Geotechnical Investigation at
Southeast Landfill
Hillsborough County, Florida

Ardaman & Associates, Inc.

OFFICES

Orlando, 8008 S. Orange Avenue, Orlando, Florida 32809, Phone (407) 855-3860
Bartow, 1525 Centennial Drive, Bartow, Florida 33830, Phone (813) 533-0858
Cocoa, 1300 N. Cocoa Blvd., Cocoa, Florida 32922, Phone (407) 632-2503
Fort Myers, 2508 Rockfill Road, Fort Myers, Florida 33916, Phone (813) 337-1288
Miami, 2608 W. 84th Street, Hialeah, Florida 33016, Phone (305) 825-2683
Port Charlotte, 740 Tamiami Trail, Unit 3, Port Charlotte, Florida 33954, Phone (813) 624-3393
Port St. Lucie, 1017 S.E. Holbrook Ct., Port St. Lucie, Florida 34952, Phone (407) 337-1200
Sarasota, 2500 Bee Ridge Road, Sarasota, Florida 34239, Phone (813) 922-3526
Tallahassee, 3175 West Tharpe Street, Tallahassee, Florida 32303, Phone (904) 576-6131
Tampa, 1406 Tech Boulevard, Tampa, Florida 33619, Phone (813) 620-3389
West Palm Beach, 2511 Westgate Avenue, Suite 10, West Palm Beach, Florida 33409, Phone (407) 687-8200

MEMBERS:
A.S.F.E.
American Concrete Institute
American Society for Testing and Materials
American Consulting Engineers Council
Florida Institute of Consulting Engineers
American Council of Independent Laboratories

Ardaman & Associates, Inc.

Geotechnical, Environmental and
Materials Consultants

March 7, 1994
File Number 93-029

Hillsborough County
Department of Solid Waste
Post Office Box 1110
Tampa, Florida 33602

Attention: Ms. Patricia V. Berry
Landfill Services Section Manager

Subject: Geotechnical Investigation at Southeast Landfill, Hillsborough County, Florida

Dear Ms. Berry:

As requested, Ardaman & Associates, Inc. has completed a geotechnical investigation consisting of field exploration, laboratory testing and engineering evaluation for the Southeast Landfill, in Hillsborough County, Florida. The objective of the field exploration and laboratory testing was to determine the shear strength and consolidation characteristics of the waste phosphatic clay that underlies the landfill site. The engineering evaluation was performed to evaluate our previous recommendations in light of the new field and laboratory test results. The results of our field exploration, laboratory testing and engineering evaluation are summarized in this report.

This report has been prepared for the exclusive use of Hillsborough County and its consultants for the specific application to the evaluation of stability and filling schedule at the Southeast Landfill in accordance with generally accepted geotechnical engineering practice. No other warranty, expressed or implied, is made.

It has been a pleasure assisting you on this project. Please contact us if you have any questions concerning this report or if we can be of further assistance.

Very truly yours,
ARDAMAN & ASSOCIATES, INC.

Sujit K. Bhowmik

Sujit K. Bhowmik, Ph.D., P.E.
Project Engineer

Francis K. Cheung

Francis K. Cheung, P.E.
Senior Project Engineer
Florida Registration No. 36382

John E. Garlanger

John E. Garlanger, Ph.D., P.E.
Principal

Enclosures

cc: Mr. Steven M. Hamilton, SCS Engineers ✓

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF APPENDICES

1.0 INTRODUCTION

1.1 Site Location

The Southeast Landfill site is located within Sections 14, 15, 22 and 23 of Township 31 South, Range 21 East, in Hillsborough County, Florida. More specifically, the landfill site is located between Picnic and Pinecrest, about 2 miles west of State Road 39 and about 0.5 miles north of County Road 672. The approximate site location, as superimposed on a reproduction of the United States Geological Survey (USGS) quadrangle map of Lithia (1955, Photorevised 1987), Florida, is shown in Figure 1.

1.2 Site History and Project Background

The Southeast Landfill is constructed directly above a waste clay settling area at a former phosphate mine known as Lonesome Phosphate Mine or "Boyette Mine". The settling area, also known as Settling Area No. 1, was built on natural ground within a perimeter dike constructed of sand borrowed from the surrounding areas. Waste phosphatic clay was deposited in the settling area for a number of years during the mining operation.

A comprehensive study consisting of extensive field exploration, laboratory testing and geotechnical analyses was conducted by Ardaman & Associates, Inc. during 1981-83 to evaluate the feasibility of constructing a landfill above the settling area. The results and conclusions from our 1981-83 study are documented in an Ardaman & Associates report titled "Hydrogeological Investigation, Southeast County Landfill, Hillsborough County, Florida", dated February 22, 1983 (Ardaman & Associates' File Number 81-159). Based on the results of our 1981-83 study, the waste phosphatic clay was deposited on unmined natural ground sandy soils and, before landfill construction, the thickness of the phosphatic clay varied between 4 and 18 feet. A diagram showing the original thickness of the phosphatic clay within the landfill site is reproduced in Figure 2 from our February 22, 1983 report.

Subsequently, additional geotechnical analyses were performed by Ardaman & Associates, Inc. in 1989 to evaluate the landfill stability and the projected filling schedule. The results and conclusions from our 1989 study are documented in an Ardaman Associates report titled "Evaluation of Filling Schedules and Stability Analyses for Southeast Sanitary Landfill, Hillsborough County, Florida", dated July 13, 1989 (Ardaman & Associates' File Number 89-036).

1.3 Landfill Operating Plan

As part of our 1981-83 study, a filling plan was developed under which the entire landfill area would be divided into three phases and landfilling within a phase would be conducted gradually and only to a point where an adequate factor of safety against sliding failure of the landfill slope could be maintained. Filling within a phase area above a previous lift would resume only after the clay had consolidated and experienced sufficient increase in strength to support the additional load. The current filling plan used by the county is a modified version of the plan developed in our 1981-83 study.

In accordance with the current landfill operating plan, the entire landfill area has been divided into six phases (Phases I through VI) and each phase would be filled in three different stages (Stages

1 through 3). The present layouts of the six phases of the landfill are depicted in Figure 3. According to the present filling plan, all exterior slopes would be 4 horizontal to 1 vertical and all interior slopes would be 10 horizontal to 1 vertical. Beyond the crest of the exterior slopes, the landfill surface would be graded to a slope of 3 to 5 percent until the peak elevation is reached near the center of the landfill.

The Stage 1 operation of the landfill was planned to begin within the Phases I and II areas and continue until a crest elevation of +145 feet (NGVD) was reached along the perimeter side slope. The interior side slope was to be filled to a crest elevation of approximately +155 feet (NGVD). The peak elevation near the center within the Phases I and II areas was designed at +174 feet (NGVD). Upon completion of filling in these areas, the Stage 1 operation would continue onto Phase III and Phase IV areas to a crest elevation of +145 feet (NGVD) along the exterior slope and +153 feet (NGVD) along the interior face. The peak elevation near the center of the Phases III and IV areas during the Stage 1 filling was planned to be +183 feet (NGVD).

Upon completion of Stage 1 filling in Phases III and IV, the Stage 2 operation would begin and involve raising of the Phases I through IV areas to a crest elevation of about +160 feet (NGVD) along the outside slope and +170 feet (NGVD) along the inside slope. The peak elevation that could be reached in these areas upon completion of the Stage 2 filling would be approximately +196 feet (NGVD).

After completion of Stage 2 filling above the Phases I through IV areas, filling was scheduled to begin in the Phases V and VI areas and continued until a crest elevation of about +160 feet (NGVD) along the perimeter slope and a peak elevation of +204 feet (NGVD) near the center were attained.

The Stage 3 filling would begin following the completion of Phases V and VI filling to the elevations indicated above. As laid out in the landfill operating plan, the Stage 3 operation would involve raising the entire landfill to a crest elevation of about +220 feet (NGVD) along the perimeter slope and a peak elevation of about +250 feet (NGVD) near the center of the landfill.

1.4 Present Site Layout and Elevations

The present layout of the landfill phases obtained from a topographic map (i.e., aerial photography flown on January 15, 1992) of the landfill site provided to us by SCS Engineers is presented in Figure 3. Based on the elevation contours in Figure 3, the approximate maximum elevations of the landfill/ground surface in the Phases I through VI areas were +159, +170, +171, +137, +127 and +130 feet (NGVD), respectively, on the date the aerial photograph was taken. No landfill material has been placed to-date in the Phases V and VI areas. An approximately 4-foot thick layer of sand in the Phase V area and 8-foot thick layer of sand in the Phase VI area have been placed to preload the phosphatic clay deposit before landfilling.

1.5 Objectives and Scope of Present Study

The objectives of the present study were: (i) to determine the thickness and shear strength and consolidation characteristics of the phosphatic clay, and to compare the results with those of our 1981-83 study and (ii) to evaluate our previous recommendations on landfill stability and filling schedule in light of the new field and laboratory test results.

The services provided by Ardaman & Associates, Inc. for this project consisted of the following:

- Review of existing engineering reports and topographic maps to compile available information on the landfill site and to make an assessment on the type of information to be gathered as part of the present project.

- Planning and performance of a field exploration program to determine the thickness and *in situ* properties of the waste phosphatic clay and to collect undisturbed samples for use in laboratory soil testing.

- Planning and performance of a laboratory testing program to determine relevant properties of representative waste phosphatic clay samples obtained during the field sampling operation.

- Engineering evaluation of our previous recommendations on landfill stability and filling schedule in light of the present field and laboratory test results.

- Preparation of this report to document the information gathered for this project and to summarize the results of our evaluation.

This report has been prepared for the exclusive use of Hillsborough County and its consultants for the specific application to the evaluation of stability and filling schedule at the Southeast Landfill in accordance with generally accepted geotechnical engineering practice. No other warranty, expressed or implied, is made.

The analyses and recommendations presented herein are based in part on the projected filling schedule and the refuse/residue unit weight data currently available to us. In the event any changes occur or additional data become available, we should be notified to reevaluate the analyses and recommendations presented herein.

2.0 FIELD EXPLORATION PROGRAM

2.1 General

The objectives of the field exploration program were to determine the *in situ* shear strength and consolidation properties of the phosphatic clay using field tests and to collect undisturbed samples of the phosphatic clay for use in laboratory testing. The field exploration program consisted of piezocone penetration testing at 6 locations (PC-1 through PC-6), piezoprobe installations at 2 locations (PP-2 and PP-6), vane shear testing at 2 locations (VS-2 and VS-6) and undisturbed sampling at 3 locations (TH-1, TH-2 and TH-6). Approximate locations of the field tests are shown in Figure 3. At each test location, a hollow stem auger was used to drill through the landfill material and/or the sand tailings above the clay layer. The elevations of the landfill surface at Test Sites 1 through 6, as surveyed by the Hillsborough County staff in February, 1994, were +157.9, +148.7, +149.2, +134.2, +129.1 and +129.7 feet (NGVD), respectively.

2.2 Piezocone Penetration Tests

The piezocone consists of a conical point attached to a steel rod and a friction sleeve. The test is performed by pushing the assembly into the soil at a constant rate of penetration. Resistance to penetration at the cone tip and on the friction sleeve are measured by load cells placed within the assembly and the pore pressure in the soil is measured using a pressure transducer connected to the porous element placed near the cone tip.

Six piezocone penetration tests (PC-1 through PC-6) were performed at the landfill site during the present field exploration program to determine the thickness and to estimate the undrained shear strength of the phosphatic clay. Results of the piezocone penetration tests are presented in Figures 4 through 9. As shown in the figures, results are presented in the form of tip resistance (i.e., the resistance to penetration at the cone tip), sleeve resistance (i.e., the resistance to penetration of the friction sleeve), pore pressure (i.e., the total pore pressure including the pore pressure generated due to penetration of the cone) and friction ratio (i.e., the ratio of sleeve resistance to tip resistance) versus depth. The piezocone tests were started near the top of the clay layer and terminated near the bottom of the clay layer. Since there are sandy soils both at the top and bottom of the phosphatic clay, the thickness of the clay layer were determined by examining the variation of tip resistance and pore pressure with depth. The tip resistance and the pore pressure in a clay layer are expected to be lower and higher, respectively, than those in a sand layer. The sudden changes in tip resistance and pore pressure, as shown in Figures 4 through 9, are expected to occur at the transitions between the sand tailings and the phosphatic clay. Based on these criteria, the thicknesses of the phosphatic clay at locations PC-1, PC-2, PC-3 and PC-6 were estimated to be 3.5, 8, 7.5 and 14 feet, respectively. At locations PC-4 and PC-5, the piezocone tests were started slightly below the top of the clay layer and, therefore, clay thicknesses at these two sites could not be determined precisely. The clay thicknesses at these two sites were determined to be greater than 7.5 and 13 feet, respectively.

2.3 Piezoprobe Tests

The piezoprobe consists of a conical point attached to a steel rod. The conical point is fitted with a porous element which is connected to a pressure transducer for pore pressure measurement. The test is performed by pushing the piezoprobe assembly into the soil at a constant rate. When the probe is pushed into the soil, excess pore pressures are generated in the soil due to penetration of the probe. The continuous pore pressure profile generated during piezoprobe penetration can be used to define soil type and stratification. If the probe is pushed to a certain depth and then held stationary, the excess pore pressure dissipates and eventually the pore pressure reaches the actual pore pressure in the soil before probe penetration. The rate of dissipation of excess pore pressure can be used to estimate the coefficient of consolidation of the clay.

Two piezoprobe tests (PP-2 and PP-6) were performed at the landfill site during the present field exploration program to estimate the existing pore pressure in the phosphatic clay. Results of the piezoprobe tests PP-2 and PP-6 are presented in Figures 10 and 11, respectively, in the form of normalized pore pressure (i.e., the ratio of initial pore pressure after piezoprobe penetration to the pore pressure at any time after penetration) versus time. As shown in the figures, the pore pressures at some depths reached near-equilibrium conditions at the end of the monitoring period (i.e., the time period during which the piezoprobe was held stationary). The equilibrium pore pressures at various depths at the two test locations (i.e., PP-2 and PP-6) were obtained by extrapolating the pore pressure versus time curves as shown in Figures 10 and 11. The equilibrium pore pressures so obtained are plotted versus depth in Figures 12 and 13 for test locations PP-2 and PP-6, respectively. Also shown in Figures 12 and 13 are the estimated static pore pressure lines in the phosphatic clay assuming the groundwater table to be at 2 feet above the top of the clay layer. As expected, the equilibrium pore pressures were higher than the corresponding estimated static pore pressures. The difference between the equilibrium and static pore pressures is the excess pore pressure due to incomplete consolidation of the clay under the weight of the landfill material and/or sand tailings.

Based on the excess pore pressures obtained from Figure 12, the consolidation at the center of the clay layer at location PP-2 under the weight of the landfill was estimated to be 96 percent complete. Considering the original thickness of the clay layer at location PP-2 to be 10.5 feet and the filling period to be between December 1985 and April 1987, the *in situ* coefficient of consolidation of the phosphatic clay at the test location PP-2 was estimated to be 1.3×10^{-4} cm^2/sec. Similarly, based on the excess pore pressures obtained from Figure 13, the consolidation at the center of the clay layer at location PP-6 under the weight of the preloading was estimated to be about 40 percent complete. Considering the original thickness of the clay layer at location PP-6 to be 18 feet and the preloading period to be between December 1989 and January 1990, the *in situ* coefficient of consolidation of the phosphatic clay at the test location PP-6 was estimated to be 1.5×10^{-4} cm^2/sec.

2.4 Vane Shear Tests

The vane shear testing equipment consists of four metal blades attached to a vertical shaft. The test is carried out by pushing the blade assembly into the soil and then applying a torque to the vertical shaft. The undrained shear strength of the soil is computed from the torsional force required to shear a cylindrical surface by the vane.

Vane shear tests were performed at two different depths at location VS-2 and at five different depths at location VS-6. The undrained shear strengths obtained from the vane shear tests are presented in Table 1. For highly plastic clays, the undrained shear strength obtained from vane shear tests is generally higher than that actually mobilized in the field. Bjerrum (1972)[1] recommended correction factors as a function of the plasticity index of the clay to estimate the mobilized shear strength from the vane shear strength. The corrected undrained shear strengths (using a correction factor of 0.6) are also presented in Table 1.

To determine the undrained shear strength ratios (i.e., the ratio of undrained shear strength to effective vertical stress) from vane shear test results, the effective vertical stresses at the test depths were estimated using the excess pore pressure distributions presented in Figures 12 and 13. As shown in Table 1, the predominant range of undrained shear strength ratios obtained from the vane shear test results was 0.19 to 0.27.

---

[1] Bjerrum, L. (1972), "Embankments on Soft Ground", State-of-the-Art Report, Proc. ASCE Spec. Conf. on Performance of Earth and Earth-Supported Structures, Lafayette, Vol. II, pp. 1-54.

2.5 Undisturbed Sampling

During the field exploration program, undisturbed samples of the phosphatic clay were obtained using Shelby tube samplers from test locations TH-1, TH-2 and TH-6. A total of 2, 4 and 8 Shelby tube samples were obtained from locations TH-1, TH-2 and TH-6, respectively. Soil specimens prepared from the undisturbed samples were used to perform index, shear strength, consolidation and permeability tests in the laboratory.

3.0 LABORATORY TESTING PROGRAM

3.1 General

The undisturbed Shelby tube soil samples obtained during the field sampling operation were transported to our laboratory for visual examination and laboratory testing. The laboratory tests were performed to determine various index, strength, deformation and hydraulic properties of phosphatic clay at different depths. All laboratory tests, where applicable, were performed in accordance with ASTM Standards.

3.2 Index Tests

The laboratory index tests performed on the phosphatic clay samples included determinations of moisture content, dry density and Atterberg limits (i.e., liquid limit and plastic limit). The results are presented in Table 2. As shown in the table, the moisture contents, liquid limits and plasticity indices of the tested samples of phosphatic clay ranged from 66.6 to 198.4, 141 to 252 and 103 to 203 percent, respectively. The dry density of the tested samples ranged from 26.4 to 61.2 lb/ft^3. The Atterberg limits are also presented on a Plasticity Chart in Figure 14. As shown in the figure, the phosphatic clay classifies as a highly plastic clay.

3.3 Permeability Tests

Constant head permeability tests were performed in the laboratory on 3 phosphatic clay specimens obtained from the sampling location TH-6. The results are presented in Table 3. As shown in the table, the test specimens were isotropically consolidated to pressures of 1065, 4195 and 7200 lb/ft^2 before permeation to simulate the range of *in situ* stress conditions. The measured coefficient of permeability of the phosphatic clay ranged from 3.3×10^{-8} to 3.5×10^{-9} cm/sec.

3.4 Unconfined Compression Tests

Unconfined compression tests were performed on a total of 8 phosphatic clay specimens obtained from the sampling locations TH-1, TH-2 and TH-6. The undrained shear strengths obtained from the eight tests are presented in Table 4 and the stress-strain curves are included in Appendix 1. The *in situ* effective vertical stress at the specimen depth at TH-1 was estimated assuming a degree of consolidation corresponding to a c_v-value of 1.5×10^{-4} cm^2/sec. For samples obtained from locations TH-2 and TH-6, the *in situ* effective vertical stresses at the depths of the test specimens were estimated using the equilbrium pore pressure distributions presented in Figures 12 and 13. The undrained shear strength ratios (i.e., the ratio of undrained shear strength to effective vertical stress) for the eight test specimens are presented in Table 4.

As shown, the predominant range of undrained shear strength ratios obtained from the unconfined compression test results was 0.20 to 0.31.

3.5 Consolidated Undrained Triaxial Compression Tests

A total of five consolidated undrained triaxial compression tests were performed on test specimens prepared from the Shelby tube samples obtained from test locations TH-1, TH-2 and TH-6. Before applying shear stress, the specimens were isotropically consolidated to stresses ranging from 1025 to 10250 lb/ft^2. Results of the tests are presented in Appendix 2 and summarized in Table 5. These test results were used to compute the undrained shear strength ratios (i.e., the ratio of undrained shear strength to effective vertical stress under normally consolidated condition) for the tested specimens. As shown, the predominant range of undrained shear strength ratios obtained from the consolidated undrained triaxial compression tests was 0.22 to 0.26.

3.6 Consolidated Undrained Direct Simple Shear Tests

Two consolidated undrained direct simple shear tests were performed on phosphatic clay specimens obtained from the sampling location TH-5. The direct simple shear tests were performed by J. T. Germaine and Associates of Lexington, Massachusetts. The test specimens were isotropically consolidated to stresses of 2728 and 8244 lb/ft^2 before applying shear stress and the undrained shear strength ratios obtained from the two tests were 0.24 and 0.20, respectively.

3.7 One-Dimensional Consolidation Tests

Four one dimensional consolidation tests were performed on phosphatic clay specimens obtained from the sampling locations TH-2 and TH-6. Results of the consolidation tests are presented in the form of strain versus effective stress and coefficient of consolidation (c_v) versus effective stress relationships in Figures 15 through 18. As shown in the figures, the c_v-values for the phosphatic clay specimens ranged from 5.9×10^{-5} to 1.1×10^{-3} cm^2/sec. In the overconsolidated range, the coefficient of consolidation is generally higher than that in the normally consolidated range. The c_v-values obtained from the four tests are plotted versus stress level (i.e., the ratio of vertical consolidation pressure to maximum past pressure) in Figure 19. Also shown in the figure is the average c_v versus stress level relationship obtained from the results of three tests. The c_v-values obtained from sample US-1A of boring TH-6 were not used in developing this relationship because this sample was recovered from near the top of the phosphatic clay where the clay was overconsolidated probably due to desiccation and, therefore, the c_v-values of this sample are not representative of those of the phosphatic clay inside the deposit. As shown in Figure 19, the average c_v-value in the normally consolidated range was determined to be approximately 1×10^{-4} cm^2/sec.

4.0 ENGINEERING EVALUATION

4.1 Undrained Shear Strength

The undrained shear strength of the phosphatic clay was determined using field vane shear tests, unconfined compression tests, consolidated undrained triaxial compression tests and consolidated undrained direct simple shear tests. Results of all these tests are summarized in Figure 20 where the undrained shear strength is plotted versus effective vertical stress. Based on the best-fit straight line through the data points, as shown in the figure, the average undrained shear strength ratio (i.e., the ratio of undrained shear strength to effective vertical stress) was determined to be 0.22. For comparison, the undrained shear strength ratio recommended in our February 22, 1983 report and used in all our previous stability analyses was 0.21. Thus, there is no significant difference between the undrained shear strength ratio obtained from the present study and that used in our previous stability analyses.

4.2 Coefficient of Consolidation

The coefficient of consolidation of the phosphatic clay was determined from one-dimensional consolidation tests performed in the laboratory and the results of piezoprobe tests performed at two test locations at the site. An average coefficient of consolidation of $1x10^{-4}$ cm^2/sec in the normally consolidated range was obtained from the consolidation test results. From piezoprobe test results, the *in situ* coefficient consolidation was estimated to range between $1.3x10^{-4}$ and $1.5x10^{-4}$ cm^2/sec. For comparison, the coefficient of consolidation recommended in our February 22, 1983 report and used in all our previous analyses was $1.5x10^{-4}$ cm^2/sec, which falls within the range of c_v-values obtained from the present investigation.

4.3 Landfill Stability and Filling Schedule

At the Southeast Landfill, the stability of the landfill slopes and the landfill filling schedule depend on the shear strength and consolidation properties of the underlying phosphatic clay. The two major parameters that govern the stability and filling schedule are the undrained shear strength of the phosphatic clay and the rate of excess pore pressure dissipation in the clay.

The undrained shear strength determines the steepest slope and the maximum height to which the landfill can be raised. The higher the undrained shear strength, the steeper is the slope and the greater is the maximum height of the fill that can be attained.

The rate at which the excess pore pressure in a clayey soil dissipates is governed by the coefficient of consolidation of the clay. The higher the coefficient of consolidation, the faster is the rate of excess pore pressure dissipation. When landfill material is placed on the phosphatic clay, excess pore pressure is generated in the clay. With time, the excess pore pressure dissipates and the shear strength of the clay increases. When the increase in undrained shear strength is sufficient, a second lift can be placed on top of the first lift. Thus, the faster the rate of pore pressure dissipation (i.e., the higher the coefficient of consolidation), the shorter would be the waiting period between successive lifts.

As explained in Sections 4.1 and 4.2, the undrained shear strength and the coefficient of consolidation of the phosphatic clay determined from the present investigation are in good agreement with those used in all our previous analyses. Therefore, the recommendations on stability and filling schedule (i.e., a waiting period of 7 years between placement of successive lifts) of the Southeast Landfill provided in our previous reports have been confirmed by the recent field and laboratory testing program and remain unchanged.

Table 1

Results of Field Vane Shear Tests

Test Site	Test Depth (feet)	Estimated *in situ* Effective Vertical Stress, $\bar{\sigma}_v$ (lbs/ft²)	Measured Undrained Shear Strength (lbs/ft²)	Corrected* Undrained Shear Strength, s_u (lbs/ft²)	Undrained Shear Strength Ratio** ($s_u/\bar{\sigma}_v$)
VS-2	29.5	2335	746	450	0.19
	31.5	2390	777	465	0.20
VS-6	11.5	495	207	125	0.25
	13.5	465	207	125	0.27
	17.5	605	116	70	0.12
	19.5	775	323	195	0.25
	21.5	995	376	225	0.23

* Corrected using the reduction factor proposed by Bjerrum (1972).
** Ratio of undrained shear strength to estimated *in situ* effective vertical stress.

Bjerrum (1972)

$S_u(\text{design}) = \lambda S_u(\text{vane shear})$ — B. DAS (1985) Prin. of Geo Eng.

λ = correction factor $= 1.7 - 0.54 \log(PI)$

VS-6
in-situ, $\bar{\sigma}_v' = 495$ lb/ft²

Table 2

Results of Laboratory Index Tests on Phosphatic Clay Samples

Ken
3/17/97

Test Hole Number	Sample Number	Sample Depth (feet)	Moisture Content ~~(feet)~~ %	Dry Density (lb/ft³)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)
TH-1	US-1*	39.7 - 40.0	91.4	-	158	39	119
	US-2	41.3 - 42.0	103.3	44.1	252	49	203
TH-2	US-1**	26.6 - 27.5	66.6	61.2	-	-	-
	US-2	28.0 - 29.5	92.5	47.6	197	53	144
	US-4	31.8 - 33.5	86.0	48.8	193	51	142
TH-6	US-1	9.9 - 10.5	87.4	51.3	141	38	103
	US-1A	9.5 - 10.5	81.5	54.0	148	33	115
	US-2	11.2 - 13.0	198.4	26.4	227	41	186
	US-3	13.8 - 15.0	171.8	30.0	217	19	198
	US-4	15.2 - 17.0	156.5	32.6	207	41	166
	US-6	19.1 - 21.0	110.8	42.5	154	30	124

* Only about 16 percent of the [illegible] this sample.
** This sample was sandy; Atte[illegible]

TH-6 8-ft of sand surcharge

	Sample Depth	Clay Depth	
US-1	9.9 - 10.5	1.9 - 2.5	
US-1A	9.5 - 10.5	1.5 - 2.5	upper
US-2	11.2 - 13.0	3.2 - 5.0	
US-3	13.8 - 15.0	5.8 - 7.0	mid
US-4	15.2 - 17.0	7.2 - 9.0	
US-6	19.1 - 21.0	11.1 - 13.0	low

Table 3

Results of Laboratory Permeability Tests on Phosphatic Clay Samples

Test Hole Number	Sample Number	Specimen Depth (feet)	Initial Specimen Condition		Effective Isotropic Consolidation Pressure, $\bar{\sigma}_c$ (lb/ft^2)	Final Specimen Condition		Coefficient of Permeability (cm/sec)
			Moisture Content (%)	Dry Density (lb/ft^3)		Moisture Content (%)	Dry Density (lb/ft^3)	
TH-6	US-2	12.3	221.1	23.8	1065	178.4	28.6	3.3×10^{-8}
	US-4	16.0	159.0	32.1	4195	95.7	47.1	7.9×10^{-9}
	US-6	20.7	106.4	44.0	7200	70.3	60.7	3.5×10^{-9}

TH-6
$\sigma_v' = 1065$ lb/ft^2

Table 4

Results of Unconfined Compression Tests on Phosphatic Clay Samples

Test Hole Number	Sample Number	Specimen Depth (feet)	Estimated *in situ* Effective Vertical Stress, $\bar{\sigma}_v$ (lb/ft^2)	Moisture Content (%)	Fines Content (%)	Dry Density (lb/ft^3)	Undrained Shear Strength, s_u (lb/ft^2)	Undrained Shear Strength Ratio* $(s_u/\bar{\sigma}_v)_{NC}$
TH-1	US-2	41.6	3600	106.0	-	44.1	460	0.13
TH-2	US-2	28.8	2320	97.7	-	46.2	490	0.21
	US-4	31.8	2400	92.8	-	48.7	470	0.20
	US-4	33.0	2500	110.3	-	42.2	720	0.29
TH-6	US-1	10.2	600	85.9	100	50.6	480	0.31**
	US-2	11.8	500	199.1	99	27.1	125	0.25
	US-4	15.6	500	154.0	100	33.0	115	0.23
	US-6	19.5	775	122.8	100	39.6	220	0.28

* Ratio of undrained shear strength to estimated *in situ* effective vertical stress.

** The clay at this depth was overconsolidated probably due to desiccation. From a consolidation test, the overconsolidation ratio (OCR) was determined to be 3.3. The undrained shear strength ratio under normally consolidated (NC) condition for this sample was computed as $(s_u/\bar{\sigma}_v)_{NC} = (s_u/\bar{\sigma}_v)/OCR^{0.8}$.

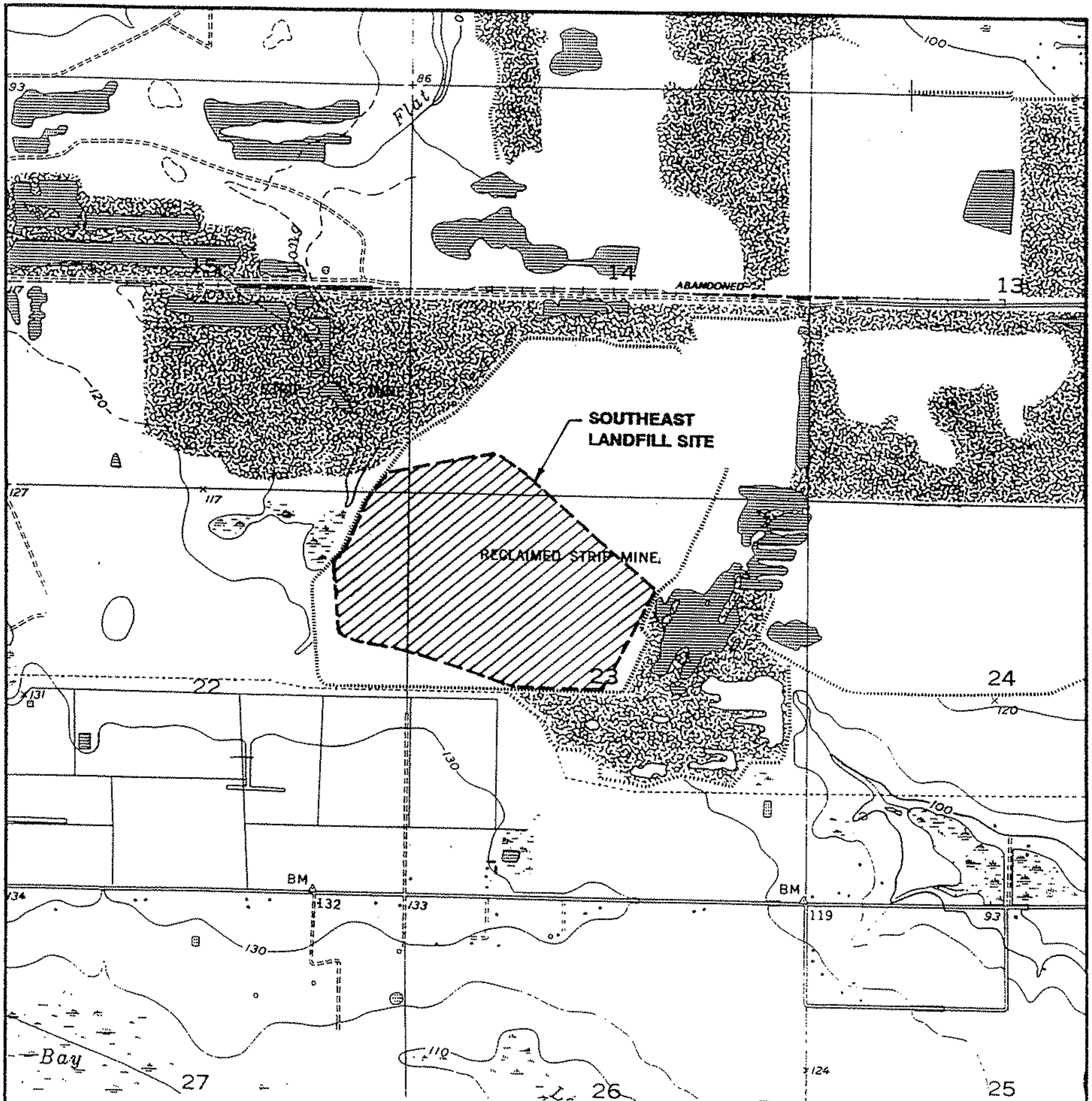
Table 5

Results of Consolidated Undrained Triaxial Compression Tests on Phosphatic Clay Samples

Test Hole Number	Sample Number	Specimen Depth (feet)	Initial Specimen Condition		Effective Isotropic Consolidation Pressure, $\bar{\sigma}_c$ (lb/ft^2)	Pre-Shear Specimen Condition		Undrained Shear Strength Ratio* $(s_u/\bar{\sigma}_{vc})_{NC}$
			Moisture Content (%)	Dry Density (lb/ft^3)		Moisture Content (%)	Dry Density (lb/ft^3)	
TH-1	US-2	41.8	105.0	44.7	7180	75.4	55.8	0.24
TH-2	US-4	32.5	120.2	40.1	10250	77.5	54.8	0.26
TH-6	US-1A**	9.9	85.6	52.6	1025	83.3	52.1	0.44
	US-3	14.8	164.6	31.4	4100	89.2	49.7	0.23
	US-6	20.0	127.5	38.5	5125	81.2	53.1	0.22

* Ratio of undrained shear strength to effective vertical consolidation stress for normally consolidated condition.

** This sample was obtained from near the top of the phosphatic clay deposit where the clay was probably overconsolidated due to desiccation.



SITE LOCATION MAP

TOWNSHIP 31 SOUTH
RANGE 21 EAST
SECTIONS 14, 15, 22 AND 23

OBTAINED FROM U.S.G.S. QUAD MAP: LITHIA, FLORIDA 1955
(PHOTOREVISED 1987)

NORTH

0 1000 2000

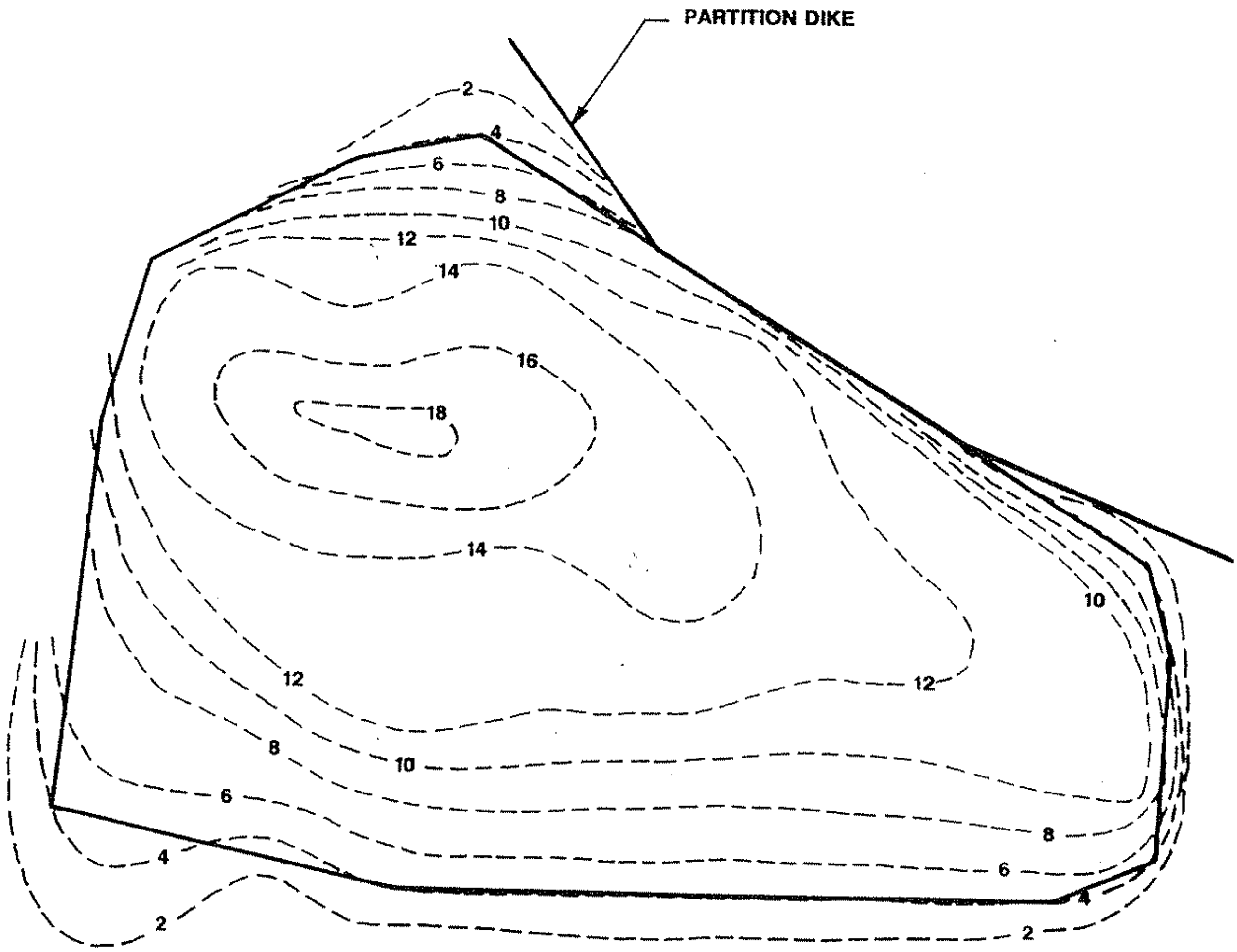
SCALE —— FEET

Ardaman & Associates, Inc.
Geotechnical, Environmental and Materials Consultants

GEOTECHNICAL INVESTIGATION
SOUTHEAST LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

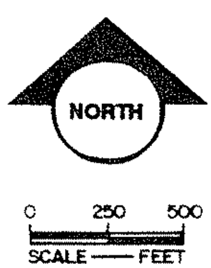
DRAWN BY: SEF	CHECKED BY: SKB	DATE: 03/02/94	
FILE NO. 93-029	APPROVED BY:		FIGURE: 1

N03676

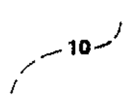


THICKNESS OF PHOSPHATIC CLAY BEFORE LANDFILL CONSTRUCTION

(REPRODUCED FROM ARDAMAN & ASSOCIATES' 1981-1983 STUDY)



LEGEND



CONTOUR OF CLAY THICKNESS IN FEET

Ardaman & Associates, Inc.
Geotechnical, Environmental and Materials Consultants

GEOTECHNICAL INVESTIGATION
SOUTHEAST LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

DRAWN BY: TS	CHECKED BY: SKB	DATE: 03/02/94
FILE NO. 93-029	APPROVED BY:	FIGURE 2

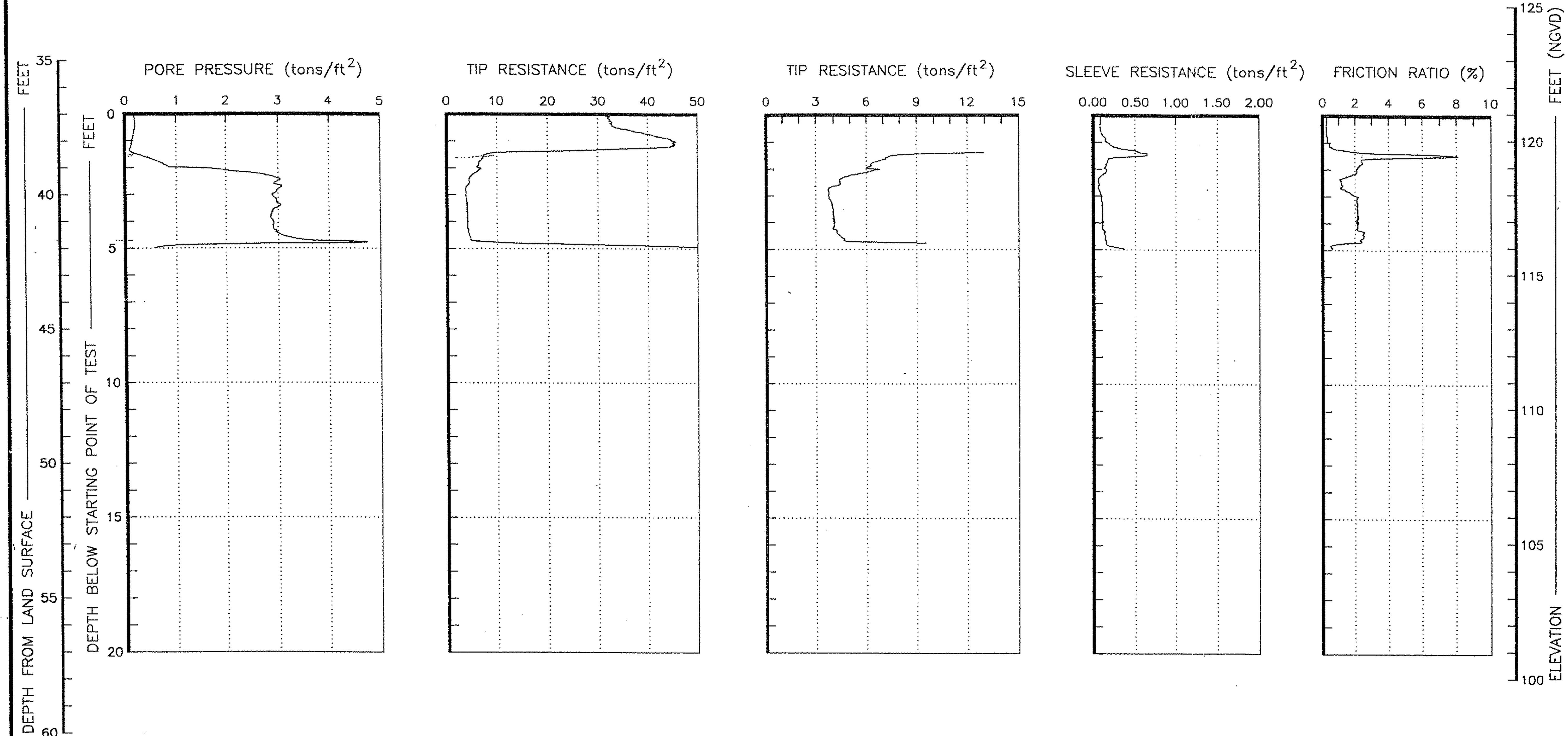
N03676

LANDFILL LAYOUT AND FIELD TEST LOCATION MAP

Ardaman & Associates, Inc.
Geotechnical, Environmental and Materials Consultants

GEOTECHNICAL INVESTIGATION
SOUTHEAST LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

DRAWN BY RBE	CHECKED BY. SKB	DATE 2/24/94
FILE NO. 93-029	APPROVED BY	FIGURE: 3



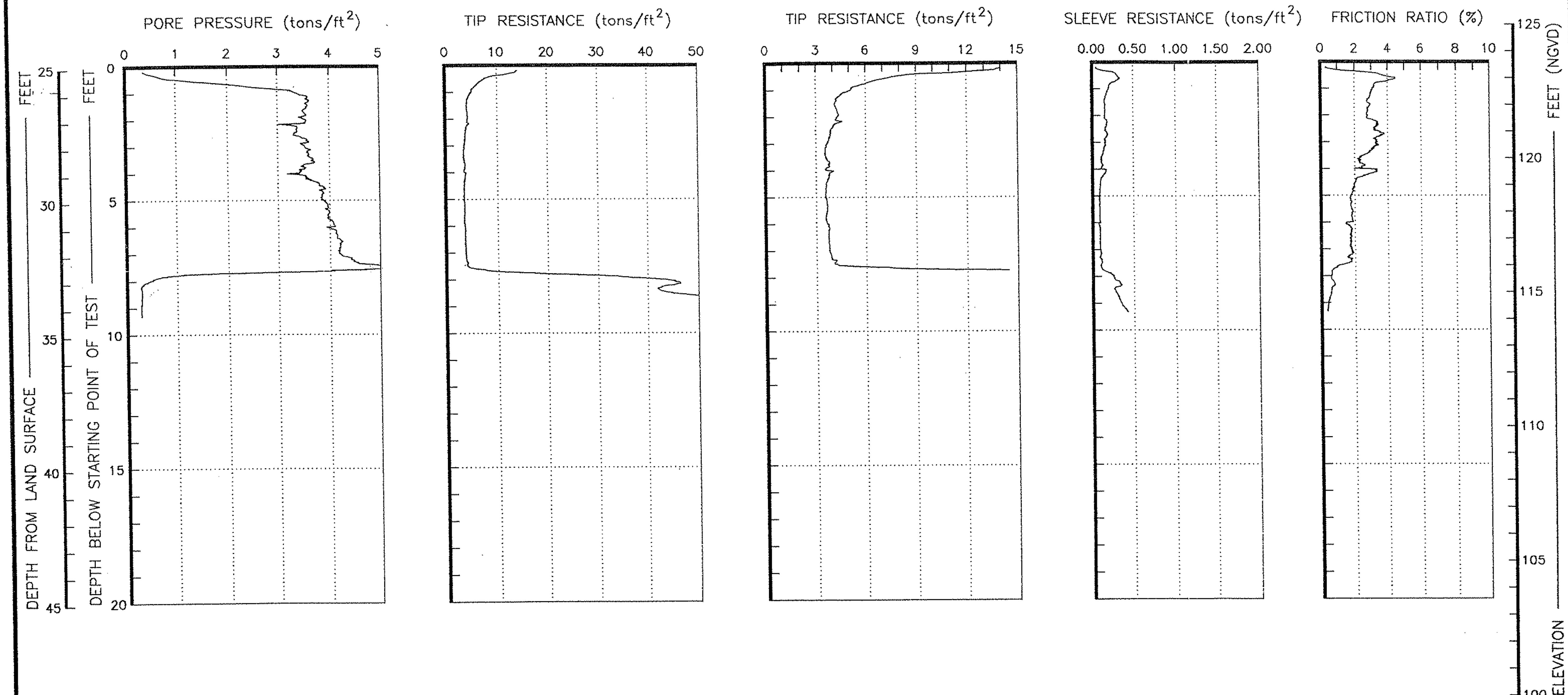
RESULTS OF PIEZOCONE TEST AT PC-1

Ardaman & Associates, Inc.
Geotechnical, Environmental and Materials Consultants

GEOTECHNICAL INVESTIGATION
SOUTHEAST LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

DRAWN BY: RBE	CHECKED BY: RW	DATE: 12/17/93
FILE NO.: 93-029	APPROVED BY:	FIGURE: 4

FILE: 93-029 DATE: 03/04/94



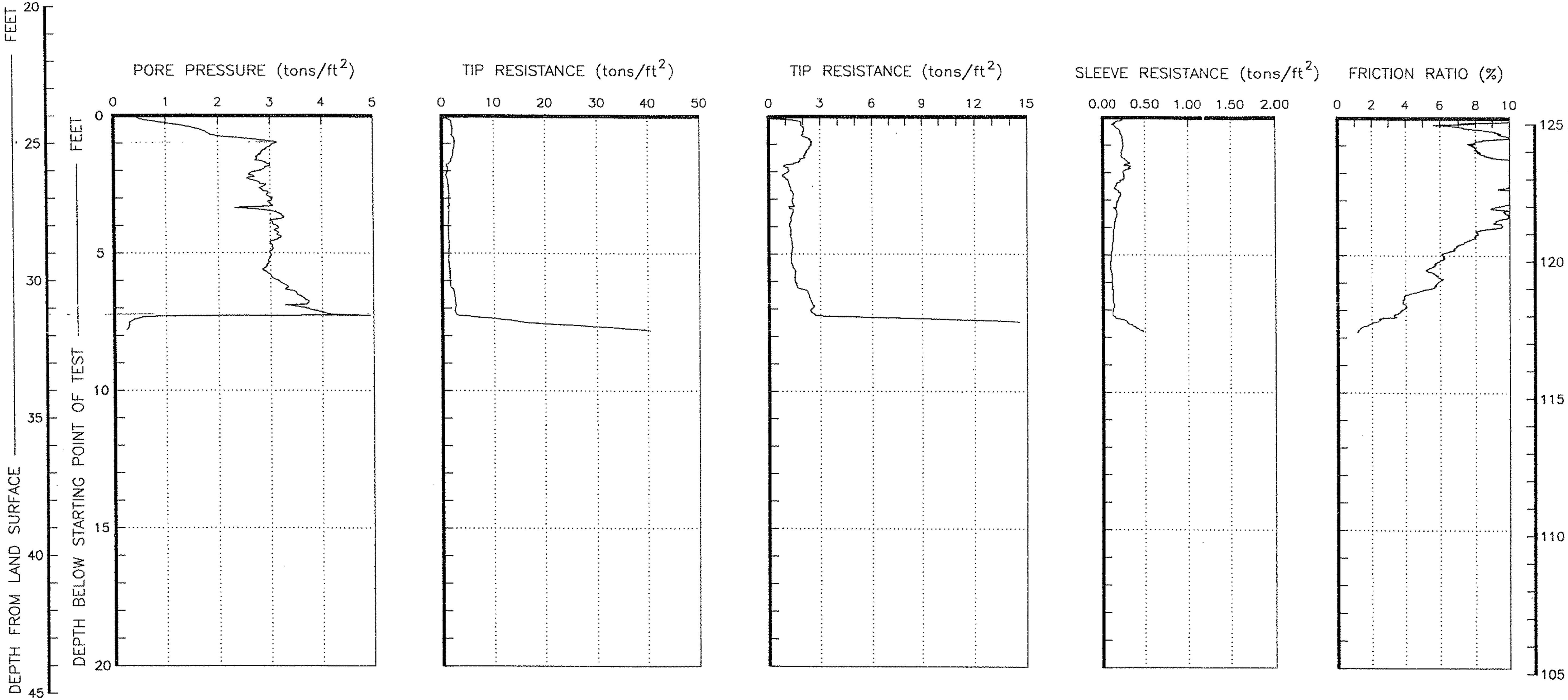
RESULTS OF PIEZOCONE TEST AT PC-2

Ardaman & Associates, Inc.
Geotechnical, Environmental and Materials Consultants

GEOTECHNICAL INVESTIGATION
SOUTHEAST LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

DRAWN BY: RBE	CHECKED BY: RW	DATE: 12/17/93
FILE NO.: 93-029	APPROVED BY:	FIGURE: 5

FILE: 93-029 DATE: 03/04/94



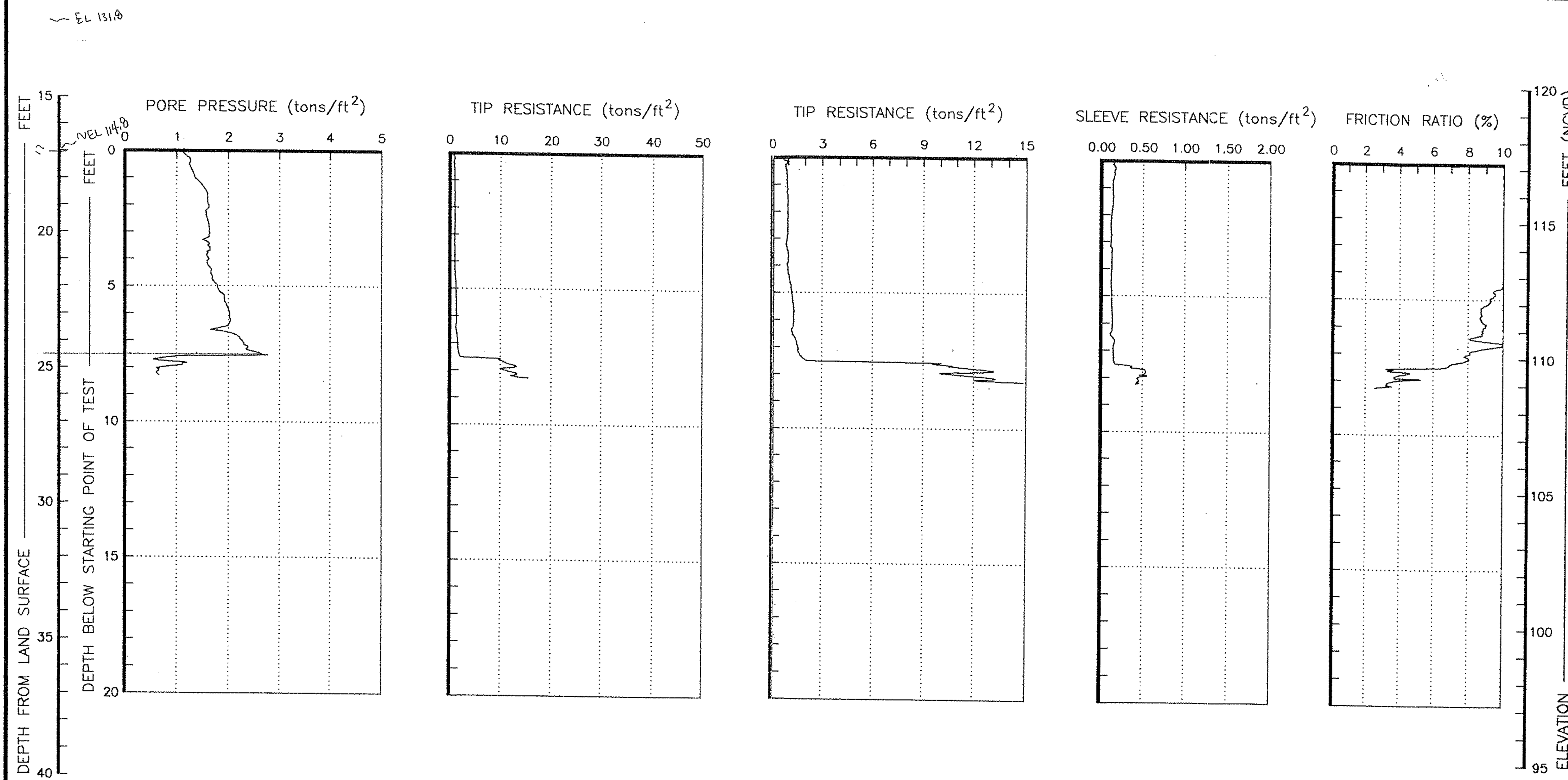
RESULTS OF PIEZOCONE TEST AT PC-3

Ardaman & Associates, Inc.
Geotechnical, Environmental and Materials Consultants

GEOTECHNICAL INVESTIGATION
SOUTHEAST LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

DRAWN BY: RBE	CHECKED BY: RW	DATE: 12/17/93
FILE NO.: 93-029	APPROVED BY:	FIGURE: 6

FILE: 93-029 DATE: 03/04/94



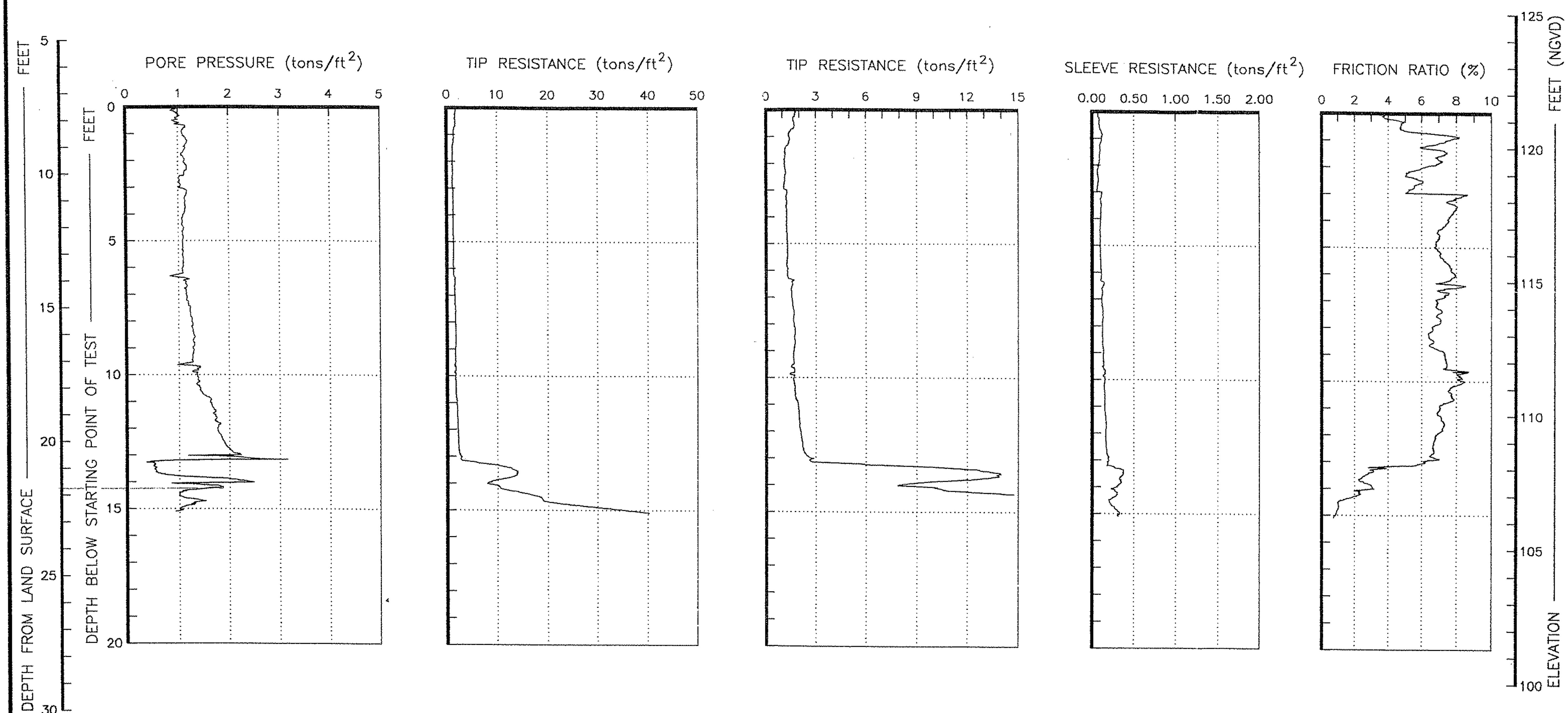
RESULTS OF PIEZOCONE TEST AT PC-4

Ardaman & Associates, Inc.
Geotechnical, Environmental and Materials Consultants

GEOTECHNICAL INVESTIGATION
SOUTHEAST LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

DRAWN BY: RBE	CHECKED BY: RW	DATE: 12/17/93
FILE NO.: 93-029	APPROVED BY:	FIGURE: 7

FILE: 93-029 DATE: 03/04/94



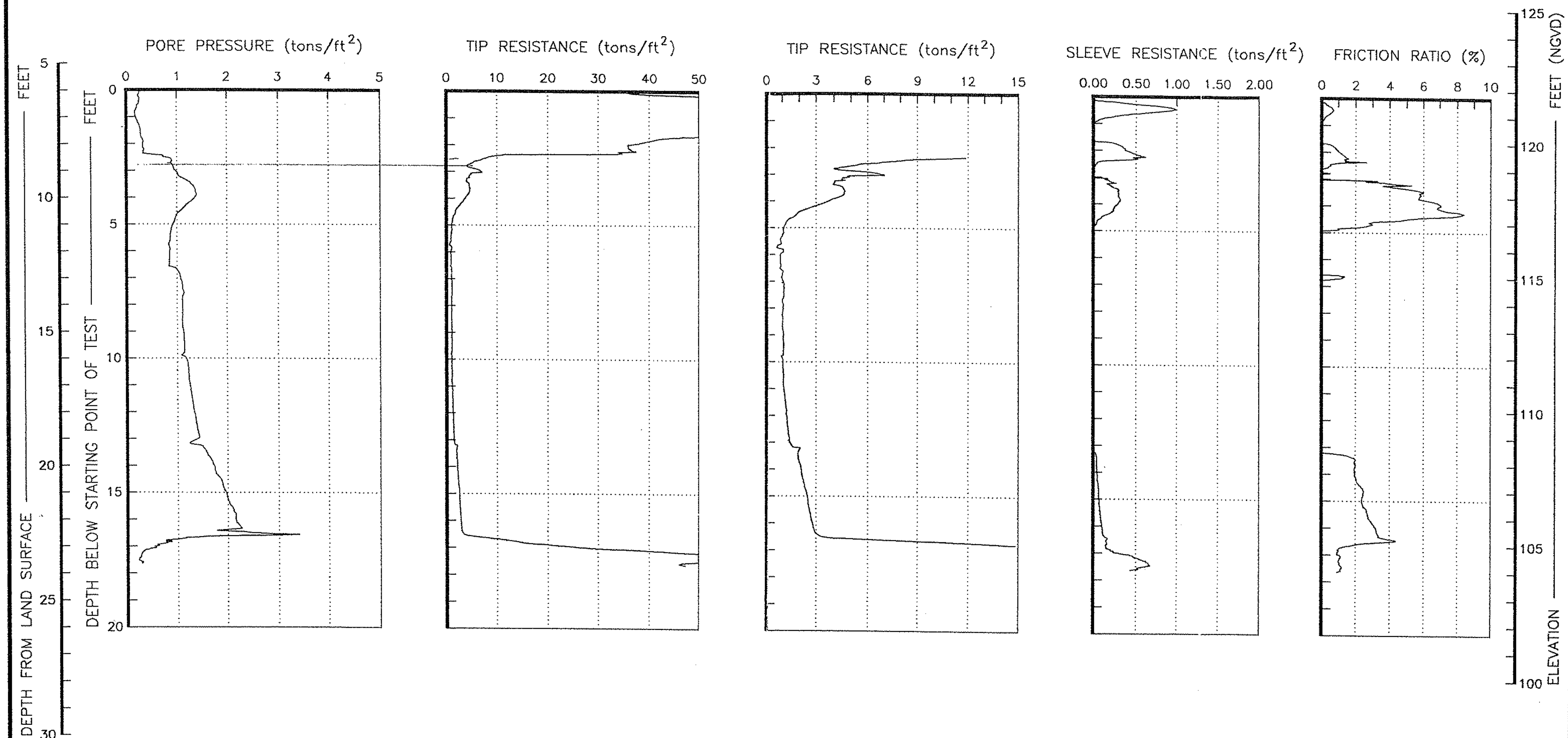
RESULTS OF PIEZOCONE TEST AT PC-5

Ardaman & Associates, Inc.
Geotechnical, Environmental and Materials Consultants

GEOTECHNICAL INVESTIGATION
SOUTHEAST LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

DRAWN BY: RBE	CHECKED BY: RW	DATE: 12/17/93	
FILE NO.: 93-029	APPROVED BY:		FIGURE: 8

FILE: 93-029 DATE: 03/04/94



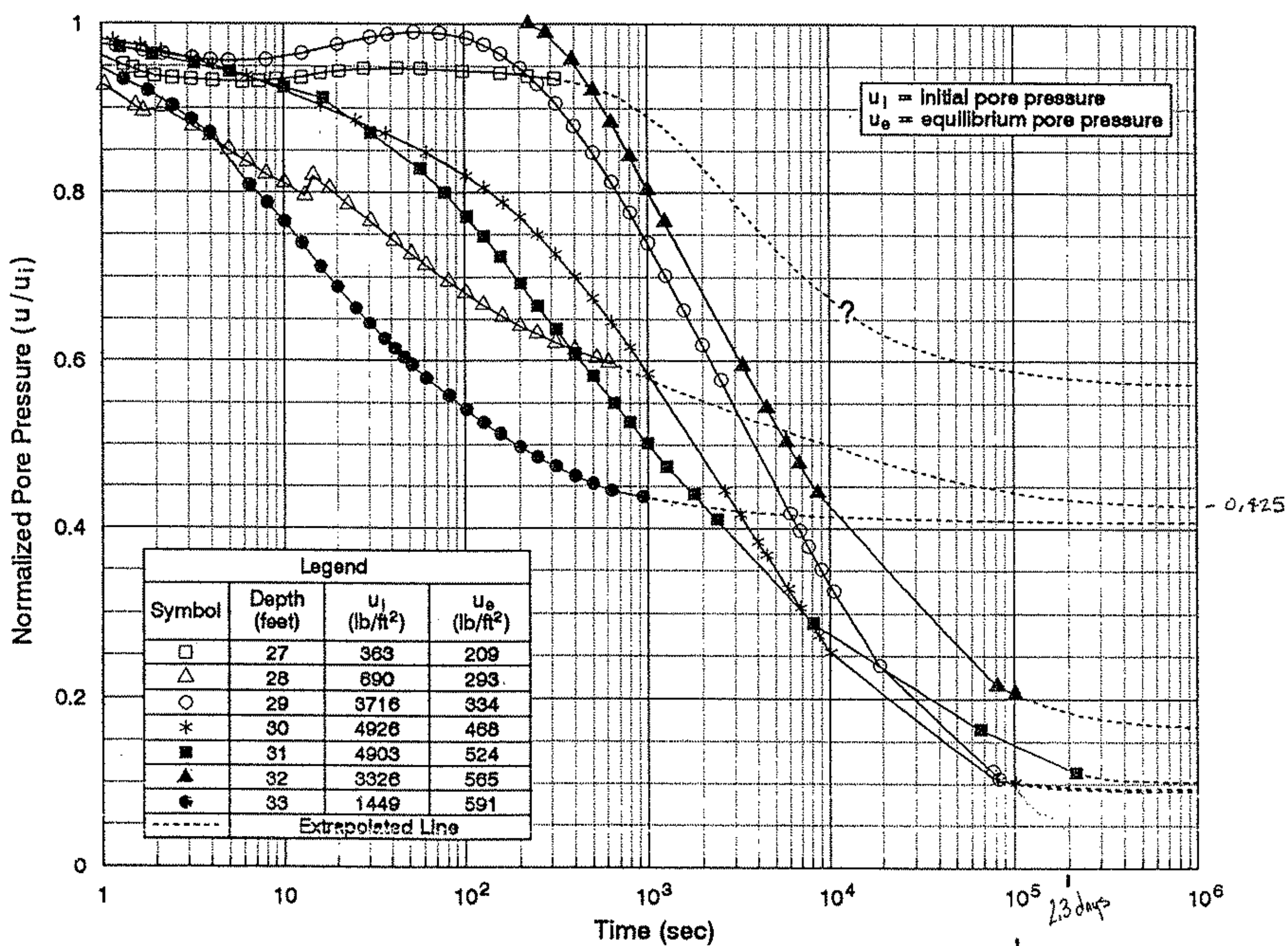
RESULTS OF PIEZOCONE TEST AT PC-6

Ardaman & Associates, Inc.
Geotechnical, Environmental and Materials Consultants

GEOTECHNICAL INVESTIGATION
SOUTHEAST LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

DRAWN BY: RBE	CHECKED BY: RW	DATE: 12/17/93
FILE NO.: 93-029	APPROVED BY:	FIGURE: 9

FILE: 93-029 DATE: 03/04/94



1.2 days

$u_1(\#) = u_e$

$\triangle_{28ft} = 690\ (0.425) = 293$

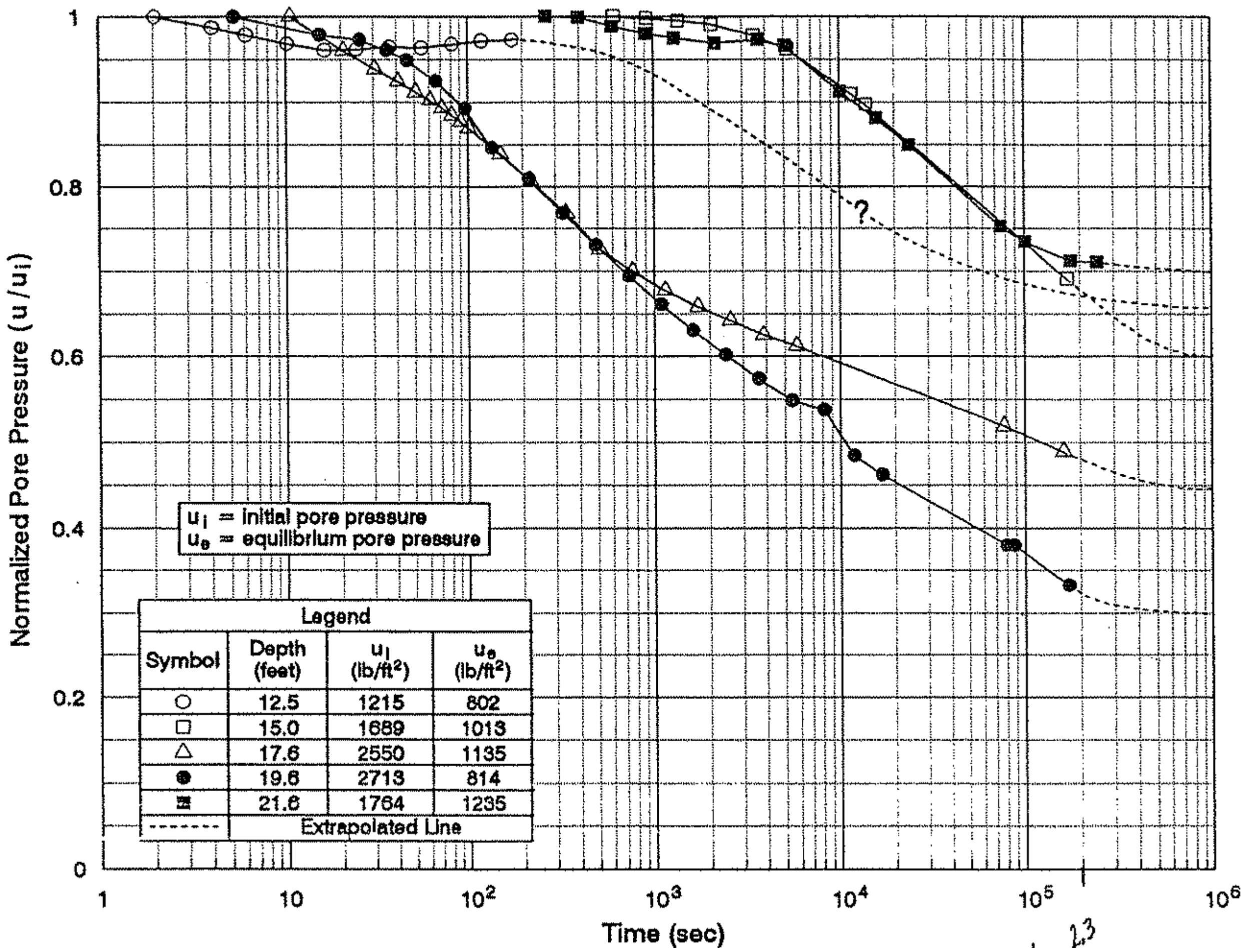
PORE PRESSURE VERSUS TIME RELATIONSHIPS FROM PIEZOPROBE TEST RESULTS AT PP-2

Ardaman & Associates, Inc.
Geotechnical, Environmental and Materials Consultants

GEOTECHNICAL INVESTIGATION
SOUTHEAST LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

DRAWN BY: SKB	CHECKED BY: SKB	DATE: 03-03-94
FILE NO. 93-029	APPROVED BY:	FIGURE NO. 10

E:\FL\FILES\93029.HBC\PP-T 2.DRW



Legend			
Symbol	Depth (feet)	u_i (lb/ft²)	u_e (lb/ft²)
○	12.5	1215	802
□	15.0	1689	1013
△	17.6	2550	1135
●	19.6	2713	814
▦	21.6	1764	1235
-------	Extrapolated Line		

1 2,3

1.2 days

E:\FLFILES\9302...\PP-T 6.DRW

PORE PRESSURE VERSUS TIME RELATIONSHIPS FROM PIEZOPROBE TEST RESULTS AT PP-6

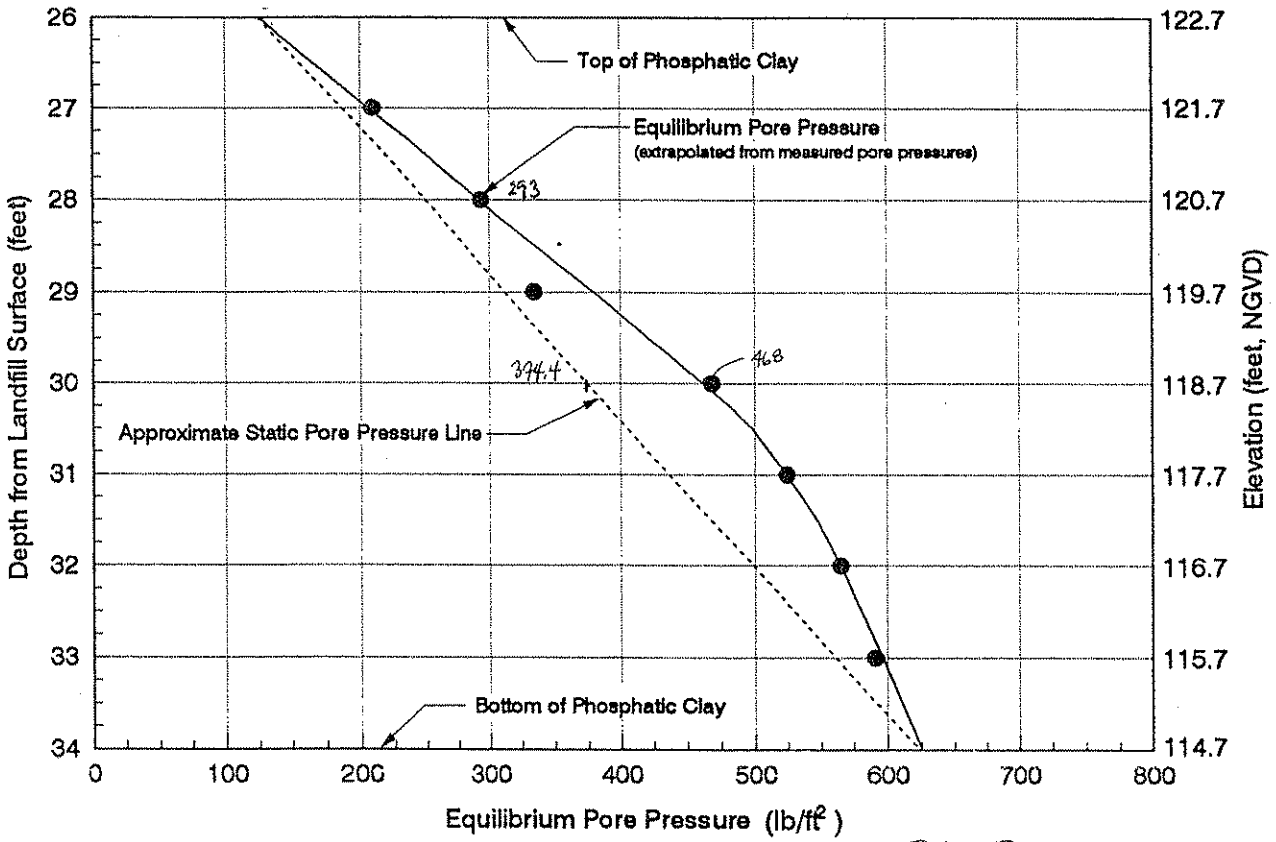
Ardaman & Associates, Inc.
Geotechnical, Environmental and Materials Consultants

GEOTECHNICAL INVESTIGATION
SOUTHEAST LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

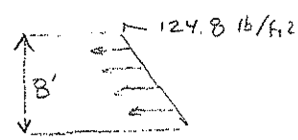
DRAWN BY: SKB	CHECKED BY: SKB	DATE: 03-03-94
FILE NO. 93-029	APPROVED BY:	FIGURE NO. 11

Assumes 2 ft
standing leachate
above clay

$2\text{ ft}\left(62.4\,\frac{lb}{ft^3}\right) = 124.8\,\frac{lb}{ft^2}$



8 ft Thick clay



$8 \times 62.4 = 499.2$
$+\ 124.8$
$624.0\ lb/ft^2$

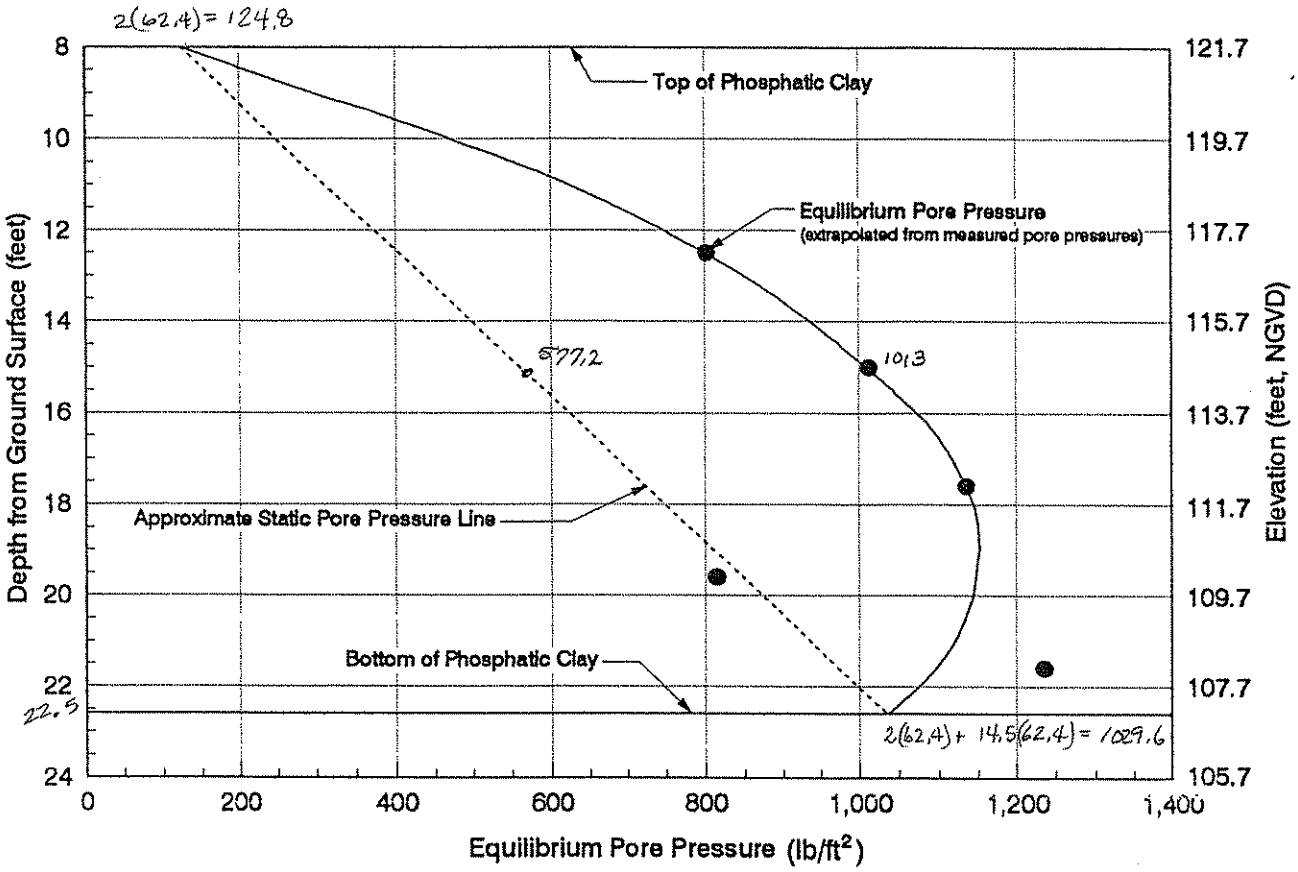
PORE PRESSURE VERSUS DEPTH RELATIONSHIP FROM PIEZOPROBE TEST RESULTS AT PP-2

Ardaman & Associates, Inc.
Geotechnical, Environmental and Materials Consultants

GEOTECHNICAL INVESTIGATION
SOUTHEAST LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

DRAWN BY: SKB	CHECKED BY: SKB	DATE: 03-03-94
FILE NO. 93-029	APPROVED BY:	FIGURE NO. 12

E:\FL\FILES\93029.rad\C\PP-D 2.DRW



Clay 14.5 ft THick

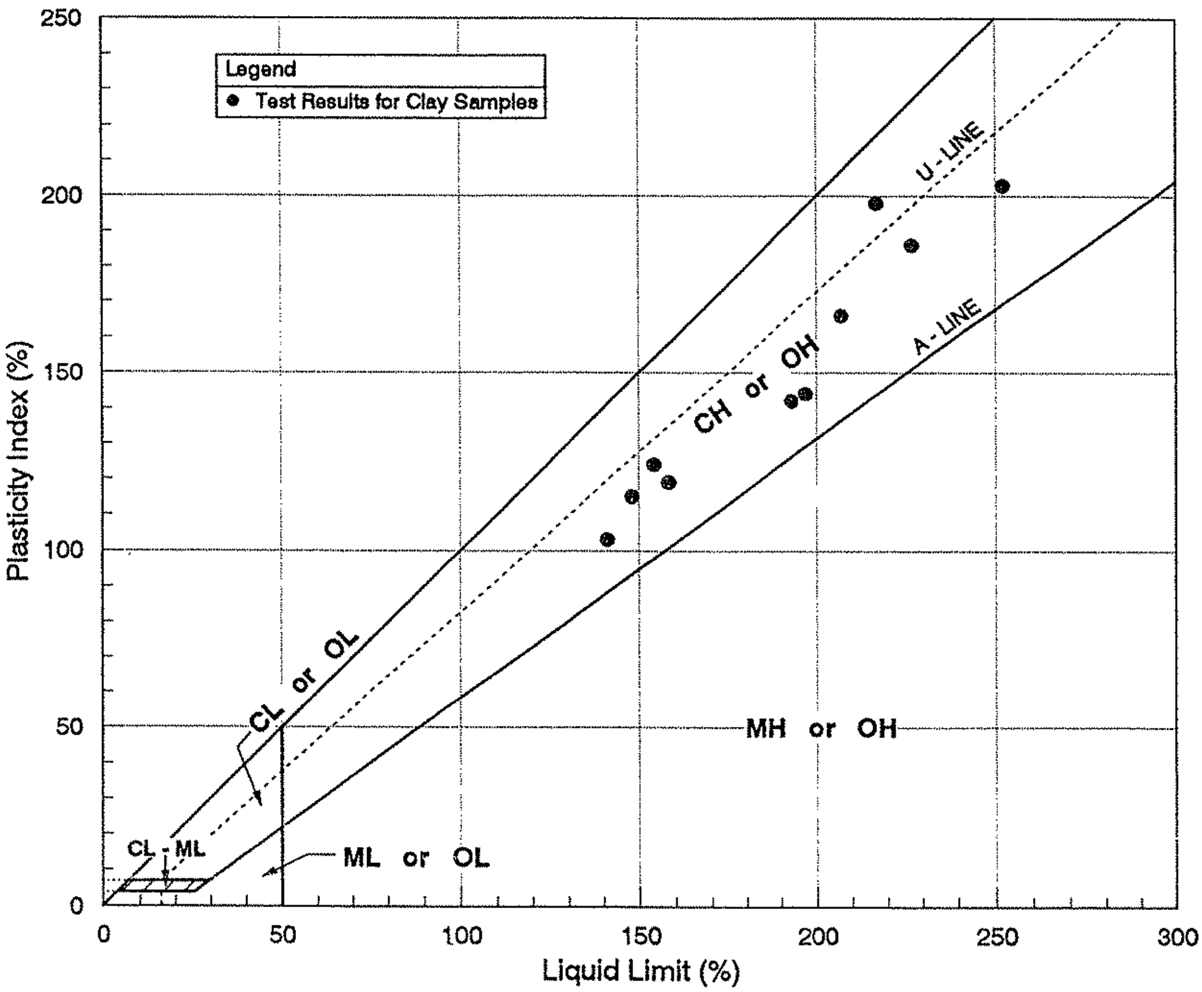
PORE PRESSURE VERSUS DEPTH RELATIONSHIP FROM PIEZOPROBE TEST RESULTS AT PP-6

Ardaman & Associates, Inc.
Geotechnical, Environmental and Materials Consultants

GEOTECHNICAL INVESTIGATION
SOUTHEAST LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

DRAWN BY: SKB	CHECKED BY: SKB	DATE: 03-03-94
FILE NO. 93-029	APPROVED BY:	FIGURE NO. 13

E:\FLFILES\93029\...SC\PP-D 6.DRW



PLASTICITY OF CLAY SAMPLES

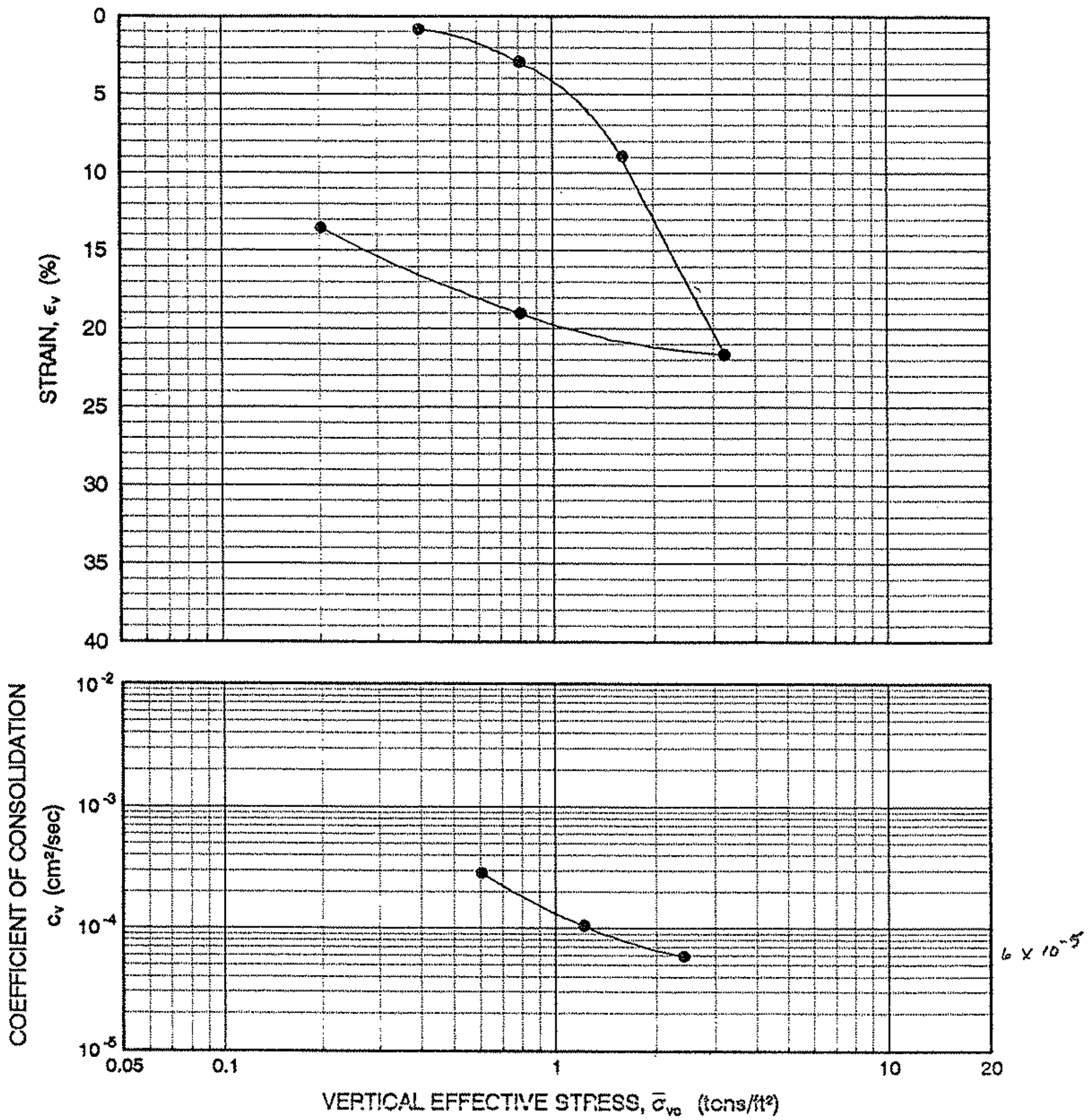
(Plasticity Chart Reproduced from ASTM D 2487 - 90)

Ardaman & Associates, Inc.
Geotechnical, Environmental and Materials Consultants

GEOTECHNICAL INVESTIGATION
SOUTHEAST LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

DRAWN BY: SKB	CHECKED BY: SKB	DATE: 03-03-94
FILE NO. 93-029	APPROVED BY:	FIGURE NO. 14

E:\FLFILES\93029.HBC\PLCHART.DRW



SAMPLE DATA

BORING NO.: TH-2
DEPTH (FEET): 31.9
DESCRIPTION: GRAY CLAY

SPECIMEN CONDITIONS	INITIAL	FINAL
MOISTURE CONTENT (%):	120.5	105.4
DRY DENSITY (lb/ft³):	40.1	45.9
VOID RATIO:	3.3	2.7

INDEX PROPERTIES

LIQUID LIMIT (%):	193
PLASTIC LIMIT (%):	51
PLASTICITY INDEX (%):	142
% PASSING NO. 200	100
SPECIFIC GRAVITY:	2.75 (estimated)

CONSOLIDATION PROPERTIES

IN SITU VERTICAL EFFECTIVE STRESS, $\bar{\sigma}_{vo}$ (lb/ft²):	2500
PRECONSOLIDATION PRESSURE, $\bar{\sigma}_{vm}$ (lb/ft²):	2500
VIRGIN COMPRESSION RATIO, CR:	0.41
SWELLING RATIO, SR:	0.07

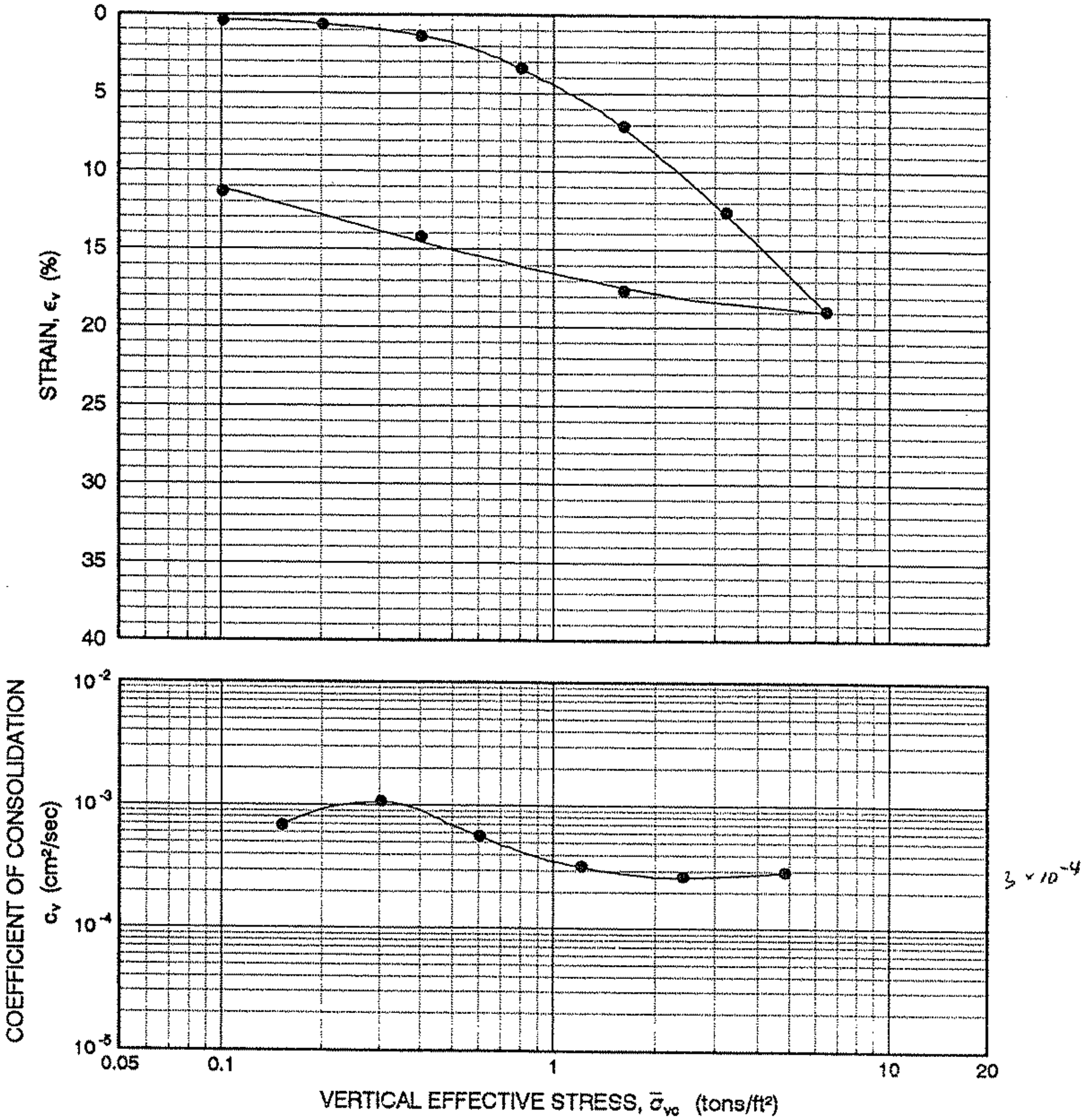
Ardaman & Associates, Inc.
Geotechnical, Environmental and Materials Consultants

CONSOLIDATION TEST RESULTS FOR SAMPLE US-4 OF BORING TH-2

GEOTECHNICAL INVESTIGATION
SOUTHEAST LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

DRAWN BY: SKB	CHECKED BY: SKB	DATE: 03-03-94
FILE NO. 93-029	APPROVED BY:	FIGURE NO. 15

E:\FLFILES\93029\HBC\CON-2_4.DRW



SAMPLE DATA

BORING NO.: TH-6
DEPTH (FEET): 10.3
DESCRIPTION: GRAY CLAY WITH VERTICAL SANDY CLAY SEAM

SPECIMEN CONDITIONS

SPECIMEN CONDITIONS	INITIAL	FINAL
MOISTURE CONTENT (%):	77.2	67.7
DRY DENSITY (lb/ft³):	54.7	61.3
VOID RATIO:	2.14	1.78

INDEX PROPERTIES

LIQUID LIMIT (%):	148
PLASTIC LIMIT (%):	33
PLASTICITY INDEX (%):	115
% PASSING NO. 200	99
SPECIFIC GRAVITY:	2.75 (estimated)

CONSOLIDATION PROPERTIES

IN SITU VERTICAL EFFECTIVE STRESS, $\bar{\sigma}_{vo}$ (lb/ft²):	605
PRECONSOLIDATION PRESSURE, $\bar{\sigma}_{vm}$ (lb/ft²):	2000
VIRGIN COMPRESSION RATIO, CR:	0.21
SWELLING RATIO, SR:	0.04

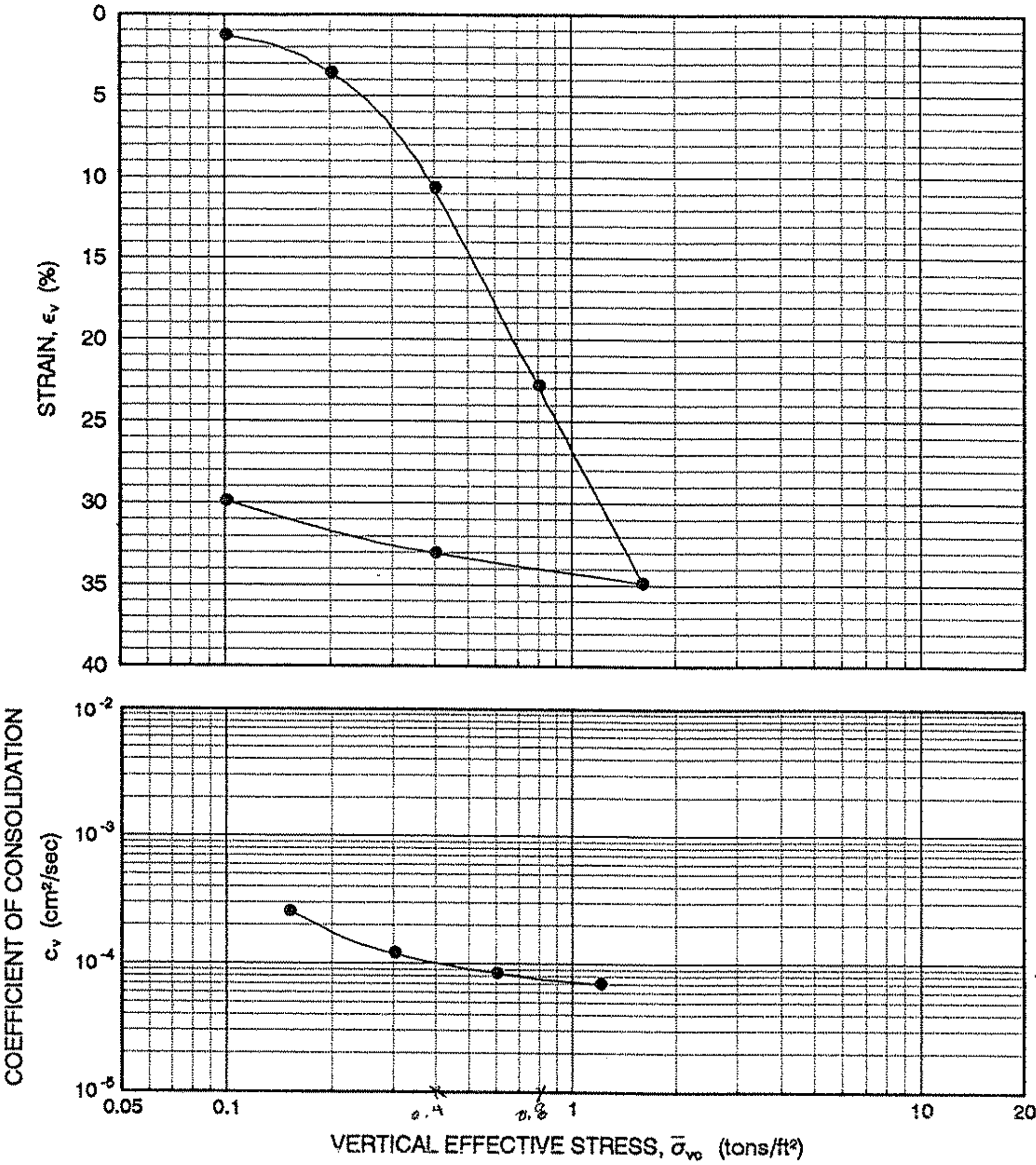
CONSOLIDATION TEST RESULTS FOR SAMPLE US-1A OF BORING TH-6

Ardaman & Associates, Inc.
Geotechnical, Environmental and Materials Consultants

GEOTECHNICAL INVESTIGATION
SOUTHEAST LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

DRAWN BY: SKB	CHECKED BY: SKB	DATE: 03-03-94
FILE NO. 93-029	APPROVED BY:	FIGURE NO. 16

E:\FLFILES\93\CON-6 1A.DRW



SAMPLE DATA

BORING NO.: TH-6
DEPTH (FEET): 16.5
DESCRIPTION: GRAY CLAY

SPECIMEN CONDITIONS	INITIAL	FINAL
MOISTURE CONTENT (%):	189.8	127.7
DRY DENSITY (lb/ft³):	27.7	39.3
VOID RATIO:	5.2	3.35

INDEX PROPERTIES

LIQUID LIMIT (%):	207
PLASTIC LIMIT (%):	41
PLASTICITY INDEX (%):	166
% PASSING NO. 200	100
SPECIFIC GRAVITY:	2.75 (estimated)

CONSOLIDATION PROPERTIES

IN SITU VERTICAL EFFECTIVE STRESS, $\bar{\sigma}_{vo}$ (lb/ft²):	560
PRECONSOLIDATION PRESSURE, $\bar{\sigma}_{vm}$ (lb/ft²):	560
VIRGIN COMPRESSION RATIO, CR:	0.40
SWELLING RATIO, SR:	0.04

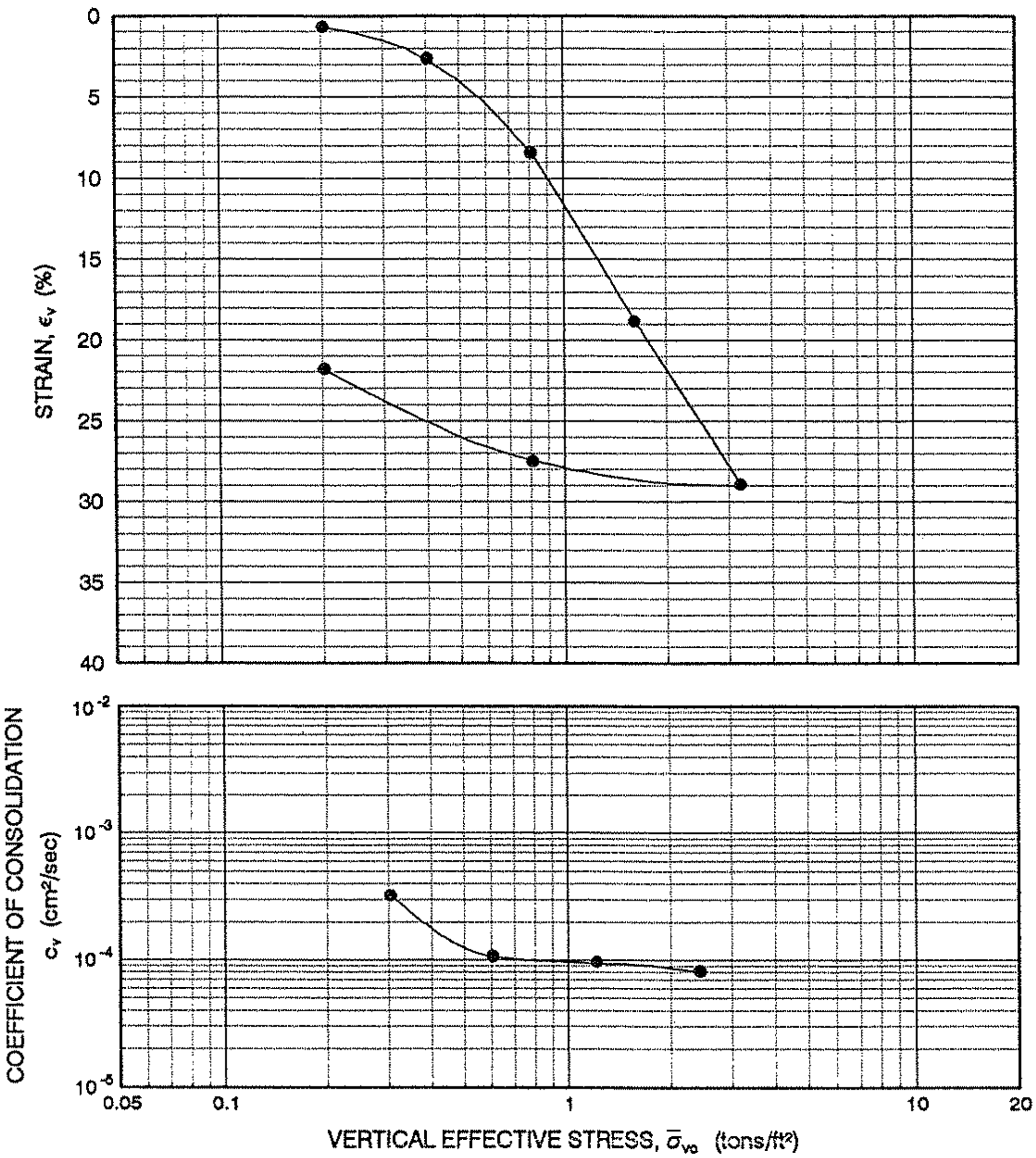
E:\FLFILES\930...\CON-6 4.DRW

CONSOLIDATION TEST RESULTS FOR SAMPLE US-4 OF BORING TH-6

Ardaman & Associates, Inc.
Geotechnical, Environmental and Materials Consultants

GEOTECHNICAL INVESTIGATION
SOUTHEAST LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

DRAWN BY: SKB	CHECKED BY: SKB	DATE: 03-03-94
FILE NO. 93-029	APPROVED BY:	FIGURE NO. 17



8×10^{-5}

SAMPLE DATA

BORING NO.: TH-6
DEPTH (FEET): 20.3
DESCRIPTION: GRAY CLAY

SPECIMEN CONDITIONS	INITIAL	FINAL
MOISTURE CONTENT (%):	120.9	90.6
DRY DENSITY (lb/ft³):	39.9	50.8
VOID RATIO:	3.3	2.36

INDEX PROPERTIES

LIQUID LIMIT (%):	154
PLASTIC LIMIT (%):	30
PLASTICITY INDEX (%):	124
% PASSING NO. 200	100
SPECIFIC GRAVITY:	2.75 (estimated)

CONSOLIDATION PROPERTIES

IN SITU VERTICAL EFFECTIVE STRESS, $\bar{\sigma}_{vo}$ (lb/ft²):	850
PRECONSOLIDATION PRESSURE, $\bar{\sigma}_{vm}$ (lb/ft²):	1150
VIRGIN COMPRESSION RATIO, CR:	0.34
SWELLING RATIO, SR:	0.06

E:\FLFILES\9302... \CON-6 6.DRW

CONSOLIDATION TEST RESULTS FOR SAMPLE US-6 OF BORING TH-6

Ardaman & Associates, Inc.
Geotechnical, Environmental and Materials Consultants

GEOTECHNICAL INVESTIGATION
SOUTHEAST LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

DRAWN BY: SKB	CHECKED BY: SKB	DATE: 03-03-94
FILE NO. 93-029	APPROVED BY:	FIGURE NO. 18

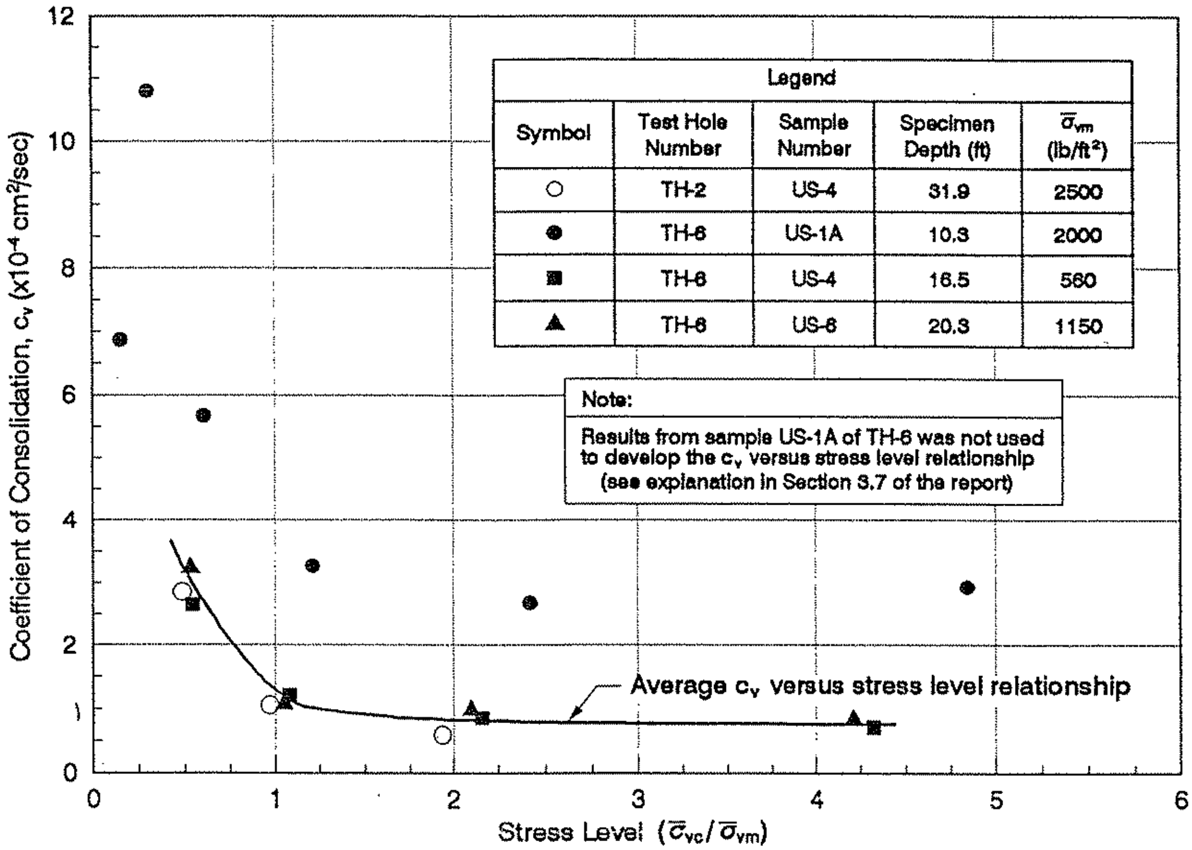
SCS ENGINEERS

SHEET ______ OF ______

CLIENT HCSWMD	PROJECT SCLF	JOB NO. 09200020.11
SUBJECT Clay cap evaluation	BY Sheila	DATE 4-4-01
	CHECKED	DATE

FROM ARDAMAN REPORT "GEOTECHNICAL INVESTIGATION AT SOUTHEAST LANDFILL, HILLSBOROUGH COUNTY, FLORIDA" DATED MARCH 7, 1994, DETERMINE TOP OF CLAY, BOTTOM OF CLAY FROM RESULTS OF PIEZOCONE TESTS FOR PC-1 ... PC-6.

PIEZO	EST LAND SURFACE	DEPTH FROM LS (ARDAMAN) TOP	DEPTH FROM LS (ARDAMAN) BOTTOM	CLAY TOP EL	CLAY BOTTOM EL	CLAY THICK FT
PC-1	156	39	41.75	156-39 = 117	156-41.75 = 114.25	2.75
PC-2	146	25.8	32.3	120.2	113.7	6.5
PC-3	145	24.0	31.2	121.0	113.8	7.2
PC-4	131.8	< 17	24.5	< 114.8	107.3	> 7.5
PC-5	125.7	< 7.5	21.6	< 118.2	104.1	> 14.1
PC-6	127	8.8	22.5	118.2	104.5	13.7



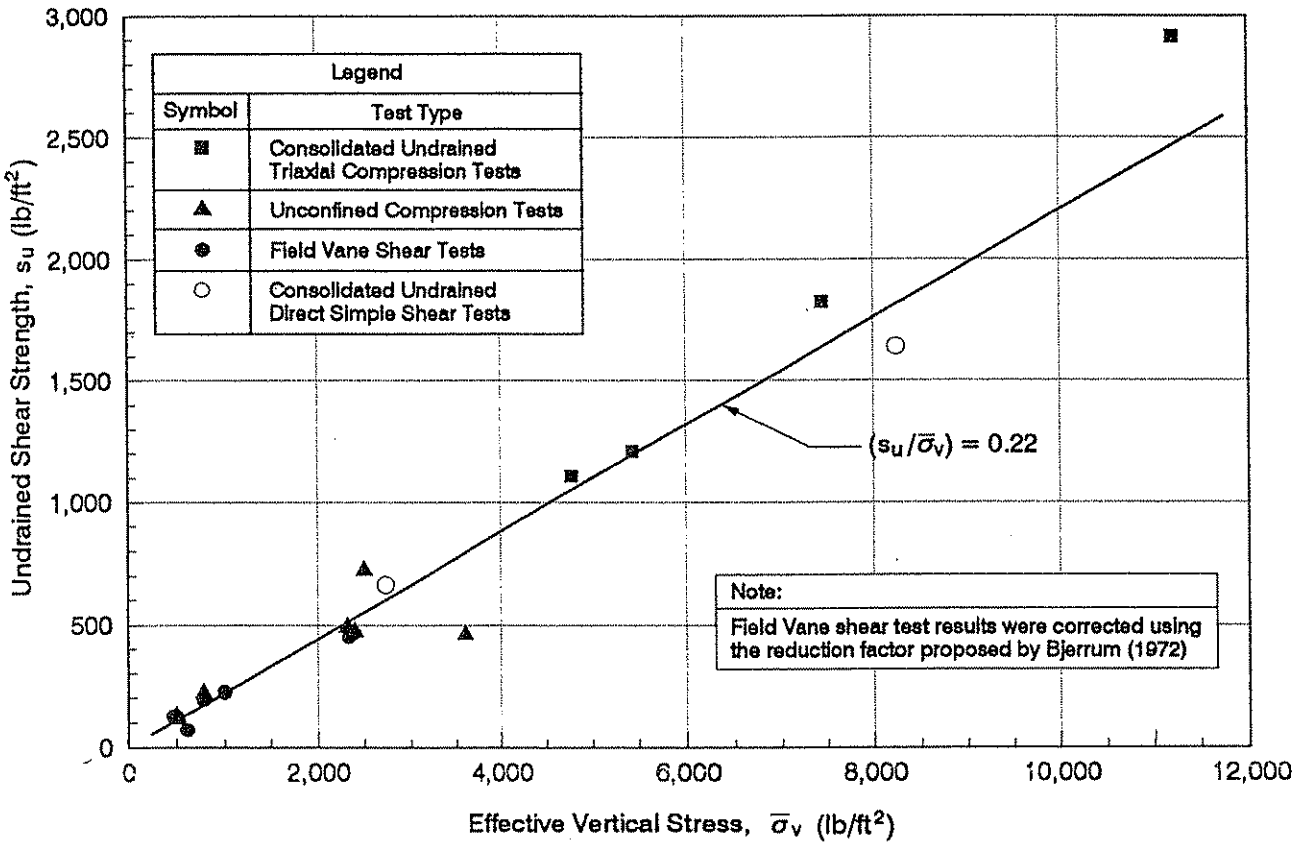
E:\FLFILES\93029.HBC\COEF-CON.DRW

COEFFICIENT OF CONSOLIDATION VERSUS STRESS LEVEL FOR CLAY SPECIMENS

Ardaman & Associates, Inc.
Geotechnical, Environmental and Materials Consultants

GEOTECHNICAL INVESTIGATION
SOUTHEAST LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

DRAWN BY: SKB	CHECKED BY: SKB	DATE: 03-03-94
FILE NO. 93-029	APPROVED BY:	FIGURE NO. 19



UNDRAINED SHEAR STRENGTH VERSUS EFFECTIVE VERTICAL STRESS RELATIONSHIP

Ardaman & Associates, Inc.
Geotechnical, Environmental and Materials Consultants

GEOTECHNICAL INVESTIGATION
SOUTHEAST LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

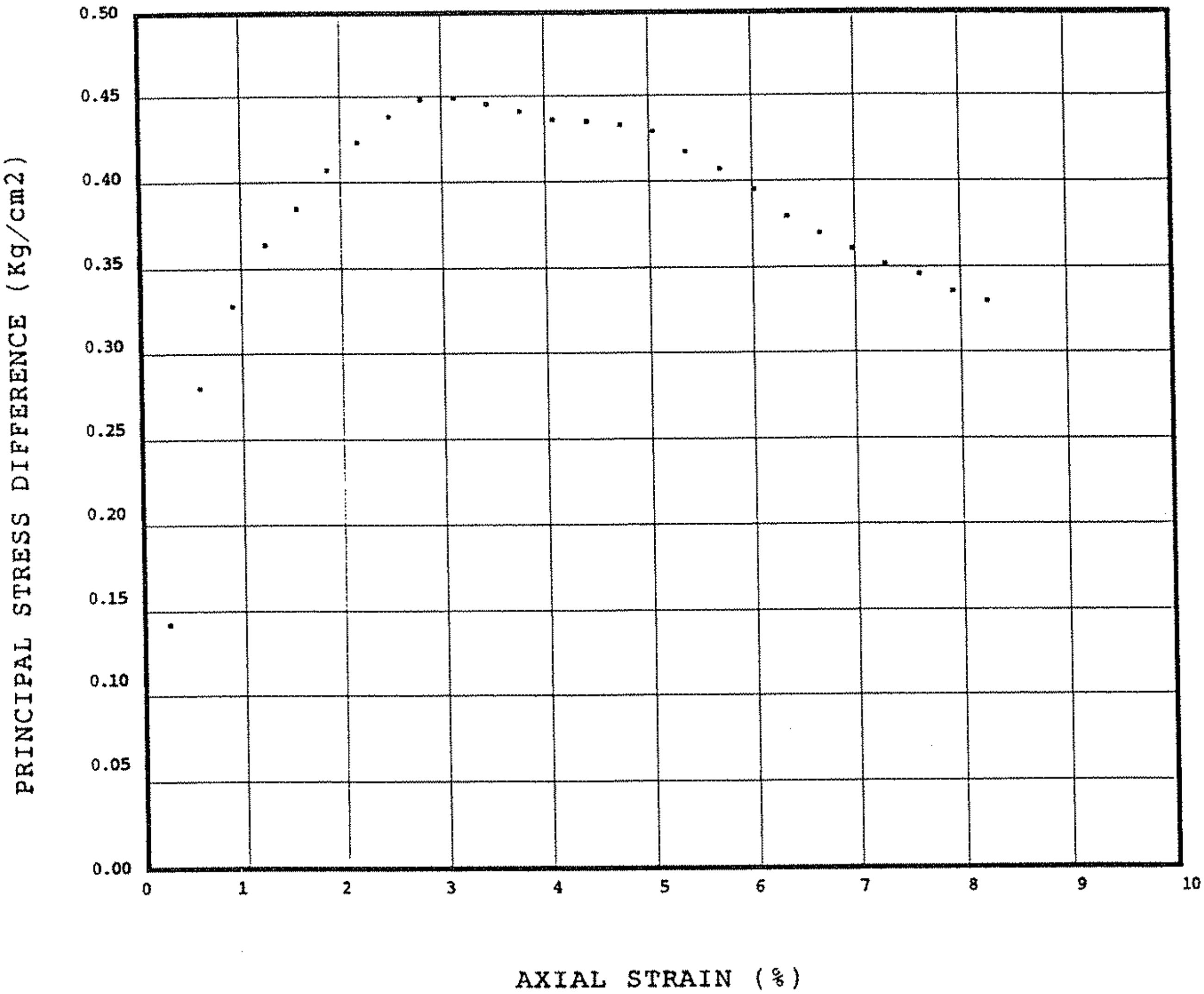
DRAWN BY: SKB	CHECKED BY: SKB	DATE: 03-03-94
FILE NO. 93-029	APPROVED BY:	FIGURE NO. 20

E:\FLFILES\93029.HBC\SU-SIGV.DRW

Appendix 1

Results of Unconfined Compression Tests

Sample Name : SITE 1, US 2, BOTTOM



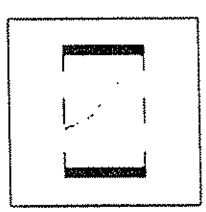
Dry density 44.1 pcf

Water content 106.0 %

Saturation 101.4 %

Cell pressure 0.00 kg/cm2

Strain rate 1.00%/min



TYPE OF FAILURE

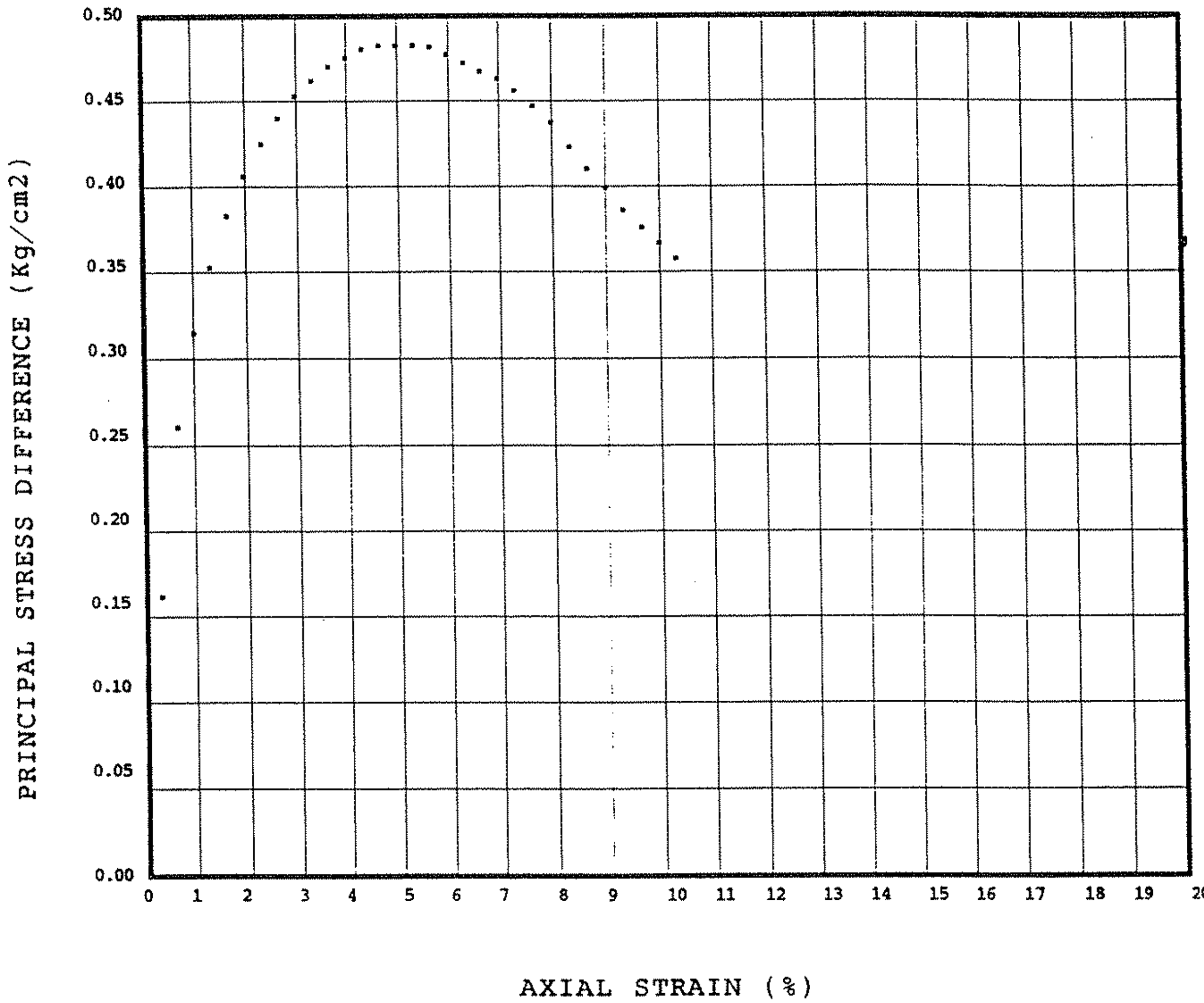
UNCONSOLIDATED – UNDRAINED TRIAXIAL COMPRESSION TEST

Ardaman & Associates Inc.

GEOTECHNICAL INVESTIGATION
SOUTHEAST LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

DRAWN BY:	CHECKED BY:	DATE: 3/2/94
FILE NO. 93-029	APPROVED BY:	

Sample Name : SITE 2, US 2, QU



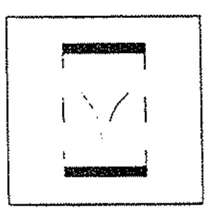
Dry density 46.2 pcf

Water content 97.7 %

Saturation 99.6 %

Cell pressure 0.00 kg/cm2

Strain rate 1.00%/min



TYPE OF FAILURE

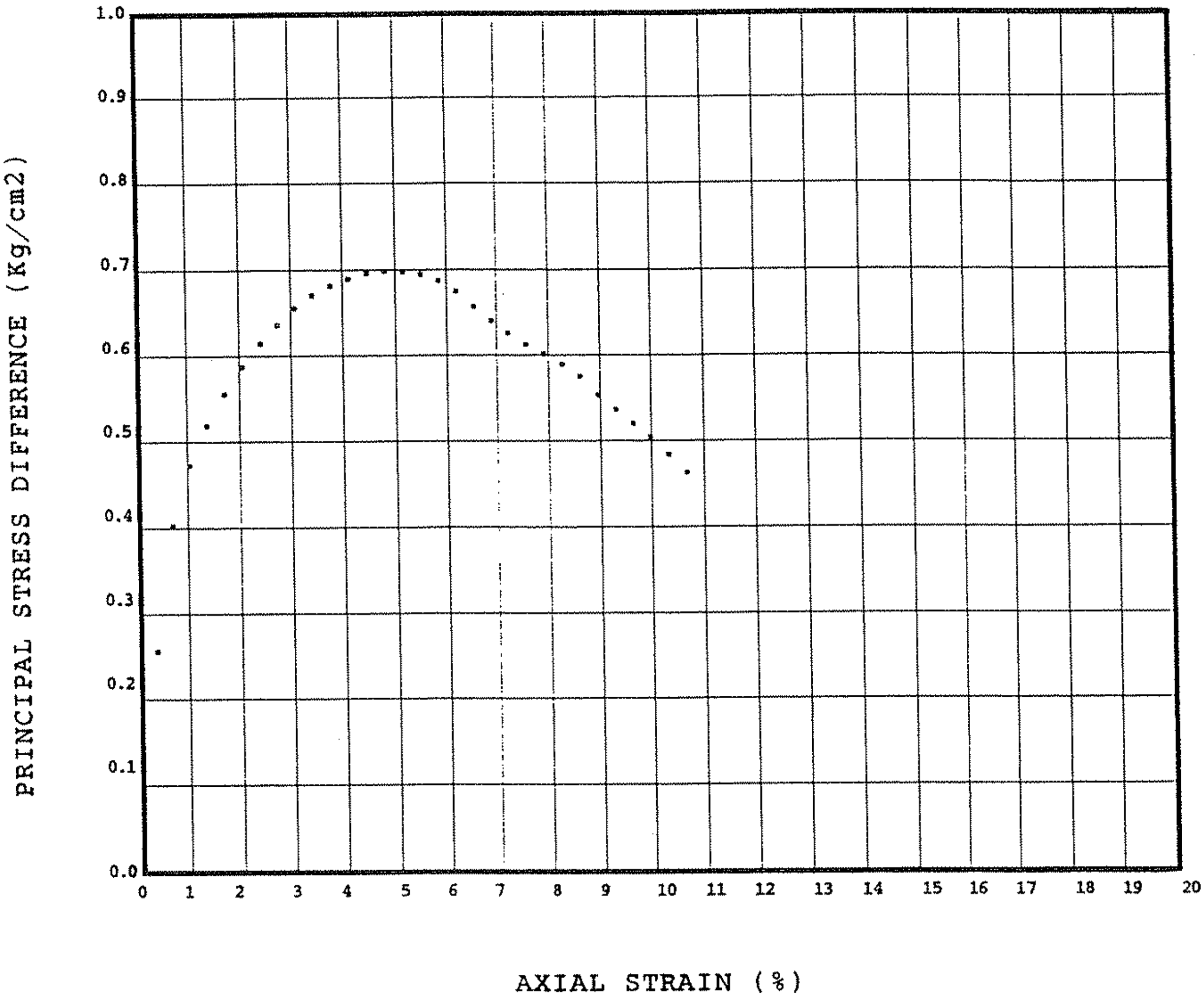
UNCONSOLIDATED - UNDRAINED TRIAXIAL COMPRESSION TEST

Ardaman & Associates Inc.

GEOTECHNICAL INVESTIGATION
SOUTHEAST LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

DRAWN BY:	CHECKED BY:	DATE: 3/2/94
FILE NO. 93-029	APPROVED BY:	

Sample Name : SITE 2, US 4, B4, QU 1



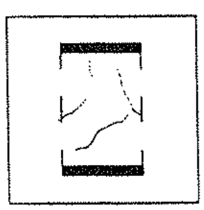
Dry density 42.2 pcf

Water content 110.3 %

Saturation 99.5 %

Cell pressure 0.00 kg/cm2

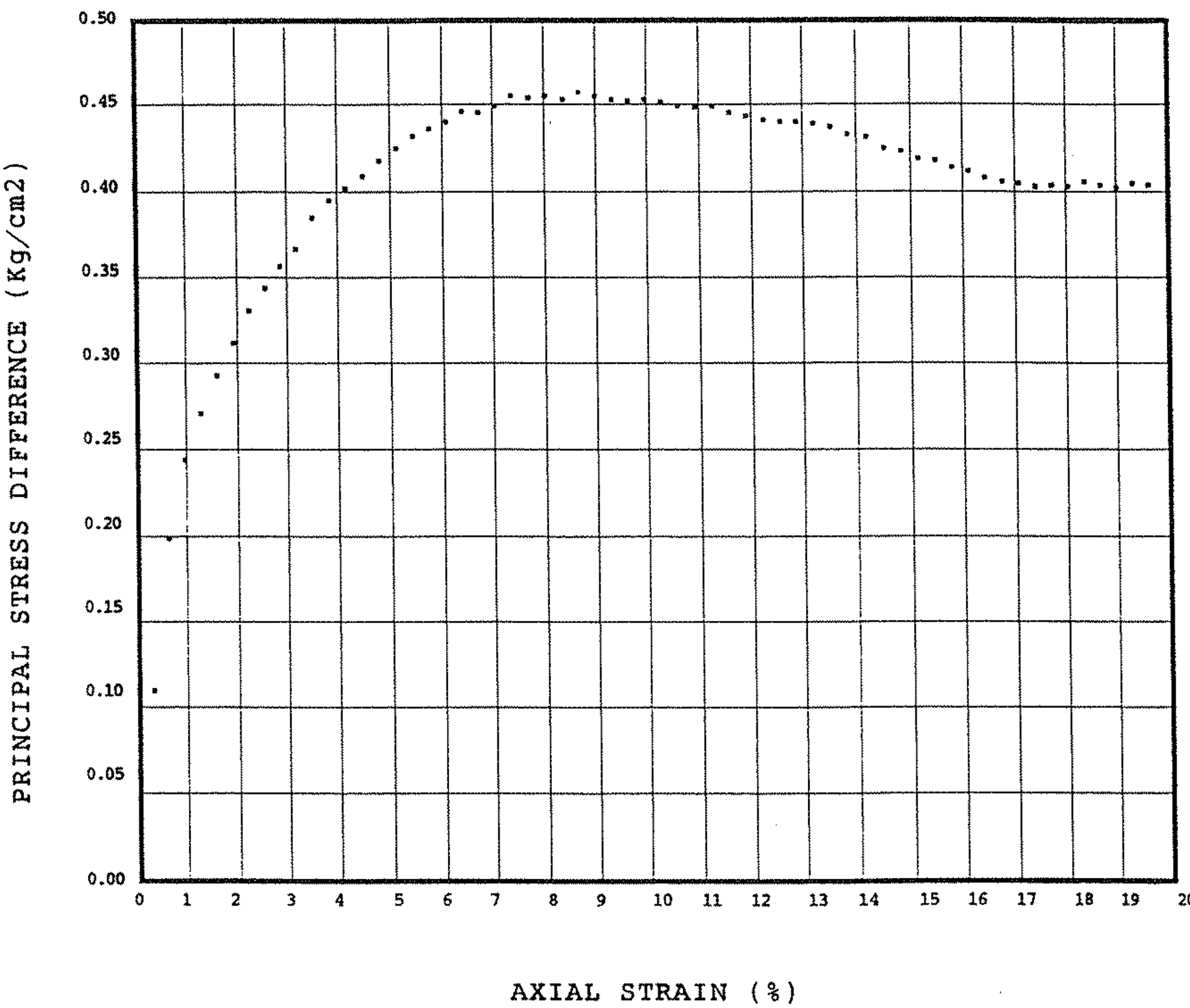
Strain rate 1.00%/min



TYPE OF FAILURE

UNCONSOLIDATED - UNDRAINED TRIAXIAL COMPRESSION TEST	
Ardaman & Associates Inc.	
GEOTECHNICAL INVESTIGATION SOUTHEAST LANDFILL HILLSBOROUGH COUNTY, FLORIDA	
DRAWN BY:	CHECKED BY:
DATE: 3/2/94	
FILE NO. 93-029	APPROVED BY: [signature]

Sample Name : SITE 2, US 4, B 1, QU 2



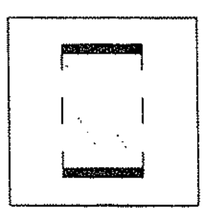
Dry density 48.7 pcf

Water content 92.8 %

Saturation 101.8 %

Cell pressure 0.00 kg/cm2

Strain rate 1.00%/min



TYPE OF FAILURE

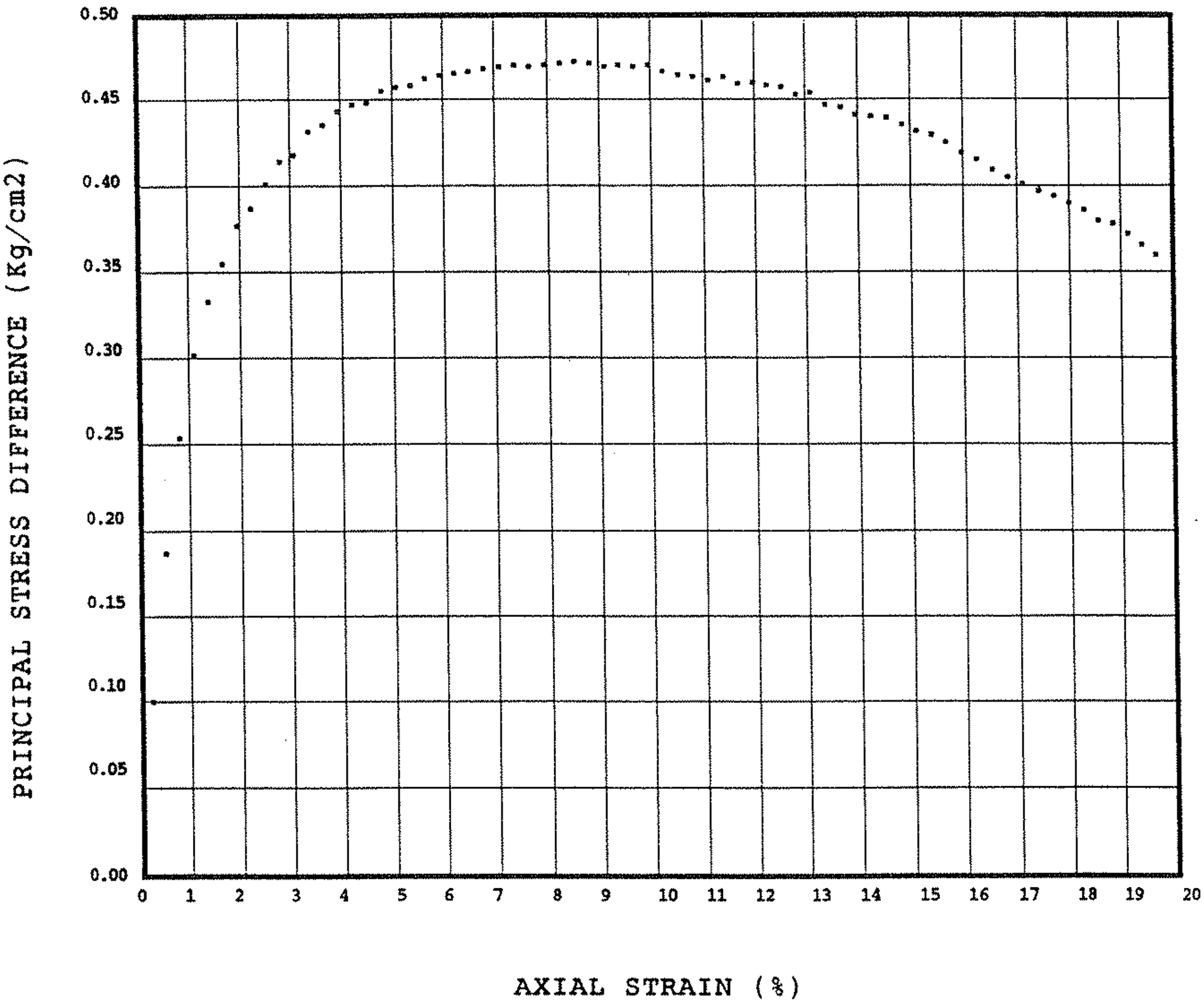
UNCONSOLIDATED - UNDRAINED TRIAXIAL COMPRESSION TEST

Ardaman & Associates Inc.

GEOTECHNICAL INVESTIGATION
SOUTHEAST LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

DRAWN BY:	CHECKED BY:	DATE: 3/2/94
FILE NO. 93-029	APPROVED BY:	

Sample Name : TH-6, US 1 8.5-10.5



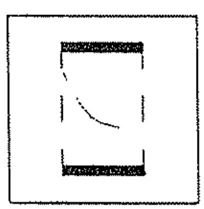
Dry density 50.6 pcf

Water content 85.9 %

Saturation 98.8 %

Cell pressure 0.00 kg/cm2

Strain rate 1.00%/min



TYPE OF FAILURE

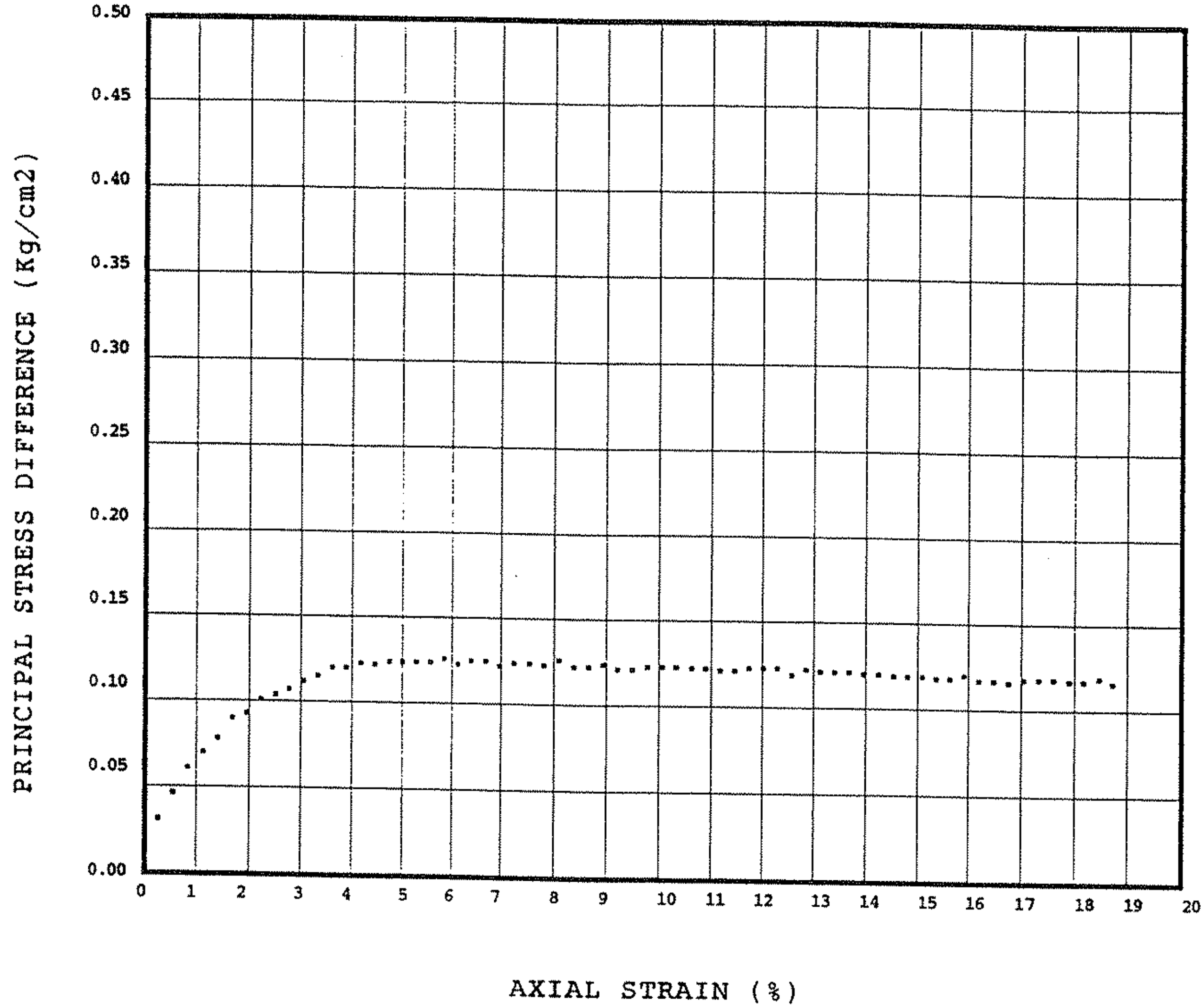
UNCONSOLIDATED - UNDRAINED TRIAXIAL COMPRESSION TEST

Ardaman & Associates Inc.

GEOTECHNICAL INVESTIGATION
SOUTHEAST LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

DRAWN BY:	CHECKED BY:	DATE: 3/2/94
FILE NO. 93-029	APPROVED BY:	

Sample Name : TH-6, US 2, 11-13



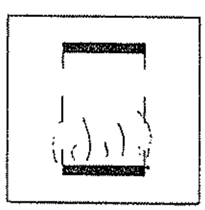
Dry density 27.1 pcf

Water content 199.1 %

Saturation 102.8 %

Cell pressure 0.00 kg/cm2

Strain rate 1.00%/min



TYPE OF FAILURE

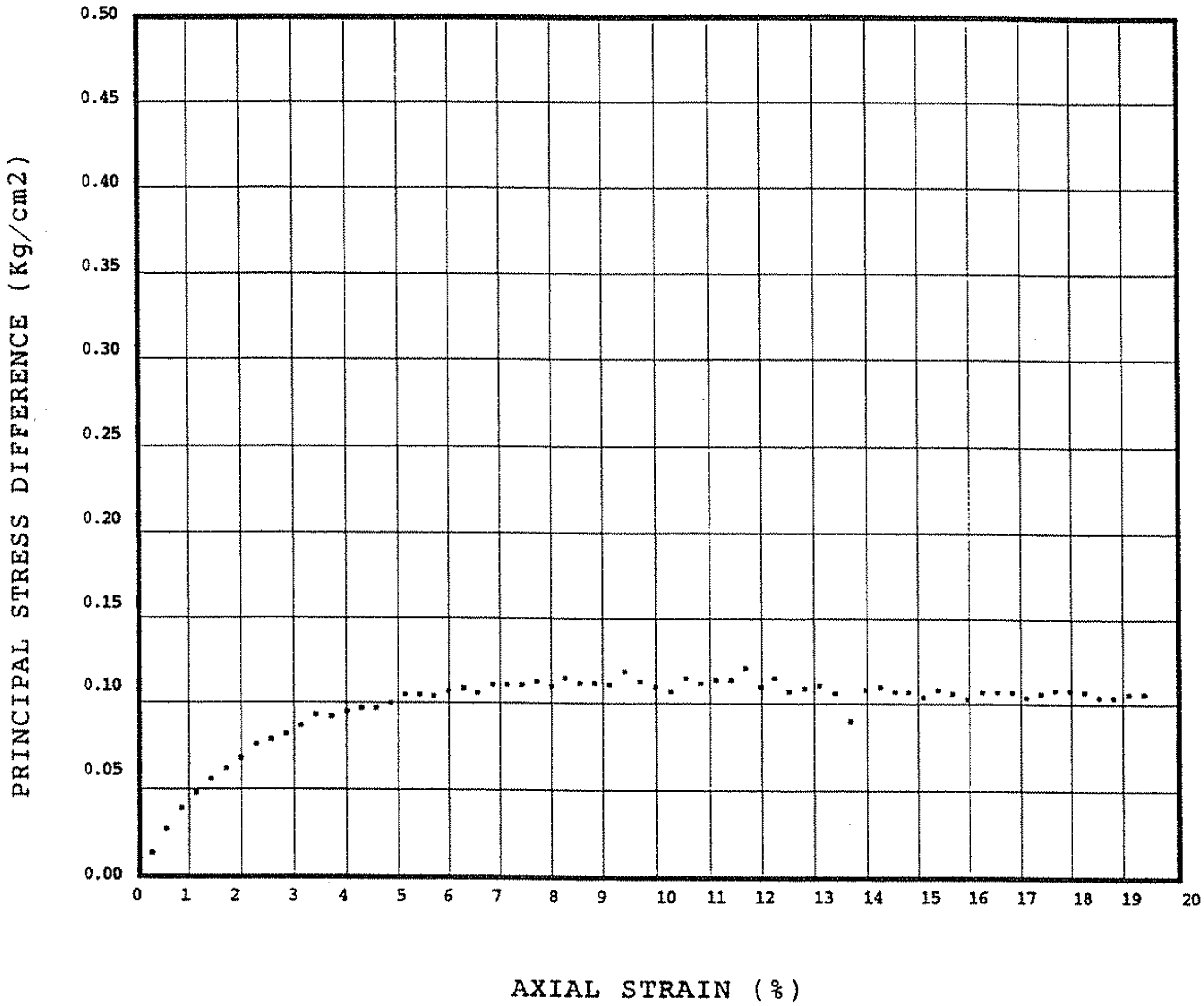
UNCONSOLIDATED - UNDRAINED TRIAXIAL COMPRESSION TEST

Ardaman & Associates Inc.

GEOTECHNICAL INVESTIGATION
SOUTHEAST LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

DRAWN BY:	CHECKED BY:	DATE: 3/2/94
FILE NO. 93-029	APPROVED BY:	

Sample Name : TH-6, US 4, 15-17



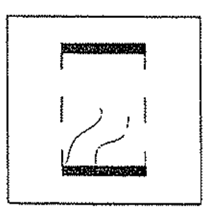
Dry density 33.0 pcf

Water content 154.0 %

Saturation 100.9 %

Cell pressure 0.00 kg/cm2

Strain rate 0.00%/min



TYPE OF FAILURE

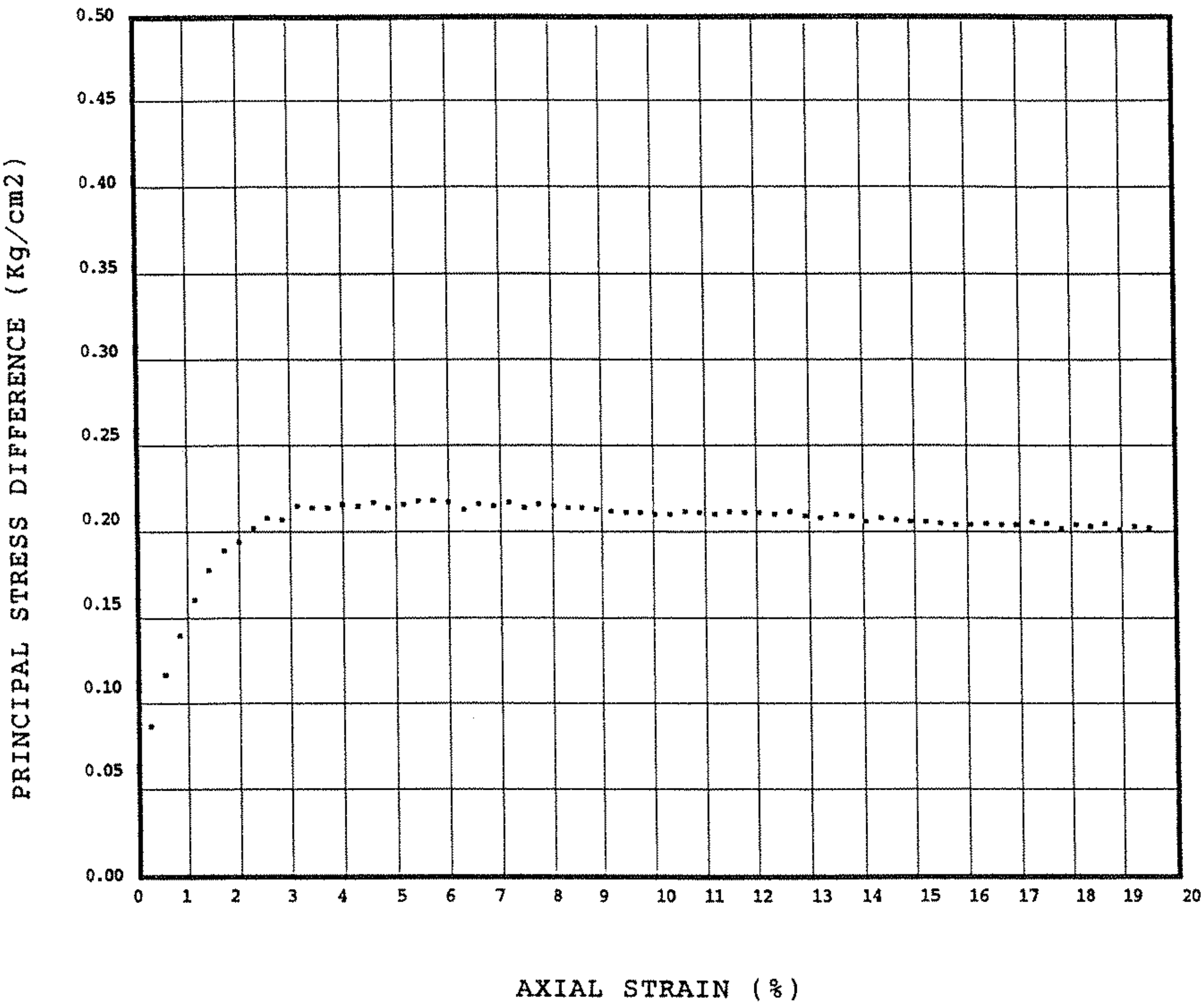
UNCONSOLIDATED - UNDRAINED TRIAXIAL COMPRESSION TEST

Ardaman & Associates Inc.

GEOTECHNICAL INVESTIGATION
SOUTHEAST LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

DRAWN BY:	CHECKED BY:	DATE: 3/2/94
FILE NO. 93-029	APPROVED BY:	

Sample Name : TH-6 US 6, 19-21



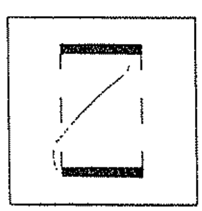
Dry density 39.6 pcf

Water content 122.8 %

Saturation 101.3 %

Cell pressure 0.00 kg/cm2

Strain rate 1.00%/min



TYPE OF FAILURE

UNCONSOLIDATED - UNDRAINED TRIAXIAL COMPRESSION TEST

Ardaman & Associates Inc.

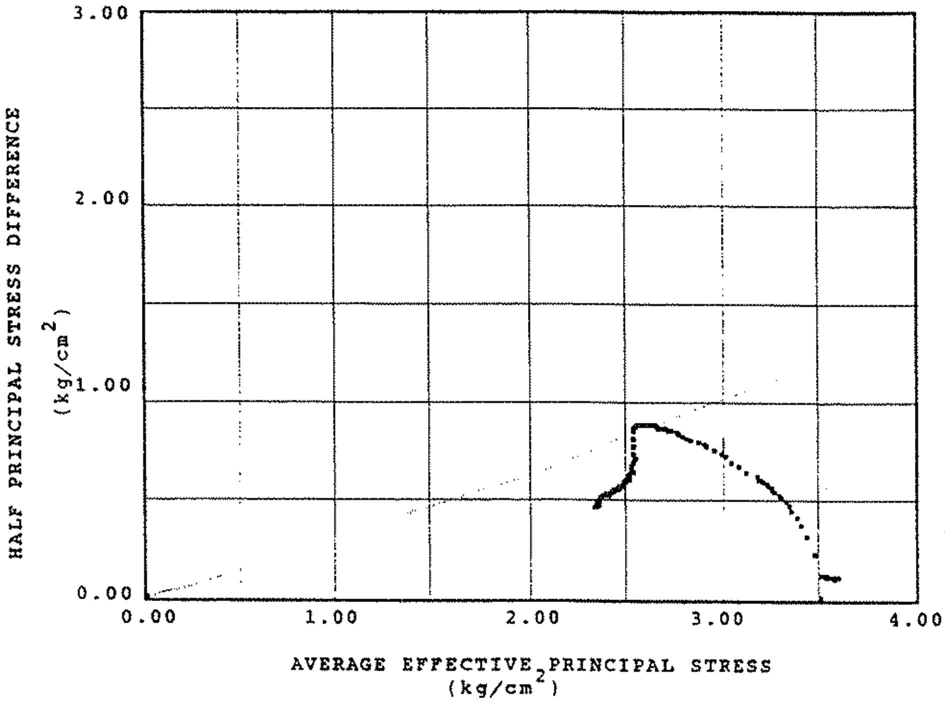
GEOTECHNICAL INVESTIGATION
SOUTHEAST LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

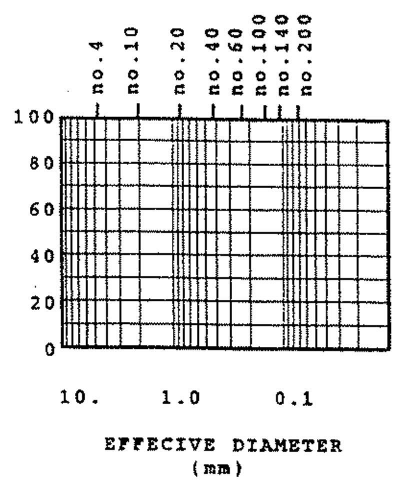
DRAWN BY:	CHECKED BY:	DATE: 3/2/94
FILE NO. 93-029	APPROVED BY:	

Appendix 2

Results of Consolidated Undrained Triaxial Compression Tests

CONSOLIDATED - UNDRAINED TRIAXIAL COMPRESSION TEST

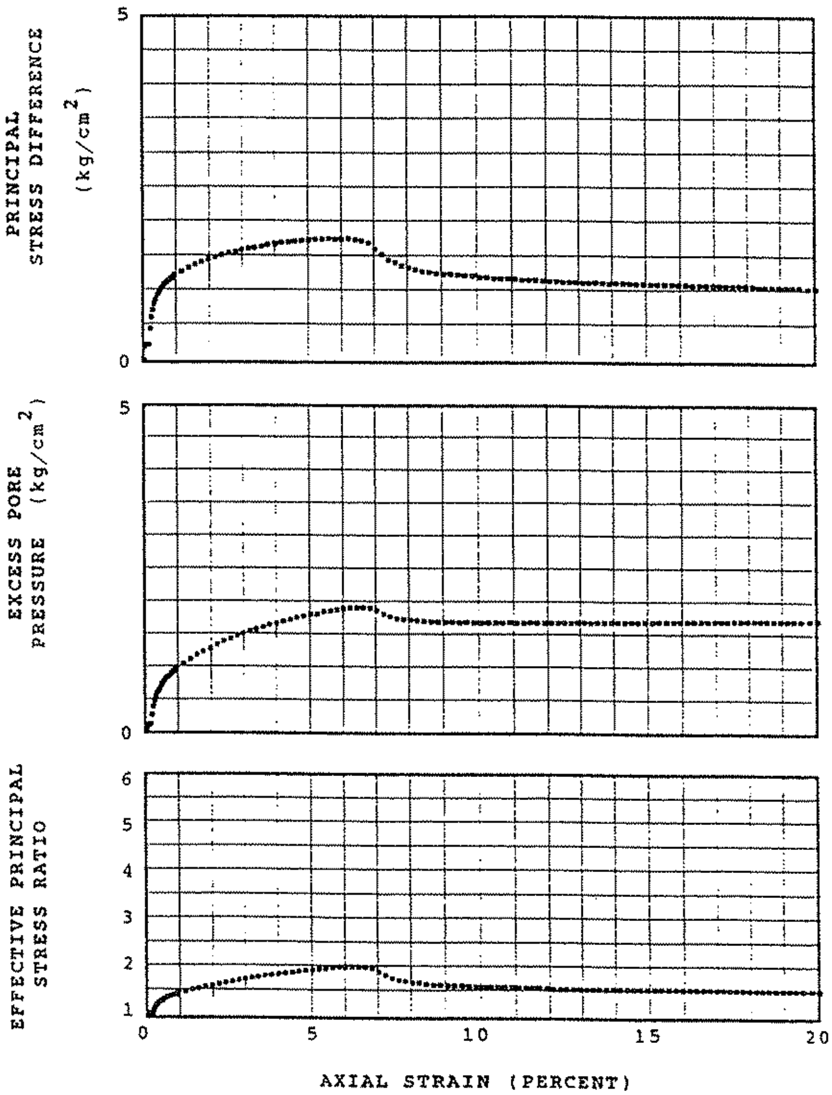




ASSUMED SPECIFIC GRAVITY 2.75

SAMPLE DESCRIPTION

clay slime



SAMPLE ID
SITE1, US2,
TOP

TYPE OF SPECIMEN Undisturbed
STRAIN RATE 1.11%/hr
EFFECTIVE CONSOLIDATION
PRESSURE 3.5 kg/cm^2 (7170 psf
BACK PRESSURE 10.5 kg/cm^2

INTIAL CONDITIONS
WATER CONTENT 105.0%
DRY DENSITY 44.7 pcf
VOID RATIO 2.839
SATURATION 100.0%

PRE-SHEAR CONDITIONS
WATER CONTENT 75.4%
DRY DENSITY 55.8 pcf
VOID RATIO 2.075
SATURATION 99.9%
B - FACTOR 98.0%

MAXIMUM STRESS RATIO
AXIAL STRAIN 6.17%
σ'_1 3.44 kg/cm^2
σ'_3 1.67 kg/cm^2

A - FACTOR 1.06 Normal Consolidation

MAXIMUM STRESS DIFFERENCE
AXIAL STRAIN 6.17%
σ'_1 3.44 kg/cm^2
σ'_3 1.67 kg/cm^2

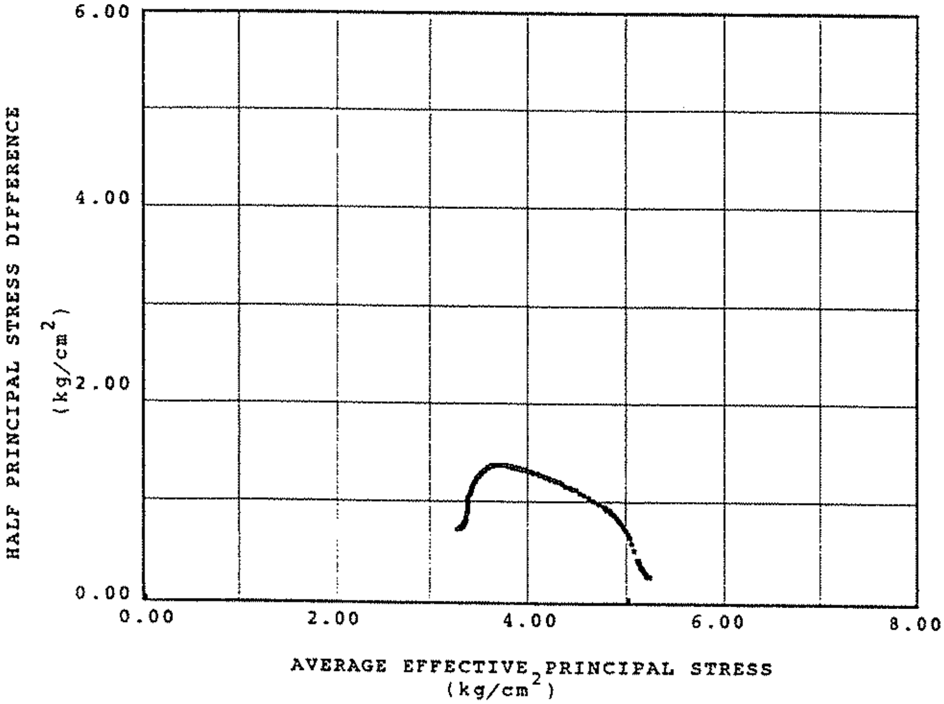
A - FACTOR 1.06

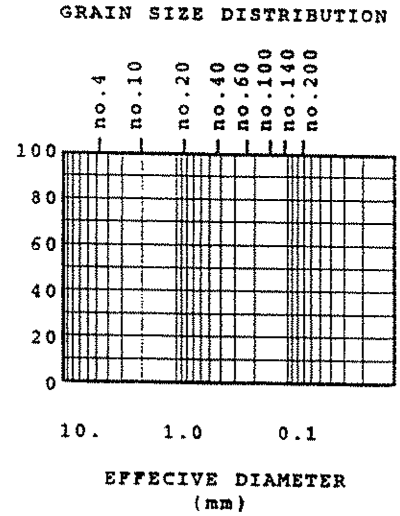
Ardaman & Associates Inc.
Consulting Engineers

GEOTECHNICAL INVESTIGATION
SOUTHEAST LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

CHECKED BY:	DATE: 3/2/94
FILE NO. 93-029	APPROVED BY: [signature]

CONSOLIDATED - UNDRAINED TRIAXIAL COMPRESSION TEST

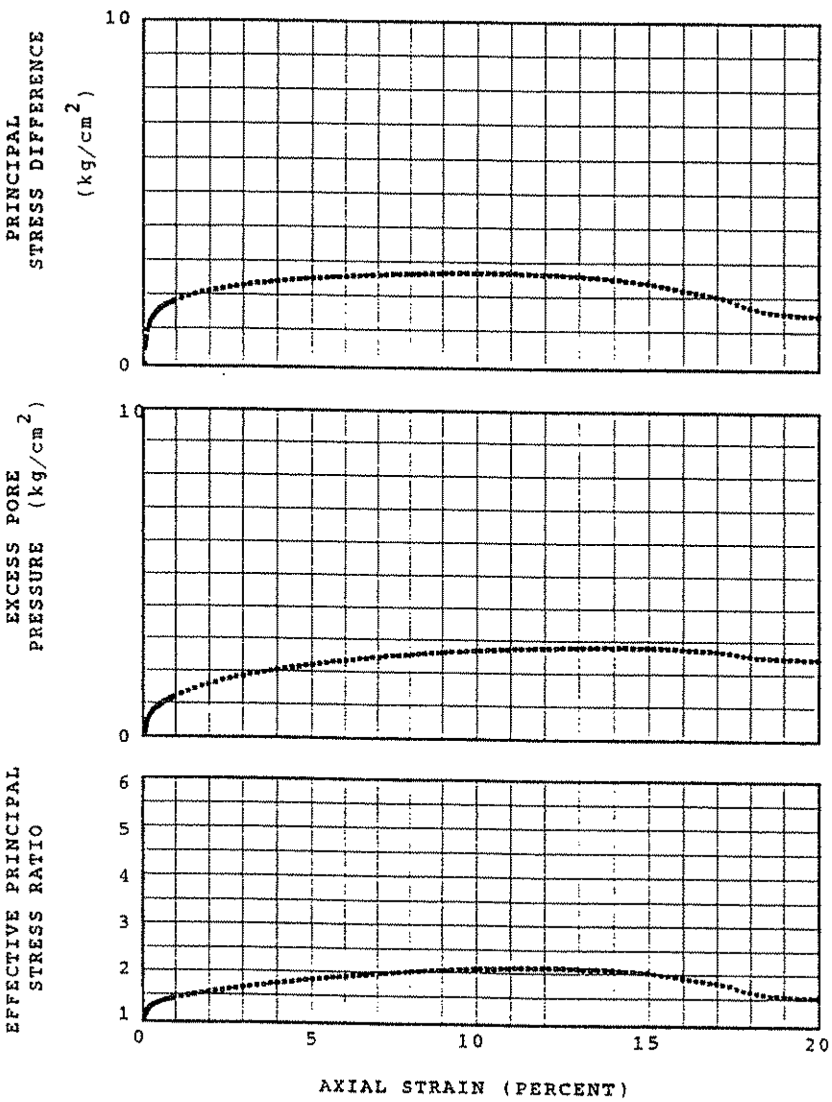




ASSUMED SPECIFIC GRAVITY 2.75

SAMPLE DESCRIPTION

Clay slime



SAMPLE ID

SITE 2 , US4,
B3,

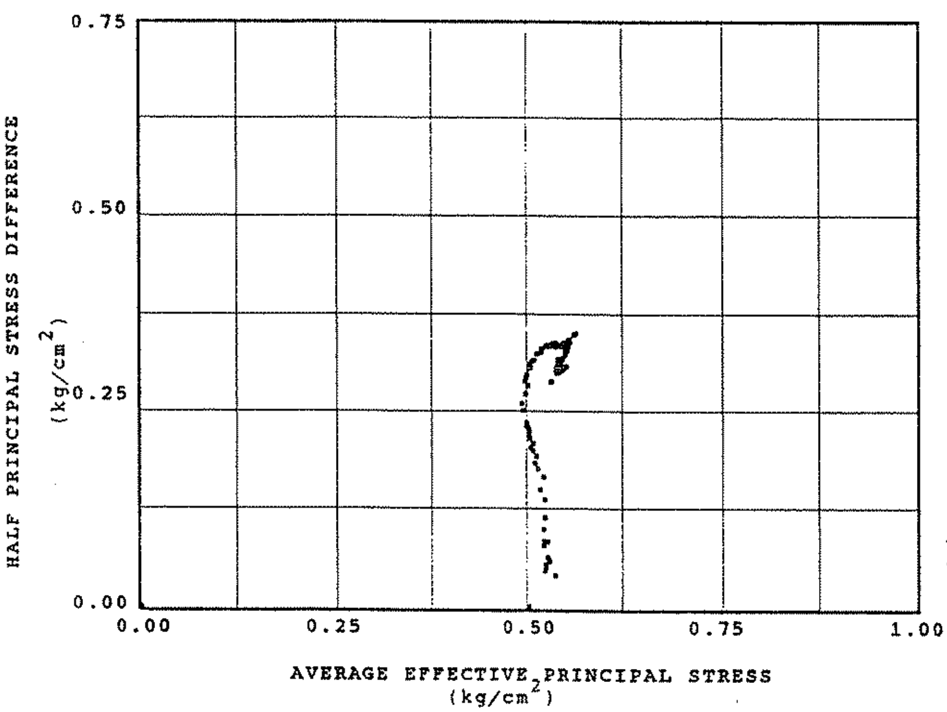
TYPE OF SPECIMEN	Undisturbed
STRAIN RATE	1.25%/hr
EFFECTIVE CONSOLIDATION PRESSURE	5.0 kg/cm^2
BACK PRESSURE	9.0 kg/cm^2
INTIAL CONDITIONS	
WATER CONTENT	120.2%
DRY DENSITY	40.1 pcf
VOID RATIO	3.279
SATURATION	100.0%
PRE-SHEAR CONDITIONS	
WATER CONTENT	77.5%
DRY DENSITY	54.8 pcf
VOID RATIO	2.131
SATURATION	100.0%
B - FACTOR	99.0%
MAXIMUM STRESS RATIO	
AXIAL STRAIN	11.06%
σ'_1	4.99 kg/cm^2
σ'_3	2.27 kg/cm^2
A - FACTOR	1.00 Normal Consolidated
MAXIMUM STRESS DIFFERENCE	
AXIAL STRAIN	9.03%
σ'_1	5.09 kg/cm^2
σ'_3	2.38 kg/cm^2
A - FACTOR	0.96

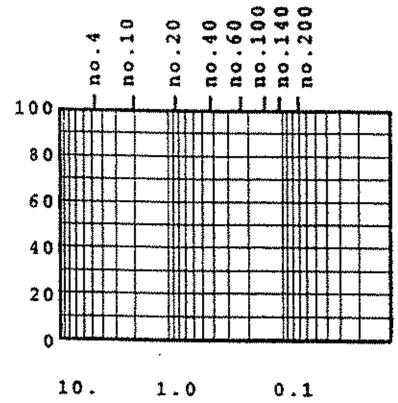
Ardaman & Associates Inc.
Consulting Engineers

GEOTECHNICAL INVESTIGATION
SOUTHEAST LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

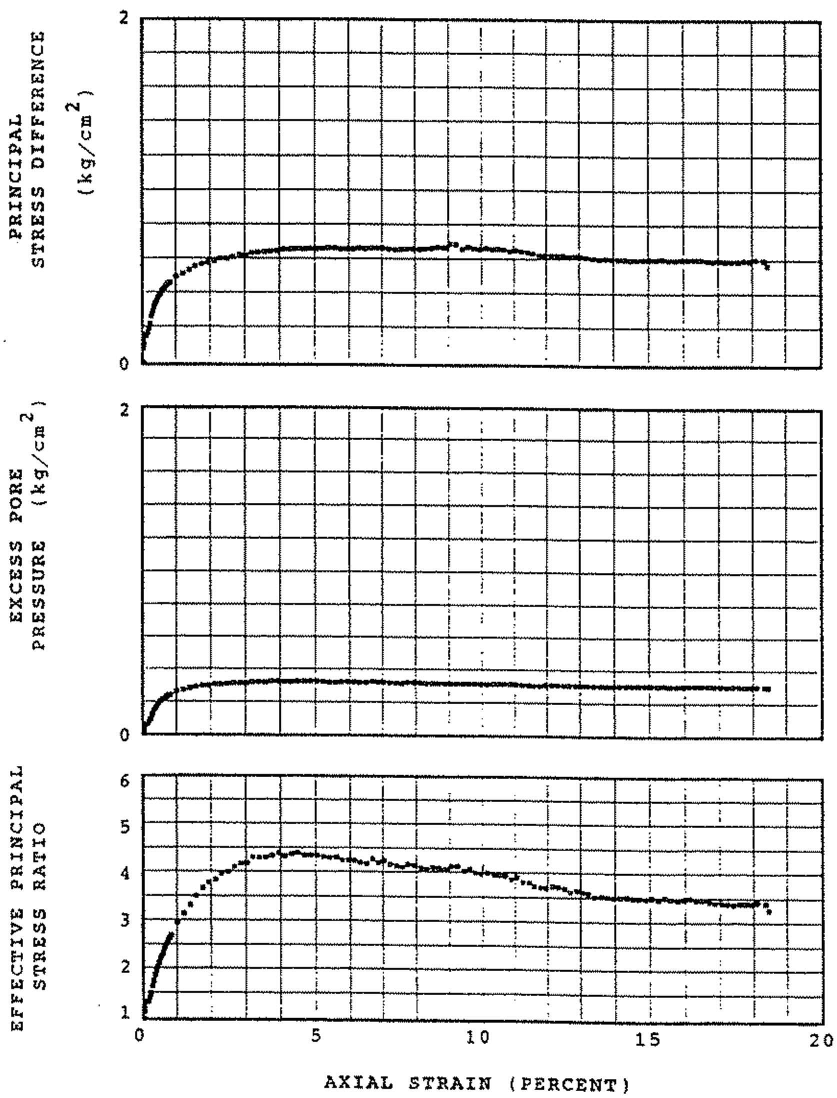
CHECKED BY:	DATE: 3/2/94
FILE NO. 93-029	APPROVED BY: [signature]

CONSOLIDATED - UNDRAINED TRIAXIAL COMPRESSION TEST





ASSUMED SPECIFIC GRAVITY 2.75
SAMPLE DESCRIPTION
Gray clay with roots



SAMPLE ID
TH-6, US1A,
.5-10.5'

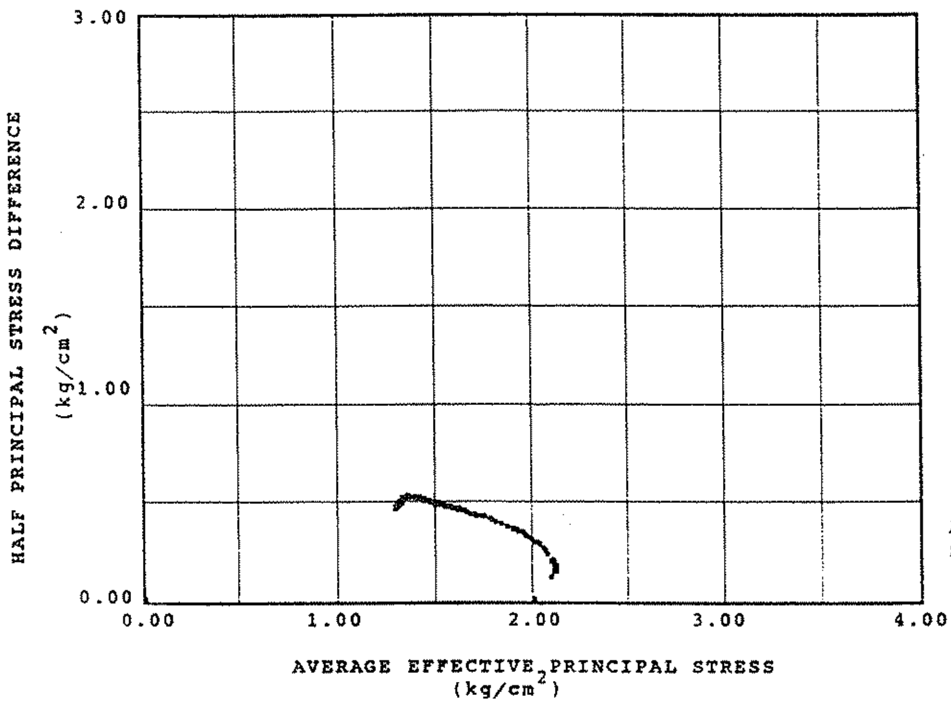
TYPE OF SPECIMEN	Undisturbed
STRAIN RATE	1.16%/hr
EFFECTIVE CONSOLIDATION PRESSURE	0.5 kg/cm^2
BACK PRESSURE	12.0 kg/cm^2
INTIAL CONDITIONS	
WATER CONTENT	85.6%
DRY DENSITY	52.6 pcf
VOID RATIO	2.262
SATURATION	100.0%
PRE-SHEAR CONDITIONS	
WATER CONTENT	83.3%
DRY DENSITY	52.1 pcf
VOID RATIO	2.294
SATURATION	99.9%
B - FACTOR	99.0%
MAXIMUM STRESS RATIO	
AXIAL STRAIN	3.88%
σ'_1	0.85 kg/cm^2
σ'_3	0.19 kg/cm^2
A - FACTOR	0.48 overconsolidat.
MAXIMUM STRESS DIFFERENCE	
AXIAL STRAIN	8.99%
σ'_1	0.91 kg/cm^2
σ'_3	0.21 kg/cm^2
A - FACTOR	0.44

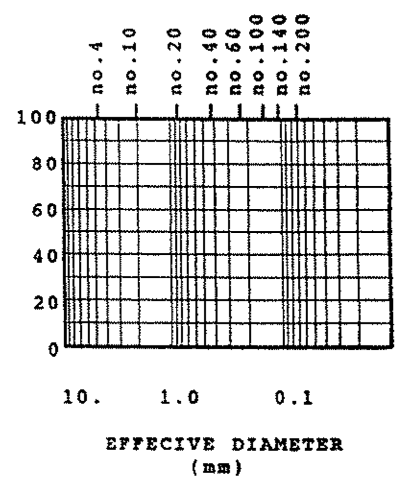
Ardaman & Associates Inc.
Consulting Engineers

GEOTECHNICAL INVESTIGATION
SOUTHEAST LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

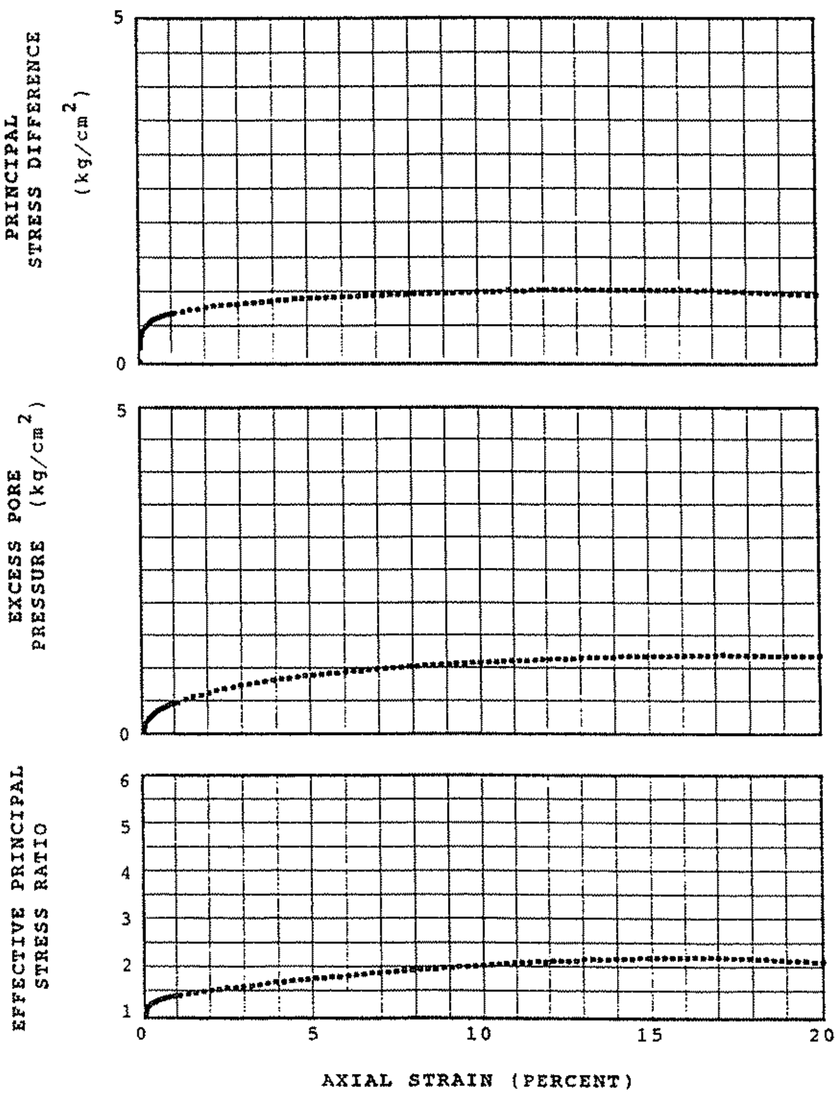
CHECKED BY:	DATE: 3/2/94
FILE NO. 93-029	APPROVED BY:

CONSOLIDATED - UNDRAINED TRIAXIAL COMPRESSION TEST





ASSUMED SPECIFIC GRAVITY 2.75
SAMPLE DESCRIPTION
Gray clay



SAMPLE ID
TH-6 US3,
13-15'

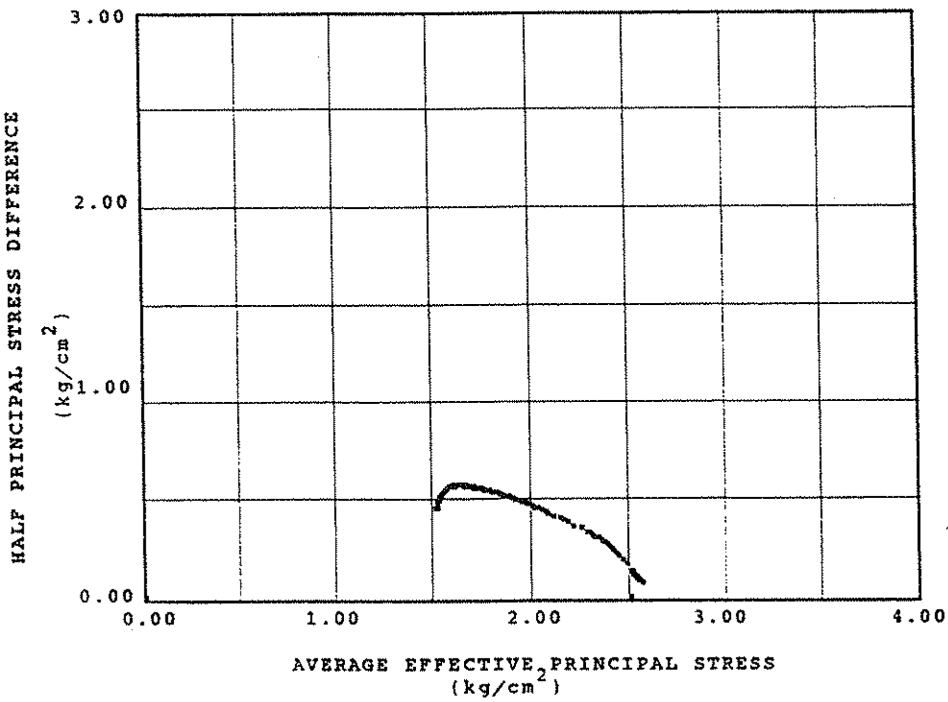
TYPE OF SPECIMEN	Undisturbed
STRAIN RATE	1.48%/hr
EFFECTIVE CONSOLIDATION PRESSURE	2.0 kg/cm² (4097.7 psf)
BACK PRESSURE	12.0 kg/cm²
INTIAL CONDITIONS	
WATER CONTENT	164.6%
DRY DENSITY	31.4 pcf
VOID RATIO	4.465
SATURATION	100.0%
PRE-SHEAR CONDITIONS	
WATER CONTENT	89.2%
DRY DENSITY	49.7 pcf
VOID RATIO	2.453
SATURATION	100.0%
B - FACTOR	99.0%
MAXIMUM STRESS RATIO	
AXIAL STRAIN	16.70%
σ'_1	1.85 kg/cm²
σ'_3	0.81 kg/cm²
A - FACTOR	1.13 Normal Consolidated
MAXIMUM STRESS DIFFERENCE	
AXIAL STRAIN	15.30%
σ'_1	1.86 kg/cm²
σ'_3	0.82 kg/cm²
A - FACTOR	1.12

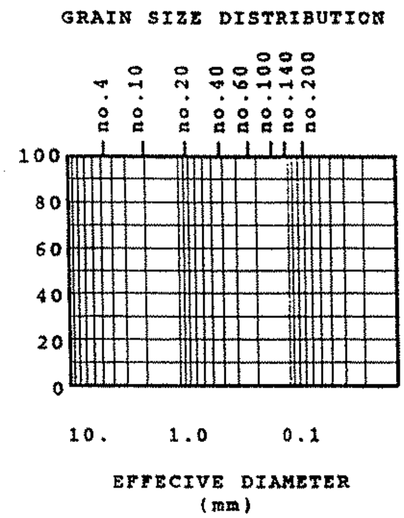
Ardaman & Associates Inc.
Consulting Engineers

GEOTECHNICAL INVESTIGATION
SOUTHEAST LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

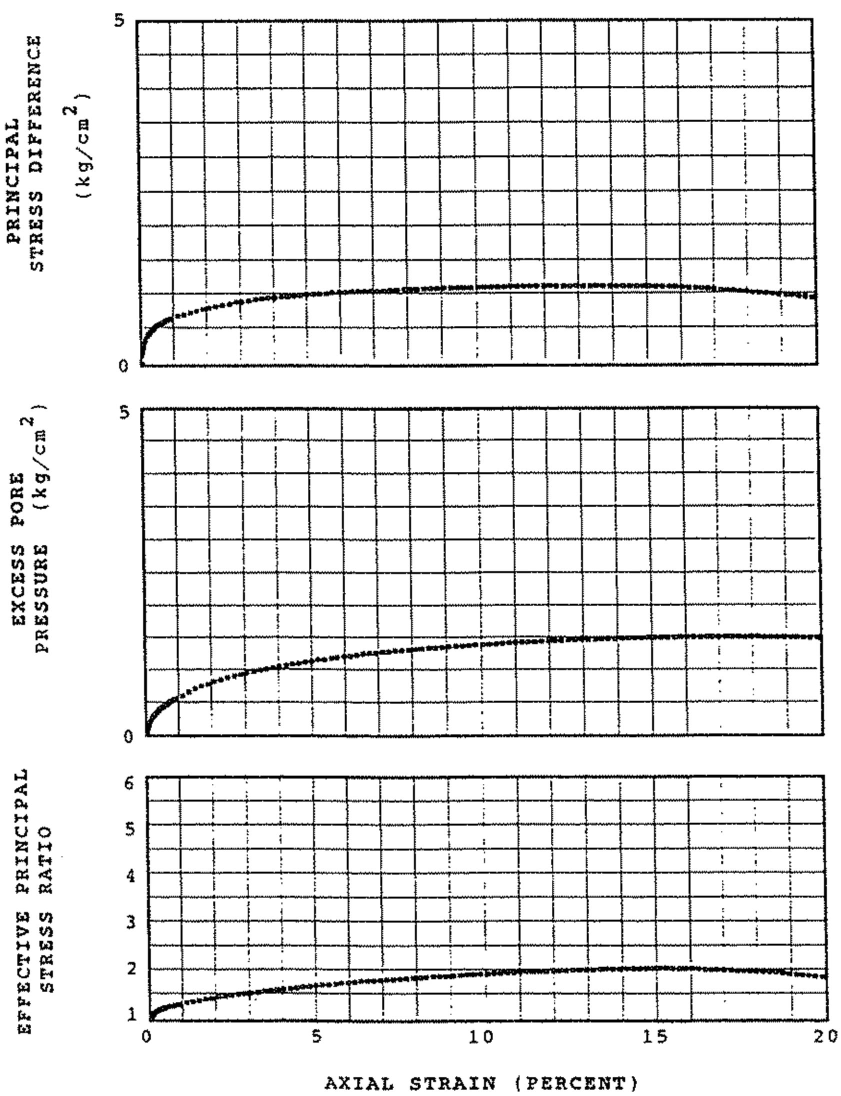
CHECKED BY:	DATE: 3/2/94
FILE NO. 93-029	APPROVED BY:

CONSOLIDATED - UNDRAINED TRIAXIAL COMPRESSION TEST





ASSUMED SPECIFIC GRAVITY 2.75
SAMPLE DESCRIPTION
Gray clay



SAMPLE ID
TH-6, US6,
19–21'

TYPE OF SPECIMEN	Undisturbed
STRAIN RATE	1.21%/hr
EFFECTIVE CONSOLIDATION PRESSURE	2.5 kg/cm^2
BACK PRESSURE	11.0 kg/cm^2
INTIAL CONDITIONS	
WATER CONTENT	127.5%
DRY DENSITY	38.5 pcf
VOID RATIO	3.457
SATURATION	100.0%
PRE-SHEAR CONDITIONS	
WATER CONTENT	81.2%
DRY DENSITY	53.1 pcf
VOID RATIO	2.232
SATURATION	100.0%
B - FACTOR	99.0%
MAXIMUM STRESS RATIO	
AXIAL STRAIN	15.21%
σ'_1	2.15 kg/cm^2
σ'_3	1.03 kg/cm^2
A - FACTOR	1.32
MAXIMUM STRESS DIFFERENCE	
AXIAL STRAIN	13.47%
σ'_1	2.18 kg/cm^2
σ'_3	1.05 kg/cm^2
A - FACTOR	1.28

Normal Consolidated

Ardaman & Associates Inc.
Consulting Engineers

GEOTECHNICAL INVESTIGATION
SOUTHEAST LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

CHECKED BY:	DATE: 3/2/94
FILE NO. 93-029	APPROVED BY: