Design of the Crossroads Landfill over Presumpscot Formation

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ABSTRACT

In 1989 large-scale rapid displacement (slope failure) of a 50-acre landfill in Norridgewock, Maine occurred due to shear failure in the underlying Presumpscot Clay foundation strata. Ten years later, the owner undertook a project to remove the waste and construct a new lined landfill in the same area, and to a height that would be 100 ft higher than the original landfill. Detailed geotechnical analyses and the rigorous application of data from geotechnical instrumentation were used to design: (i) allowable excavation configurations to safely relocate the unlined waste; (ii) a system of prefabricated vertical drains (wick drains) to facilitate consolidation of the Presumpscot Clay during subsequent construction and waste filling, and (iii) mechanically stabilized earth (MSE) perimeter berms to increase the loading and associated strength gain in the Presumpscot Clay around the landfill perimeter. Ongoing analyses using the geotechnical instrumentation data and load tracking have been conducted since then to develop and justify modifications to the original waste-slope designs.

This paper was prepared in concert with a companion paper by Richard T. Reynolds titled *25-Years of Loading an Old Presumpscot Clay Landslide: A Case History.* Where applicable, references are made herein to the "Reynolds Companion Paper".

1 INTRODUCTION

1.1 *Site Conditions / Landfill History*

The Crossroads Landfill is a 430-acre site located in Norridgewock, Maine that has served as a solid waste disposal facility since 1976. The current owner, Waste Management Disposal Services of Maine (WMDSM) acquired the facility in October 1990 and has operated the Site since then in accordance with solid waste management permits issued by the Maine Department of Environmental Protection (DEP). Since 1990, WMDSM has constructed several new landfill cells and has closed areas of the Site where no further waste disposal will take place.

The Site geology is characterized by glaciomarine deposits from the Presumpscot Formation which can be divided into an upper olive-brown layer and an underlying gray stratum. The subsurface stratigraphy is shown in Figure 1, and is summarized as follows (from top to bottom).

- The uppermost stratum (herein referred to as the "olive-brown crust") is an upper surficial or near-surface weathered portion of the Presumpscot Formation. This material classifies as clayey silt (ML) with relatively low moisture content and often exhibits a wide range of post-depositional features such as desiccation fissures, disruption by roots, frost fracturing, and expansion fracturing. The thickness of the olive-brown layer ranges from about 5 to 12 feet in areas of the Site where previous construction activities have left the layer intact. The average (geometric

mean) hydraulic conductivity of the olive-brown crust layer at the Site is governed by macro features and ranges from about 3×10^{-5} to 5×10^{-5} cm/s. The strength of the olive-brown crust is addressed later in this paper.

- The gray Presumpscot stratum (herein referred to as the "gray clay") occurs beneath the olive-brown crust and consists of clayey silt with scattered fine sand seams. Similar to the upper olive-brown crust, the "gray clay" actually classifies as clayey silt (ML), but with much higher moisture content, much lower hydraulic conductivity on the order of 5×10^{-8} cm/s, and considerably lower strength. The thickness of the gray clay stratum ranges from fairly thin in some areas to more than 75 feet under much of the Site. The transition from the stiff upper olive-brown crust to the underlying gray clay can be quite distinct, or in areas of fluctuating groundwater levels can be a more gradual transition marked by a mottled brown/gray appearance. The strength of the gray clay is addressed in more detail later in this paper.

- Glacial till underlies the Presumpscot Formation and ranges in thickness from 5 to 35 feet at the Site. The till contains a significant range of stratified sand, gravel, and cobbles, and with an average hydraulic conductivity of approximately 2×10^{-3} cm/s serves as an aquifer for nearby wells. The Glacial Till deposit exhibits very high Standard Penetration Test (SPT) blow counts indicating dense to very dense in-place consistency and strengths represented by an angle of internal friction ranging from 38° to 45°.

- Bedrock at the Site consists of meta-sedimentary rocks of the Sangerville Formation and intrusive igneous rocks. The bedrock dips to the southeast and is fractured in the upper 50 feet. The primary hydraulic conductivity of the bedrock is negligible; however, as a result of fracturing the upper portion exhibits a secondary hydraulic conductivity of approximately 2×10^{-4} cm/s.

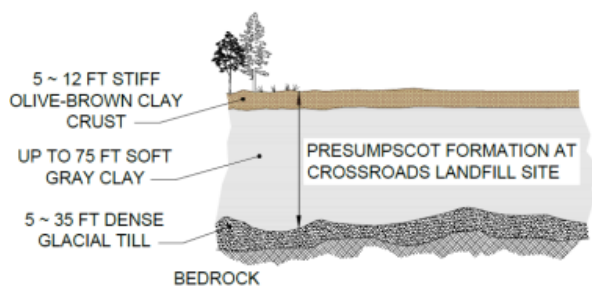


Figure 1. Site stratigraphy in the area of the landfill.

Groundwater at the Site is characterized in three hydrostratigraphic units: the phreatic surface; the potentiometric conditions in the glacial till; and the potentiometric conditions in the bedrock. The phreatic surface at the Site exists at depths ranging from zero to 15 ft depending on seasonal fluctuations. The glacial till is a confined aquifer in most areas of the Site due to the overlying gray clay, and because of the local topography and recharge conditions, groundwater in the till exhibits upward gradients during certain times of the year.

1.2 *1989 Stability Failure*

In 1989, prior to WMDSM acquiring the Site, a massive landslide occurred in the unlined MSW Landfill unit due to removal of buttressing soil and unloading at the toe of a waste slope compounded by rapid loading of the adjacent waste slope and heavy rains that had persisted for several days prior. This resulted in the waste footprint increasing from about 8 acres to nearly 15 acres in less than two minutes. Luckily no one was hurt, since the failure took place at about 06:00 AM shortly before waste disposal trucks or operations personnel were on the landfill.

The slide was a classic retrogressive failure [Richardson & Reynolds, 1990]. A small slice of waste near the toe moved first, which by itself wouldn't have been catastrophic. But this reduction in the toe buttress triggered movement of a bigger adjacent slice of the landfill. This caused movement of yet another slice, etc. resulting in a massive landslide. The retrogressive failure mechanism can be discerned from the chunky appearance of the MSW pile after the failure shown in Figures 2a and 2b.

Figure 2a. Aerial view of 1989 landslide caused by retrogressive failure of the Presumpscot Clay foundation under the landfill (waste moved from left to right in photographs).

Figure 2b. Side view clearly showing the retrogressive failure mechanism.

1.3 *Post-Slide Landfill Development*

In the years following the slide, the surface of the failed MSW pile was graded smooth and a composite clay/geomembrane final cover system was installed in 1993. The Crossroads facility was reopened to accept waste and additional disposal units (Phases 1-6, Phase 7, and Phase 9 – shown in Figure 3a) were constructed and filled with new waste in much of the surrounding area adjacent to the MSW Landfill.

Nearly a decade later, WMDSM embarked on a mission to remove the unlined waste in the previously failed MSW Landfill and create one large contiguous lined area (named "Phase 8") by piggybacking onto the surrounding lined disposal units. As shown in Figures 3a, 3b, and 3c, the Phase 8 landfill would occupy the area from which the unlined waste in the MSW Landfill would be removed, and would eventually include filling over portions of the surrounding disposal units such that the individual units are combined into one large fully-lined disposal area.

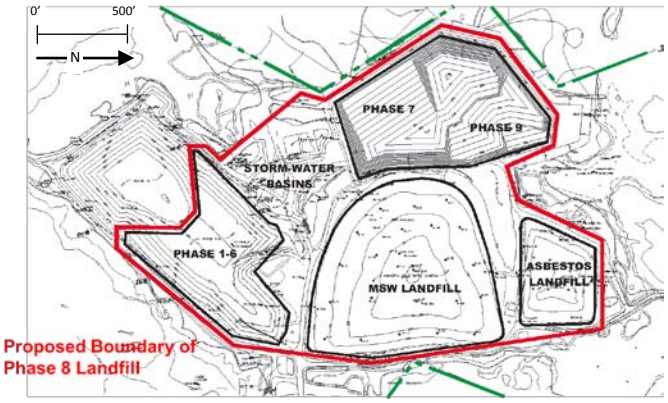


Figure 3a. Plan view of MSW Landfill and surrounding waste disposal units constructed and filled after 1989 slide.

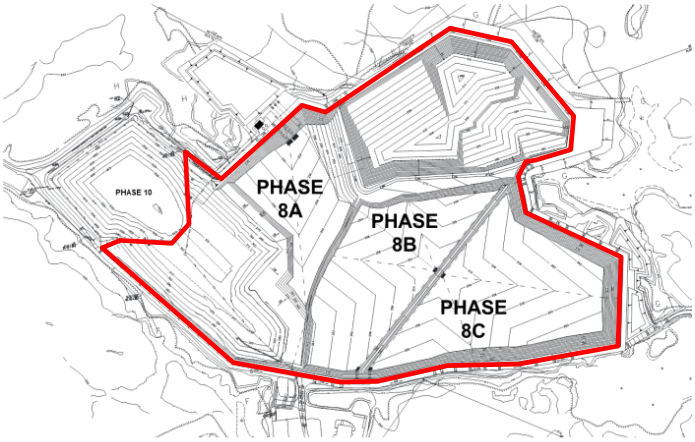


Figure 3b. Liner grades of proposed Phase 8 landfill unit.

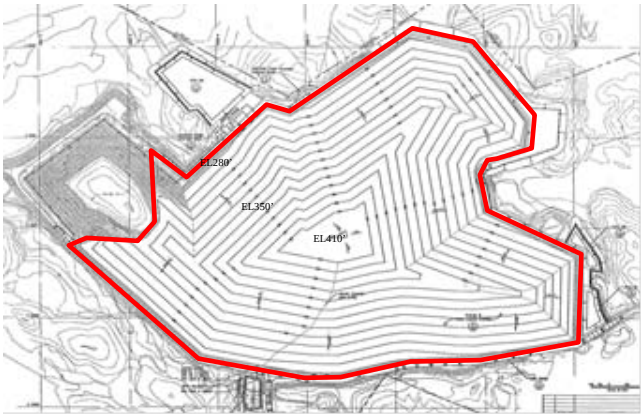


Figure 3c. Proposed Phase 8 waste grades.

2 FEATURES OF PHASE 8 DESIGN

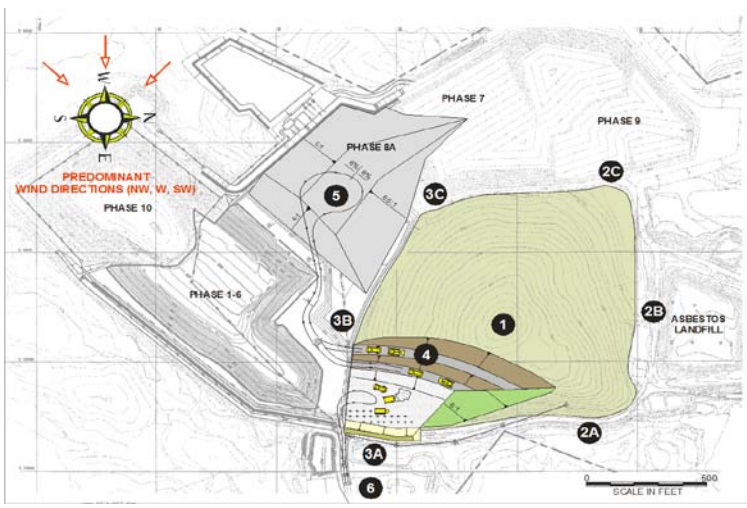
2.1 *Overview of Design Challenges*

In addition to the typical challenges of landfill design and construction, there were several unique challenges associated with designing Phase 8 to be positioned over such a thick soft deposit of the Presumpscot Clay (and to overcome the stigma of redeveloping the site of a previous catastrophic failure). Specific geotechnical engineering analyses of the Presumpscot Clay for Phase 8 were required for: (i) incremental construction of lined cells sequenced with excavation and relocation of the waste from the failed MSW pile into the newly lined areas; (ii) use of prefabricated vertical drains (PVDs – often call "wick drains") to accelerate consolidation and strength gain in the underlying Presumpscot Clay; and (iii) mechanically stabilized earth (MSE) berms around much of the Phase 8 perimeter to increase disposal capacity and enhance stability. These challenges/features are described in more detail below; details of the geotechnical analyses for each are presented in Section 3 of this paper.

2.2 *Incremental Construction Sequencing*

To construct the lined landfill cells shown in Figure 3b, the Phase 8A cell had to be constructed first, followed by excavation/ relocation of the waste from the southern half of the MSW pile into Phase 8A. This allowed construction of Phase 8B, subsequent excavation/relocation of the northern half of the MSW pile into Phase 8B, and then construction of Phase 8C.

The incremental construction/relocation activities had to be accomplished concurrent with ongoing disposal of new gate-receipt waste being received at the landfill. This process of relocating more than 1 million cubic yards of waste (illustrated in Figures 4a and 4b) took place over a two-year period, often being performed around-the-clock during winter months to maximize the beneficial effects of frozen ground for equipment trafficability and to take advantage of snow in suppressing odors.



Figures 4a (above) and 4b (below). Illustration of waste relocation from unlined MSW Landfill into lined cells.

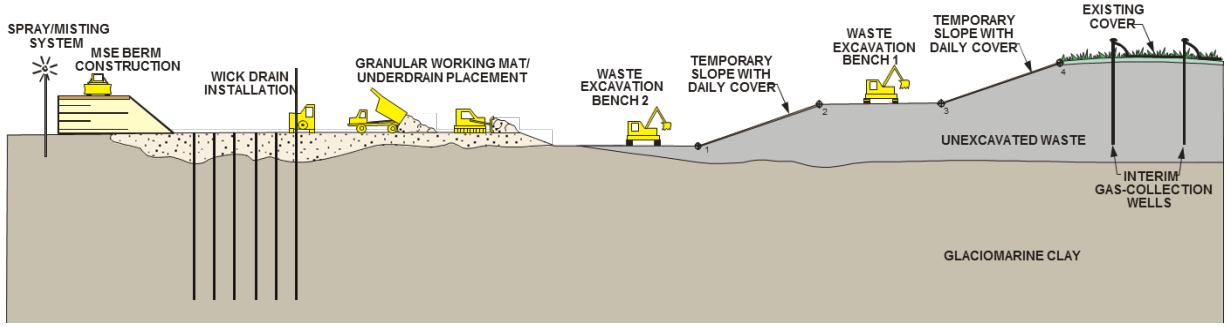


Figure 5a. Rig used to install prefabricated vertical (wick) drains.

Figure 5b. Tops of prefabricated vertical drains (PVDs) being cut off at the elevation of drainage blanket (underdrain) layer during installation

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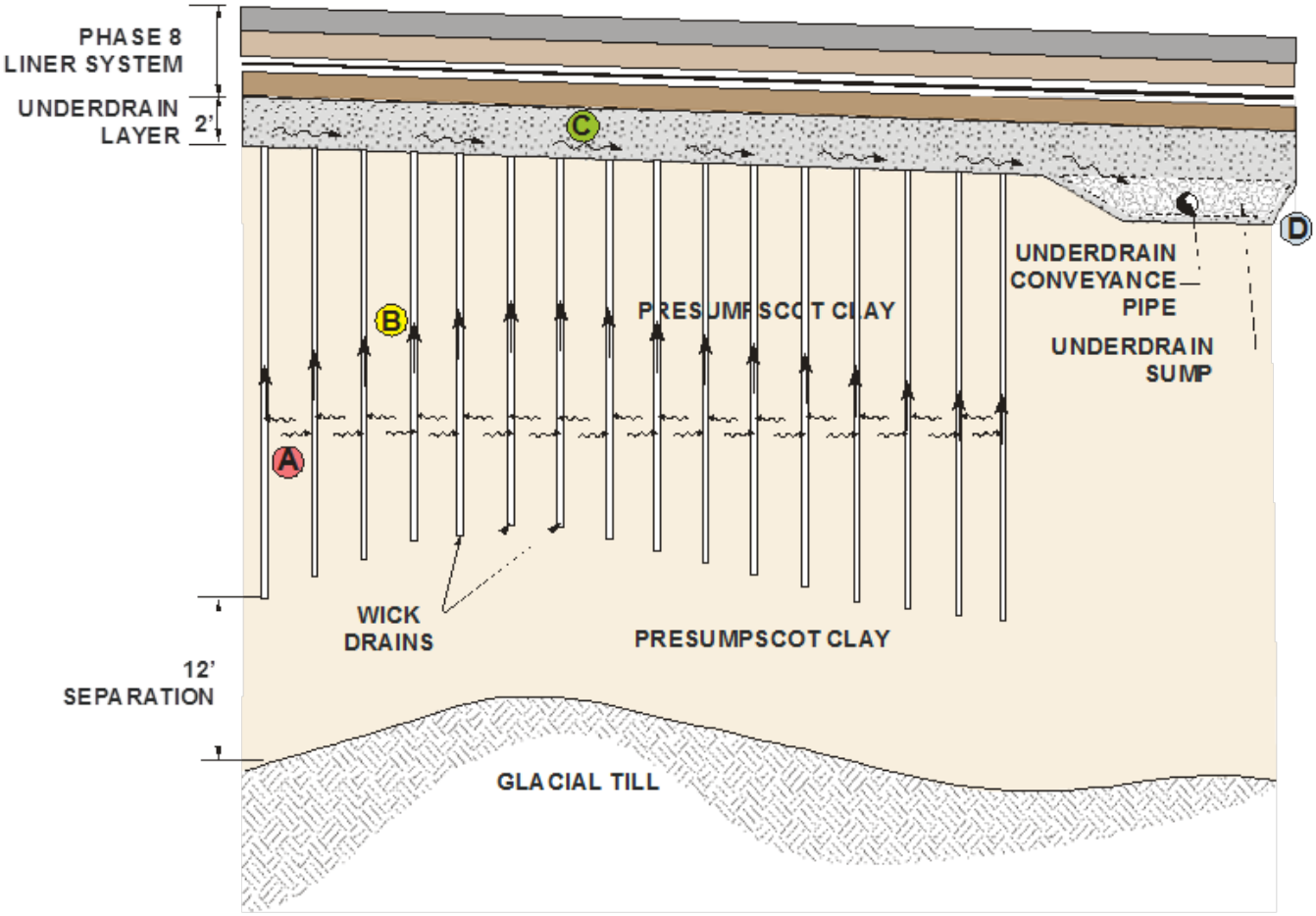

Figure 6. Profile illustration of prefabricated vertical drains (wick drains) and blanket underdrain beneath landfill liner system

2.3 *Pre-Fabricated Vertical (Wick) Drains*

The proposed Phase 8 design required confident engineering reliance on the increase in strength that would occur in the Presumpscot Clay as it consolidates. To accelerate this process, more than 75,000 PVDs were installed prior to constructing the liners in the new Phase 8A, 8B, and 8C cells (see Figures 5a and 5b).

The PVDs were installed at a horizontal spacing of 5~10 ft on center, and to depths ranging from 20 ft to 75 ft such that the bottoms of the PVDs extended no closer than 12 ft above the underlying Glacial Till. The 12-ft vertical separation zone was maintained as a hydraulic barrier over the till, thereby not allowing upward migration of water from the till aquifer into the PVDs nor the downward migration of potentially impacted groundwater from the area of the previously unlined MSW pile into the till aquifer (see Figure 6). The PVDs were also installed diagonally outward near the edge of landfill prior to construction of the perimeter berms to facilitate rapid strength gain in the Presumpscot Clay foundation over a larger lateral area.

A drainage blanket (underdrain) into which the top of the PVDs terminate was installed prior to and directly beneath the Phase 8A, 8B, and 8C liners. The underdrain consisted of a minimum 2-ft thick layer of clean sand ($k \geq 5\times10^{-3}$ cm/s). A network of perforated outlet pipes in the underdrain directed consolidation water from the PVDs to a specific location at the landfill perimeter where the water could be tested and discharged as clean surface water or collected and treated if deemed environmentally necessary.

2.4 *MSE Perimeter Berms*

The lateral boundaries of the Phase 8 area were constrained by existing landfill disposal units, property lines, and wetlands. As such, it was important to maximize the vertical extent to which the waste could be placed within the lateral constraints.

In this pursuit, mechanically stabilized earth (sometimes referred to as reinforced soil) berms were designed around much of the perimeter which, as illustrated in Figure 7, greatly increased the disposal capacity of Phase 8. The average height of the MSE perimeter berms was 20 ft, which provided over 500,000 yd^3 of additional disposal capacity compared to what would have been provided for roughly the same lined footprint using conventional berms.

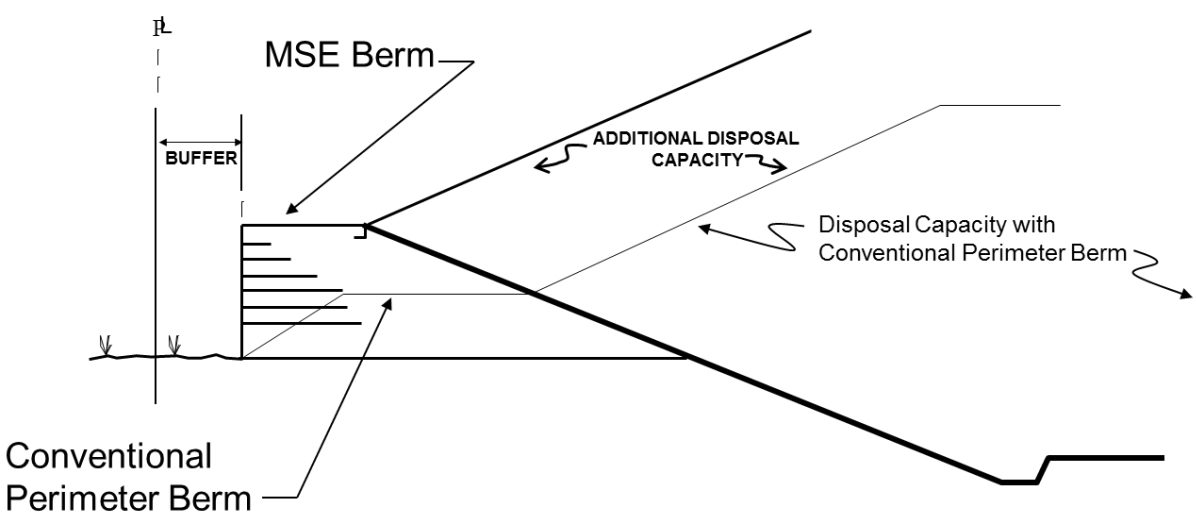

Figure 7. Cross section showing additional disposal capacity provided by Mechanically Stabilized Earth perimeter berm.

As importantly, the MSE berms provided additional loading (and therefore consolidation and increased strength gain of the underlying Presumpscot Clay) near the edge of the landfill where the added clay strength is very important for global stability but is often difficult to achieve

with conventional toe-of-waste and unreinforced berm configurations.

3 GEOTECHNICAL DESIGN ANALYSES

3.1 *Approach to Geotechnical Pre-Design Investigation and Design Analyses*

A noteworthy strategy for designing the Phase 8 landfill was based on a collective agreement by the designer, owner, and regulatory personnel to focus the pre-design investigation on defining the critical stratigraphic geometry of the subsurface conditions such as the thickness of the olive-brown crust, the thickness of the gray clay, and the depth to the glacial till. However, rather than running an excessive number of laboratory strength tests on samples of the Presumpscot Clay, the strengths would be based on a less extensive (but strategically located) number of undisturbed samples and laboratory direct simple shear and consolidation tests, but then geotechnical field instrumentation would be installed and monitored during and after construction to corroborate and refine the design during waste filling. This rationale was, to some degree, in recognition that much pre-design information for the Phase 8 area could be gleaned from the geotechnical information that had been obtained immediately following the 1989 failure and during the design and construction of cells surrounding the MSW pile during the 1990s.

To supplement the existing information, dozens of borings and direct push probes were performed to further define the thickness of the upper olive-brown crust and the gray clay, and the depth to the underlying glacial till. Additional soil test borings were also advanced around select portions of the perimeter to yield more precise information about the thickness and continuity of the upper olive-brown crust, and in a few locations through the MSW pile into the underlying Presumpscot formation to investigate strength gain that had presumably taken place as a result of the 1989 shear failure and ensuing decade of consolidation. An average (and relatively uniform) unconfined shear strength of 2000 psf was measured using vane shear testing on the upper olive-brown crust. The strength of the gray clay was calculated using the SHANSEP method [Ladd & Foote, 1974] which included reconsolidation of the Shelby tube samples to the virgin state (i.e. the preconsolidation pressure) then unloading to the desired overconsolidation stress and testing in direct simple shear. The results were used to develop shear strength profiles for the underlying gray clay. A profile showing the subsurface stratigraphy in one area of the Site is shown in Figure 8a and the accompanying soil strength profile is shown in Figure 8b. (See Reynolds Companion Paper for more details of how the shear strength profiles were developed).

3.2 *Geotechnical Analyses for Incremental Cell Construction and Waste Relocation*

As previously mentioned, construction of the lined cells for Phase 8 required incremental (sequential) cell construction and waste relocation from the unlined MSW Landfill. Since the 1989 failure had resulted in waste blocks that had shifted (rotated) down as they moved laterally, the excavation of waste had to be carefully pre-engineered to ensure the surrounding waste slopes would be stable during the excavation and relocation activities. This involved design and analyses of many excavation configurations (see Figure 9) to ensure:

- stability of excavation sideslopes and of the unexcavated portion of the MSW pile;
- stability of existing landfill units surrounding the excavation area; and
- heave or excessive clay-strength reduction would not occur near the bottom of the excavation due to water pressure in the underlying glacial till.

The stability analyses were performed using various limit-equilibrium stability software programs available at that time (STABL®, UTEXAS®, and SLIDE®). Strengths for the waste were estimated from literature values, and strengths for the Presumpscot Clay were calculated using the SHANSEP method as previously described. The factors of safety using the modified Janbu method analyses and Spencer's method for circular and block failure surfaces were evaluated, and allowable excavation depths, slope configurations, and incremental backfill requirements were established prior to starting the waste relocation activities.

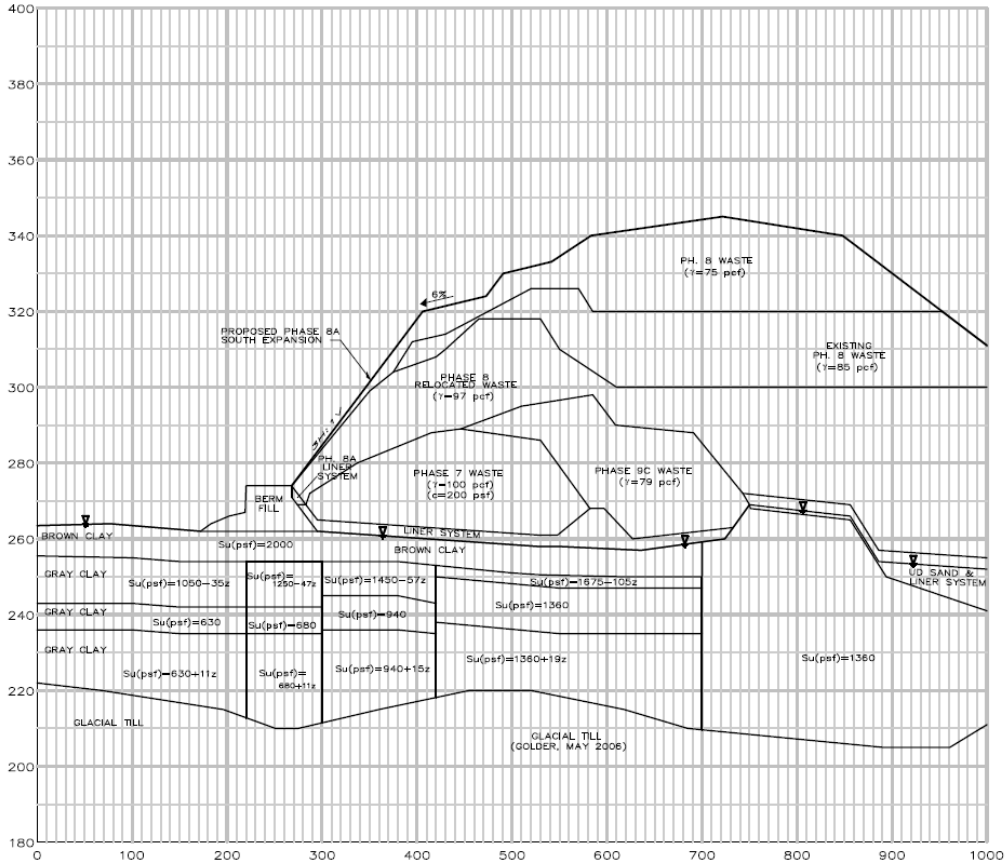

Figure 8a. Subsurface stratigraphy for one stability section of Phase 8.

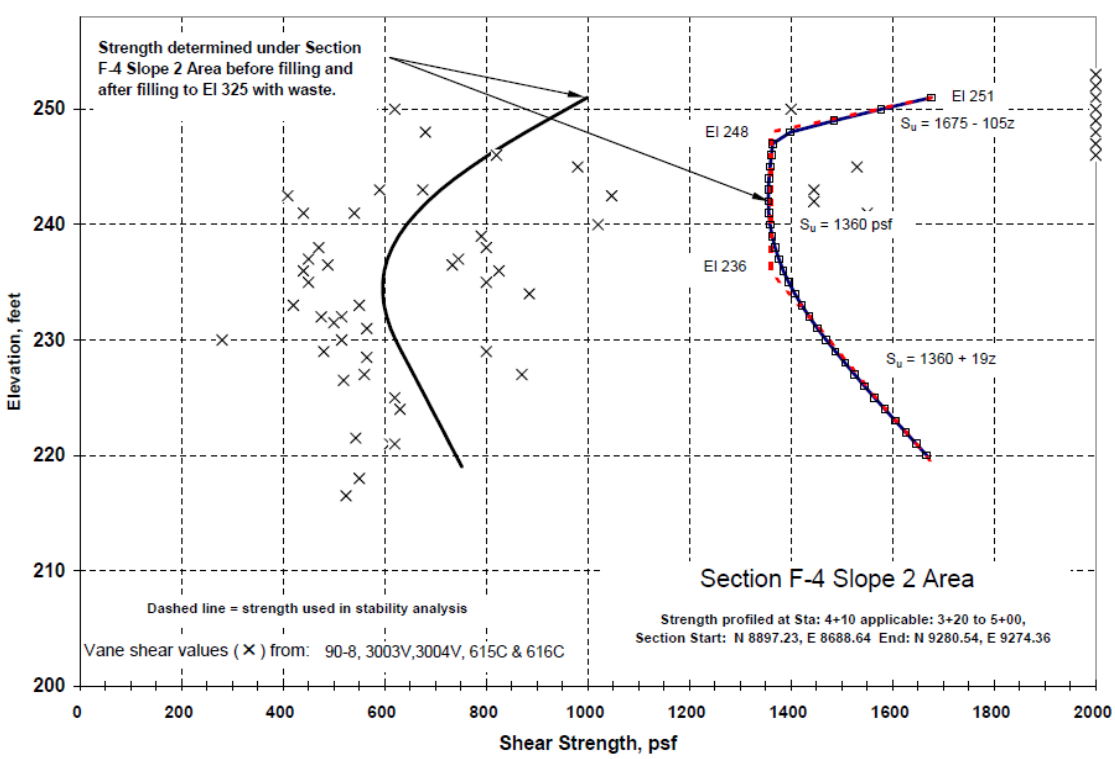

Figure 8b. Strength of Presumpscot Clay as a function of depth and stress history at section shown in Figure 8a.

In addition to the attention given to the strength of the Presumpscot Clay under the landfill, an equal amount of attention was paid to reducing uncertainty in the loading conditions that would occur from the various waste configurations during the excavation / relocation activities. This required increasing the confidence in the density of the existing waste prior to and after being relocated. Accordingly, nearly 200 test pits were excavated to measure the density of the relocated waste in the newly lined cells during relocation. The test pits were carefully excavated using a backhoe to depths ranging from 2 to 3 ft with lateral dimensions of 10 to 12 ft wide and 11 to 15 ft long. The waste from each test pit was loaded into a truck that was weighed at the gate scales (before and after) to provide the weight of the waste from the test pit, thereby allowing the waste density to be calculated using the dimensions from each test pit. The test pits revealed an average density of 94 lb/ft^3 from areas of the MSW pile where the waste had intermixed with clay as a result of the 1989 slope failure, and 81 lb/ft^3 in areas where the waste had not mixed with the underlying clay. The results of the test-pit densities were used to back-check the original analyses, and adjustments were made to the allowable excavation and relocation configurations during construction to increase construction efficiency while maintaining acceptable factors of safety against failure of the temporary excavation.

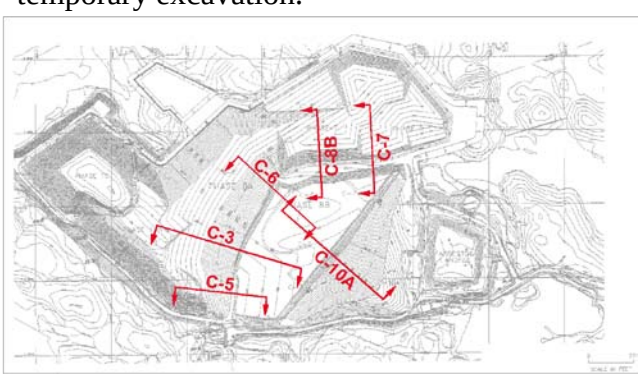

Figure 9. Locations of critical stability sections used to establish and evaluate waste excavation/relocation configurations that would be allowed during construction

3.3 *Geotechnical Analyses for Prefabricated Vertical (Wick) Drains*

As noted previously, the Presumpscot Clay at the Site is up to 75 ft thick with hydraulic conductivity in the mid-to-low range of 10^{-8} cm/s. Very long periods of time (many years) would be required for the pore water to flow upward to near-surface drainage features or downward to the glacial till via consolidation gradients. Strength gain in the Presumpscot Clay from consolidation had to be achieved in shorter periods of time to minimize the size of the lined areas into which the existing waste from the MSW pile would be relocated and into which new gate-receipt waste could be concurrently deposited. Furthermore, there was concern that the consolidation water from under the MSW pile would be impacted by leachate from the overlying unlined MSW pile and may need to be collected. Two design objectives had to be accomplished: (i) accelerate consolidation by shortening the flow paths of the pore water; and (ii) convey possibly contaminated pore water under the MSW pile away from the glacial till aquifer to a controlled point where it could be tested and collected for treatment or discharged as clean surface water. These objectives were accomplished by installing PVDs under the Phase 8 area prior to liner construction.

The spacing of the PVDs was initially calculated based on the anticipated rate of waste placement and a coefficient of consolidation (C_v) for the gray clay of 40 ft^2/yr. The C_v was calculated using the pore pressures measured in piezometers that had been installed after the 1989 slide and in areas of known loading prior to the Phase 8 design. The 40 ft^2/yr value was conservatively less than measured in the laboratory, and led to an intentionally close PVD spacing of 5 ft on center (OC) for the first cell (Phase 8A). The rate of consolidation was then re-evaluated using settlement plates, extensometers, and piezometers under and around Phase 8A during waste filling and the spacing of the PVDs was subsequently reduced to 8 ft OC for Phase 8B and Phase 8C.

Differential settlement of the base (liner) grades was carefully evaluated to ensure a proper flow direction of leachate in the leachate collection layer of Phases 8A, 8B, and 8C would be maintained at all times. This also required that the areas and timing of waste placement in various areas of the cells be pre-planned and controlled on an on-going basis throughout filling of Phase 8 with waste. (See Reynolds Companion Paper for more details of how

normalized settlement ratios were developed and used to facilitate the calculation and evaluation of differential settlements during waste filling.)

3.4 *Geotechnical Analyses for MSE Berms*

As previously introduced in Section 2.4, the Phase 8 design included MSE berms to maximize the disposal capacity and to provide more loading (and therefore more strength gain in the Presumpscot Clay) near the outer edge of the landfill footprint. The MSE berm design included analyses to ensure internal stability of the reinforced zone thereby optimizing the required quantity and tensile strength of geogrid reinforcing layers. Relative to the underlying Presumpscot Clay, and because the MSE berm was initially considered to be a wall structure, bearing capacity analyses were also performed. The calculations included traditional bearing capacity analyses (Figure 10a) and punching shear analyses (Figure 10b) in an attempt to model the stiff olive-brown crust over the gray clay. The bearing capacity analyses however were problematic because neither the traditional bearing capacity nor the punching shear equations are well suited to handle foundation soil strengths that vary with depth.

There was also considerable (albeit quite valuable) debate amongst Geosyntec's design engineers and the Maine DEP geotechnical reviewers over what the acceptable values for bearing capacity should be. Finally after much discussion, all parties agreed that limit-equilibrium stability analyses using one or more of the numerical computer programs (STABL®, UTEXAS®, and SLIDE®) would be used in lieu of bearing capacity analyses because the programs are much more capable of accounting for complex foundation stratification and variable strengths, and because the Maine regulatory requirements for minimum factors of safety for landfill containment systems could be applied to the MSE berms the same way they were being applied to the other aspects of the landfill stability. The bearing capacity interaction between Geosyntec designers and the Maine DEP geotechnical reviewers proved to be very beneficial since it led to a better understanding of potential failure mechanisms and a more rigorous defining of the search boundaries applied in the limit-equilibrium programs during design.

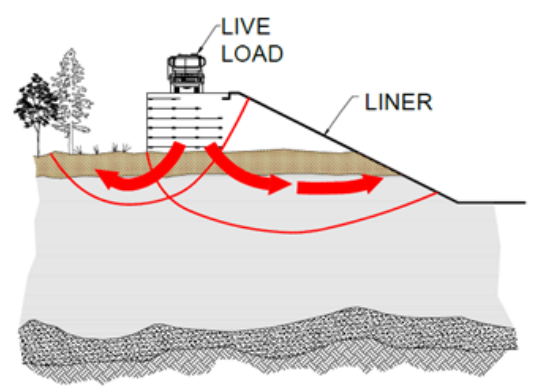

Figure 10a. Inward and outward traditional bearing capacity failure modes analyzed for the MSE berm at designated locations around the Phase 8 perimeter.

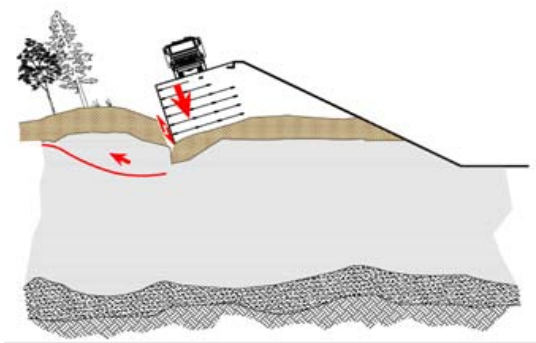

Figure 10b. Punching shear bearing capacity failure mode analyzed for the MSE berm at designated locations around the Phase 8 perimeter.

The design analyses showed that the stability of the MSE berms was a direct function of the thickness of the olive-brown crust, and to a lesser degree, the strength of the crust (which was taken to be S_u = 2000 psf based on the vane shear results). Parametric analyses actually showed that the strength in the uppermost 15 to 20 ft of the underlying gray clay had more effect on the ability of the crust to support the berms than the strength of the crust itself (within the range of plausible strengths for the crust). In fact, the analyses showed that the initial strength of the gray clay would not be sufficient to support the full height of the MSE berms along some portions of the Phase 8 perimeter. The design was therefore developed to allow two-staged berm construction, such that the berms would be initially built to approximately 50~75% of the designed height with construction temporarily suspended until the affected portion of the gray clay could consolidate and gain strength before proceeding with construction of the berms to full

height. The consolidation of the gray clay was hastened by the PVDs under the berms, and was carefully controlled during construction by measuring and tracking the in-place density of the compacted soil in the berms in order to determine when construction could safely proceed up to full height after pausing at the first stage. Given the more than 4,500 ft of MSE berms for Phase 8, this required considerable diligence to balance the competing interests of ensuring stability versus the desire to finish construction as soon as possible.

3.5 *Geotechnical Analyses for Waste Filling*

As previously described, the approach to ensuring stability of the landfill over the Presumpscot Clay focused heavily on the use of in-situ instrumentation to supplement pre-design investigation field and laboratory data. This approach required that the waste slopes be quite flat (5 horizontal to 1 vertical (5H:1V)) compared to conventional landfill designs (typically 3H:1V). Figure 11 shows the waste configuration initially envisioned during design and the locations of several critical sections (F-1 through F-11) at which the geotechnical stability of the landfill was evaluated. These locations were chosen by considering the proposed height and angle of the waste slopes coupled with the strength and thickness of the Presumpscot Clay at various locations under the landfill.

Similar to the stability analyses of the interim conditions, the proposed final configurations at the critical sections were analyzed using various limit-equilibrium stability software, including STABL®, UTEXAS®, and SLIDE®.

Strengths for the waste were estimated from literature values, and strengths for the Presumpscot Clay were calculated using the SHANSEP method. The factors of safety (FS) were evaluated using the modified Janbu and Spencer's methods for circular and block failure surfaces.

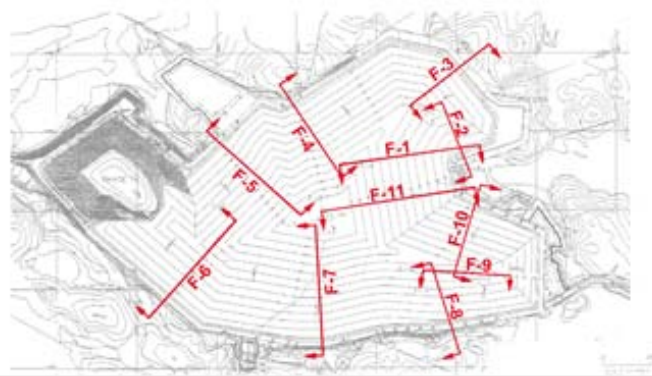

Figure 11. Phase 8 waste-configuration design and locations of critical sections for stability analyses.

Analyses were performed for static and seismic conditions to establish design slopes that would maximize disposal capacity in the landfill while meeting the regulatory required minimum factors of safety for interim conditions (FS≥1.3 static and ≥1.1 seismic) and for post closure (FS≥1.5 static and ≥1.0 seismic) set forth in the Maine Solid Waste Management Rules. The stability analyses have nearly always indicated that the minimum FS values are found using block failure searches where the critical surface passes steeply down through the waste and the gray clay to just above the top of the glacial till, where it then runs roughly parallel to the top of the till layer and then rises to the ground surface outside the landfill perimeter.

A summary of the initial minimum factors of safety for the eleven stability sections is presented in Table 1.

The stability analyses demonstrated that the Phase 8 waste configuration would be stable using conservative strength parameters for the Presumpscot Clay. This led to the conclusion that the projected disposal capacity of 4 million yd^3 would be an economically feasible investment for the owner, and the Crossroads facility would be able to provide more than a decade of disposal capacity for the residents and businesses of central Maine.

Table 1. Summary of initial stability factors of safety.

Section	Minimum Calculated Factor of safety
F-1	1.54
F-2	1.52
F-3	1.66
F-4	1.56
F-5	1.50
F-6	1.57
F-7	1.51
F-8	1.50
F-9	1.59
F-10	1.55
F-11	1.55

The real windfall, however, has been in the continued use of the geotechnical instrumentation to provide reassurance that the Presumpscot Clay has reacted to the various loadings (and even unloading/reloading in the vicinity of the MSW pile) in the manner expected, and to guide the designer and owner in establishing and continuing to update procedures for operating a safe and economically viable landfill facility (see Section 4).

4 POST-CONSTRUCTION MONITORING, ANALYSES, AND MODIFICATIONS

4.1 *Overview*

As introduced in Section 3.1 of this paper, a critical component to the Phase 8 design was the owner's commitment to using geotechnical instrumentation during waste filling to monitor movement and pore-pressures in the Presumpscot Clay. Slope Inclinometers (SIs) and/or Slope Accelerometer Arrays (SAAs) were installed at the toe of the stability sections (see Figure 11 and Figure 12) to monitor movement, and nested vibrating-wire piezometers were installed adjacent to the SIs or SAAs to provide valuable information about pore-pressures at designated elevations in the Presumpscot Clay during construction and subsequent landfilling.

The monitoring procedures were codified in a Stability Monitoring Plan (SMP) that defined allowable rates of displacement in the clay and response actions that would be implemented if movements were detected (see Section 4.2).

A second and equally valuable purpose of the geotechnical instrumentation was to use the data to strategically plan where and when waste should be placed in the landfill such that consolidation and strength gain of the Presumpscot Clay would be optimized for future waste placement, and to modify the original waste configuration such that considerably more disposal capacity could be achieved within the same lined footprint (see Section 4.3).

4.2 *Stability Monitoring Plan*

A stability monitoring plan (SMP) was developed to guide the acquisition and interpretation of the data in monitoring the stability of the landfill. The SMP defines the zones of influence in which the Presumpscot Clay, and therefore the instrumentation in any given area, could be affected by waste placement in that zone. The SMP then prescribes the frequency that data from the instrumentation must be obtained when waste is being placed within each zone of influence illustrated in Figure 12.

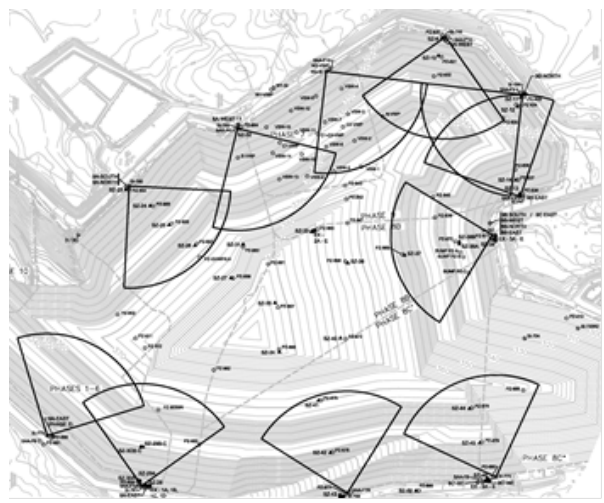

Figure 12. Zones of influence for monitoring stability using instrumentation during waste placement.

If predetermined levels of deformation are exceeded within designated timeframes, then a "yellow alert" status is activated for the Site personnel. This requires additional readings and assessment of pore pressures in the Presumpscot Clay to promptly be performed. Then if additional deformations above designated levels are measured (especially if coupled with continued increases in pore pressures), a "red alert" is triggered which requires that waste

placement within the zone of influence be suspended until the rate of deformation and pore pressures have decreased. The SMP is revisited every couple of years to adjust/refine the frequency of readings, the threshold levels, and required response actions for the various alert levels.

4.3 *Use of Geotechnical Instrumentation for Strategic Planning and Design Modifications*

In addition to monitoring the stability of the landfill, the ongoing instrumentation data has been used to guide the owner in where and when waste should be placed, how high it should be placed in any given area, and how long waste placement in designated areas would need to be suspended to allow the underlying Presumpscot Clay to consolidate and gain strength prior to resuming waste filling in the subject areas. This required the owner to estimate future tonnage of gate-receipt waste that would be disposed of in the landfill, thereby allowing predictions of how long given areas could viably sit prior to needing to be re-opened for more disposal. The process was further complicated by the owner's understandable desire to limit the amount of Phase 8 open at any given time so leachate quantities and air emissions would be minimized.

During and after waste placement in the designated areas of Phase 8, the geotechnical instrumentation data was used to assess whether design modifications could be implemented. Specifically, the instrumentation results were used to refine the steepness and height to which waste slopes could be placed, the rate at which waste filling could take place (i.e., the rate at which the loading could be applied), and the required time periods during which each loading needed to remain in place prior to steepening or raising the slopes.

4.4 *Waste Sideslope Modifications*

During the decade since the permit for Phase 8 was approved and completion of Phase 8A, 8B, and 8C′ construction, several modifications have been accomplished whereby the landfill sideslopes have been steepened from the original 5H:1V inclination to as steep as 2.5H:1V. To date, the sideslope modifications have resulted in more than 1 million yd^3 of additional landfill capacity without increasing the landfill footprint.

These modifications have been possible by rigorously tracking the waste density, applied use of the data from the geotechnical instrumentation, advanced sampling and strength assessment of the gray foundation clays, and staged construction of terraced landfill slopes. These factors are further described below.

As previously mentioned, average densities ranging from 81 pcf to 94 pcf for the relocated waste from the MSW Landfill had been measured using test pits during waste relocation. This range of densities had been used to confirm the initial design slopes. As new gate-receipt waste is placed in Phase 8, the density of the new waste has been routinely recalculated by dividing the recorded tonnage (cumulative weight from the truck scales) during any selected period by the volume consumed in Phase 8 during the same period calculated from aerial or ground surveys. The analyses have demonstrated that the recent new waste density is ~75 pcf (considerably less than the value of 85 pcf used in the original design). The density calculations have also been refined using a mass balance relationship to account for the addition of weight from precipitation, and reductions in weight from evaporation as well as removal of leachate and landfill gas condensate from the landfill.

The geotechnical instrumentation data has been used to assess when strain in the Presumpscot Clay resulting from waste loading (measured with the inclinometers) is occurring under drained or undrained conditions (assessed with pore pressures measured with the vibrating-wire piezometers), and in turn when the strength gain resulting from the load is complete. The increased strength of the clay has then been confirmed by drilling and sampling the clay near the toe of the landfill and testing it in direct simple shear using the SHANSEP method.

Like the initial stability design analyses, the analyses presented for slope modifications have included staged loading with multiple interim grading plans as well as final closure grades. Stability analyses for each stage of loading are dependent on the strength gain resulting from the previous stage. The strength gain was calculated using the SHANSEP method and includes strength gain resulting from shear strain (i.e. Shear Induced Strength Gain (SISGa) described

in the Reynolds Companion Paper). Progression to subsequent waste fill stages is dependent on the time required for strength gain. The slope modifications have specified the time between stages, and also mandated that the progression to the next fill stage be dependent on achieving confirmatory results of strain and pore pressures from the geotechnical instrumentation monitoring and/or additional strength testing. Stability terraces in the waste sideslopes, typically about 65 ft wide (see Figure 13), have been included in some of the modified slope designs to force the location of the critical failure surfaces further toward the middle of the landfill.

As a result of the extra million yd^3 of disposal capacity, the sideslope modifications have allowed WMDSM to delay placing waste in unfilled areas, thereby postponing capital expenditures for new cell construction and providing considerable savings in leachate treatment costs.

5 SUMMARY AND CONCLUSIONS

5.1 *Project Status*

To date, the liners and leachate collection systems for Phases 8A and 8B have been constructed and waste has been placed to full or partial height. The liner system has been constructed in the southern portion of Phase 8C with waste placement ongoing. Design modifications to the northern part of Phase 8C are underway, requiring further analyses of stability of the Presumpscot Clay formation. Based on the approved and envisioned modifications, the Phase 8 landfill is projected to provide disposal capacity until the year 2025.

5.2 *Take-Away Points*

The Crossroads Landfill has provided (and continues to provide) valuable opportunities for geotechnical engineers to gain detailed experience in design techniques and methods for the Presumpscot Formation, just a few of which are summarized below.

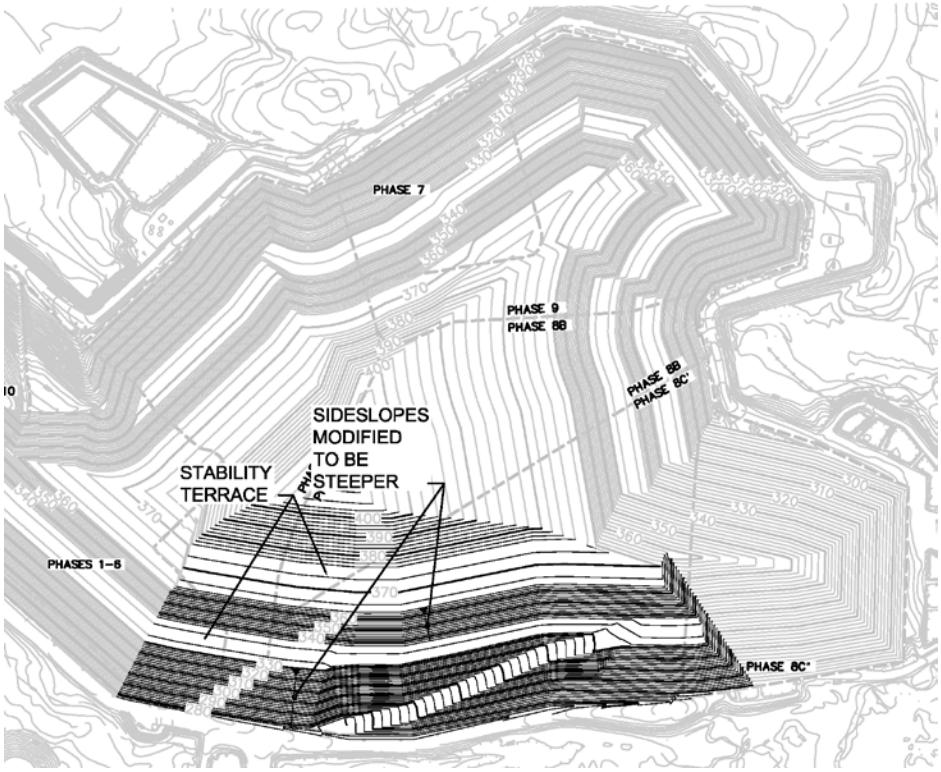

Figure 13. Example of sideslope modifications using steeper sideslopes and stability terraces.

- Maintaining stability while deconstructing a landfill (i.e., waste excavation/relocation) when performed over the Presumpscot Formation can be just as challenging as constructing and filling a landfill over the Presumpscot Formation.
- In addition to accelerating the time required for consolidation, Prefabricated Vertical Drains (PVDs, aka wick drains) are also quite valuable for increasing the designer's confidence in uniform consolidation and therefore uniform strength gain in large areas of the Presumpscot Clay. This is paramount in being able to trust the factors of safety calculated using the various slope stability programs.
- Maximizing load near the edge of landfill (i.e., consolidation and strength gain near the toe of slope) increases overall stability and provides considerably more disposal capacity for the same lined footprint. The edge loading, however, must be done carefully to prevent localized failure and a possible retrogressive failure from developing. For landfills, this may be accomplished using MSE berms, PVDs, incremental construction of the berms, and careful consideration of the support provided by the upper crust. The application of traditional bearing capacity analyses (equations) may not be as accurate as using limit-equilibrium slope stability programs, but the process of understanding bearing capacity and punching shear failure modes can help guide the geotechnical investigation and the designer's search routines used in the slope stability programs.
- The stiff upper olive-brown clay crust of Presumpscot Clay can be used to our advantage, but must be done so carefully. Variations in the thickness of the crust and the strength of the gray clay immediately beneath the crust must be considered.
- Continued monitoring throughout the active life of these types of facilities is a vital part of design to: (i) confirm the original design assumptions; (ii) provide stability alerts with pre-planned response actions as necessary; (iii) guide waste-fill sequencing such that optimum loading can be achieved for present and future configurations; and (iv) provide information to justify steeper and higher slopes (sideslope modifications) after some strength gain in the Presumpscot Clay has been achieved.

ACKNOWLEDGEMENTS

The authors would like acknowledge many members of the WMDSM/Geosyntec team who have contributed significantly to the success of the Phase 8 project. Many of these acknowledgements were already articulated at the end of the Reynolds Companion Paper and are therefore not repeated here.

In addition to those sentiments, we would be remiss if we did not mention Be Schonewald for her geotechnical contributions to the Crossroads facility over many years.

The lead author of this paper would also like to emphasize his professional appreciation for the co-authors of this paper, and especially for Richard (Dick) Reynolds. Dick's foresight has been noteworthy to be sure, but the manner in which he relentlessly pursues and rigorously applies geotechnical principles is even more exemplary and has been a catalyst for many of the accomplishments described in this paper.

REFERENCES

Richardson, G., and Reynolds, RT, (1990), "Design of a Lined Landfill on Compressible Clays", First Materials Engineering Congress - Serviceability and Durability of Construction Materials, Denver CO, ASCE Construction Institute.

Ladd, C.C. and Foote, R. (1974), New Design Procedure for Stability of Soft Clays", Journal of Geotechnical Engineering, ASCE, Vol 100 (7): pp 763-786.