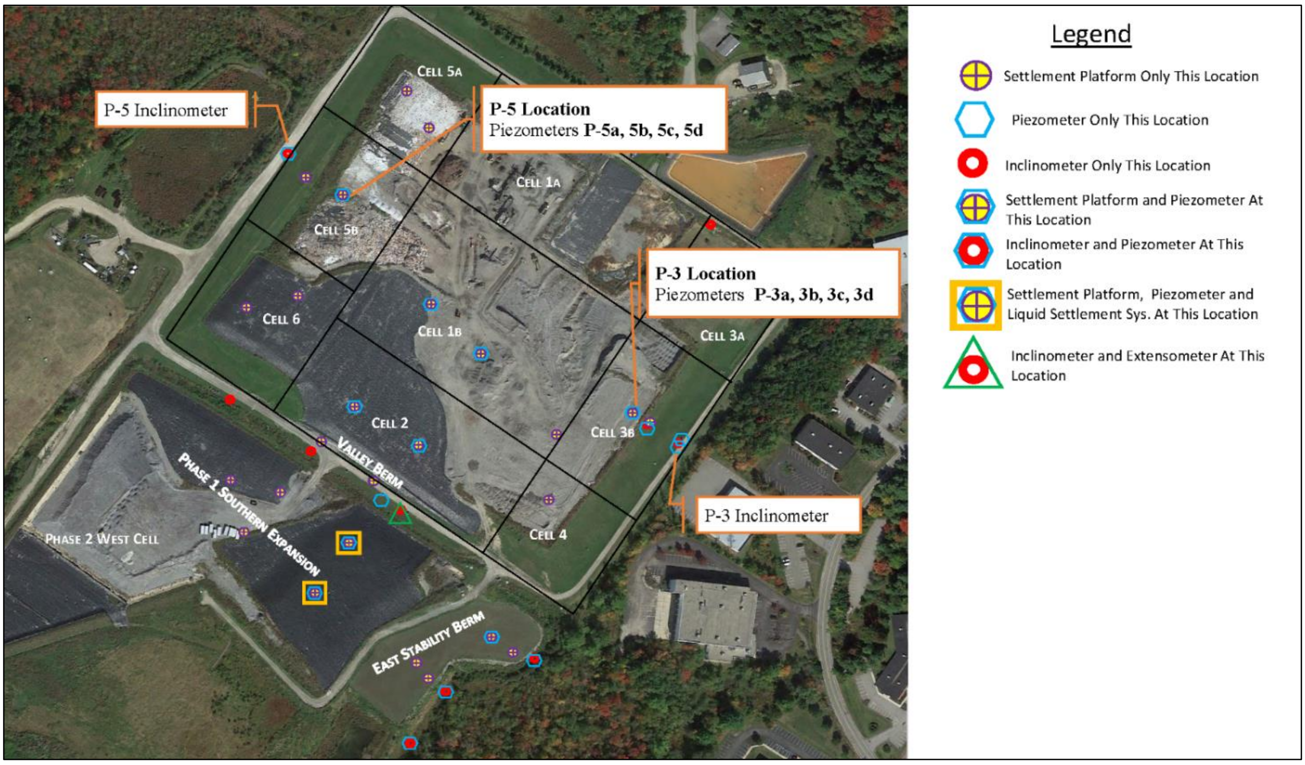

Figure 2. Exploration and Instrumentation Plan

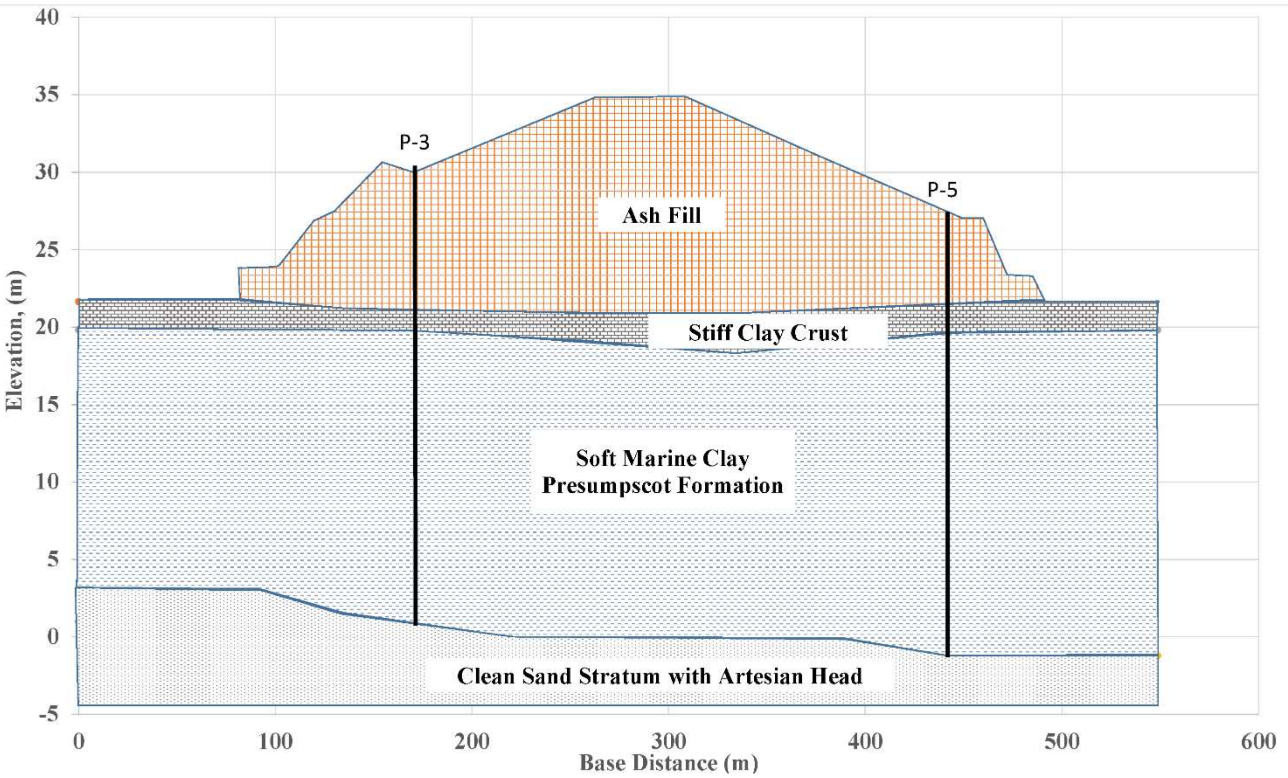

Figure 3. Typical Cross Section

$$\frac{S_u}{\sigma'_{vo}} = S\,(OCR)^{0.8} \qquad (1)$$

where: Su is the undrained shear strength, σ'_{vo} is the existing effective overburden stress, S is the normalized stress ratio and OCR is the overconsolidation ratio.

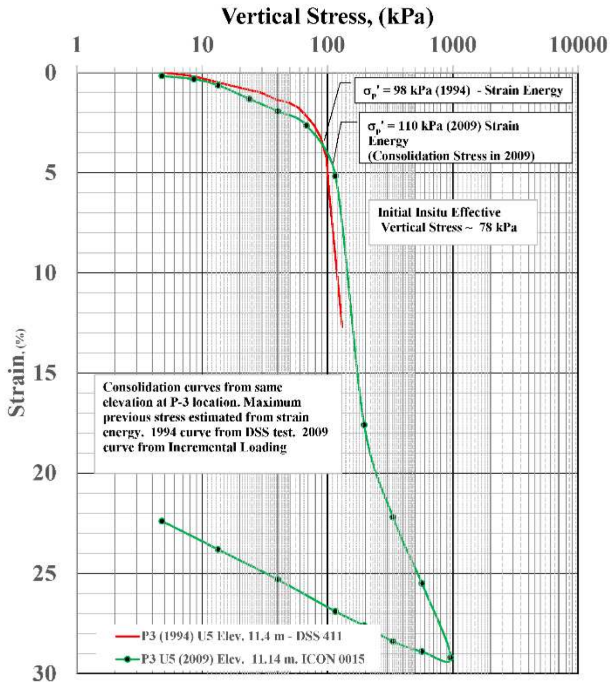

Figure 4. Consolidation Testing at Elevation 11.1 m.

The initial stress history at the site is shown on the middle plot on Figure 5. The existing vertical stress was developed based on the unit weights from Shelby tube samples and the initial pore pressures from three vibrating wire piezometers installed in the clay stratum and one installed in the underlying sand stratum. The underlying sand stratum exhibits artesian heads of 1.2 to 2.4 m above the ground surface prior to site development. The existing vertical effective stress was developed to reflect this condition. The maximum previous stress σ'_p (prior to site loading) was plotted from eight incremental consolidation tests at various depths. The σ'_{vo} from the consolidation phase of the eight DSS tests was also plotted. These test results indicated that the initial clay was heavily overconsolidated from the surface crust down to approximately elevation 15 m. From elevation 15 to the base of the stratum, the clay was judged to be overconsolidated by a uniform stress of about 14.3 kPa. The overconsolidation is believed to be the result of fluctuations in the groundwater table over time. It is noted that there were a few consolidation tests that indicated normally consolidated or slightly under consolidated conditions.

The DSS testing was performed to measure the normalized stress ratio, "S". The DSS testing was conducted to simulate stress history conditions reflecting "aged" and "young" or staged construction conditions. The DSS test is also the most relevant failure mode when comparing laboratory strengths to the VST data.

The concept of "aged" and "young" clays was discussed by Bjerrum (1972). Clays initially deposited and left under a constant effective stress for hundreds or thousands of years will undergo secondary consolidation. Clays which have undergone significant secondary consolidation are considered "aged". The clay at the Ecomaine site in its virgin state (prior to any site loading) was "aged" and "slightly overconsolidated" due to fluctuations in the groundwater table over the site's geological history. A clay which has been recently

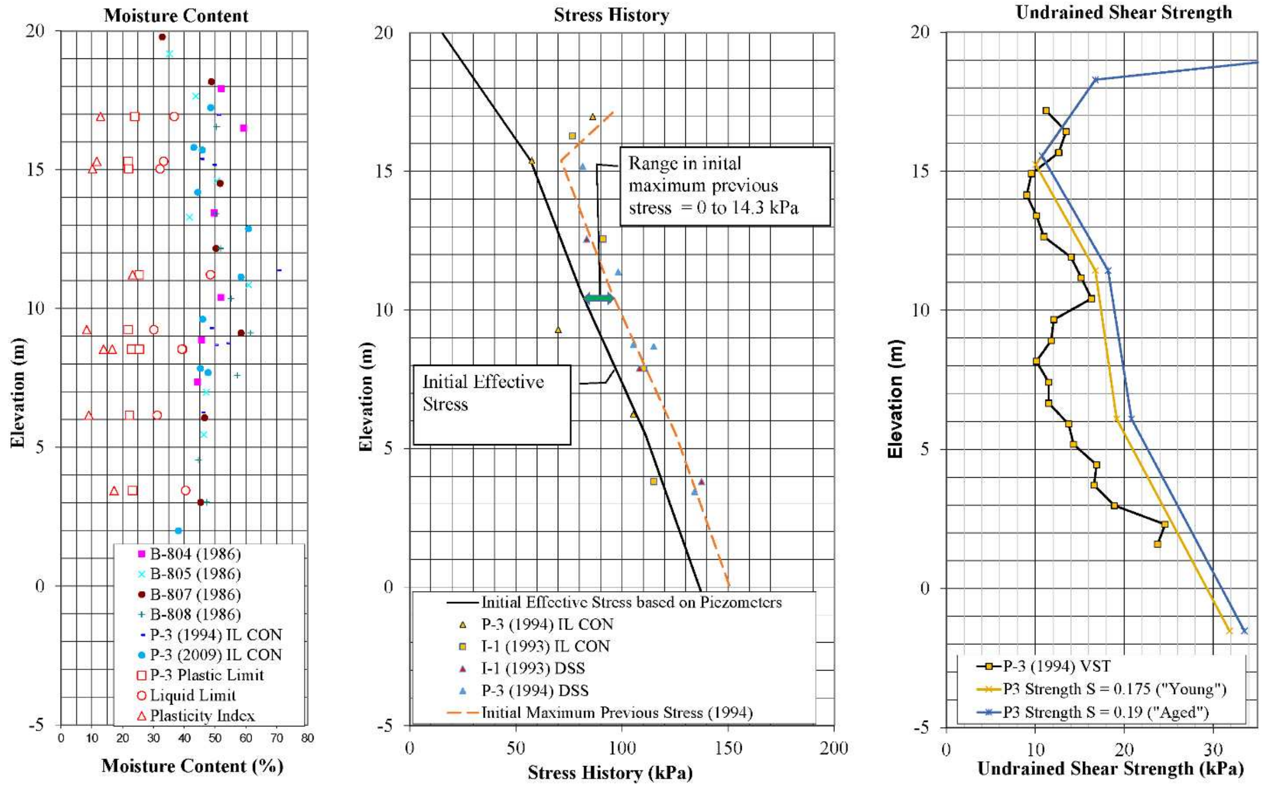

Figure 5. Stress History and Strength at P-3 Location

deposited and has come to equilibrium under its own weight, but has not undergone significant secondary consolidation, is classified as "young" or normally consolidated clay. The clays at the Ecomaine site are currently characterized as "young" clays, due to the ongoing consolidation.

"Aged" conditions for the DSS testing were induced by consolidating the test specimins well past maximum previous stress, then holding the stress for about 24 hours, or about one log cycle of secondary consolidation. The samples were then sheared. "Young" conditions were induced by consolidating the sample well past the maximum previous stress, holding the stress for typically 1-hour then shearing the sample.

The first round of DSS testing resulted in an "aged": stress ratio "S" of 0.19 and a "young" stress ratio of 0.175. Subsequent testing at the site since these initial tests were performed indicate an average "young" stress ratio of 0.167.

2.2. Field Vane Testing

The original investigations conducted in the mid-1980's measured the in situ undrained shear strength using an Acker torque wrench with thick drillers vanes. This equipment is crude and unreliable in soft sensitive clays. Beginning in 1994, the undrained shear strength of the soft clay at the site was measured in situ using the Geonor vane borer H-10 testing apparatus (Fig. 6). Testing was carried out during three separate episodes: 1994 (episode 1), 2002 (episode 2), and between 2009 and 2013 (episode 3).

The vane borer is a cylindrical tube 950 mm in length which houses the vane. While the unit is forced into the clay, the vane is protected by a shoe. For testing, the vane is pushed ahead of the protection shoe a distance of 50 cm to allow testing in nearly undisturbed material. The protection shoe allows thinner blades to be used, thus reducing the area ratio which is directly proportional to the induced disturbance. The protection shoe also cleans the vane automatically between each test.

The in situ vane shear testing was performed in accordance with ASTM D2573. The vane torque head was attached to the inner rods and the vane was rotated at a rate of 0.1 degree per second (6 degrees per minute). Torque readings were made every 30 seconds. The torque head was rotated using a hand crank for the testing performed by UNH in 1994 and 2002, and using a DC motor for the testing performed by Soil Metrics LLC between 2009 and 2013. The test was continued past peak torque until an approximate constant residual value was obtained. The typical total test interval was about 10 minutes although some tests in the deeper boreholes extended to as much as 30 minutes. After the peak strength was obtained, the vane was rotated in-place for 10 full rotations, and a remolded test was performed. The peak remolded gauge reading was recorded after about 3 to 5 minutes.

Extensive calibrations of the torque heads (UNH and Soil Metrics) were carried out using a frame designed and built by Soil Metrics.

3. Initial Strength and Stress History

The laboratory DSS and incremental consolidation tests were used alongside field vane testing to assess the strength of the clay stratum to determine the waste grades to ensure a factor of safety of 1.5 against failure.

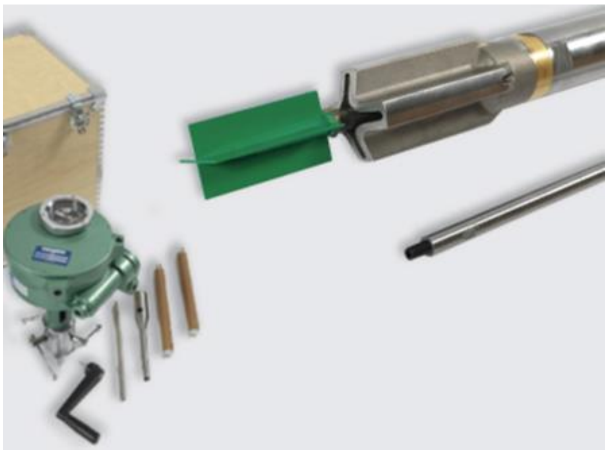

Figure 6. Geonor H-10 vane borer apparatus

Fig. 5 shows the initial stress history and strength profile developed from the laboratory and field tests. The laboratory strength for the stratum was estimated for the P-3 location and "aged" conditions, based on the constant overconsolidation stress of 14.3 kPa with depth, the existing vertical stress shown, adjusted for the artesian conditions at the base, and the stress ratio "S" of 0.19. The undrained shear strength for the "young", staged construction condition was estimated using the stress ratio of 0.175.

The peak undrained shear strength from the in situ VST conducted at roughly 0.75 m intervals (total 22 tests) is plotted with depth. Between elevation 15 and about 10.5 m, the lab strength was estimated to be about 20 to 40 percent higher than the in situ VST strength. Between elevations 10.5 to 8 m, the undrained shear strength measured from the VST dropped significantly by about 8 kPa, then began to increase linearly with increasing depth. Similar lower strength zones were also measured at other locations during the 1994 VST program.

Santamaria et al. (2015) observed a similar shift in undrained strength at coastal sites in Portsmouth, Dover and Newington, NH using the field vane, the flat plate dilatometer and the piezocone. This shift was first reported in Ladd (1972) during the investigation for a test embankment in the sensitive clay in Portsmouth, NH. The shift is unusual and appears to a regional rather than a local feature. The study by Santamaria et al. (2015) concluded that there was evidence of a significant difference in soil composition above and below the change in undrained shear strength based on X-ray diffraction of samples obtained in Newington, NH. Using the work of Birch (1984) and Oakley (2000) they also hypothesized that the strength change may be attributed to a sea level lowstand that may have exposed and eroded part of the Presumpscot clay producing this unconformity. Seismic activity may have also been a possible factor in creating this unusual zone.

This zone of reduced strength measured by the VST is confirmed by inclinometer measurements made during the same time period at the perimeter of the landfill as shown in Fig. 7. The low strength zone coincided with the most incremental horizontal displacement. The incremental displacement plot is shown to highlight the zones of high shear strain. These observations led to an increase use of the field vane for stability analyses and with a factor of safety initially reduced to 1.3. It was found that this lower strength zone controlled the design heights even with the lower factor of safety. Since this initial work, subsequent in situ VST's at other locations across the site have all indicated the drop in strength with depth at similar elevations.

It is clear from Figure 4 that the laboratory strength profile in the low strength zone estimated undrained shear strengths from SHANSEP as much as 80 percent higher at some elevations (e.g. elevation 8 m.). As a result of this site-wide behaviour and the low rate of consolidation, stability analyses conducted after the initial analyses used the undrained shear strength from the in-situ VST and a factor of safety of 1.5.

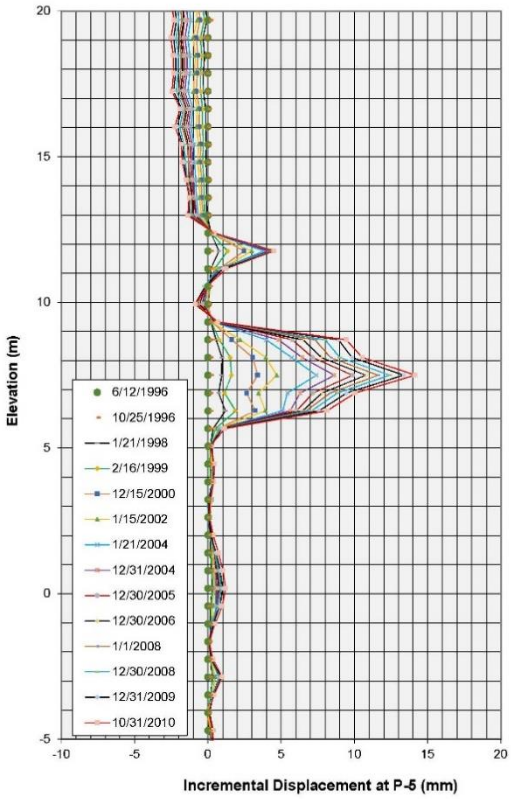

Figure 7. Incremental displacement at P-5

4. Strength Increase

Two additional episodes of VST were conducted at the site in 2002 (episode 2), and between 2009 and 2013 (episode 3). The measured undrained shear strength from these two episodes is shown at two locations P-3 (Fig. 8) and P-5 (Fig. 9). The results of these additional episodes of VST indicate that the in situ strength generally increased between 2 to 4 kPa from episodes 1 and 3 over about a 15 year period. There were some locations where no strength increase was measured and a couple locations where a small strength decrease was measured. The low measured shear strengths in the upper five tests performed in 2011 at the P-5 location were attributed to metal being dragged into the clay stratum from the overlying ash deposit.

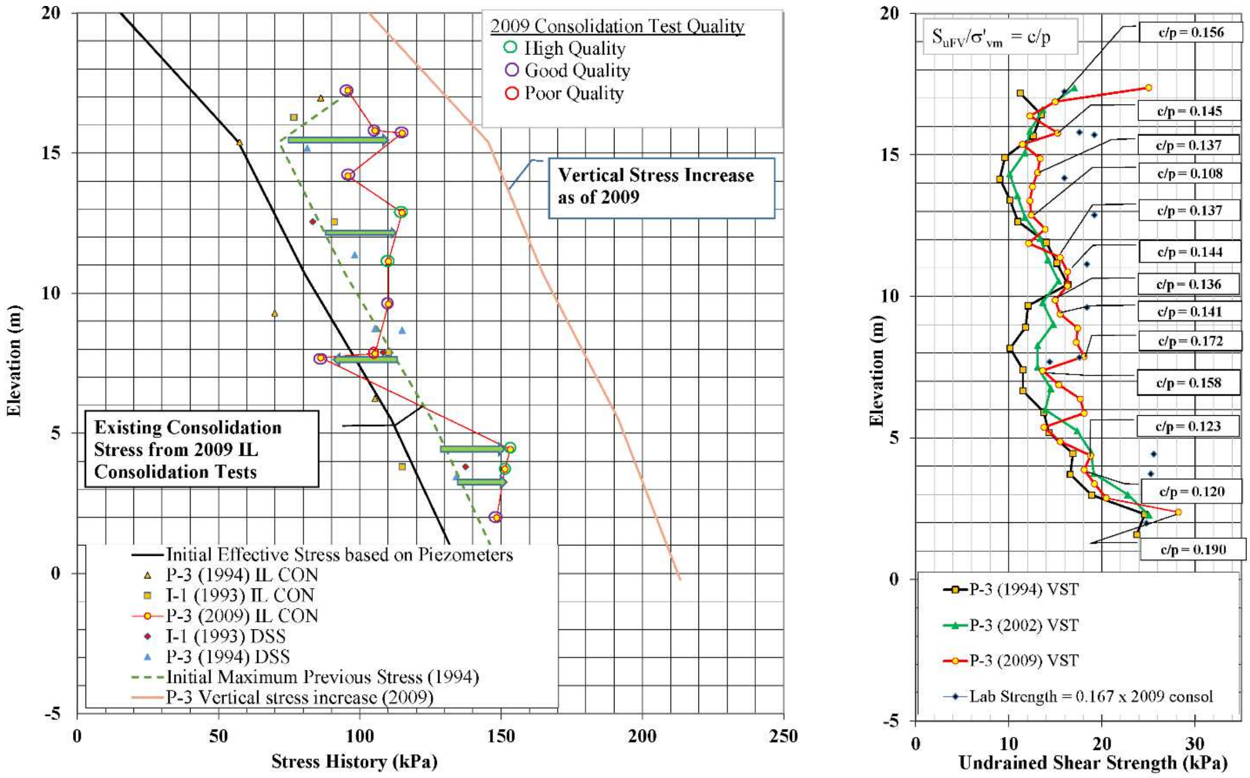

Figure 8. Stress History and Strength Increase at P-3 Location

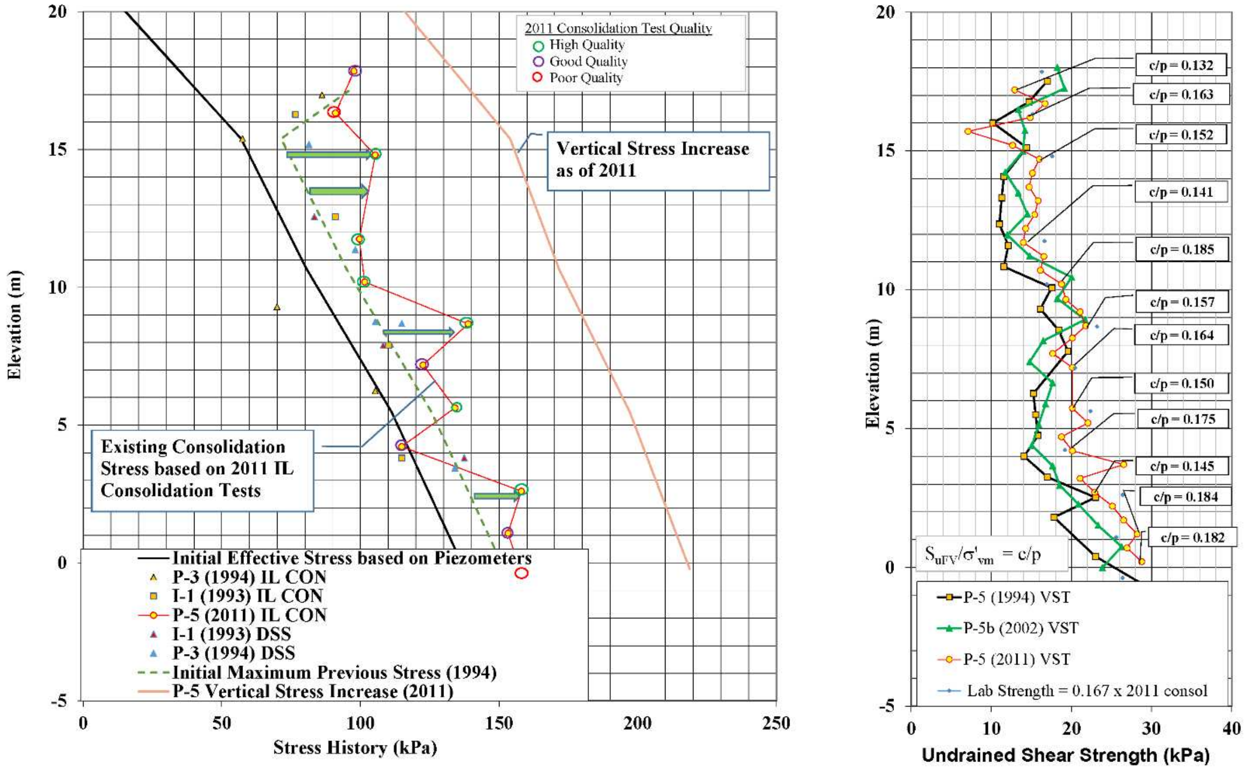

Figure 9. Stress History and Strength Increase at P-5 Location

In order to provide some corroboration of the measured in situ strengths, the investigation for episode 3 included retrieving Shelby tube samples at 1.5 m intervals at several of the VST locations. Incremental and Constant Rate of Strain consolidation tests were performed on these Shelby tube samples in order to estimate the existing consolidation stress at each depth. The maximum previous stress from these consolidation tests is plotted on the center plot on Figures 8 and 9. The consolidation test results were qualitatively assessed using a high, good and poor quality based on the shape of the consolidation curve.

The initial stress history plot shown on Fig. 5 indicated that approximately 15 kPa of overconsolidation existed at the site prior to loading. This overconsolidation was quickly erased during the initial stages of site loading and the clay stratum became normally consolidated. Normally consolidated conditions indicate that the existing overburden stress σ'_{vo} is equal to the maximum previous stress σ'_{vm}.. Referring to the SHANSEP equation, $Su/\sigma'_{vo} = S\ (OCR)^{0.8}$, the term OCR becomes 1.0 and the equation to characterize the strength at the site becomes: $S_u = S\ \sigma'_{vo,}$ where the effective stress is equal to the maximum previous consolidation stress.

The strength gain occurs as a result of consolidation over time which has been monitored on site using settlement plates and piezometers. Fig. 19 shows a graph of settlement versus time for the period of 1995 to 2016 in response to waste loading.

Fig. 9 also shows excess pore pressure. It can be seen that during that time period, a settlement of about 1 m under loading of 6 m of waste or approximately 95 kPa occurred at that particular location. The pore pressures illustrate the low hydraulic conductivity of the Presumpscot clay which leads to very slow settlement rates and strength gain.

The strength increase from SHANSEP was estimated by multiplying the maximum previous stress from the episode 3 consolidation tests by the previously developed site-wide stress ratio 0.167 for "young" samples.

5. Comparison of Laboratory strength to In Situ Strength

The shear strength plots shown on Figs. 8 and 9 indicate that the measured in situ strengths are consistently lower than the laboratory measured strengths. The different strength measurements were compared using the undrained strength ratios. The field vane strength ratio $S_{u\text{-}FV}/\sigma'_{vm}$ is calculated by dividing the undrained shear strength from the vane shear test by the maximum previous stress at the closest consolidation test. These strength ratios, sometimes referred to as c/p ratios are plotted on the shear strength plots on Figures 8 and 9. The field vane ratios ranged from 0.09 to 0.22, with an average of 0.144, as shown on Table 1.

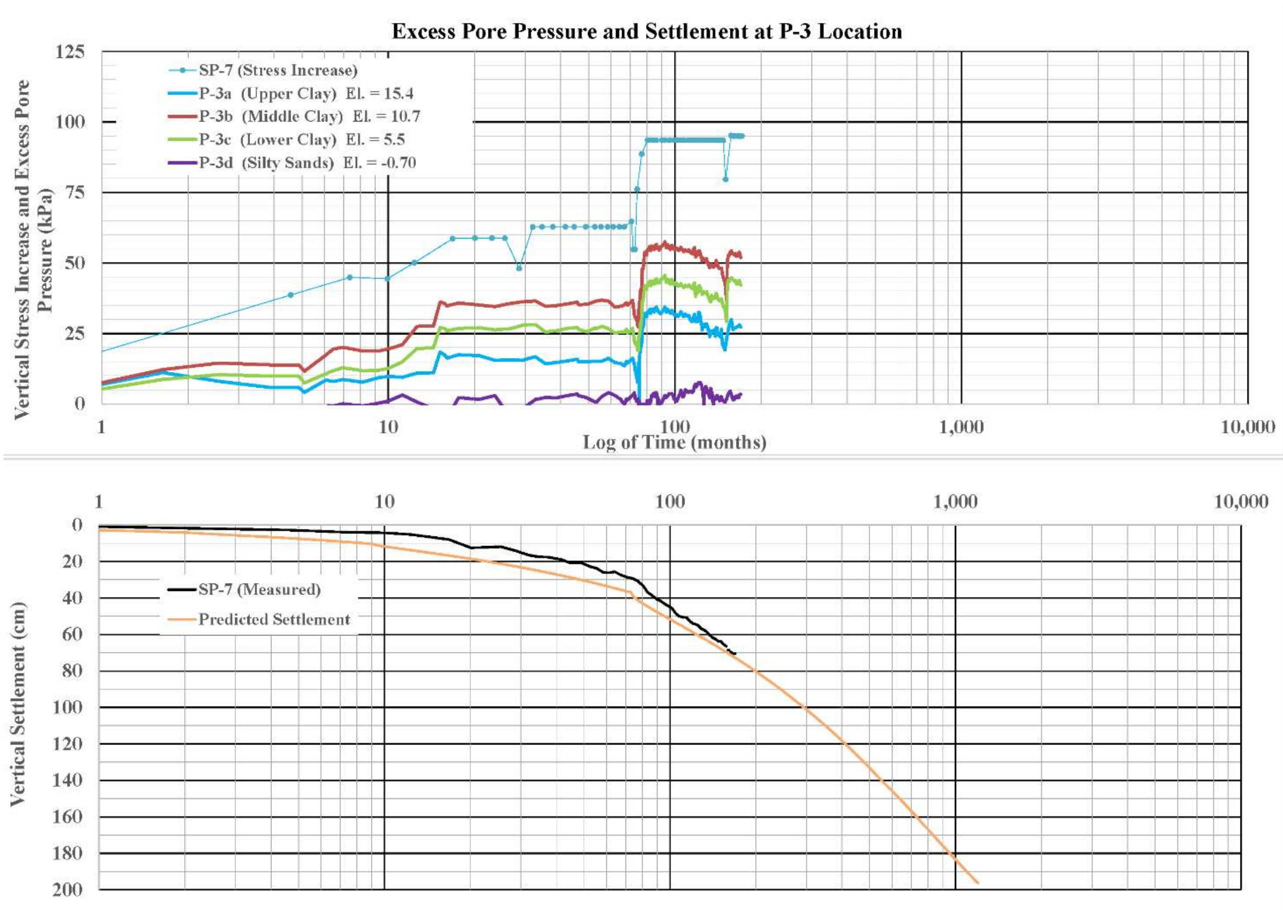

Figure 10. Excess Pore Pressure and Settlement at P-3 Location

RATIO: S_u from In Situ Vane Shear Testing/σ'_{vm} from Consolidation Tests

Ecomaine Ashfill/Balefill Facility

Vane Shear Test No	Depth (m)	Elevation (m)	S_{uFV} (peak) (kPa)	σ'_{vm} (kPa)	S_{uFV}/σ'_{vm}
P-3-2	10.0	16.9	15.0	95.8	0.156
P-3-4	11.1	15.8	15.2	105.3	0.145
P-3-7	12.5	14.4	13.1	95.8	0.137
P-3-10	14.0	12.9	12.4	114.9	0.108
P-3-13	15.5	11.4	15.5	113.0	0.137
P-3-14	16.0	10.9	16.3	113.0	0.144
P-3-16	17.0	9.9	15.0	110.1	0.136
P-3-17	17.5	9.4	15.5	110.1	0.141
P-3-20	19.0	7.9	18.1	105.3	0.172
P-3-21	19.5	7.4	13.6	86.2	0.158
P-3-27	22.5	4.4	18.8	153.2	0.123
P-3-28	23.0	3.9	18.1	151.3	0.120
P-3-31	24.5	2.4	28.2	148.4	0.190
		Average High and Good			**0.141**
B1-1	14.0	16.7	43.5	148.4	0.293
B1-4	15.5	15.2	21.4	143.6	0.149
B1-8	17.5	13.2	18.8	95.8	0.197
B1-11	19.0	11.7	17.1	76.6	0.223
B1-14	20.5	10.2	17.8	105.3	0.169
B1-17	22.0	8.7	19.8	138.9	0.143
B1-20	23.5	7.2	19.1	124.5	0.154
B1-26	26.5	4.2	17.4	124.5	0.140
B1-29	28.0	2.7	20.5	134.1	0.153
B1-32	29.5	1.2	27.9	114.9	0.243
B1-35	31.0	-0.3	36.1	177.2	0.204
		Average High and Good			**0.151**
B4-1	10.6	17.2	16.3	113.0	0.144
B4-5	12.5	15.3	15.8	91.0	0.173
B4-9	14.5	13.3	15.8	86.2	0.183
B4-12	16.0	11.8	16.8	114.9	0.147
B4-16	18.0	9.8	16.3	105.3	0.155
B4-18	19.0	8.8	15.0	105.3	0.142
B4-24	22.0	5.8	17.4	158.0	0.110
B4-27	23.5	4.3	19.8	153.2	0.129
B4-30	25.0	2.8	27.3	167.6	0.163
		Average High and Good			**0.147**
P5-1	11.9	17.2	12.9	97.7	0.132
P5-3	12.9	16.2	14.8	91.0	0.163
P5-6	14.4	14.7	16.0	105.3	0.152
P5-12	17.4	11.7	14.0	99.6	0.141
P5-15	18.9	10.2	18.8	101.5	0.185
P5-18	20.4	8.7	21.8	138.9	0.157
P5-21	21.9	7.2	20.1	122.6	0.164
P5-22	23.3	5.7	20.1	134.1	0.150
P5-25	24.9	4.2	20.1	114.9	0.175
P5-28	26.4	2.7	22.9	158.0	0.145
P5-31	27.9	1.2	28.2	153.2	0.184
P5-33	28.9	0.2	28.8	158.0	0.182
		Average High and Good			**0.158**

Vane Shear Test No	Depth (m)	Elevation (m)	Su (peak) (kPa)	σ'_{vm} (kPa)	S_{uFV}/σ'_{vm}
I5-3	5.4	18.0	25.7	114.9	0.224
I5-6	7.0	16.5	10.6	119.7	0.089
I5-9	8.4	15.0	10.4	86.2	0.120
I5-11	9.5	14.0	10.9	105.3	0.104
I5-15	11.5	12.0	12.5	92.9	0.135
I5-17	12.5	11.0	13.9	113.0	0.123
I5-21	14.5	9.0	20.7	127.4	0.162
I5-24	15.9	7.5	16.7	114.9	0.145
I5-27	17.5	6.0	16.7	105.3	0.159
I5-30	19.0	4.5	18.5	151.3	0.123
I5-33	20.5	3.0	21.7	158.0	0.137
I5-36	21.9	1.5	25.4	141.7	0.179
I5-39	23.5	0.0	30.1	162.8	0.185
		Average High and Good			**0.145**
B13-1-1	6.8	17.4	17.4	177.2	0.098
B13-1-3	8.0	16.3	11.7	105.3	0.111
B13-1-8	10.5	13.8	11.5	91.0	0.126
B13-1-12	12.5	11.8	15.0	81.4	0.184
B13-1-17	15.0	9.3	18.0	162.8	0.110
B13-1-21	17.0	7.3	15.8	143.6	0.110
B13-1-26	19.5	4.8	21.2	162.8	0.130
B13-1-30	21.5	2.8	20.8	162.8	0.128
		Average High and Good			**0.125**

Consolidation Test Quality

- High Quality
- Good Quality
- Poor Quality

Average High and Good (all tests) 0.144

Table 1. Summary of C/P Ratios

Several investigators addressed the issue of field strengths versus laboratory strength (Chandler, 1988, Aas et al., 1986, Bjerrum and Simons, 1960, Bjerrum, 1972, Ladd, 1972). The under prediction of the undrained strength by the vane shear test is generally attributed to vane disturbance during insertion in clays with high sensitivity and low plasticity. The Ecomaine clay is highly sensitive with a low plasticity. This issue is discussed extensively by Chandler and by Aas who suggested that the field vane in these types of clays can yield an undrained strength that is as much as 25 % below the undisturbed value. The ratio of average DSS test stress ratio for young clays (0.167) to the average field vane stress ratio 0.144 is 1.16, implying an average field vane correction value of 1.16. The average plasticity index (PI) for the site soils is 13.4. This correction factor for vane shear strengths is plotted on the well known Bjerrum correction factor plot shown on Figure 11 taken from Ladd and DeGroot, 2002. The Ecomaine data plots just above the Ladd and Foott, (1974) point which was for the I-95 test Embankment in Portsmouth, NH, also a Presumpscot Formation Clay site.

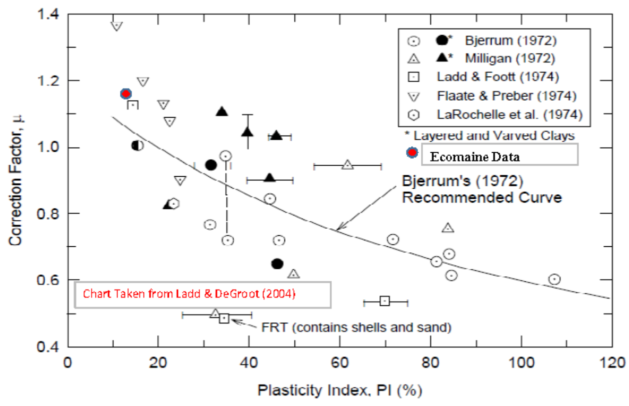

Figure 11. Bjerrum's Correction Factor for VST

While we believe that on-average, the in situ VST results will result in more conservative values of the undrained shear strength, the body of evidence at the Ecomaine site indicates that there are zones of lower strength clay at some elevations throughout the site that should be considered in the analyses. Simply characterizing the site based on average values to predict future strength, or to average out these lower strength zones in stability analyses is not recommended. All previous stability analyses indicate that these lower strength zones control the design heights of the Ashfill. It is further noted that the inclinometer data clearly indicates that the horizontal deformation around the perimeter is largely occurring in these low strength zones.

6. Conclusions

The field vane testing at the Ecomaine landfill site founded on soft sensitive clay of low plasticity was a key test in the design and monitoring of waste loading at this facility. The results, although conservative compared to laboratory testing, have been able to identify zones of lower strength in the stratum that needed to be considered in stability analyses and waste loading grades.

The episode 3 vane shear testing testing indicated that the undrained shear strength is generally increasing in most principal test locations. It is noted however that some of the episode 3 test locations results indicated some zones where little or no strength increase has occurred, even though up to 40 to 50 percent consolidation has occurred to date. Some of this lower strength could be related to disturbance due to vane insertion in the sensitive clays, however, it could also be due to a loss in strength due to strain softening. Future testing at the site will include Cone Penetration Testing (CPT) calibrated to the VST results to more closely monitor clay strength in the next dozen years prior to any site expansions.

7. References

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