Geotechnical Study
Northern Solid Waste Management Center – 1, Pigeon Point Disposal Area
Wilmington, Delaware

(SEA Reference 985176.06)

Schnabel Engineering

Schnabel Engineering

Schnabel Engineering Associates, Inc.
510 East Gay Street
West Chester, PA 19380
610-696-6066 • Fax 610-696-7771

December 10, 1999

Delaware Solid Waste Authority
1101 Lambson Lane
New Castle, DE 19720

Attn: Mr. Rodney Alexander, P.E.

Subject: Contract 985176.06, Geotechnical Study, Northern Solid Waste Management Center – 1, Pigeon Point Disposal Area, Wilmington, Delaware

Gentlemen:

We are pleased to submit this report for the geotechnical evaluation performed for the Pigeon Point Landfill site. These services were provided in accordance with our agreement dated July 8, 1998.

Subsurface exploration using in-situ testing and sampling was conducted at the Pigeon Point Landfill site; soil laboratory testing was conducted on several samples obtained from the field; and previous reports for the site were reviewed as part of this study. The data acquired from the exploration, testing, and research was used to derive geotechnical parameters and subsurface profiles for the slope stability analysis to evaluate the impact to filling the site with stabilized sludge materials. The slope stability of the site was analyzed using computer software that considered the geotechnical parameters, the subsurface stratigraphy, and the proposed final loading condition. A summary of our findings and conclusions is contained in the attached report.

We have endeavored to prepare this report in accordance with generally accepted geotechnical engineering practice and make no warranties, either express or implied, as to the professional advice provided under the terms of our agreement and included in this report.

We appreciate the opportunity to be of service for this project. Please do not hesitate to contact either of the undersigned if clarification is needed for any aspect of this report.

Very truly yours,

SCHNABEL ENGINEERING ASSOCIATES, INC.

Michael G. Gill, HCF

Michael G. Gill, P.E.
Project Engineer

Christopher M. Reith, P.E.
Associate

Allen W. Cadden, P.E.
Senior Associate

MGG:CMR:AWC:hcf

cc: Delaware Solid Waste Authority (3)
Attn: Mr. Rodney Alexander

GEOTECHNICAL STUDY
NORTHERN SOLID WASTE MANAGEMENT CENTER - 1
PIGEON POINT DISPOSAL AREA
WILMINGTON, DELAWARE

Table of Contents

1.0 EXECUTIVE SUMMARY

Schnabel Engineering Associates (SEA) has performed a geotechnical study for the Pigeon Point Disposal Area at the Delaware Solid Waste Authority, Northern Solid Waste Management Center – 1, in Wilmington, Delaware. The analysis considered the placement of up to about 1,500,000 tons (1,300,000 cy) of stabilized sludge or similar DNREC approved materials on top of the existing landfill.

The subsurface exploration included cone penetration, dilatometer, field vane shear, standard penetration, and soil laboratory tests. The field tests correlated well with one another and were used in conjunction with previous data presented in reports prepared by Edward H. Richardson Associates, to estimate the parameters required for slope stability, bearing capacity, and settlement analysis.

Given the results of the field exploration and soil laboratory testing, the recent deposits and dredge spoils are normally to slightly underconsolidated. The in-situ and laboratory testing indicates that these materials have gained considerable strength due to consolidation from the landfill loading.

The computer modeling software, PCSTABL, was employed to evaluate potential failure surfaces for four critical slope cross sections utilizing the new strength parameters measured in this exploration. The estimated factor of safety from the analysis was satisfactory to allow the placement of the stabilized sludge materials using the limitations discussed herein.

2.0 INTRODUCTION

This study was conducted for the Delaware Solid Waste Authority (DSWA), Pigeon Poin Landfill, for the purpose of evaluating the geotechnical stability of the area for the additional stabilized sludge material placement. The stability evaluation considered the increase in loading resulting from the placement of up to about 1,500,000 tons of stabilized sludge or similar DNREC approved materials over the existing landfill.

2.1 Site Description

The Pigeon Point Landfill is located at the end of Resource and Energy Lanes in New Castle County, Delaware. The landfill is bordered by the old Delaware Reclamation Plant to the northwest, the Delaware River to the east, the Delaware Memorial Bridge to the south, and the Penn Central Railroad Tracks to the west.

The existing landfill area has a 120 acre footprint with about a 60 acre plateau. The landfill was operational from about 1971 to 1985 and accepted shredded and unshredded solid waste. It appears that soil, sludge, and dredge spoil materials were used as daily and intermediate cover and for the final soil cap. The landfill is generally overgrown with brush, small trees, and reeds, although periodically stabilized sludge materials are placed on top of the landfill to grade out the surface depressions. Several monitoring wells and leachate observation wells are located within and around the landfill footprint. A gas collection and flare system, and leachate collection system are currently in operation.

The landfill was originally constructed with a soil liner, and a perimeter leachate collection system was added after initial filling operations started. Additions and modifications to the original leachate collection system were performed on several occasions. A preliminary design report was prepared for the second phase of the landfill in 1973 by Edward H. Richardson Associates, which included subsurface exploration, laboratory testing, geotechnical engineering analysis, as well as grading plans and construction details. A Closure Plan was prepared by Duffield Associates in 1985.

The base grades are believed to range from EL 5 to EL 10 and the current closure grades range from EL 55 to EL 61, which indicate that up to about 50 to 55 ft of waste were placed in the landfill.

2.2 Proposed Construction

We understand that DSWA would like to increase the thickness of the cap by placing stabilized sludge and/or other DNREC approved materials on the existing landfill. As detailed in the contract, the estimated amount of this material was expected to range from 90,000 to 160,000 tons per year for seven years. This amounted to about 525,000 to 934,000 cy, for a lift averaging about three to five feet over the entire 120 acre landfill footprint.

However, considering slope stability issues and minimum top slopes of about five percent for improved run-off, this results in a variable depth of new fill over the current landfill grades. The maximum height of the landfill was limited to EL 90 to control the stress increase on the subgrade and differential settlements. This configuration results in about 1,300,000 cy of additional air space neglecting additional settlement of the subgrade and existing waste and the two foot rooting zone and topsoil layer.

2.3 Review of Previous Reports and Existing Data

There are reports containing subsurface, groundwater, lab testing, and design information for this site that were prepared in the early 1970's by Edward H. Richardson Associates. Modifications were made to the leachate collection system in the early 1980's. A Closure Plan, groundwater monitoring reports, and an evaluation of the buried solid waste were prepared by Duffield Associates. The reports, plans and correspondence were provided to us by DSWA and were used in our site evaluation. In addition to the materials provided by DSWA, historical information was reviewed at the office of the Delaware Geological Survey. A bibliography of the reports and correspondence that were used in our analysis is provided in Appendix E.

A 1948 United States Geological Survey map of the site shows that the area is predominantly marshland with Magazine Ditch and another un-named drainage channel crossing through the site. The surface grades are shown ranging from EL 0 to EL 10. A 1954 air photo shows the site as generally unchanged. A 1962 air photo shows that the site has been filled and that the drainage channels are gone. Magazine Ditch has been realigned to its present location, parallel and adjacent to the southern span of the Delaware Memorial Bridge.

The design report prepared by Richardson states that eight to 12 ft of hydraulically placed dredge spoil had been placed at this site prior to landfilling, and this material was estimated to be underconsolidated in 1973, based on the subsurface exploration and laboratory test results. Given the available information, the original subgrade of the landfill is believed to have been about EL 11 to EL 18 prior to construction.

3.0 SUBSURFACE EXPLORATION AND CONDITIONS

3.1 Subsurface Exploration

A program of subsurface exploration was designed and conducted to evaluate the current shear strengths of the soils underlying the landfill. These measurements were also compared to the strengths estimated in 1973. The subsurface exploration was conducted from August 10 to 20, 1998, under the supervision of SEA. In-Situ Soil Testing, L.C., performed nine cone penetration test soundings and two dilatometer test soundings. Hardin-Huber, Inc., performed two test borings with field vane shear tests, and three additional test borings with Standard Penetration Tests. The locations of the field tests are shown in the Location Plan, Figure A-1, included in Appendix A.

The cone penetration tests were performed at locations on the perimeter dike and two locations within the disposal area. The cone apparatus consisted of an electric penetrometer capable of recording pore pressures, tip resistance, and sleeve friction. The tests were performed in general accordance with ASTM D3441 Standard Method for Deep, Quasi-Static, Cone and Friction-Cone Penetration Tests of Soil. In addition to gathering tip resistance, friction sleeve resistance and pore pressures, pore pressure dissipation tests and down hole seismic tests were conducted at select locations.

The flat dilatometer tests and the field vane shear tests were performed at the locations of SEA-9 and SEA-13. The dilatometer tests were performed in general accordance with the Suggested Method for Performing the Flat Dilatometer Test as recommended by ASTM Subcommittee 18.02, and the field vane shear tests were performed in general accordance with ASTM 2573 Standard Test Method for Field Vane Shear Test in Cohesive Soil.

Standard Penetration Tests were taken in the same boring as the field vane shear tests and at other locations around the landfill perimeter. We have included logs of the subsurface exploration obtained from this investigation in Appendix A.

3.2 Subsurface Conditions

The subsurface exploration and previous reports indicate that the on-site soils can be grouped into general strata to facilitate the discussion and analysis of this study. Reworked or compacted fill materials are present in the top 10 to 15 ft of the dike areas and are defined as Stratum A. Below the fill soil of Stratum A, hydraulic fill soils and recent alluvial deposits of Stratum B are present. The soils of Stratum B are generally classified as elastic silt containing organic matter. The upper portion of Stratum B is believed to consist of hydraulic fill or dredged soil and is generally present from about EL -5 to EL 10, except where the old stream channels were filled. Below the dredged soil, the lower portion of Stratum B is considered to be recent alluvial deposits. These materials were encountered down to elevations ranging from approximately EL -10 to EL -50. Below the recent deposits, the generally granular soils of Stratum C, the Columbia Formation, or the soils of the Stratum D, the Potomac Formation, are present. Both of these strata possess a much higher shear strength and lower compressibility than the Stratum B materials.

3.3.2 Dilatometer Tests

Dilatometer tests (DMTs) were performed at two test locations, SEA-9 and SEA-13. The DMTs were performed in general accordance with the Suggested Method for Performing the Flat Dilatometer Test as recommended by ASTM Subcommittee 18.02. The test consists of pushing a flat rectangular plate, with the approximate dimensions of 240 x 95 x 15 mm, into the soil and inflating a circular membrane, approximately 60 mm in diameter, while simultaneously recording the resisting soil stress.

The dilatometer test can be used to estimate a wide range of soil properties. The properties of primary concern for our study are the soil classification and the undrained shear strength, S_u. Other properties that the DMT can be used to estimate include: coefficient of earth pressure at rest (K_o), drained plane strain friction angle (ϕ'_{ps}), preconsolidation pressure (σ'_{pc}), dilatometer modulus (E_D), and tangent modulus (M). All of these estimated properties are presented in Appendix A as plots with depth and as tabular data for each of the dilatometer tests. The derivation of the soil classification and the undrained shear strength are provided below. The references for the other estimated soil properties are also provided.

3.3.2.1 Soil Classification

The material index, I_D, is the computed parameter that determines the soil classification for DMTs. The material index is expressed as (Marchetti, 1980):

$$I_D = (p_1 - p_o)/(p_o - u_o)$$

where:

p_1 = corrected soil pressure at 1.1 mm membrane expansion
p_o = corrected pressure at 0.0 mm membrane expansion
u_o = pre-insertion water pressure, (unit weight of water) x (assumed depth below water)

The material index versus depth plots consider soils with an $I_D < 0.6$ to behave like a clay, and soils with an $I_D > 1.2$ to behave like a sand. The dilatometer classification table used for the plots is shown as Table C-1 in Appendix C. The tabular data includes classification of the soil by a more complex relationship between material index and dilatometer modulus. This chart is shown as Figure C-2 in Appendix C (Marchetti and Crapps, 1981).

3.3.2.2 Undrained Shear Strength

Undrained shear strength, S_u, is determined from the horizontal stress index, K_D, and the assumed effective vertical shear stress, σ'_{vo}, as follows (Marchetti, 1980):

$$S_u = 0.22\sigma'_{vo}\,(0.5K_D)^{1.25}$$

where: σ'_{vo} = calculated effective stress

K_D is calculated as follows:

$$K_D = (p_o - u_o) / \sigma'_{vo}$$

The total unit weights were computed using the relationship shown in the above referenced chart and correlate well with the average unit weight measured by previous laboratory testing by others at this site. Undrained shear strength versus elevation plots are contained in Appendix C for Test Locations SEA-9 and SEA-13.

3.3.2.3 Additional Soil Properties

Additional estimated soil properties are included in the dilatometer output data presented in Appendix A. The references for the estimated properties are as follows:

Estimated Soil Property using the DMT	Reference for Correlations
Coefficient of Earth Pressure at Rest for Clays, K_o	Marchetti, 1980
Coefficient of Earth Pressure at Rest for Sands, K_o	Schmertmann, 1983
Plane Strain Drained Friction Angle, ϕ'_{ps}	Schmertmann, 1982
Preconsolidation Pressure, P_c	Marchetti, 1980
Dilatometer Modulus, E_D	Marchetti, 1980
Tangent Modulus, M	Marchetti, 1980

3.3.3 Cone Penetration Tests

Cone penetration tests (CPTs) were performed at Test Locations SEA-6 through SEA-15. The cone equipment used was an approximately 36 mm diameter electric penetrometer with a pore pressure transducer located between the tip and friction sleeve to allow for the measurement of pore pressures. The CPTs were performed in general accordance with ASTM D3441, Standard Method for Deep, Quasi-Static, Cone and Friction-Cone Penetration Tests of Soil. As the cone is pushed into the soil, the cone resistance, sleeve resistance and pore pressures are continuously measured. Correlations exist for the derivation of soil classification and undrained shear strength, S_u, from the measured data.

3.3.3.1 Soil Classification

The soil classification was determined by using the classification chart included in Figure C-3 and Table C-1 of Appendix C (Robertson, 1990). The normalized cone resistance, Q_t, is shown relative to friction ratio, F_r, and to pore pressure ratio, B_q. The definitions of these parameters are as follows:

$$Q_t = (q_t - \sigma_{vo}) / (\sigma_{vo} - u_o)$$

$$F_r = f_s / (q_t - \sigma_{vo})$$

$$B_q = (u_2 - u_o) / (q_t - \sigma_{vo})$$

where: q_t = measured corrected cone resistance for unequal end area effects
σ_{vo} = total vertical stress = (assumed unit weight) x (depth)
u_o = equilibrium pore pressure = (unit weight of water) x (assumed depth below water)
f_s = measured sleeve friction
u_2 = pore pressure measured between cone and the friction sleeve

The plots in the above referenced chart and table classify the soil into seven to nine soil behavior types. Plots of the soil classifications and the measured data including f_s, Q_t, F_r, and u_2 are shown with depth for each test location in Appendix A.

3.3.3.2 Undrained Shear Strength

The undrained shear strength, S_u, of the soil can also be derived from the cone penetration test data. The corrected cone resistance, q_t, can be correlated with undrained shear strength based on a cone factor, N_{kt}. The undrained shear strengths from the field vane shear tests and the dilatometer tests were used to establish the cone factor, N_{kt}. The cone factor is expressed as follows (Lunne, *et al.*, 1994):

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

Utilizing the undrained shear strength, S_u, from the other test methods, an average N_{kt} was established for this site. The N_{kt} calculated from the field vane shear tests and dilatometer tests are shown as Figure C-4 in Appendix C. The average N_{kt} was calculated to be 11 for this site using the field test data. The average N_{kt} agrees well with the literature that suggests N_{kt} has been observed to vary from 11 to 19 for normally consolidated marine clay with the field vane as the reference test (Lunne and Kleven, 1981).

The cone factor, $N_{kt} = 11$, was then used to determine the undrained shear strengths from the cone penetration data at this site. The undrained shear strengths versus elevation are plotted in Appendix C for each CPT location.

3.3.3.3 Pore Pressure Dissipation Tests

At CPT Test Locations SEA-14 and SEA-16, pore pressure dissipation tests were conducted. At a specific depth, the cone position was fixed and an initial excess pore pressure was measured. Then, the initial excess pore pressure dissipation was measured with time, and the hydraulic properties of the soil were computed. The computed properties include the coefficient of horizontal consolidation, C_h, and the coefficient of horizontal hydraulic conductivity, k_h. The pore pressure dissipation test results are presented in Appendix A.

These results indicate a horizontal coefficient of permeability of about 8.6 x 10^{-7} cm/s for the dredge spoils and recent deposits, and a coefficient of horizontal compressibility of about 0.15 in^2/min. As expected from the anisotropic nature of these materials, the values are higher in the horizontal direction than those measured in the vertical direction from the consolidation tests.

3.3.4 Seismic Cone Penetration Tests

At three of the cone penetration test locations, SEA-9, SEA-12, and SEA-13, cone equipment was used that included a geophone to record shear waves produced at the ground surface. These tests are more specifically referred to as seismic cone penetration tests (SCPTs). The additional data recorded by these tests consists of shear wave velocities and shear moduli. The tabulated data and plots for the SCPTs are included in Appendix A.

3.4 Soil Laboratory Testing

Four undisturbed thin wall tube samples obtained from Boring SEA-9 were tested in our soils laboratory. Laboratory testing consisted of natural moisture content, natural density, grain size analysis, Atterberg Limits, consolidation, unconfined compression, and consolidated undrained triaxial shear strength. The physical property tests aided us in confirming the visual classification of these materials in accordance with ASTM D2487, and in selecting parameters for use in the geotechnical analysis and recommendations. The consolidation tests were used as a check on the in-situ testing data and to estimate settlements. The unconfined compression testing was done as a comparison to the in-situ testing data. The triaxial testing was conducted to estimate the drained shear strength properties of the dredge material and recent deposits. The results are shown on the Summary of Soil Laboratory Tests contained in Appendix B.

The soil classifications, percent passing the No. 200 sieve, Atterberg Limits, and natural densities were consistent with those recorded from the previous studies. The undrained shear strengths measured by the unconfined compression tests agreed well with those measured by the in-situ techniques, and are included on the shear strength plot of Location SEA-9. An effective friction angle of about 31 degrees was measured from the consolidated undrained triaxial compression tests conducted on the dredge spoils and recent deposits. The consolidation tests indicated that the materials were normally consolidated to slightly underconsolidated. The estimated compression index, C_c, ranged from 0.79 to 0.87, and the estimated coefficient of compressibility, C_v, ranged from 0.006 to 0.01 in^2/min.

Groundwater levels in the subsurface are monitored by wells around the landfill perimeter and leachate observation wells within the landfill footprint. The groundwater levels observed at the time of exploration are noted on the logs and were used to supplement the existing well data. The groundwater levels observed during the subsurface exploration varied widely, but for analytical purposes generally range from about EL 5 to about EL 15 in the recent deposits of Stratum B and granular soils of Stratum C within the landfill footprint. Elevated water levels were observed in SEA-14 and SEA 16 which may be attributable to 'ponding' of water within the permeable waste and relatively impermeable daily and intermediate cover soils, and/or excess pore pressures. This water was assumed to exist as a phreatic surface in the slope stability analysis, since this is a more conservative case.

The site history and construction procedures of the landfill have influenced the consistency or strength of the underlying soils. The perimeter dike was likely constructed from the hydraulic fill that was allowed to dry and desiccate, then scraped off and piled on the dike, and from imported materials. Therefore, the materials around the perimeter have been dried out and reworked, resulting in strengthening of these soils. The upper fill soils in the perimeter dike area generally exhibit medium to stiff consistency based on the Standard Penetration Test (SPT) blowcounts and in-situ testing. The dredged soil and recent deposits below the dike and landfill were not allowed to desiccate, and thus exhibit a very soft to soft consistency. The variations in shear strength can be observed in the plots of Appendix C.

Empirical correlations of the measured undrained shear strength with estimated effective stress (assuming a steady state phreatic surface) generally indicate that the interior dredge soils and recent deposits are essentially normally consolidated to slightly underconsolidated.

3.3 <u>In-Situ Test Results</u>

The three methods of in-situ testing conducted at the site were field vane shear tests, dilatometer tests and cone penetration tests. The testing procedure and the data reduction are discussed in this section.

3.3.1 Field Vane Shear Tests

The field vane shear tests (FVSTs) were conducted at Test Locations SEA-9 and SEA-13. The FVSTs were used to determine the undrained shear strength, S_u, of the soil. Both peak and remolded undrained shear strengths were measured.

The FVSTs were performed in general accordance with ASTM D2573, Standard Method for Field Vane Shear Test in Cohesive Soil. A 4¼ inch ID hollow stem auger was drilled to a depth above the determined test elevation; then the vane was pushed 2.5 ft into the soil below the auger and allowed to sit 10 minutes before the test was conducted. The vane was then turned by a hand operated Acker geared drive at the shear rate of about 0.1° per second. The maximum torque observed was used in the peak S_u calculation. After the peak test, the vane was rotated 10 times and immediately tested again at a shear rate of 0.1° per second. The observed maximum torque after 10 rotations was used in the remolded S_u calculations. Friction tests were also conducted to determine the maximum torque required to turn the rod without the vane. A gauge located on a moment arm measured force. The torque was calculated by multiplying the force by the length of the moment arm. Rotational displacement was measured by counting the number of cranks turned in the geared drive.

The dimensions of the vanes that were used differed slightly from the dimensions recommended by ASTM. Two vane sizes were used for testing. One vane had a 3⅝ inch diameter and a six inch height, the other vane had a 2½ inch diameter and a 4½ inch height. Both were tapered and were attached to a ¾ inch diameter rod.

The undrained shear strength is calculated from the measured torque by the following equation (ASTM D2573):

$$S_u = k(T - f)$$

where:

S_u = undrained shear strength, lb/ft^2 (peak or remolded)
k = vane constant reflecting vane dimensions, ft^{-3}
T = measured maximum torque from vane test, lb-ft (peak or remolded)
f = measured maximum torque from friction test, lb-ft (peak or remolded)

k is calculated as follows:

$$k = 1728[(2/(\pi D^2H)) + (2.70/(2D^3\text{-}d^3))]$$

where:

D = vane diameter, inch
H = vane height, inch
d = rod diameter, inch

The vane constant, k, for the 3⅝ inch diameter vane was calculated to be 10.88 ft^{-3}; and the vane constant for the 2½ inch diameter vane was calculated to be 31.08 ft^{-3}. The vane shear test results are presented on the boring logs at the vane tip elevations. Logs of actual test results and plots of strength and torque versus rotation are also contained in Appendix A.

It has been suggested in the technical literature that the undrained shear strength measured by FVST, S_u (FVST), should be corrected to more accurately reflect the undrained shear strength mobilized in the field, S_u (mob). Bjerrum (1973) recommends that the S_u (FVST) be corrected according to the Plasticity Index. The recommended correction equation is as follows:

$$S_u\,(\text{mob}) = \mu\, S_u\,(\text{FVST})$$

The correction coefficient versus Plasticity Index is shown in Figure C-1, in Appendix C. Previous laboratory testing conducted by Richardson indicates the average Plasticity Index for the dredged soil and recent deposits of Stratum B soil is about 20%. The corresponding μ, from Bjerrum's chart, is equal to about 1.0. Therefore, the undrained shear strength measured from the FVST is essentially equal to the undrained shear strength mobilized within an embankment slope.

4.0 GEOTECHNICAL ENGINEERING ANALYSIS

4.1 General

A geotechnical analysis was conducted to evaluate the slope stability of the Pigeon Point Landfill considering placement of stabilized sludge material over the top of the landfill. The soil material properties have been evaluated considering our subsurface exploration, in-situ testing, soil laboratory testing, and our review of previous data. The soil strata have been categorized into subgroups having similar strength and unit weight properties. The computer software, PCSTABL, was employed to evaluate the possible failure surfaces and their corresponding factors of safety.

4.2 Geotechnical Properties of the Slope Materials

The geotechnical properties have been evaluated for the soil and municipal solid waste to develop input parameters for the slope stability analysis. The properties needed to analyze slope stability are unit weight and shear strength.

4.2.1 Unit Weights

The estimated unit weights for the soil groups and municipal solid waste are presented in Table 1. The moist unit weights for the recent deposits, the perimeter dike soils, and the interior dredge soils were estimated from the previous and new soil laboratory data, and in-situ testing. The moist unit weight data for the Columbia and Potomac Formations were estimated from soil classifications, the DMT results, and our past experience with these soils. Soils believed to be saturated when tested were given the same saturated unit weight as the moist unit weight. Saturated unit weights were estimated for soils believed to have been tested in an unsaturated state.

A parametric evaluation using various unit weight values for the municipal solid waste was used in the analysis. A moist unit weight of 38 pcf was used in the original analysis in 1973 (Richardson, 12/73). Moist unit weight values ranging from 35 to 100 pcf were evaluated in the slope stability, but only the analysis using the tabulated values is presented in this report since it represents the more conservative results. A summary of unit weight values used in this analysis is shown in Table 1.

Table 1
Unit Weight Parameters used in Slope Stability Analysis

Material Type	Moist Unit Weight, γ_m (pcf)	Saturated Unit Weight, γ_{sat} (pcf)
Recent Deposits and Dredge Spoils	96	96
Columbia and Potomac Formation	130	130
Municipal Solid Waste	70*	100*
Stabilized Sludge	88**	95**

* From general parameters given for municipal solid waste in Sharma and Lewis, *Waste Containment Systems, Waste Stabilization and Landfills, Design and Evaluation*, 1994.

** Provided by VFL Laboratories.

4.2.2 *Shear Strength*

The undrained shear strengths for the recent deposits, the perimeter dike soils, and the interior dredge soils were estimated using the CPT, DMT, FVST, and the soil laboratory test data obtained during the subsurface exploration under this contract. The estimated undrained shear strength versus elevation plots are presented in Appendix C for each test location.

The ratio of shear strength to the overburden pressure, or c/p ratio, typically ranges from 0.22 to 0.28 for normally consolidated clays. A c/p ratio of 0.27 was estimated for these soils and is shown on the undrained shear strength plots at the location where multiple field test methods were performed. Where this line matches the field measured shear strength, the materials are normally consolidated. Where the field measured shear strength trend line is steeper, the soils are underconsolidated.

Table 2 presents a summary of shear strength parameters selected for use in the analysis. The soils of the Columbia Formation, municipal solid waste, and stabilized sludge are believed to behave as a drained material due to their permeability; therefore, no undrained parameters are presented in Table 2. Where a range of undrained shear strength is presented in Table 2, the material was modeled as having an increasing strength with depth.

Table 2
Shear Strength Parameters used in Slope Stability Analysis

Material Type	Undrained Condition	Drained Condition†	
	Undrained Shear Strength, S_u (psf)	Cohesion Intercept, C (psf)	Drained Friction Angle, ϕ (degrees)
Recent Deposits and Dredge Spoils around the Perimeter	400 to 750	0	30
Recent Deposits and Dredge Spoils below the Landfill	400 to 2,500	0	30
Soils of the Columbia and Potomac Formations	n/a	0	35
Municipal Solid Waste	n/a	400†	20†
Stabilized Sludge	n/a	0	30

† From general parameters given for municipal solid waste in Sharma and Lewis, *Waste Containment Systems, Waste Stabilization and Landfills, Design and Evaluation,* 1994.

The drained strength parameters for the municipal solid waste are the generally recommended published values (Sharma, *et al.*, 1994). The shear strengths were developed for each cross section analyzed as they differed widely with location around the site.

4.3 Slope Stability Analysis

A stability analysis was conducted to evaluate the potential for shear failure to occur through the foundation, embankment, and landfill materials under the static loading of the stabilized sludge. Four critical cross sections were analyzed for stability and their locations are shown on Figure A-1. The existing ground surface data for the slope cross sections was taken from the topographic plan created by L. Robert Kimball and Associates in June 1996. Slope stability was performed using the computer program PCSTABL and the Modified Bishop limit equilibrium analytical procedure for random circular failure surfaces.

The four cross sections were analyzed using the drained and undrained parameters. The phreatic surface was estimated for the effective stress (long term) analysis. This analysis considered the existing side slopes with an increased slope height and increasing the side slopes where they are currently relatively flat.

Factors of safety were computed for several hundred failure surfaces and the minimum factor of safety for the surfaces was found to be greater than 1.3 under static loading conditions using total strength parameters where the existing side slopes are steeper than 4H:1V. A minimum factor of safety of 1.3 was calculated for areas where the existing slopes were steepened to 4H:1V, using total strength parameters. The long term analysis indicates a factor of safety greater than 2.0 for all cases analyzed using the effective strength parameters. The results of the slope stability analysis are included in Appendix D. We consider the above stated minimum factors of safety acceptable, and this is consistent with previous analyses at this and other sites.

The final grading should consider the following restrictions: new fills at the top of the landfill can be as steep as 4H:1V without causing a slope stability problem; where the existing slopes are currently steeper than 4H:1V they can be left as is; the top of the landfill can be graded at up to a five percent slope, provided that the maximum elevation does not exceed about EL 90.

4.4 Settlement Analysis

It appears that subgrade settlement on the order of three to five feet was originally predicted, based on the Phase II Preliminary Design Report prepared by Richardson. The time for primary consolidation was also predicted to be on the order of 40 to 50 years. The actual settlement at any given location under the landfill will be a function of loading or waste height, thickness of compressible deposits, and time. Subgrade settlement can impact the gravity leachate collection system, and settlement within the waste mass can effect the gas collection system and final cover.

Based on a review of previous elevation data and site observations, it appears that the waste has undergone differential settlement relative to the subgrade on the order of seven to nine feet at the leachate observation wells, and up to about 10 to 12 ft based on the proposed final grades in the closure plan and the site grades in 1996. The waste mass will continue to settle under its own weight and due to the addition of new stabilized sludge. There is no accurate method of predicting what the final settlement will be due to compression of the waste mass. The effects of additional relatively large magnitude settlements need to be taken into consideration in the modifications to the current gas collection system where the grades are raised.

We estimated subgrade settlement due to the existing waste using the previous subsurface exploration data from Richardson and the new subsurface data gathered as part of this study. This was done as a check on the previous estimates, since it appears that Richardson used a unit weight of 38 pcf for the waste material, which would underestimate the increase in stress in the subsoils. We estimated the settlement to be about 5.2 ft under loading from 50 ft of waste, where 50 ft of compressible material exists under the landfill. The higher unit weight results in slightly higher settlements than previously given by Richardson, but they are in the same general magnitude.

Additional settlement of the subgrade due to placement of zero to 30 ft of new cap material was estimated to range from zero to two feet, considering an average unit weight of 90 pcf and the proposed final grades. This settlement is due to primary consolidation of the dredge materials and soft recent deposits, and is expected to take from 30 to 60 years. The magnitude of subgrade settlement that occurs due to this new loading is expected to be relatively minor during the active life of about 10 years. Secondary settlement is expected to be negligible during this period.

The existing waste will also settle under the load of the new material. It is difficult to predict settlement of a waste mass since settlement occurs due to self weight and new loads by several mechanisms, including mechanical, raveling, physical chemical change, and biochemical change. These mechanisms are also load and time dependent and influenced by type of waste, age, thickness, initial density, placement methods, type and amount of cover soils used, moisture content, temperature, and gas generation.

We used the Sower's Method, the Gibson and Lo Model, and the Power Creep Law to estimate the range of settlement expected to occur in the existing waste mass under the load of the new material. The range of settlement under the full height loading at the center of the landfill ranges from about four to 10 ft using the three models. Settlement at the crest of the new fill, EL 60, is estimated to range from about one to two feet. An average gain in useable air space during the active filling period of two feet due to compression of the existing waste is believed to be a conservative estimate.

4.5 Impacts to Leachate Collection System

The existing leachate collection system consists of a series of gravity flow pipes that drain into two lift stations, and three pump stations that ultimately discharge into a force main. Design slopes of the gravity piping system ranged from 0.4% up to about 12% where shown on the various plans; however, most of the piping is located around the perimeter rather than below the landfill. A schematic leachate collection system piping layout is included in Appendix C.

The additional cap material should not increase the stress around the landfill perimeter by more than 10 to 25 percent over the original design estimates considering an average unit weight of 90 pcf. This is due to the proposed final side slopes being flatter than originally designed, the original closure grades being close to the proposed final grades, and the minimum suggested unit weight of 1,200 lbs per cy for compaction of the landfill material referenced in the design report. Additional settlements over the total originally predicted amounts on the order of one to two feet were estimated. This could result in additional differential settlements of up to about one to two feet, which result in a maximum change in angular distortion of about 0.002 ft/ft. This could change the pitch of any gravity flow piping under the landfill by up to 0.2 percent.

There are two leachate collection systems that will be impacted by the additional material. These consist of the single pipe that drains from Manhole MH F-2 to the east pump station and the D-series manholes that drain to the west pump station. The invert of MH F-2 was EL 13.0 in 1983, based on survey data by Duffield. The east pump station invert is about EL 5, based on the design drawings. This represents a pitch of about 0.45 percent. The total primary settlement estimated by Richardson for the existing conditions will cause the eastern half of this pipe to be lower than the invert. The settlement estimates for the new closure grades result in about one foot of additional settlement along this pipe. This will likely result in a negative slope over portions of this pipe, and a sag of two to three feet below the pump station invert sometime during the 30 year post closure monitoring period with or without the new material.

The D-series manholes that represent the west collection system have inverts as high as EL 36.6, based on survey data by Duffield. This implies that several of these manholes and the leachate pipes are founded within the waste. A settlement profile for this system under the existing conditions and proposed fill heights indicates that subgrade settlement will not cause the pipes to have a negative slope. The impacts of the waste settlement on the manholes will also not likely cause the average slope to become negative.

Although not affected by the proposed placement of additional fill, it should be noted that the northeast collection system also has leachate headers and manholes founded within the existing waste. The perimeter pump stations and leachate collection lines will not be affected by the proposed additional filling. Therefore, the additional fill material should not have a net impact on the functionality of the existing leachate collection system during the operating life and post closure monitoring period. Some of the existing manholes that are within the areas of new fill placement will require raising if they are not going to be abandoned.

The new fill and the final grading will greatly reduce the amount of infiltration into the existing waste, which will cut down the quantity of leachate. This will be a better condition than the flat top which currently allows very little run-off.

4.6 Impacts to Gas Collection System

The existing landfill gas collection system consists of a series of wells and surface pipes that go to a flare near the west pump station. The well heads and piping system will have to be raised in areas of new fill placement. The stabilized sludge or other approved fill materials will be inert and will not generate any landfill gas. Once the final cap is complete, it will reduce the available moisture which may cause a long term reduction in landfill gas production.

5.0 OPERATIONS PLAN AND FINAL CLOSURE GRADES

Camp Dresser & McKee, Inc., (CDM) under subcontract to us, has prepared a Contouring Plan for placement of stabilized sewage sludge or other DNREC approved materials at this site. The revised closure grades and filling sequence are based on their review of the pertinent site data and the geotechnical evaluation reported herein. The final closure grades and filling sequence were developed to achieve the required additional capacity without adversely affecting slope stability, the gas collection system, or the leachate collection system. The proposed closure grades should also reduce the amount of infiltration into the landfill and reduce the amount of leachate currently being pumped that is directly due to surface water infiltration.

The Contouring Plan addresses the issues of waste filling sequence, temporary and permanent haul roads, gas collection system, leachate collection system manholes, leachate observation wells, and stormwater control. This consists of five (5) phases, having a total capacity of about 1,500,000 tons of stabilized sludge. At an annual placement rate of 160,000 tons per year it is estimated to take about 9.4 years to completely bring the site to the final contours defined in the plan. The Contouring Plan is a 14 sheet set of drawings included as Appendix F.

6.0 CONCLUSIONS

Based on the subsurface exploration, the document review, and the slope stability analysis, the following conclusions and recommendations have been developed:

- The CPT, DMT, FVST, and soil laboratory tests conducted at this site indicate good correlation between the methods and provide greater confidence in the validity of the results. These results also indicate that the soils have undergone additional consolidation and strength gain over the 25 years since the original study.

- The dredge soils and recent alluvial deposits are believed to be essentially normally consolidated to slightly underconsolidated under the entire landfill footprint.

- The change in driving force is relatively small due to the additional stabilized sludge. The factors of safety for slope stability are estimated to be greater than 1.3 under short term loading conditions using total strength parameters, and greater than 2.0 using long term effective strength parameters, considering the added loading given the geometry restrictions discussed herein. As such, the proposed additional stabilized sludge material may be placed safely on top of the landfill.

- The final closure grades should be developed to accommodate the additional 1,300,000 cy of stabilized sludge or other DNREC approved materials, such that the exterior side slopes are not increased to steeper than 4H:1V (horizontal:vertical) where they are currently flatter. Where the existing slopes are currently steeper than 4H:1V, they do not need to be flattened out. The top can be graded with up to a five percent slope, provided the maximum height does not exceed about EL 90.

- The additional fill placement should not affect the current or future functionality of the existing leachate collection system. The existing manholes will require raising in areas where additional material is to be placed.

- The Contouring Plan was prepared by CDM and is included as Appendix F. The volume and life estimates do not consider any volume gained due to settlement of the existing waste mass due to the new loading.

- The existing gas collection system well heads and piping will need to be raised in areas where new fill will be placed.

7.0 LIMITATIONS

Recommendations contained in this report are based on data obtained from available site plans, site reconnaissance, previous reports and additional subsurface explorations performed as detailed herein. This report does not reflect variations between borings that may become evident by future exploration.

This report has been prepared to assist your office in the operation of this facility. It is intended for use with regard to the specific project described herein.

In the event that changes in the nature, design, or location of the facilities are planned, the conclusions and recommendations contained in this report should not be considered valid unless the changes are reviewed and conclusions of this report modified or verified in writing by us. SEA is not responsible for any claims, damages, or liability associated with interpretation of subsurface data or reuse of the subsurface data or engineering analyses without expressed written authorization.