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TOMOKA FARMS ROAD LANDFILL
NORTH CELL EXPANSION
GEOTECHNICAL INVESTIGATION

Prepared for

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SECTION 1
INTRODUCTION

SCS Engineers (SCS) has completed a subsurface geotechnical investigation of the North Cell Class I Landfill Expansion Area of the Tomoka Farms Road Landfill to identify and characterize the soil foundation. The investigation was conducted at the recommendation of SCS due to concerns about previous predictions of significant settlement at the site and the associated problems with liner and leachate collection systems stability and integrity.

A preliminary geotechnical analysis submitted to the Florida Department of Environmental Protection (FDEP) with the original construction permit application predicted a maximum settlement of 2.5 feet to 3.0 feet in the center of the expansion area. Settlement of this magnitude could jeopardize the integrity of the liner and leachate collection systems. However, this geotechnical analysis was done without the benefit of site-specific investigation of the North Cell area and was intentionally conservative. Slope stability was not addressed in the preliminary analysis.

For this geotechnical analysis, SCS reviewed previous site investigations conducted for the facility and the FDEP construction permit documents for the North Cell area. SCS then prepared a geotechnical investigation plan for the North Cell area to comply with the requirements of Rule 62-701.410, Florida Administrative Code (F.A.C), which addresses hydrogeological and geotechnical investigation requirements for landfills.

1.1 Site Description

The North Cell expansion area is directly north of the existing active Class I disposal area. Existing maintenance roads define the south and west sides of the cell. Drainage trenches define the north and east side of the cell. The area enclosed by the above boundaries is approximately 30 acres.

The North Cell was formerly used as a borrow source and was excavated approximately ten to fifteen feet below natural grade. Volusia County (County) has excavated and rough-graded the bottom of the cell to the elevations previously permitted.

Surrounding the cell are drainage trenches which, upon construction of the cell, will be used to control stormwater runoff. Currently the County pumps water from the trenches into a storage pond located east of the cell. Groundwater elevations across the cell are influenced by the pumping activities in the trenches. During the site investigation standing water was encountered in the approximate locations of the leachate laterals and sump areas.

1.2 Regulatory Requirements

Rule 62-701.410 (2) - (4), F.A.C, requires that a geotechnical investigation of a landfill site be conducted prior to construction. This geotechnical investigation defines the engineering properties of the site that are necessary for the design, construction, and support of the landfill leachate collection and removal system and the stability and integrity of the proposed liner system.

The following requirements of Section 62-701.410 have been previously addressed to the satisfaction of FDEP and conditions at the facility have not changed significantly so as to effect the information presented in the previous permit applications:

- Section 62-701.410 (2)b - Explore and address the presence of muck, previously filled areas, soft ground, lineaments, and sinkholes.

- Section 62-701.410 (2)c - Evaluate and address fault areas, seismic impact zones, and unstable areas.

- Section 62-701.410 (2)d - Estimate average and maximum high groundwater table across the site.

The groundwater levels are influenced by both on-site drainage ditches and pumping activities surrounding the expansion cell. For the purpose of this report the groundwater level was conservatively assumed to be approximately the same elevation as the excavation. The assumption is conservative because standing water, due to the clayey sand in the surficial stratum, was observed during the investigation but water levels in the adjacent trenches and borings were two to four feet lower than the bottom of the expansion area excavation.

After reviewing information submitted in prior permit applications for the facility, the following requirements of Rule 62-701.410 were specifically addressed in this geotechnical investigation:

- Section 62-701.410 (2)a - Explore and describe subsurface stratigraphy and groundwater table conditions.

- Section 62-701.410 (2)e - Include a foundation analysis to include foundation bearing capacity, subgrade settlement and subgrade slope stability.

1.3 Assumptions

For the geotechnical analysis, SCS has estimated that the compacted in-place waste density will be approximately 1,500 pounds per cubic yard (PCY) or 51.85 pounds per cubic foot (PCF). This density is a conservative estimate based on the current waste characteristics, equipment and operational procedures used at the facility, and published literature. Density information is included in Appendix A.

The elevations of the bottom liner system were taken from construction drawings for the North Cell. Final closure elevations were taken from the permitted closure elevations. The proposed maximum height of the North Cell is elevation 155.0 feet NGVD. At the center of the North Cell, the maximum depth of waste will be approximately 140 feet.

Using the above information, SCS has completed the geotechnical investigation of the site.

SECTION 2
SITE INVESTIGATION

In addition to previous subsurface geotechnical investigations preformed on the Tomoka Farms Road Landfill site, SCS conducted a detailed investigation within the proposed limits of the North Cell expansion. A detailed investigation was performed to establish subsurface soil stratigraphy and the engineering design parameters directly related to the expansion area.

2.1 Methods of Subsurface Investigation

A subsurface geotechnical investigation plan was prepared by SCS and carried out by Universal Engineering Sciences (Universal) from December 4 through 11, 1996. SCS observed drilling operations during the entire on-site investigation. Operational equipment for Universal included a drilling rig and a pickup truck equipped with a water tank. The drilling rig was capable of performing Standard Penetration Tests (SPT) and retrieving subsurface soil samples both with split spoon and Shelby tube samplers. SPT penetration resistance, or N-values, were recorded continuously for the upper ten feet and subsequently at every five foot increment of depth within the boring. A representative soil sample was retrieved at each increment using a two-inch diameter split spoon sampler and was containerized for laboratory analysis. A 3-inch diameter Shelby tube was used to collect undisturbed samples. Cuttings were removed from the borehole by flushing the borehole with water provided from the water truck and rotary drilling mud was circulated to stabilize the boring sidewalls. Upon completion of the subsurface investigation, all boreholes were grouted, from the boring terminus to natural grade, with a pumped mixture of portland cement and bentonite.

2.2 Soil Boring Locations

As part of the investigation, ten soil borings were drilled. The locations of the borings and site conditions at the time of the investigation are shown in Figure 2-1. The depth of individual borings are indicated in Table 2-1.

2.3 Soil Stratigraphy and Current Groundwater Tables

The subsurface stratigraphy beneath the North Cell expansion area is represented by two cross sections shown in Figure 2-2, based on information presented in the boring logs. The lines designating the interface between various stratum are approximate and based on split spoon samples retrieved at various depths within the boring. In addition, transitions between stratum are assumed to be gradual and therefore a single profile was compiled to reflect subsurface transitions across the site. Groundwater tables were measured in the borehole using a measuring rod. The soil boring logs and groundwater table depths at the time of the investigation were completed by Universal and are shown in Appendix B.

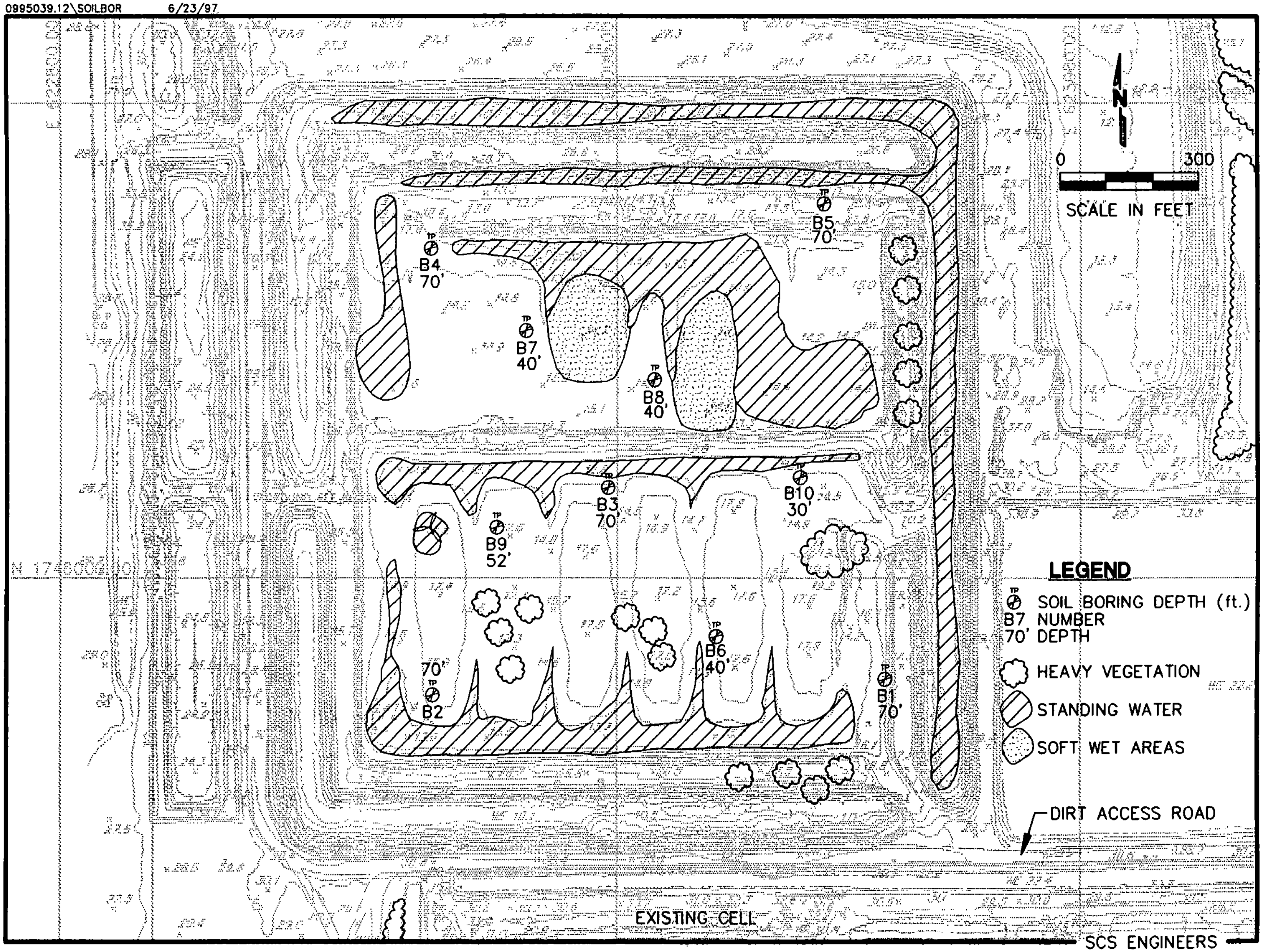

Figure 2–1. Boring Location Map (Site Conditions On 12/10/96).

TABLE 2-1 BORING DEPTHS

Boring Number	Approximate Ground Elevation (ft NGVD)	Depth of Boring (ft)	Approximate Boring Terminus Elevation (ft NGVD)
B-1	16.0	70.0	-54.0
B-2	16.0	70.0	-54.0
B-3	14.0	70.0	-56.0
B-4	14.0	70.0	-56.0
B-5	14.5	70.0	-55.5
B-6	16.0	40.0	-24.0
B-7	15.0	40.0	-25.0
B-8	15.0	40.0	-25.0
B-9	17.5	52.0	-34.5
B-10	14.0	30.0	-16.0

Note: NGVD - National Geodetic Vertical Datum

For the purpose of determining subsidence across the expansion area, a composite subsurface stratigraphy was compiled using information contained in the boring logs. The composite subsurface profile is shown in Figure 2-3. The soil borings generally encountered four separate strata. Limited variations in the thicknesses of different strata were encountered across the expansion area. From the ground surface to approximately ten feet in depth, a medium dense, silty to clayey sand was encountered. The second layer, approximately fifteen feet in thickness, consisted of a loose, poorly graded to silty sand with traces of shells. The third layer, approximately twenty feet in thickness, consisted of medium dense poorly graded sand with significant traces of shell fragments. The fourth layer, approximately twenty feet in thickness, consisted of dense poorly graded to silty sand. Approximately 65 to 70 feet below the surface, a very dense

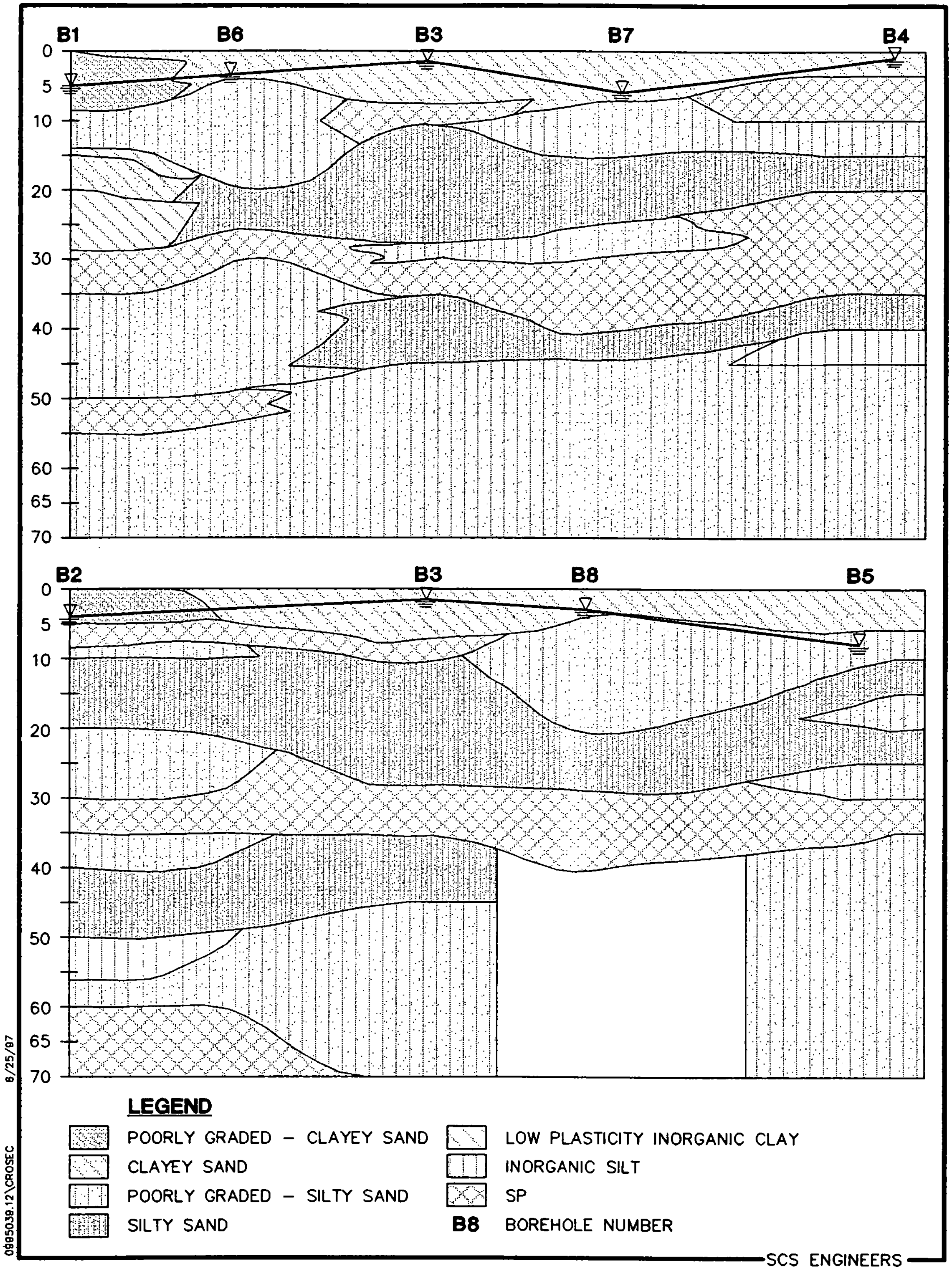

Figure 2-2. Subsurface Cross Section Profile.

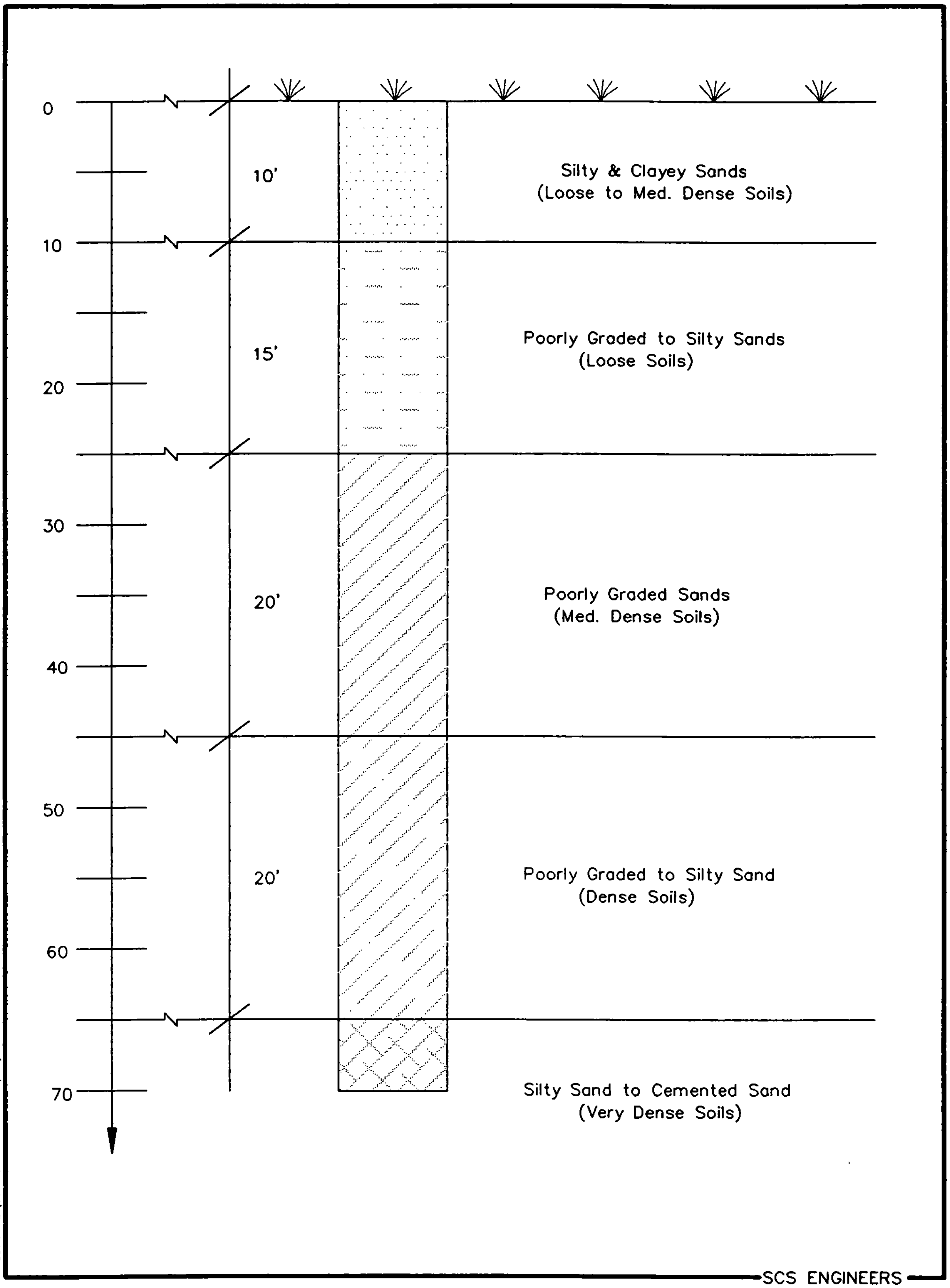

Figure 2–3. Composite Subsurface Profile.

cemented sand with shell fragments was encountered. Groundwater elevations, at the time of the investigation, varied from within two to seven feet below land surface.

2.4 Laboratory Analysis and Test Results

Split spoon samples were collected during the investigation and were visually field classified by Universal. The soil boring logs are presented in Appendix B. Containerized samples were tested by Universal according to schedule outlined in Tables 3-2 and 3-3.

Representative containerized soil samples were identified by SCS for further laboratory analysis. The analyses included determining grain size distributions and establishing Atterberg limits of potentially cohesive fine grained samples.

Results of the laboratory analysis were used to classify the soil according to the Unified Soil Classification System (USCS). SPT N-values and the soil classifications were used by SCS to estimate unit weights, in situ internal friction angles, and other engineering properties. Laboratory sample test results are presented in Appendix C.

During the investigation, several attempts were made by Universal to collect undisturbed samples using a 3-inch diameter Shelby tube. The undisturbed samples chosen by SCS for analysis were to be taken within the upper green-silty sand layer and the dense, gray-silty sand, at approximately 25 and 45 feet below land surface, respectively. Several tubes samples were lost upon removal from the borehole.

Table 2-2 . Tomoka Farms Road Landfill - North Cell Expansion

Sieve Analysis (ASTM D 2487)

Boring Number	Sample Number	Approximate Natural Ground Elevation (ft NGVD)	Depth of Sample (ft bls)	Approximate Elevation of Sample (ft NGVD)
B - 1	3	16.0	5	11.0
	8	16.0	20	- 4.0
	12	16.0	40	-24.0
B - 2	7	16.0	15	1.0
	10	16.0	30	-14.0
	13	16.0	45	-29.0
	15	16.0	55	-39.0
B - 3	8	14.0	20	-6.0
	9	14.0	25	-11.0
	10	14.0	30	-16.0
	12	14.0	40	-26.0
B - 4	8	14.0	20	-6.0
	13	14.0	45	-31.0
	16	14.0	60	-46.0
	18	14.0	70	-56.0
B - 5	15	14.5	55	-40.5
	16	14.5	60	-45.5
B - 6	11	16.0	35	-19.0
B - 7	6	15.0	10	5.0
B - 8	2	15.0	4	11.0
B - 9	8	17.5	20	-2.5
	14	17.5	50	-32.5
B - 10	7	14.0	15	-1.0

Table 2-3. Tomoka Farms Road Landfill - North Cell Expansion
Atterberg Limit (ASTM D 4318)

Boring Number	Sample Number	Natural Ground Elevation (ft NGVD)	Depth of Sample (ft bls)	Elevation of Sample (ft NGVD)
B - 3	9	14.0	25	-11.0
	12	14.0	40	-26.0
B - 4	8	14.0	20	-6.0
	13	14.0	45	-31.0
B - 9	8	17.5	20	-2.5
	14	17.5	50	-32.5

Note: bls - below land surface

Three tubes were successfully retrieved from the boreholes and sent to Universal for consolidation and triaxial strength tests. Upon extraction of the materials from the tubes, the tests were conducted but due to the poor quality of the samples and associated difficulty in testing, SCS did not consider the values obtained by testing to be representative of subsurface conditions.

SECTION 3
FOUNDATION ANALYSIS

SCS conducted foundation analysis that addressed both differential and total settlement, subgrade slope stability, and foundation bearing capacity. Analysis included short-term, end of construction, and long term stability and settlement. The following sections present the results of SCS's analysis of subgrade, liner system and slope stability.

3.1 Settlement

The rate of pore water dissipation determines when the majority of the settlement will occur within the North Cell expansion area. Soils with a high permeability dissipate pore water pressure faster than lower permeability soils. Sandy soils generally have high permeability and thus settlements occur just after the application of the overburden stress. Clayey soils generally have low permeability and settle, or consolidate, over a greater period of time. Secondary settlements are the result of continuing long term settlement.

A review of the soil boring logs and laboratory analysis for the subsurface investigation of the North Cell indicated the majority of the subsurface soils are poorly graded to silty sands. Immediate settlement of the site will predominate over consolidation and secondary settlement. Pore water pressure should dissipate rapidly with the application of waste material in North Cell. Therefore, only immediate settlement upon application of the waste was considered and the long term effects of consolidation and secondary settlement were considered negligible. The expected maximum subsidence will occur shortly after the final elevations are achieved.

3.1.1 Immediate Settlement --

SCS compiled the boring logs into a representative composite subsurface profile, as shown in Figure 2-3, and estimated the insitu dry unit weights. Once the dry unit weights of the strata were estimated, the initial void ratio was computed using the following equation:

$$\gamma_d = (G_s \gamma_w) / (1+e)$$

Where;

γ_d = Dry Unit Weight (lb/ft^3)

G_s = Specific Gravity

γ_w = Unit Weight of Water (lb/ft^3)

e = Void Ratio

In Appendix D similar soil types, void ratios and dry unit weights are shown.

To estimate the subsidence within the expansion area, the consolidation theory was used. The compression index, the change in void ratio with applied stress of the soil, was estimated using the soil type and density. Using the following equation and the estimated overburden stress induced by the overlying waste material, the settlement was computed in the middle and perimeter of the expansion area.

$$\Delta h = H\ C_c\ /(1+e_o)\ \log(1+(\Delta P/P_i))$$

Where;

Δh = Subsidence (ft)

H = Thickness of Layer (ft)

C_c = Compression Index

e_0 = Initial Void Ratio

ΔP = Change in Applied Stress (lb/ft^2)

P_i = Initial Stress (lb/ft^2)

Results from the settlement calculations indicate the greatest subsidence, approximately 8.9 inches, occurs approximately in the center portion of the expansion area. The highest elevations of the landfill and overburden stresses due to the overlying waste material occur in the center of expansion area. Thus larger settlement are expected in the vicinity of the center of highest stress. Around the perimeter, approximately 2.9 inches of settlement is expected. Appendix E contains the immediate settlement calculations. Figure 3-1 depicts the results of the settlement analysis across the North Cell.

3.1.2 Differential Settlement --

For long term integrity of the liner system and leachate collection system differential settlement across the site should be limited. Due to the loading conditions, subsidence within the expansion area will be uniform with the largest differential settlement occurring between the perimeter of the expansion area and the interior. A differential settlement of six inches over the length of the expansion area was considered by SCS to acceptable. After subsidence of approximately nine inches within the interior, a minimum elevation gradient of 0.35% will still be maintained.

3.2 Subgrade and Liner System Stability

Several subgrade stability failure modes for the North Cell area were investigated by SCS. The first mode of failure was a noncircular (wedge) failure analysis of the potential failure planes along the various geosynthetic components of the double composite liner system. The second mode of failure was a circular failure through the waste material. The third mode of failure was a circular failure propagating through the subsurface foundation stratum.

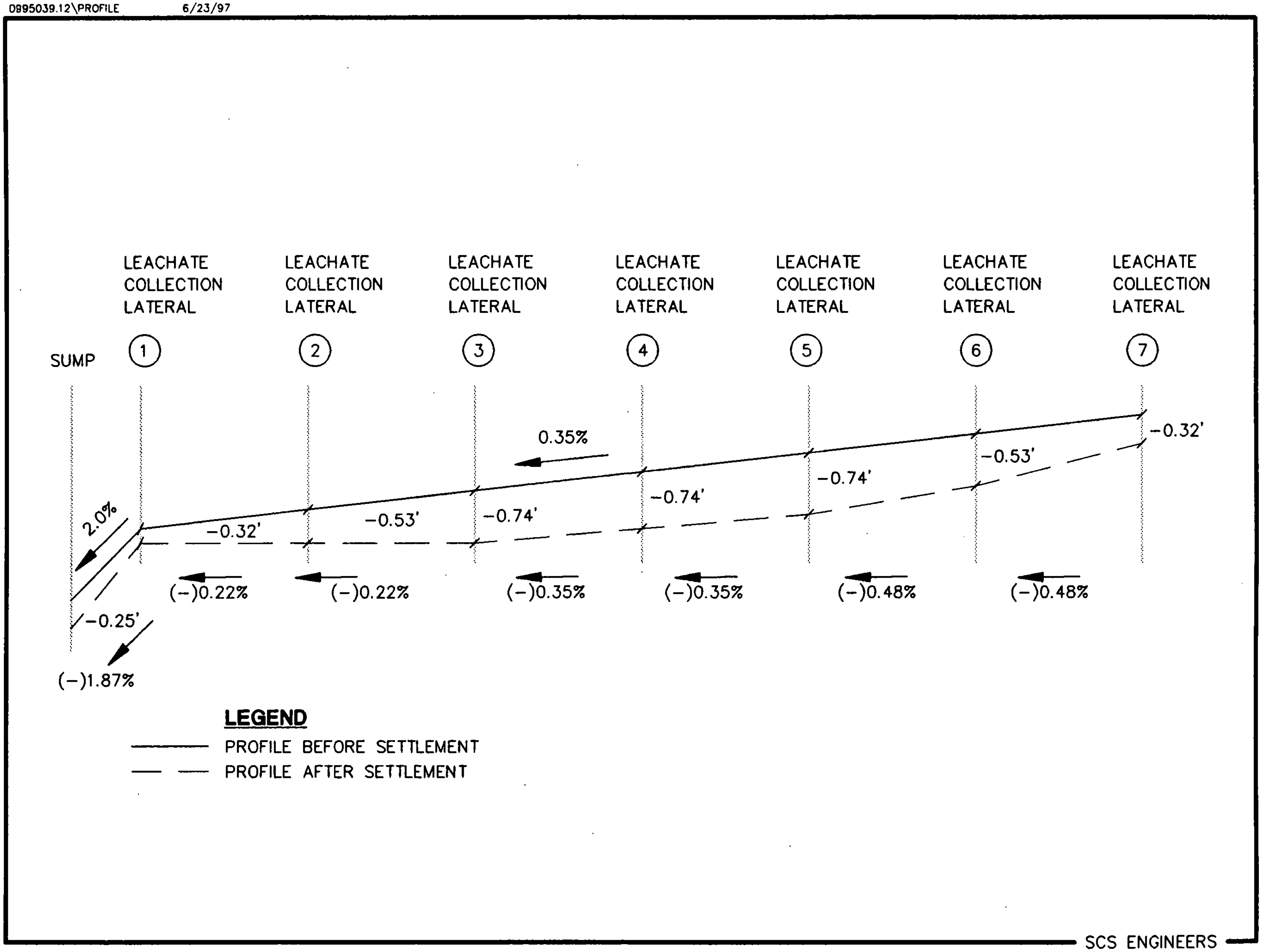

Figure 3-1. Settlement Profile of Collection Header.

For the analyses, SCS considered a minimum long term static stability of 1.5 or greater as an acceptable limit for the design.

All of the analyses were performed using published typical geosynthetic interface friction angles, hydrated internal friction angles for the geosynthetic clay liner (GCL), and typical soil and waste characteristics. The foundation soil will increase in strength as the subsurface is compacted and the lower loose soils begin to consolidate and densify.

3.2.1 Method of Analysis for Foundation Stability (Non Circular and Circular) --

Foundation stability analysis was performed using the computer program PCSTABL5M. Program input parameters included:

- Geotechnical description (including unit weight, cohesion, and angle of internal friction (ϕ) of the municipal solid waste (MSW), geomembrane components of the landfill, and the foundation strata.

- Landfill Geometric Data.

- Subsurface (Foundation) Strata Profile Data.

- Piezometric Surface Data.

The above information was input into the computer program, and the computer used a search routine to find the worst-case failure plane for the input conditions.

3.2.2 Input Parameters used in Analyses (Non Circular and Circular) --

Municipal Solid Waste --

Strength parameters were selected based upon review of numerous reports included in the ASTM text titled "Geotechnics of Waste Fills, Theory and Practice" (Landva, Knowles. 1990). Specifically, the report titled "Evaluation of the Stability of Sanitary Landfills" (Singh and Murphy) addresses predicted strength parameters for MSW based upon laboratory test data and back-calculation of field tests and operational records. A copy of the applicable sections of the report are included in Appendix F.

Based upon the information presented in the referenced report, SCS selected a conservative strength property for the MSW. The following number were used to model the waste material:

- internal friction angle of waste = 25 degrees.

- cohesion of waste = 0 psf. (Typically this value is assumed to be zero to be conservative).

Interface Between Geosynthetic Layers of Bottom Liner System --

For the analysis, SCS reviewed published interface friction angles for the various geosynthetic components of the bottom liner system. Sources included manufacturer material data, professional experience, and information presented in the text titled "Designing with Geosynthetics" (Koerner, 1990). Various values of interface friction angles between the geosynthetics and internal friction angles of hydrated GCL are presented in Appendix G.

The following values were used to model the various geosynthetic interfaces:

- internal friction angle of hydrated unreinforced GCL = 4 degrees. This value is based on the available published data.

- internal friction angle of hydrated reinforced GCL = 16 degrees. This value is based on the available published data.

- cohesion of GCL (unreinforced or reinforced) = 0 psf. Typically this value is assumed to be zero to be conservative.

The following values were used to model the various Interface Shear Strengths (Adhesion assumed to be zero in all cases):

- interface friction angle between geocomposite/textured HDPE = 30 degrees.

- interface friction angle between non-woven geotextile/sand = 21 degrees.

- interface friction angle between smooth HDPE FML/GCL = 14 degrees.

- interface friction angle between textured HDPE FML/GCL = 22 degrees.

- interface friction angle between textured HDPE FML/sand = 25 degrees.

The critical interface friction angles between various geosynthetic materials and/or soil layers were estimated based on our experience and knowledge of the typical soil/geosynthetic characteristics. However the internal or the interface friction angles are product dependent and should be verified by testing of the actual interface materials prior to start of construction (or once the project-specific materials are known). It is recommended that the geosynthetic/soil or geosynthetic/geosynthetic interface direct shear test be performed on the anticipated low valued interfaces, using project-specific materials. The test procedure used should be in accordance with ASTM D 5321, "Determining the Coefficient of Geosynthetic/Geosynthetic and Soil/Geosynthetic Friction by the Direct Shear Method".

Foundation Strata Profile Data --

The geotechnical engineering properties of the various subsurface strata underlying the North Cell landfill foundation were estimated from SPT blow counts and published references. The representative composite profile of the foundation is shown in Figure 2-3.

SCS selected a typical cohesionless strength property for the underlying sandy foundation soils. The internal friction angle of foundation soil below liner system used in the slope stability model was 28 degrees.

Landfill Geometric Data --

For stability of the design, representative fill heights were analyzed for various phases of operation. For long term stability, final proposed operational grades of North Cell expansion area were used from the permitted operational plans. Final and bottom elevations of the expansion are presented in Appendix G and Figure 3-1.

Piezometric Line Data --

Piezometric line data representing a water table at the most conservative location is approximately equal to the ground surface of the existing excavation. This elevation represents a conservative estimate for rotational failure through the landfill foundation subgrade.

3.2.3 Non Circular Stability of Liner System --

Critical Failure Plane --

To determine the stability of the double liner system, the failure plane was modeled to pass parallel to the liner system. This failure plane determined the critical interface friction angle required between geosynthetic components to maintain stability. Both the input and output files for the critical failure mode are included in Appendix G.

Results of Analysis --

The results of the slope stability analysis using PCSTABL5M computer program are summarized below:

SCS first investigated if unreinforced GCL could be used as a part of the sideslope containment system. Assuming a critical internal friction angle of 4 degrees for the hydrated unreinforced GCL, the factor of safety of 0.9 was calculated using a block-type failure surface within the GCL. This is an unstable condition, so SCS then investigated using reinforced GCL on the sideslope and extending this system into the expansion area for greater stability. The factor of safety was increased to 1.5 if at least 260 feet length (measured perpendicularly from the base of the perimeter berm toward the landfilling area) of reinforced GCL is installed. This reinforced GCL was assumed to have an internal friction angle of 16 degrees.

Proposed Double Liner System --

Based on the results of the analyses, SCS minimized weak interface friction angles, that is, any interface which has a friction angle of less than 16 degrees within the 260 feet zone from the toe of the perimeter berm. The minimum interface or internal friction angle of the individual geosynthetic components used bottom liner system should be at a minimum 16 degrees to provide a factor of safety of at least 1.5. The proposed double liner system is shown in Figure 3-2.

Proposed Liner System

The proposed liner system on the side slope consists of the following layers (from bottom to top):

- Compacted subgrade;
- Reinforced Secondary Geosynthetic Clay Liner (GCL);
- 60-mil High Density Polyethylene (HDPE) textured Secondary Geomembrane - textured on both sides;
- A Tri-Planar Geocomposite (thermally bonded 6-oz non-woven geotextile to a tri-planar geonet);
- Reinforced Primary GCL;
- 60-mil HDPE textured Primary Geomembrane - textured on both sides;
- Geocomposite Drainage layer; and
- 2 feet of Drainage Sand.

The proposed liner system in the interior of the landfill consists of the following layers (from bottom to top):

- Compacted subgrade;

- Unreinforced Secondary Geosynthetic Clay Liner (GCL) - needled punched;
- 60-mil HDPE Secondary Liner (smooth);
- A Tri-Planar Geocomposite (thermally bonded 6-oz non-woven geotextile to a tri-planar geonet);
- Unreinforced Primary GCL;
- 60-mil HDPE Primary Liner (smooth);
- Geocomposite Drainage layer; and
- 2-feet of Drainage Sand.

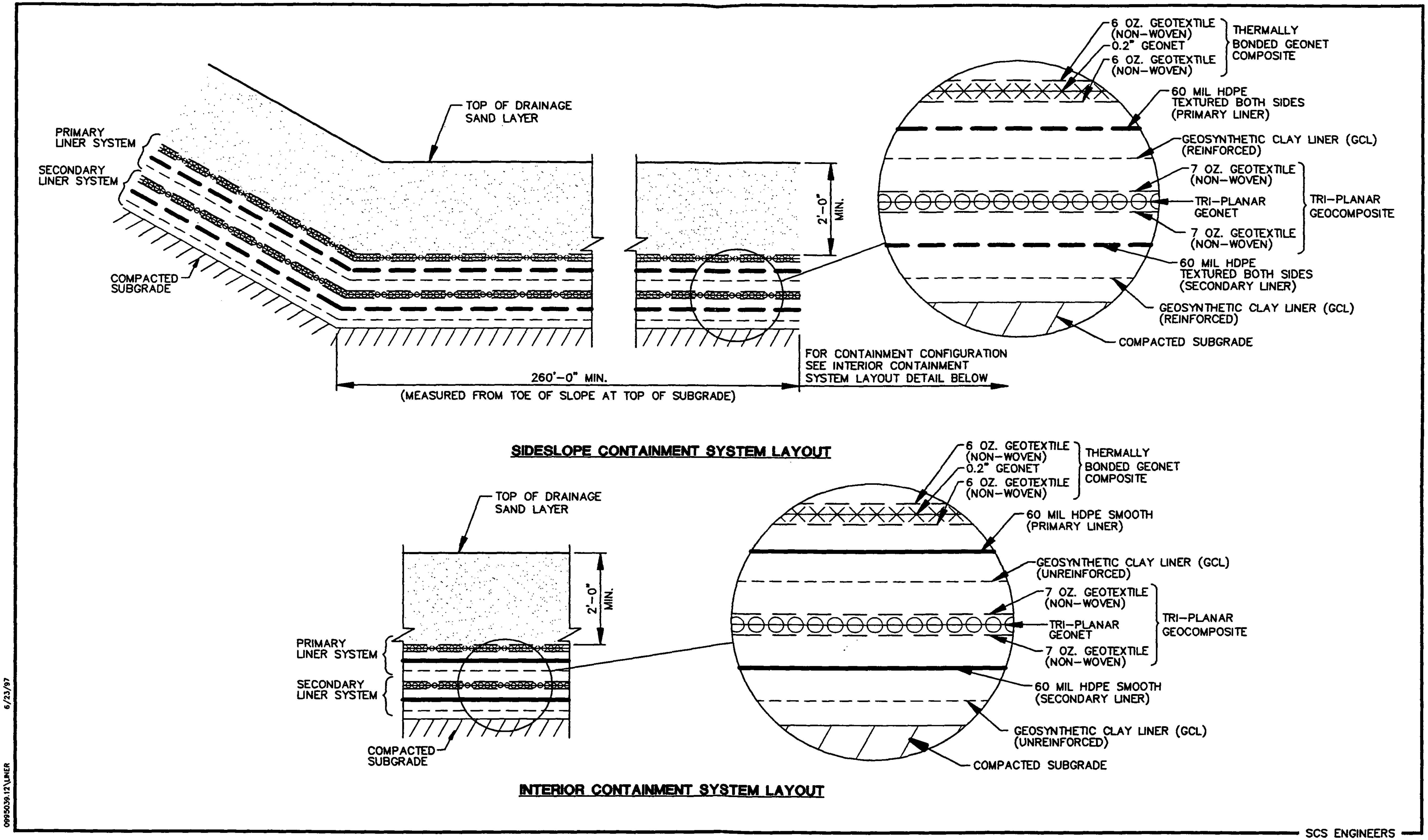

Figure 3-2. Proposed Liner System.

3.2.4 Circular Sideslope and Landfill Foundation Stability --

Critical Failure Plane --

A circular search routine was used to search for the stability of the sideslope and a wedge failure plane analysis was used to evaluate the foundational stability of the expansion area. The circular routine searched through the waste material above the liner system and the failure plane analysis analyzed failure propagating through the foundation subgrade.

When a circular-type toe failure surface is assumed, a factor of safety of 1.9 was computed, using at least 260 feet of reinforced GCL measured from the inner toe of the perimeter berm. The failure surface was found to propagate within the waste material and close to the final surface of the side slope.

When a deep-seated, block-type failure mode is analyzed within the underlying foundation soils below the liner system, a factor of safety equal to 2.5 is calculated. This indicates that the foundation soils are stable under the proposed landfill loading conditions and thus a deep-seated block-type failure poses minimal risk.

3.3 <u>Bearing Capacity</u>

Definition of Bearing Capacity --

SCS computed the bearing capacity of the site by determining the net allowable increase in the applied loading on the landfill subbase that would cause critical design limitations to be exceeded. The net allowable increase in pressure was defined as the difference between the applied pressure, causing design failure, and the proposed permitted applied pressure. Using 1,500 PCY as the estimated MSW unit weight and a maximum permitted landfill elevation of + 155 feet NGVD, the maximum applied loading exerted on the bottom of the landfill will occur in the middle interior of the North Cell.

Approximately 7778 pounds per square foot is estimated to be exerted on the interior bottom of the expansion area. The critical design limitation identified by SCS was excessive settlement of the collection pipes so as to limit the scouring velocity below 2 feet per second.

Results of Bearing Capacity Analysis

An increase in the waste unit weight, from 1,500 to 2,604 pounds per cubic yard, increased the applied loading of the foundation sufficient to cause velocities within the collection system to approach the minimum scour velocity.

An increase in the unit weight of the waste material from 1,500 to 2,604 pounds per cubic yard is a net increase of approximately 1,103 pounds per cubic yard. At an estimated depth of 140 feet, the total bearing capacity of the site was conservatively estimated to be 13,500 pounds per square foot, leaving a net bearing capacity of 5,700 pounds per square foot. Refer to Appendix H for calculations.

SECTION 4
ADDITIONAL ANALYSES

4.1 Dewatering Plan

During the site investigation, SCS noted that the ground water levels were within two to seven feet of the surface of the bottom of the expansion area. Groundwater levels were elevated in the central interior of the expansion area. During the drilling of borehole B-3, the clayey sandy soils exhibited significant pumping action due to the high water table and equipment loadings. During construction of the expansion area, this pumping action will inhibit compaction efforts and the expelled water from compaction will migrate toward the surface. Water at or near the surface of the subgrade will hydrate geocomposite clay liner prior to installation. The hydrated GCL will swell without confining pressure and the permeability of the GCL will be increased.

Based upon field observations, such as standing surficial water, pumping action of the subgrade, soil classifications, and water levels in the boreholes and adjacent stormwater ditches, SCS has developed a preliminary dewatering plan, for County use only, to be implemented prior to and during construction. Figures 4-1 through Figure 4-4 outline the proposed dewatering plan for construction of Phases I through V. During construction, modifications to the dewatering plan can be made based upon actual field observations and construction scheduling.

Figures 4-1 through 4-4 delineate construction of three dewatering trenches, in the same location as the collection headers, which will serve as static dewatering ditches. The ditch bottoms should be excavated below the surficial clayey sand layer, approximately 5 to 7 feet below the existing natural grade, to promote collection of water. The vacuum assisted dewatering pump can be rotated from the three ditches, with collected water be pumped into the adjacent stormwater channels. When the ditches are to be filled in, the well point dewatering systems should be activated until the ditch is dry and then backfilled with the excavated clayey sand soils and compacted in lifts as outlined Section 4.2.

Well point dewatering is recommended prior to construction so that water levels can be lowered rapidly and stabilized below the surrounding water elevations. Dewatering in the area of the sumps will require the water elevations to be lowered below elevation +6.0 feet NGVD or approximately nine to eleven feet below the bottom of the excavated cell. Dewatering efforts should proceed prior to compaction of the subgrade through completion of the liner system to prevent hydration of the GCL and uplift forces. Upon completion of the liner system, the sump area should be immediately backfilled with a minimum of 2 to 3 feet of drainage sand to prevent uplift and unconfined hydration of the GCL. The well point system in the interior portion of the expansion area is recommended so that water levels and potential pumping actions in the subgrade are minimized during compaction and liner system installation. The well screen should be set below the clayey sand layers and preferably into the loose brown sands, approximately 7 to 10 feet below the existing surface of the expansion area, as shown in borehole number B-3 and B-10.

A dewatering pump placed along the eastern embankment of the expansion area is recommended so water levels within the stormwater channel can be lowered, preferably below elevation + 12.0 feet, and pumped into the stormwater pond directly east of the expansion area. A siltation basin and floating turbidity boom is recommended to avoid silt settlement within, and discharging from, the pond.

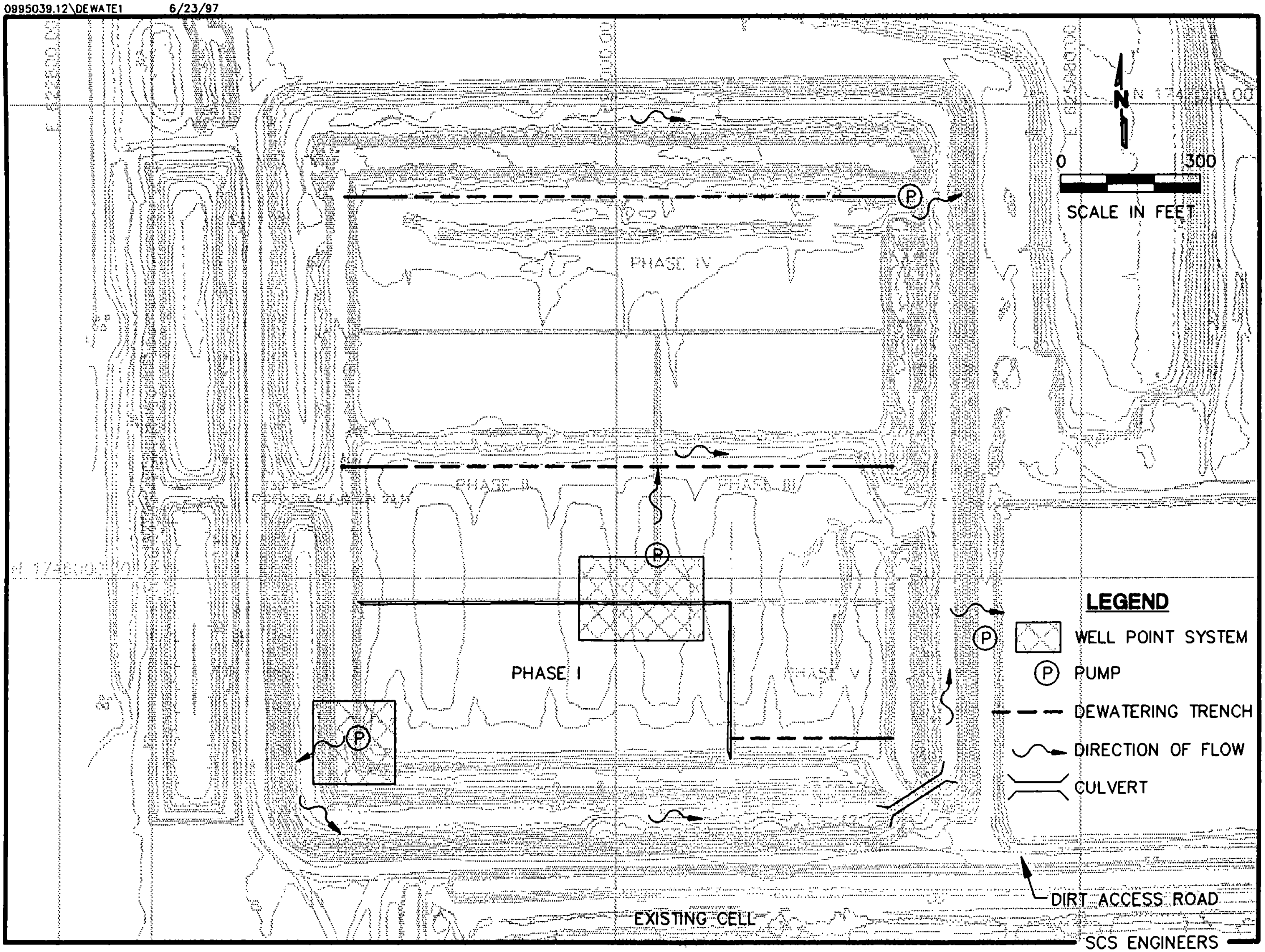

Figure 4–1. Step I Dewatering Plan (Site Conditions On 12/10/96).

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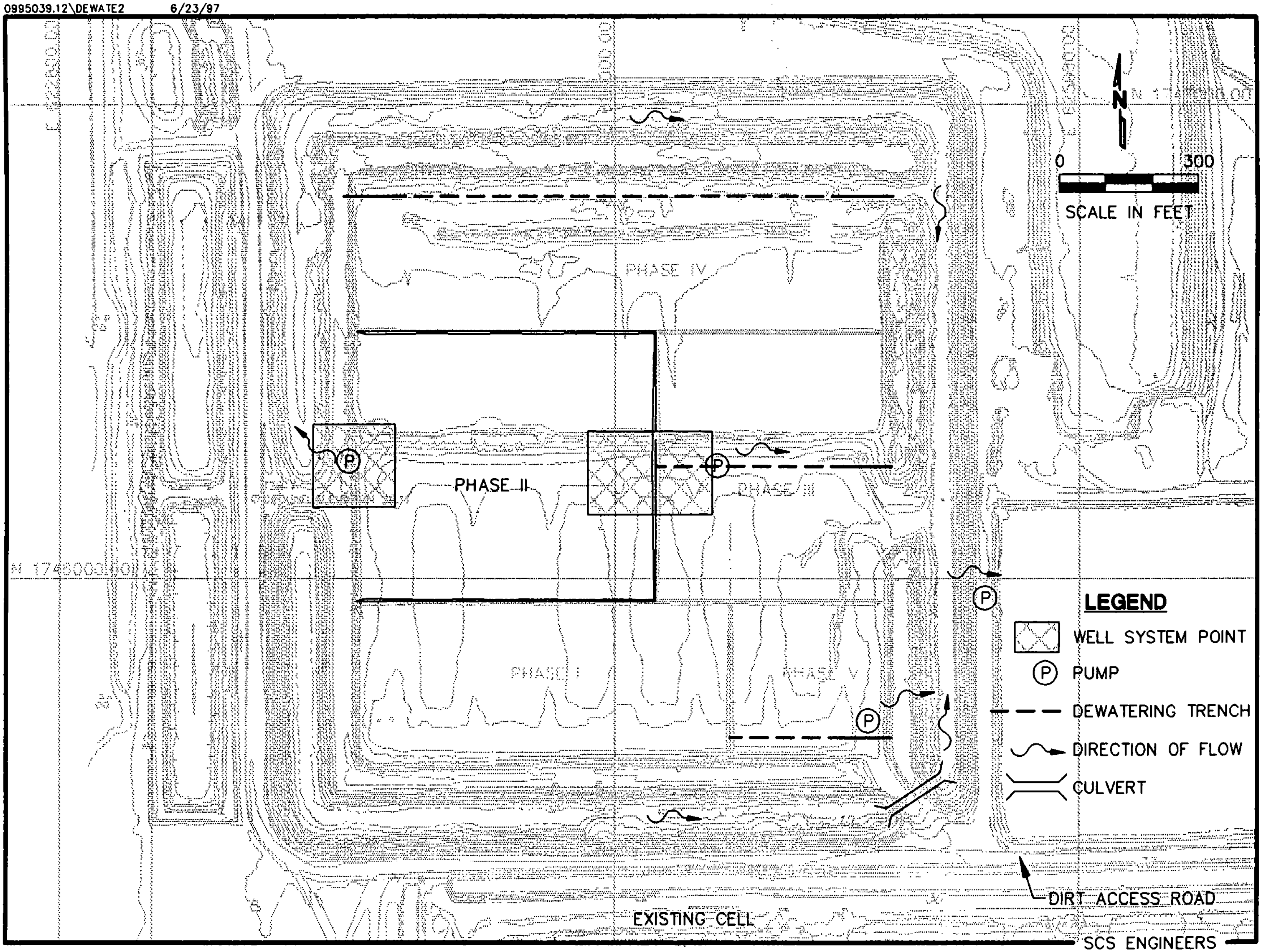

Figure 4–2. Step II Dewatering Plan (Site Conditions On 12/10/96).

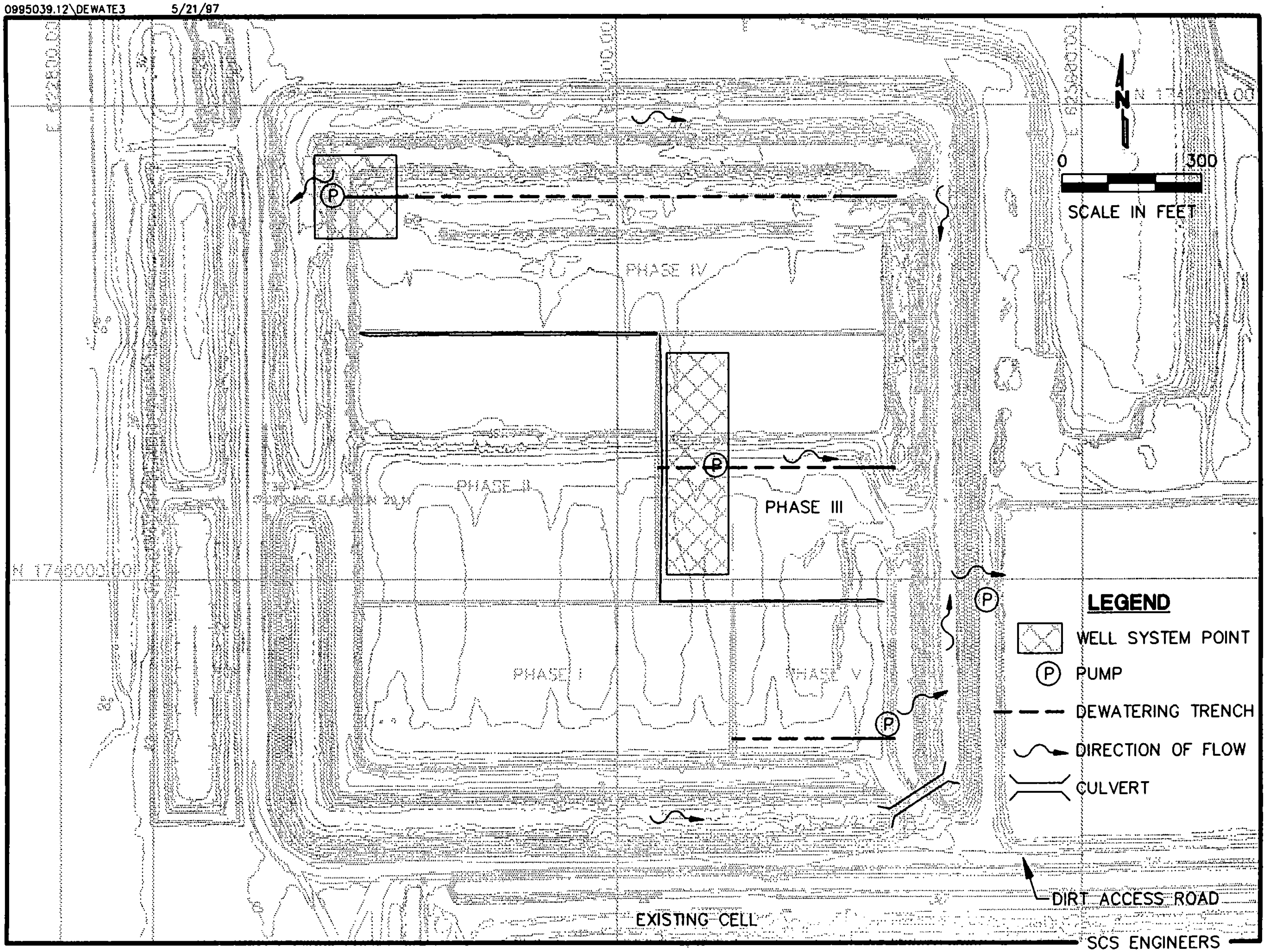

Figure 4–3. Step III Dewatering Plan (Site Conditions On 12/10/96).

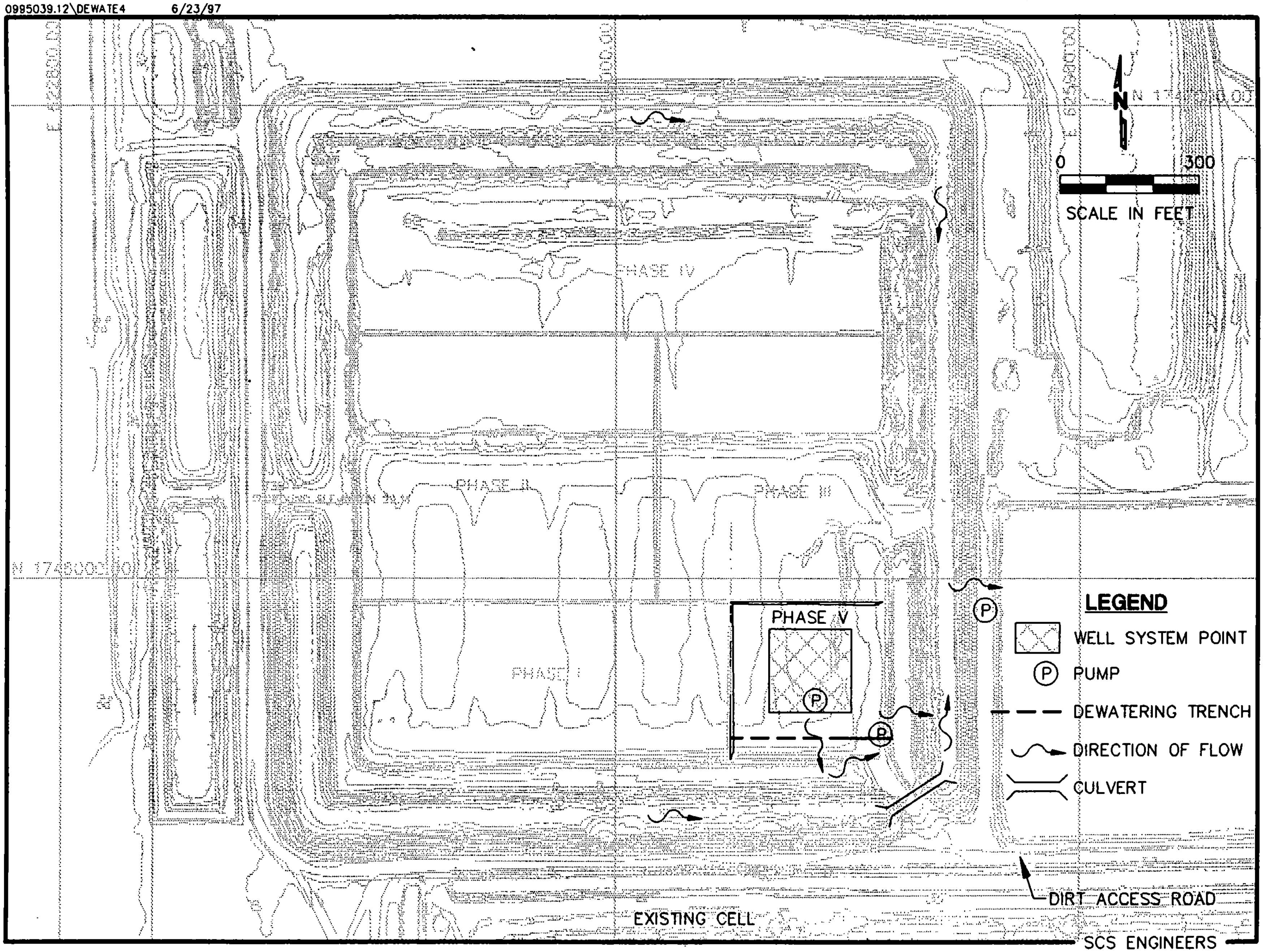

Figure 4–4. Step IV Dewatering Plan (Site Conditions On 12/10/96).

4.2 Subgrade Compaction

As part of the preparation of the subgrade prior to placement of the secondary geosynthetic clay liner, in conjunction with the dewatering plan, SCS recommends the following guidelines;

1) Strip existing vegetation and organic debris, approximately the upper six inches to one foot from the surface of the existing area.

2) Proof roll the stripped surface with a vibratory rubber tired roller, at a slow speed (3-4 mile per hour), a minimum of four to six passes. The roller should have a minimum weight of 10 tons with a vibratory frequency between 25 to 30 hertz (Hz). Upon proof rolling, soft areas and areas with excessive settlement should be overexcavated and backfilled with clean fill. Borehole numbers B-3 and B-8 may contain soft subgrade layers.

3) Place additional clean fill, preferably a clayey sand, in a maximum of 8-inch loose lifts. Spread and mix the fill with a tracked dozer and compact with a rubber tired vibratory roller a minimum of four to six passes at 3 to 4 miles per hour. The density of the subgrade should be compacted to a minimum of 95 percent of the Standard Proctor and from approximately -2 to + 5 percent of the optimum moisture content.

4) Repeat Step 3 until a uniform, smooth subgrade is brought to the elevations shown on the permit drawings.

SECTION 5
RECOMMENDATIONS

Based on the results of this investigation, SCS recommends the following:

1. The leachate collection system and bottom liner system should be constructed to the grades described in Section 3 of this report to allow for proper functioning after the anticipated subgrade settlement.

2. The liner system should be constructed as described in Section 3 of this report to provide the required interface friction angles and foundation slope stability.

3. The dewatering plan described in Section 4 of this report should be used as a guide to provide a subgrade that is suitable for construction.

4. The recommendations in Section 4 of this report for subgrade compaction should be followed to achieve the desired subgrade compaction.

APPENDIX A

TYPICAL MUNICIPAL SOLID WASTE DENSITY INFORMATION

SHEET 1 of 5

GEOTECHNOLOGY of Waste Management

Issa S. Oweis . Raj P. Khera

Butterworths

"GEOTECHNOLOGY OF WASTE MANAGEMENT"

SHEET 2 of 5

Table 4.9 Unit weights for landfill materials

Description and state	Total unit weight lb/ft³	kN/m³
Municipal waste		
Poor compaction	18–20	2.8–3.1
Moderate to good compaction	30–40	4.7–6.3
Good to excellent compaction	55–60	8.6–9.4
Baled waste	37–67	5.5–10.5
Shredded and compacted	41–67	6.4–10.5
In situ density	35–44	5.5–6.9
Active landfill with leachate mound	42	6.6
North-east US active landfill	30–40	4.6–6.3
Incinerator residue		
Poorly burnt	46	7.2
Intermediate burnt	75	11.8
Well burnt	81	12.6
Ashes	41–52	6.4–8.2
Hazardous waste landfill site		
75 ft deep waste with soil cover	101	15.9
40–50 ft deep dry dust and soil	30–110	4.6–17.3
62 ft deep waste average		
Kiln dust, sludge tar, creosote and soil	73	11.5
Dust	46	7.2
Tars	104	16.3
Contaminated soils	69	10.8
75 ft deep chemical solutions and scrap metals mixed with contaminated soil	63–74	9.9–11.6
30–40 ft deep landfill with 90–95% waste in metal drums	90	14.1

Avg 50 lb/ft³
1350 lb/cy

AVG
800 – 1100 lb/cy

Sources: Bromwell, 1978; Collins, 1977; Oweis and Khera, 1986; Peirce *et al.*, 1986; Sargunan *et al.*, 1986; Shoemaker, 1972; Sowers, 1973; Tchobanoglous *et al.*, 1977.

4.5.2 Field test sections

This technique requires an area about 300 ft long and 150 ft wide to be filled with refuse using usual daily field parameters (layer thickness, compaction equipment and passes). The weight of refuse is determined from the number of trucks delivering the refuse. The volume is determined by optical surveys of dimensions.

This technique is reasonably reliable for assessing the unit weight before advanced decomposition. The results are typically 35–45 lb/ft³ (5.5–7.1 kN/m³) for average compaction.

4.5.3 Refuse inventories

For a landfill where good records are kept, the weight of refuse dumped can be reasonably assessed. The volume is determined from optical surveys and approximate estimates of the settlement of the foundation where it comprises compressible materials.

STP 1070

Geotechnics of Waste Fills—Theory and Practice

Arvid Landva and G. David Knowles, editors

ASTM
1916 Race Street
Philadelphia, PA 19103

SHEET 3 of 5

Mass densities or unit weights of natural soils are readily obtained from relatively undisturbed soil samples. On the other hand mass densities or unit weights of refuse fills are not as easily determined because they must be indirectly estimated from weigh station records and volume changes of the landfill over a given period of time. The volume changes can be estimated from historic topographic relief maps of the landfill surface that are periodically prepared by land surveyors. The average unit weight of the compacted refuse placed at the Richmond landfill was estimated to be 46 pcf (736.9 kg/m³). Although, the unit weight of refuse fill varies considerably this value is in reasonable agreement with typical values reported by others as summarized in Table 3.

Geotechnic of waste fills - Theory & Practice

TABLE 3 -- Refuse Fill Average Unit Weights

SOURCE	REFUSE PLACEMENT CONDITIONS	UNIT WEIGHT (kg/m³)	UNIT WEIGHT (pcf)	
NAVFAC [5]	Sanitary Landfill			
	a) Not Shredded			
	· Poor Compaction	320	20	1080 lb/cy
	· Good Compaction	641	40	
	· Best Compaction	961	60	1620 lb/cy
	b) Shredded	881	55	
Sowers [6]	Sanitary Refuse: Depending on Compaction Effort	481-961	30-60	
NSWMA [7]	Municipal Refuse:			
	· In a landfill	705-769	44-49	
	· After Degradation and Settlement	1009-1121	63-70	
Landva and Clark[a] [8]	Refuse Landfill (Refuse to soil cover ratio varied from about 2:1 to 10:1)	913-1346	57-84	
EMCON Associates[b] [9]	For 6:1 refuse to daily cover soil	737	46	

a These values were obtained from test pit measurements of refuse at eleven municipal landfills located in Canada. Values measured for the Halifax landfill and the August 1983 measurements at the Edmonton and Calgary landfills have not been included, as suggested by the authors.

b Based on tonnage records and areal survey maps recorded during the period from April 1988 through April 1989.

SHEET 4 of 5

SCS ENGINEERS

CLIENT	PROJECT		JOB NO.
Volusia County			
SUBJECT		BY	DATE
Estimated Waste Density			
		CHECKED	DATE

From "Geotechnology of Waste Management"

assume good to excellent compaction — 1,350 lb/cy

(conservative for geotechnical calculations)

From "Geotechnics of Waste Fills - Theory and Practice"

Best compaction — 1,620 lb/cy

(conservative weight calculation)

For report use $\left(\frac{1{,}350 + 1{,}620}{2}\right)$

$= 1{,}485$ lb/cy

density of waste $\gamma_{waste} = 1{,}500$ lb/cy

APPENDIX B

SOIL BORING LOGS

B-8
is not
same as
MW. B-8

UNIVERSAL ENGINEERING SCIENCES
BORING LOG

PROJECT NO.:	41011-100-01
REPORT NO.:	56732
APPENDIX:	A- 1

PROJECT: GEOTECHNICAL EVALUATION
TOMOKA LANDFILL NORTH CELL EXPANSION
VOLUSIA COUNTY, FLORIDA

BORING DESIGNATION: **B-1**   SHEET: **1 of 2**

CLIENT: VOLUSIA COUNTY
LOCATION: AS DIRECTED BY SCS ENGINEERS
REMARKS: * = INDICATES 6" OF TRAVEL BY WEIGHT OF HAMMER

G.S. ELEVATION (ft):
WATER TABLE (ft): 5.0
DATE OF READING: 12/4/96
EST. W.S.W.T. (ft):

DATE STARTED: 12/4/96
DATE FINISHED: 12/5/96
DRILLED BY: U.E.S. DRILLING
TYPE OF SAMPLING: ASTM D-1586

DEPTH (FT.)	SAMPLE	BLOWS PER 6" INCREMENT	N (BLOWS/ FT.)	W.T.	SYMBOL	DESCRIPTION	-200 (%)	MC (%)	ATTERBERG LIMITS LL	ATTERBERG LIMITS PI	K (FT./ DAY)	ORG. CONT. (%)
0						Medium dense brown SAND with CLAY (SP-SC)						
	X	4-7-9	16									
	X	11-12-13	25			—Grayish brown	9.8					
5	X	9-11-16	27	▼								
	X	7-8-10	18									
	X	5-6-6	12			Medium dense to loose grayish brown SAND with traces of SILT (SP-SM)						
10	X	3-4-4	8									
						Soft dark gray CLAY (CL)						
15	X	2-2-2	4			Loose gray SAND and SILT (SP-SM)						
						Very soft dark gray CLAY with SAND (SC)	67.8					
20	X	1-*-2	2									
						Stiff dark gray CLAY with some SHELL (CL)						
25	X	2-6-3	9									
						Dense gray SAND and SHELL (SP)						
30	X	12-16-21	37									
35	X	14-17-22	39									
						Dense gray SAND with SHELL and SILT (SP-SM)	12.8					
40	X	11-14-18	32									
						Medium dense to loose dark gray SILT and SAND with a trace of SHELL (SP-SM)						
45	X	6-5-6	11									
50	X	5-5-4	9									

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UNIVERSAL ENGINEERING SCIENCES
BORING LOG

PROJECT NO.:	41011-100-01
REPORT NO.:	56732
APPENDIX:	A- 1

PROJECT: GEOTECHNICAL EVALUATION
TOMOKA LANDFILL NORTH CELL EXPANSION
VOLUSIA COUNTY, FLORIDA

BORING DESIGNATION: **B-1**

SHEET: **2 of 2**

DEPTH (FT.)	SAMPLE	BLOWS PER 6" INCREMENT	N (BLOWS/ FT.)	W.T.	SYMBOL	DESCRIPTION	-200 (%)	MC (%)	ATTERBERG LIMITS LL	ATTERBERG LIMITS PI	K (FT./ DAY)	ORG. CONT. (%)
50						Medium dense grayish brown SAND (SP)						
55	X	10-14-12	26			Medium dense gray SAND and SHELL with a trace of CEMENTED SAND and SILT (SP-SM)						
60	X	8-7-8	15			Medium dense light brownish gray SAND and SHELL (SP)						
65	X	9-11-12	23			Loose gray SAND with SILT and a trace of SHELL (SP-SM)						
70	X	4-4-5	9			END OF BORING: 70.0'						
75												
80												
85												
90												
95												
100												

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UNIVERSAL ENGINEERING SCIENCES
BORING LOG

PROJECT NO.:	41011-100-01
REPORT NO.:	56732
APPENDIX:	A- 2

PROJECT: GEOTECHNICAL EVALUATION
TOMOKA LANDFILL NORTH CELL EXPANSION
VOLUSIA COUNTY, FLORIDA

BORING DESIGNATION: **B-2** SHEET: **1 of 2**

CLIENT: VOLUSIA COUNTY
LOCATION: AS DIRECTED BY SCS ENGINEERS
REMARKS: * = INDICATES 6" OF TRAVEL BY WEIGHT OF HAMMER

G.S. ELEVATION (ft):
WATER TABLE (ft): 4.0
DATE OF READING: 12/4/96
EST. W.S.W.T. (ft):

DATE STARTED: 12/4/96
DATE FINISHED: 12/4/96
DRILLED BY: U.E.S. DRILLING
TYPE OF SAMPLING: ASTM D-1586

DEPTH (FT.)	SAMPLE	BLOWS PER 6" INCREMENT	N (BLOWS/FT.)	W.T.	SYMBOL	DESCRIPTION	-200 (%)	MC (%)	ATTERBERG LIMITS LL	PI	K (FT./DAY)	ORG. CONT. (%)
0						Loose to medium dense brownish gray SAND with CLAY (SP-SC)						
	X	2-3-3	6									
	X	5-6-5	11	▼								
5	X	4-7-7	14			Medium dense light brown SAND (SP) —Brown to light brown						
	X	5-6-7	13									
	X	4-5-5	10			Loose gray to dark gray SAND with SILT (SP-SM)						
10	X	2-3-3	6									
						Very loose dark gray SILTY SAND (SM)	37.7					
15	X	1-1-2	3									
20	X	5-3-*	3									
						Loose dark gray SILT with SAND and SHELL (SP-SM)						
25	X	1-1-6	7									
						Dense gray SAND with a trace of SHELL and SILT (SP-SM)	7.7					
30	X	15-21-20	41									
						Medium dense dark gray SAND and SHELL (SP)						
35	X	10-10-13	23									
						Medium dense dark gray SAND with SILT and SHELL (SP-SM)						
40	X	6-9-8	17									
						Medium dense dark gray SILTY SAND with SHELL and CEMENTED SAND (SM)	31.6					
45	X	7-6-6	12									
50	X	5-5-7	12									

UNIVERSAL ENGINEERING SCIENCES
BORING LOG

PROJECT NO.:	41011-100-01
REPORT NO.:	56732
APPENDIX:	A- 2

PROJECT: GEOTECHNICAL EVALUATION
TOMOKA LANDFILL NORTH CELL EXPANSION
VOLUSIA COUNTY, FLORIDA

BORING DESIGNATION: **B-2**

SHEET: **2 of 2**

DEPTH (FT.)	SAMPLE	BLOWS PER 6" INCREMENT	N (BLOWS/FT.)	W.T.	SYMBOL	DESCRIPTION	-200 (%)	MC (%)	ATTERBERG LIMITS LL	PI	K (FT./DAY)	ORG. CONT. (%)
50						Loose dark gray SILT (ML)	84.7					
55	X	2-2-2	4			Medium dense gray SHELL with SAND and SILT (SP-SM)						
60	X	11-14-11	25			Medium dense dark gray SAND and SHELL (SP)						
65	X	7-8-8	16			Dense gray to light brown CEMENTED SAND and SHELL (SP)						
70	X	19-20-14	34			END OF BORING: 70.0'						
75												
80												
85												
90												
95												
100												

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UNIVERSAL ENGINEERING SCIENCES
BORING LOG

PROJECT NO.: 41011-100-01
REPORT NO.: 56732
APPENDIX: A- 3

PROJECT: GEOTECHNICAL EVALUATION
TOMOKA LANDFILL NORTH CELL EXPANSION
VOLUSIA COUNTY, FLORIDA

BORING DESIGNATION: **B-3**

SHEET: **1 of 2**

CLIENT: VOLUSIA COUNTY
LOCATION: AS DIRECTED BY SCS ENGINEERS
REMARKS:

G.S. ELEVATION (ft):
WATER TABLE (ft): 1.5
DATE OF READING: 12/4/96
EST. W.S.W.T. (ft):

DATE STARTED: 12/4/96
DATE FINISHED: 12/4/96
DRILLED BY: U.E.S. DRILLING
TYPE OF SAMPLING: ASTM D-1586

DEPTH (FT.)	SAMPLE	BLOWS PER 6" INCREMENT	N (BLOWS/FT.)	W.T.	SYMBOL	DESCRIPTION	-200 (%)	MC (%)	ATTERBERG LIMITS LL	ATTERBERG LIMITS PI	K (FT./DAY)	ORG. CONT. (%)
0						Very loose to loose dark grayish brown CLAYEY SAND (SC)						
	X	2-2-3	5	▼								
	X	2-1-2	3									
5	X	2-3-4	7									
	X	3-4-4	8									
	X	4-5-6	11			Medium dense to loose brown SAND (SP)						
10	X	3-4-5	9									
						Loose to medium dense dark gray SILTY SAND (SM)						
15	X	3-3-4	7									
20	X	4-6-8	14				16.8					
						Loose greenish gray SILTY SAND (SM)						
25	X	1-2-2	4				45.7		41.0	20.2		
						Very dense gray SAND with SHELL and SILT and a trace of CEMENTED SAND (SP-SM)	12.6					
30	X	19-23-29	52									
						Very dense gray SAND with SHELL (SP)						
35	X	15-31-42	73									
						Medium dense dark gray SILTY SAND with SHELL (SM)	45.5		NP	NP		
40	X	6-4-7	11			(Layers of SHELL)						
						(Trace of CEMENTED SAND)						
45	X	5-4-6	10									
						Medium dense dark gray SILT with SAND and SHELL (SP-SM)						
50	X	4-5-6	11									

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UNIVERSAL ENGINEERING SCIENCES
BORING LOG

PROJECT NO.:	41011-100-01
REPORT NO.:	56732
APPENDIX:	A- 3

PROJECT: GEOTECHNICAL EVALUATION
TOMOKA LANDFILL NORTH CELL EXPANSION
VOLUSIA COUNTY, FLORIDA

BORING DESIGNATION: **B-3**

DEPTH (FT.)	SAMPLE	BLOWS PER 6" INCREMENT	N (BLOWS/ FT.)	W.T.	SYMBOL	DESCRIPTION	-200 (%)	MC (%)	ATTERBERG LIMITS LL	ATTERBERG LIMITS PI	K (FT./ DAY)	ORG. CONT. (%)
50												
55	X	9-15-16	31									
60	X	11-14-17	31									
						Medium dense gray to light brown SAND with SHELL and a trace of SILT (SP-SM)						
65	X	9-11-12	23			(Some CEMENTED SAND)						
70	X	9-9-13	23									
						END OF BORING: 70.0'						
75												
80												
85												
90												
95												
100												

BL3ORG

UNIVERSAL ENGINEERING SCIENCES
BORING LOG

PROJECT NO.:	41011-100-01
REPORT NO.:	56732
APPENDIX:	A- 4

PROJECT: GEOTECHNICAL EVALUATION
TOMOKA LANDFILL NORTH CELL EXPANSION
VOLUSIA COUNTY, FLORIDA

BORING DESIGNATION: **B-4**   SHEET: **1 of 2**

CLIENT: VOLUSIA COUNTY
LOCATION: AS DIRECTED BY SCS ENGINEERS
REMARKS:

G.S. ELEVATION (ft):
WATER TABLE (ft): 1.0
DATE OF READING: 12/5/96
EST. W.S.W.T. (ft):

DATE STARTED: 12/4/96
DATE FINISHED: 12/4/96
DRILLED BY: U.E.S. DRILLING
TYPE OF SAMPLING: ASTM D-1586

DEPTH (FT.)	SAMPLE	BLOWS PER 6" INCREMENT	N (BLOWS/FT.)	W.T.	SYMBOL	DESCRIPTION	-200 (%)	MC (%)	ATTERBERG LIMITS LL	ATTERBERG LIMITS PI	K (FT./DAY)	ORG. CONT. (%)
0				▼		Loose grayish brown CLAYEY SAND (SC)						
	X	2-2-2	4									
	X	2-3-5	8			Loose grayish light brown SAND with a trace of SILT (SP-SM)						
5	X	3-4-5	9									
	X	4-5-5	10									
	X	4-3-5	8			Loose grayish brown SAND (SP)						
10	X	3-3-4	7									
						Very loose dark gray SAND with SILT (SP-SM)						
15	X	2-1-2	3									
						Very loose dark gray to dark brown SILTY SAND (SM)	37.7		29.1	4.7		
20	X	1-1-1	2									
						Loose dark gray SILT with a trace of SHELL (SP)						
25	X	1-1-9	10									
						Very dense gray to dark gray SAND with SHELL (SP)						
30	X	15-33-31	64									
						Dense gray SAND with SHELL and traces of CEMENTED SAND (SP)						
35	X	19-20-24	44									
						Medium dense dark gray SILTY SAND and SHELL with a trace of CEMENTED SAND (SM)	45.5					
40	X	8-7-8	15									
						Medium dense dark gray SILT with SHELL (ML)	55.9		NP	NP		
45	X	4-5-7	12									
						Medium dense dark gray SAND and SHELL with SILT (SP-SM)						
50	X	5-5-6	11									

UNIVERSAL ENGINEERING SCIENCES
BORING LOG

PROJECT NO.:	41011-100-01
REPORT NO.:	56732
APPENDIX:	A- 4

PROJECT: GEOTECHNICAL EVALUATION
TOMOKA LANDFILL NORTH CELL EXPANSION
VOLUSIA COUNTY, FLORIDA

BORING DESIGNATION: **B-4**

SHEET: **2 of 2**

DEPTH (FT.)	SAMPLE	BLOWS PER 6" INCREMENT	N (BLOWS/ FT.)	W.T.	SYMBOL	DESCRIPTION	-200 (%)	MC (%)	ATTERBERG LIMITS LL	ATTERBERG LIMITS PI	K (FT./ DAY)	ORG. CONT. (%)
50						Medium dense dark gray SILT with a trace of SHELL and SAND (SP-SM)						
55	X	4-3-10	13			Dense to medium dense dark gray SAND with SILT and SHELL (SP-SM)	6.4					
60	X	8-10-23	33									
65	X	9-8-8	16			Medium dense light brownish gray SILTY SAND with a trace of SHELL and CEMENTED SAND (SP-SM)	18.5					
70	X	2-10-7	17			END OF BORING: 70.0'						
75												
80												
85												
90												
95												
100												

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UNIVERSAL ENGINEERING SCIENCES
BORING LOG

PROJECT NO.:	41011-100-01
REPORT NO.:	56732
APPENDIX:	A- 5

PROJECT: GEOTECHNICAL EVALUATION
TOMOKA LANDFILL NORTH CELL EXPANSION
VOLUSIA COUNTY, FLORIDA

BORING DESIGNATION: **B-5** SHEET: **1 of 2**

CLIENT: VOLUSIA COUNTY
LOCATION: AS DIRECTED BY SCS ENGINEERS
REMARKS:

G.S. ELEVATION (ft):
WATER TABLE (ft): 8.0
DATE OF READING: 12/5/96
EST. W.S.W.T. (ft):

DATE STARTED: 12/5/96
DATE FINISHED: 12/5/96
DRILLED BY: U.E.S. DRILLING
TYPE OF SAMPLING: ASTM D-1586

DEPTH (FT.)	SAMPLE	BLOWS PER 6" INCREMENT	N (BLOWS/ FT.)	W.T.	SYMBOL	DESCRIPTION	-200 (%)	MC (%)	ATTERBERG LIMITS LL	ATTERBERG LIMITS PI	K (FT./ DAY)	ORG. CONT. (%)
0						Loose to medium dense brown CLAYEY SAND (SC)						
	X	2-3-5	8									
	X	3-4-4	8									
5	X	5-6-8	14									
	X	6-7-5	12			Medium dense dark gray SAND with SILT (SP-SM)						
	X	5-5-6	11	▼								
	X	4-5-5	10									
10						Very loose greenish gray SAND and SILT (SM)						
	X	3-1-1	2									
15						Loose greenish gray SAND with a trace of SILT (SP-SM)						
	X	3-2-3	5									
20						Very loose dark gray to dark brown SILT with a trace of SAND (SM)						
	X	3-1-1	2									
25						Dense dark gray SAND and SHELL with a trace of SILT (SP-SM)						
	X	8-19-22	41									
30						Medium dense gray to light gray SAND and SHELL with a trace of CEMENTED SAND (SP)						
	X	14-15-12	27									
35						Medium dense dark gray SAND and SHELL with some SILT (SP-SM)						
	X	8-5-9	14									
40												
	X	4-9-10	19									
45						Medium dense dark gray SAND and SILT with SHELL (SP-SM)						
	X	5-6-6	12									
50												

BL3ORG

UNIVERSAL ENGINEERING SCIENCES
BORING LOG

PROJECT NO.:	41011-100-01
REPORT NO.:	56732
APPENDIX:	A- 5

PROJECT: GEOTECHNICAL EVALUATION
TOMOKA LANDFILL NORTH CELL EXPANSION
VOLUSIA COUNTY, FLORIDA

BORING DESIGNATION: **B-5**

DEPTH (FT.)	SAMPLE	BLOWS PER 6" INCREMENT	N (BLOWS/ FT.)	W.T.	SYMBOL	DESCRIPTION	-200 (%)	MC (%)	ATTERBERG LIMITS LL	ATTERBERG LIMITS PI	K (FT./ DAY)	ORG. CONT. (%)
50						Very dense dark gray SAND with SHELL and a trace of SILT (SP-SM)	7.8					
55	X	11-24-30	54									
						Medium dense greenish gray SAND and SHELL with SILT (SP-SM)	10.4					
60	X	12-13-16	29									
65	X	9-10-13	23									
						Very dense greenish gray SAND and SHELL with CEMENTED SAND and a trace of SILT (SP-SM)						
70	X	19-26-50	76									
						END OF BORING: 70.0'						
75												
80												
85												
90												
95												
100												

BL30RG

UNIVERSAL ENGINEERING SCIENCES
BORING LOG

PROJECT NO.:	41011-100-01
REPORT NO.:	56732
APPENDIX:	A- 6

PROJECT: GEOTECHNICAL EVALUATION
TOMOKA LANDFILL NORTH CELL EXPANSION
VOLUSIA COUNTY, FLORIDA

BORING DESIGNATION: **B-6** SHEET: **1 of 1**

CLIENT: VOLUSIA COUNTY
LOCATION: AS DIRECTED BY SCS ENGINEERS
REMARKS:

G.S. ELEVATION (ft):
WATER TABLE (ft): 3.5
DATE OF READING: 12/5/96
EST. W.S.W.T. (ft):

DATE STARTED: 12/5/96
DATE FINISHED: 12/5/96
DRILLED BY: U.E.S. DRILLING
TYPE OF SAMPLING: ASTM D-1586

DEPTH (FT.)	SAMPLE	BLOWS PER 6" INCREMENT	N (BLOWS/ FT.)	W.T.	SYMBOL	DESCRIPTION	-200 (%)	MC (%)	ATTERBERG LIMITS LL	PI	K (FT./ DAY)	ORG. CONT. (%)
0						Loose to medium dense brown CLAYEY SAND (SC)						
	X	3-4-4	8									
	X	9-9-11	20	▼								
5	X	12-10-17	27			Medium dense light brown SAND with a trace of SILT (SP-SM)						
	X	9-11-12	23									
	X	7-5-5	10									
	X	4-5-7	12									
10						Loose greenish gray SAND with SILT (SP-SM)						
15	X	1-2-2	4			Loose dark gray SILT with SAND (SP-SM)						
20	X	2-2-3	5			Very loose dark gray SILT with a trace of SAND (SM)						
25	X	1-1-1	2			Dense gray SAND with SHELL (SP)						
30	X	13-15-27	42			Very dense gray SAND with SHELL and a trace of SILT (SP-SM)	5.4					
35	X	25-36-42	78			Medium dense dark gray SAND and SHELL with SILT and a trace of CEMENTED SAND (SP-SM)						
40	X	8-9-6	15			END OF BORING: 40.0'						
45												
50												

BL3ORG

UNIVERSAL ENGINEERING SCIENCES
BORING LOG

PROJECT NO.:	41011-100-01
REPORT NO.:	56732
APPENDIX:	A- 7

PROJECT: GEOTECHNICAL EVALUATION
TOMOKA LANDFILL NORTH CELL EXPANSION
VOLUSIA COUNTY, FLORIDA

BORING DESIGNATION: **B-7** SHEET: **1 of 1**

CLIENT: VOLUSIA COUNTY
LOCATION: AS DIRECTED BY SCS ENGINEERS
REMARKS:

G.S. ELEVATION (ft):
WATER TABLE (ft): 6.0
DATE OF READING: 12/6/96
EST. W.S.W.T. (ft):

DATE STARTED: 12/6/96
DATE FINISHED: 12/6/96
DRILLED BY: U.E.S. DRILLING
TYPE OF SAMPLING: ASTM D-1586

DEPTH (FT.)	BLOWS PER 6" INCREMENT	N (BLOWS/ FT.)	W.T.	DESCRIPTION	-200 (%)	MC (%)	ATTERBERG LIMITS LL	ATTERBERG LIMITS PI	K (FT./ DAY)	ORG. CONT. (%)
0				Loose to medium dense brown CLAYEY SAND (SC)						
	2-4-5	9								
	4-7-4	11								
5	4-5-9	14		Medium dense to loose grayish brown SAND with SILT (SP-SM)						
	4-4-6	10	▼							
	5-5-6	11								
10	5-6-8	14			13.7					
15	4-4-5	9								
				Loose to very loose dark gray SILT with SAND (SM)						
20	3-3-5	8								
25	1-1-1	2								
				Dense dark gray SAND and SHELL with a trace of SILT (SP-SM)						
30	17-16-22	38								
				Very dense dark gray SAND with a trace of SHELL and CEMENTED SAND (SP)						
35	19-26-27	53								
				Medium dense dark gray SAND with a trace of SHELL and CEMENTED SAND (SP)						
40	15-10-8	18								
				END OF BORING: 40.0'						
45										
50										

BL3ORG

UNIVERSAL ENGINEERING SCIENCES
BORING LOG

PROJECT NO.:	41011-100-01
REPORT NO.:	56732
APPENDIX:	A- 8

PROJECT: GEOTECHNICAL EVALUATION
TOMOKA LANDFILL NORTH CELL EXPANSION
VOLUSIA COUNTY, FLORIDA

BORING DESIGNATION: **B-8** SHEET: **1 of 1**

This is not log for MW B-8 mH 7/6/07

CLIENT: VOLUSIA COUNTY
LOCATION: AS DIRECTED BY SCS ENGINEERS
REMARKS:

G.S. ELEVATION (ft):
WATER TABLE (ft): 3.0
DATE OF READING: 12/6/96
EST. W.S.W.T. (ft):

DATE STARTED: 12/6/96
DATE FINISHED: 12/6/96
DRILLED BY: U.E.S. DRILLING
TYPE OF SAMPLING: ASTM D-1586

see App E for map Soil Borings

DEPTH (FT.)	SAMPLE	BLOWS PER 6" INCREMENT	N (BLOWS/ FT.)	W.T.	SYMBOL	DESCRIPTION	-200 (%)	MC (%)	ATTERBERG LIMITS LL	ATTERBERG LIMITS PI	K (FT./ DAY)	ORG. CONT. (%)
0						Loose brown CLAYEY SAND (SC)						
	X	2-3-4	7									
	X	2-2-3	5	▼		Very loose to loose grayish brown SAND with SILT (SP-SM)	10.3					
5	X	2-1-2	3									
	X	2-3-3	6									
	X	2-2-3	5									
10	X	1-2-2	4									
						Medium dense greenish gray SAND with SILT (SP-SM)						
15	X	3-4-6	10									
20	X	4-5-7	12									
						Very loose dark gray SILT with traces of SAND (SM)						
25	X	1-1-1	2									
						Dense dark gray SAND with SHELL (SP)						
30	X	2-18-19	37									
35	X	7-17-22	39									
40	X	15-17-17	34									
						END OF BORING: 40.0'						
45												
50												

UNIVERSAL ENGINEERING SCIENCES
BORING LOG

PROJECT NO.:	41011-100-01
REPORT NO.:	56732
APPENDIX:	A- 9

PROJECT: GEOTECHNICAL EVALUATION
TOMOKA LANDFILL NORTH CELL EXPANSION
VOLUSIA COUNTY, FLORIDA

BORING DESIGNATION: **B-9**   SHEET: **1 of 2**

CLIENT: VOLUSIA COUNTY
LOCATION: AS DIRECTED BY SCS ENGINEERS
REMARKS: * = INDICATES 6" OF TRAVEL BY WEIGHT OF HAMMER

G.S. ELEVATION (ft):
WATER TABLE (ft): 7.0
DATE OF READING: 12/6/96
EST. W.S.W.T. (ft):

DATE STARTED: 12/6/96
DATE FINISHED: 12/6/96
DRILLED BY: U.E.S. DRILLING
TYPE OF SAMPLING: ASTM D-1586

DEPTH (FT.)	SAMPLE	BLOWS PER 6" INCREMENT	N (BLOWS/ FT.)	W.T.	SYMBOL	DESCRIPTION	-200 (%)	MC (%)	ATTERBERG LIMITS LL	ATTERBERG LIMITS PI	K (FT./ DAY)	ORG. CONT. (%)
0						Loose to medium dense dark grayish brown SAND with traces of CLAY and SHELL (SP-SC)						
	X	2-3-4	7									
	X	3-3-4	7									
5	X	4-5-6	11									
	X	5-5-8	13	▼								
	X	6-5-6	11									
	X	4-5-5	10									
10						Medium dense gray SAND with traces of SILT (SP-SM)						
	X	6-7-11	18									
15						Loose gray SILTY SAND (SM)						
	X	4-3-3	6				22.8		34.2	9.2		
20						Very loose dark gray SILT with traces of SAND (SM)						
	X	*-*-1	1									
25												
						Dense dark gray SAND with SHELL (SP)						
	X	15-21-24	45									
30						—Greenish gray						
	X	13-18-24	42									
35						Medium dense dark gray SAND and SILT with SHELL (SP-SM)						
	X	10-12-9	21									
40						Medium dense dark gray SILT with traces of SAND (SM)						
	X	5-6-11	17									
45						LOST SAMPLE						
	X	3-4-5	9						29.9	11.1		
50												

BL30RG

UNIVERSAL ENGINEERING SCIENCES
BORING LOG

PROJECT NO.:	41011-100-01
REPORT NO.:	56732
APPENDIX:	A- 9

PROJECT: GEOTECHNICAL EVALUATION
TOMOKA LANDFILL NORTH CELL EXPANSION
VOLUSIA COUNTY, FLORIDA

BORING DESIGNATION: **B-9**

DEPTH (FT.)	SAMPLE	BLOWS PER 6" INCREMENT	N (BLOWS/ FT.)	W.T.	SYMBOL	DESCRIPTION	-200 (%)	MC (%)	ATTERBERG LIMITS LL	PI	K (FT./ DAY)	ORG. CONT. (%)
50												
						END OF BORING: 52.0'						
55												
60												
65												
70												
75												
80												
85												
90												
95												
100												

BL3OR6

UNIVERSAL ENGINEERING SCIENCES
BORING LOG

PROJECT NO.: 41011-100-01
REPORT NO.: 56732
APPENDIX: A- 10

PROJECT: GEOTECHNICAL EVALUATION
TOMOKA LANDFILL NORTH CELL EXPANSION
VOLUSIA COUNTY, FLORIDA

BORING DESIGNATION: **B-10**   SHEET: **1 of 1**

CLIENT: VOLUSIA COUNTY
LOCATION: AS DIRECTED BY SCS ENGINEERS
REMARKS:

G.S. ELEVATION (ft):
WATER TABLE (ft): 2.5
DATE OF READING: 12/9/96
EST. W.S.W.T. (ft):

DATE STARTED: 12/9/96
DATE FINISHED: 12/9/96
DRILLED BY: U.E.S. DRILLING
TYPE OF SAMPLING: ASTM D-1586

DEPTH (FT.)	SAMPLE	BLOWS PER 6" INCREMENT	N (BLOWS/FT.)	W.T.	SYMBOL	DESCRIPTION	-200 (%)	MC (%)	ATTERBERG LIMITS LL	PI	K (FT./DAY)	ORG. CONT. (%)
0						Loose brown CLAYEY SAND (SC)						
	X	2-3-3	6	▼								
	X	2-2-4	6			Loose to very loose grayish brown SAND with a trace of SILT (SP-SM)						
5	X	2-2-3	5									
	X	2-3-4	7									
	X	2-1-2	3									
10	X	2-2-2	4			Very loose dark gray CLAYEY SAND (SC)						
							18.2					
15	X	1-1-1	2			Very loose to loose dark brownish gray SILT with a trace of SAND and SHELL (SM)						
20	X	1-1-2	3									
25	X	2-3-4	7			Very dense dark gray SAND and SHELL (SP)						
30	X	18-31-34	65			END OF BORING: 30.0'						
35												
40												
45												
50												

APPENDIX C

LABORATORY RESULTS

UES Project No. 41011-100-01
UES Report No. 56732
Date: January 13, 1997

REPORT OF SOIL GRADATION

Client: VOLUSIA COUNTY

Project: TOMOKA LANDFILL, NORTH CELL EXPANSION

Boring No.: B-1

Sample No.: S-3 Sample Description: Brown sand with clay [SP-SC]

Date: 12/4/96

TEST RESULTS

Sieve Sizes Passing	Percent Retained	Percent Passing
3/8"	0	100.0
No. 4	0	100.0
No. 10	0	100.0
No. 40	0.9	99.1
No. 60	3.2	96.8
No. 100	51.3	48.7
No. 200	90.2	9.8

UES Project No. 41011-100-01
UES Report No. 56732
Date: January 13, 1997

REPORT OF SOIL GRADATION

Client: VOLUSIA COUNTY

Project: TOMOKA LANDFILL, NORTH CELL EXPANSION

Boring No.: B-1

Sample No.: S-12 Sample Description: Gray sand with shell and silt [SP-SM]

Date: 12/4/96

TEST RESULTS

Sieve Sizes Passing	Percent Retained	Percent Passing
3/8"	0.5	99.5
No. 4	1.1	98.9
No. 10	3.4	96.6
No. 40	13.5	86.5
No. 60	16.3	83.7
No. 100	23.0	77.0
No. 200	87.2	12.8

UES Project No. 41011-100-01
UES Report No. 56732
Date: January 13, 1997

REPORT OF SOIL GRADATION

Client: VOLUSIA COUNTY

Project: TOMOKA LANDFILL, NORTH CELL EXPANSION

Boring No.: B-2

Sample No.: S-10 Sample Description: Gray sand with a trace of shell and silt [SP-SM]

Date: 12/4/96

TEST RESULTS

Sieve Sizes Passing	Percent Retained	Percent Passing
3/8"	0.6	99.4
No. 4	0.9	99.1
No. 10	1.5	98.5
No. 40	2.9	97.1
No. 60	5.6	94.4
No. 100	43.5	56.5
No. 200	92.3	7.7

UES Project No. 41011-100-01
UES Report No. 56732
Date: January 13, 1997

REPORT OF SOIL GRADATION

Client: VOLUSIA COUNTY

Project: TOMOKA LANDFILL, NORTH CELL EXPANSION

Boring No.: B-2

Sample No.: S-15 Sample Description: Dark gray silt [ML]

Date: 12/4/96

TEST RESULTS

Sieve Sizes Passing	Percent Retained	Percent Passing
3/8"	0.2	99.8
No. 4	0.4	99.6
No. 10	1.2	98.8
No. 40	2.7	97.3
No. 60	3.7	96.3
No. 100	7.1	92.9
No. 200	15.3	84.7

UES Project No. 41011-100-01
UES Report No. 56732
Date: January 13, 1997

REPORT OF SOIL GRADATION

Client: VOLUSIA COUNTY

Project: TOMOKA LANDFILL, NORTH CELL EXPANSION

Boring No.: B-3

Sample No.: S-8 Sample Description: Dark gray silty sand [SM]

Date: 12/4/96

TEST RESULTS

Sieve Sizes Passing	Percent Retained	Percent Passing
3/8"	0	100.0
No. 4	0	100.0
No. 10	0.1	99.9
No. 40	3.5	96.5
No. 60	15.6	84.4
No. 100	34.5	65.5
No. 200	83.2	16.8

UES Project No. 41011-100-01
UES Report No. 56732
Date: January 13, 1997

REPORT OF SOIL GRADATION

Client: VOLUSIA COUNTY

Project: TOMOKA LANDFILL, NORTH CELL EXPANSION

Boring No.: B-3

Sample No.: S-10

Sample Description: Gray sand with shell and silt with a trace of cemented sand [SP-SM]

Date: 12/4/96

TEST RESULTS

Sieve Sizes Passing	Percent Retained	Percent Passing
3/8"	0	100.0
No. 4	0.8	99.2
No. 10	3.6	96.4
No. 40	6.5	93.5
No. 60	7.6	92.4
No. 100	17.2	82.8
No. 200	87.4	12.6

UES Project No. 41011-100-01
UES Report No. 56732
Date: January 13, 1997

REPORT OF SOIL GRADATION

Client: VOLUSIA COUNTY

Project: TOMOKA LANDFILL, NORTH CELL EXPANSION

Boring No.: B-3

Sample No.: S-12 Sample Description: Dark gray silty sand with shell [SM]

Date: 12/4/96

TEST RESULTS

Sieve Sizes Passing	Percent Retained	Percent Passing
3/8"	0	100.0
No. 4	2.3	97.7
No. 10	3.4	96.6
No. 40	5.2	94.8
No. 60	5.6	94.4
No. 100	7.1	92.9
No. 200	54.5	45.5

UES Project No. 41011-100-01
UES Report No. 56732
Date: January 13, 1997

REPORT OF SOIL GRADATION

Client: VOLUSIA COUNTY

Project: TOMOKA LANDFILL, NORTH CELL EXPANSION

Boring No.: B-4

Sample No.: S-8 Sample Description: Dark gray silty sand [SM]

Date: 12/5/96

TEST RESULTS

Sieve Sizes Passing	Percent Retained	Percent Passing
3/8"	0	100.0
No. 4	0.1	99.9
No. 10	0.2	99.8
No. 40	0.3	99.7
No. 60	10.6	89.4
No. 100	23.8	76.2
No. 200	62.3	37.7

UES Project No. 41011-100-01
UES Report No. 56732
Date: January 13, 1997

REPORT OF SOIL GRADATION

Client: VOLUSIA COUNTY

Project: TOMOKA LANDFILL, NORTH CELL EXPANSION

Boring No.: B-4

Sample No.: S-12

Sample Description: Dark gray silty sand and shell with a trace of cemented sand [SP]

Date: 12/5/96

TEST RESULTS

Sieve Sizes Passing	Percent Retained	Percent Passing
3/8"	0	100.0
No. 4	2.3	97.7
No. 10	3.4	96.6
No. 40	5.2	94.8
No. 60	5.6	94.4
No. 100	7.1	92.9
No. 200	54.5	45.5

UES Project No. 41011-100-01
UES Report No. 56732
Date: January 13, 1997

REPORT OF SOIL GRADATION

Client: VOLUSIA COUNTY

Project: TOMOKA LANDFILL, NORTH CELL EXPANSION

Boring No.: B-4

Sample No.: S-13 Sample Description: Dark gray silt with shell [ML]

Date: 12/5/96

TEST RESULTS

Sieve Sizes Passing	Percent Retained	Percent Passing
3/8"	1.3	98.7
No. 4	3.5	96.5
No. 10	11.0	89.0
No. 40	12.6	87.4
No. 60	14.8	85.2
No. 100	27.9	72.1
No. 200	44.1	55.9

UES Project No. 41011-100-01
UES Report No. 56732
Date: January 13, 1997

REPORT OF SOIL GRADATION

Client: VOLUSIA COUNTY

Project: TOMOKA LANDFILL, NORTH CELL EXPANSION

Boring No.: B-4

Sample No.: S-16

Sample Description: Dark gray sand with silt and shell [SP-SM]

Date: 12/5/96

TEST RESULTS

Sieve Sizes Passing	Percent Retained	Percent Passing
3/8"	0	100.0
No. 4	0	100.0
No. 10	0.1	99.9
No. 40	3.3	96.7
No. 60	15.7	84.3
No. 100	66.6	33.4
No. 200	93.6	6.4

UES Project No. 41011-100-01
UES Report No. 56732
Date: January 13, 1997

REPORT OF SOIL GRADATION

Client: VOLUSIA COUNTY

Project: TOMOKA LANDFILL, NORTH CELL EXPANSION

Boring No.: B-6

Sample No.: S-11 Sample Description: Gray sand with shell and a trace of silt [SP-SM]

Date: 12/5/96

TEST RESULTS

Sieve Sizes Passing	Percent Retained	Percent Passing
3/8"	0	100.0
No. 4	0.2	99.8
No. 10	1.8	98.2
No. 40	4.2	95.8
No. 60	10.6	89.4
No. 100	58.4	41.6
No. 200	94.6	5.4

UES Project No. 41011-100-01
UES Report No. 56732
Date: January 13, 1997

REPORT OF SOIL GRADATION

Client: VOLUSIA COUNTY

Project: TOMOKA LANDFILL, NORTH CELL EXPANSION

Boring No.: B-7

Sample No.: S-6

Sample Description: Grayish brown sand with silt [SP-SM]

Date: 12/6/96

TEST RESULTS

Sieve Sizes Passing	Percent Retained	Percent Passing
3/8"	0	100.0
No. 4	0	100.0
No. 10	0	100.0
No. 40	1.0	99.0
No. 60	2.1	97.9
No. 100	29.2	70.8
No. 200	86.3	13.7

UES Project No. 41011-100-01
UES Report No. 56732
Date: January 13, 1997

REPORT OF SOIL GRADATION

Client: VOLUSIA COUNTY

Project: TOMOKA LANDFILL, NORTH CELL EXPANSION

Boring No.: B-8

Sample No.: S-2 Sample Description: Grayish brown sand with silt [SP-SM]

Date: 12/6/96

TEST RESULTS

Sieve Sizes Passing	Percent Retained	Percent Passing
3/8"	0	100.0
No. 4	0	100.0
No. 10	0	100.0
No. 40	3.4	96.6
No. 60	14.9	85.1
No. 100	52.0	48.0
No. 200	89.7	10.3

UES Project No. 41011-100-01
UES Report No. 56732
Date: January 13, 1997

REPORT OF SOIL GRADATION

Client: VOLUSIA COUNTY

Project: TOMOKA LANDFILL, NORTH CELL EXPANSION

Boring No.: B-9

Sample No.: S-8 Sample Description: Gray silty sand [SM]

Date: 12/6/96

TEST RESULTS

Sieve Sizes Passing	Percent Retained	Percent Passing
3/8"	0	100.0
No. 4	0	100.0
No. 10	0	100.0
No. 40	0.7	99.3
No. 60	5.1	94.9
No. 100	29.8	70.2
No. 200	77.2	22.8

UES Project No. 41011-100-01
UES Report No. 56732
Date: January 13, 1997

REPORT OF SOIL GRADATION

Client: VOLUSIA COUNTY

Project: TOMOKA LANDFILL, NORTH CELL EXPANSION

Boring No.: B-10

Sample No.: S-7 Sample Description: Dark gray clayey sand [SC]

Date: 12/9/96

TEST RESULTS

Sieve Sizes Passing	Percent Retained	Percent Passing
3/8"	0	100.0
No. 4	0	100.0
No. 10	0.1	99.9
No. 40	5.6	94.4
No. 60	9.7	90.3
No. 100	38.0	62.0
No. 200	81.8	18.2

REPORT ON ATTERBERG INDEX
LIQUID LIMIT / PLASTICITY INDEX

Client: Volusia County

Project: Tomoka Landfill, North Cell Expansion

Location: Volusia County, Florida

Soil Description: See Below

Date Tested: 12/26/96 **Tested By:** R. Cleland

Date Sampled: 12/15/96 **Sample No.:** See Below

Technician: R. Haire/D. Adkins

TEST RESULTS

Boring No.	Sample No.	Depth	Soil Description	Liquid Limit	Plastic Limit	Plasticity Index
B-3	S-9	25'	Grayish brown silty sand with some clay	36.0	16.9	19.1
B-3	S-12	40'	Grayish blue silty sand	N.P.	N.P.	N.P.
B-4	S-8	20'	Light grayish brown silty sand	29.1	24.4	4.7
B-4	S-13	45'	Grayish blue silty sand with traces of shell fragments	N.P.	N.P.	N.P.

Rick G. Kushner, P.E.
P.E. Number 38705
District Manager

REPORT ON ATTERBERG INDEX
LIQUID LIMIT / PLASTICITY INDEX

Client: Volusia County

Project: Tomoka Landfill, North Cell Expansion

Location: Volusia County, Florida

Soil Description: See Below

Date Tested: 12/26/96 **Tested By:** R. Cleland

Date Sampled: 12/6/96 **Sample No.:** See Below

Technician: R. Haire/D. Adkins

TEST RESULTS

Boring No.	Sample No.	Depth	Soil Description	Liquid Limit	Plastic Limit	Plasticity Index
B-9	S-8	20'	Grayish brown silty sand with clay	34.2	25.0	9.2
B-9	S-14	52'	Bluish gray silty sand with clay and shell fragments	29.9	18.8	11.1

Rick G. Kushner, P.E.
P.E. Number 38705
District Manager

APPENDIX D

TYPICAL SOIL PROPERTIES

Soil Mechanics

T. William Lambe • Robert V. Whitman

Massachusetts Institute of Technology

1969

JOHN WILEY & SONS, New York • Chichester • Brisbane • Toronto • Singapore

Source: "Soil Mechanics"

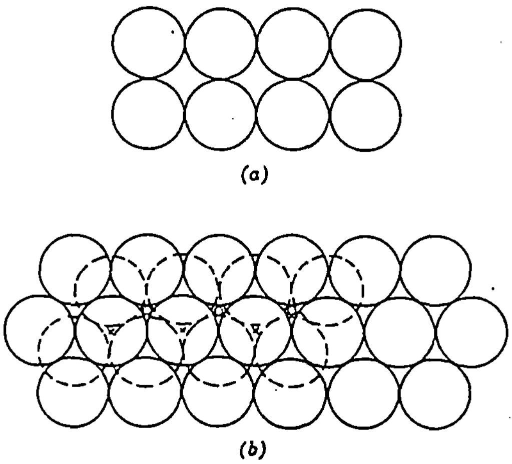


Fig. 3.2 Arrangements of uniform spheres. (*a*) Plan and elevation view: simple cubic packing. (*b*) Plan view: dense packing. Solid circles, first layer; dashed circles, second layer; ∘, location of sphere centers in third layer: face-centered cubic array; ×, location of sphere centers in third layer: close-packed hexagonal array. (From Deresiewicz, 1958.)

these simple packings can be computed from the geometry of the packings, and the results are given in Table 3.2. This table also gives densities for some typical granular soils in both the "dense" and "loose" states. A variety of tests have been proposed to measure the maximum and minimum void ratios (Kolbuszewski, 1948). The test to determine the maximum density usually involves some form of vibration. The test to determine minimum density usually involves pouring oven-dried soil into a container. Unfortunately, the details of these tests have not been entirely standardized, and values of the maximum density and minimum density for a given granular soil depend on the procedure used to determine them. By using special measures, one can obtain densities greater than the so-called maximum density. Densities considerably less than the so-called minimum density can be obtained, especially with very fine sands and silts, by slowly sedimenting the soil into water or by fluffing the soil with just a little moisture present.

Table 3.2 Maximum and Minimum Densities for Granular Soils

Description	Void Ratio		Porosity (%)		Dry Unit Weight (pcf)	
	e_{max}	e_{min}	n_{max}	n_{min}	$\gamma_{d\,min}$	$\gamma_{d\,max}$
Uniform spheres	0.92	0.35	47.6	26.0	—	—
Standard Ottawa sand	0.80	0.50	44	33	92	110
Clean uniform sand	1.0	0.40	50	29	83	118
Uniform inorganic silt	1.1	0.40	52	29	80	118
Silty sand	0.90	0.30	47	23	87	127
Fine to coarse sand	0.95	0.20	49	17	85	138
Micaceous sand	1.2	0.40	55	29	76	120
Silty sand and gravel	0.85	0.14	46	12	89	146

The smaller the range of particle sizes present (i.e., the more nearly uniform the soil), the smaller the particles, and the more angular the particles, the smaller the minimum density (i.e., the greater the opportunity for building a loose arrangement of particles). The greater the range of particle sizes present, the greater the maximum density (i.e., the voids among the larger particles can be filled with smaller particles).

A useful way to characterize the density of a natural granular soil is with *relative density* D_r, defined as

$$D_r = \frac{e_{max} - e}{e_{max} - e_{min}} \times 100\%$$
$$= \frac{\gamma_{d\,max}}{\gamma_d} \times \frac{\gamma_d - \gamma_{d\,min}}{\gamma_{d\,max} - \gamma_{d\,min}} \times 100\% \qquad (3.1)$$

where

e_{min} = void ratio of soil in densest condition
e_{max} = void ratio of soil in loosest condition
e = in-place void ratio
$\gamma_{d\,max}$ = dry unit weight of soil in densest condition
$\gamma_{d\,min}$ = dry unit weight of soil in loosest condition
γ_d = in-place dry unit weight

Table 3.3 characterizes the density of granular soils on the basis of relative density.

Table 3.3 Density Description

Relative Density (%)	Descriptive Term
0–15	Very loose
15–35	Loose
35–65	Medium
65–85	Dense
85–100	Very dense

moisture test

$\omega = \frac{M_s}{M_w}$

Values of water content for natural granular soils vary from less than 0.1% for air-dry sands to more than 40% for saturated, loose sand.

Typical Values of Phase Relationships for Cohesive Soils

The range of values of phase relationships for cohesive soils is much larger than for granular soils. Saturated sodium montmorillonite at low confining pressure can exist at a void ratio of more than 25; saturated clays

Source: Soil Mechanics; Lambe & Whitman 1969

SHEET 1

GEOTECHNICAL ENGINEERING INVESTIGATION MANUAL

ROY E. HUNT
Consulting Engineer

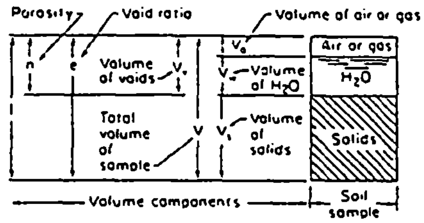
McGraw-Hill Book Company
New York St. Louis San Francisco Auckland Bogotá Hamburg Johannesburg London Madrid Mexico Montreal New Delhi Panama Paris São Paulo Singapore Sydney Tokyo Toronto

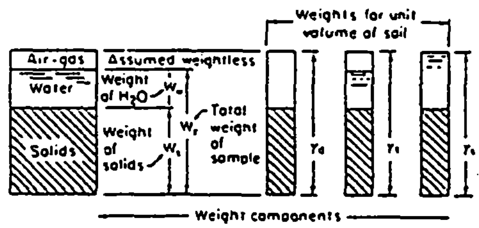
INDEX PROPERTIES
EQUATIONS

Source "Geotechnical Engineering Investigation Manual"

TABLE 3.8
VOLUME-WEIGHT RELATIONSHIPS FOR SOILS*

	Property	Saturated sample (W_s, W_w, G_s are known)	Unsaturated sample (W_s, W_w, G_s, V are known)	Illustration of sample
Volume components	Volume of solids V_s	$\frac{W_s}{G_s\gamma_w \dagger}$		
	Volume of water V_w	$\frac{W_w}{\gamma_w \ddagger}$		
	Volume of air or gas V_a	zero	$V - (V_s + V_w)$	
	Volume of voids V_v	$\frac{W_w}{\gamma_w \ddagger}$	$V - \frac{W_s}{G_s\gamma_w}$	
	Total volume of sample V	$V_s + V_w$	Measured	
	Porosity n	$\frac{V_v}{V}$ or $\frac{e}{1+e}$		
	Void ratio e	$\frac{V_v}{V_s} = \cancel{\frac{G_s\gamma_w}{\gamma_d} - 1}$ $\frac{G_s\gamma_w}{\gamma_d} - 1$		
Weights for specific sample	Weight of solids W_s	Measured		
	Weight of water W_w	Measured		
	Total weight of sample W_t	$W_s + W_w$		
Weights for sample of unit volume	Dry-unit weight γ_d	$\frac{W_s}{V_s + V_w}$	$\frac{W_s}{V}$	
	Wet-unit weight γ_t	$\frac{W_s + W_w}{V_s + V_w}$	$\frac{W_s + W_w}{V}$	
	Saturated-unit weight γ_s	$\frac{W_s + W_w}{V_s + V_w}$	$\frac{W_s + V_v\gamma_w}{V}$	
	Submerged (buoyant) unit weight γ_b	$\gamma_s - \gamma_w \ddagger$		
Combined relations	Moisture content w	$\frac{W_w}{W_s}$		
	Degree of saturation S	1.00	$\frac{V_w}{V_v}$	$\gamma_d = \frac{\gamma_t}{1+w}$ $\boxed{\gamma_s = \gamma_d + \gamma_w\left(\frac{e}{1+e}\right)}$
	Specific gravity G_s	$\frac{W_s}{V_s\gamma_w}$		





*After NAVFAC (1971).[6]

†γ_w is unit weight of water, which equals 62.4 pcf for fresh water and 64 pcf for sea water (1.00 and 1.025 g/cm³). Where noted with ‡ the actual unit weight of water surrounding the soil is used. In other cases use 62.4 pcf. Values of w and s are used as decimal numbers.

SATURATED UNIT WEIGHT OF SOIL

Source:
Same as
Sheet

TABLE 3.31
TYPICAL PROPERTIES OF COMPACTED SOILS*

Group symbol	Soil type	Range of maximum dry unit weight, pcf	Range of optimum moisture, %	Typical value of compression: Percent of original height, At 1.4 tsf (20 psi)	Typical value of compression: Percent of original height, At 3.6 tsf (50 psi)	Typical strength characteristics: Cohesion (as compacted), psf	Typical strength characteristics: Cohesion (saturated), psf	Typical strength characteristics: Effective stress envelope φ, degrees	Typical strength characteristics: tan φ	Typical coefficient permeabil ft/min
GW	Well-graded clean gravels, gravel-sand mixtures	125–135	11–8	0.3	0.6	0	0	>38	>0.79	5 × 1
GP	Poorly graded clean gravels, gravel-sand mix	115–125	14–11	0.4	0.9	0	0	>37	>0.74	10⁻
GM	Silty gravels, poorly graded gravel-sand silt	120–135	12–8	0.5	1.1	...	...	>34	>0.67	>10⁻
GC	Clayey gravels, poorly graded gravel-sand-clay	115–130	14–9	0.7	1.6	...	...	>31	>0.60	>10⁻
SW	Well-graded clean sands, gravelly sands	110–130	16–9	0.6	1.2	0	0	38	0.79	>10⁻
SP	Poorly-graded clean sands, sand-gravel mix	100–120	21–12	0.8	1.4	0	0	37	0.74	>10⁻
SM	Silty sands, poorly graded sand-silt mix	110–125	16–11	0.8	1.6	1050	420	34	0.67	5 × 1
SM-SC	Sand-silt clay mix with slightly plastic fines	110–130	15–11	0.8	1.4	1050	300	33	0.66	2 × 1
SC	Clayey sands, poorly graded sand-clay mix	105–125	19–11	1.1	2.2	1550	230	31	0.60	5 × 1
ML	Inorganic silts and clayey silts	95–120	24–12	0.9	1.7	1400	190	32	0.62	10⁻
ML-CL	Mixture of inorganic silt and clay	100–120	22–12	1.0	2.2	1350	460	32	0.62	5 ×
CL	Inorganic clays of low to medium plasticity	95–120	24–12	1.3	2.5	1800	270	28	0.54	10⁻
OL	Organic silts and silt-clays, low plasticity	80–100	33–21	...	...	...	...	...	...	...
MH	Inorganic clayey silts, elastic silts	70–95	40–24	2.0	3.8	1500	420	25	0.47	5 ×
CH	Inorganic clays of high plasticity	75–105	36–19	2.6	3.9	2150	230	19	0.35	10⁻
OH	Organic clays and silty clays	65–100	45–21	...	...	...	...	...	...	...

*From NAVFAC Manual DM 7 (1971).[6] All properties are for condition of "standard Proctor" maximum density, except values of k and CBR which are for "modified Proctor" maximum density. Typical strength characteristics are for effective strength envelopes and are obtained from USBR data. Compression values are for vertical loading with complete lateral confinement. (...) Indicates insufficient data available for an estimate.

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COMMON PROPERTIES OF COHESIONLESS SOILS

Material	Compactness	D_R, %	N*	γ dry,† g/cm^3	Void ratio e	Strength‡ ϕ
GW: well-graded gravels, gravel-sand mixtures	Dense	75	90	2.21	0.22	40
	Medium dense	50	55	2.08	0.28	36
	Loose	25	<28	1.97	0.36	32
GP: poorly graded gravels, gravel-sand mixtures	Dense	75	70	2.04	0.33	38
	Medium dense	50	50	1.92	0.39	35
	Loose	25	<20	1.83	0.47	32
SW: well-graded sands, gravelly sands	Dense	75	65	1.89	0.43	37
	Medium dense	50	35	1.79	0.49	34
	Loose	25	<15	1.70	0.57	30
SP: poorly graded sands, gravelly sands	Dense	75	50	1.76	0.52	36
	Medium dense	50	30	1.67	0.60	33
	Loose	25	<10	1.59	0.65	29
SM: silty sands	Dense	75	45	1.65	0.62	35
	Medium dense	50	25	1.55	0.74	32
	Loose	25	<8	1.49	0.80	29
ML: inorganic silts, very fine sands	Dense	75	35	1.49	0.80	33
	Medium dense	50	20	1.41	0.90	31
	Loose	25	<4	1.35	1.0	27

*N is blows per foot of penetration in the SPT. Adjustments for gradation are after Burmister (1962).[13] See Table 3.23 for general relationships of D_R vs. N.

†Density given is for G_s = 2.65 (quartz grains).

‡Friction angle ϕ depends on mineral type, normal stress, and grain angularity as well as D_R and gradation (see Fig. 3.63).

Source: Same as Sheet

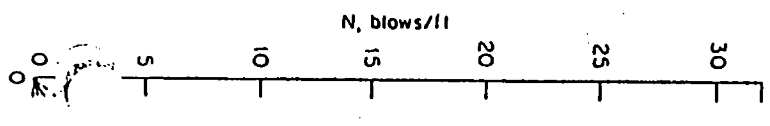

FIG. 3.64

APPENDIX E

SETTLEMENT CALCULATIONS

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SETTLEMENT ANALYSIS FOR TOMOKA LANDFILL

3/20/97
RHI

SCOPE: Estimate settlement of subsurface soils due to final landfill loadings.

SUBSURFACE CONDITIONS:

1. Review of borings B-1 to B-10 by Universal Engineering Services indicates soils are predominantly sandy with varying percentages of silt and clay. Descriptions include sand, silty sand, clayey sand and silt, also containing shell fragments. Unified Soil Classifications include SM, SP, SP-SM and ML.

2. Standard penetration test values range from 1 to 73. Soils are described as medium dense to dense in general, although there is a loose zone in each of the 10 borings as follows:

	A	B	C	D	E
1	Boring	SPT Range	Average SPT	"Loose" Zone	Soil Type
2	B-1	2 to 39	18	14' to 28'	Clay with Sand
3	B-2	3 to 41	14	9' to 25'	Silty Sand
4	B-3	4 to 73	18	10 to 27'	Silty Sand
5	B-4	2 to 64	16	10' to 25'	Silt; Silty Sand
6	B-5	2 to 76	20	10' to 25'	Sand; Silt
7	B-6	2 to 78	20	10' to 25'	Sand; Silt
8	B-7	2 to 53	16	15' to 25'	Silt with Sand
9	B-8	2 to 39	14	15' to 25'	Silt
10	B-9	1 to 45	16	15' to 27'	Silt
11	B-10	2 to 65	11	5' to 25'	Clayey Sand; Silt
12					

3. Grain size tests indicate P200 and USCS Classifications as follows:

	A	B	C	D
1	Boring	Sample	P200	USCS
2	B-1	S-3	9.8	SP-SC
3	B-1	S-12	12.8	SP-SM
4	B-2	S-10	7.7	SP-SM
5	B-2	S-15	84.7	ML
6	B-3	S-8	16.8	SM
7	B-3	S-10	12.6	SP-SM
8	B-3	S-12	45.5	SM
9	B-4	S-8	37.7	SM
10	B-4	S-12	45.5	SP
11	B-4	S-13	55.9	ML
12	B-4	S-16	6.4	SP-SM
13	B-6	S-11	5.4	SP-SM
14	B-6	S-6	13.7	SP-SM
15	B-8	S-2	10.3	SP-SM
16	B-9	S-8	22.8	SM
17	B-10	S-7	18.2	SC

4. Atterberg Limits test values as follows:

	A	B	C	D
1	Boring	Sample	Liquid Limit	Plasticity Index
2	B-3	S-9	36	19.1
3	B-3	S-12	NP	NP
4	B-4	S-8	29.1	4.7
5	B-4	S-13	NP	NP
6	B-9	S-8	34.2	9.2
7	B-9	S-14	29.9	11.1

5. Geologic Cross Section B-B' depicts soils extending to elevation -55 feet. Limestone bedrock below soil.

6. Groundwater was encountered within 10 feet of the ground surface.

FINAL LANDFILL LOADING CONDITIONS:

Based on final grading plans, it is assumed that the landfill will have a maximum waste depth of 140 feet , which occurs at the central portion. At the outer edges the minimum waste depth will be about 25 feet. Waste density is assumed to be 1500 PCY which reduces to about 56 PCF, which is average for municipal solid waste.

Due to the relatively large dimensions of the facility, these loadings are distributed to a depth greater than the soil depth with little reduction. That is, the soil depth of about 60 feet is much smaller than the landfill with of approximately 1300 feet. Therefore, it is conservatively assumed that the full waste loading occurs throughout the soil column.

It is assumed that the limestone bedrock is relatively incompressable in comparison to the overlying soils.

METHOD OF SETTLEMENT ANALYSIS:

Estimate settlement based on consolidation theory.

Use the relationship: $$\Delta H = H \frac{C_c}{1+e_o} \log\left(1 + \frac{\Delta P}{P_i}\right)$$

where, ΔH = change in layer thickness due to compressions

H = original layer thickness

e_o = initial void ration (in-place)

C_c = compression index

Δp = induced stress at center of layer due to loading

p_i = initial stress at center of layer due overburden

Consider 3 main layers.

Layer 1 includes the uppermost 10 feet of sandy and clayey sand soil. It is in a loose to medium dense condition, and is fully submerged. Assume a unit dry density of 95 pcf and initial void ratio of 0.72 , based a G = 2.63 (per Universal). Coefficient of compression is estimated to be 0.03, based on empirical data from Hough, 1957.

Layer 2 includes the underlying loose layer of sandy and silty material that averages 15 feet thick. It is also fully submerged. Assume a unit dry density of 87 pcf and initial void ratio of 0.88 , based a G = 2.63 (per Universal). Coefficient of compression is estimated to be 0.06, based on empirical data from Hough, 1957.

Layer 3 consists of the remaining 40 feet of sandy soil, but broken up into two layers of 20 feet each (i.e., Layer 3A and 3B). It is in a medium dense to very dense condition based on blow count data. Assume a unit dry weight of 105 pcf and initial void ratio of 0.56. Coefficient of compression is estimated to be 0.02.

For these layers, the original stress at the center of each layer assumes submerged unit weights. However, since excavation has already occurred, an additional loading equivalent to 15 feet of 100 pcf soil has been added to cover this unloading.

Based on these parameters the total and differential settlements have been calculated and are shown on the attached spreadsheet.

Consolidation Settlement Estimate for Tomoka Farms Landfill 4/20/97 RHI

Layer	Depth to Center of Layer	Thickness	Coeff. of Compression	Initial Void Ratio	Original Stress at Center	Induced Stress at Center	Change in Thickness at Center	Induced Stress at Edge	Change in Thickness at Edge
		H	Cc	e_o	Pi	DP	DH	DP	DH
	(feet)	(feet)			(psf)	(psf)	(inches)	(psf)	(inches)
95 1	5	10	0.03	0.72	1665	7778	1.58	1389	0.55
87 2	12.5	15	0.06	0.88	2017.5	7778	3.94	1389	1.31
105 3A	35	20	0.02	0.56	2635	7778	1.84	1389	0.57
105 3B	55	20	0.02	0.56	3495	7778	1.56	1389	0.45
	Total Soil Thickness=	65		Total Settlement in Inches =			8.92		2.87
				Differential Settlement in Inches =					6.05

Initial (15 ft)(100 #/ft3) = 1500 psf
+ 5(95-62.4) = 1,665 psf

INDUCED STRESS = (140 ft of WASTE)(1500/27 lb/ft3) = 7778 psf MIDPOINT

= (25 ft of WASTE)(1500/27 lb/ft3) = 1389 psf TOE

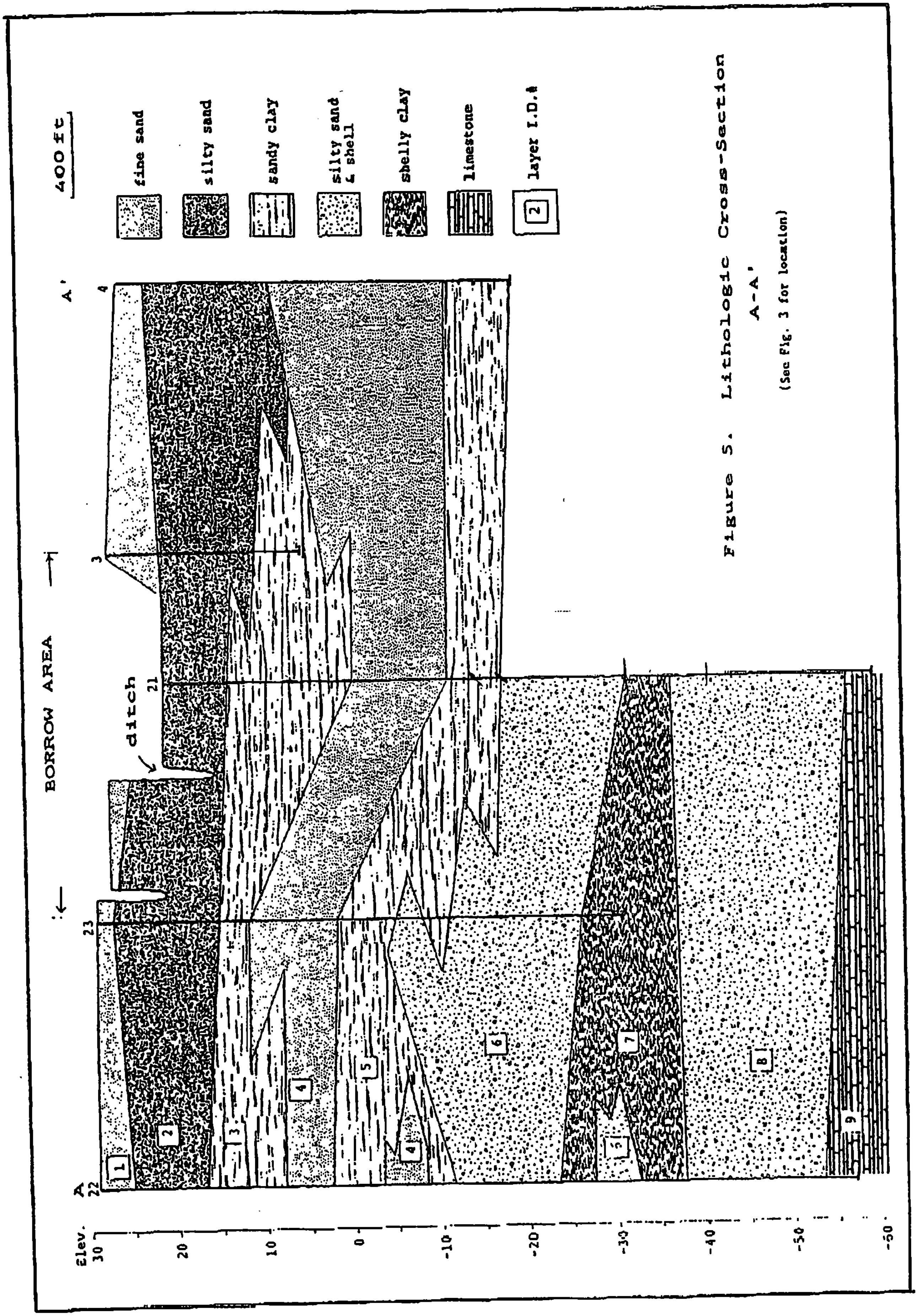


Figure 5. Lithologic Cross-Section A-A'

(See Fig. 3 for location)

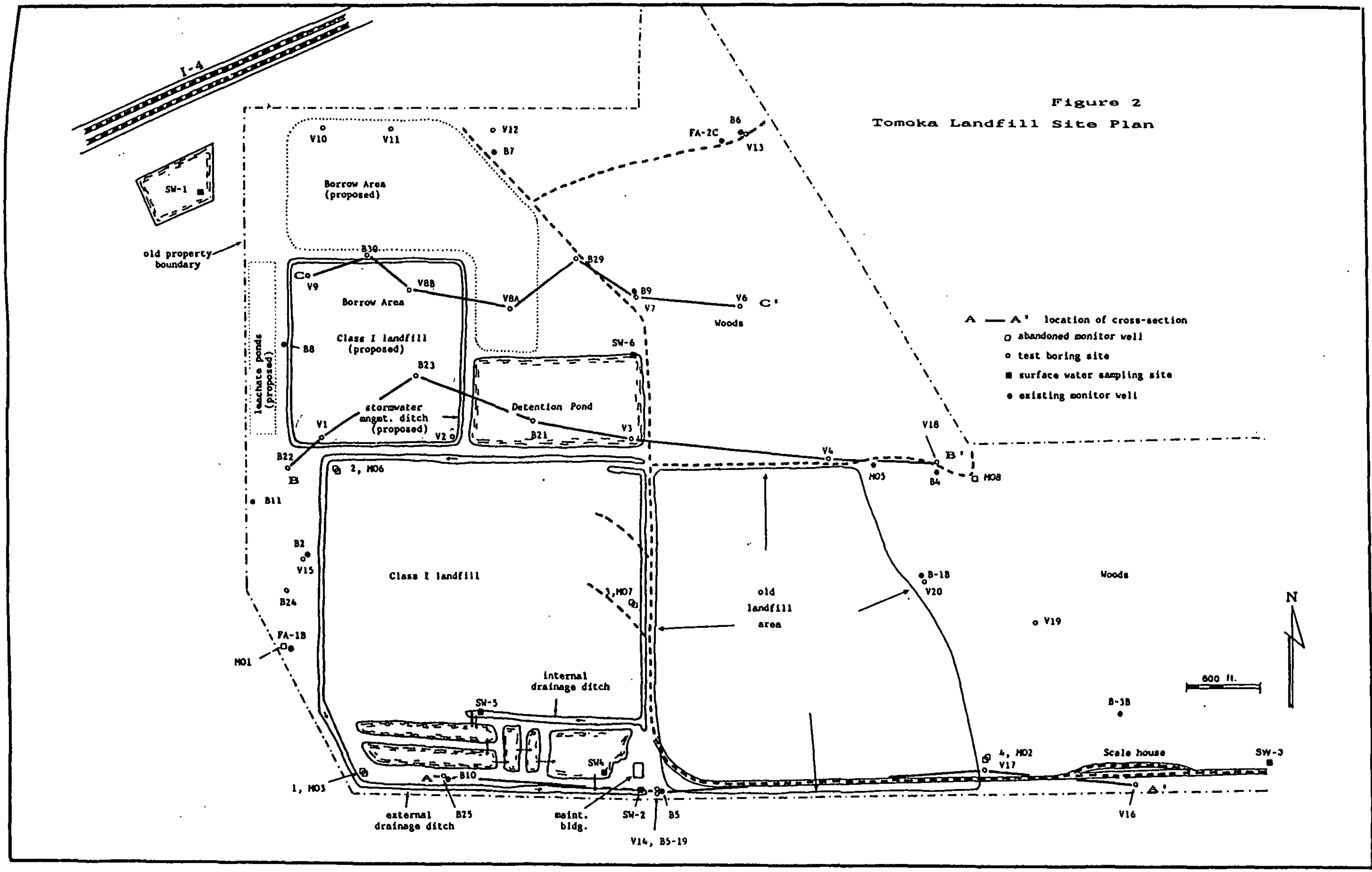


Figure 2
Tomoka Landfill Site Plan
I-4
SW-1
old property boundary
leachate ponds (proposed)
Borrow Area (proposed)
Borrow Area
Class I landfill (proposed)
stormwater mngmt. ditch (proposed)
Detention Pond
Woods
Class I landfill
old landfill area
internal drainage ditch
external drainage ditch
maint. bldg.
Scale house
A — A' location of cross-section
abandoned monitor well
test boring site
surface water sampling site
existing monitor well
600 ft.
N
V10
V11
V12
B7
FA-2C
B6
V13
B30
C
V9
V8B
V8A
B29
B9
V7
V6
C'
B8
B23
SW-6
V1
V2
B21
V3
V4
V18
B'
B22
B
2, MO6
MO5
B4
MO8
B11
B2
V15
B24
1, MO7
B-1B
V20
V19
FA-1B
MO1
SW-5
B-3B
SW4
4, MO2
V17
SW-3
1, MO3
A
B10
B25
SW-2
B5
V14, B5-19
A'
V16

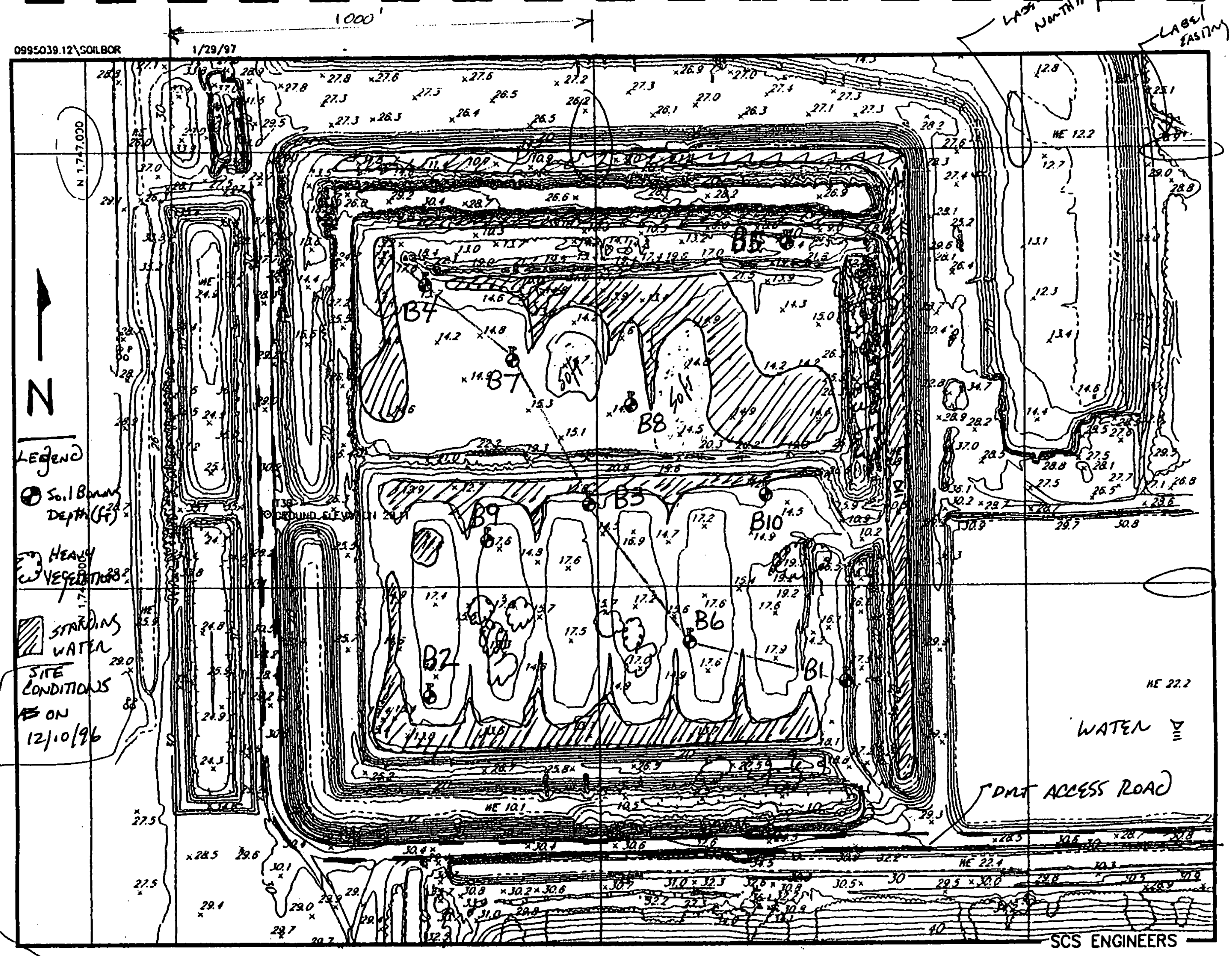


Figure _. ~~General Site Plan, Tampa, Florida~~.
BORING LOCATION MAP (SITE CONDITIONS ON 12/10/96)
1" ≃ 300'

SCS ENGINEERS
11260 Roger Bacon Drive
Reston, Virginia 20190
703 471-6150
FAX 703 471-6676

JOB Tomolka LF
SHEET NO. ______ OF ______
CALCULATED BY RI DATE 3/20/97
CHECKED BY ______ DATE ______
SCALE ______

Compression Index Estimates

From Hough, 1957 :

$C_c = a(e_o - b)$

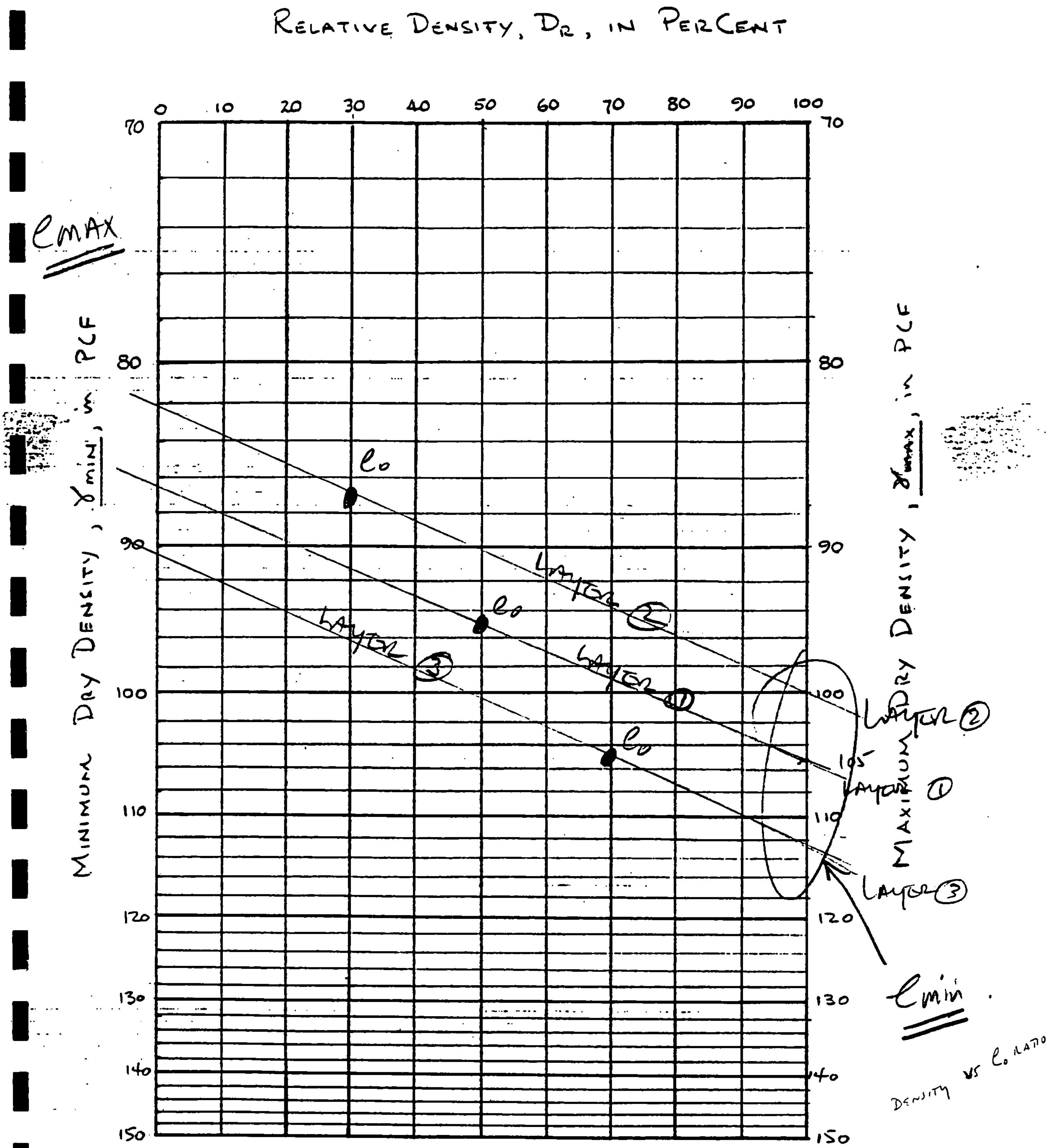
e_o = initial void ratio

a, b } constants based on grain size, minimum void ratio, etc.

Inplace Dr	Layer	Inplace γ_{dry}	Inplace e_o	a	b (e_{min})	C_c
50%	1	95 pcf	0.72	0.20	0.57	0.03
30%	2	89 pcf	0.88	0.25	0.64	0.06
70%	3	105 pcf	0.56	0.20	0.46	0.02

b ⇒ e_{min} (densest condition)

estimate e_{min} based on Relative Density (Dr) of soil in place and approx. max density for soil type.



$$D_R = \left(\frac{\gamma_{max}}{\gamma}\right)\left(\frac{\gamma - \gamma_{min}}{\gamma_{max}-\gamma_{min}}\right)\cdot 100$$

Calc. b (emin) from This....

[redacted]

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BASIC SOILS ENGINEERING

B. K. HOUGH

SECOND EDITION

THE RONALD PRESS COMPANY • NEW YORK

ditions, the slope of the recompression diagram gives a more realistic indication of the compressibility of the formation than the slope of the virgin curve. One would then represent in the diagram the anticipated load increment Δp and establish the prospective change in void ratio as the difference between the values of e_1 and e_2. For these conditions, the change in thickness of a compressible soil layer would be calculated by substitution of these values in Eq. (5-4).

5-14. LIMITATIONS OF COMPRESSION TESTING

In order to evaluate the compression index of soil in the manner described above, suitable specimens must be obtained or prepared and one or more laboratory compression tests must be conducted. In most cases, undisturbed specimens are considered necessary. Because of the limitations of present-day sampling equipment, however, especially the equipment in the hands of most contract drillers, it is for all practical purposes impossible to obtain undisturbed samples except in stone-free clay and silt formations. Testing equipment is at present also similarly limited to use with these particular soil types. Thus there remains the problem of establishing the compression index or some similar parameter for mixed soils containing significant amounts of gravel or stone fragments as well as clay or silt, and for cohesionless formations in general. There has been some tendency in the past to dismiss this problem with the assertion that the last-mentioned soil types are relatively incompressible. While this is true in certain cases (as with hardpan or dense sand and gravel formations), there are many occasions when the problem cannot be thus dismissed. The fact is that *all* particulate materials are compressible to some degree. Some fine-grained cohesionless soil formations, especially those containing significant amounts of mica or organic matter, for example, are considerably more compressible than certain clays while many others are at least equally compressible. Furthermore, with unusual combinations of loading and settlement limitations,[12] the compressibility of even the most compact sand and gravel formation or compacted fill may become a matter of practical importance.

Perhaps the most important consideration, however, is that what is known as the *allowable bearing capacity* of soil formations for support of spread foundations is directly related to soil compressibility. Evaluation of bearing capacity, which is an essential preliminary step in the design of spread foundations (footings in particular), cannot be accom-

[12] See *Jour. Soil Mech. & Fdns. Div.*, ASCE, April 1960, discussion by Lev Zetlin of paper by B. K. Hough, "Compressibility as the Basis for Soil Bearing Value."

plished except by the most empirical procedures, unless the compressibility of the bearing materials is known at least approximately; this is true whether the soil happens to be stoney or stone-free.

An alternative to use of data from conventional compression tests for evaluating the compression index is therefore an evident necessity in many cases. Even with stone-free, cohesive materials, some alternative is often desirable since there are many occasions when preliminary settlement estimates or bearing capacity evaluations must be made before laboratory testing programs can be completed or even initiated. The following section deals with one such alternative.

Compression Index as a Function of Initial Density

5-15. SUPPORTING EVIDENCE AND DEVELOPMENT OF RELATIONSHIP

Virgin compression curves and typical C_c values for specimens of many different types of soil are presented in Fig. 5-11. Some of the specimens were undisturbed (U); some remolded (R). Examination of the converging pattern of these curves clearly indicates that, in a general way, compressibility varies with initial void ratio; the looser the specimen initially, the more compressible it is over any given loading range, and vice versa.

The nature of the relationship between the compression index, C_c, and no-load void ratio, e_0, for certain types of material can be established by conducting tests on remolded specimens prepared at densities which vary over a significant range. It is then possible to plot C_c as a function of e_0. In Fig. 5-12, curves plotted on this basis for remolded specimens of four different types of sand are presented. For each individual type and within the range of densities characteristic of the type, the relationship appears to be approximately linear. When this is true, the relationship may be expressed by the equation

$$C_c = a(e_0 - b) \tag{5-7}$$

In Eq. (5-7), the terms C_c and e_0 are the dependent variables, the terms a and b constants for a particular soil type. From presently available information it appears that the term a, which represents the slope of a given diagram, is dependent chiefly on particle shape, size, and gradation. The term b, the value of the intercept on the X-axis, is apparently a close approximation of the minimum void ratio of the material. Values of a and b for the sand specimens represented in Fig. 5-12 are given in the figure and values for other materials are given in a later section.

Similar soils.

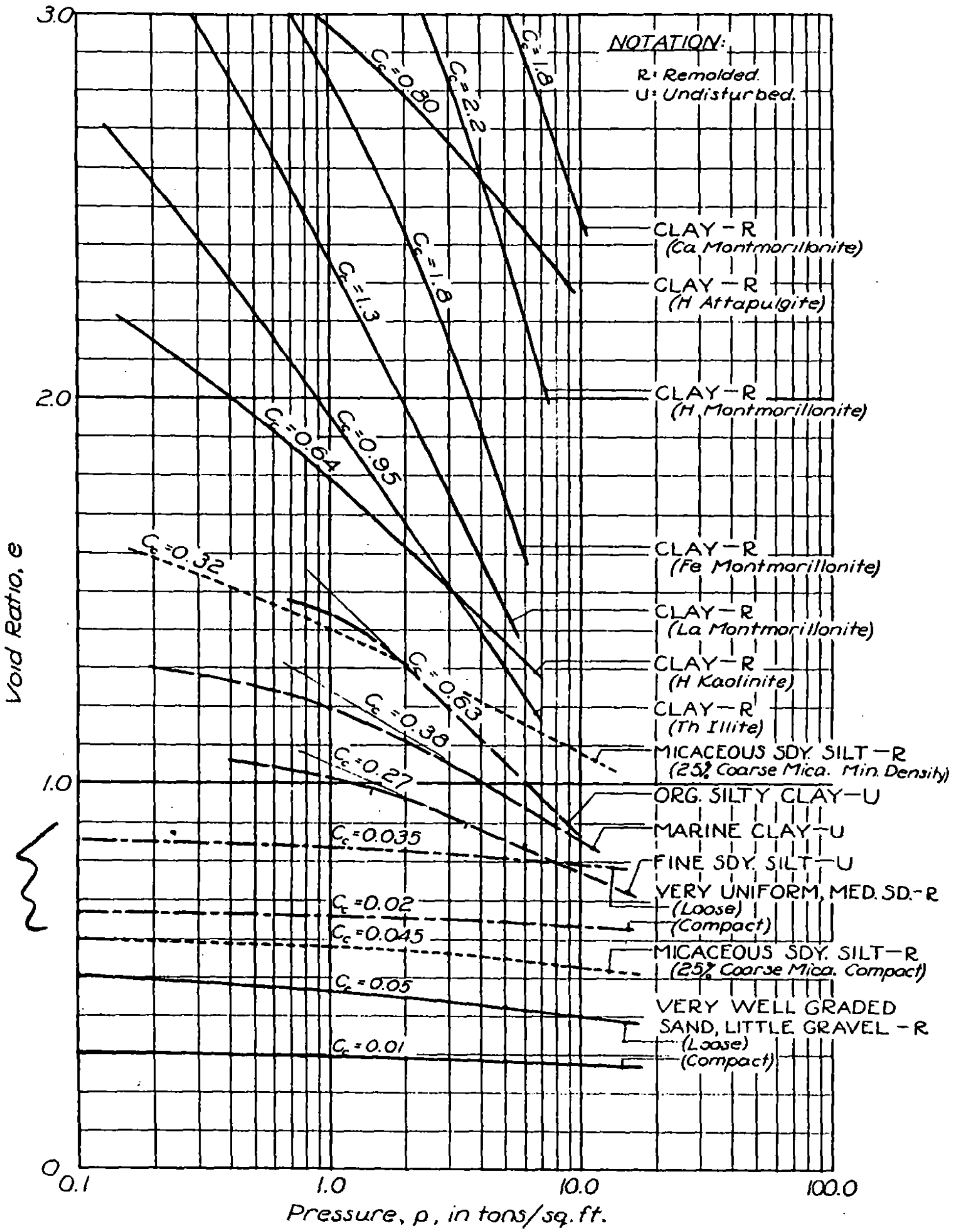


Fig. 5-11. Variation in slope of *p-e* curves with initial void ratio.

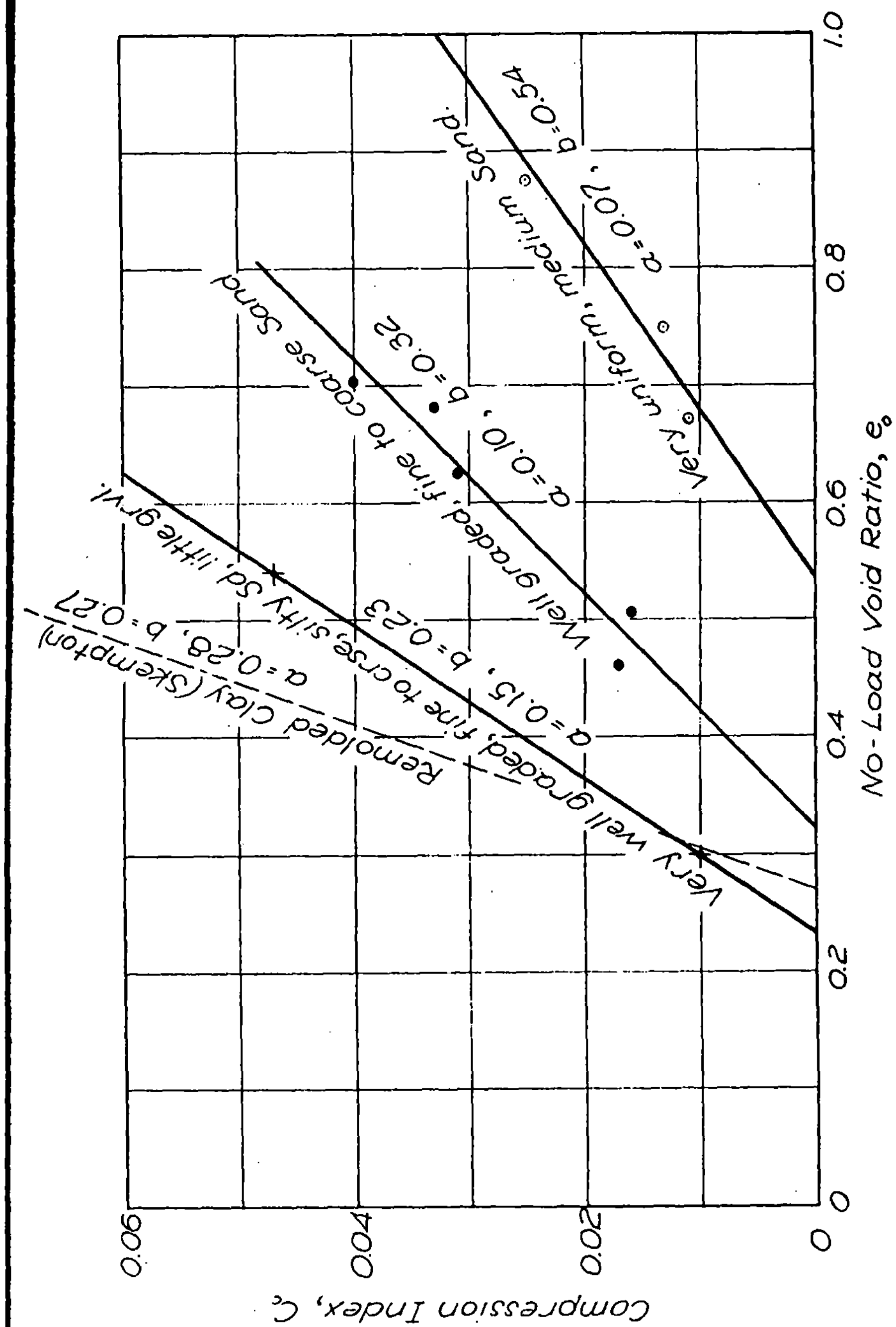


Fig. 5-12. Variation of C_c with no-load void ratio, e_o.

values 0.285 and 0.270, respectively; a curve plotted on this basis is included in Fig. 5-12 for comparative purposes.

Values of the constants a and b of Eq. (5-7) obtained from tests on laboratory prepared specimens of many different soil types, including those described above, are summarized in Table 5-1. The values given

TABLE 5-1
Values of the Constants of Equation (5-7) for Typical Materials

Type of Soil	Value of Constant	
	a	b*
Uniform cohesionless material ($C_u \leqq 2$)		
Clean gravel	0.05	0.50
Coarse sand	0.06	0.50
Medium sand	0.07	0.50
Fine sand	0.08	0.50
Inorganic silt	0.10	0.50
Well-graded, cohesionless soil		
Silty sand and gravel	0.09	0.20
Clean, coarse to fine sand	0.12	0.35
Coarse to fine silty sand	0.15	0.25
Sandy silt (inorganic)	0.18	0.25
Inorganic, cohesive soil		
Silt, some clay; silty clay; clay	0.29	0.27
Organic, fine-grained soil		
Organic silt, little clay	0.35	0.50

* The value of the constant b should be taken as e_{min} whenever the latter is known or can conveniently be determined. Otherwise, use tabulated values as a rough approximation.

for materials such as sand and gravel, which are too coarse for testing in consolidometers of conventional size, represent assumptions based on study of available settlement records.

5-16. GENERALIZATIONS AS TO COMPRESSIBILITY

Before describing procedures for utilizing Eq. (5-7) for evaluation of the compression index in practical applications, it may be instructive to consider certain general aspects of compressibility which are evident from the discussion which has thus far been presented. These generalities may be stated in the following manner.

At a given void ratio, a (confined) *uniform* material is less compressible than one which is well graded.

Considering (confined) uniform materials at a given void ratio, the finer the particle size, the more compressible is the material.

Soils in general with bulky, angular, or rounded particles are less compressible than those with flat particles.

Clays with needle-shaped particles, such as attapulgite (and to a lesser degree, halloysite), are less compressible than those with plate-shaped particles, montmorillonite (plate-shaped particles plus expanding lattice) in particular.

Materials of any given type which include significant amounts of mica and/or organic matter are more (sometimes considerably more) compressible than those of the same type which do not.

As an overall generalization, the greater its void ratio prior to loading, the greater is the compressibility of any given soil type; and vice versa.[15]

5-17. INITIAL DENSITY OF SOIL FORMATIONS

It is evident that information on the original, "no-load" void ratio of a formation must be available if the C_c, e_0 relationship is to be used directly for estimating soil compressibility. A rather general impression apparently exists to the effect that sedimentary formations, at least, are laid down initially in a condition approximating their maximum void ratio. Skempton's work suggests that this is true in the case of fine-grained sedimentary formations, clay in particular. Coupled with this belief is the assumption that the present, in-place condition of such formations is entirely the result of loading subsequent to deposition. If these assumptions could be completely accepted, the value e_{max} could be substituted for e_0 in Eq. (5-7) and application of the equation would be greatly simplified.

Unfortunately, there are many reasons for doubting the general applicability of such assumptions as the above. For example, in a texturally uniform deposit of fine-grained sand or silt, if these assumptions were valid, the void ratio of the material would steadily decrease with depth and at any given depth would have the same value at points which laterally are some distance apart. The finding of such a condition in a natural formation, however, is very much more the exception than the rule. In many cases, void ratio varies quite unpredictably both laterally and with depth. Most surprising to the layman, perhaps, is the finding that void ratio often *increases* with depth, loose sand layers being found beneath more compact surface layers and soft clay intervals underlying stiff clay.

The construction of compression diagrams based on use of the C_c, e_0 relationship in the manner described in the next section is often helpful

[15] This, of course, is the justification for the expenditure of considerable sums of money to compact both earth fills and natural soil formations prior to loading.

[redacted]s a[redacted]ma[redacted]ses [redacted]ovi[redacted]mea[redacted]esti[redacted]g th[redacted]pre[redacted] sion index without recourse to undisturbed sampling and laboratory testing.

Field Compression Diagrams

5–18. DEFINITION

As the term is used in this book, a field compression diagram is a pressure-void ratio curve originating at or passing through a point which represents the in-place density of an element in a natural soil formation or earth fill and the existing overburden pressure.

5–19. CONSTRUCTION AND UTILIZATION

The recommended construction should be performed on semilog paper with pressure and void ratio scales appropriate to the conditions of the problem. The void ratio scale should cover the range from e_{max} to e_{min} for the material in question. For the pressure scale, it is usually sufficient to make provision for two logarithmic cycles ranging from 0.1 to 1.0 and from 1.0 to 10.0 tons per sq. ft., respectively.

A pressure-void ratio curve originating at $e = e_{max}$ and $p = 0.1$ ton per sq. ft. is then constructed as shown in Fig. 5–14, by utilization of the relationship,

$$C_c = a(e_{max} - b)$$

For clay soils, e_{max} can be taken as the void ratio at the liquid limit. For other soil types, an indication of e_{max} can be obtained by reference to Table 2–3 or by test on representative material. Although of less practical importance, it may be of interest to draw a second diagram, originating at e_{min}. The latter may be assumed to be a horizontal line.

The two diagrams described above establish limits on the area within which a point representing the in-place condition of the soil will fall except in a very few cases, which are mentioned later. Points A, B, and C in Fig. 5–14 represent examples of in-place condition points for ordinary situations.

If a plotting of the in-place void ratio and overburden pressure for a soil element of any type results in a point such as point A, close to the uppermost limiting diagram, it may reasonably be assumed that the material was laid down in an approximation of its loosest condition and that the subsequent reduction in void ratio was due entirely to weight of present overburden. If the soil is a cohesive type it would [redacted]ther[redacted] (5–[redacted] or [redacted](5–7[redacted]bstit[redacted] e_m[redacted]$_o$, c[redacted]e us[redacted] to obtain an approximation of the compression index for this material.

If the soil is a clay which is in such a condition that the in-place void ratio and pressure plot at point B, it should be presumed, initially at least, that it is precompressed and that the field compression diagram

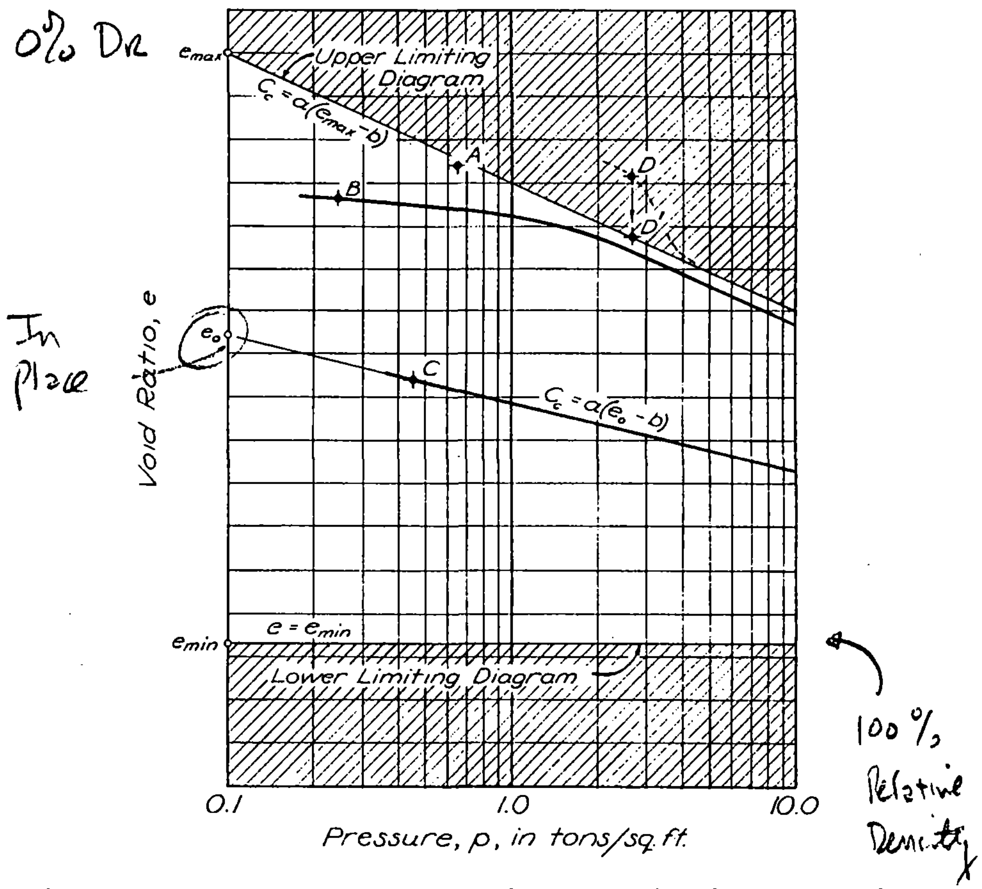


Fig. 5–14. Illustration of procedure for constructing field compression diagrams.

will resemble that shown by the full line diagram through B in Fig. 5–14. This plotting provides a reasonable basis for recommending a program of undisturbed sampling and laboratory testing even though greater than ordinary expense may be involved.

APPENDIX F

MUNICIPAL SOLID WASTE STRENGTH CHARACTERISTICS

SHEET 1/2

Geotechnics of WASTE FILLS

Theory and Practice

Landva/Knowles, editors

ASTM

STP 1070

Landva/Knowles

Geotechnics of Waste Fills
Theory and Practice

STP 1070

ASTM

Source "Same As Sheet 1/2"

SHEET 2of2

of 15 was used by Dames & Moore (20) after rejecting values larger than 50 that may represent the encounter of obstructions. Earth Tech Corporation (9) reported the results of a vane shear test and a standard penetration test. These results are shown in Figure 3.

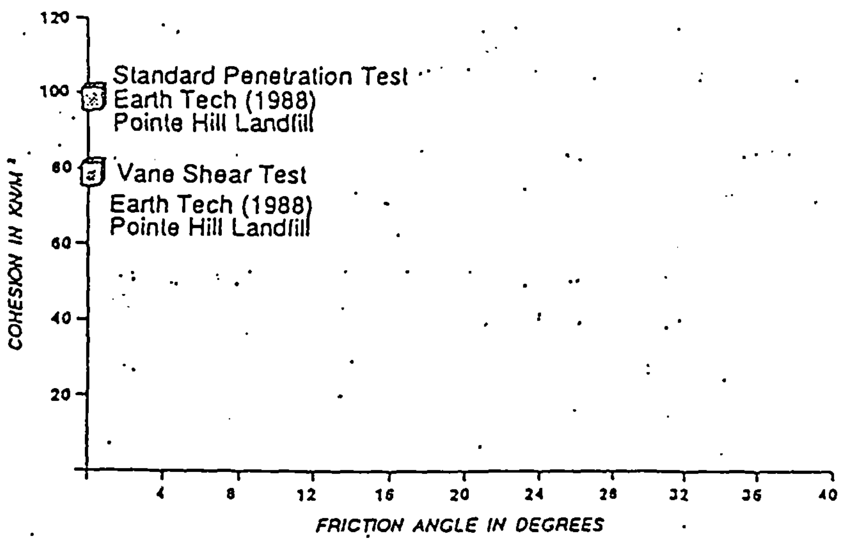


Fig. 3 - Strength Parameters from In-Situ Testing

Finally, the results of all the foregoing tests are plotted in Figure 4. Because of the scatter and scarcity of the data, it is difficult to draw any definitive conclusions on the shear strength characteristics of sanitary fill material.

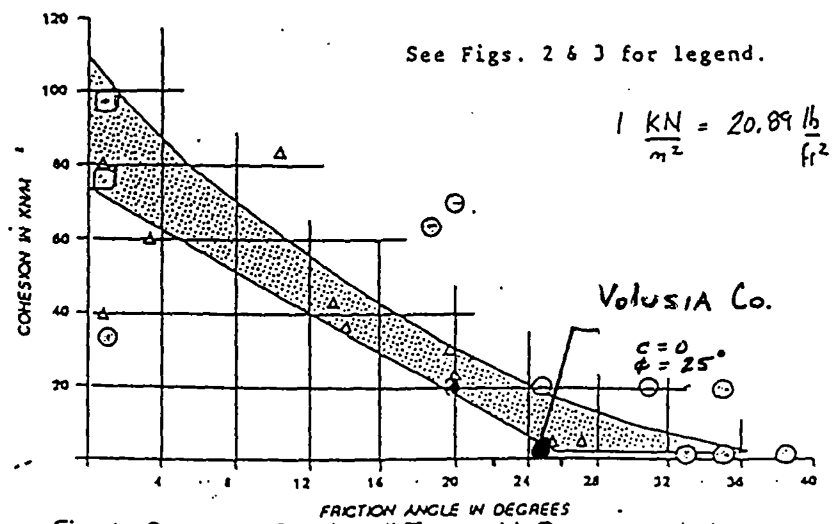


Fig. 4 - Summary Plot for All Tests with Recommended Parameters Shaded

MSW Typical Values

APPENDIX G

SLOPE STABILITY MODEL - INPUTS AND OUTPUT VALUES

April 3, 1997
File No. 0995039.12

MEMORANDUM

TO: Joe O'Neill

FROM: James Law/Bob Isenberg

SUBJECT: Tomoka Farms Road Landfill
Slope Stability Analysis

INTRODUCTION

This memorandum presents the slope stability analyses for the proposed bottom liner system at the Tomoka Farms Road Landfill, Volusia County, Florida.

OBJECTIVE

The objective of the slope stability analysis is to evaluate the static stability of the proposed liner systems both on the side slope and at the base of the landfill.

APPROACH

The approach used in achieving the above objective was to:

- obtain published testing data and/or assumption regarding the internal friction angles of the soil or geosynthetic clay liner (GCL) and interface friction angles of the geosynthetic/geosynthetic or soil/geosynthetic interfaces;

- select a critical cross-section for slope stability analyses; and

- evaluate the slope stability of the critical slope section assuming hydrated GCL under approximately 20 psi confining pressure, using PCSTABL5M computer program.

<u>Proposed Liner System</u>

The proposed liner system on the side slope consists of the following layers (from bottom to top):

- Compacted subgrade;

- Geosynthetic Clay Liner (GCL) - needled punched;

- 60-mil HDPE Secondary Liner - textured on both sides;
- 6-ounce-per-square-yard spunbonded, non-woven geotextile;
- Drainage net which consists of a 200-mil thick, HDPE geonet;
- 6-ounce-per-square-yard spunbonded, non-woven geotextile;
- Geosynthetic Clay Liner (GCL) - needled punched; and
- 60-mil HDPE Primary Liner - textured on the side facing down.

The proposed liner system at the bottom of the landfill consists of the following layers (from bottom to top):

- Prepared compacted subgrade;
- Geosynthetic Clay Liner (GCL) - needled punched;
- 60-mil HDPE Secondary Liner (smooth);
- Drainage net which consists of a 200-mil thick, HDPE geonet;
- 6-ounce-per-square-yard spunbonded, non-woven geotextile;
- Geosynthetic Clay Liner (GCL) - needled punched;
- 60-mil HDPE Primary Liner (smooth);
- Drainage net which consists of a 200-mil thick, HDPE geonet;
- 6-ounce-per-square-yard spunbonded, non-woven geotextile; and
- 2-feet Drainage Sand.

ASSUMPTIONS

<u>Material Shear Strengths</u>

The following values are based on available published data and our experience with similar materials:

- internal friction angle of waste = 25 degrees.
- cohesion of waste = 0 psf. (Typically this value is assumed to be zero to be conservative).
- internal friction angle of foundation soil below liner system = 28 degrees.

- internal friction angle of hydrated unreinforced GCL = 4 degrees. This value is based on the available published data.

- internal friction angle of hydrated reinforced GCL = 16 degrees. This value is based on the available published data.

- cohesion of GCL (unreinforced or reinforced) = 0 psf. Typically this value is assumed to be zero to be conservative.

<u>Interface Shear Strengths (Adhesion assumes to be zero in all cases)</u>

- interface friction angle between drainage net/textured HDPE = 6 degrees.

- interface friction angle between drainage net/smooth HDPE = 8 degrees.

- interface friction angle between non-woven geotextile/sand = 21 degrees.

- interface friction angle between smooth HDPE FML/GCL = 14 degrees.

- interface friction angle between textured HDPE FML/GCL = 22 degrees.

- interface friction angle between drainage net/geotextile = 25 degrees.

TYPICAL CROSS-SECTION

A typical cross-section, shown Exhibit 1, was selected for the analysis. This section has a final slope of approximately 3(H) to 1(V). The final slope height is about 150 feet measured from the inner toe of the perimeter berm. The location of this cross-section A-A' is shown in Exhibit 2.

METHOD OF ANALYSIS

Slope stability analysis involves the calculation of the minimum (critical) safety factor for assumed failure surfaces through representative slope cross sections. The safety factor is defined as the ratio of the available shear strength to the shear strength required for stability. A safety factor of 1.0 (unity) indicates a condition of impending slope failure; that is, where the available shear strength of the soil or waste, or along any single interface is equal to the strength required for stability. A minimum safety factor of 1.5 is the generally accepted minimum value recommended in the industry for static slope stability.

The method used to evaluate the slope stability was to calculate the factor of safety utilizing Janbu's method of slices in the PCSTABL5M computer program. A block-type failure was assumed and the failure surface was assumed to be along the weakest interface which has the lowest interface friction angle. The surfaces are randomly generated within the specified weakest interface.

A critical failure surface is automatically determined for selected cross sections by the PCSTABL5M program. The calculated critical failure surface defines a slope mass with the lowest static safety factor.

RESULTS OF ANALYSIS

The results of the slope stability analysis using PCSTABL5M computer program are summarized as follows:

- When assuming a critical internal friction angle of 4 degrees for the hydrated unreinforced GCL, the factor of safety of 0.9 was calculated using a block-type failure surface within the GCL. Consider an area adjacent to the inner toe of the perimeter berm. This factor of safety can be drastically increased to 1.5 if at least 260 feet length (measured perpendicularly from the inner toe of the perimeter berm toward the landfilling area) of reinforced GCL is used in that area. This reinforced GCL is assumed to have an internal friction angle of 16 degrees.

- When a circular-type toe failure surface is assumed, a factor of safety of 1.7 is calculated, assuming using at least 260 feet of reinforced GCL measured from the inner toe of the perimeter berm. The failure surface is found to be located within the waste and close to the final surface of the side slope.

- When a deep-seated, block-type failure mode is assumed within the underlying foundation soils below the liner system, the factor of safety is found to be 2.5. The result indicates that the foundation soils are very stable under the proposed landfill loading conditions and thus a deep-seated block-type failure is not likely.

CONCLUSIONS AND RECOMMENDATIONS

Based on the results of the analyses, it is concluded and recommended that:

- From the results of the analysis, it is observed that, to eliminate weak interface friction angles, any interface which has a friction angle of less than 16 degrees within the 260 feet zone from the inner toe of the perimeter berm should be eliminated from the proposed liner system. Therefore, the minimum interface or internal friction angle of the bottom liner system has to be at least 16 degrees to provide a factor of safety of at least 1.5.

- As a result, the smooth HDPE FML, geonet, and filter fabric layers used in the proposed liner system must be replaced with a textured HDPE FML and a drainage geocomposite layer. The geocomposite drainage net is defined as a geonet with non-woven geotextile heat-bonded to either side of the geonet. The smooth FML or the unreinforced GCL may be used in an inner area of the landfill footprint that is beyond the 260 feet distance from the inner toe of the perimeter berm.

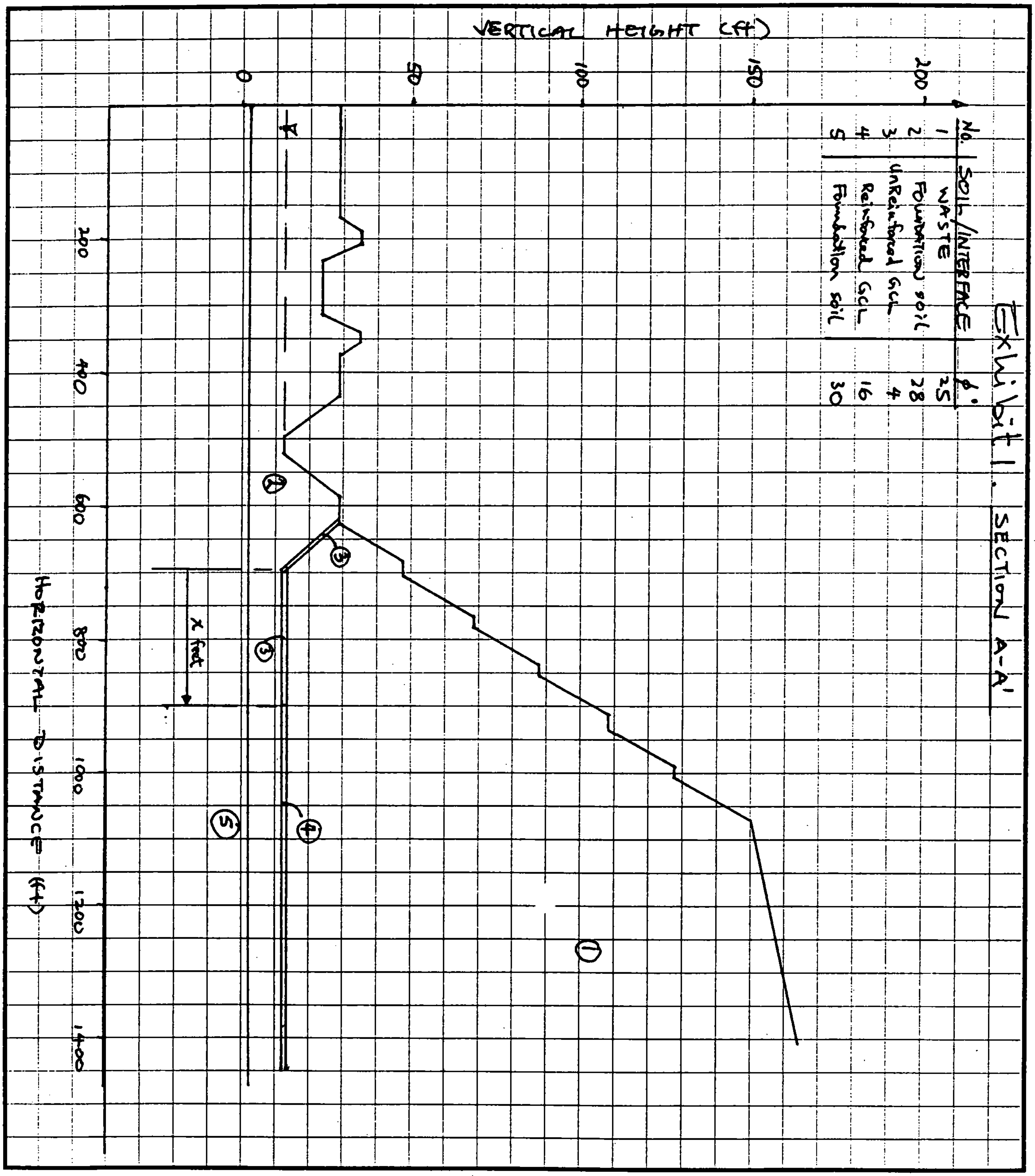
- The geocomposite drainage net above the secondary liner may potentially be clogged with the hydrated or saturated GCL if the GCL is fully hydrated over time. If the GCL has to be used, a high flow capacity geocomposite drainage net or thicker geonet with a larger void space should be considered to minimize potential fully hydration of the GCL.

The critical interface friction angles between various geosynthetic materials and/or soil layers were estimated based on our experience and knowledge of the typical soil/geosynthetic characteristics. However the internal or the interface friction angles are product dependent and should be verified by testing of the actual interface materials prior to start of construction (or once the project-specific materials are known). It is recommended that the geosynthetic/soil or geosynthetic/geosynthetic interface direct shear test be performed on the anticipated low valued interfaces, using project-specific materials. The test procedure used should be in accordance with ASTM D 5321, "Determining the coefficient of geosynthetic/geosynthetic and soil/geosynthetic friction by the direct shear method".

SCS ENGINEERS

11260 Roger Bacon Drive
Reston, Virginia 22090
703 471-6150
FAX 703 471-6676

JOB ______
SHEET NO. ______ OF ______
CALCULATED BY ______ DATE ______
CHECKED BY ______ DATE ______
SCALE ______



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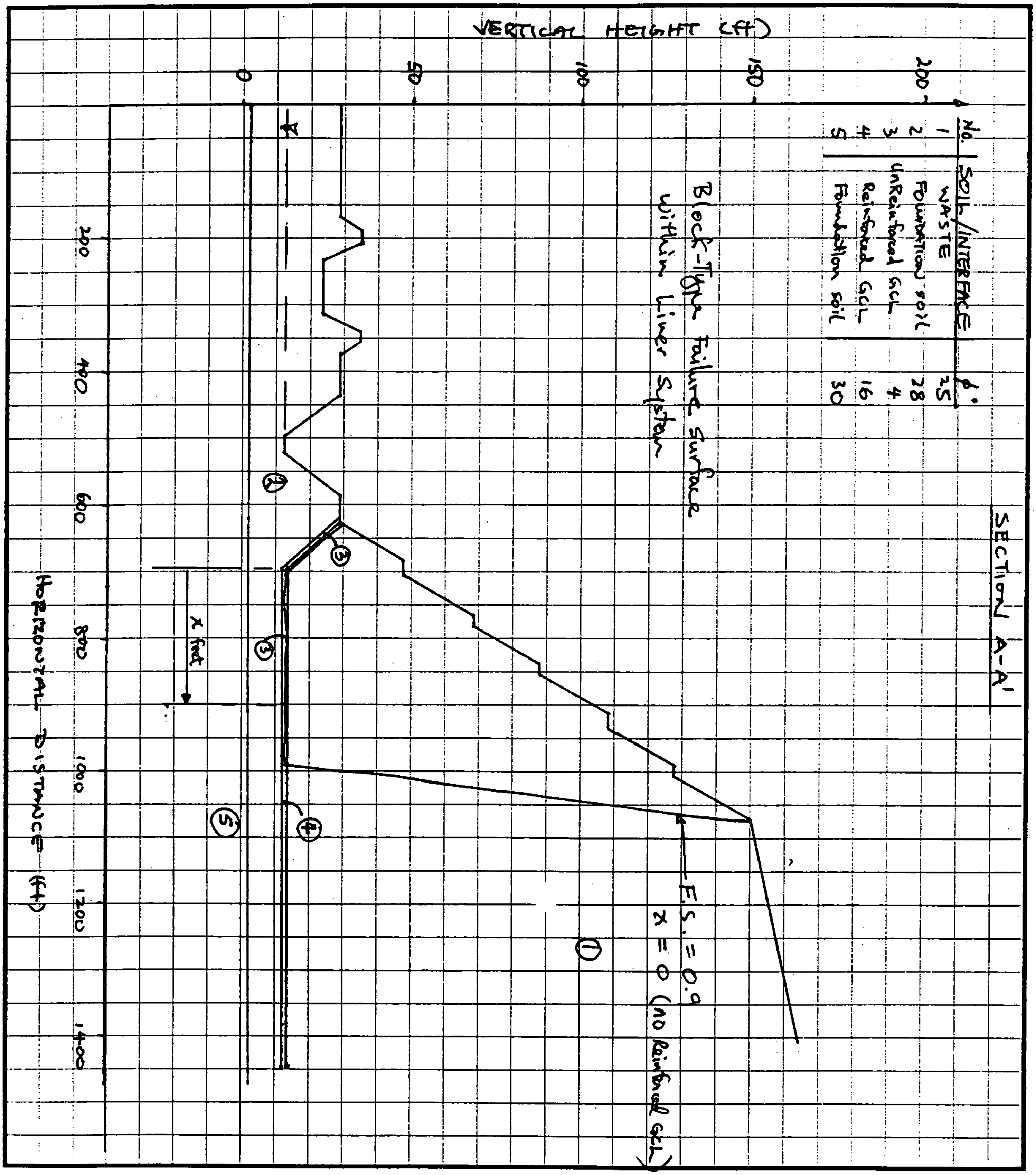

** PCSTABL5M **

by
Purdue University

--Slope Stability Analysis--
Simplified Janbu, Simplified Bishop
or Spencer`s Method of Slices

Run Date: 4-3-97
Time of Run:
Run By: hjl
Input Data Filename: a:\tf15a.i
Output Filename: a:\tf15a.o

PROBLEM DESCRIPTION Tomaka Farms Road Landfill - North Cell
Final Buildout - East/West X-Section

Block type Failure Surface - Unreinforced GCL used at the base

BOUNDARY COORDINATES

26 Top Boundaries
35 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	.00	229.00	170.00	229.00	2
2	170.00	229.00	190.00	235.00	2
3	190.00	235.00	205.00	235.00	2
4	205.00	235.00	235.00	225.00	2
5	235.00	225.00	315.00	225.00	2
6	315.00	225.00	345.00	235.00	2
7	345.00	235.00	355.00	235.00	2
8	355.00	235.00	375.00	229.00	2
9	375.00	229.00	440.00	229.00	2
10	440.00	229.00	508.00	212.00	2
11	508.00	212.00	528.00	212.00	2
12	528.00	212.00	596.00	229.00	2
13	596.00	229.00	630.00	229.00	2
14	630.00	229.00	690.00	249.00	1
15	690.00	249.00	705.00	249.00	1
16	705.00	249.00	765.00	269.00	1
17	765.00	269.00	780.00	269.00	1
18	780.00	269.00	840.00	289.00	1
19	840.00	289.00	855.00	289.00	1
20	855.00	289.00	915.00	309.00	1
21	915.00	309.00	930.00	309.00	1
22	930.00	309.00	990.00	329.00	1
23	990.00	329.00	1005.00	329.00	1
24	1005.00	329.00	1065.00	349.00	1
25	1065.00	349.00	1200.00	355.00	1

26	1200.00	355.00	1335.00	349.00	1
27	677.00	214.00	1200.00	214.00	3
28	630.00	229.00	675.00	214.00	4
29	675.00	214.00	677.00	214.00	4
30	677.00	214.00	677.10	213.80	4
31	630.00	229.00	630.40	228.80	2
32	630.40	228.80	675.00	213.80	2
33	675.00	213.80	677.10	213.80	2
34	677.10	213.80	1200.00	213.80	2
35	.00	203.00	1200.00	203.00	5

ISOTROPIC SOIL PARAMETERS

5 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	56.0	56.0	.0	25.0	.00	.0	1
2	92.0	102.0	.0	28.0	.00	.0	1
3	58.6	58.6	.0	4.0	.00	.0	1
4	58.6	58.6	.0	16.0	.00	.0	1
5	105.0	115.0	.0	30.0	.00	.0	1

1

1 PIEZOMETRIC SURFACE(S) HAVE BEEN SPECIFIED

Unit Weight of Water = 62.40

Piezometric Surface No. 1 Specified by 2 Coordinate Points

Point No.	X-Water (ft)	Y-Water (ft)
1	.00	212.00
2	1335.00	212.00

Janbus Empirical Coef is being used for the case of c=0

1

A Critical Failure Surface Searching Method, Using A Random Technique For Generating Sliding Block Surfaces, Has Been Specified.

100 Trial Surfaces Have Been Generated.

3 Boxes Specified For Generation Of Central Block Base

Length Of Line Segments For Active And Passive Portions Of
Sliding Block Is 25.0

Box No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Height (ft)
1	630.20	228.90	674.90	213.90	.10
2	675.00	213.90	675.00	213.90	.10
3	700.00	213.90	1000.00	213.90	.10

Factor Of Safety Calculation Has Gone Through Ten Iterations

The Trial Failure Surface In Question Is Defined
By The Following 6 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	644.85	233.95
2	660.44	218.79
3	675.00	213.91
4	709.28	213.91
5	709.76	238.90
6	718.55	253.52

Factor Of Safety For The Preceding Specified Surface = 14.904

1

Following Are Displayed The Ten Most Critical Of The Trial
Failure Surfaces Examined. They Are Ordered - Most Critical
First.

* * Safety Factors Are Calculated By The Modified Janbu Method * *

Failure Surface Specified By 11 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	632.63	229.88
2	635.57	227.09
3	675.00	213.87
4	984.16	213.89
5	1000.10	233.15

6	1013.84	254.03
7	1029.44	273.57
8	1037.35	297.28
9	1052.43	317.23
10	1065.36	338.62
11	1075.60	349.47

*** .855 ***

Individual data on the 25 slices

Slice No.	Width Ft(m)	Weight Lbs(kg)	Water Force Top Lbs(kg)	Water Force Bot Lbs(kg)	Tie Force Norm Lbs(kg)	Tie Force Tan Lbs(kg)	Earthquake Force Hor Lbs(kg)	Earthquake Force Ver Lbs(kg)	Surcharge Load Lbs(kg)
1	2.8	290.9	.0	.0	.0	.0	.0	.0	.0
2	.1	19.1	.0	.0	.0	.0	.0	.0	.0
3	39.4	37437.9	.0	.0	.0	.0	.0	.0	.0
4	2.0	3412.6	.0	.0	.0	.0	.0	.0	.0
5	.1	112.5	.0	.0	.0	.0	.0	.0	.0
6	12.9	23889.3	.0	.0	.0	.0	.0	.0	.0
7	15.0	29513.7	.0	.0	.0	.0	.0	.0	.0
8	60.0	151647.7	.0	.0	.0	.0	.0	.0	.0
9	15.0	46310.2	.0	.0	.0	.0	.0	.0	.0
10	60.0	218833.8	.0	.0	.0	.0	.0	.0	.0
11	15.0	63106.7	.0	.0	.0	.0	.0	.0	.0
12	60.0	286019.8	.0	.0	.0	.0	.0	.0	.0
13	15.0	79903.2	.0	.0	.0	.0	.0	.0	.0
14	54.2	315884.3	.0	.0	.0	.0	.0	.0	.0
15	.1	598.2	.0	.0	.0	.0	.0	.0	.0
16	5.7	35568.0	.0	.0	.0	.0	.0	.0	.0
17	10.1	57650.1	.0	.0	.0	.0	.0	.0	.0
18	4.9	25293.4	.0	.0	.0	.0	.0	.0	.0
19	8.8	41179.3	.0	.0	.0	.0	.0	.0	.0
20	15.6	61792.1	.0	.0	.0	.0	.0	.0	.0
21	7.9	23509.5	.0	.0	.0	.0	.0	.0	.0
22	15.1	29575.8	.0	.0	.0	.0	.0	.0	.0
23	12.6	13574.0	.0	.0	.0	.0	.0	.0	.0
24	.4	216.1	.0	.0	.0	.0	.0	.0	.0
25	10.2	2978.4	.0	.0	.0	.0	.0	.0	.0

Failure Surface Specified By 11 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	629.91	229.00
2	630.24	228.89
3	675.00	213.85
4	976.54	213.92
5	994.00	231.82
6	1011.60	249.58
7	1026.20	269.87
8	1036.06	292.84

9	1046.60	315.51
10	1063.61	333.83
11	1074.25	349.41

*** .862 ***

Failure Surface Specified By 11 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	632.34	229.78
2	634.73	227.40
3	675.00	213.87
4	925.79	213.93
5	942.18	232.81
6	958.32	251.89
7	974.37	271.07
8	990.28	290.35
9	1007.11	308.83
10	1019.74	330.41
11	1024.82	335.61

*** .872 ***

Failure Surface Specified By 11 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	633.57	230.19
2	637.24	226.55
3	675.00	213.87
4	987.16	213.89
5	997.56	236.62
6	1015.20	254.34
7	1029.53	274.83
8	1045.38	294.16
9	1050.45	318.64
10	1067.58	336.85
11	1073.16	349.36

*** .879 ***

Failure Surface Specified By 11 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	639.76	232.25
2	649.75	222.38
3	675.00	213.90
4	952.81	213.94
5	969.63	232.44
6	986.55	250.85
7	1003.71	269.02
8	1015.24	291.21
9	1032.29	309.48
10	1048.12	328.84
11	1060.03	347.34

*** .886 ***

Failure Surface Specified By 11 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	618.75	229.00
2	640.63	225.40
3	675.00	213.94
4	961.90	213.85
5	979.20	231.89
6	996.01	250.40
7	1008.06	272.30
8	1015.41	296.20
9	1031.81	315.07
10	1049.33	332.90
11	1049.71	343.90

*** .888 ***

Failure Surface Specified By 10 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	630.90	229.30
2	636.25	226.82
3	675.00	213.86
4	950.10	213.87
5	960.88	236.43

6	977.97	254.67
7	990.17	276.50
8	1000.97	299.04
9	1006.82	323.35
10	1006.86	329.62

*** .892 ***

Failure Surface Specified By 10 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	627.91	229.00
2	642.81	224.64
3	675.00	213.94
4	910.92	213.94
5	924.27	235.08
6	940.07	254.44
7	957.58	272.29
8	974.53	290.67
9	987.10	312.28
10	987.15	328.05

*** .894 ***

Failure Surface Specified By 12 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	641.28	232.76
2	644.81	229.57
3	666.28	216.76
4	675.00	213.93
5	962.92	213.85
6	975.85	235.25
7	991.85	254.46
8	1006.49	274.72
9	1024.15	292.42
10	1036.34	314.25
11	1043.21	338.29
12	1048.32	343.44

*** .896 ***

Failure Surface Specified By 10 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	631.01	229.34
2	640.19	225.55
3	675.00	213.91
4	912.09	213.85
5	926.16	234.52
6	941.16	254.52
7	958.71	272.33
8	968.46	295.35
9	974.08	319.71
10	979.84	325.61

*** .898 ***

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                Y           A     X     I     S           F     T

                .00     166.88     333.75     500.63     667.50     834.38

X        .00 +---------+-*W*-----+---------+---------+---------+
             -
             -
             -
             -
             -
      166.88 +             *
             -             *
             -            *
             -
             -
             -            *
A     333.75 +             *
             -             *
             -
             -
             -             *
             -
X     500.63 +            *
             -            *
             -
             -             *
             -            .6
             -            5*
I     667.50 +            *.
             -            .  *
             -            ...
             -            ....
             -            ...*
             -            .....
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S    834.38 +             ....*
            -             ....*.
            -             ......
            -             38....*
            -             5338...
            -             15538088
    1001.25 +             .115373*
            -               415153
            -                  411*
            -                     1
            -
            -
F   1168.13 +
            -            **       *
            -
            -
            -
            -
T   1335.00 +             W       *
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SCS ENGINEERS

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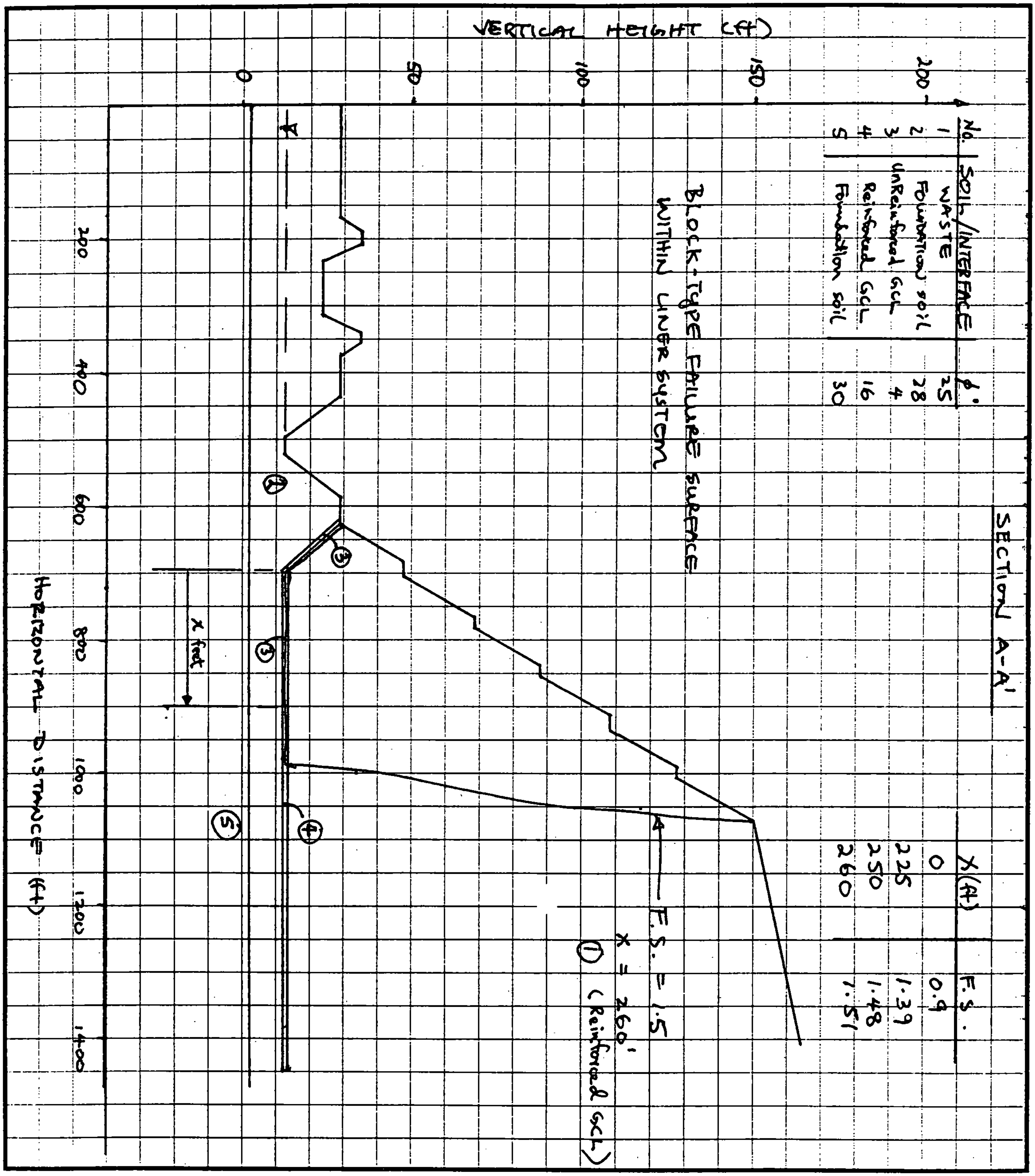
JOB ______
SHEET NO. ______ OF ______
CALCULATED BY ______ DATE ______
CHECKED BY ______ DATE ______
SCALE ______



** PCSTABL5M **

by
Purdue University

--Slope Stability Analysis--
Simplified Janbu, Simplified Bishop
or Spencer`s Method of Slices

Run Date: 4-1-97
Time of Run:
Run By: hjl
Input Data Filename: a:\tf55.i
Output Filename: a:\tf55.o

PROBLEM DESCRIPTION Tomaka Farms Road Landfill - North Cell
Final Buildout - East/West X-Section

Block type failure. x = 260' of reinforced GCL used at the base.

BOUNDARY COORDINATES

26 Top Boundaries
35 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	.00	229.00	170.00	229.00	2
2	170.00	229.00	190.00	235.00	2
3	190.00	235.00	205.00	235.00	2
4	205.00	235.00	235.00	225.00	2
5	235.00	225.00	315.00	225.00	2
6	315.00	225.00	345.00	235.00	2
7	345.00	235.00	355.00	235.00	2
8	355.00	235.00	375.00	229.00	2
9	375.00	229.00	440.00	229.00	2
10	440.00	229.00	508.00	212.00	2
11	508.00	212.00	528.00	212.00	2
12	528.00	212.00	596.00	229.00	2
13	596.00	229.00	630.00	229.00	2
14	630.00	229.00	690.00	249.00	1
15	690.00	249.00	705.00	249.00	1
16	705.00	249.00	765.00	269.00	1
17	765.00	269.00	780.00	269.00	1
18	780.00	269.00	840.00	289.00	1
19	840.00	289.00	855.00	289.00	1
20	855.00	289.00	915.00	309.00	1
21	915.00	309.00	930.00	309.00	1
22	930.00	309.00	990.00	329.00	1
23	990.00	329.00	1005.00	329.00	1
24	1005.00	329.00	1065.00	349.00	1
25	1065.00	349.00	1200.00	355.00	1

26	1200.00	355.00	1335.00	349.00	1
27	935.00	214.00	1200.00	214.00	3
28	630.00	229.00	675.00	214.00	4
29	675.00	214.00	935.00	214.00	4
30	935.00	214.00	935.10	213.80	4
31	630.00	229.00	630.40	228.80	2
32	630.40	228.80	675.00	213.80	2
33	675.00	213.80	935.10	213.80	2
34	935.10	213.80	1200.00	213.80	2
35	.00	203.00	1200.00	203.00	5

ISOTROPIC SOIL PARAMETERS

5 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	56.0	56.0	.0	25.0	.00	.0	1
2	92.0	102.0	.0	28.0	.00	.0	1
3	58.6	58.6	.0	4.0	.00	.0	1
4	58.6	58.6	.0	16.0	.00	.0	1
5	105.0	115.0	.0	30.0	.00	.0	1

1

1 PIEZOMETRIC SURFACE(S) HAVE BEEN SPECIFIED

Unit Weight of Water = 62.40

Piezometric Surface No. 1 Specified by 2 Coordinate Points

Point No.	X-Water (ft)	Y-Water (ft)
1	.00	212.00
2	1335.00	212.00

Janbus Empirical Coef is being used for the case of c=0

1

A Critical Failure Surface Searching Method, Using A Random Technique For Generating Sliding Block Surfaces, Has Been Specified.

100 Trial Surfaces Have Been Generated.

3 Boxes Specified For Generation Of Central Block Base

Length Of Line Segments For Active And Passive Portions Of Sliding Block Is 25.0

Box No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Height (ft)
1	630.20	228.90	674.90	213.90	.10
2	675.00	213.90	675.00	213.90	.10
3	700.00	213.90	1000.00	213.90	.10

Factor Of Safety Calculation Has Gone Through Ten Iterations

The Trial Failure Surface In Question Is Defined By The Following 6 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	644.85	233.95
2	660.44	218.79
3	675.00	213.91
4	709.28	213.91
5	709.76	238.90
6	718.55	253.52

Factor Of Safety For The Preceding Specified Surface = 16.440

1

Following Are Displayed The Ten Most Critical Of The Trial Failure Surfaces Examined. They Are Ordered - Most Critical First.

* * Safety Factors Are Calculated By The Modified Janbu Method * *

Failure Surface Specified By 11 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	632.63	229.88
2	635.57	227.09
3	675.00	213.87
4	984.16	213.89
5	1000.10	233.15

6	1013.84	254.03
7	1029.44	273.57
8	1037.35	297.28
9	1052.43	317.23
10	1065.36	338.62
11	1075.60	349.47

*** 1.514 ***

Individual data on the 25 slices

Slice No.	Width Ft(m)	Weight Lbs(kg)	Water Force Top Lbs(kg)	Water Force Bot Lbs(kg)	Tie Force Norm Lbs(kg)	Tie Force Tan Lbs(kg)	Earthquake Force Hor Lbs(kg)	Earthquake Force Ver Lbs(kg)	Surcharge Load Lbs(kg)
1	2.8	290.9	.0	.0	.0	.0	.0	.0	.0
2	.1	19.1	.0	.0	.0	.0	.0	.0	.0
3	39.4	37437.9	.0	.0	.0	.0	.0	.0	.0
4	15.0	27414.4	.0	.0	.0	.0	.0	.0	.0
5	15.0	29513.7	.0	.0	.0	.0	.0	.0	.0
6	60.0	151647.7	.0	.0	.0	.0	.0	.0	.0
7	15.0	46310.2	.0	.0	.0	.0	.0	.0	.0
8	60.0	218833.8	.0	.0	.0	.0	.0	.0	.0
9	15.0	63106.7	.0	.0	.0	.0	.0	.0	.0
10	60.0	286019.8	.0	.0	.0	.0	.0	.0	.0
11	15.0	79903.2	.0	.0	.0	.0	.0	.0	.0
12	5.0	26867.6	.0	.0	.0	.0	.0	.0	.0
13	.1	316.3	.0	.0	.0	.0	.0	.0	.0
14	49.1	288700.5	.0	.0	.0	.0	.0	.0	.0
15	.1	598.2	.0	.0	.0	.0	.0	.0	.0
16	5.7	35568.0	.0	.0	.0	.0	.0	.0	.0
17	10.1	57650.1	.0	.0	.0	.0	.0	.0	.0
18	4.9	25293.4	.0	.0	.0	.0	.0	.0	.0
19	8.8	41179.3	.0	.0	.0	.0	.0	.0	.0
20	15.6	61792.1	.0	.0	.0	.0	.0	.0	.0
21	7.9	23509.5	.0	.0	.0	.0	.0	.0	.0
22	15.1	29575.8	.0	.0	.0	.0	.0	.0	.0
23	12.6	13574.0	.0	.0	.0	.0	.0	.0	.0
24	.4	216.1	.0	.0	.0	.0	.0	.0	.0
25	10.2	2978.4	.0	.0	.0	.0	.0	.0	.0

Failure Surface Specified By 11 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	629.91	229.00
2	630.24	228.89
3	675.00	213.85
4	976.54	213.92
5	994.00	231.82
6	1011.60	249.58
7	1026.20	269.87
8	1036.06	292.84

9	1046.60	315.51
10	1063.61	333.83
11	1074.25	349.41

*** 1.515 ***

Failure Surface Specified By 11 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	633.57	230.19
2	637.24	226.55
3	675.00	213.87
4	987.16	213.89
5	997.56	236.62
6	1015.20	254.34
7	1029.53	274.83
8	1045.38	294.16
9	1050.45	318.64
10	1067.58	336.85
11	1073.16	349.36

*** 1.549 ***

Failure Surface Specified By 11 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	639.76	232.25
2	649.75	222.38
3	675.00	213.90
4	952.81	213.94
5	969.63	232.44
6	986.55	250.85
7	1003.71	269.02
8	1015.24	291.21
9	1032.29	309.48
10	1048.12	328.84
11	1060.03	347.34

*** 1.577 ***

1

Failure Surface Specified By 11 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	641.03	232.68
2	654.53	220.70
3	675.00	213.94
4	973.18	213.88
5	990.64	231.77
6	1001.90	254.09
7	1010.68	277.50
8	1028.26	295.28
9	1043.67	314.97
10	1058.01	335.44
11	1071.79	349.30

*** 1.583 ***

Failure Surface Specified By 12 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	641.28	232.76
2	644.81	229.57
3	666.28	216.76
4	675.00	213.93
5	962.92	213.85
6	975.85	235.25
7	991.85	254.46
8	1006.49	274.72
9	1024.15	292.42
10	1036.34	314.25
11	1043.21	338.29
12	1048.32	343.44

*** 1.603 ***

1

Failure Surface Specified By 11 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	618.75	229.00
2	640.63	225.40
3	675.00	213.94
4	961.90	213.85

5	979.20	231.89
6	996.01	250.40
7	1008.06	272.30
8	1015.41	296.20
9	1031.81	315.07
10	1049.33	332.90
11	1049.71	343.90

*** 1.608 ***

Failure Surface Specified By 10 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	629.71	229.00
2	631.27	228.54
3	675.00	213.94
4	868.59	213.89
5	885.65	232.16
6	902.70	250.44
7	919.24	269.19
8	934.53	288.97
9	950.56	308.15
10	954.14	317.05

*** 1.633 ***

Failure Surface Specified By 11 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	620.68	229.00
2	623.62	226.50
3	648.35	222.80
4	675.00	213.94
5	979.69	213.94
6	995.25	233.51
7	1002.32	257.49
8	1019.88	275.28
9	1031.02	297.66
10	1044.90	318.46
11	1046.70	342.90

*** 1.639 ***

Failure Surface Specified By 11 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	632.34	229.78
2	634.73	227.40
3	675.00	213.87
4	925.79	213.93
5	942.18	232.81
6	958.32	251.89
7	974.37	271.07
8	990.28	290.35
9	1007.11	308.83
10	1019.74	330.41
11	1024.82	335.61

*** 1.640 ***

1

```
           Y           A     X     I     S            F     T

            .00   166.88    333.75    500.63    667.50    834.38

X       .00 +---------+-*W*-----+---------+---------+---------+
            -
            -
            -
            -
            -
     166.88 +             *
            -             *
            -            *
            -
            -
            -            *
A    333.75 +             *
            -             *
            -
            -
            -             *
            -
X    500.63 +            *
            -            *
            -
            -             *
            -            .7
            -            4*
I    667.50 +            *.
            -            .  *
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            -            ...*
            -            .....
S    834.38 +            ....*
            -            8...*.
            -            .88...
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            -            *00.888
            -            1440....
    1001.25 +            .1145.0*
            -              314140
            -                 311*
            -                    1
            -
            -
F   1168.13 +
            -           **       *
            -
            -
            -
            -
T   1335.00 +            W       *
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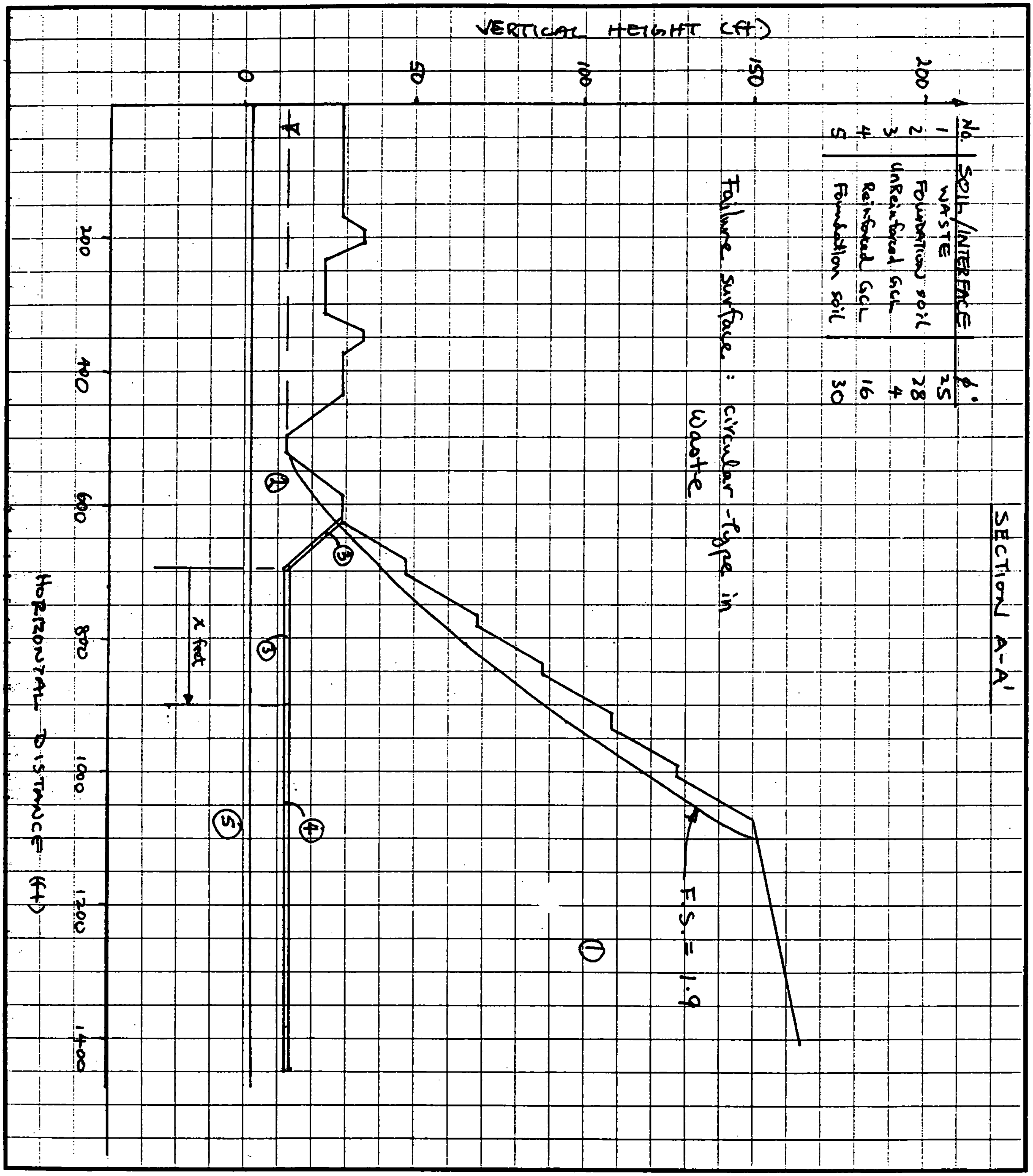
JOB ____________________
SHEET NO. ____________ OF ____________
CALCULATED BY ____________ DATE ____________
CHECKED BY ____________ DATE ____________
SCALE ____________________



** PCSTABL5M **

by
Purdue University

--Slope Stability Analysis--
Simplified Janbu, Simplified Bishop
or Spencer`s Method of Slices

Run Date: 4-1-97
Time of Run:
Run By: hjl
Input Data Filename: a:\tf85.i
Output Filename: a:\tf85.o

PROBLEM DESCRIPTION Tomaka Farms Road Landfill - North Cell
Final Buildout - East/West X-Section

CIRCULAR SURFACE, NEAR SURFACE OF SIDE SLOPE

BOUNDARY COORDINATES

26 Top Boundaries
35 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	.00	229.00	170.00	229.00	2
2	170.00	229.00	190.00	235.00	2
3	190.00	235.00	205.00	235.00	2
4	205.00	235.00	235.00	225.00	2
5	235.00	225.00	315.00	225.00	2
6	315.00	225.00	345.00	235.00	2
7	345.00	235.00	355.00	235.00	2
8	355.00	235.00	375.00	229.00	2
9	375.00	229.00	440.00	229.00	2
10	440.00	229.00	508.00	212.00	2
11	508.00	212.00	528.00	212.00	2
12	528.00	212.00	596.00	229.00	2
13	596.00	229.00	630.00	229.00	2
14	630.00	229.00	690.00	249.00	1
15	690.00	249.00	705.00	249.00	1
16	705.00	249.00	765.00	269.00	1
17	765.00	269.00	780.00	269.00	1
18	780.00	269.00	840.00	289.00	1
19	840.00	289.00	855.00	289.00	1
20	855.00	289.00	915.00	309.00	1
21	915.00	309.00	930.00	309.00	1
22	930.00	309.00	990.00	329.00	1
23	990.00	329.00	1005.00	329.00	1
24	1005.00	329.00	1065.00	349.00	1
25	1065.00	349.00	1200.00	355.00	1

26	1200.00	355.00	1335.00	349.00	1
27	935.00	214.00	1200.00	214.00	3
28	630.00	229.00	675.00	214.00	4
29	675.00	214.00	935.00	214.00	4
30	935.00	214.00	935.10	213.80	4
31	630.00	229.00	630.40	228.80	2
32	630.40	228.80	675.00	213.80	2
33	675.00	213.80	935.10	213.80	2
34	935.10	213.80	1200.00	213.80	2
35	.00	203.00	1200.00	203.00	5

ISOTROPIC SOIL PARAMETERS

5 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	56.0	56.0	.0	25.0	.00	.0	1
2	92.0	102.0	.0	28.0	.00	.0	1
3	58.6	58.6	.0	4.0	.00	.0	1
4	58.6	58.6	.0	16.0	.00	.0	1
5	105.0	115.0	.0	30.0	.00	.0	1

1

1 PIEZOMETRIC SURFACE(S) HAVE BEEN SPECIFIED

Unit Weight of Water = 62.40

Piezometric Surface No. 1 Specified by 2 Coordinate Points

Point No.	X-Water (ft)	Y-Water (ft)
1	.00	212.00
2	1335.00	212.00

A Critical Failure Surface Searching Method, Using A Random Technique For Generating Circular Surfaces, Has Been Specified.

Janbus Empirical Coef. is being used for the case of c=0
800 Trial Surfaces Have Been Generated.

20 Surfaces Initiate From Each Of 40 Points Equally Spaced Along The Ground Surface Between X = 500.00 ft.
and X = 550.00 ft.

Each Surface Terminates Between X =1100.00 ft.
and X =1330.00 ft.

Unless Further Limitations Were Imposed, The Minimum Elevation At Which A Surface Extends Is Y = .00 ft.

40.00 ft. Line Segments Define Each Trial Failure Surface.

Following Are Displayed The Ten Most Critical Of The Trial Failure Surfaces Examined. They Are Ordered - Most Critical First.

* * Safety Factors Are Calculated By The Modified Janbu Method * *

Failure Surface Specified By 16 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	533.33	213.33
2	572.89	219.30
3	612.36	225.77
4	651.75	232.73
5	691.05	240.18
6	730.25	248.12
7	769.35	256.55
8	808.35	265.48
9	847.22	274.89
10	885.98	284.79
11	924.61	295.17
12	963.10	306.03
13	1001.46	317.38
14	1039.67	329.21
15	1077.73	341.51
16	1105.23	350.79

*** 1.883 ***

Individual data on the 30 slices

Slice No.	Width Ft(m)	Weight Lbs(kg)	Water Force Top Lbs(kg)	Water Force Bot Lbs(kg)	Tie Force Norm Lbs(kg)	Tie Force Tan Lbs(kg)	Earthquake Force Hor Lbs(kg)	Earthquake Force Ver Lbs(kg)	Surcharge Load Lbs(kg)

1	39.6	7130.2	.0	.0	.0	.0	.0	.0	.0
2	23.1	10453.3	.0	.0	.0	.0	.0	.0	.0
3	16.4	6882.7	.0	.0	.0	.0	.0	.0	.0
4	17.6	2718.8	.0	.0	.0	.0	.0	.0	.0
5	.2	1.6	.0	.0	.0	.0	.0	.0	.0
6	.1	.5	.0	.0	.0	.0	.0	.0	.0
7	21.5	2216.4	.0	.0	.0	.0	.0	.0	.0
8	38.2	13438.4	.0	.0	.0	.0	.0	.0	.0
9	1.0	524.6	.0	.0	.0	.0	.0	.0	.0
10	14.0	5788.9	.0	.0	.0	.0	.0	.0	.0
11	25.3	10814.9	.0	.0	.0	.0	.0	.0	.0
12	34.7	22068.6	.0	.0	.0	.0	.0	.0	.0
13	4.4	3149.2	.0	.0	.0	.0	.0	.0	.0
14	10.6	6693.8	.0	.0	.0	.0	.0	.0	.0
15	28.3	18239.8	.0	.0	.0	.0	.0	.0	.0
16	31.7	25553.6	.0	.0	.0	.0	.0	.0	.0
17	7.2	6061.6	.0	.0	.0	.0	.0	.0	.0
18	7.8	5713.2	.0	.0	.0	.0	.0	.0	.0
19	31.0	23130.0	.0	.0	.0	.0	.0	.0	.0
20	29.0	25152.4	.0	.0	.0	.0	.0	.0	.0
21	9.6	8136.6	.0	.0	.0	.0	.0	.0	.0
22	5.4	3946.4	.0	.0	.0	.0	.0	.0	.0
23	33.1	24386.1	.0	.0	.0	.0	.0	.0	.0
24	26.9	21845.6	.0	.0	.0	.0	.0	.0	.0
25	11.5	8544.2	.0	.0	.0	.0	.0	.0	.0
26	3.5	2194.1	.0	.0	.0	.0	.0	.0	.0
27	34.7	21231.8	.0	.0	.0	.0	.0	.0	.0
28	25.3	16275.3	.0	.0	.0	.0	.0	.0	.0
29	12.7	7006.4	.0	.0	.0	.0	.0	.0	.0
30	27.5	6199.4	.0	.0	.0	.0	.0	.0	.0

Failure Surface Specified By 16 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	541.03	215.26
2	580.89	218.54
3	620.67	222.71
4	660.36	227.74
5	699.92	233.65
6	739.34	240.42
7	778.60	248.06
8	817.69	256.56
9	856.58	265.91
10	895.26	276.12
11	933.70	287.17
12	971.89	299.07
13	1009.81	311.80
14	1047.44	325.36
15	1084.76	339.75
16	1112.40	351.11

*** 1.920 ***

Failure Surface Specified By 16 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	538.46	214.62
2	578.39	216.96
3	618.25	220.34
4	658.00	224.77
5	697.63	230.24
6	737.10	236.74
7	776.38	244.28
8	815.45	252.84
9	854.29	262.42
10	892.86	273.01
11	931.14	284.61
12	969.11	297.21
13	1006.73	310.79
14	1043.98	325.36
15	1080.84	340.90
16	1102.41	350.66

*** 1.933 ***

Failure Surface Specified By 17 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	550.00	217.50
2	589.69	222.46
3	629.31	227.97
4	668.85	234.04
5	708.29	240.67
6	747.64	247.85
7	786.89	255.58
8	826.02	263.86
9	865.04	272.69
10	903.92	282.06
11	942.67	291.98
12	981.28	302.44
13	1019.74	313.45
14	1058.04	324.99
15	1096.17	337.07
16	1134.13	349.68
17	1142.01	352.42

*** 1.939 ***

Failure Surface Specified By 17 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	537.18	214.29
2	577.14	216.15
3	617.03	219.05
4	656.84	223.00
5	696.52	227.98
6	736.07	234.00
7	775.44	241.06
8	814.62	249.14
9	853.57	258.24
10	892.27	268.36
11	930.69	279.49
12	968.80	291.62
13	1006.59	304.75
14	1044.02	318.85
15	1081.07	333.94
16	1117.71	349.98
17	1120.90	351.48

*** 1.964 ***

Failure Surface Specified By 17 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	532.05	213.01
2	572.02	214.63
3	611.93	217.28
4	651.76	220.97
5	691.48	225.68
6	731.07	231.43
7	770.49	238.21
8	809.72	246.00
9	848.74	254.81
10	887.52	264.63
11	926.02	275.45
12	964.24	287.27
13	1002.13	300.08
14	1039.68	313.86
15	1076.86	328.62
16	1113.65	344.33
17	1130.15	351.90

*** 1.993 ***

Failure Surface Specified By 17 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	530.77	212.69
2	570.69	215.24
3	610.54	218.67
4	650.31	222.96
5	689.98	228.13
6	729.52	234.16
7	768.92	241.06
8	808.16	248.82
9	847.22	257.43
10	886.08	266.90
11	924.73	277.22
12	963.14	288.38
13	1001.30	300.38
14	1039.19	313.21
15	1076.78	326.86
16	1114.07	341.34
17	1140.72	352.37

*** 1.995 ***

Failure Surface Specified By 17 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	534.62	213.65
2	574.59	215.03
3	614.52	217.44
4	654.37	220.90
5	694.12	225.40
6	733.73	230.93
7	773.19	237.49
8	812.46	245.07
9	851.53	253.68
10	890.35	263.31
11	928.91	273.94
12	967.18	285.58
13	1005.13	298.21
14	1042.75	311.83
15	1079.99	326.42
16	1116.84	341.98
17	1139.64	352.32

*** 2.007 ***

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Failure Surface Specified By 17 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	541.03	215.26
2	580.97	217.32
3	620.86	220.28
4	660.68	224.13
5	700.39	228.88
6	739.99	234.52
7	779.46	241.04
8	818.77	248.45
9	857.90	256.74
10	896.84	265.90
11	935.56	275.94
12	974.04	286.84
13	1012.27	298.60
14	1050.23	311.22
15	1087.90	324.69
16	1125.25	339.00
17	1160.03	353.22

*** 2.020 ***

Failure Surface Specified By 18 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	532.05	213.01
2	571.68	218.44
3	611.25	224.28
4	650.76	230.52
5	690.21	237.18
6	729.58	244.24
7	768.87	251.71
8	808.09	259.59
9	847.22	267.87
10	886.27	276.56
11	925.22	285.66
12	964.08	295.15
13	1002.83	305.05
14	1041.48	315.35
15	1080.03	326.05
16	1118.46	337.15
17	1156.77	348.64
18	1173.39	353.82

*** 2.035 ***

```
               Y            A     X     I     S           F     T

                 .00     166.88     333.75     500.63     667.50     834.38
X          .00 +---------+-*W*-----+---------+---------+---------+
               -
               -
               -
               -
               -
        166.88 +             *
               -             *
               -            *
               -
               -
               -            *
A       333.75 +             *
               -             *
               -
               -
               -             *
               -
X       500.63 +            *
               -          ..*
               -        ....4
               -       .....1*
               -       .....21
               -      ......6*
I       667.50 +     .......*2
               -    .........1*
               -    .........51
               -   ..........24
               -   ..........51*
               -  ............21
S       834.38 +  ............71*
               -  ............52*
               -  .............31
               -  .............831*
               -  ..........*...2
               -   .............51
       1001.25 +   ..............51*
               -   ...............41
               -    ..............2 *
               -     .............91.
               -     ..............71
               -      ..............4
F      1168.13 +       .............9
               -         ..**.......*
               -          ...........
               -            .........
               -              .......
               -                .....
T      1335.00 +            W      .*
```

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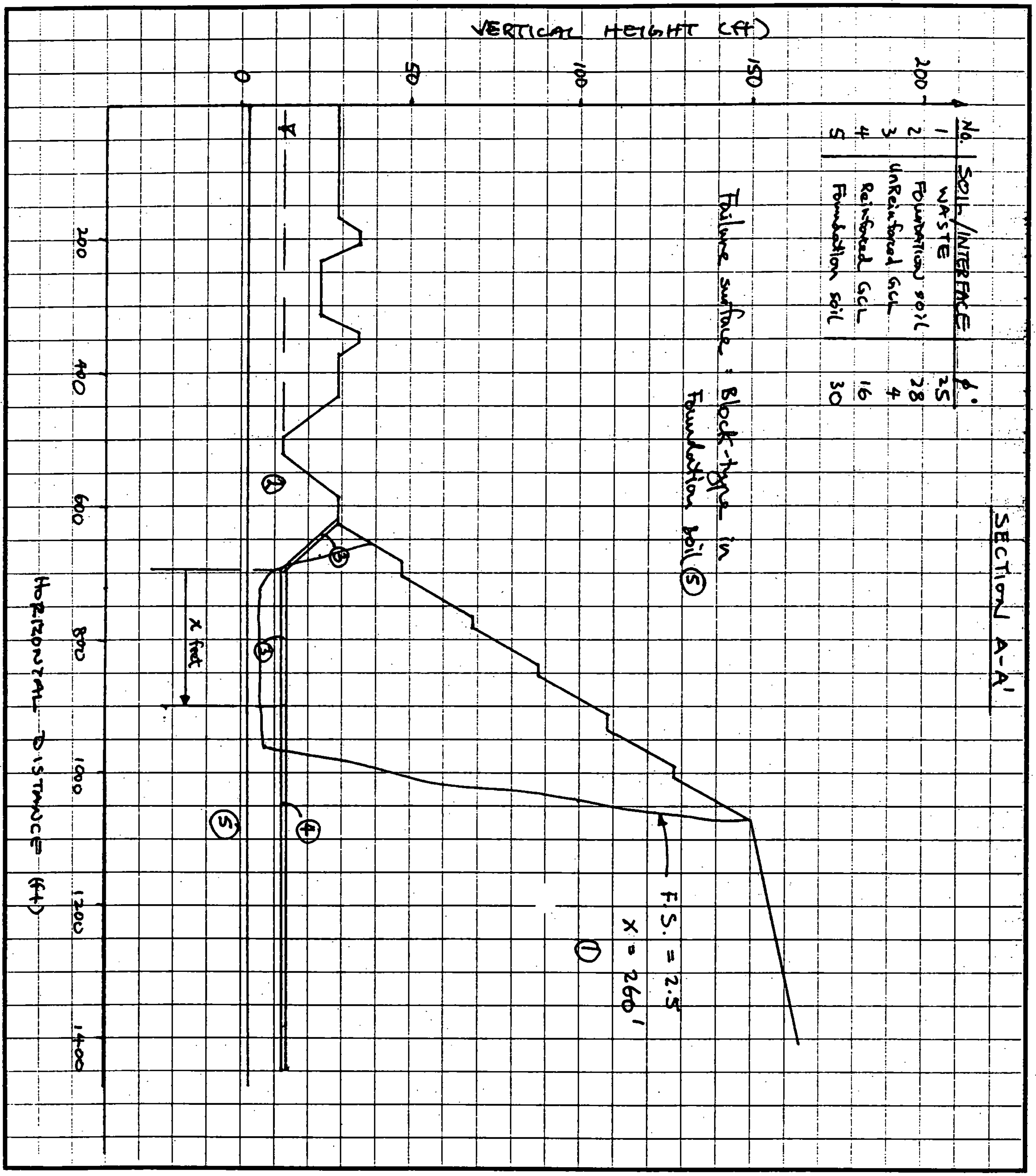
JOB ______________________
SHEET NO. ______________ OF ______
CALCULATED BY ______________ DATE ______
CHECKED BY ______________ DATE ______
SCALE ______________________



Chicago Cincinnati Kansas City Los Angeles New York Phoenix Richmond
San Francisco Seattle Tampa Vancouver, B.C. Virginia Beach Washington, D.C.

** PCSTABL5M **

by
Purdue University

--Slope Stability Analysis--
Simplified Janbu, Simplified Bishop
or Spencer`s Method of Slices

Run Date: 4-1-97
Time of Run:
Run By: hjl
Input Data Filename: a:\tf95.i
Output Filename: a:\tf95.o

PROBLEM DESCRIPTION Tomaka Farms Road Landfill - North Cell
Final Buildout - East/West X-Section

BLOCK FAILURE WITHIN FOUNDATION SOIL

BOUNDARY COORDINATES

26 Top Boundaries
35 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	.00	229.00	170.00	229.00	2
2	170.00	229.00	190.00	235.00	2
3	190.00	235.00	205.00	235.00	2
4	205.00	235.00	235.00	225.00	2
5	235.00	225.00	315.00	225.00	2
6	315.00	225.00	345.00	235.00	2
7	345.00	235.00	355.00	235.00	2
8	355.00	235.00	375.00	229.00	2
9	375.00	229.00	440.00	229.00	2
10	440.00	229.00	508.00	212.00	2
11	508.00	212.00	528.00	212.00	2
12	528.00	212.00	596.00	229.00	2
13	596.00	229.00	630.00	229.00	2
14	630.00	229.00	690.00	249.00	1
15	690.00	249.00	705.00	249.00	1
16	705.00	249.00	765.00	269.00	1
17	765.00	269.00	780.00	269.00	1
18	780.00	269.00	840.00	289.00	1
19	840.00	289.00	855.00	289.00	1
20	855.00	289.00	915.00	309.00	1
21	915.00	309.00	930.00	309.00	1
22	930.00	309.00	990.00	329.00	1
23	990.00	329.00	1005.00	329.00	1
24	1005.00	329.00	1065.00	349.00	1
25	1065.00	349.00	1200.00	355.00	1

26	1200.00	355.00	1335.00	349.00	1
27	935.00	214.00	1200.00	214.00	3
28	630.00	229.00	675.00	214.00	4
29	675.00	214.00	935.00	214.00	4
30	935.00	214.00	935.10	213.80	4
31	630.00	229.00	630.40	228.80	2
32	630.40	228.80	675.00	213.80	2
33	675.00	213.80	935.10	213.80	2
34	935.10	213.80	1200.00	213.80	2
35	.00	203.00	1200.00	203.00	5

1

ISOTROPIC SOIL PARAMETERS

5 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	56.0	56.0	.0	25.0	.00	.0	1
2	92.0	102.0	.0	28.0	.00	.0	1
3	58.6	58.6	.0	4.0	.00	.0	1
4	58.6	58.6	.0	16.0	.00	.0	1
5	105.0	115.0	.0	30.0	.00	.0	1

1

1 PIEZOMETRIC SURFACE(S) HAVE BEEN SPECIFIED

Unit Weight of Water = 62.40

Piezometric Surface No. 1 Specified by 2 Coordinate Points

Point No.	X-Water (ft)	Y-Water (ft)
1	.00	212.00
2	1335.00	212.00

Janbus Empirical Coef is being used for the case of c=0

1

A Critical Failure Surface Searching Method, Using A Random Technique For Generating Sliding Block Surfaces, Has Been Specified.

100 Trial Surfaces Have Been Generated.

3 Boxes Specified For Generation Of Central Block Base

Length Of Line Segments For Active And Passive Portions Of Sliding Block Is 25.0

Box No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Height (ft)
1	650.00	208.00	700.00	208.00	4.00
2	725.00	208.00	725.00	208.00	4.00
3	800.00	208.00	1000.00	208.00	4.00

Factor Of Safety Calculation Has Gone Through Ten Iterations

The Trial Failure Surface In Question Is Defined By The Following 11 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	613.83	229.00
2	613.95	228.89
3	638.08	222.34
4	658.33	207.69
5	725.00	208.98
6	872.99	206.37
7	873.03	231.37
8	875.39	256.26
9	889.40	276.97
10	902.84	298.05
11	904.05	305.35

Factor Of Safety For The Preceding Specified Surface = 6.930

Following Are Displayed The Ten Most Critical Of The Trial Failure Surfaces Examined. They Are Ordered - Most Critical First.

* * Safety Factors Are Calculated By The Modified Janbu Method * *

Failure Surface Specified By 13 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)

1	660.81	239.27
2	675.38	224.99
3	694.60	209.00
4	725.00	206.31
5	964.15	207.17
6	981.66	225.01
7	999.34	242.69
8	1016.36	261.00
9	1033.51	279.19
10	1047.29	300.05
11	1058.42	322.43
12	1070.99	344.05
13	1076.40	349.51

*** 2.505 ***

Individual data on the 31 slices

Slice No.	Width Ft(m)	Weight Lbs(kg)	Water Force Top Lbs(kg)	Water Force Bot Lbs(kg)	Tie Force Norm Lbs(kg)	Tie Force Tan Lbs(kg)	Earthquake Force Hor Lbs(kg)	Earthquake Force Ver Lbs(kg)	Surcharge Load Lbs(kg)
1	14.6	7808.0	.0	.0	.0	.0	.0	.0	.0
2	13.2	19855.7	.0	.0	.0	.0	.0	.0	.0
3	.2	467.0	.0	.0	.0	.0	.0	.0	.0
4	1.2	2342.0	.0	.0	.0	.0	.0	.0	.0
5	1.0	2092.7	.0	.0	.0	.0	.0	.0	.0
6	3.6	8255.1	.0	438.4	.0	.0	.0	.0	.0
7	10.4	25892.1	.0	2252.1	.0	.0	.0	.0	.0
8	20.0	56275.9	.0	6016.4	.0	.0	.0	.0	.0
9	40.0	138268.9	.0	14015.3	.0	.0	.0	.0	.0
10	15.0	57300.0	.0	5163.4	.0	.0	.0	.0	.0
11	60.0	261977.1	.0	20150.4	.0	.0	.0	.0	.0
12	15.0	73688.6	.0	4911.7	.0	.0	.0	.0	.0
13	60.0	327531.5	.0	19143.6	.0	.0	.0	.0	.0
14	15.0	90077.1	.0	4660.0	.0	.0	.0	.0	.0
15	5.0	30240.8	.0	1542.2	.0	.0	.0	.0	.0
16	.1	609.3	.0	30.8	.0	.0	.0	.0	.0
17	29.0	184789.8	.0	8848.1	.0	.0	.0	.0	.0
18	4.7	30458.1	.0	1019.6	.0	.0	.0	.0	.0
19	1.8	10877.9	.0	.0	.0	.0	.0	.0	.0
20	.2	1195.1	.0	.0	.0	.0	.0	.0	.0
21	10.8	63503.8	.0	.0	.0	.0	.0	.0	.0
22	8.3	45967.6	.0	.0	.0	.0	.0	.0	.0
23	9.3	47572.7	.0	.0	.0	.0	.0	.0	.0
24	5.7	26403.0	.0	.0	.0	.0	.0	.0	.0
25	11.4	48336.1	.0	.0	.0	.0	.0	.0	.0
26	17.1	62950.9	.0	.0	.0	.0	.0	.0	.0
27	13.8	39511.2	.0	.0	.0	.0	.0	.0	.0
28	11.1	21012.6	.0	.0	.0	.0	.0	.0	.0
29	6.6	7297.0	.0	.0	.0	.0	.0	.0	.0
30	6.0	3430.5	.0	.0	.0	.0	.0	.0	.0
31	5.4	791.2	.0	.0	.0	.0	.0	.0	.0

Failure Surface Specified By 12 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	630.31	229.10
2	641.46	218.18
3	665.07	209.95
4	725.00	209.05
5	931.19	208.76
6	948.40	226.89
7	965.86	244.79
8	983.22	262.78
9	995.81	284.37
10	1007.20	306.63
11	1024.85	324.34
12	1037.83	339.94

*** 2.535 ***

Failure Surface Specified By 13 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	636.85	231.28
2	654.78	219.56
3	676.41	207.02
4	725.00	206.82
5	977.33	206.54
6	991.55	227.10
7	1004.41	248.54
8	1019.18	268.71
9	1036.02	287.18
10	1053.66	304.90
11	1070.21	323.64
12	1080.76	346.31
13	1084.19	349.85

*** 2.560 ***

Failure Surface Specified By 12 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	655.56	237.52
2	673.41	226.87
3	691.12	209.23

4	725.00	208.62
5	952.58	209.51
6	969.36	228.03
7	982.29	249.43
8	998.30	268.63
9	1012.93	288.90
10	1030.60	306.59
11	1042.78	328.42
12	1046.94	342.98

*** 2.602 ***

1

Failure Surface Specified By 11 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	631.39	229.46
2	637.33	224.57
3	656.37	208.37
4	725.00	206.96
5	864.34	206.35
6	881.80	224.24
7	899.48	241.92
8	915.54	261.08
9	923.66	284.72
10	940.14	303.52
11	953.18	316.73

*** 2.627 ***

Failure Surface Specified By 13 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	617.82	229.00
2	627.57	221.70
3	650.95	212.86
4	675.67	209.09
5	725.00	206.72
6	951.70	208.21
7	967.91	227.24
8	985.54	244.97
9	996.55	267.41
10	1013.23	286.03
11	1019.67	310.19
12	1036.36	328.81
13	1042.17	341.39

*** 2.633 ***

Failure Surface Specified By 14 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	630.22	229.07
2	633.61	225.72
3	654.66	212.24
4	679.50	209.39
5	725.00	208.85
6	962.34	208.29
7	977.56	228.13
8	994.36	246.64
9	1000.71	270.82
10	1018.10	288.79
11	1035.63	306.61
12	1052.47	325.09
13	1068.36	344.39
14	1070.04	349.22

*** 2.638 ***

Failure Surface Specified By 12 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	635.19	230.73
2	648.18	220.59
3	670.71	209.75
4	725.00	208.59
5	964.69	206.40
6	981.25	225.13
7	996.88	244.64
8	1007.47	267.28
9	1018.90	289.52
10	1032.24	310.66
11	1046.04	331.51
12	1058.77	346.92

*** 2.640 ***

1

Failure Surface Specified By 10 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	640.50	232.50
2	662.67	224.20
3	680.38	206.56
4	725.00	206.08
5	822.11	208.08
6	839.42	226.11
7	853.11	247.03
8	868.36	266.85
9	883.16	286.99
10	892.81	301.60

*** 2.672 ***

Failure Surface Specified By 13 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	630.54	229.18
2	635.27	224.68
3	654.64	208.88
4	679.56	206.86
5	725.00	207.55
6	941.49	206.78
7	957.43	226.04
8	971.17	246.92
9	986.77	266.46
10	994.68	290.18
11	1009.76	310.12
12	1022.69	331.52
13	1027.34	336.45

*** 2.674 ***

1

```
              Y           A     X     I     S           F     T

                 .00   166.88    333.75    500.63    667.50    834.38

 X        .00 +---------+-*W*----+---------+---------+---------+
              -
              -
              -
```

```
              -
              -
     166.88   +             *
              -             *
              -            *
              -
              -
              -            *
A    333.75   +             *
              -             *
              -
              -
              -             *
              -
X    500.63   +            *
              -            *
              -
              -            .*
              -            .6
              -           .2*
I    667.50   +           3*1
              -           .1.*
              -           12
              -
              -               *
              -           ...
S    834.38   +           9.9..*
              -           5..99*.
              -           .55..99
              -           .2..55.*
              -           6*2...55
              -           11422....
    1001.25   +           ..314220*
              -             ..11422
              -               ..118*
              -                   .1
              -
              -
F   1168.13   +
              -           **       *
              -
              -
              -
              -
T   1335.00   +            W       *
```

FAX COVER

TO:

DATE: 3/25/97

NAME:

COMPANY NAME:

FAX NUMBER:

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COMMENTS: James Law

CETCO

COLLOID ENVIRONMENTAL TECHNOLOGIES COMPANY

TR-408

TECHNICAL DATA SHEET

410 995-4045

BENTOMAT

DIRECT SHEAR TESTING SUMMARY

Revised 9-30-93

1350 W. Shure Drive • Arlington Heights, Illinois 60004-1440 • (708) 392-5800 • FAX (708) 506-6150

A wholly owned subsidiary of American Colloid Company

SUMMARY OF BENTOMAT DIRECT SHEAR TEST DATA

Lab[1]	Date	Interface[2]	Normal Stresses (psi)	Moisture Condition[3]	Shear Rate	Friction Angle(deg)
J & L	05-30-90	NW/Sand	1 / 2 / 3	Hydrated	0.02 in/min	35
		NW/Sand	1 / 2 / 3	Dry	"	28
		NW/Clay	1 / 2 / 3	Hydrated	"	41
		NW/Clay	1 / 2 / 3	Dry	"	31
STS	09-11-90	NW/40-mil Text. HDPE	35 / 52 / 70	Dry	0.2 in/min	18
		NW/80-mil Text. HDPE	35 / 52 / 70	Dry	"	37
		W/80-mil Text. HDPE	35 / 52 / 70	Dry	"	24
J & L	11-06-90	NW/Sandy Soil	2 / 3.5 / 5	Dry	0.02 in/min	23
GRI	04-18-91	Internal	0.5 / 1 / 2 / 5 / 10 / 20	Dry	0.035 in/min	42
		"	0.12 / 0.5 / 1/ 5 / 10	Hydrated	"	37
		"	0.12 / 0.5 / 1 / 5 /10	Hydrated[4]	"	39
STS	05-28-91	NW/40-mil Text. HDPE	35 / 52 / 70	Hydrated	0.2 in/min	20
		W/80-mil Text. HDPE	35 / 52 / 70	Hydrated	"	19
UTA	8-12-91	Internal	6 / 9 / 14 / 19	Hydrated	0.02 mm/hr	26
J & L	9-9-91	W/Soil Cover	0.6 / 1.25 / 1.88	Hydrated	0.035 in/min	22.5
		W/Geonet	0.6 / 1.25 / 1.88	Hydrated	"	17
		NW/2B Stone	0.6 / 1.25 / 1.88	Hydrated	"	53
TRI	5-6-92	W/60-mil Text. VLDPE	2 / 8 / 14	Hydrated	0.04 in/min	22
		W/60-mil Smooth VLDPE	2 / 8 / 14	Hydrated	"	14
TRI	11-12-92	W/40-mil Text. VLDPE	3.5 / 7 / 14	Hydrated	0.2 in/min	25

SUMMARY OF BENTOMAT DIRECT SHEAR TEST DATA (Continued)

Lab[1]	Date	Interface[2]	Normal Stresses (psi)	Moisture Condition[3]	Shear Rate	Friction Angle(deg)
TRI	3-16-93	WP/Saturated Soil	1 / 2 / 3	Hydrated	0.04 in/min	20
		WP/Dry Soil	1 / 2 / 3	Hydrated	0.04 in/min	22
		NW/Drainage Geocomposite	1 / 2 / 3	Hydrated	0.2 in/min	17.2
GA	9-4-92	W/60-mil Smooth HDPE	0.5 / 1 / 2 / 4 / 10	Hydrated	0.02 in/min	8 ←
		Internal	0.5 / 1 / 2 / 4 / 10	Hydrated	0.0025 in/min	27
		W/Drainage Geocomposite	0.5 / 1 / 2 / 4 / 10	Hydrated	0.02 in/min	21
		W/Textured HDPE	0.5 / 1 / 2 / 4 / 10	Hydrated	0.02 in/min	28
TRI	7-1-93	W/30-mil PVC	1 / 3 / 5	Dry	0.04 in/min	24
		W/30-mil PVC	1 / 3 / 5	Hydrated	0.04 in/min	13
GC	9-28-93	Internal	0.35 / 1 / 2 / 3.5	Dry	0.04 in/min	57
		Internal	0.5 / 1 / 2 / 4 / 10	Hydrated	0.04 in/min	59

Notes:

[1] J & L = J & L Testing Company, Inc., Canonsburg, PA (used a 3-inch Wykeham Farrance direct shear device)
STS = STS Consultants Ltd., Northbrook, IL (used a custom-made 12-inch shear box)
GRI = Geosynthetic Research Institute, Drexel University, Philadelphia, PA (used a Wykeham Farrance device)
UTA = University of Texas at Austin, Civil Engineering Laboratory (used a 2.4-inch direct shear box)
TRI = TRI Environmental, Inc., Austin, Texas (used a 12-inch direct shear box)
GA - Golder Associates, Denver, Colorado (12-inch direct shear box)
GC - GeoSyntec Consultants, Atlanta, Georgia

[2] NW = Non-woven geotextile of Bentomat.
W = Woven geotextile of Bentomat.
WP = Woven geotextile of Bentomat Pink.

[3] "Dry" = sample tested in the as-received moisture state, which is typically 12 percent.
"Hydrated" = sample was hydrated prior to testing, *although the actual hydration methods and durations vary.*
Samples were hydrated with distilled water unless otherwise noted.

[4] Hydrated in leachate.

CETCO

COLLOID ENVIRONMENTAL TECHNOLOGIES COMPANY

TR-408cm
10-13-95

TECHNICAL DATA SHEET

CLAYMAX

DIRECT SHEAR TESTING SUMMARY

NOTE:

This data is for informational purposes only and is not intended to replace project-specifc interface testing, which CETCO emphatically recommends. Variability in this data may be attributed to variability in the test setup and specimen preparation procedures, which has been shown to affect results. For this reason, CETCO makes no warranty as to the usefulness of the data.

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SUMMARY OF CLAYMAX DIRECT SHEAR TEST DATA

Lab[1]	Report Date	Interface Tested[2]	Normal Stresses (psi)	Bentomat Moisture[3]	Shear Rate (in/min)	Peak Friction Angle (deg)	Apparent Cohesion (psf)	Residual Friction Angle (deg)[4]	Apparent Cohesion (psf)
GSC	11-6-92	Internal	0.35 - 1.4 - 2.8 - 4.9 - 6.9	Hydrated	0.04	0	500	Not Determined	
			0.35 - 1.4 - 2.8 - 4.9 - 6.9	Hydrated	0.04	0	570	Not Determined	
GSC	11-24-92	W/60 mil sm. HDPE	0.35 - 1.4 - 2.8	Hydrated	0.04	11	10	11	10
		W/60 mil Text. HDPE	0.35 - 1.4 - 2.8	Hydrated	0.04	35	10	26	20
		W/60 mil Text. VLDPE	0.35 - 1.4 - 2.8	Hydrated	0.04	37	20	28	10
		W/Soaked Sandy Clay	0.35 - 1.4 - 2.8	Hydrated	0.04	25	50	25	50
GSC	11-24-92	W/60-mil sm. HDPE	2.8 - 4.9 - 6.9	Hydrated	0.04	11	10	11	10
		W/60-mil Text. HDPE	2.8 - 4.9 - 6.9	Hydrated	0.04	24	100	11	20
		W/60-mil Text. VLDPE	2.8 - 4.9 - 6.9	Hydrated	0.04	27	100	20	80
		W/Soaked Sandy Clay	2.8 - 4.9 - 6.9	Hydrated	0.04	25	60	25	60
GSC	07-16-93	W/Gecomposite Drain (NSC TN3002-1125)	0.69 - 1.4 - 2.8	Hydrated	0.04	12	30	11	20
GSC	09-21-93	Internal	0.35 - 1.4 - 2.8 - 4.9 - 6.9 - 13.9 - 20.8	Hydrated	0.04	4	485	Not Determined	
		Internal	0.35 - 1.4 - 2.8 - 4.9 - 6.9 - 13.9 - 20.8	Hydrated	0.04	0	565	Not Determined	
GSC	10-21-93	Internal	0.35 - 1.4 - 2.8 - 4.9 - 6.9 - 13.9 - 20.8 - 69.4 - 142.4	Hydrated	0.04	5	485	5	485
JCC	11-93	W/60 mil Text. HDPE	20 - 35	Hydrated	0.04	7.2	277	5.8	157
		Internal	1 - 10 - 50	Hydrated	0.04	14.5	686	10.5	686

SUMMARY OF CLAYMAX DIRECT SHEAR TEST DATA (Continued)

Lab[1]	Report Date	Interface Tested[2]	Normal Stresses (psi)	Bentomat Moisture[3]	Shear Rate (in/min)	Peak Friction Angle (deg)	Apparent Cohesion (psf)	Residual Friction Angle (deg)[4]	Apparent Cohesion (psf)
GSC	3-21-94	Internal	1.4 - 2.8 - 10 - 50 - 145	Hydrated	0.04	5	570	5	570
EMC	03-21-94	W/Ione Clay	14 - 28 - 62.5	Hydrated	0.04	18.4	500	12.5	500
EMC	03-26-94	Internal	13.9 - 27.8 - 62.5	Hydrated	0.04	14	400	9	800
EMC	04-01-94	W/Text. HDPE	13.9 - 27.8 - 62.5	Hydrated	0.04	6.3	600	6	300
GSC	04-02-94	Internal	1.4 - 2.8 - 10 - 50 - 145	Hydrated	0.04	5	570	5	570
JCC	04-04-94	W/60-mil Text. HDPE	1 - 10 - 50	Hydrated	0.04	8.4	110	4.6	110
GSC	08-10-94	Internal	0.69 - 1.4 - 2.1	Hydrated	0.04	34	453	34	453
GSC	08-19-94	W/30-mil sm. PVC	1 - 2 - 3	Hydrated	0.04	20	70	19	75
GSC	08-29-94	W/Site Soil	1 - 2 - 4	Hydrated	0.04	20	53	17	53
JCC	09-06-94	W/40-mil Text. HDPE	1 - 2 - 4	Hydrated	0.04	19	32	19	20
JCC	09-06-94	W/30-mil sm. PVC	1 - 2 - 4	Hydrated	0.04	19	22	19	15
GSC	09-20-94	W/40-mil Text. HDPE	1 - 2 - 4	Hydrated	0.04	20	53	17	53
			1 - 2 - 4	Hydrated	0.04	21	37	17	13
			1 - 2 - 4	Hydrated	0.04	18	5	17	5
AGP	09-21-94	W/Silty Clay	0.69 - 1.4 - 2.1	Hydrated	0.04	38	112	39	100
		W/Slag	0.69 - 1.4 - 2.1	Hydrated	0.04	49	12	47	13
CETCO	08-08-95	Internal	0.5 - 1 - 2 - 4 - 10	Hydrated	0.04	5	608	4	630

GEOSYNTHETIC CLAY LINERS

MUST CONSIDER INTERFACE AND INTERNAL STRENGTH

INTERNAL STRENGTH DEPENDS SIGNIFICANTLY ON SATURATION, CONFINING PRESSURE

- DRY, REINFORCED, 20 PSI: $\phi_r = 25°$
- DRY, REINFORCED, 100 PSI: $\phi_r = 15°$
- SAT., REINFORCED, 20 PSI: $\phi_r = 16°$
- SAT., REINFORCED, 100 PSI: $\phi_r = 8°$
- SAT., UNREINFORCED: $\phi_r = 4°$

INTERFACE SHEAR STRENGTHS

TYPICAL VALUES

Table 1
Summary of Interface-Shear-Strength Tests:
Kettleman Hills Repository

Interface Components	Conditions	Direct Shear Tests Residual Friction Angle: ϕ_r	Pullout-Box Tests Residual Friction Angle: ϕ_r
HDPE liner/geotextile	Dry, unpolished	9.5° to 12.5°	9.5° (1 Test)
	Dry, partly polished	9.0° to 11.0°	---
	Dry, polished	8.5° to 10.5°	8.0° (1 Test)
	Wet, unpolished	8.0° to 10.0°	7.0° to 10.5°
	Wet, polished	7.0 to 9.5°	6.5° to 9.0°
HDPE liner/geonet (transverse shear)	Dry	7.0° to 8.0° +	--
	Submerged	7.0° to 10.0°	8.0° to 9.0°
HDPE liner/geonet (aligned shear)	Submerged	5.0° to 8.0°	6.0° to 8.0°
Geotextile/geonet	Dry	>20°	
	Submerged	10° to 14° (+	
HDPE liner/HDPE liner	Dry	6.0° to 13.0°	
	Submerged	6.0° to 11.0°	

Geo

APPENDIX H

BEARING CAPACITY

Tomoka Farms Road Landfill

Initial Induced Stress = 1500 psf (15 feet of sand @ 100 pcf)

Layer	Unit Weight of Soil (pcf)	Thickness H (feet)	Depth to Center of Layer (feet)	Coefficient of Compression Cc	Initial Void Ratio e	Original Stress @ Center Pi (psf)	Induced Stress @ Center DP (psf)	Change in Thickness DH (inches)	Induced Stress @ Edge DP (psf)	Change in Thickness DH (inches)
1	95	10	5	0.03	0.72	1663	13500	2.01	1389	0.55
2	87	15	17.5	0.06	0.88	2010.5	13500	5.10	1389	1.31
3A	105	20	35	0.02	0.56	2621	13500	2.43	1389	0.57
3B	105	20	55	0.02	0.56	3473	13500	2.12	1389	0.45
	Total Thickness	65 ft					Total Settlement	11.65 in	Total Settlement	2.88 in
							Differential Settlement			8.77 in

Note: 140 feet of MSW X 1500 pcy / 27cf/y = 7778 psf at center of landfill.
13500 psf is the induced stress that causes desing limits for settlement to be exceeded.
11.65 inches = 0.97 feet
2.88 inches = 0.24 feet

Tomoka Farms Road Landfill
Volusia County, Florida
North Cell Expansion Area

Trench Invert Elevations

	Sump	Lateral 1	Lateral 2	Lateral 3	Lateral 4	Lateral 5	Lateral 6	Lateral 7
Distance between (ft)		40	160	160	160	160	160	160
Elevation	9.00	9.80	10.36	10.92	11.48	12.04	12.60	13.16
Slope 0.0035								
Settlement	-0.25	-0.49	-0.73	-0.97	-0.97	-0.97	-0.73	-0.49
Elevation	8.75	9.31	9.63	9.95	10.51	11.07	11.87	12.67
Final Slope		0.0140 ft/ft	0.0020 ft/ft	0.0020 ft/ft	0.0035 ft/ft	0.0035 ft/ft	0.0050 ft/ft	0.0050 ft/ft
(Percent)		1.40%	0.20%	0.20%	0.35%	0.35%	0.50%	0.50%

| Edge | | Center of Landfill | | Edge |

SHEET 3 OF 3

Circular Channel Analysis & Design
Solved with Manning's Equation

Open Channel - Uniform flow

Worksheet Name: Tomoka Farms Rd LF

Comment: Tomoka Farms Road Landfill - Collection Pipe

Solve For Full Flow Capacity

Given Input Data:

Diameter..........	1.00 ft	(12-inch Leachate Collection Pipe)
Slope.............	0.0020 ft/ft	
Manning's n.......	0.013	
Discharge.........	1.59 cfs	

Computed Results:

Full Flow Capacity.....	1.59 cfs	
Full Flow Depth........	1.00 ft	
Velocity..........	2.03 fps	(2.03 fps > 2.00 fps) ok
Flow Area.........	0.79 sf	
Critical Depth....	0.54 ft	
Critical Slope....	0.0063 ft/ft	
Percent Full......	100.00 %	
Full Capacity.....	1.59 cfs	
QMAX @.94D........	1.71 cfs	
Froude Number.....	FULL	