25-years of loading an old Presumpscot Fm landslide: a case history

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ABSTRACT: This case history discusses why a landslide occurred when about 70 feet of fill were placed on a like thickness of the Presumpscot Formation. The case history then summarizes steps that have been taken at the site to avoid another slide, while permitting a new waste pile to grow to over 100 feet in height. Important examples of some of these steps are discussed in detail, including: how typical field and laboratory testing techniques can be modified to yield better data, how much undrained shear strain of the Presumpscot Formation is too much, how long-term drained strain will strengthen the clay, what less than obvious precautions, should be taken to avoid landslides when placing many stages of load on the Presumpscot Formation, and how automated geotechnical data collection systems can help maintain stable slopes, when manually collected data is economically unfeasible.

1 INTRODUCTION

In the mid-1970s, on a site in Norridgewock, Maine, the owner of the site began placing waste materials on top of the Presumpscot Formation after first removing a thin veneer of topsoil and eolian sand. At this particular location, the Presumpscot Formation comprised about 8 ft of stiff mottled olive brown clay over 50 ft of soft gray clay, all underlain by a dense glacial till.

By the mid-1980s, the landfill had grown to cover about 15 acres and environmental regulations required that any future expansion of the landfill footprint be underlain with an HDPE liner. While expansion plans and permits were being prepared, waste was placed higher and higher on the same footprint, such that by the late 1980s, the pile of waste had grown to a peak height of about 70 feet.

Up until that time, only a few borings had been drilled at the site for the purpose of determining the strength of the clay under the rapidly rising mountain of waste. Moreover, little information had been collected regarding the likely density of the waste. A few inclinometers and piezometers had been installed along the north, east and south sides of the waste pile in areas where future expansions would not require the removal of the instruments.

Construction of a westerly lined expansion area of the landfill began in spring 1989. The site preparation included excavation of 4 to 6 feet of the stiff mottled olive-brown clay to provide for waste capacity and inclination of a leachate collection system. In addition, a drainage trench was dug through the olive-brown clay along the western toe of the existing landfill area to collect leachate from the unlined area.

On 16 August 1989 a large landside dislocated about 1 million cubic yards of waste from the existing pile, plus some of the clay underneath, onto the landfill expansion area. Shortly thereafter, the owner, Consolidated Waste

Services, Inc. (CWS), regraded the debris to form a gentle slope over the former waste pile and the expansion area. The resulting clay covered waste pile was then about 40 feet high at its peak.

Investigations to pin-point the cause of the 1989 landslide concluded that the slide was likely triggered by sloughing of clay into the drainage trench next to the existing waste pile. The sloughed clay left a somewhat higher and steeper unsupported slope, which quickly slid onto the area where the olive-brown clay had been excavated. This in turn left an even higher unsupported slope, which also failed. By the time the retrogressive sliding stopped, one could clearly see 6 distinct blocks of waste from the former waste pile, "floating" on a base of remolded soft gray clay. In front of the 6 blocks was a wide debris field of waste mixed with remolded clay. Attachment A contains an air photo taken just after the landslide.

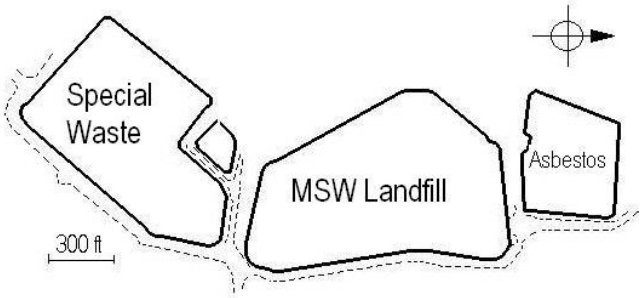

Figure 1 – Site plan of landfill at Norridgewock, Maine before the 16 August 1989 MSW Landslide. The small polygon next to the Ash/Sludge (Special Waste) Landfill was a leachate storage pond.

When the landslide occurred, CWS was operating three different landfill units at the Norridgewock site: the MSW landfill, which broke up during the landslide, a Special Waste landfill, and an Asbestos landfill. Figure 1 indicates the relative location and proximity of these three landfill units. These landfill units occupy only a fraction of the 400+ acre site that CWS owned, and other portions the site were considered as future possibilities for landfill units.

2 POST-LANDSLIDE INVESTIGATIONS

Whereas there was a scarcity of data gathered for geotechnical purposes before the landslide, since the slide there have been more than 700 borings, 2500 vane shear tests, and 600 Shelby tubes retrieved at the site. In discussing the investigative work that has been undertaken, it is convenient to address the work in the following order:

1) Work to assess the cause(s) and extent of the 1989 slide and to obtain Maine DEP authorization to reopen the Special Waste and Asbestos landfill units, which did not appear to have been affected by the landslide.

2) Work to plan, permit, construct and operate new landfill units.

3) Work to assess strength gain and side slope modifications of existing and future landfill units.

The scopes of these tasks are summarized below along with some of the more important findings. It should be noted that near the end of the landslide assessment phase of the work, Waste Management Disposal Services of Maine (WMDSM) purchased the entire site from CWS and renamed it their Crossroads Landfill site. WMDSM has subsequently and successfully planned, permitted, constructed and operated additional landfill units on the site, first adjacent to the slide area, then completely away from the slide area, and finally over top of the three landfill units shown in Figure 1.

2.1 *Assessing the MSW Landslide Causes and the stability of Special Waste and Asbestos Landfills.*

Time was of the essence for this work. Several drill rigs were mobilized to the site and drilled borings to:

- Determine if the landslide shear surface had carried waste down to the surface of the glacial till.

- Determine if the landslide had impacted the stability of the Special Waste Landfill or its Leachate Storage Pond, or the Asbestos Landfill.

- Install slope inclinometers and piezometers in and around the MSW debris flow, and around and under Special Waste and Asbestos Landfills.

The work also involved:

- Building a 4 ft x 4 ft shear box that was used to measure the shear strength of waste.

- Digging test pits with measurable dimensions so that waste could be weighed and volumetrically measured for density determination.

- Performing about 600 "Maine Test Boring" vane shear tests, a single Geonor H-10 Vane Borer test, 4 monotonic and 4 cyclic Direct Simple Shear (DSS) laboratory strength tests and performing cross-hole geophysical tests.

- Making mass balance estimates for the waste densities in the MSW, Special Waste and Asbestos landfill units from topographical surveys and scale weights.

- Running numerous stability analyses of the MSW slide and the slopes of the Asbestos and Special Waste Landfills

- Evaluating the potential for liquefaction of the Presumpscot Formation below the stability berms of the Asbestos landfill.

- "Reconstructing" the MSW landfill using the data from pre- and post- landslide topographic maps along with vane shear tests to generate the likely topography of the intact clay surface under the MSW Landslide area.

Some of the more significant findings from the assessment of the causes of the MSW landslide are discussed below, while detailed results of the investigations may be found in Reynolds (1991) and Richardson & Reynolds, (1991).

2.1.1 *Landslide causes*

It is likely that the retrogressive landslide would not have occurred if both the toe drain trench and the excavation of most of the stiff olive brown clay had not occurred. Moreover these excavations' effects were exacerbated by the previous rapid filling of the waste pile, the storage of excavated clay on top of the landfill and many inches of rainfall that fell just prior to the slide. The heavy rains elevated the ground water heads in the glacial till, thus increasing or causing upward seepage forces in the remaining clay. It was concluded that the shear strength along the entire shear surface of the retrogressive slide was originally sufficient to hold back the entire sliding mass, but because just a portion of the sliding mass was able to slide and then quickly move out of the way, further sliding could easily occur. Had excavations and backfilling at the toe of the waste pile taken place in small segments, the overall effect of the clay removal would have been much less.

2.1.2 *Locating the shear surface*

The availability of air photos and topographic maps before and after the MSW Landslide proved to be invaluable in the sketching of Figures 2 and 3. A few borings were drilled through the old waste pile to search for remolded clay. The technique of reconstructing the original landfill prior to the landslide, used by H.B. Seed, et. al. (1975) in their analysis of the Lower San Fernando Valley Dam failure was used to substantiate the depth of the sliding surface. As Seed, et. al. (1975) did, Figures 2 and 3 were drawn to scale. They are not merely artistic renditions of the slide. An excavation and

relocation of the MSW years later verified the location of the shear surface shown in Figure 3.

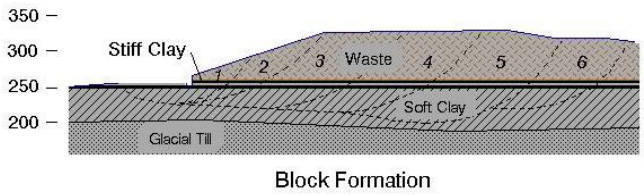

Figure 2 – MSW waste and clay blocks before breaking apart in the 1989 MSW Landslide.

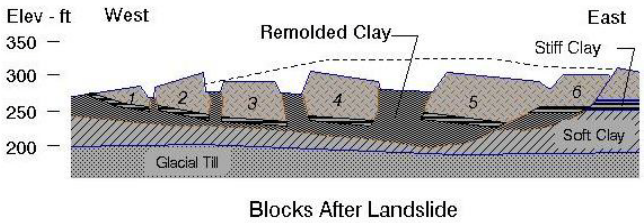

Figure 3 – MSW waste blocks "floating" in remolded and intact clay after the 1989 MSW Landslide. Enlarged versions of Figures 2 and 3 are included in Attachment A.

2.1.3 *Extent of damage*

A slope inclinometer installed within the debris flow from the MSW Landslide just after the slide, sheared off at a shallow depth a few days later. Pore pressures equivalent to more than 20 ft of water were measured in the clays remolded during the landslide. The maximum heads would have been higher since the piezometer measurements did not start immediately after the clays were remolded. Some of the excess pore water pressure dissipated upwards, appearing as small boils.

Slope inclinometer and survey monument measurements taken between the edge of the debris flow and the Special Waste and Asbestos Landfills and the Leachate Storage Pond indicated no impact of the landslide outside of the edge of the debris flow.

2.1.4 *Assessing stability of adjacent landfills to gain approval for their reopening*

Vane shear data were collected and used in stability analyses of the Special Waste and Asbestos landfills to demonstrate the safety of their reopening. A side by side comparison of vane shear strengths measured with the conventional "Maine Test Borings" vane and the Geonor H-10 Vane Borer indicated average vane shear values that were 15% higher when the Vane Borer was used. Based on this finding, after April 1990, the author's use of the MTB vane was minimal, and for vane shear testing, vanes from the H-10 vane borer along with light-weight actuator rods and frequently calibrated torque wrenches with memory dials were the rule. The primary reason why MTB vane indicated lower vane shear strengths was believed to be disturbance of the soft clay when the heavy-duty vane was pushed into the clay prior to torqueing.

It was difficult to demonstrate a reasonable factor of safety of the slopes around the Asbestos Landfill using the pseudo-static stability analysis approach, when considering an earthquake. Consequently, the author used a liquefaction analysis to show that without liquefaction, the slopes would remain stable. Liquefaction was shown to be unlikely using a SHAKE analysis and cyclic shear strengths. This required cross-hole tests to measure shear wave velocities of the fill (600 fps), clay (800 fps for olive brown, and 600 to 400 fps for gray) and glacial till (1200 fps). These velocities were used in SHAKE along with the characteristics of the 1988 Saguenay Earthquake to estimate the induced cyclic shear stresses. The induced cyclic shear stresses were typically in the range of 100 to 200 psf. The shear stresses needed to cause liquefaction of the clay, based on cyclic DSS shear test results, were typically in the range of 300 to 500 psf.

2.1.5 *Difficulties of drilling through landfills*

It was very expensive to drill borings through the MSW landfill, not only because of the obstructions that slowed down progress, but because a considerable amount of drilling

equipment was lost. When waste was caught in the drill bits and jammed in the lower portions of the steel casings, several feet of the driller's tools could unscrew in the bore hole, without him even knowing the tools, including diamond bits, were being unscrewed (until it was too late).

2.2 *Work to plan, permit, construct, and operate new landfill units*

This work was significantly impacted by landfill regulations that came into effect shortly after the 1989 MSW Landslide. The regulations allow for waste to be placed in higher and higher stages with intervening consolidation periods, but require corroborating field and laboratory data that support the shear strengths used in determining the stability of all stages. For the Crossroads Landfill, there was an additional requirement that an independent review by a geotechnical consultant would take place before waste placement in each stage. The reviews were under the direction of the Maine DEP, but paid for by WMDSM. The investigative work to conform to these requirements included:

- Drilling borings to determine the Presumpscot Formation thickness under the future landfill units.
- Running vane shear tests to estimate the virgin clay shear strengths, and attempting to measure the effects of consolidation under landfill berms.
- Running oedometer tests, and a few more DSS tests and triaxial compression and extension tests on virgin clays.
- Installing slope inclinometers and piezometers at the toe of slope on critical cross sections, along with piezometers and a few settlement plates under slope areas of these sections.
- Responding to the implications of the 1991 Federal Subtitle D regulations for the seismic design of landfills.

2.2.1 *Shear strength testing for new landfill units*

The use of the vane shear test to collect shear strength information was driven by two factors. First, while the ultimate goal was to gather sufficient undrained direct simple shear (DSS) and stress history (consolidation) data so that the SHANSEP method (Ladd & Foote, 1974) could be used to determine the design shear strengths, we lacked time and consistently high quality samples for consolidation testing to accomplish this goal in a reasonable time-frame. Second, at that time, the independent geotechnical reviewers, SW Cole Engineering, were more familiar with the use of vane shear tests than the SHANSEP method for determining undrained shear strengths.

During the investigations for the design of new landfill units, there were more than 2000 vane shear tests, using the Geonor H-10 vanes, light weight actuator rods and calibrated memory dial torque wrenches. The bore holes for these tests were usually drilled using weighted drilling fluids. For field-lab corroboration purposes, Shelby tube samples, collected from some of the same borings, were tested in standard oedometer consolidation devices. Just over 50 oedometer tests were performed.

While the quality of the sampling, specimen preparation, and testing for the 50 oedometer tests was sufficient to demonstrate corroborating field and laboratory test results of the shear strengths, it suggested several opportunities for improvements. Those steps were taken later when it came time to use consolidation tests to demonstrate strength gains that were needed for future lift stages and side slope modifications.

One goal that was not met by vane shear testing was to demonstrate strength gain that occurred under stability berms that had been in place for several years. This may have been due to disturbance of the low OCR clays below the berms during drilling and/or the direction in which shear strength is measured by a vane. These topics are discussed later in this paper.

2.2.2 *Monitoring stability and strength gain*

Inclinometers and piezometers installed in the clay below the landfill berms at the toe of waste slopes have been used to monitor stability during and after waste placement. (Settlement plates and piezometers under the waste piles are used to monitor consolidation.) WMDSM technicians read the instruments, typically weekly during waste placement, and provide timely and fairly precise estimates of the waste elevations above the instruments to those of us who interpret the movement and pore pressure data.

The question of how much movement or pore pressure during stability monitoring is acceptable is often based on experience, which can change with interpretation after the readings are made. At Crossroads, the question was addressed before filling using the following rationale:

- Based on DSS testing of soft gray virgin clays from the site, the typical undrained strain at peak strength averages about 2%, and the minimum observed has been approximately 1%.

- Strains measured in the field are presumed to be undrained if the pore pressures are increasing and drained if the pore pressures are dissipating.

- Presuming some undrained strain would occur before and after a peak strain rate occurs, 0.25% strain was decided to be the limit before remedial action is required.

- The typical time needed to react to an excessive rate of movement, including the evaluation, design, and construction of a response would be about 1 week.

- If measurements are taken weekly, it is possible up to 7 days of reaction time could elapse before an issue is observed.

Given this situation, remedial action is required if 0.25% undrained shear strain in two weeks is seen. This strain rate was termed the Red Alert level. If a Red Alert is confirmed, WMDSM would stop all filling activity and start taking steps to increase the stability of the area in question. Note that different clays, or different abilities to respond, or different monitoring frequencies might require different maximum movement rates.

There have been no Red Alerts at Crossroads. There have been two cases when a strain rate of approximately 0.1% in two weeks has occurred, and WMDSM took action. In the first case, WMDSM stopped waste filling in the area and increased the size of the stability berm, which cut the strain rate in half over a period of a few weeks. In the second case, WMDSM slowed down the rate of final cover placement, which also cut the strain rate in half.

Pore pressures are always measured when inclinometer measurements are taken. Other than using the pore pressure trends to decide if there are drained or undrained conditions, no limit has been set on the maximum size of a pore pressure spike. This is primarily because the amount of pore pressure build up at peak strength in the dozens of DSS tests that have been run on the Presumpscot Formation has been quite variable.

2.2.3 *Satisfying Subtitle D requirements*

The US Code of Federal Regulations, Title 40, Subtitle – D (1991) established new criteria for the seismic design of solid waste landfills. Essentially, one must show that the slopes of a waste pile will be stable after the earthquake, the movement as a result of the earthquake will not harm the leachate collection system, and the soils under the waste pile will not liquefy.

Makdisi, F.I., et.al. (1978) provided a rather simple procedure for estimating movements of a slope which can be utilized to evaluate the question of whether or not the design earthquake will damage the leachate collection system. If it can be shown that the soils under the landfill lose only a minor amount of shear strength due to the shaking, the question of liquefaction becomes moot. Thus the Subtitle D requirements for

Crossroads generally boil down to determining the strength loss after the earthquake shaking.

Recognizing that the Presumpscot Formation can lose strength if shaken hard enough, we set about to prepare a procedure that could be used at Crossroads to estimate the shear strength loss due to shaking in the range of motions expected from earthquakes in Maine. The work resulted in a simple plot of the soil strength loss ratio as a function of the stresses existing under a slope just before shaking and the soil's peak cyclic shear strain that is induced by an earthquake.

Reynolds (1995) presents details of this work. In summary, the plot for estimating the strength loss is provided in Figure 4. The steps that are followed are listed below Figure 4.

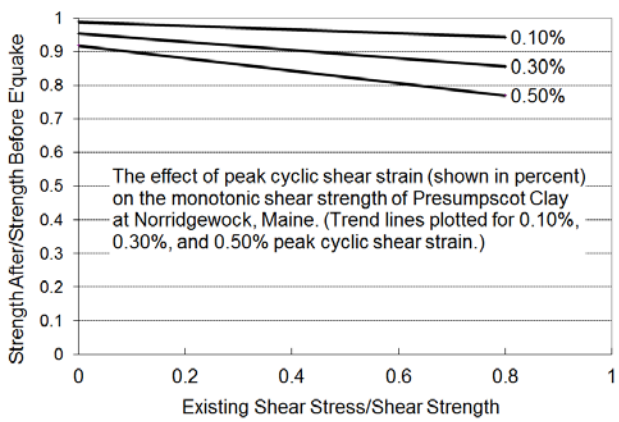

Figure 4 - Monotonic shear strength loss as a function of the peak cyclic shear strain and existing shear stress ratio (from Reynolds, 1995)

Steps for determining the strength loss at Crossroads after earthquake shaking:

1. Develop a shear strength profile under the embankment – the SHANSEP method is typically used for this determination.

2. Determine the characteristics of the design earthquake – Weston Geophysical Corp. provided an acceleration time-history for this purpose.

3. Use SHAKE and the Weston acceleration time-history to compute the peak cyclic shear stresses in the clay.

4. Use the Plaxis finite element program to compute the pre-existing shear stresses under the embankment (or use a slope stability program to compute shear plane stresses and then estimate pre-existing shear stresses.

5. Use Figure 4 to determine the loss of shear strength as a function of depth and then run a traditional stability analysis without any seismic coefficients applied to determine the minimum factor of safety.

2.3 *Assessing strength gain and side slope modifications of existing and future landfill units*

While hundreds of borings were drilled for geotechnical purposes in the process of investigating the causes of the MSW Landslide and for planning, permitting, and placing instrumentation during the construction of new landfills, a relatively few borings (35) have been drilled to assess strength gain and the potential for side slope modifications. Those borings that have been drilled have typically been located on the critical cross sections, through the stability berms and on virgin ground outside the toe of slope on these sections. In each of these borings, numerous Shelby tube samples (a total of 180) have been recovered. At some locations, the driller has gone back to collect more tubes after the passage of time so that consolidation testing could be done to assess strength gain or to try out a new procedure to collect higher quality intact samples.

It should be noted at this point that "strength gain" of the clays is not always due to consolidation – a process that may be verified with field sampling and consolidation testing. In other words, some of the apparent strength gain that has been shown has in fact been due to the improved sampling, specimen handling and

testing techniques used. Some has been due to actual consolidation. And, some has been measured only through strength testing of clay that had not undergone consolidation, but has been subjected to shear loading.

2.3.1 *Improvements in the sampling process*

When many geotechnical investigations were started right after the 1989 Landslide, borings were typically advanced by driving 4 inch diameter casing and then flushing the casing with water to clean out the cuttings and expose "virgin" clay at the bottom of the casing. For intact samples, a Shelby tube was attached to the driller's N-Rod and pushed into the bottom of the borehole with the weight of the drill rig.

When the sampling process resulted in many of the Shelby tubes filled with disturbed clay in obviously virgin ground, the first change was to use bentonite drilling mud in all borings that were drilled for geotechnical sampling purposes. This meant that no longer could geotechnical borings also be used for installing ground water sampling wells, which was the prior practice. The benefit of using the drilling mud to flush the cuttings was to provide resistance to heaving of the clay at the bottom of the borehole upward into the steel casing.

After noting that some of the Shelby tubes still encountered disturbed clay at the bottom of the bore hole, barite was added to the drilling mud to increase the density of the drilling fluid. While this provided more resistance to the clay heaving, we still retrieved too many Shelby tubes with disturbed soil in the top portion of the sample tube. We then changed from simply pushing the Shelby tube with the drill rod to using an Acker GUS hydraulically actuated fixed piston sampler.

The sampling changes noted above took place over a span of about 10 years and with them we were often able to retrieve relatively good quality intact samples, especially when sampling soft clays that had an OCR of greater than about 1.3. When the OCR is less than about 1.3, we observed that the bottom portion of the sample would fall out of the Shelby tube and hairline cracks would form in the remaining clay. Some engineers have had success in minimizing the hairline cracking by minimizing the turned-in edge of the Shelby tube so that little clearance and more friction exists between the clay and the tube. We tried this modification of the tubes, but still had difficulty retrieving crack-free samples in the low OCR clays.

While making the changes in field sampling process, we also gradually moved all of the specimen selection and preparation work (as well as the consolidation testing) to the Geotechnical Laboratory at MIT. At the time, that lab was the only lab in New England equipped with X-ray facilities to make radiographs of the sample tubes, which enabled us to see the hairline cracks and other indications of disturbance.

The radiographs also permitted us to select the more clayey specimens for consolidation testing. Note that the tubes often contain alternating layers of more clayey and then more silty soil. Selecting the more clayey soil, while perhaps biasing the compression indices of the results, provides specimen that are less easily disturbed when preparing the soil for consolidation testing, as well as consolidation curves that more readily indicate the preconsolidation pressure. And it is the preconsolidation pressure (stress history) that we are most interested in knowing when using the SHANSEP method to determine shear strengths.

Note that after landfill construction, low OCR clays often exist below the stability berms due to the weight added by the berms. Because of their location, these clays also provide significant resistance to sliding of the adjacent slope. Consequently knowing the shear strengths of these clays, or if there has been an increase in the strengths of these clays is quite important. Starting in 2011, to overcome the difficulties of retrieving high quality intact samples of clay when sampling low OCR clays, we made an additional change to the sampling process.

The author knew from hand boring sampling of the soft gray clay below the water table that the

suction force on the bottom of the sampler at times is almost more than a person could overcome (lift). Measurements of the force (tension on the drill rig cable) when removing the fixed piston sampler from the borehole have indicated, net of the weight of the sampler and drill rods, and net of the friction on the sides of samples, a suction force of about 175 lbs.

A suction force of about 175 lbs. at the bottom of the sampler suggests that quite likely cavitation occurs in the cavity below the sampler when the samples are withdrawn from the bottom of the borehole. And the suction on the clay in the sampler would be on the order of 1500 to 2000 psf. This suction could easily cause portions of the soil in tube to fall out and/or cracking or stretching of the soil. We often observe both of these phenomena when sampling soft clay. Stretching will cause lower preconsolidation pressures to be measured in the ensuing consolidation tests.

There have been various tools developed to break the suction at the bottom of the sampler, e.g. the Laval sampler. We chose to make our own tool with a very small diameter jet drill rod that would just fit between the fixed piston sampler and the 4" diameter casing. Figures 5 and 6 illustrate the suction breaker jet rod.

Figure 5 – Jet rod with side discharge tip attached to bottom of Acker GUS sampler. Once the sampler is resting on the bottom of the borehole, the jet rod is held up so it does not advance when the Shelby tube is hydraulically pushed.

Figure 6 – Geologist, Bob Estes, using an up-down and rotating motion on the jet rod to wash a small hole from the bottom of the borehole to the tip of the GUS sampler.

Determining the correct jet pressure is a matter of trial and error. Too much pressure can wash too large of a hole at the tip of the sampler – too little pressure will not sufficiently create a small boring alongside the sampler to completely release the suction. The jet pressures tend to range between 30 and 100 psi. The size of the jet at the bottom of the 3/8" IP drill rod is also a matter of trial and error. A 1/8" hole seems to be about right.

Since the suction breaking tools have been used, the issue of retrieving intact samples with cracks has generally been overcome.

Figure 7 provides a comparison of three preconsolidation pressure curves for Presumpscot Formation specimens from essentially the same location. The curves were based on the data available when they were drawn. All three curves used least-squares curve fitting for their respective data. The red curve was drawn, based on data collected through about 1998 – the data points are shown in small circles. The sampling and testing up to 1998 basically comprised conventional push Shelby tubes and oedometer tests.

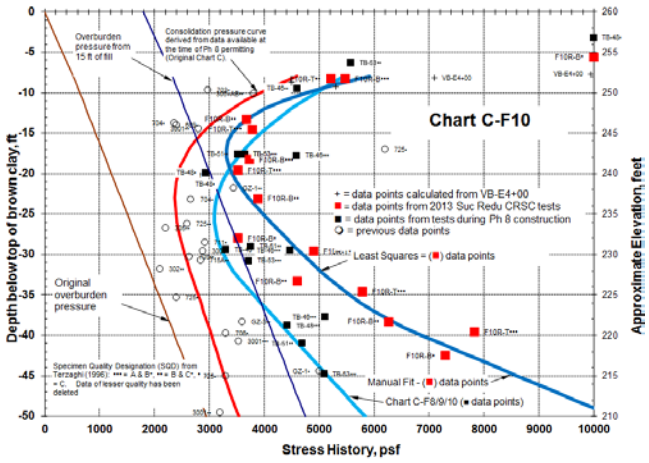

Figure 7- Stress history or preconsolidation stresses vs. depth for specimen obtained before 1998 (red curve), between 2000 and 2005 (light blue curve) and in 2013 (dark blue curve. Enlarged versions of Figures 7, 8, and 9 are included in Attachment A.

The light blue curve is based on data collected between about 2000 and 2005. The data points from which the light blue curve was drawn are shown with black squares. Those data are from CRSC (constant rate of strain consolidation) tests on tubes collected with the Acker GUS sampler, where the specimens were x-rayed. The dark blue curve is based on data collected in 2013 using all of the improvements in the sampling and specimen selection discussed above. These data also come from CRSC testing.

2.3.2 *Shear induced strength gains*

In late 2003, the author was having a conversation with the independent geotechnical reviewer, Tony Hersh of S W Cole, Inc, when he asked why we saw a buildup in pore pressure at the toe of a landfill in early 2001. That was long after the stability berm fill had been placed at this location (in 1999). Figure 8 shows the pore pressure curves he was asking about. The burgundy curve (horizontal line after 1999) indicates the fill level over the piezometers at this location.

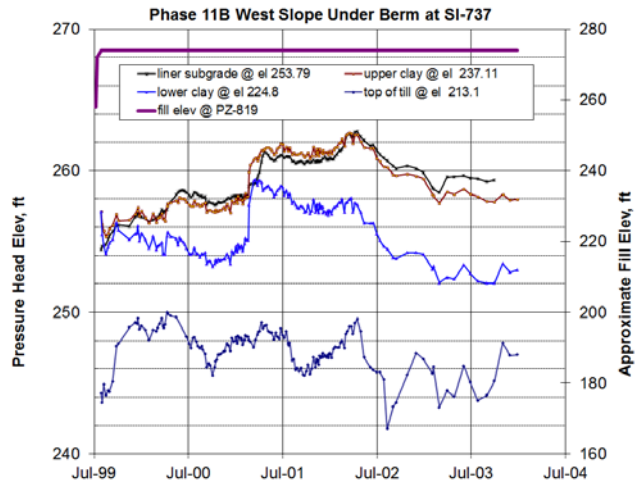

Figure 8 – PZ 819 located under the stability berm at the toe of the waste slope.

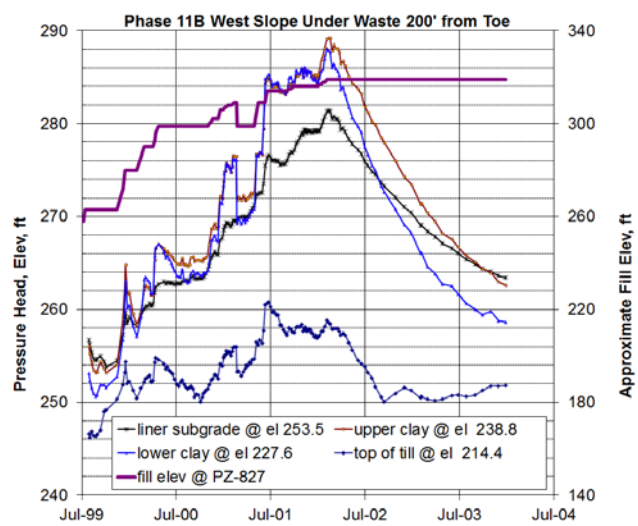

Figure 9 – PZ 827 pore pressures and waste elevation about 200 ft away from PZ 819

Figure 9 illustrates the timing of waste placement next to the berm at this location. The initial response to his question was that the pore pressure increase was probably due to horizontal compressive stress on the clays under the berm due to waste loading placed next to the berm. This response was adequate for the time being, however, his question raised other questions, such as, if there is pore pressure buildup, and then dissipation, how much shear strength would be gained under the berm due to that consolidation process? And why did the pore pressures increase in the deeper clays when there was little horizontal movement at that level?

In looking for answers to those questions, the author read a paper by DeGroot, et. al. (1996), which included the results of DSS tests that showed that the undrained strength ratio of Boston Blue Clay increases when shear induced pore pressures have built up and dissipated. They ran tests where the clays were: 1) vertically consolidated, 2) subjected to drained horizontal shear stress, and 3) sheared to failure without further drainage. When the drained shear stress (τ_{hc}) was about 50% of the original undrained shear strength, the subsequent undrained shear strength (τ_h) increased by about 10%. When τ_{hc} was increased to about 95% of the original undrained shear strength, τ_h increased by about 40%.

The DeGroot, et. al. (1996) results were presented to WMDSM along with a request to run a few similar special DSS tests to investigate how the clay would gain strength after: 1) being vertically consolidated, 2) being subjected to undrained shear stresses, 3) being allowed to dissipate the pore pressured built up during that shearing, and 4) then sheared again to failure. WMDSM agreed with the request and actually authorized several special DSS tests that would replicate the way waste is placed in stages at Crossroads.

Figure 10 shows typical stress-strain curves from the one and two-stage DSS tests that were run. The single stage test (blue data) indicated that the undrained strength ratio (S_1) of the specimen would be approximately 0.19. The initial stage of the two-stage test (red data) was run with a shear stress ratio of 0.15, which corresponds to a strength to stress ratio (factor of safety) of 1.26.

As shown by the red data points in Figure 11, after the specimen was stressed to a shear stress ratio of 0.15, it was allowed to undergo drained strain. This is typically what happens during the landfill waste placement, where a stage of waste is placed and then remains at that elevation for some time while drained strain of the clay takes place.

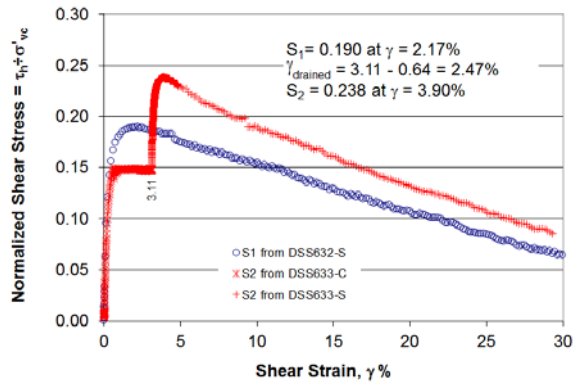

Figure 10 – Stress-strain curves from one and two-stage DSS tests with results of stage 1 in blue and stage 1-2 in red. S_1 and S_2 are the peak normalized shear stresses.

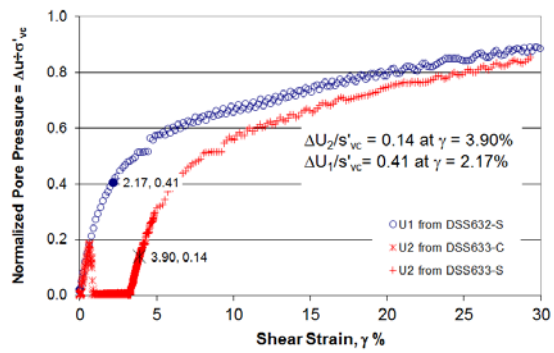

Figure 11 – Pore pressure curves from one and two-stage DSS tests with results of stage 1 in blue and stage1-2 in red. See Attachment A for enlarged versions of Figures 10 -11.

After the constant stress and zero pore pressure waiting period shown in Figures 10 and 11 (where the stress ratio was held at 0.15), new shear stresses were applied until the specimen failed, not at an undrained strength ratio of 0.19 as it would have in a single stage test, but at a ratio of 0.238. This represented a strength gain of approximately 25% due to the application of the drained shear stresses.

While WMDSM had been counting on strength gain of the Presumpscot Formation under the waste piles from the application of vertical stresses, it had not been counting on a 25% increase in the shear strength under the berms and landfill slope areas. This possibility justified the expenditure of more funds to more fully investigate SISGa (Shear Induced Strength Gain) as the phenomenon is abbreviated.

To date, a total of 16 sets of two-stage, with a corresponding single-stage, DSS test have been performed on the Presumpscot Formation from Norridgewock. Three of these multi-stage tests have been run with a third stage of loading to investigate how much additional SISGa occurs after two drained strain waiting periods.

Reynolds and Germaine (2007) provide many more details of the multi-stage DSS testing. Briefly, Figure 10 shows the results of a set of multi-stage DSS tests where the ratio of shear stress to shear strength during the drained strain period was 0.15 (the consolidation stress, σ_{vc}, was constant throughout the set of tests). In this test, the drained strain was 2.47%.

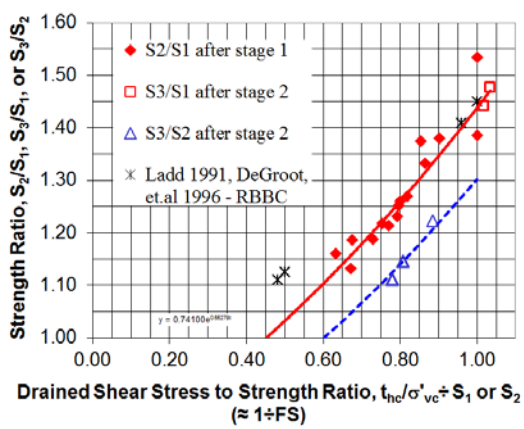

Figure 12 – Recommended plots for determining shear induced strength gain (SISGa). Red and blue data points for Presumpscot Formation, black asterisks for Resedimented Boston Blue Clay. See Attachment A for an enlarged version of Figure 12.

Figure 12 graphically shows the results of the 16 sets of multi-stage DSS tests that have been run. In these tests, it was found that the amount of drained strain did not influence the SISGa, whereas the shear stress to shear strength ratio did have a significant effect on the SISGa. In other words, the closer the clay gets to failing in the initial loading, the stronger it is after the pore pressures from shearing dissipate. This is the same trend that DeGroot, et. al. (1996) found for the Boston Blue Clay. In addition, Professor Charles C Ladd, in his 1986 Terzaghi Lecture also briefly mentioned SISGa and found the same trend (Ladd, 1991).

To use Figure 12 in the design process, the precise determination of the shear stress during drained strain period would require considerable effort, e.g. using numerical analysis (a finite element model) for each profile where SISGa is to be used. From a practical standpoint, however, the average shear stress to strength ratio, at least in the zone of potential shearing of a sliding mass, is simply 1÷FS. FS is the factor of safety of the slope in question, which can be determined from slope stability analysis. Thus, the ordinate of the Figure 12 plot includes this approximation, and as long as one does not stray too far away from the potential shear zone that existed during the drained strain period, the average shear stress to strength ratio to be used in Figure 12 will be known.

On Figures 10 and 12, the terms S_1, S_2, and S_3 refer to the undrained strength ratio, S for the first, second and third stages of fill slope construction. Ladd and Foote (1976) discussed S and how it is to be used for computing undrained shear strengths in the SHANSEP equation. That equation is:

$$S_u = S \cdot \sigma_{vc} \cdot (OCR)^m \qquad (1)$$

Where:

S_u = undrained shear strength
S = undrained strength ratio
σ_{vc} = the consolidation stress of the clay, for an example, see Figure 7.
OCR = overconsolidation ratio of the clay
m = the overconsolidation coefficient (0.8 for the Presumpscot Formation).

The application of SISGa in the design of a multistage fill slope can be summarized in the following steps:

1) Determine the shear strength profile(s) in terms of S_u vs. depth using the SHANSEP equation (Equation 1), with S_1 as the undrained strength ratio, σ_{vc} and OCR values at the end of the drained strain period.

2) Compute the Factor of Safety (FS) of the slope and make note of the location of the shear surface location associated with the minimum FS.

3) Using Figure 12 with the FS computed in step 2, determine the value of S_2.

4) Return to step 1) and compute the shear strengths using the value of S_2 determined in step 3.

5) Compute the FS with the second stage of fill in place and make note of the location of the shear surface associated with that FS.

6) If the shear surfaces in steps 2 and 5 are the same, the results from step 5 are appropriate and the process is finished.

7) If the shear surfaces from steps 2 and 5 are not the same (which is normal), fix the shear surface from step 5 into the FS analysis of step 2. Repeat step 7 until the shear surfaces from steps 2 and 5 are the same (within a few feet of each other is sufficient).

Users of Figure 12 and Steps 1) to 7) will soon note that determining the appropriate SISGa for staged construction is an iterative process. To reduce the number of iterations, it helps to estimate where the shear surface will be for the new left of fill and fix that surface into a stability analysis for the previous lift (Step 2). The users will also note that SISGa gradually declines as one moves away from the previously critical shear surface. This can be seen in the shear stresses determined by a finite element analysis of the previous slope. It can also physically been seen from the shear strains (or incremental deflections) observed in a slope inclinometer placed at the toe of a slope.

Reynolds and Germaine (2007) included a word of caution in setting limits on movement or pore pressure buildup when multi-stage fill construction occurs. The origin of this caution can be seen in Figures 10 and 11, and it applies even if SISGa is not actually used in the design of a slope.

Both the shear strains to failure, and the pore pressures built up after drained strain occurs, are reduced for the subsequent stages of filling. This means that limits on movements measured in inclinometers or pore pressures observed from piezometers that may have been established for the first stage of loading will be non-conservative for the later stages.

At Crossroads, whereas a limit (Red Alert) of 0.25% strain in two weeks was set for the initial stage of waste placement in a landfill cell, it was reduced by two-thirds to 0.083% in two weeks for the subsequent stages. The two-thirds reduction factor came from the average ratio of strain to failure in the second stage DSS tests. For example, for the clay in Figure 10, the reduction factor was (3.90% -3.11%) ÷ 2.17% = 0.36

2.3.3 *The fine print on using Figure 12 when building over an old landslide and/or placing many many stages of fill*

Figure 12 primarily summarizes the results of tests that were run the same way as those illustrated by the data on Figures 10 and 11. That is, the two- and three-stage tests were run on intact clay samples that were initially subjected to drained shear and then undrained shearing to failure.

The two data points (red diamonds) on Figure 12 that are shown with an ordinate of 1.00 were from tests run differently and were run well before an investigation of the magnitude of shear induced strength gain began. Those tests were run for the purpose of evaluating the undrained strength ratio of clays that had been completely remolded by the 1989 landslide. The testing attempted to replicate the normal and shear stress history of the clay under the MSW landfill. Thus, the specimens were:

1. Normally consolidated to a constant stress

2. Undrained sheared in a DSS apparatus to a shear strain of 10%
3. Sheared back to zero percent strain
4. Reconsolidated back to the original consolidation stress
5. Undrained sheared again to a strain of 10%

Steps 3. to 5. were repeated a second time to see if there was a change in the strength between the second and the third shears, i.e. to judge if remolding was complete. The second and third strength ratios for the test that is plotted above the trend line (with an ordinate of 1.00) on Figure 12 were the same. The test value that is plotted below the trend line (with an ordinate of 1.00) on Figure 12 fell well above the trend line during the third shear. It was concluded that upper data point on the 1.00 ordinate of Figure 12 represented a value that could be used in design analysis for building on top of the old MSW Landfill footprint as long as it was known that the clays were completely remolded and reconsolidated.

When planning for the excavation of the remains of the MSW Landfill in the early 2000's, it was necessary to evaluate the stability of the excavation, to know what excavation slopes were appropriate and to not compromise the stability of the surrounding landfills. (See Luettich, et.al. in the proceedings of the 2nd Symposium of the Presumpscot Formation.)

For those excavations and for subsequent waste fill slopes starting on the current landfill floor, two sets of strength profiles were developed for areas where the potential shear surface would extend under the old MSW Landfill footprint – one using the shear strengths of virgin clay and one using the shear strengths for remolded clay. Because of the uncertainty of knowing there would be 100% remolding of the clays, whichever strength profile indicated the lower shear strength at any particular depth was used in design. That resulted in the use of the virgin shear strengths almost everywhere.

It should be noted that the stress-strain curves for the remolded clays do not demonstrate the characteristics of a strain-softening soil the way the virgin clays do. Figure 10 illustrates how peak shear strength is developed at a fairly low strain and then the strength drops off after that. The stress-strain curve from testing remolded, reconsolidated clay tends to follow the same curve of the virgin clay up to near the peak on the virgin clay curve and then continues to rise, i.e. the clay becomes strain hardening.

The strain hardening characteristic of the remolded clay can be an important asset when working on clays that have undergone an old landslide. While the designer may not be able to consider it in the slope design process, wherever the contractor encounters it he/she will find much more favorable trafficability conditions for his/her equipment.

The author observed this favorable condition when remediating an old landslide in the Rockland, Maine area. In that case, excavation work at the toe of the slope of the old landslide caused a few inches of heaving of the ground beyond the toe, but did not precipitate a new retrogressive landslide.

Figure 12 also includes two data points that are plotted higher than a strength ratio of 1.00 in red squares with no in-fill. As the legend indicates, these points came from the results of a three-stage DSS test, where the strength from the shearing to failure was divided by the shear strength from a single loading stage test (S_3/S_1).

The purpose for including the S_3/S_1 results on Figure 12 is to demonstrate that a curve developed from two-stage testing is valid for any number of stages, as long as the last stage has been in place long enough to have had drained strain. For example, if 4 stages of fill had already been placed at a site, and the designer wanted to place a 5th stage considering SISGa, he/she would compute the FS of the 5th stage without considering SISGa. Then from the red line on Figure 12, determine a value for S_5/S_1.

The S_5/S_1 value would then be used in the iterative process indicated above when performing Step 7). The FS for the slope with 5 stages of fill, but with the shear strengths without SISGa computed from any stages would be quite low. The reciprocal of that value (1÷FS) would be quite high. Since the 1989 landslide, however, the author has never seen FS_1 values (i.e. without SISGa) less than about 1.18. That has been for the case where the final FS, considering SISGa is desired to be in the range of 1.50.

2.4 *The 2008 peer review of Crossroads Strength and Stability Analyses by Prof. Charles C. Ladd*

In early October 2008, the independent geotechnical reviewer, T. Hersh, of SW Cole Engineering, Inc. obtained concurrence from the Maine DEP that an expert with experience with SHANSEP and SISGa be retained to review the author's work at Crossroads. T. Hersh first approached Don DeGroot at U Mass Amherst, who in turn recommended that Prof. Charles C. (Chuck) Ladd be the reviewer. Ladd accepted the assignment with the understanding that he would decide the depth of the review.

The peer review lasted 2 months with Ladd spending an estimated 150 hours and reviewing approximately 200 pages of documents. The scope of his work included review of lab data, field monitoring data, spreadsheet input and graphical outputs, previous stability analyses results and the results of new analyses to judge the effects of his suggestions.

The issues cited in Ladd's report Ladd, (2008) comprised:

- How was Figure 12 developed?
- How is Figure 12 applied to OC clays?
- What consolidation state should be considered for the FS used in Figure 12?
- What field evidence is there for SISGa?
- Are sufficiently wide areas searched in the stability analyses?
- Is DSS appropriate for all potential shear zone areas?
- Should there be a limitation on the shear strength used for the clay crust?
- Wouldn't a reliability based stability analyses combined with conventional limit equilibrium analysis be better than just the limit equilibrium analysis?

While Ladd asked how we came up with Figure 12, we asked what he thought the reasons for SISGa were. We explained that we had tried to use sampling and CRSC testing to confirm an increase in the preconsolidation pressure of the clay after it had undergone drained shear in the field, but consolidation tests rarely would indicate the level of strength gain that we found in the two-stage DSS tests. Ladd's explanation was quite simple. The strength gain comes from both an increase in the preconsolidation pressure and a change in the direction of the principal stresses in the clay when it is sheared. Thus you should not expect to be able to attribute more than about 50% of the SISGa to a change in the preconsolidation pressures measured by the CRSC tests.

Ladd's response did not change the results in Figure 12, but it did change the way we computed shear strengths with the SHANSEP equation (Equation 1). Instead of increasing the OCR in the equation, we left the OCR as it can be computed from Charts like Figure 7, and we changed the values of the undrained strength ratios, S_2 and S_3. This simplified the computation of the shear strengths for Crossroads.

Other changes in the way the clay strengths at Crossroads were determined, as a result of Ladd's peer review, included:

1) For overconsolidated clays, the SISGa indicated in Figure 12 is now linearly prorated such that the full values of S_2 and S_3 shown on Figure 12 are used when the OCR is 1.0 and zero when the OCR is 2.50 or higher.

2) The consolidation state for determining shear strengths and the FS values to be used with Figure 12 now presume that

100% pore pressure dissipation under the previous stage of waste loading has occurred before the SISGa takes place (or is calculated).

3) Field evidence of SISGa should exist before it is used in constructing a slope. This may be obtained from a combination of: 1) consolidation tests, 2) DSS tests, 3) inclinometer and piezometer responses during later stages compared to earlier stages, or 4) other means to be determined. (Option 3 is often used because the data already exist from stability monitoring. The data merely have to be reinterpreted to see if the responses reflect the type of changes in strains and pore pressures depicted by Figures 10 and 11.)

4) Rather than using only DSS undrained strength ratios, values from triaxial extension (TE) are used under the toe of berm slopes, direct simple shear (DSS) under the berm and lower waste slopes, and triaxial compression (TC) under the upper waste slopes. Before applying SISGa, i.e. S_1, these values are 0.13, 0.19 and 0.29 for the TE, DSS, and TC ratios, respectively.

5) A limit of 1500 psf is used for the shear strength of the clay crust at the toe of the berm, regardless that vane shear tests may show strengths that are 2 to 4 times higher. This change was made to account for fissures in the clays and strain compatibility between the TC, DSS, and TE test results.

6) The author agrees with Ladd (2008) that reliability based design of the slopes at Crossroads would be more appropriate than limit equilibrium based designs, but the regulations would need to be changed to move away from FS values. WMDSM would likely benefit from this change because so much data exists to support the slope designs at Crossroads. Nonetheless, using current reliability methods, FS values would still have to be computed before the reliability analysis is performed.

2.5 *Geotechnical engineering time and/or cost saving ideas introduced at Crossroads landfill*

At least two ideas have been introduced at the Crossroads landfill to reduce the costs of geotechnical engineering for routine work. These comprise the use of normalized settlement ratios for predicting landfill floor settlement and the use of automated equipment for collection of movement and pore pressure data.

2.5.1 *Normalized settlement ratios*

Over three hundred consolidation tests have been run on clays from the Crossroads site. From this testing, it would be possible to perform settlement calculations using values of compression indices and preconsolidation pressures for discrete areas. This would be an onerous task and it would produce settlement estimates having a wide range of variability. For example, we have seen the Compression and Recompression Ratios vary by a factor of about three across the same landfill unit. The preconsolidation pressures of the virgin clays also vary by a factor of about three.

There have also been over 50 vibrating wire settlement plates and 5 extensometers installed in the clay below the waste and berms at Crossroads. The choice has been made to subdivide the Crossroads site into zones with similar stress history and then for each subdivision, compute a normalized settlement ratio from data already collected at the site. The normalization was by thickness of clay and the depth of the waste over of each settlement plate.

Once the normalized ratios were calculated for each settlement monitoring device, an average value for each zone was computed. Finally, judgment was used regarding the preconsolidation pressure to estimate how the

normalized ratios may change. Table 1 summarizes the current settlement ratios.

Table 1 - Settlement Ratios for Estimating Floor Settlement			
Zone	Area Description	Current Settlement	Future Settlement
A	Phase 8 and 9 undisturbed by slide, Phase 1-6, 7, 9, and asbestos	0.0005 ft/ft^2	0.0006 ft/ft^2
B	Phase 8 and 9 disturbed by slide and reconsolidated with fill at El 280	0.0009 ft/ft^2	0.0008 ft/ft^2
C	Phase 8 disturbed by slide and reconsolidated with MSW at El 280	0.0003 ft/ft^2	0.0004 ft/ft^2

Table 1 – Floor settlement in feet is computed by multiplying the depth of clay below the floor x the depth of waste above the floor x the settlement ratio from Table 1.

Inherent in the use of the Table 1 values is the idea that the values will be updated as new data are collected from areas that have experienced relatively constant waste height for a period long enough for excess pore pressures to have dissipated. Moreover, most of the waste placed in the zones has a density of around 75 pcf.

2.5.2 *Automated geotechnical data collection systems*

It is not often that a person can obtain more of what they need that is of higher quality and at a lower price. This appears to be what happened when WMDSM started buying automated geotechnical data collection systems. Currently WMDSM has 6 systems, where each comprises a data logger connected to both a stationary inclinometer and vibrating wire piezometers. The data loggers are programed to read the instruments 3 times per day.

Once a day, the geotechnical engineer calls up data loggers via the Internet through cell modems, and then in about 15 minutes, he downloads, plots and interprets the movement and pore pressure data from all 6 systems. If a potential issue is noted at one of the monitored sites, the engineer can call back the site, set the data logger to read at 10-minute intervals and then download, plot and interpret the new data. Once a week, inclinometer data and piezometer plots are uploaded to a SharePoint site, where interested parties are able to plot inclinometer data or review pore pressure plots.

WMDSM initially purchased the automated equipment on the basis of labor cost savings. A break-even point of a total of about 70 manual readings of an SI and a set of piezometers was derived, based on the cost to buy equipment, and gather, send, plot, interpret and report either manually read or automatically read instruments. Below 70 readings it makes sense to manually read the instruments and above 70 it makes more sense to have the automated equipment.

There are 26 critical cross sections at the WMDSM site. The automated data collection systems are used where the “action is”. The automated equipment is moved around the site from time to time. About 24 man-hours are required to remove, re-install, and re-program an automated system when moving it from one location to another.

Labor costs were first used to justify the purchase of the automated systems. The experience has been, however, that the higher resolution, i.e. thrice-daily readings (instead of nominal weekly readings), and precision of the fixed inclinometers are equally important benefits. Those benefits indicate the manner in which the ground at the toe of a slope responds to the addition of a minor amount of new weight upslope from the toe. The higher resolution and precision provide the opportunity to shorten or eliminate consolidation periods when it can readily be seen that movement rates have decelerated or pore pressures have quickly dissipated after loading.

Figure 13 shows the results of fixed inclinometer data recently collected during waste placement on a critical cross section.

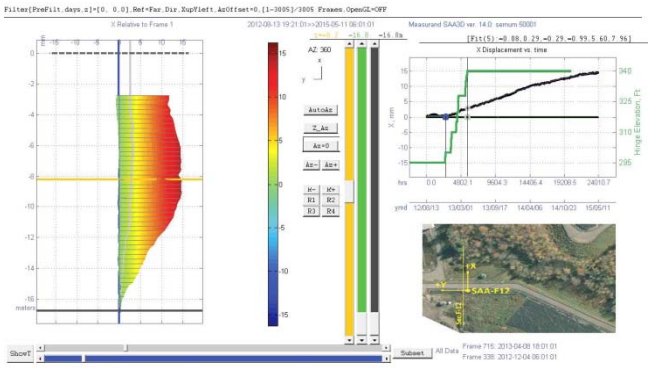

Figure 13 – Graphical results for inclinometer SAA-F12 showing displacement vs. depth on left plot and displacement and waste elevations vs. time on right plot.

Of particular interest in Figure 13 are the deflection vs. time and waste elevation plots at the upper right of the figure.

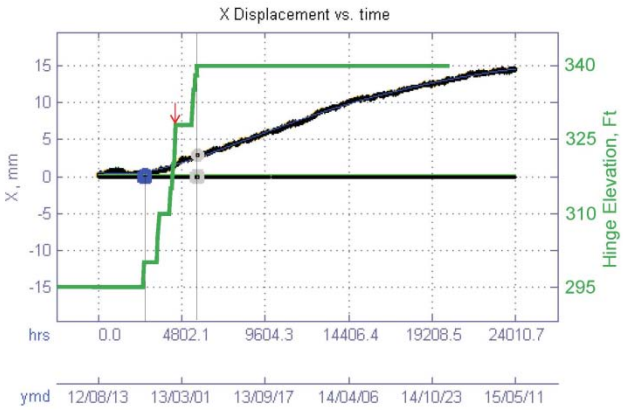

Figure 14 – Blowup of displacement and hinge elevation vs. time plot from Figure 13

Figure 14 provides a blow-up of the portion of Figure 13 showing the displacement and waste elevation plots. Note on Figure 14 that starting about 5 days before 2013-03-01, precisely 25 February 2013, the waste hinge was raised from Elevation 310 ft to 327 ft.

In a relatively short time span, the waste hinge had been raised, first to El 300, then after waiting a few days it was raised to El 310. Each time the hinge was raised, the displacement rate remained fairly constant. Then when the hinge was raised to El 327, the rate increased. (See Figure 15) The high resolution of the plot (i.e. the high frequency of the readings) permitted us to see the rates soon after the waste was placed.

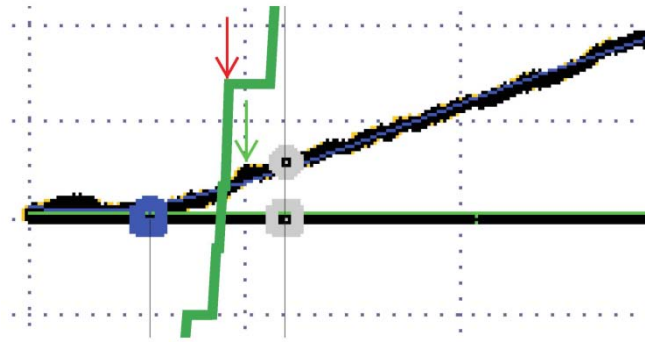

Figure 15 – Further blow up of the displacement and hinge elevation vs. time plot from Figure 13 showing the discrete data points and the displacement rate leveling off on about 1 Mar 2013 (green arrow).

The question was, did this increase in the displacement rate indicate that a longer waiting period should be used? Figure 15 helped to readily answer that question.

Along with the leveling off of the displacement rate, separate pore pressure measurements confirmed that the strains were not an issue and very little excess head was developing during the loading period.

Of course it was not necessary for WMDSM to have the automated data collection system to obtain the high resolution of displacement vs. time plot provided by Figures 13, 14 and 15. It just made it more economical from both a time and expense standpoint. The typical day to day fluctuations in the displacement data collected with the fixed inclinometers is about 0.25 to 0.50mm This can generally be seen by the small bumps that occur along the displacement vs. time plots in Figures 14 and 15.

The typical deflection fluctuations from reading to reading using a standard mobile sensor is about 1.25mm. It is estimated it would take at least 10 manual readings per week to achieve the same level of precision and resolution that is obtained when the fixed inclinometers are read daily. This would cause WMDSM to reach the break-even

point between the use of the automated data collection and fixed inclinometers, and the manually read devices within about 2 months of waste placement. As Figure 14 illustrates, waste placement requiring careful monitoring took about 4 months at this particular location.

One final note on the displacement vs. time data plotted on Figure 14. The amount of displacement taking place during the actual waste placement (loading) period will amount to only about 20% of the total displacement that will occur during this loading cycle. Based on the pore pressures that have been built up and dissipated during this cycle of loading, most of the displacement at this location has been through drained strain.

3 CONCLUSIONS

What could have been just a small slough into a trench excavation mushroomed into a 1 million cubic yard landside of a landfill built on the Presumpscot Formation because of extensive removal of the clay along the trench. Even the removal of most of the stiff Presumpscot Formation along the trench may not have caused most of the landfill to fail had the waste pile had a larger footprint so that it could have been placed more slowly, with periods of time between the stages to allow the Presumpscot Formation to gain strength. Unfortunately, the trench was dug, the stiff clay next to it was widely excavated, the waste pile was rapidly placed, and a retrogressive slide occurred. Fortunately, the slide occurred early in the morning before landfill operators and trash trucks arrived. One operator was on site and did witness the slide and reported that it all happened in a matter of a few minutes.

WMDSM is determined to never let a slope failure happen under their watch. WMDSM became a staunch supporter of using fact based stability analyses to demonstrate the stability of any future waste slope that is built on the site. This led to using numerous types of soil strength testing and sampling equipment, which in turn lead to the use of improved vane shear testing devices, improved Shelby sampling techniques, and state of the practice strength testing at the MIT Geotechnical Laboratory.

WMDSM's commitment to slope stability also led to monitoring movements and pore water pressures adjacent to and under the landfill units at 25 different locations that are deemed the most critical cross sections on the site. Moreover, when excavations have been required adjacent to a landfill toe of slope, then WMDSM has installed and monitored instrumentation next to those excavations, even though the instrumentation may only be needed for a short time. WMDSM recognizes that the time the excavation is open is not relevant because of the possibility of a retrogressive landslide.

Do not get the impression that it has been easy to convince WMDSM that new ideas should be tried in the name of stability, just because a landslide occurred at their site before they purchased it. WMDSM is part of a successful multi-billion dollar company, and feels all the pressures that come from working within such an organization. WMDSM does however; take a long-term approach and is willing to listen to good arguments for demonstrating and maintaining stable slopes in its landfills.

When those arguments can also demonstrate how more waste can safely be placed on the same landfill footprint, as was the case when higher quality sampling and testing procedures were requested, WMDSM listened carefully. The same occurred when shear induced strength gain (SISGa) was introduced at Crossroads, WMDSM listened carefully. Because SISGa has its drawbacks in the form of lower strains and pore pressures to failure, which require greater precision in stability monitoring to assure continuing stability, it was convenient that WMDSM listened to our presentations on the use of automated data collection systems. Of course it did not hurt that such systems could save WMDSM money over the long-term. The same conclusions could be applicable to widening highway embankments.

4 ACKNOWLEGMENTS

The author wishes to thank the staff at Robert G. Gerber, Inc for all the support they provided during the site investigations that followed the 1989 landslide. Those were stressful times. Bob Gerber was always there as a sounding board and the project engineers, Mike Moreau, Karen Garneau, and Ollie Muff spent endless hours in the field, the laboratory, and the office gathering and synthesizing data.

The author also wishes to thank the many engineers from Geosyntec who he has worked with at Crossroads since 2000. They have demonstrated a high degree of professionalism, are not afraid of new ideas, and have always been good team players.

Thanks go to the engineers at WMDSM, Sherwood McKenney, Paul Burns, and Steve Poggi, too. They have lobbied their management to provide financial support for ground breaking the work that was done. Even more, they deserve credit for taking the time to understand complex lectures on soil mechanics from the author.

David Burns at the Maine DEP deserves a huge amount of credit for understanding the importance of staged construction of landfills on soft clay, for requiring that corroborating field and lab data support any stability evaluation, and for having the wisdom to hire an independent geotechnical engineer to review our work. Steve Cole and then Tony Hersh brought the right approach to the project, which was to listen and be part of the solution, not just find problems.

Several individuals have made unsung contributions to CWS and WMDSM. They include Alan S. Jones, PE, Edward Brylawski, PE, Isabel Schonewald, PE and Lee Danisch, PEngr. Their work and knowledge helped turn a tragedy into a success story.

Finally, the author wishes to thank Jack and Amy Germaine for providing understated elegance from MIT to the work the author has done. Jack has consulted with the author for over 25 years on ways to test the clay and methods to improve sample quality. He also has an attitude of doing a test over – no questions asked – if he thinks he made a mistake in specimen selection or handling. Amy has carried some of Jack's load in the laboratory in more recent years and has the same attitude.

One other person from MIT also deserves credit for much of what the author has accomplished at Crossroads - the late Professor Charles C. (Chuck) Ladd. Chuck's clear, yet very detailed writing about construction on soft clay was the reason for our selection of the SHANSEP method for clay strength determination. After his peer review of our work and up until the last time I saw him, shortly before he died in August 2014, he never failed to ask when I was going to write this paper. I am only sorry he did not live to read it, and of course offer excellent and respectful review comments.

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Attachment A - Landslide Air Photo and Selected Figure Enlargements

Air Photo of August 1989 Landslide at Crossroads Landfill. North-South distance of photo is approximately 1500 feet.

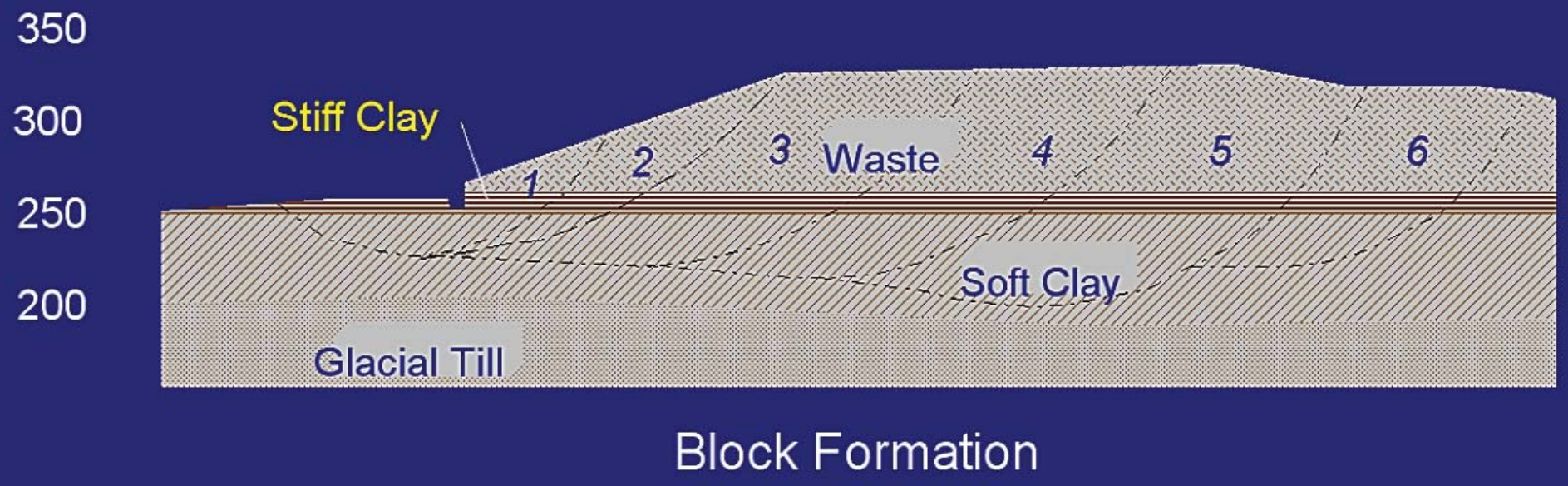

Figure 2 - MSW waste and clay blocks before breaking apart in the 1989 MSW Landslide.

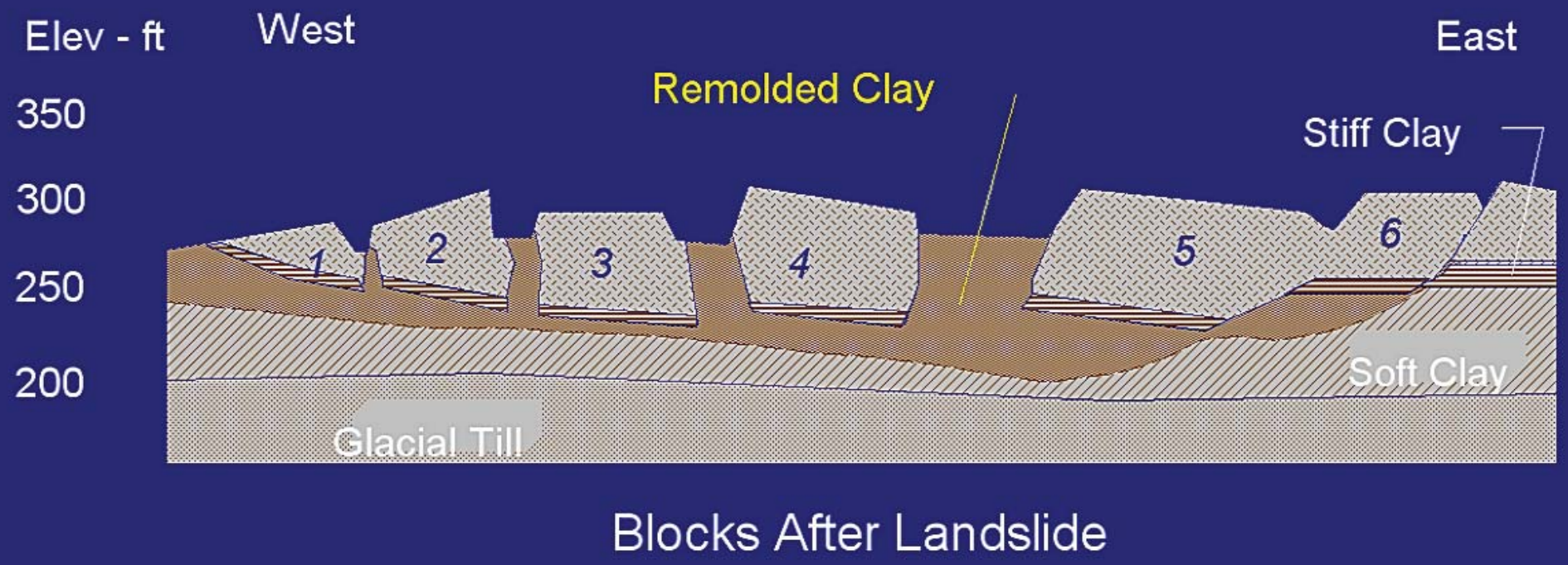

Figure 3 - MSW waste blocks "floating" in remolded and intact Presumpscot clay after 1989 MSW Landslide

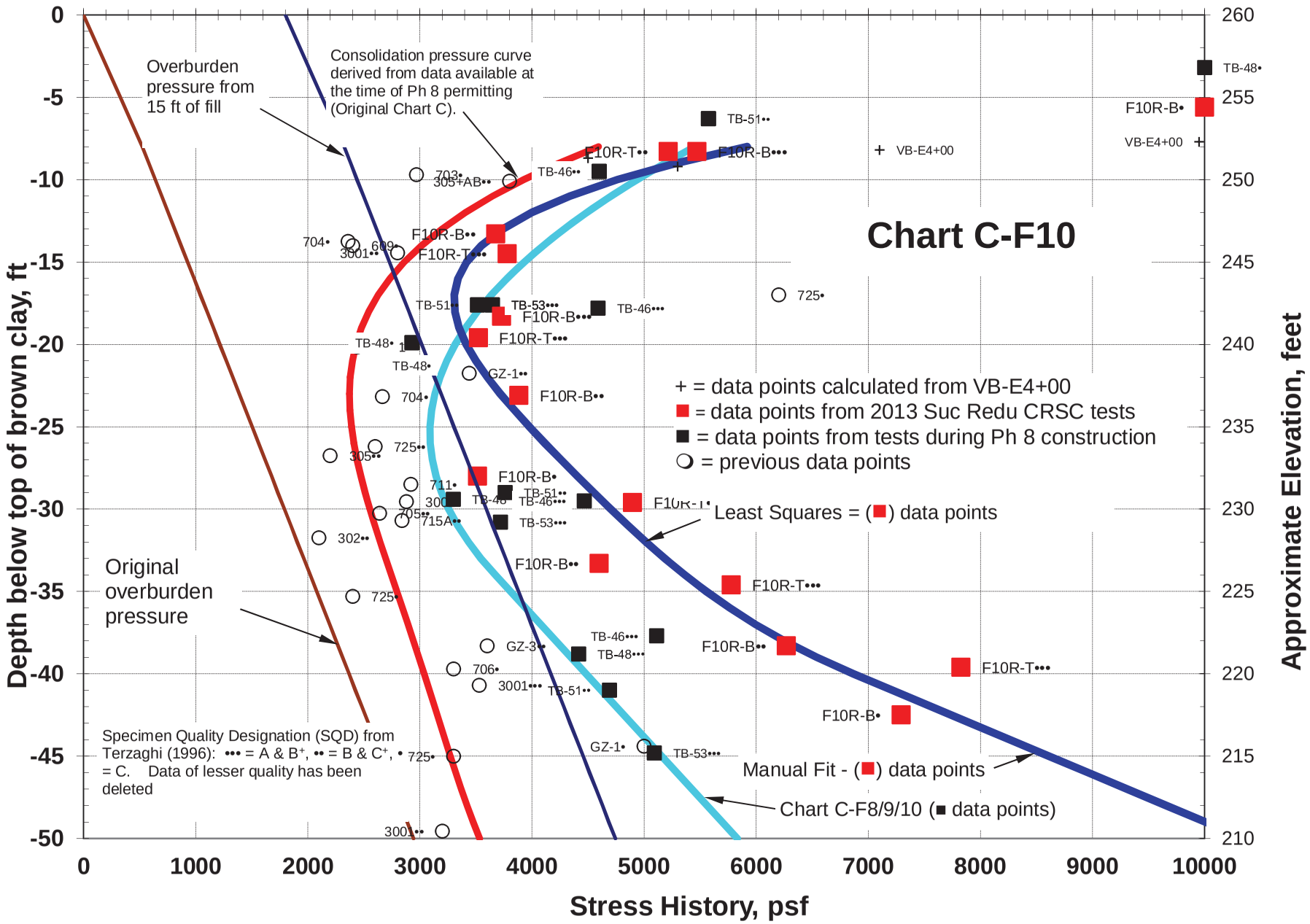

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Figure 7 - Stress history or preconsolidation stresses vs. depth for specimen obtained before 1998 (red curve), between 2000 and 2005 (light blue curve) and in 2013 (dark blue curve).

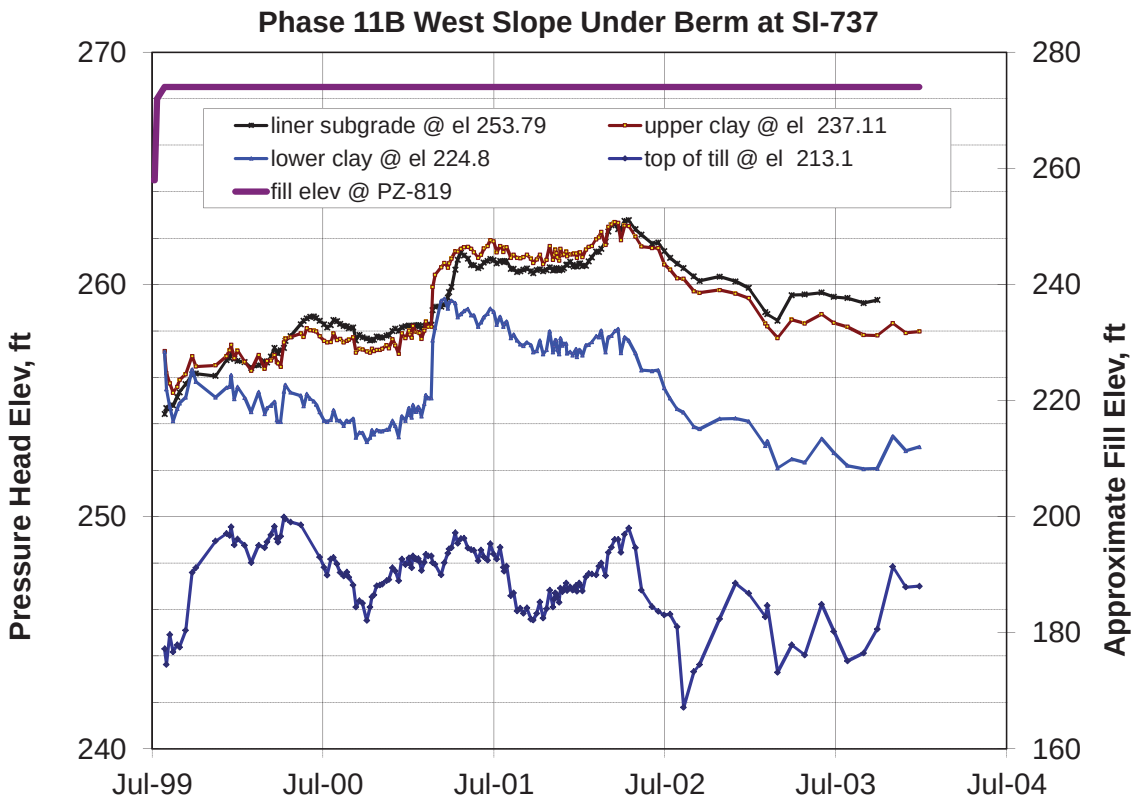

Figure 8 - PZ 819 located under stability berm at the toe of the waste slope

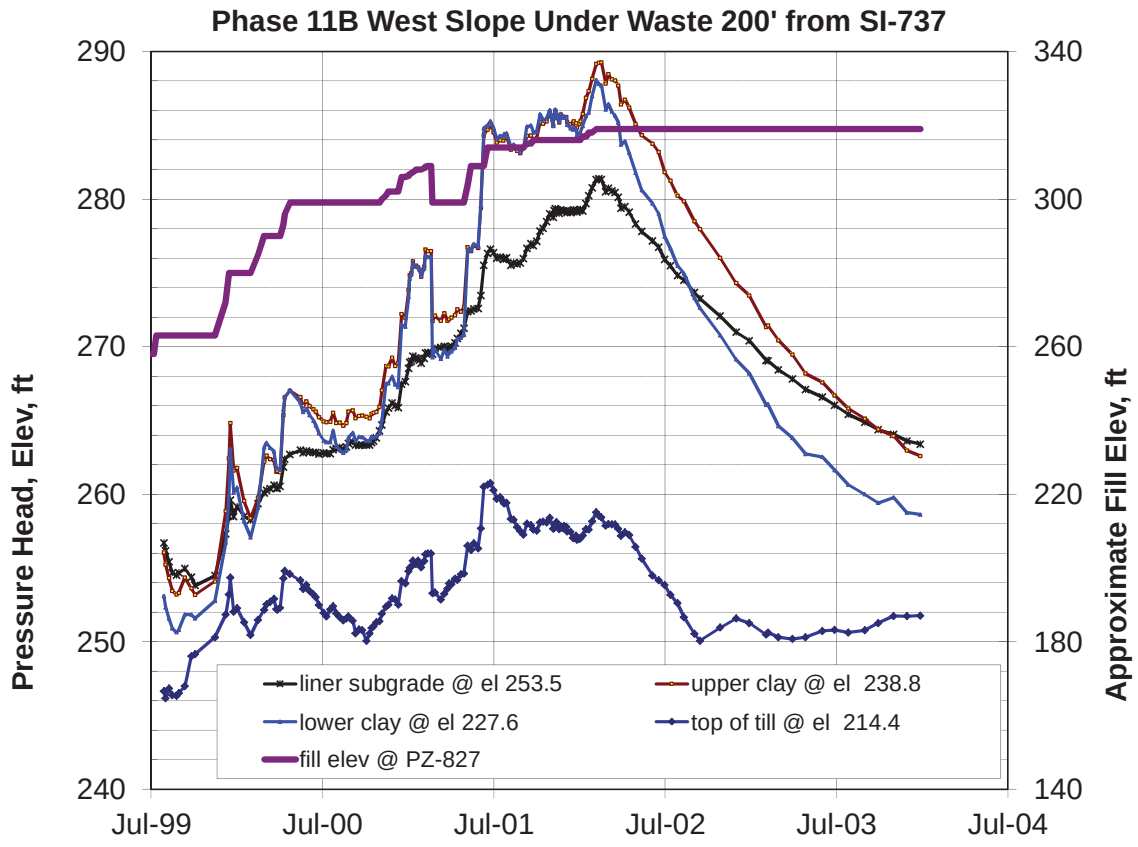

Figure 9 - PZ 827 pore pressures and waste elevations 200 ft away from PZ-819

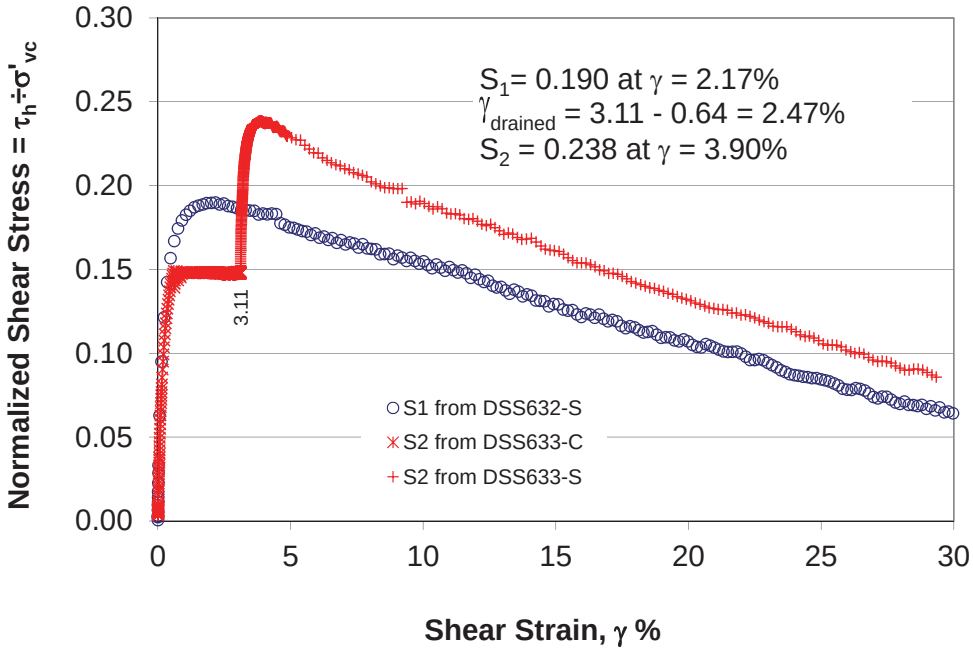

Figure 10 - Stress-strain curves from one and two-stage DSS tests with results of Stage 1 in blue circles and Stage 1-2 in red plus signs. S_1 and S_2 are the peak normalized shear stresses.

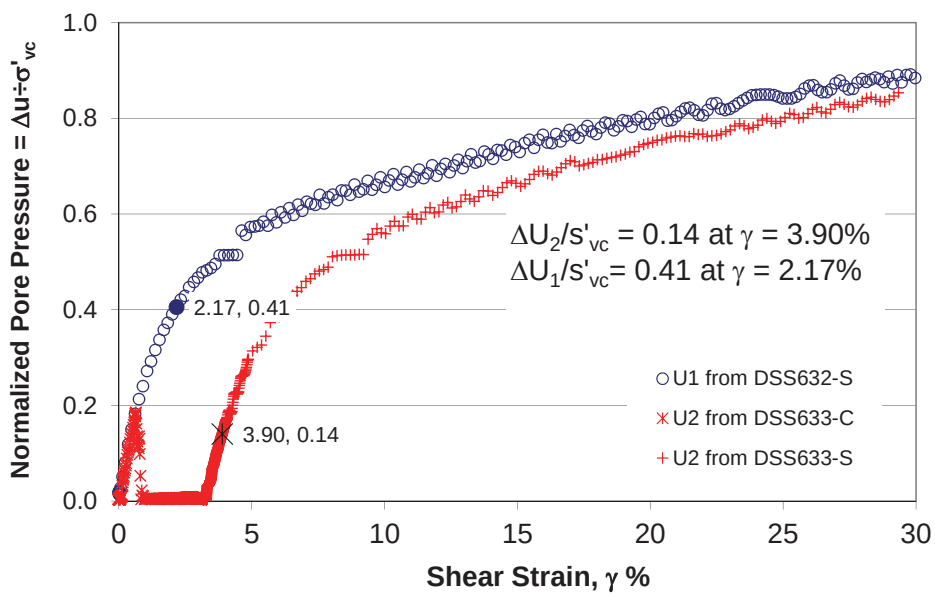

Figure 11 - Pore pressure curves from one and two-stage DSS tests with results of Stage 1 in blue circles and Stage 1-2 in red plus signs.

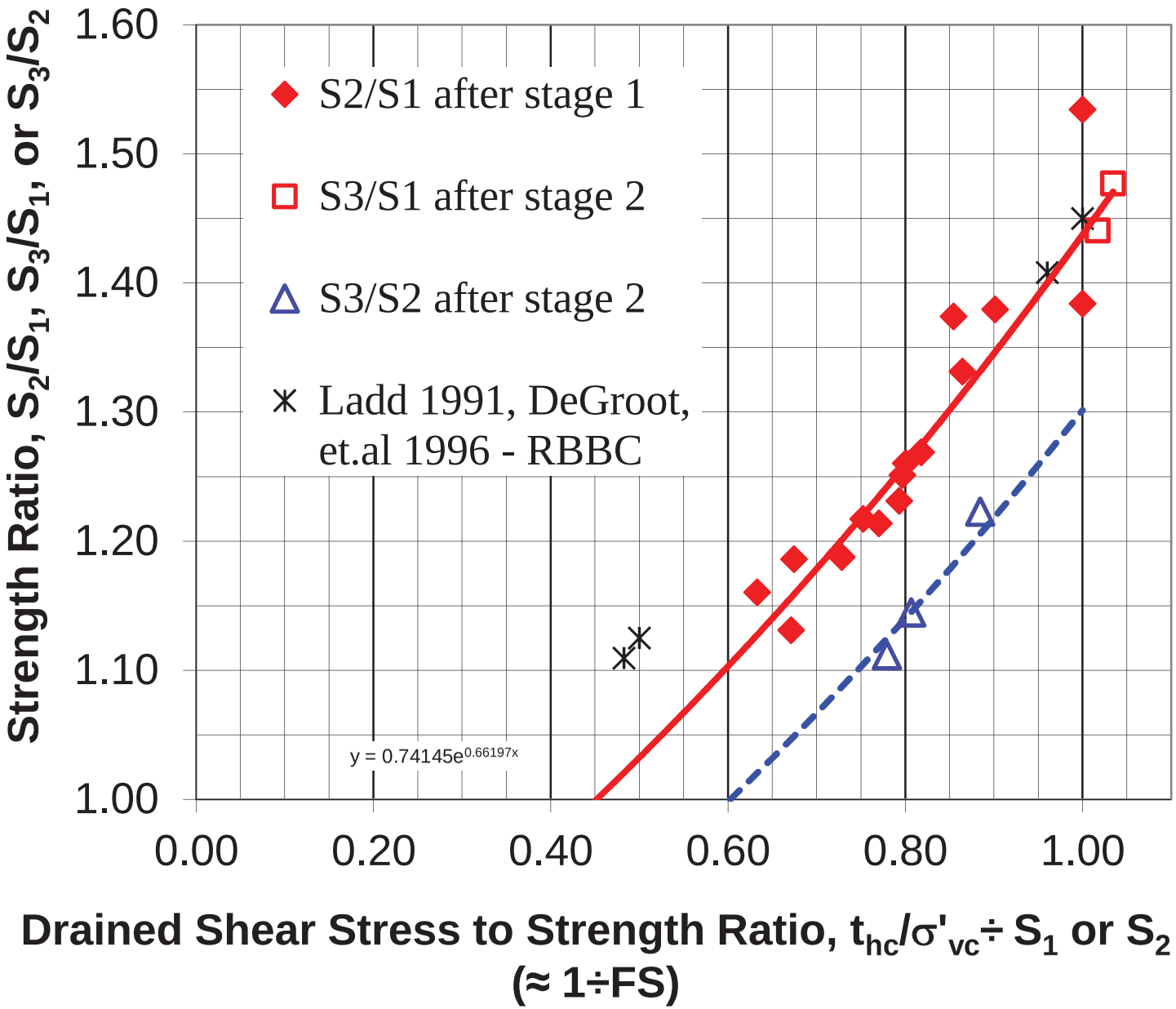

Figure 12 - Recommended plots for determining shear induced strength gain (SISGa. Red diamonds and squares and blue triangles for Presumpscot Formation, Black astericks for residimented Boston Blue clay.