Environmental Consultants and Contractors
4041 Park Oaks Boulevard, Suite 100
Tampa, FL 33610
813-621-0080
813-623-6757 FAX

SCS ENGINEERS

TO Florida Department of Environmental Protection
13051 North Telecom Parkway
Temple Terrace, Florida 33637-0926

DATE September 21, 2012
JOB NO. 09199033.23
ATTENTION Steve Morgan
Re: Hardee County Class I Landfill
Phase II Section II Expansion
Construction Permit Application Backup
Information Requested

WE ARE SENDING YOU

☒ Attached ☐ Under separate cover via ______
☐ Shop drawings ☐ Prints
☐ Copy of letter ☐ Change Order
☐ The following items: ☐ Plans ☐ Samples
☐ Specifications ☐ ______

COPIES	DATE	DESCRIPTION
3	9/21/12	Hardee County Class I Landfill Phase II Section II Expansion Construction Permit
		Application Backup Information Requested

THESE ARE TRANSMITTED as check below:

☐ For approval ☐ Approved as submitted ☐ Resubmit ______ Copies for approval
☒ For your use ☐ Approved as noted ☐ Submit ______ Copies distribution
☐ As requested ☐ Returned for corrections ☐ Return ______ Corrected prints
☐ For review and comment ☐ ______
☐ FOR BIDS DUE ______ 20 ______ ☐ PRINTS RETURNED AFTER LOAN TO US

REMARKS

COPY TO ______

SIGNED: Shane R. Fischer, P.E.

If enclosures are not as noted, kindly notify us at once.

Attachment 1

Geotechnical Engineering Services Report
September 25, 2003
Professional Services Industries, Inc.

GEOTECHNICAL ENGINEERING SERVICES REPORT

For the

PROPOSED LANDFILL EXPANSION
HARDEE COUNTY, FLORIDA

Prepared for

SCS Engineers
3012 US Highway 301 North
Suite 700
Tampa, FL 33619-2242

Prepared by

Professional Service Industries, Inc.
5801 Benjamin Center Drive
Suite 112
Tampa, Florida 33634
Telephone (813) 886-1075
Fax (813) 888-6514
Engineering Business No. 3684

PSI Project No. 775-35140

September 25, 2003

Information To Build On
Engineering • Consulting • Testing

Larry P. Moore, P.E.
Vice President
Florida Registration No. 47673

Robin Saunders, EI
Staff Engineer

Martin E. Millburg, P.E. 9/25/03
Geotechnical Department Manager
Florida Registration No. 36584

TABLE OF CONTENTS

1.0 PROJECT INFORMATION

1.1 PROJECT AUTHORIZATION

Authorization to proceed with this project was provided SCS Engineers in the form of Work Order Hardee-01, which was executed 5/1/2003. This study was conducted in accordance with the Scope of Work outlined in the PSI proposal for these services dated April 17, 2003, PSI Proposal No. 775-3G0159.

1.2 PROJECT DESCRIPTION

The existing Hardee County landfill is planned to be expanded. Geotechnical data is required to design the planned expansion. A geotechnical study with soil borings and laboratory testing has been performed to provide data to assist with the design of the planned landfill expansion.

If this information is incorrect, PSI should be notified to determine if either changes in the recommendations are required or additional deeper borings may be necessary.

1.3 PURPOSE AND SCOPE OF WORK

The following services have been provided in order to provide the requested geotechnical data:

1. Executed an program of subsurface exploration consisting of subsurface sampling and field testing. PSI performed seven (7) Standard Penetration Test (SPT) borings. One of these borings was extended to a depth of 70 feet below the ground surface. Four borings were advanced to a depth of 45 feet, and two borings were advanced to a depth of 35 feet. In each boring, samples were collected and Standard Penetration Test resistances have been measured virtually continuously for the top 10 feet and on intervals of 5 feet thereafter.

2. After the performance of the soil borings, five soil borings were performed by drilling without sampling to various depths. At those various depths, thin-walled (Shelby) tube samples were obtained. A list of the samples obtained is presented in the table below:

Boring No.	Depth, feet	Sample Name
TH-1	18.0-20.0	US1
TH-1	23.5-25.0	US2
TH-4	23.0-24.0	US3
TH-4	23.0-24.0	US4
TH-5	13.0-15.0	US5
TH-6	18.0-20.0	US6
TH-7	13.0-15.0	US7

3. Visually classified representative soil samples in the laboratory using the Unified Soil Classification System (USCS). In addition to the visual classifications, an extensive laboratory testing was performed to help define the characteristics of the subsurface materials at this site.

4. Collected groundwater level measurements and estimated normal wet seasonal high groundwater tables.

5. The results of the exploration were used in the engineering analysis and the formulation of recommendations. The results of the subsurface exploration, including the recommendations and the data on which they are based, are presented in this written report prepared by a professional engineer.

The scope of our services did not include an environmental assessment for determining the presence or absence of wetlands or hazardous or toxic materials in the soil, bedrock, groundwater, or air, on or below or around this site. Any statements in this report or on the boring logs regarding odors, colors, unusual or suspicious items or conditions are strictly for the information of our client. It is our understanding that an environmental site assessment is currently being performed at this site.

2.0 SITE AND SUBSURFACE CONDITIONS

2.1 SITE LOCATION AND DESCRIPTION

The site is located at the existing Hardee County landfill located on Airport Road, approximately 2 miles east and 2 miles north of Wauchula, Florida.

2.2 SUBSURFACE CONDITIONS

The subsurface conditions were explored using seven (7) Standard Penetration Test (SPT) borings drilled depths ranging from 35 to 70 feet below the existing ground surface. The borings were located in the field by SCS personnel who directed the location and depth of each soil boring. The approximate boring locations and soil profiles are presented on Sheet 1 in the Appendix of this report.

SPT soil borings were advanced utilizing rotary mud drilling methods and soil samples were routinely obtained at select intervals during the drilling process. Drilling and sampling techniques were accomplished in general accordance with ASTM standards. Select soil samples were returned to our laboratory for visual classification and laboratory testing. After the performance of the SPT borings, soil borings were advanced without sampling until desired depths were attained. Then, 3 inch diameter thin-wall (Shelby) tube samples were obtained at depths ranging from 13 to 25 feet.

A generalized description of the subsurface stratigraphy at this site is presented in the table below:

Depth, Feet	Description	Range of N-values
0-18	Sand, slightly silty fine sand, and silty sand (SP/SP-SM, SM)	5 - >50
18-70	Clayey sand, silty clay, sandy clay (SC, CL/CH)	5 - >50

Exceptions to this general pattern occurred at boring TH-3, where highly weathered limestone was encountered from 18 to 23 feet. Also, at TH-2, a clayey sand (SC) was found from 8 to 13 feet.

The previous descriptions are of a generalized nature to highlight the major subsurface stratification features and material characteristics. The soil profiles included on Sheet 2 should be reviewed for specific information at individual boring locations. These profiles include soil descriptions, groundwater levels, stratification, and penetration resistance. The stratifications shown on the boring profiles represent the conditions only at the actual boring locations. The stratifications represent the approximate boundary between subsurface materials and the actual transition may be gradual.

2.3 LABORATORY TESTING PROGRAM

Laboratory testing was performed as directed by SCS. Laboratory testing included moisture content, Atterberg limits, sieve analyses, tri-axial strength, permeability, standard Proctor and consolidation testing.

A summary of laboratory test results is presented in the Appendix of this report. Detailed laboratory reports are also presented in the Appendix of this report.

2.4 GROUNDWATER INFORMATION

Groundwater levels were recorded immediately after drilling, during the time of the subsurface exploration and corroborated through a visual examination of the obtained soil samples. Groundwater was found at a depth of 5 ½ to 7 feet below the current ground surface.

It should be noted that groundwater levels tend to fluctuate during periods of prolonged drought and extended rainfall and may be affected by man-made influences. A seasonal effect will occur in which higher groundwater levels are normally recorded in rainy seasons. Groundwater levels presented in this report are the levels that were measured at the time of our field activities. Based on the upper limit of the iron oxidation and the observed groundwater levels, the seasonal high depth to groundwater at this site is estimated to be on the order of 3 ½ feet below the existing ground surface at the boring locations.

3.0 REPORT LIMITATIONS

The Geotechnical Engineer warrants that the findings, recommendations, specifications, or professional advice contained herein have been made in accordance with generally accepted professional geotechnical engineering practices in the local area. No other warranties are implied or expressed.

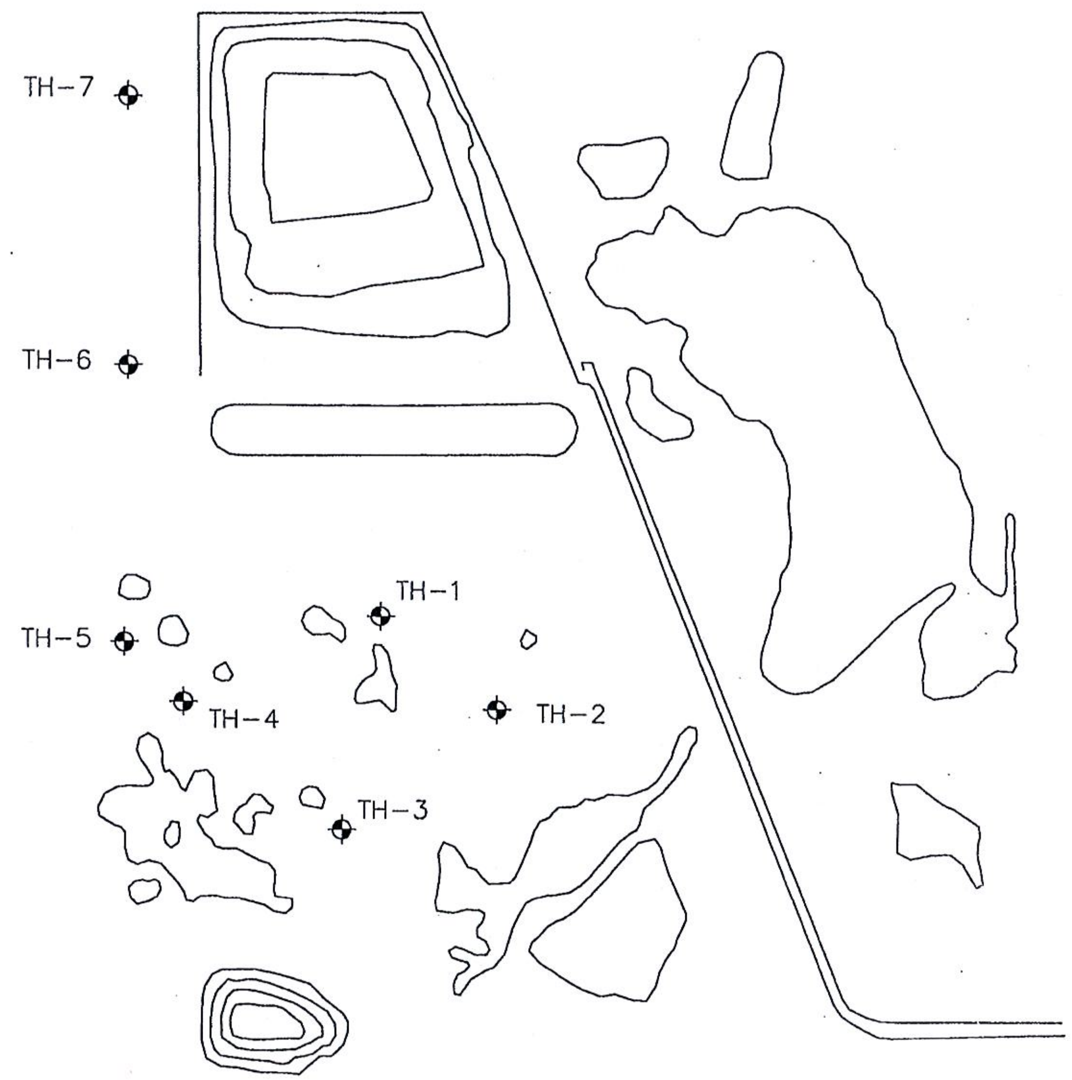
The recommendations submitted are based on the available subsurface information obtained by PSI and design details furnished by SCS Engineers for the proposed project. If there are any revisions to the plans for this project or if deviations from the subsurface conditions noted in this report are encountered during construction, PSI should be notified immediately to determine if changes in the foundation recommendations are required. The State of Florida is underlain by a soluble limestone formation. This limestone can be dissolved, resulting in the formation of sinkholes. An evaluation for the existence of or the potential for sinkhole development was not a part of the scope of services for this project.

After the plans and specifications are more complete, the Geotechnical Engineer should be retained and provided the opportunity to review the final design plans and specifications to check that our engineering recommendations have been properly incorporated into the design documents. At that time, it may be necessary to submit supplementary recommendations. This report has been prepared for the exclusive use of SCS Engineers in Hardee County, Florida.

SHEETS

BOIRNG LOCATION PLAN

SOIL PROFILES



LEGEND

⌖ Approximate SPT boring location

FLORIDA DEPARTMENT OF ENVIRONMENTAL PROTECTION
SEP 21 2012
SOUTHWEST DISTRICT TAMPA

BORING LOCATION PLAN

0 365'

APPROXIMATE PLAN SCALE

NOTE: PROVIDED TO PSI BY SCS ON 7/25/03.

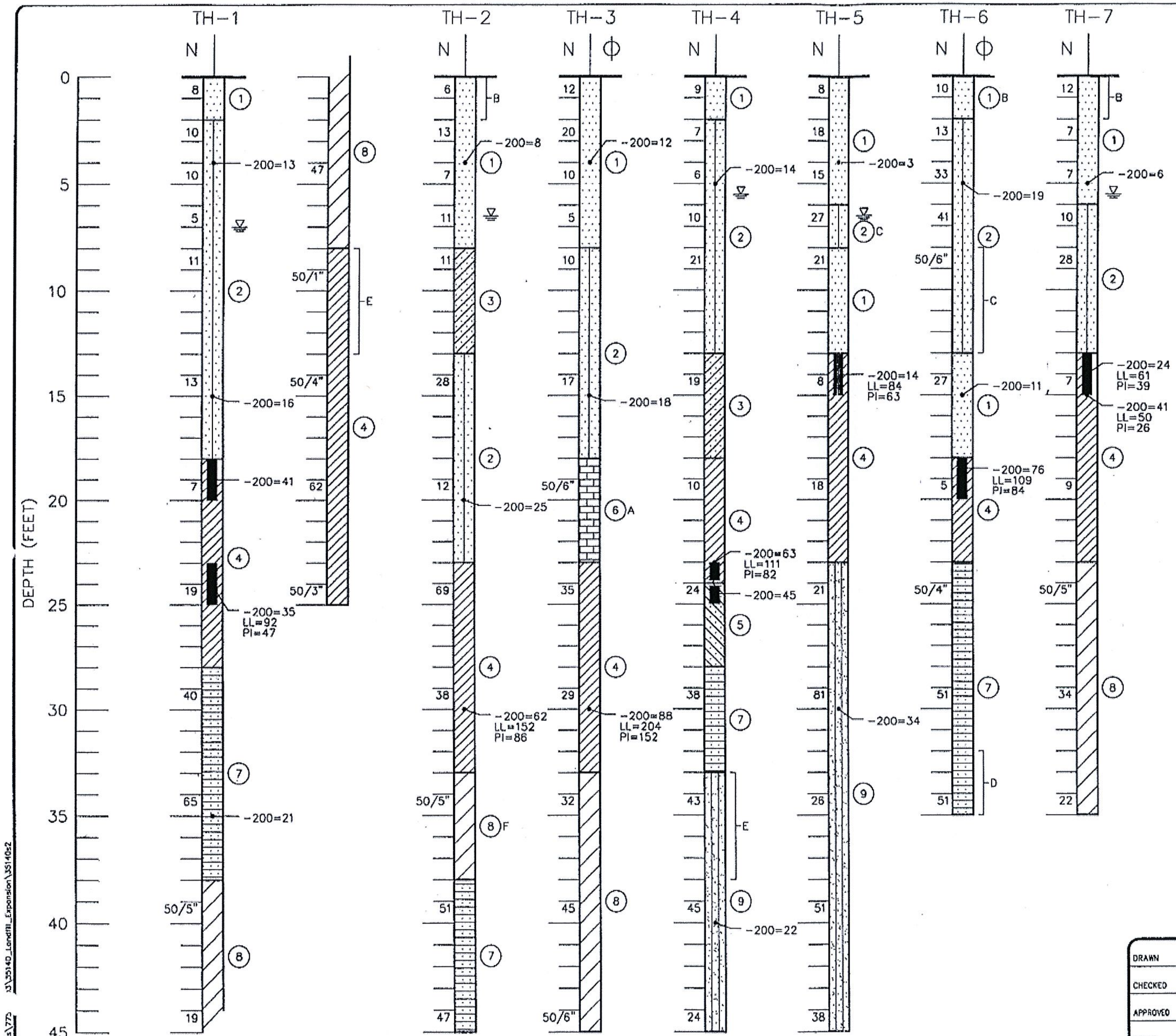
DRAWN	DJG
CHECKED	GC
APPROVED	MEM
SCALE	NOTED

GEOTECHNICAL SERVICES
PROPOSED LANDFILL EXPANSION
HARDEE COUNTY, FLORIDA

psi Information To Build On
Engineering • Consulting • Testing

DATE	PROJ. NO.	
JUNE 03	775-35140	SHEET 1

s:\geo\projects\77...\35140_Landfill_Expansion\35140s1



0 5'
VERTICAL SCALE

SOIL PROFILES

LEGEND

1. Gray, brown SAND to slightly silty SAND (SP/SP-SM)
2. Gray, brown, yellow silty SAND (SP-SM/SM)
3. Gray, brown, yellow clayey SAND (SC)
4. Gray, brown, yellow silty CLAY (CL/CH)
5. Gray silty to clayey SAND (SM/SC)
6. Highly weathered LIMESTONE (LS)
7. Gray to tan phosphatic clayey SAND (SC)
8. Gray to tan phosphatic sandy CLAY to CLAY (CL/CH)
9. Gray to tan phosphatic silty to clayey SAND (SM/SC)

SP Unified Soil Classification System (ASTM D 2487) group symbol as determined by visual review

Groundwater level, April 2003

Φ Groundwater level not measured

N SPT N-value in blows/foot

50/6" Fifty blows for six inches

Shelby tube sample location

A With phosphate fragments

B With grass roots

C With trace organic soil

D With trace cemented sand

E With chert fragments

F With limestone fragments

-200 Fines passing No. 200 sieve (%)

LL Liquid Limit (%)

PI Plasticity Index (%)

DRAWN	DJG
CHECKED	GC
APPROVED	MEM
SCALE	NOTED

GEOTECHNICAL SERVICES
PROPOSED LANDFILL EXPANSION
HARDEE COUNTY, FLORIDA

psi Information To Build On
Engineering • Consulting • Testing

DATE	PROJ. NO.	
JUNE 03	775-35140	SHEET 2

TABLES

TABLE 1
SUMMARY OF LABORATORY TEST RESULTS
PROPOSED LANDFILL EXPANSION
HARDEE COUNTY, FLORIDA
PSI PROJECT NO. 775-35140

Boring Number	Sample Depth (feet)	Sieve Analysis (%)					Atterberg Limits (%)		USGS Group	Stratum Number
		#10	#40	#60	#100	#200	LL	PI		
TH-1	4.0 - 6.0	98	93	75	48	13	-	-	SP-SM/SM	2
TH-1	13.5 - 15.0	99	71	36	22	17	-	-	SP-SM/SM	2
TH-1	18.0 - 20.0	-	100	-	85	41	-	-	CL/CH	4
TH-1	23.0 - 25.0	85	74	67	58	36	92	47	CL/CH	4
TH-1	33.5 - 35.0	99	89	64	24	21	-	-	SC	7
TH-2	2.0 - 4.0	100	95	76	50	8	-	-	SP/SP-SM	1
TH-2	18.5 - 20.0	100	81	41	27	25	-	-	SP-SM/SM	2
TH-2	28.5 - 30.0	100	100	98	87	61	152	86	CL/CH	4
TH-3	2.0 - 4.0	100	95	75	50	12	-	-	SP/SP-SM	1
TH-3	13.5 - 15.0	100	88	47	22	18	-	-	SP-SM/SM	2
TH-3	28.5 - 30.0	100	99	97	95	87	204	152	CL/CH	4
TH-4	4.0 - 6.0	100	93	71	48	17	-	-	SP-SM/SM	2
TH-4	23.0 - 24.0	100	100	-	95	63	111	82	CL/CH	4
TH-4	24.0 - 25.0	-	-	-	-	45	-	-	SM/SC	5
TH-4	38.5 - 40.0	99	86	53	25	23	-	-	SM/SC	9
TH-5	2.0 - 4.0	100	93	70	43	8	-	-	SP/SP-SM	1
TH-5	13.0 - 15.0	92	50	-	27	14	84	63	CL/CH	4
TH-5	28.5 - 30.0	99	81	49	36	34	-	-	SM/SC	9
TH-6	4.0 - 6.0	100	95	78	56	23	-	-	SP/SP-SM	2
TH-6	13.5 - 15.0	100	72	31	16	12	-	-	SP/SP-SM	1
TH-6	18.0 - 20.0	100	98	-	82	76	109	84	CL/CH	4
TH-7	4.0 - 6.0	99	95	77	52	10	-	-	SP/SP-SM	1
TH-7	13.0 - 14.0	100	96	-	32	24	61	39	CL/CH	4
TH-7	14.0 - 15.0	-	-	-	-	41	50	26	CL/CH	4

TABLE 2
SUMMARYOF CONSOLIDATION PARAMETER TEST RESULTS
PROPOSED LANDFILL EXPANSION
HARDEE COUNTY, FLORIDA
PSI PROJECT NO. 775-35140

Boring Number	Sample Number	Sample Depth (feet)	Initial Moisture Content (%)	Dry Density (pcf)	Initial Void Ratio (%)	Initial Liquid Limit (%)	Initial Plasticity Index (%)	Preconsolidation Pressure (P_c) (psf)	Conpression Index (C_c)
TH-7	US-7	13.0 – 15.0	49	74	1.28	61	39	1250	0.44

TABLE 4
SUMMARY OF PERMEABILITY TEST TABLE
PROPOSED LANDFILL EXPANSION
HARDEE, FLORIDA
PSI PROJECT NO. 775-35140

Boring Number	Sample Number	Sample Depth (ft)	Saturation (%)	Average Hydraulic Conductivity(ft/day)	USGS Group	Stratum Number	Confining Pressure, psi
TH-1	Bulk	0.0-4.0	100	2.1	SP-SM/SM	2	*
TH-7	US-7	13.0 - 14.0	98	0.039	CL/CH	4	70
TH-7	US-7	14.0 - 15.0	98	0.0003	CL/CH	4	45

TH-1 Bulk sample remolded to 95% of 112.1 pcf.

*Permeability test performed on remolded sample placed in permeameter to attain required unit weight, no confining pressure imposed.

TABLE 3
SUMMARY OF STANDARD PROCTOR COMPACTION TEST RESULTS
PROPOSED LANDFILL EXPANSION
HARDEE, FLORIDA
PSI PROJECT NO. 775-35140

Boring Number	Sample Depth (ft)	Maximum Dry Density (pcf)	Optimum Moisture Content (%)	USGS Group	Stratum Number
TH-1	0.0 - 4.0	112.1	10.5	SP-SM/SM	2

TABLE 5
SUMMARY OF TRIAXIAL COMPRESSION TEST (CONSOLIDATED-UNDRAINED) RESULTS
PROPOSED LANDFILL EXPANSION
HARDEE COUNTY, FLORIDA
PSI PROJECT NO. 775-35140

Boring Number	Sample Number	Sample Depth (feet)	Effective Stress		USGS Group	Stratum Number
			Cohesion (psf)	Angle of Friction		
TH-1	US-1	18.0-20.0	0	13	CL/CH	4
TH-4	US-4	23.0-24.0	300	9	CL/CH	4
TH-5	US-5	13.0-15.0	0	28	CL/CH	4
TH-6	US-6	18.0-20.0	0	30	CL/CH	4

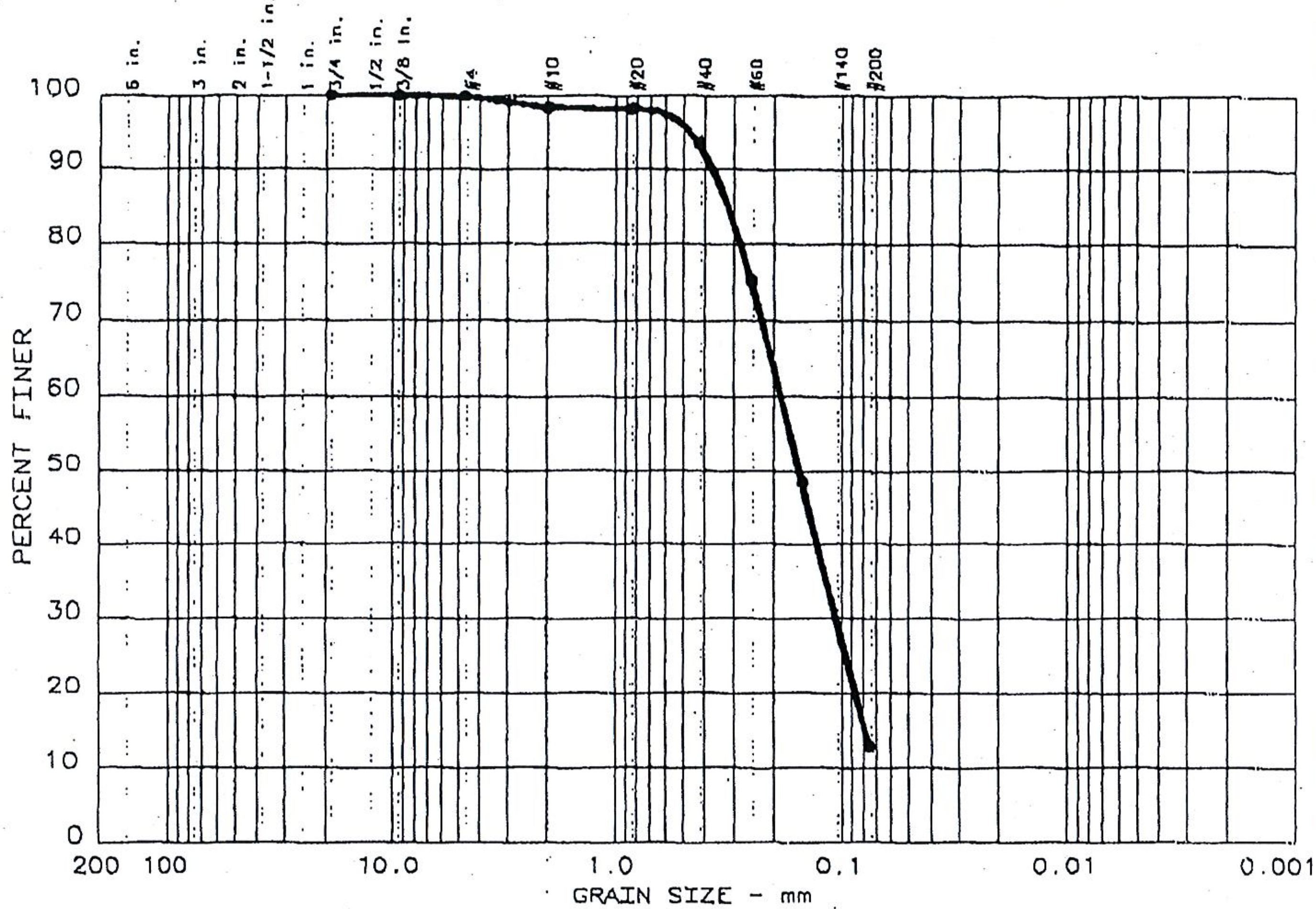
Boring Number	Sample Number	Sample Depth (feet)	Total Stress		USGS Group	Stratum Number
			Cohesion (psf)	Angle of Friction		
TH-1	US-1	18.0-20.0	500	7	CL/CH	4
TH-4	US-4	23.0-24.0	300	5	CL/CH	4
TH-5	US-5	13.0-15.0	0	16	CL/CH	4
TH-6	US-6	18.0-20.0	500	13	CL/CH	4

APPENDIX I

GRAIN SIZE ANALYSES

2/2

GRAIN SIZE DISTRIBUTION TEST REPORT



	Test	% +3"	% GRAVEL	% SAND	% SILT	% CLAY
●	2	0.0	1.7	85.3	13.0	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●			0.316	0.186	0.154	0.105	0.0781			

MATERIAL DESCRIPTION	USCS	AASHTO
● LIGHT BROWN SLIGHTLY SILTY FINE SAND	SM	A-2-4(0.2)

Project No.: 761
Project: HARDEE COUNTY LANDFILL
● Location: DELIVERED BT TAMPA LAB
TH1 4' below Surface
Date: 5-19-2003

Remarks:

GRAIN SIZE DISTRIBUTION TEST REPORT

GRAIN SIZE DISTRIBUTION TEST REPORT

PROFESSIONAL SERVICE INDUSTRIES, INC.

Project No. 35140 Date: 5/20/2003

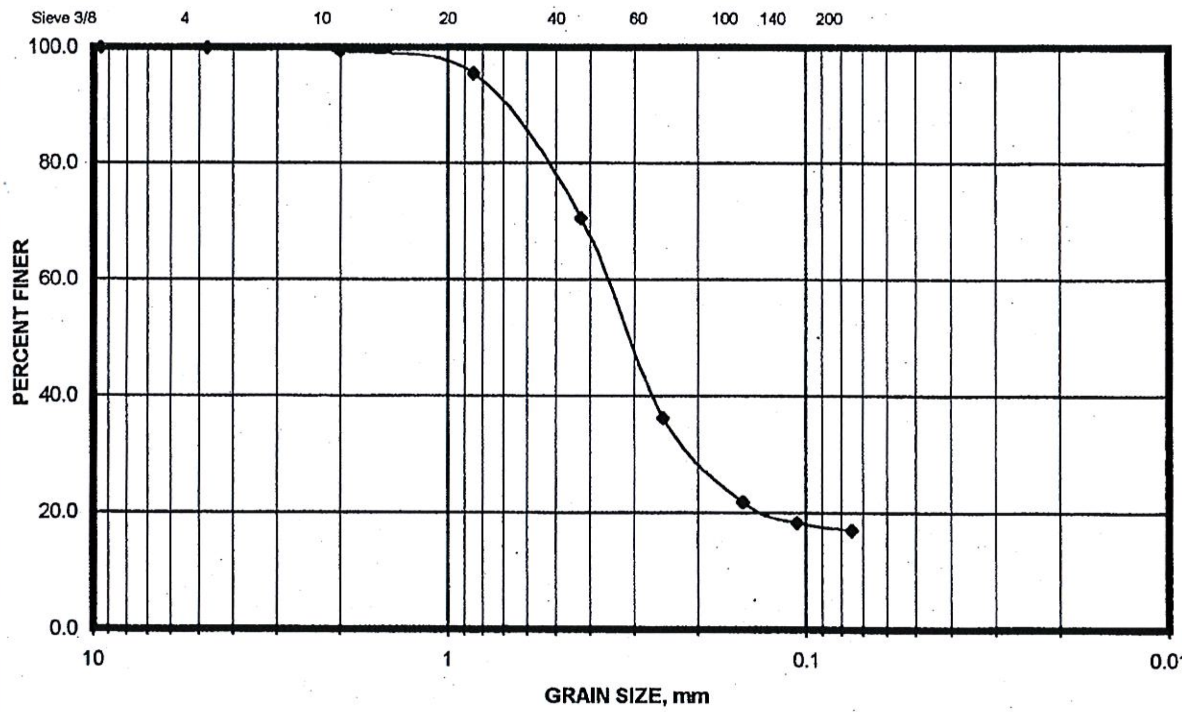
Project: Hardee County Landfill

Sample Location: TH1 15'

Soil Description: 0

Soil Classification: 0 LL ______ PI ______

GRAIN SIZE DISTRIBUTION



% Gravel	% Sand	%-200
0.0	83.0	17.0

D60	D30	D10	CC	CU

GRAIN SIZE DISTRIBUTION TEST REPORT

PROFESSIONAL SERVICE INDUSTRIES, INC.

Project No. 35140 Date: 6/11/03

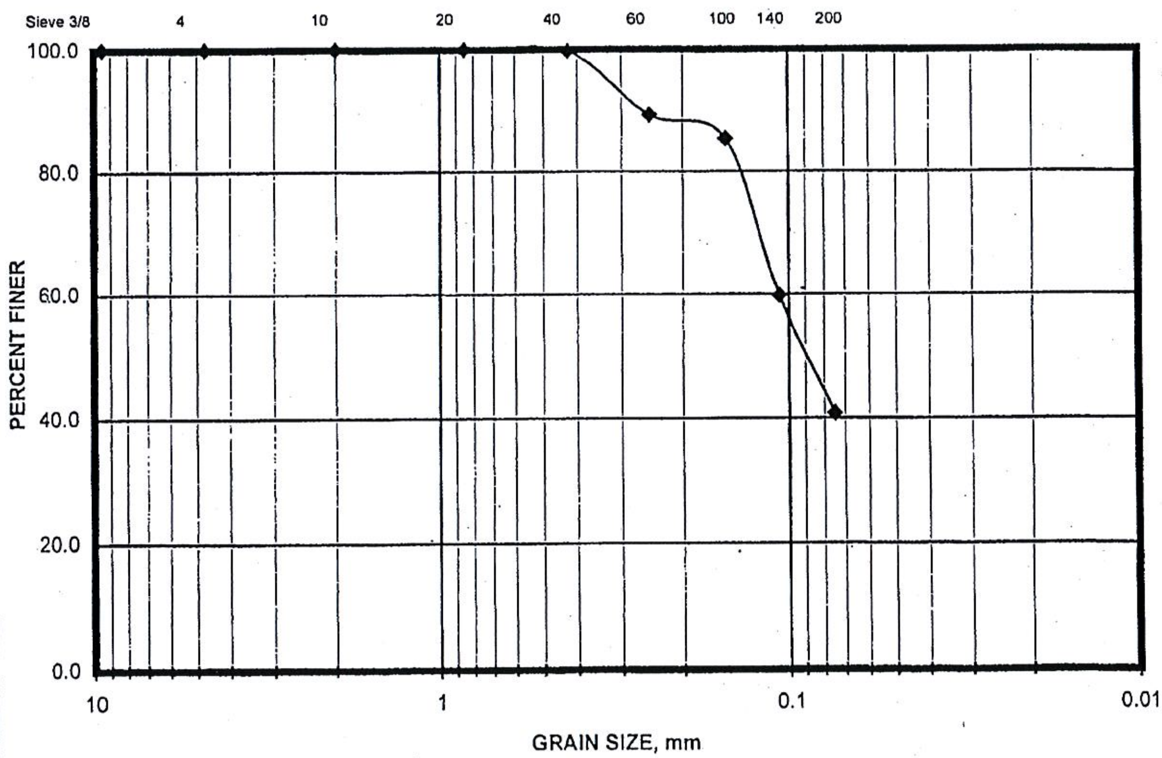
Project: Hardee County Landfill

Sample Location: TH1 18-20'

Soil Description: 0

Soil Classification: 0 LL PI

GRAIN SIZE DISTRIBUTION



% Gravel	% Sand	%-200
0.0	59.3	40.7

D60	D30	D10	CC	CU

GRAIN SIZE DISTRIBUTION TEST REPORT

PROFESSIONAL SERVICE INDUSTRIES, INC.

Project No. 775-35140 Date: 6/10/03

Project: Hardee County Landfill

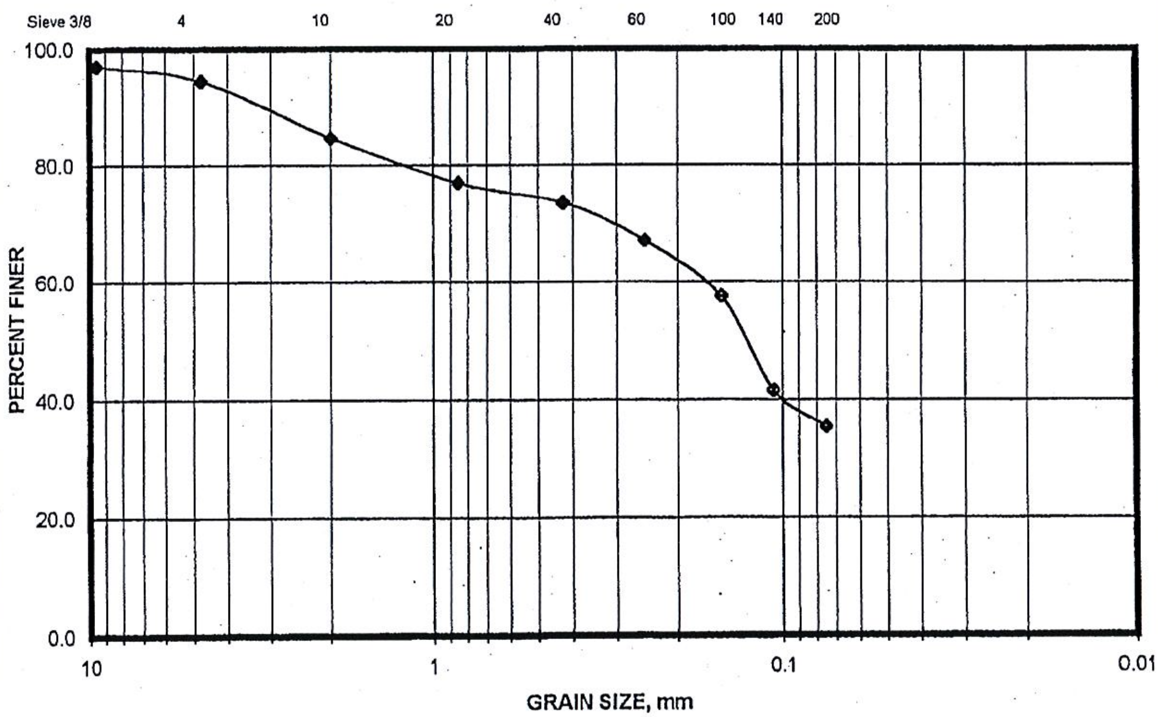
Sample Location: US-2 TH-1 @ 23.5'-25'

Soil Description: Tan and Light Green Clay With Rock

Soil Classification: 0 LL 92 PI 47

NMC % 47.4

GRAIN SIZE DISTRIBUTION



% Gravel	% Sand	%-200
5.5	59.0	35.5

D60	D30	D10	CC	CU

GRAIN SIZE DISTRIBUTION TEST REPORT

PROFESSIONAL SERVICE INDUSTRIES, INC.

Project No. 35140 Date: 5/23/2003

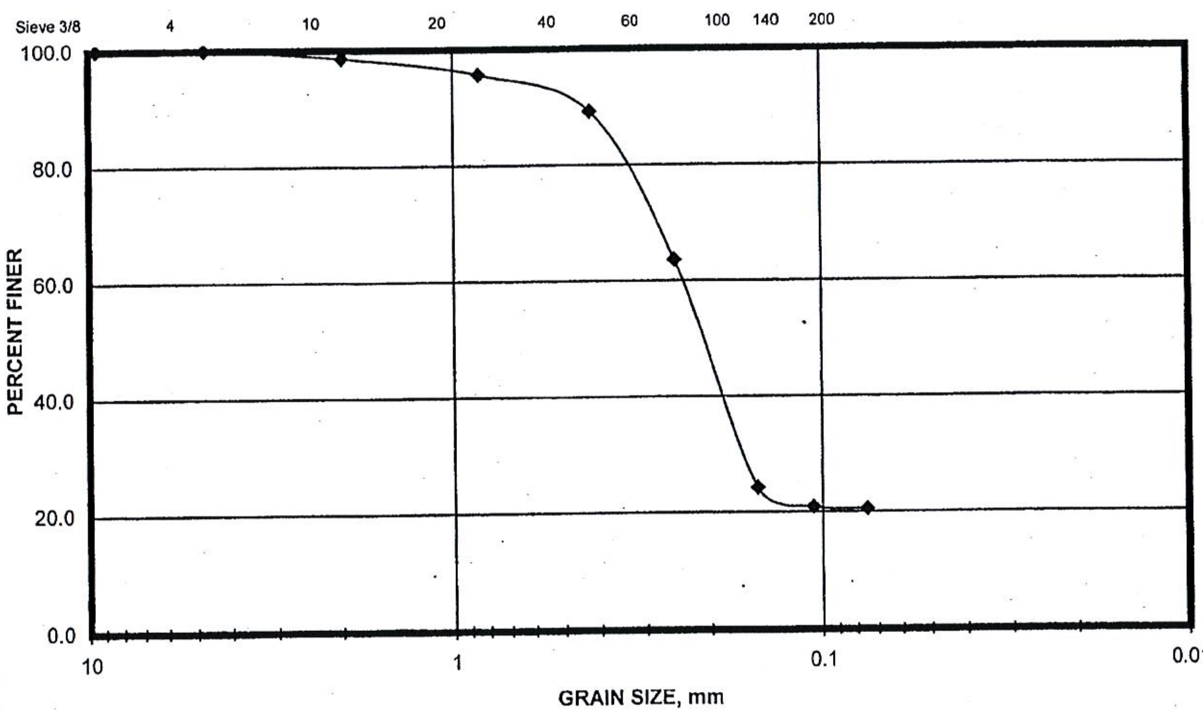
Project: Hardee County Landfill

Sample Location: TH1 35'

Soil Description: 0

Soil Classification: 0 LL ______ PI ______

GRAIN SIZE DISTRIBUTION



% Gravel		% Sand		%-200
0.0		79.5		20.5
D60	D30	D10	CC	CU

GRAIN SIZE DISTRIBUTION TEST REPORT

PROFESSIONAL SERVICE INDUSTRIES, INC.

Project No. 35140 Date: 5/23/2003

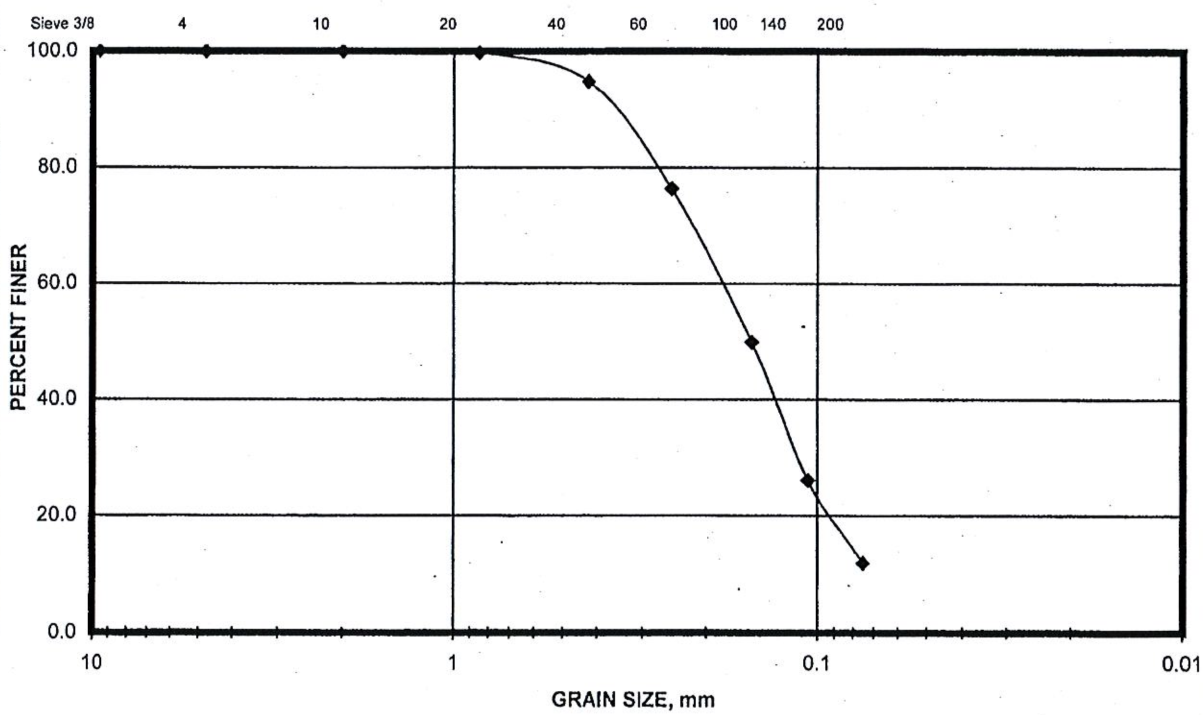
Project: Hardee County Landfill

Sample Location: TH2 4'

Soil Description: 0

Soil Classification: 0 LL ______ PI ______

GRAIN SIZE DISTRIBUTION



% Gravel	% Sand	%-200
0.0	88.1	11.9

D60	D30	D10	CC	CU

GRAIN SIZE DISTRIBUTION TEST REPORT

PROFESSIONAL SERVICE INDUSTRIES, INC.

Project No. 35140 Date: 5/23/2003

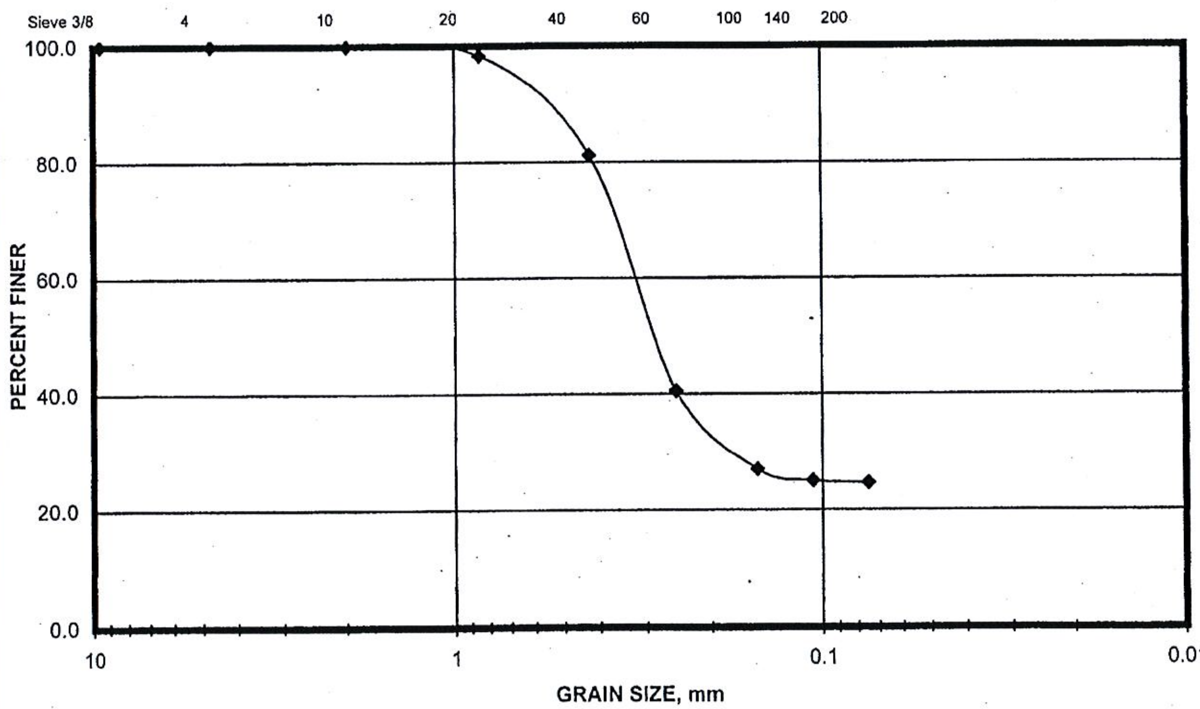
Project: Hardee County Landfill

Sample Location: TH2 20'

Soil Description: 0

Soil Classification: 0 LL ____ PI ____

GRAIN SIZE DISTRIBUTION



% Gravel		% Sand		%-200
0.0		75.3		24.7
D60	D30	D10	CC	CU

GRAIN SIZE DISTRIBUTION TEST REPORT

PROFESSIONAL SERVICE INDUSTRIES, INC.

Project No. 35140 Date: 5/21/2003

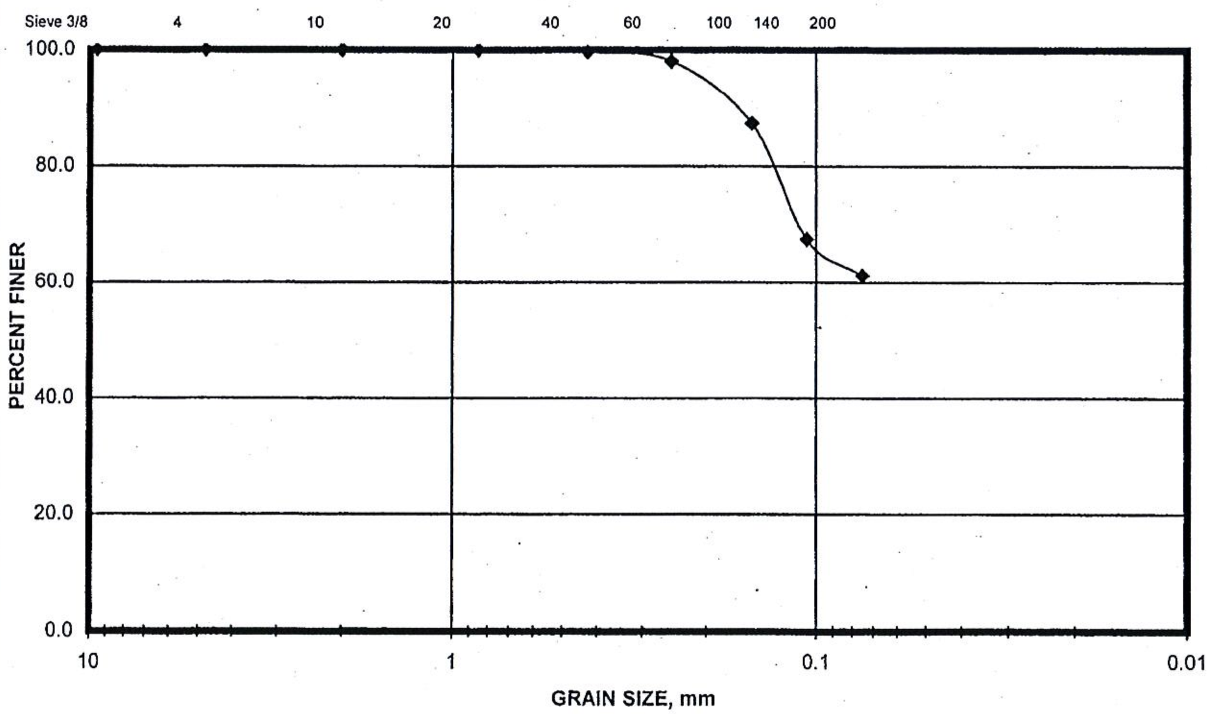
Project: Hardee County Landfill

Sample Location: TH2 30'

Soil Description: 0

Soil Classification: 0 LL ______ PI ______

GRAIN SIZE DISTRIBUTION



% Gravel	% Sand	%-200
0.0	38.8	61.2

D60	D30	D10	CC	CU

GRAIN SIZE DISTRIBUTION TEST REPORT

PROFESSIONAL SERVICE INDUSTRIES, INC.

Project No. 35140 Date: 5/22/2003

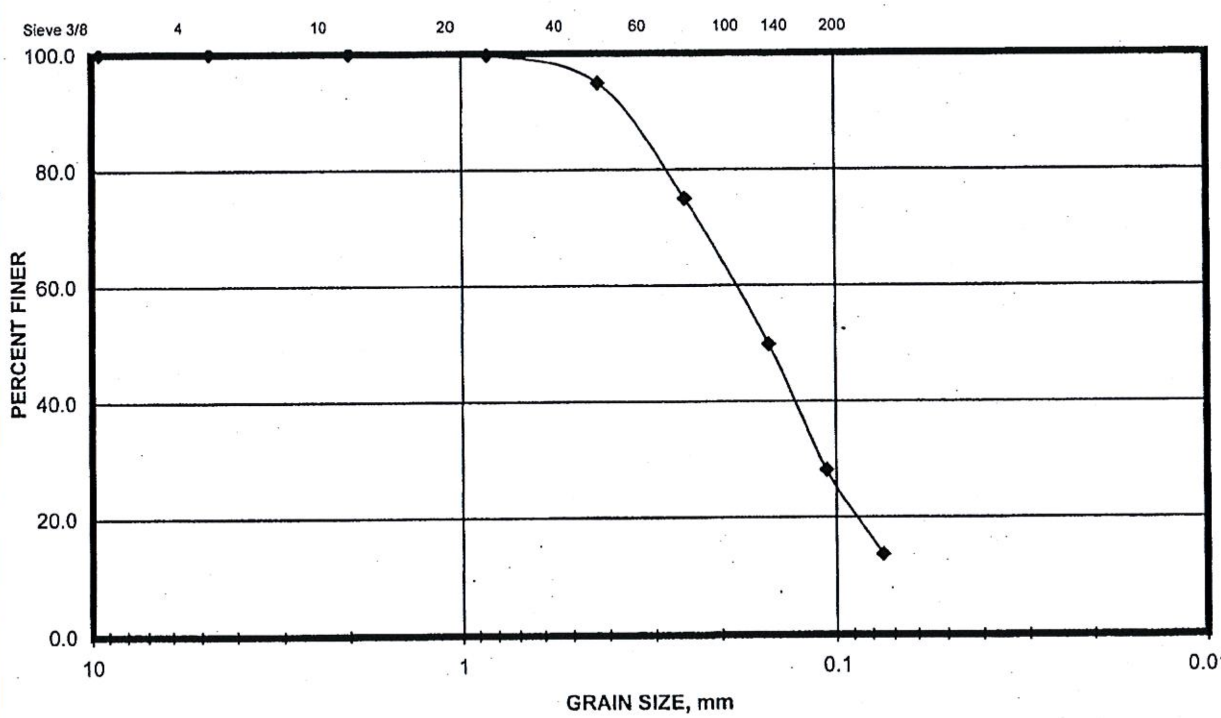
Project: Hardee County Landfill

Sample Location: TH3 4'

Soil Description: 0

Soil Classification: 0 LL ______ PI ______

GRAIN SIZE DISTRIBUTION



% Gravel	% Sand	%-200
0.0	86.3	13.7

D60	D30	D10	CC	CU

GRAIN SIZE DISTRIBUTION TEST REPORT

PROFESSIONAL SERVICE INDUSTRIES, INC.

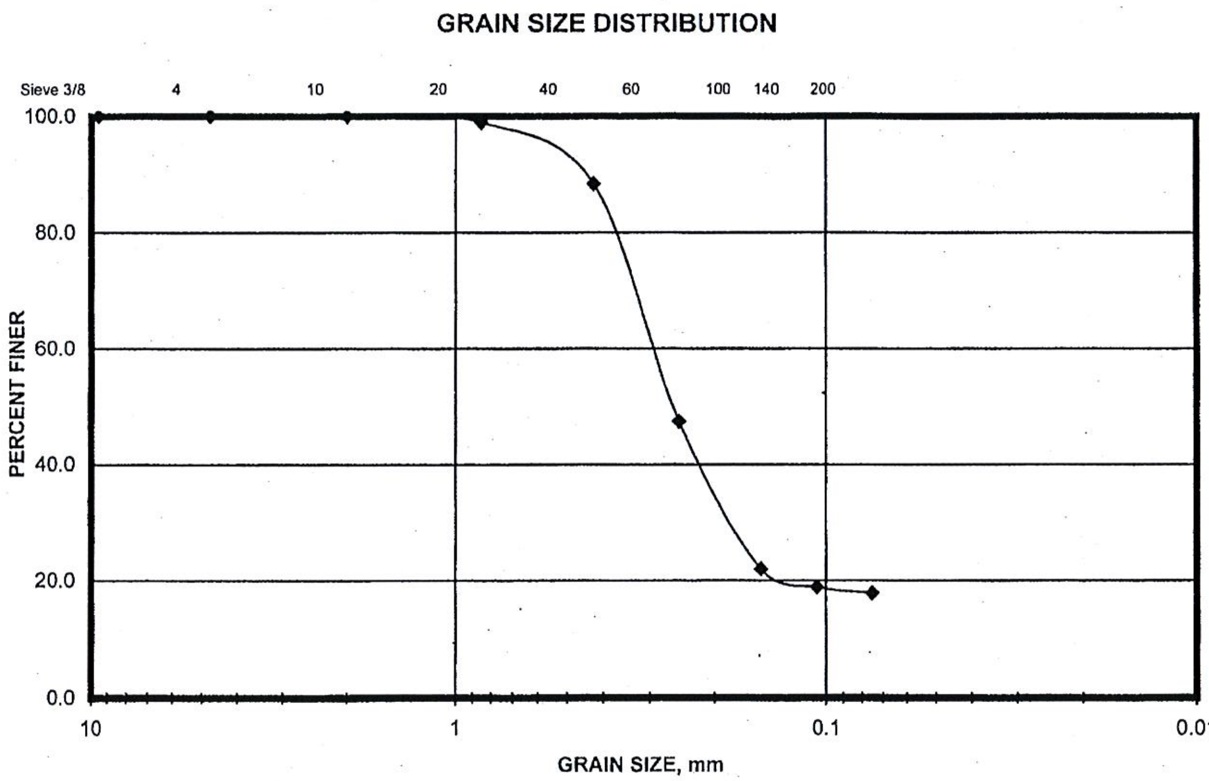
Project No. 35140 Date: 5/22/2003

Project: Hardee County Landfill

Sample Location: TH3 15'

Soil Description: 0

Soil Classification: 0 LL PI



% Gravel	% Sand	%-200
0.0	82.1	17.9

D60	D30	D10	CC	CU

GRAIN SIZE DISTRIBUTION TEST REPORT

PROFESSIONAL SERVICE INDUSTRIES, INC.

Project No. 35140 Date: 5/22/2003

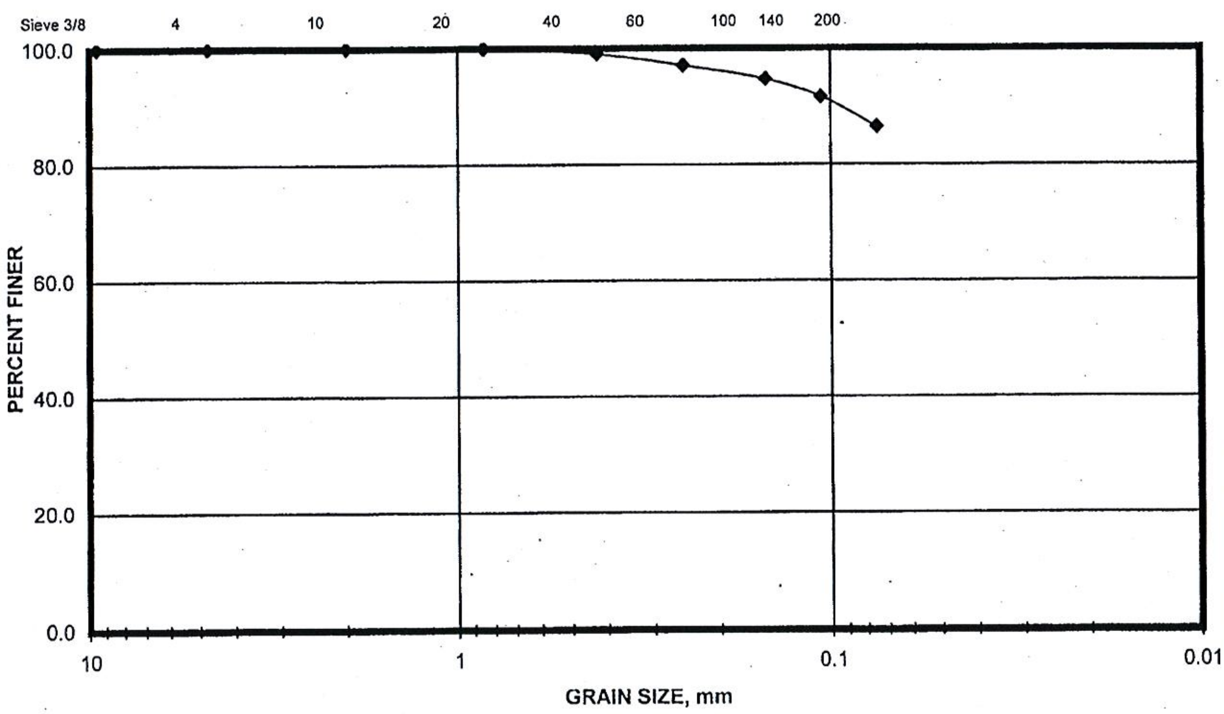
Project: Hardee County Landfill

Sample Location: TH3 30'

Soil Description: 0

Soil Classification: 0 LL ______ PI ______

GRAIN SIZE DISTRIBUTION



% Gravel	% Sand	%-200
0.0	13.5	86.5

D60	D30	D10	CC	CU

GRAIN SIZE DISTRIBUTION TEST REPORT

PROFESSIONAL SERVICE INDUSTRIES, INC.

Project No. 35140 Date: 5/23/2003

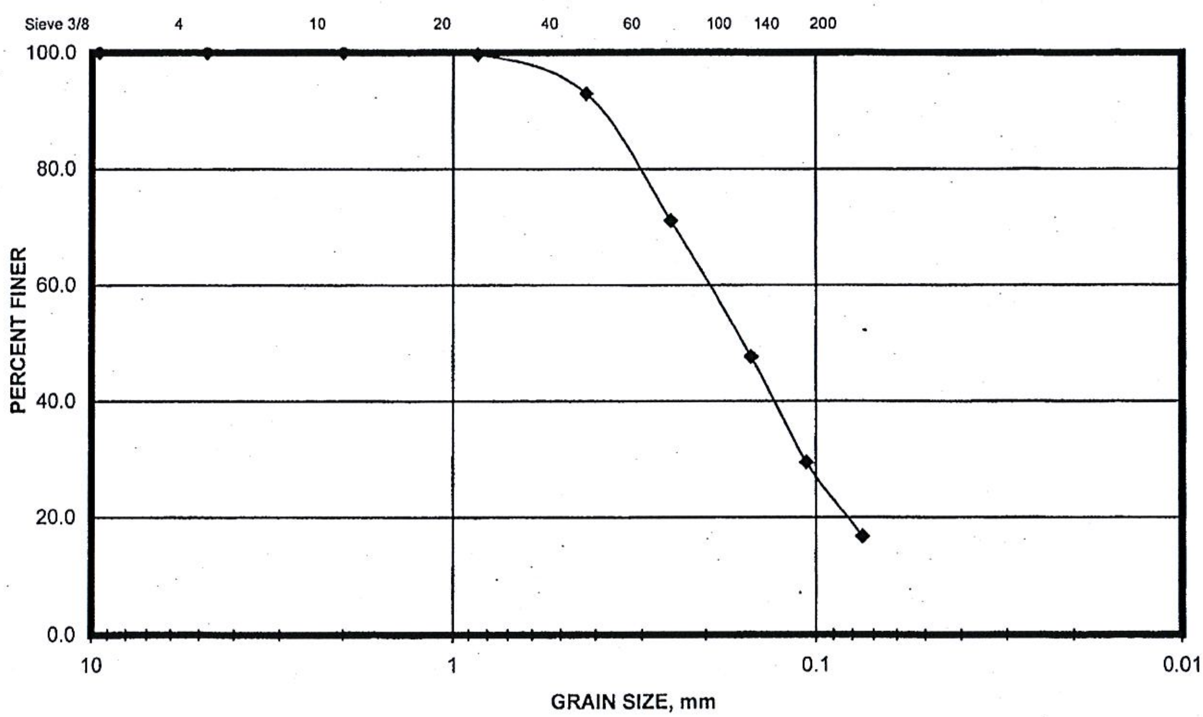
Project: Hardee County Landfill

Sample Location: TH4 5'

Soil Description: 0

Soil Classification: 0 LL PI

GRAIN SIZE DISTRIBUTION



% Gravel		% Sand		%-200
0.0		83.2		16.8
D60	D30	D10	CC	CU

GRAIN SIZE DISTRIBUTION TEST REPORT

PROFESSIONAL SERVICE INDUSTRIES, INC.

Project No. 35140 Date: 6/11/2003

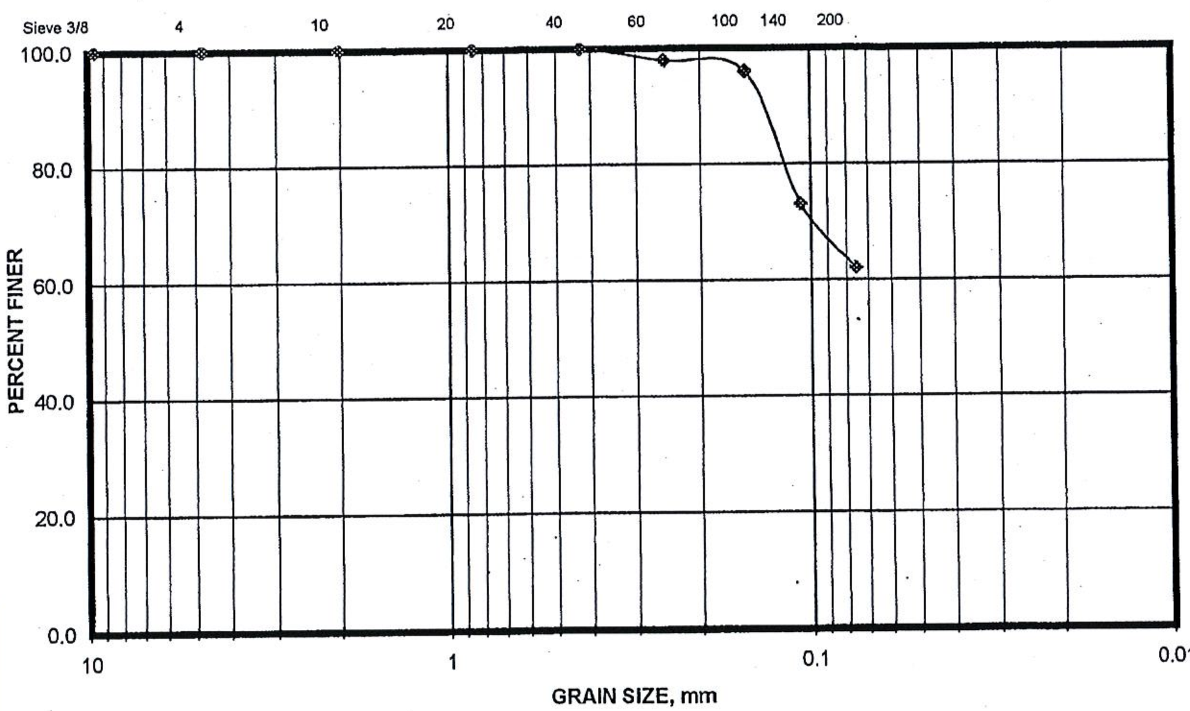
Project: Hardee County Landfill

Sample Location: TH4 23-24'

Soil Description: ____________

Soil Classification: ________ LL ________ PI ________

GRAIN SIZE DISTRIBUTION



% Gravel	% Sand	%-200
0.0	38.0	62.0

D60	D30	D10	CC	CU

GRAIN SIZE DISTRIBUTION TEST REPORT

PROFESSIONAL SERVICE INDUSTRIES, INC.

Project No. 35140 Date: 6/11/03

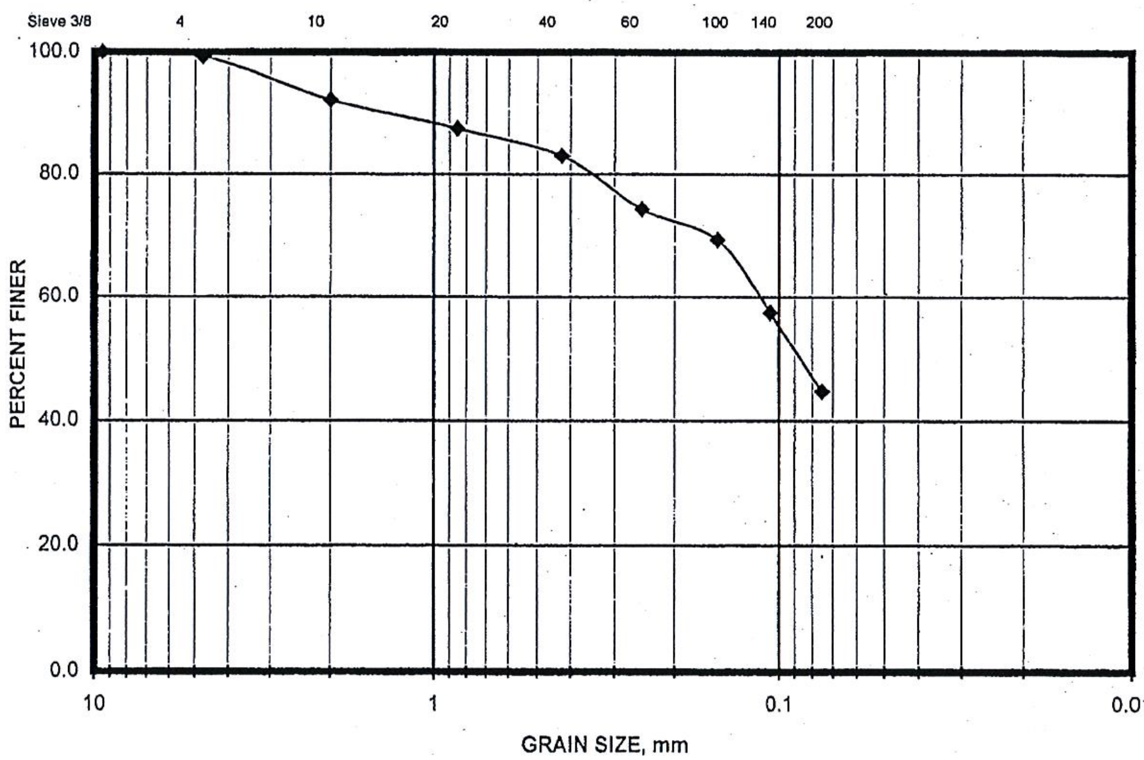
Project: Hardee County Landfill

Sample Location: TH4 24-25'

Soil Description: 0

Soil Classification: 0 LL ______ PI ______

GRAIN SIZE DISTRIBUTION



% Gravel	% Sand	%-200
0.8	54.6	44.7

D60	D30	D10	CC	CU

GRAIN SIZE DISTRIBUTION TEST REPORT

PROFESSIONAL SERVICE INDUSTRIES, INC.

Project No. 35140 Date: 5/22/2003

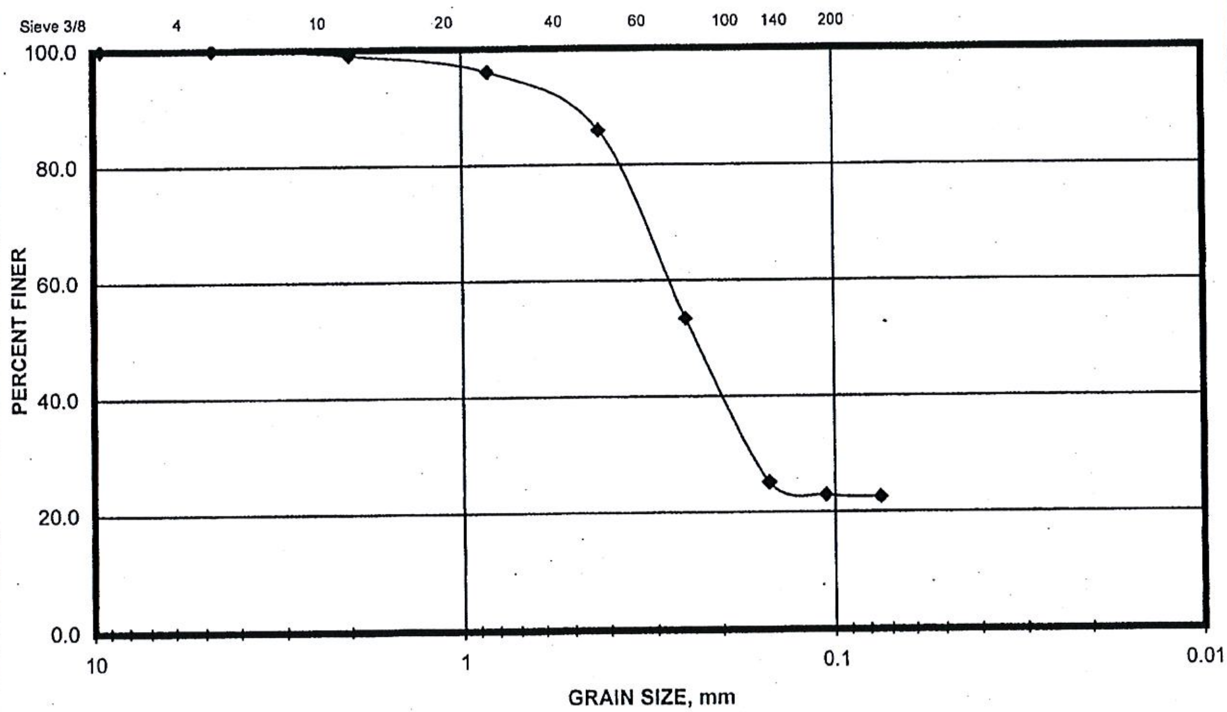
Project: Hardee County Landfill

Sample Location: TH4 40'

Soil Description: 0

Soil Classification: 0 LL ______ PI ______

GRAIN SIZE DISTRIBUTION



% Gravel	% Sand	%-200
0.0	77.4	22.6

D60	D30	D10	CC	CU

GRAIN SIZE DISTRIBUTION TEST REPORT

PROFESSIONAL SERVICE INDUSTRIES, INC.

Project No. 35140 Date: 5/27/2003

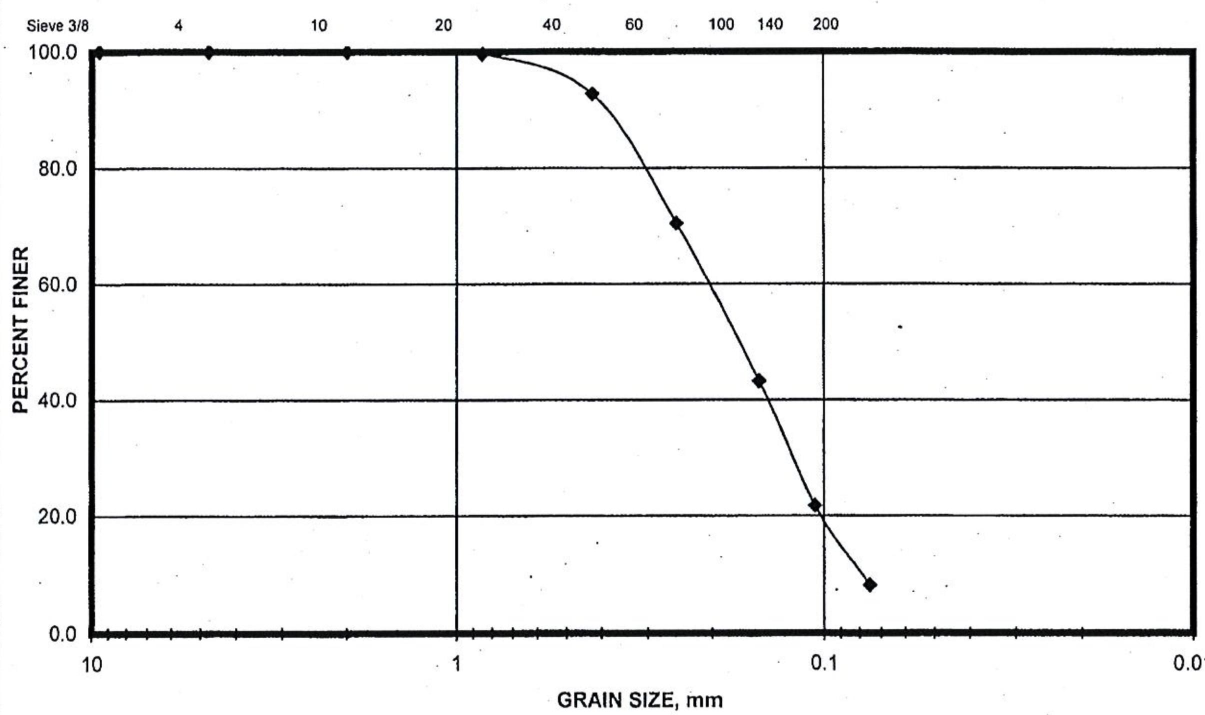
Project: Hardee County Landfill

Sample Location: TH5 4'

Soil Description: 0

Soil Classification: 0 LL ______ PI ______

GRAIN SIZE DISTRIBUTION



% Gravel	% Sand	%-200
0.0	91.9	8.1

D60	D30	D10	CC	CU

GRAIN SIZE DISTRIBUTION TEST REPORT

PROFESSIONAL SERVICE INDUSTRIES, INC.

Project No. 35140 Date: 6/11/2003

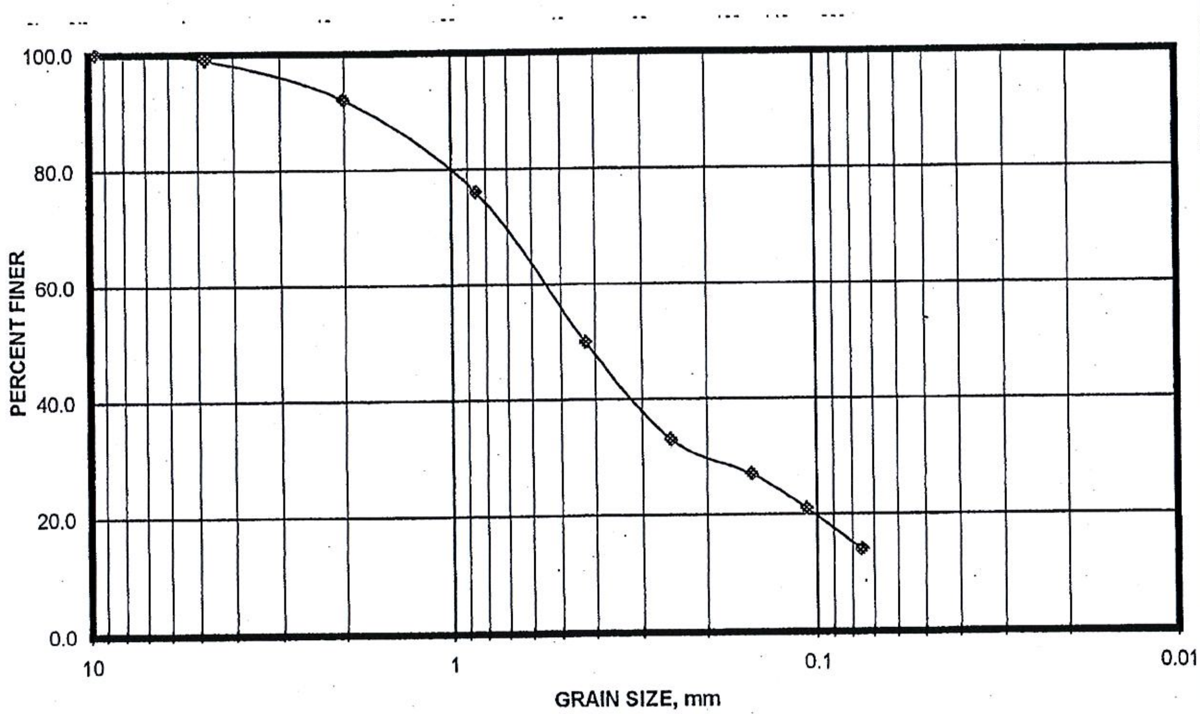
Project: Hardee County Landfill

Sample Location: TH5 13-15'

Soil Description:

Soil Classification: LL PI

GRAIN SIZE DISTRIBUTION



% Gravel	% Sand	%-200
1.0	85.0	14.0

D60	D30	D10	CC	CU

GRAIN SIZE DISTRIBUTION TEST REPORT

PROFESSIONAL SERVICE INDUSTRIES, INC.

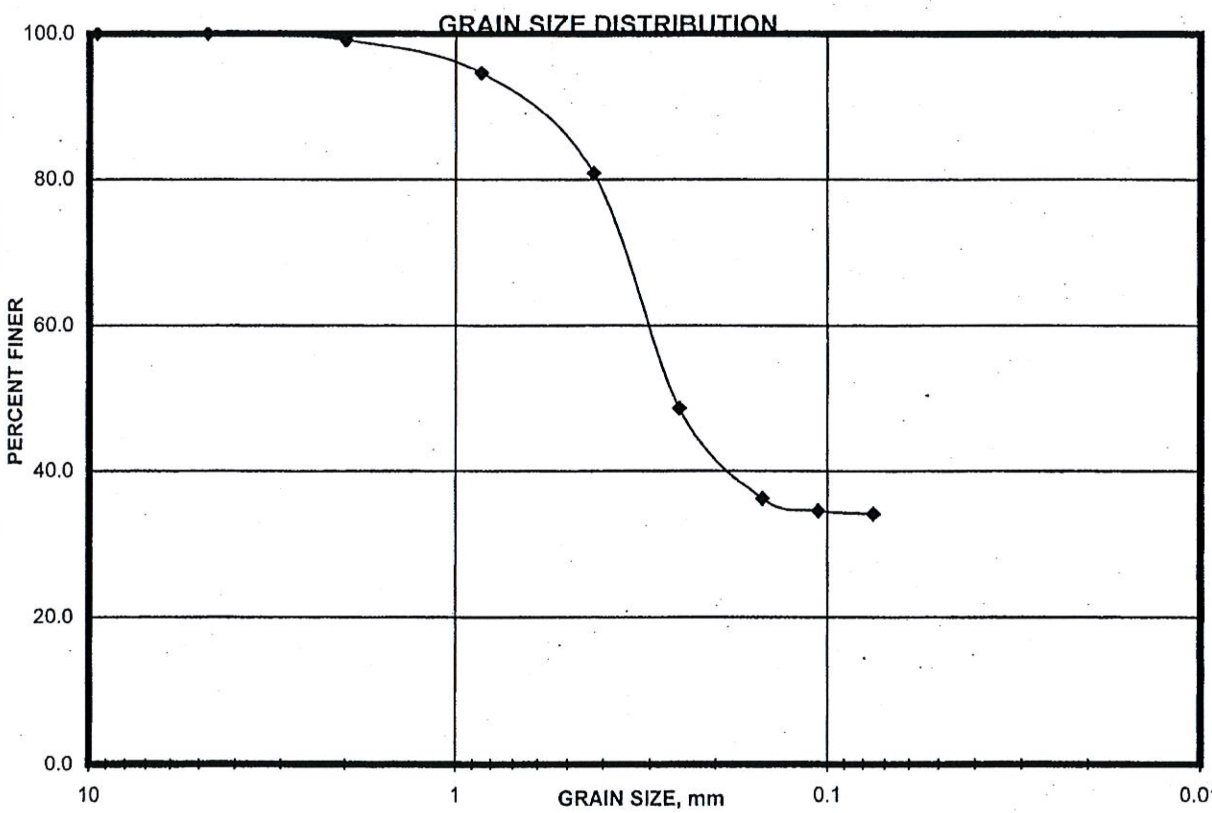
Project No. 35140 Date: 5/23/2003

Project: Hardee County Landfill

Sample Location: TH5 30'

Soil Description: 0

Soil Classification: 0 LL PI



% Gravel	% Sand	%-200
0.0	65.9	34.1

D60	D30	D10	CC	CU

GRAIN SIZE DISTRIBUTION TEST REPORT

PROFESSIONAL SERVICE INDUSTRIES, INC.

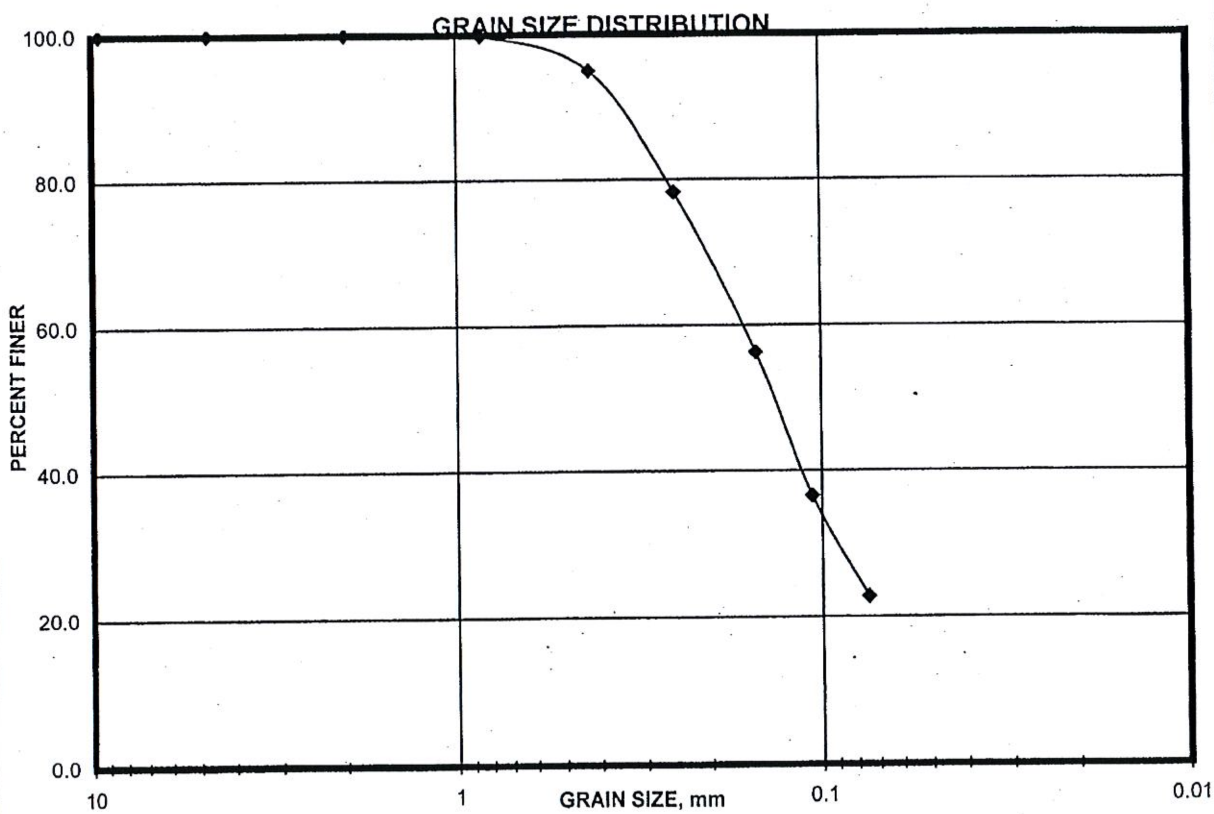
Project No. 35140 Date: 5/22/2003

Project: Hardee County Landfill

Sample Location: TH6 5'

Soil Description: 0

Soil Classification: 0 LL ______ PI ______



% Gravel		% Sand		%-200
0.0		77.3		22.7
D60	D30	D10	CC	CU

GRAIN SIZE DISTRIBUTION TEST REPORT

PROFESSIONAL SERVICE INDUSTRIES, INC.

Project No. 35140 Date: 6/11/2003

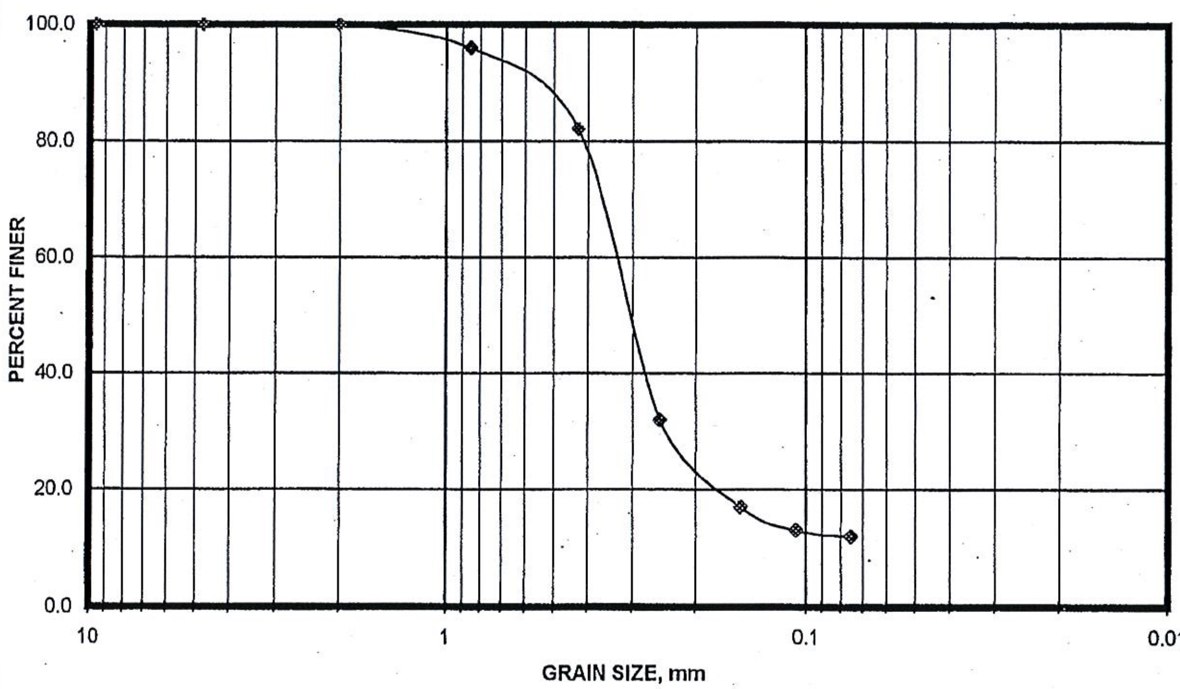
Project: Hardee County Landfill

Sample Location: TH6 15'

Soil Description:

Soil Classification: LL PI

GRAIN SIZE DISTRIBUTION



% Gravel	% Sand	%-200
0.0	88.0	12.0

D60	D30	D10	CC	CU

GRAIN SIZE DISTRIBUTION TEST REPORT

PROFESSIONAL SERVICE INDUSTRIES, INC.

Project No. 35140

Date: 6/11/03

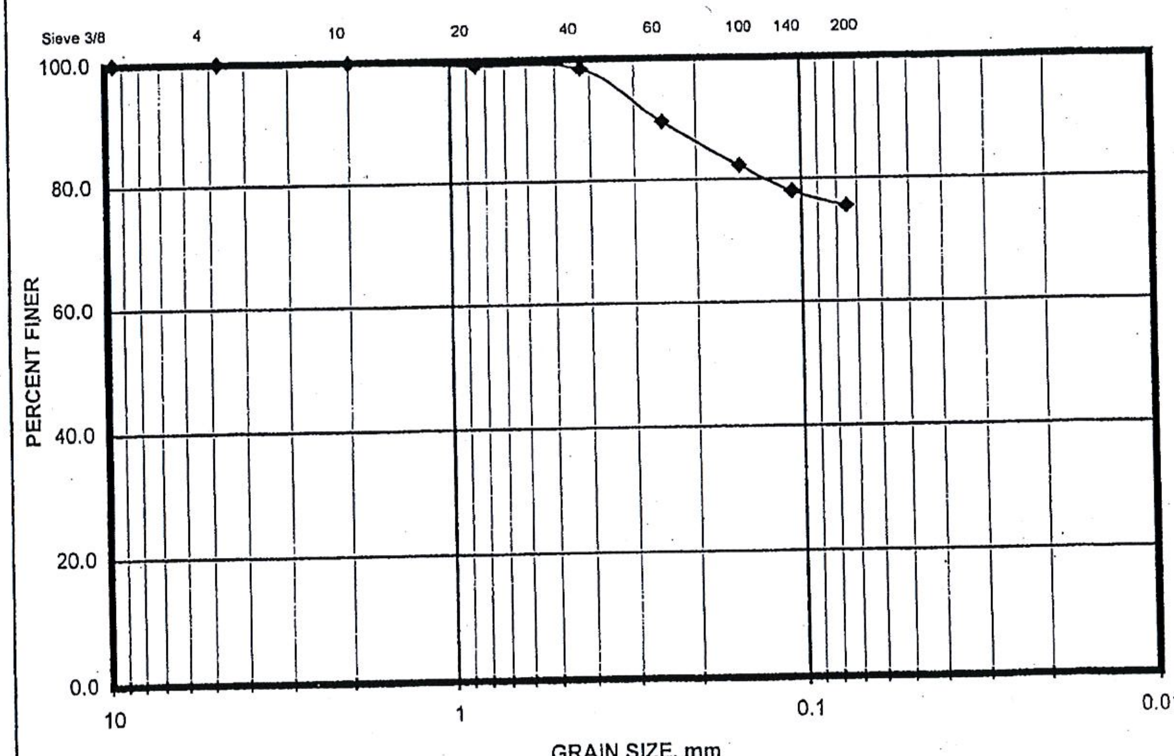
Project: Hardee County Landfill

Sample Location: TH6 18-20'

Soil Description: 0

Soil Classification: 0 LL ______ PI ______

GRAIN SIZE DISTRIBUTION



% Gravel		% Sand		%-200
0.0		24.5		75.5
D60	D30	D10	CC	CU

GRAIN SIZE DISTRIBUTION TEST REPORT

PROFESSIONAL SERVICE INDUSTRIES, INC.

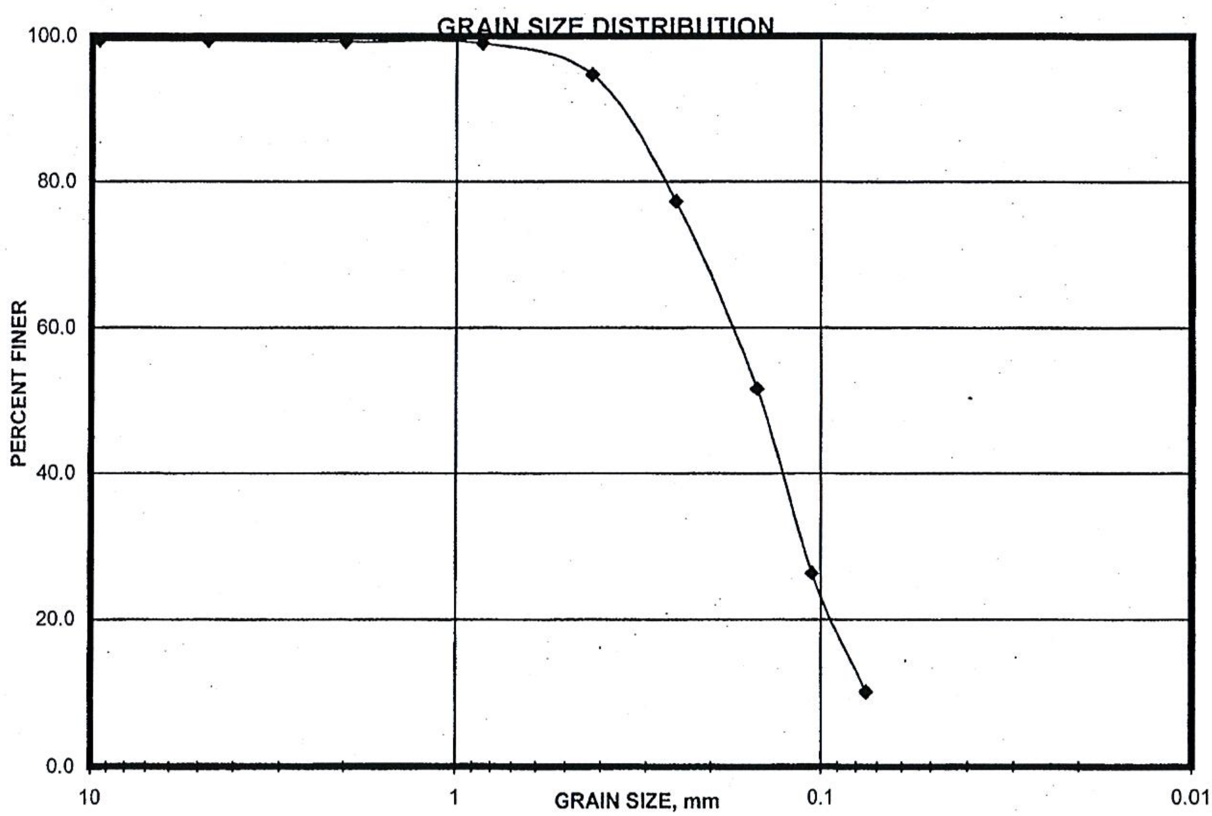
Project No. 35140 Date: 5/27/2003

Project: Hardee County Landfill

Sample Location: TH7 5'

Soil Description: 0

Soil Classification: 0 LL ______ PI ______



% Gravel		% Sand		%-200
0.6		89.3		10.1
D60	D30	D10	CC	CU

GRAIN SIZE DISTRIBUTION TEST REPORT

PROFESSIONAL SERVICE INDUSTRIES, INC.

Project No. 35140 Date: 6/11/2003

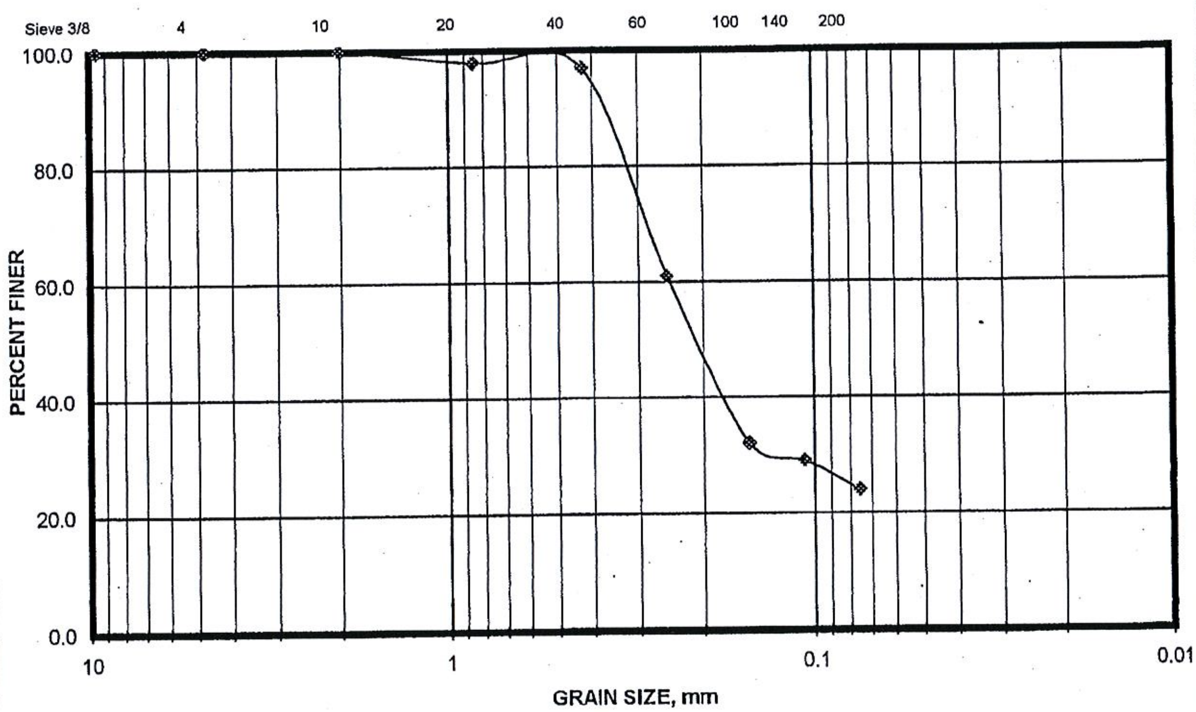
Project: Hardee County Landfill

Sample Location: TH7 13-15' CONSOL

Soil Description:

Soil Classification: LL 61 PI 39

GRAIN SIZE DISTRIBUTION



% Gravel	% Sand	%-200
0.0	76.0	24.0

D60	D30	D10	CC	CU

GRAIN SIZE DISTRIBUTION TEST REPORT

PROFESSIONAL SERVICE INDUSTRIES, INC.

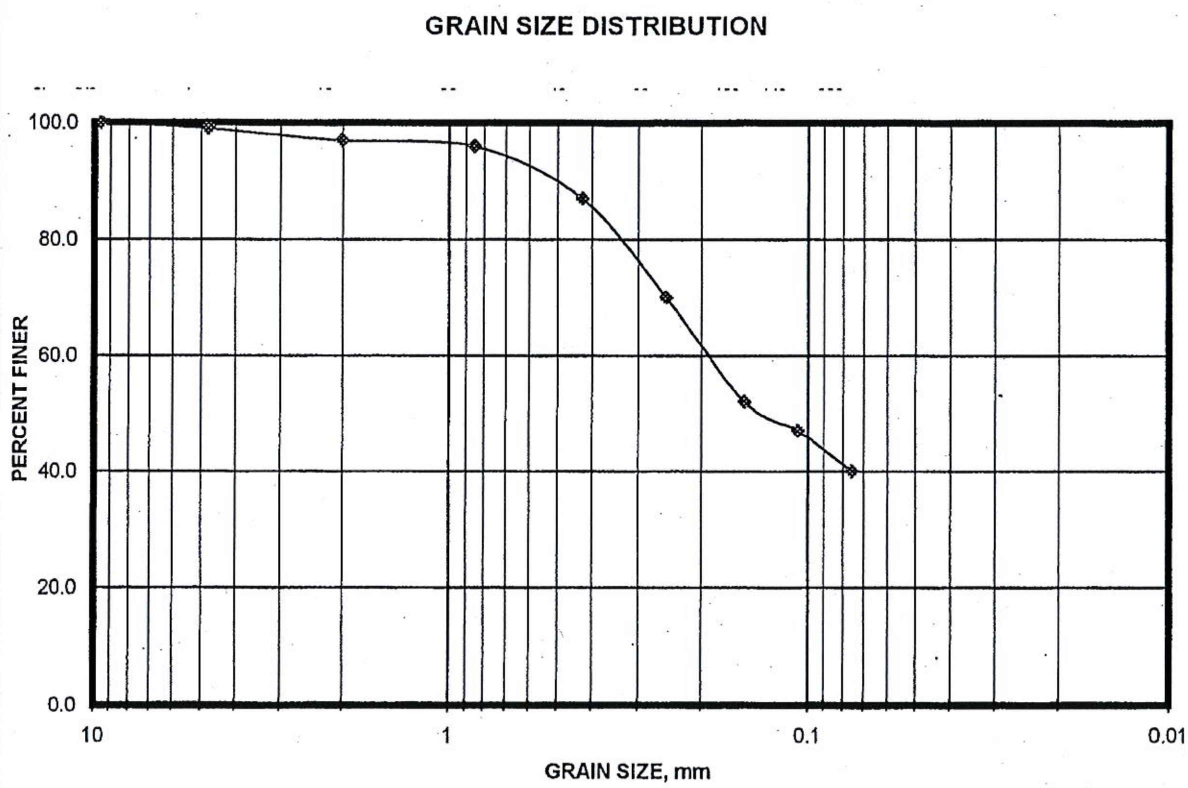
Project No. 35140 Date: 6/11/2003

Project: Hardee County Landfill

Sample Location: TH7 13-15' PERM

Soil Description:

Soil Classification: LL 50 PI 26

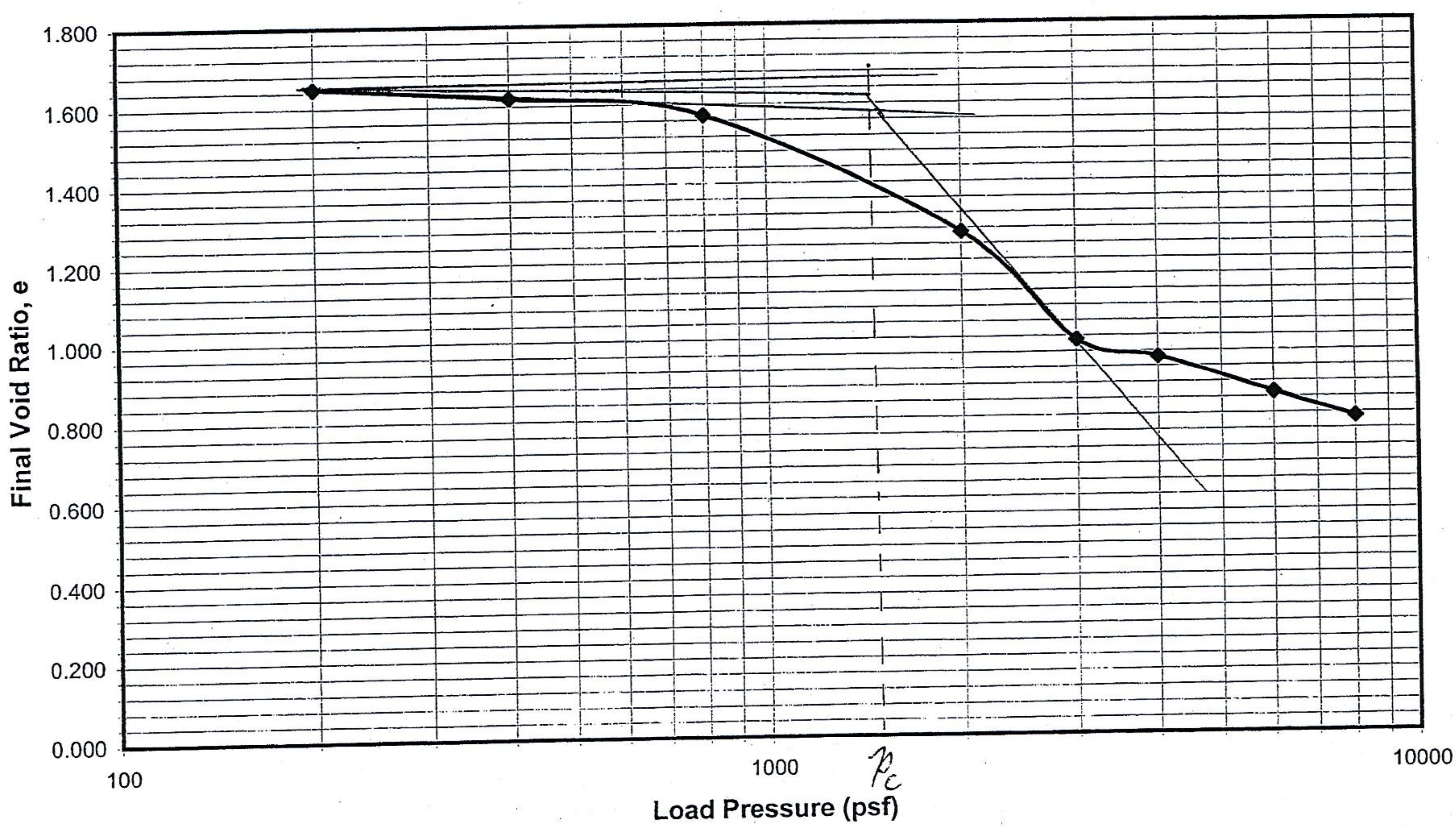


% Gravel		% Sand		%-200
1.0		59.0		40.0
D60	D30	D10	CC	CU

APPENDIX II

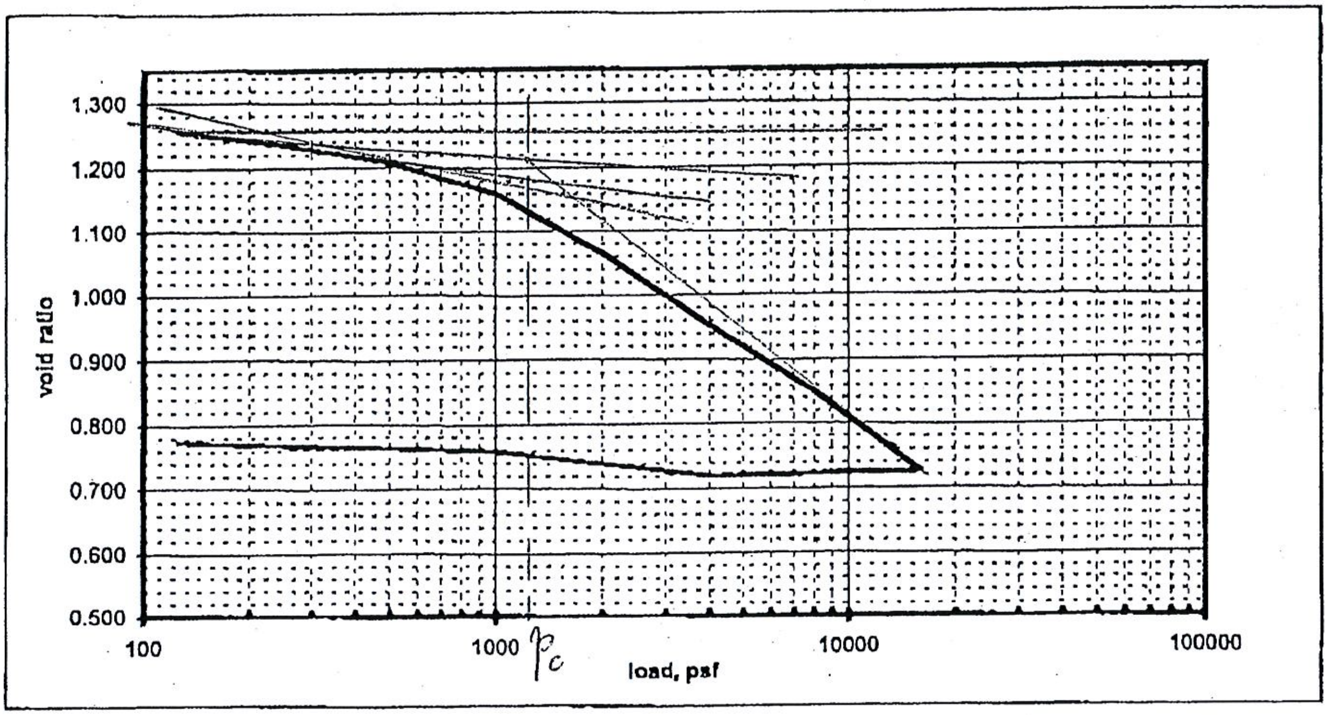
CONSOLIDATION TEST RESULTS

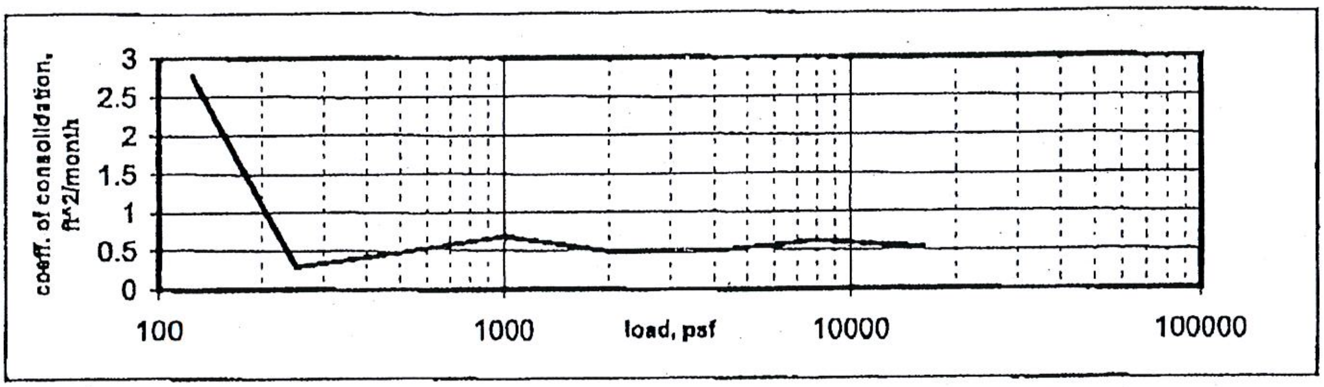
Hardee County Landfill e-log p Curve
TH-1, 23.5-25'



psi Information To Build On
Engineering - Consulting - Testing

MEMPHIS GEOTECHNICAL DEPARTMENT





CONSOLIDATION TEST RESULTS

Sample I.D.:	US-7 13'-15'		before test
Sample Classification:		Moisture, %:	48.8
Liquid Limit:	61	Void Ratio:	1.279
Plasticity Index:	39	Saturation, %:	100.0
Dry Density:	73.9 pcf	Specific Gravity:	2.7

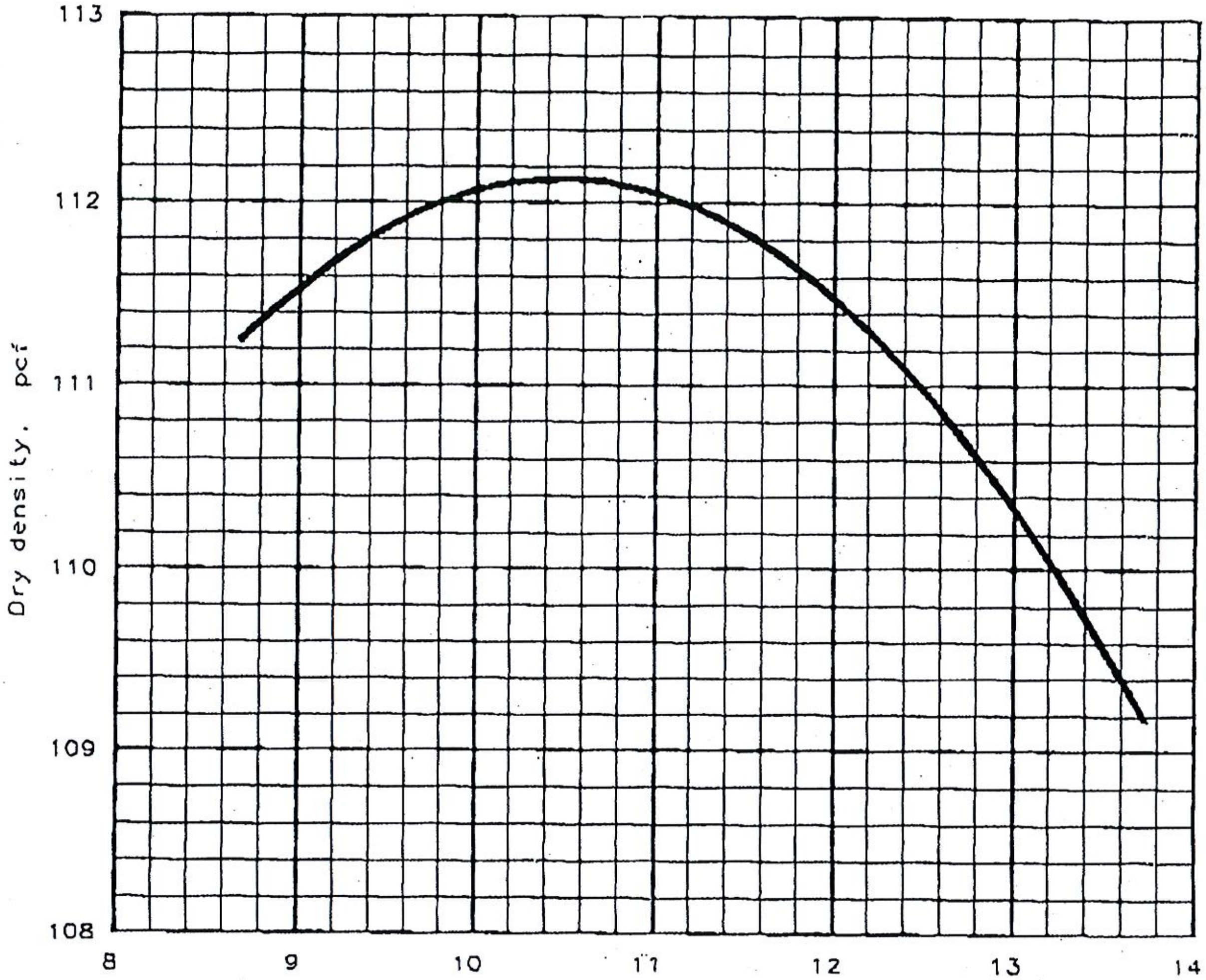
PROJECT: Hardee Landfill	FILE NO: 775-35140
	DATE: 6/10/03

APPENDIX III

MOISTURE-DENSITY (PROCTOR)

Proctor

MOISTURE-DENSITY RELATIONSHIP TEST



Test specification: ASTM D 698-91 Procedure A, Standard

Elev/ Depth	Classification USCS	Classification AASHTO	Nat. Moist.	Sp.G.	LL	PI	% > No.4	% < No.200

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 112.1 pcf Optimum moisture = 10.5 %	LT.BROWN SL.SILTY FINE SAND

Project No.: 761

Project: HARDEE COUNTY LANDFILL

Location: TH-1 (4' BELOW SURFACE)

Date: 5-19-2003

Remarks:

FILE#382

MOISTURE-DENSITY RELATIONSHIP TEST

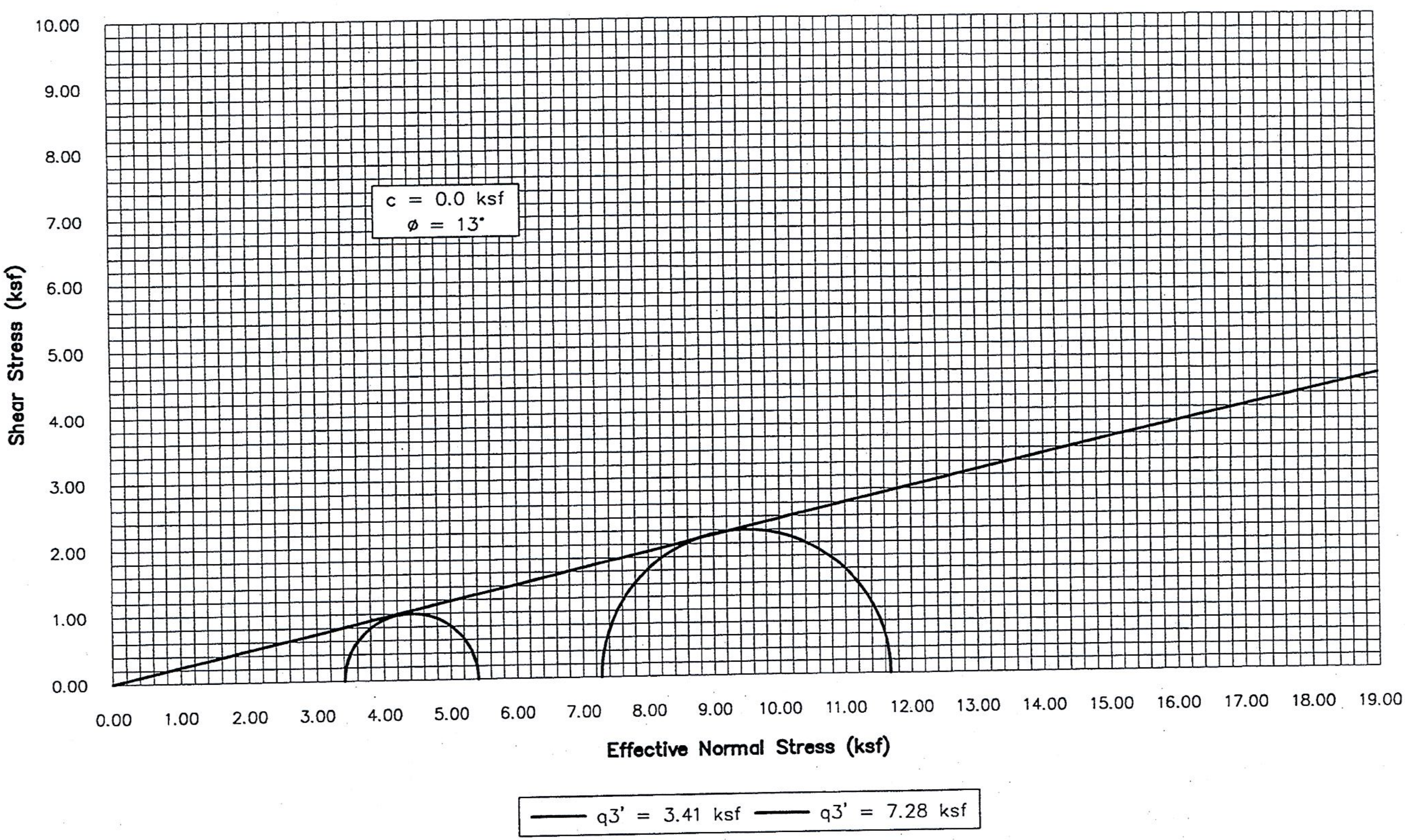
PSI, Inc.

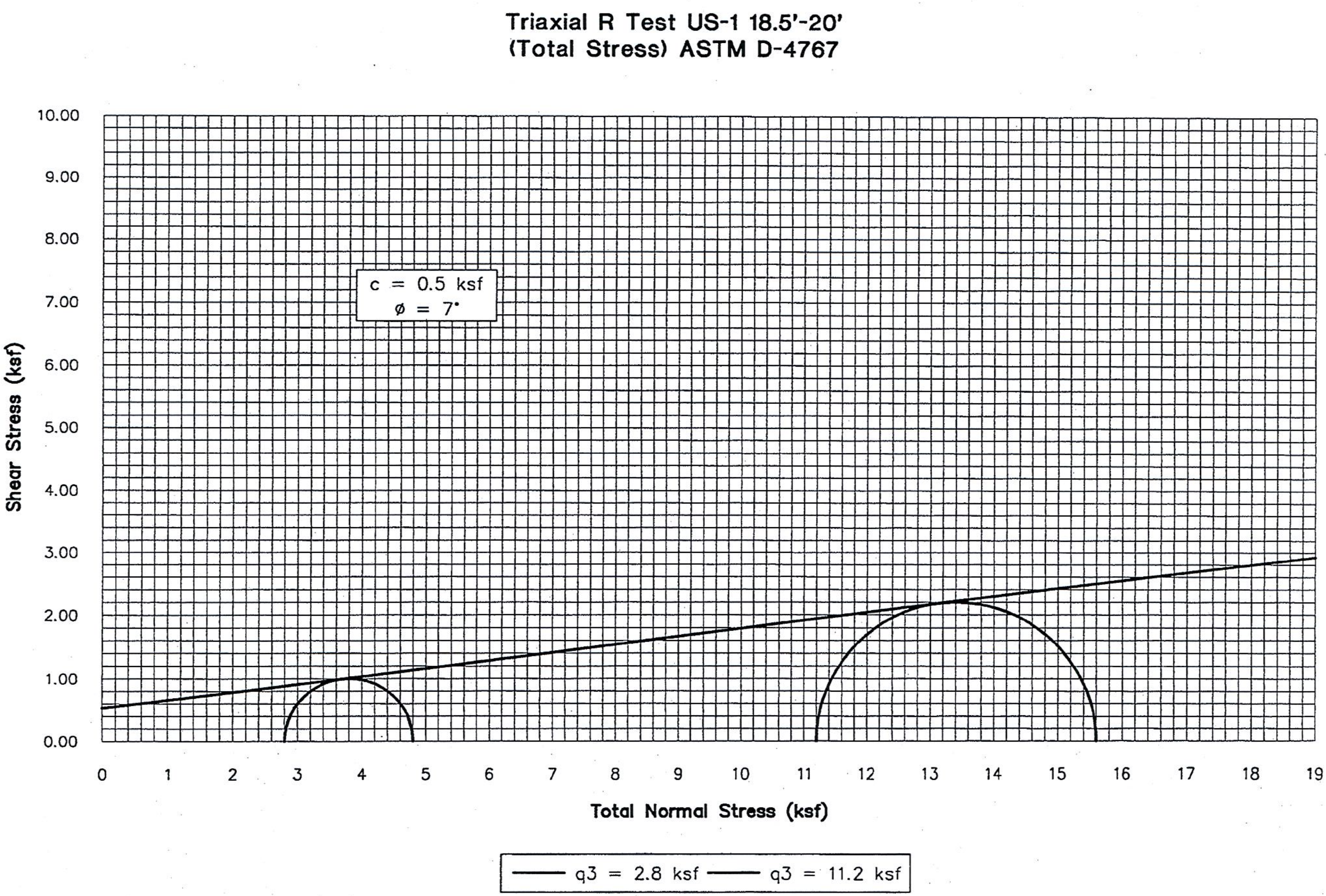
Fig. No. ______

APPENDIX IV

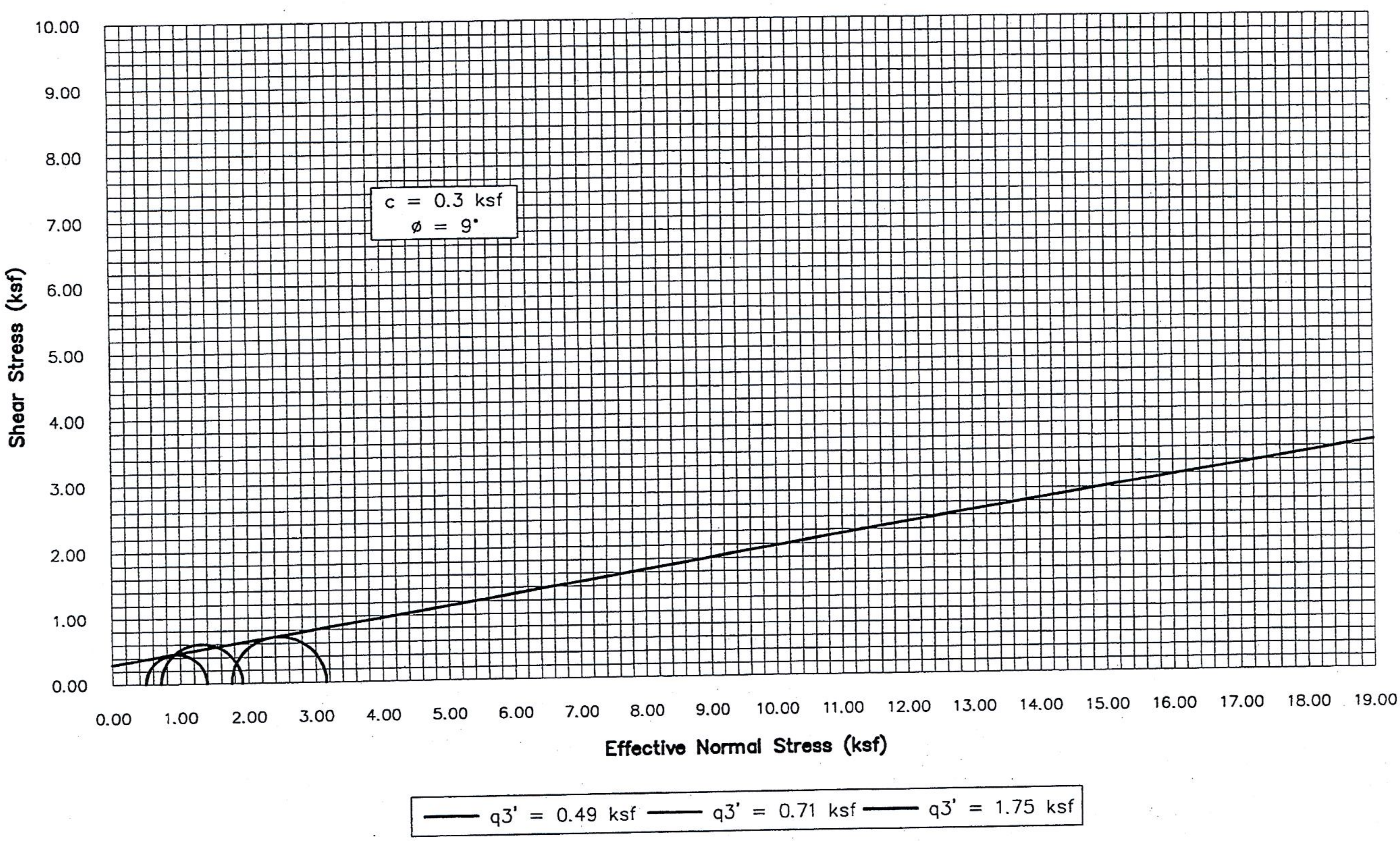
TRI-AXIAL STRENGTH TEST RESULTS

Triaxial R Test US-1 18.5'-20'
(Effective Stress) ASTM D-4767

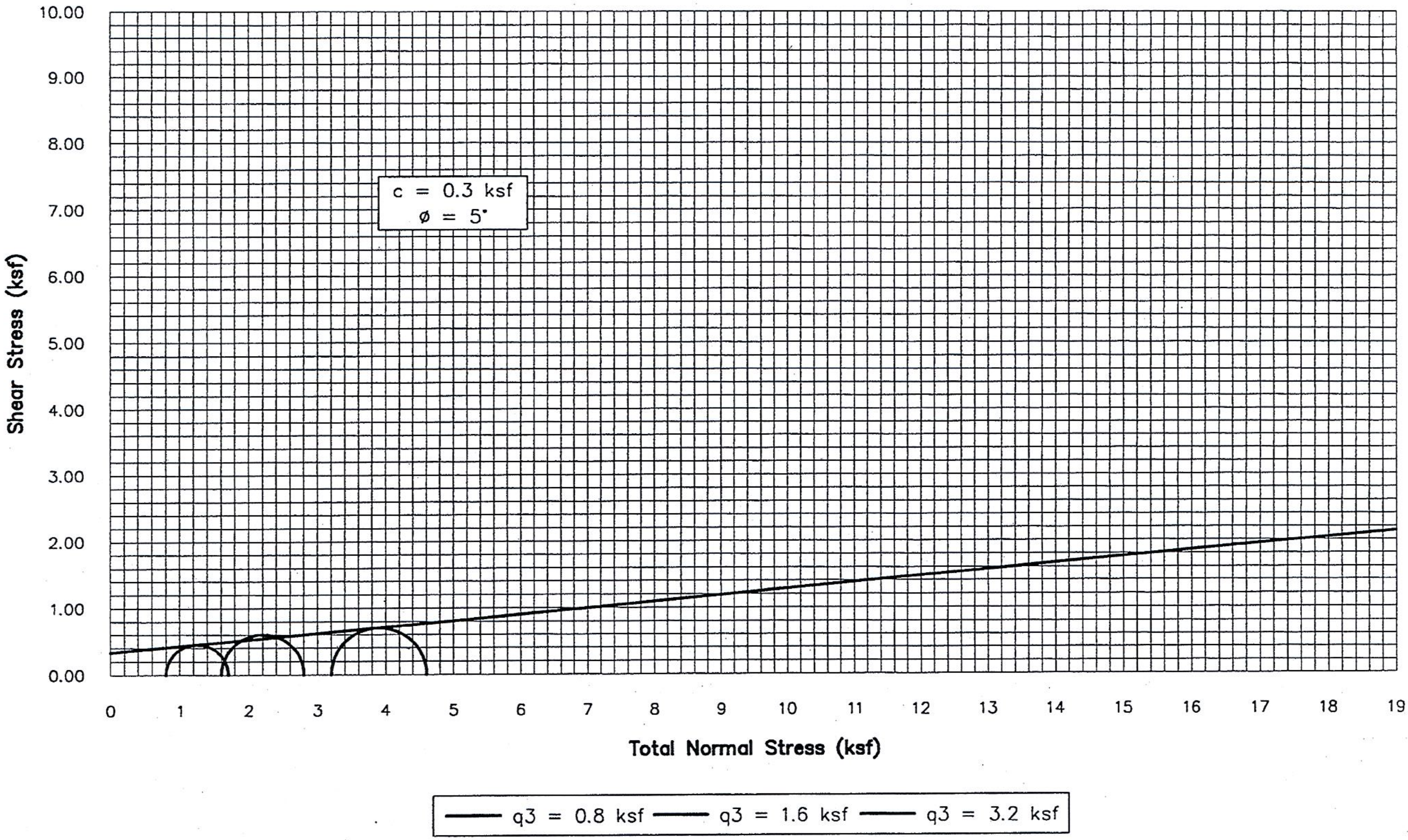



Triaxial R Test US-1 18.5'-20'
(Total Stress) ASTM D-4767
c = 0.5 ksf
ø = 7°
Shear Stress (ksf)
10.00
9.00
8.00
7.00
6.00
5.00
4.00
3.00
2.00
1.00
0.00
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
Total Normal Stress (ksf)
q3 = 2.8 ksf
q3 = 11.2 ksf

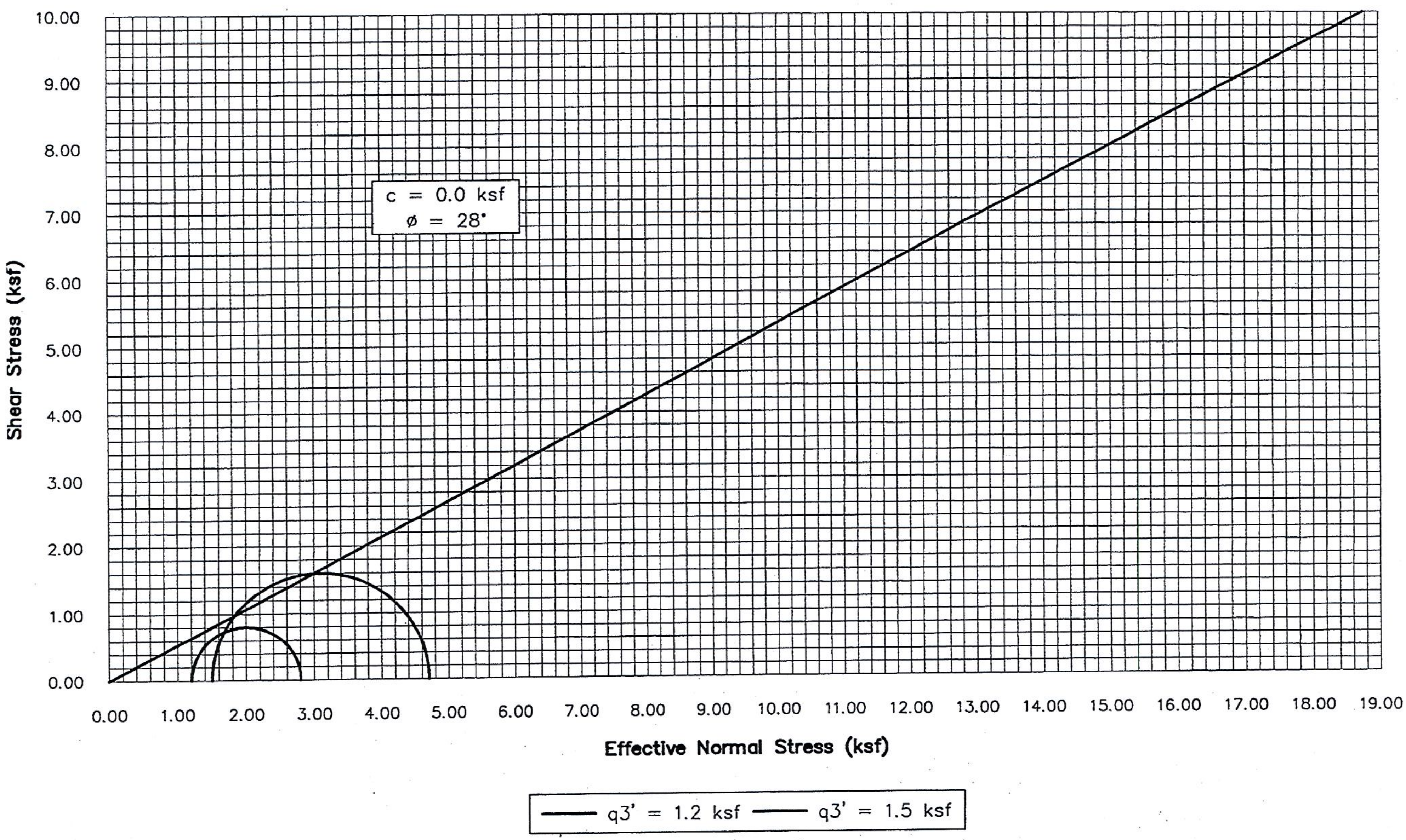
Triaxial R Test US-4 23'-24'
(Effective Stress) ASTM D-4767



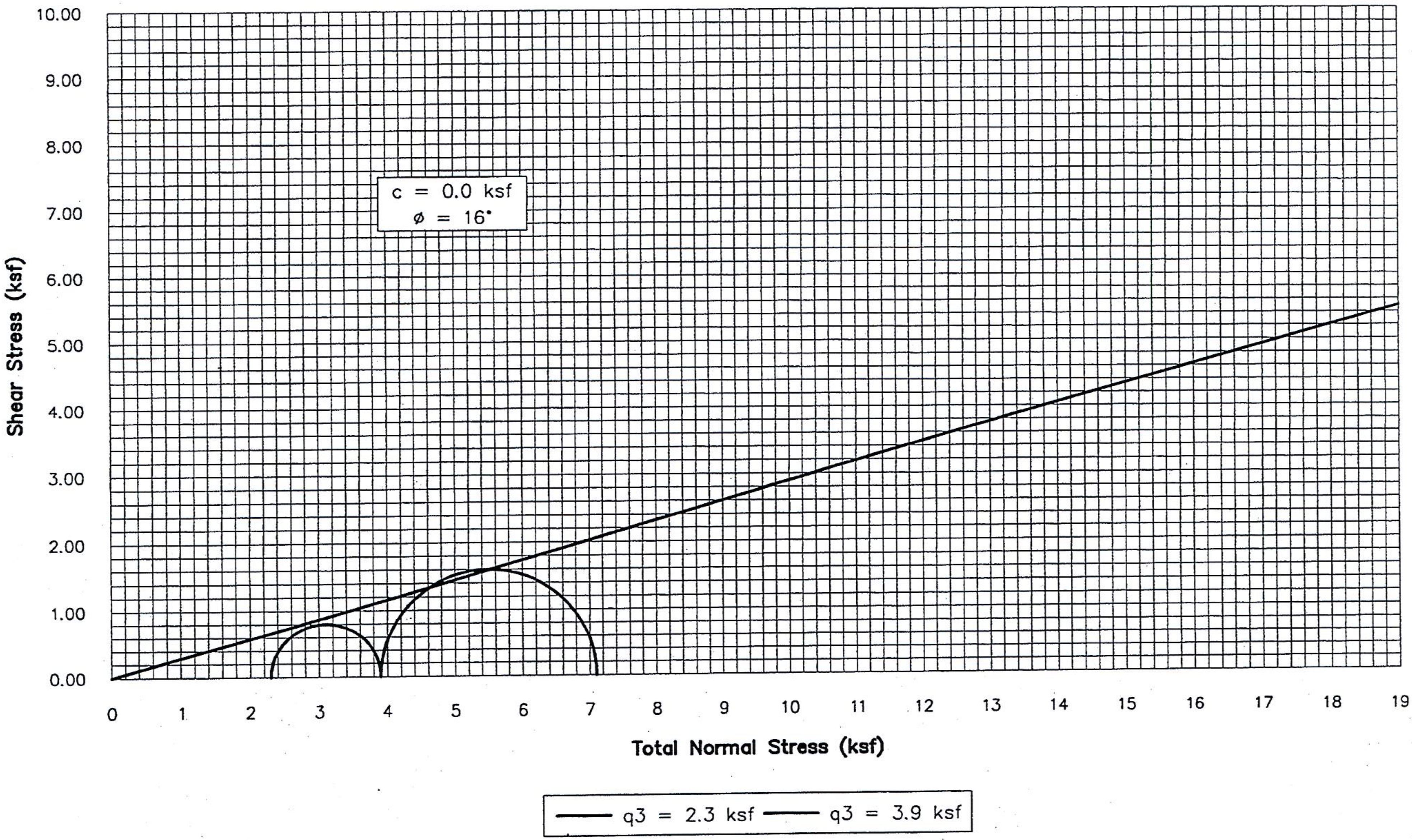
Triaxial R Test US-4 23'-24'
(Total Stress) ASTM D-4767



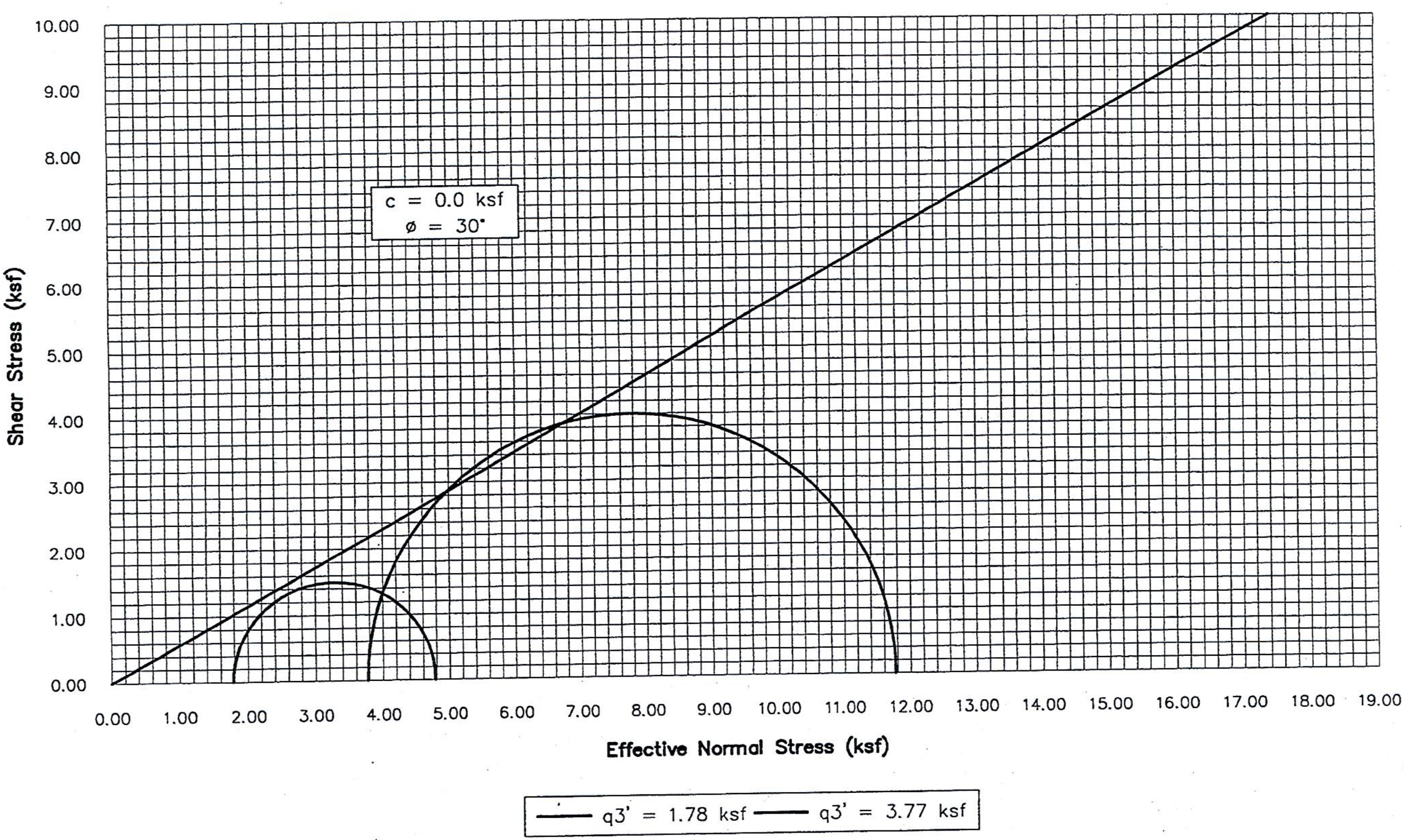
Triaxial R Test US-5 13.5'-15'
(Effective Stress) ASTM D-4767



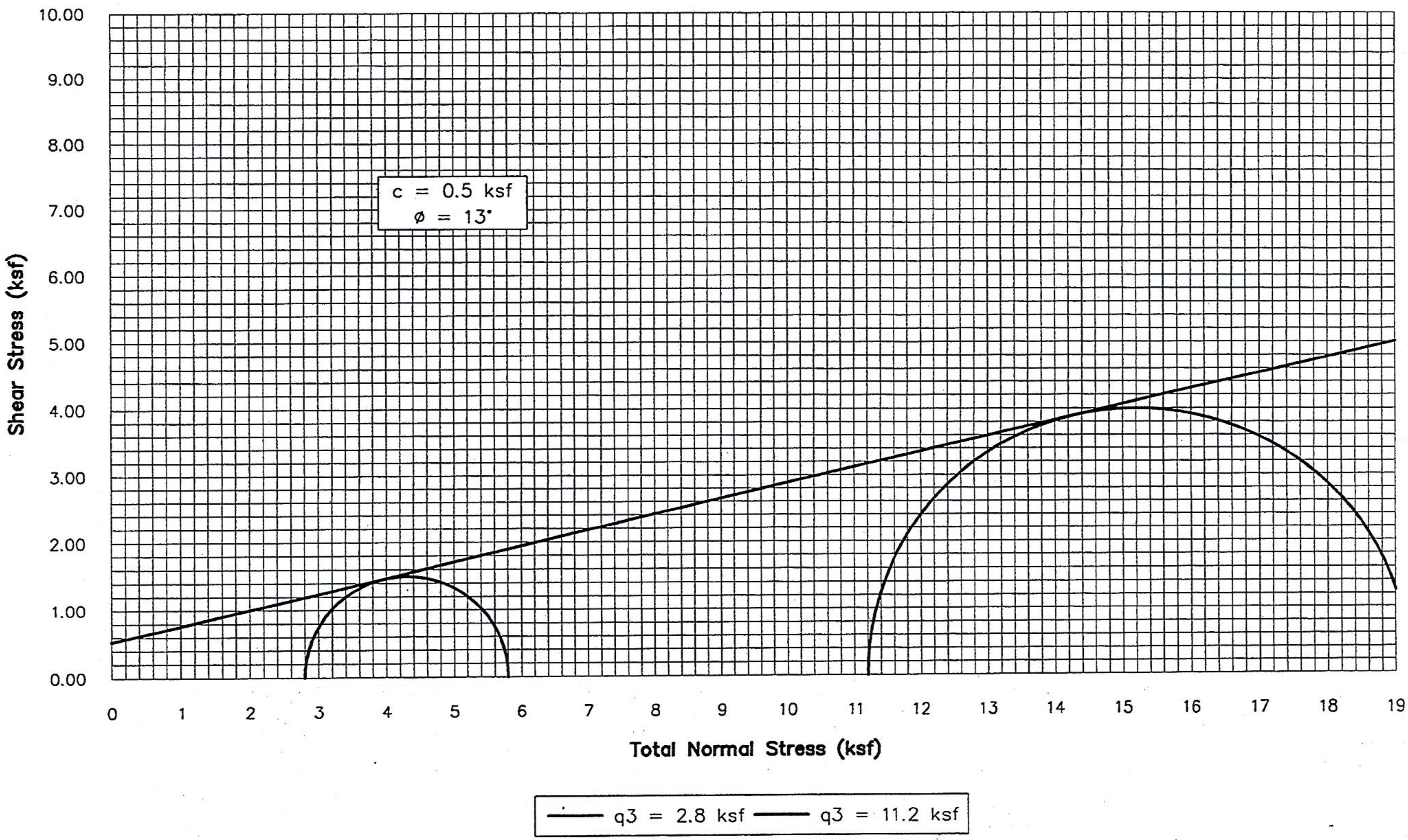
Triaxial R Test US-5 13.5'-15'
(Total Stress) ASTM D-4767



Triaxial R Test US-6 18'-20'
(Effective Stress) ASTM D-4767



Triaxial R Test US-6 18'-20'
(Total Stress) ASTM D-4767



Attachment 2

Principal of Geotechnical Engineering Das (1985)

Principal of Geotechnical Engineering, Das (1985)

Table 13.2 Approximate Correlation of Standard Penetration Number and Consistency of Clay

Standard penetration number, N	Consistency	Unconfined compression strength, q_u (ton/ft^2)
0		0
	Very soft	
2		0.25
	Soft	
4		0.5
	Medium stiff	
8		1
	Stiff	
16		2
	Very stiff	
32		4
>32	Hard	>4

Note: 1 ton/ft^2 = 95.76 kN/m^2

overburden pressure (and hence higher lateral confining pressure) at depth h_2 will contribute to a higher value of the standard penetration number. This fact has clearly been demonstrated by Gibbs and Holtz (1957). The results of their findings are shown in Figure 13.10. As an example, one can see that at $D_r \approx 80\%$, the standard penetration number is about 12 with $\sigma' = 0$ lb/ft^2. It increases to about 50 with $\sigma' = 40$ lb/in.^{2} (276 kN/m^2). For that reason, it is necessary to convert the standard penetration numbers obtained at various depths to reflect a constant effective overburden pressure. Peck, Hanson, and Thornburn (1974) proposed the following empirical correlation for converting the field standard penetration number to an effective overburden pressure of $\sigma' = 1$ ton/ft^2 (95.6 kN/m^2).

$$N' = C_N N_F = 0.77 N_F \log\left(\frac{20}{\sigma'}\right) \quad (\text{for } \sigma' > 0.25 \text{ ton/ft}^2) \tag{13.6}$$

where

N' = corrected standard penetration number
N_F = field standard penetration number
C_N = correction factor

The unit of σ' is in ton/ft^2.

In SI units, the preceding equation can be expressed as

$$N' = 0.77 N_F \log\left(\frac{20}{0.0105\,\sigma'}\right) \quad (\text{for } \sigma' > 23.9 \text{ kN/m}^2) \tag{13.7}$$

The unit of σ' in Eq. (13.7) is in kN/m^2.

Table 13.3 Approximate Relation Between Corrected Standard Penetration Number, Angle of Friction, and Relative Density of Sand

Corrected standard penetration number, N	Relative density, D_r (%)	Angle of friction, ϕ (degrees)
0–5	0–5	26–30
5–10	5–30	28–35
10–30	30–60	35–42
30–50	60–95	38–46

The standard penetration number is a very useful guideline in soil exploration and assessment of subsoil conditions, provided that the results are interpreted correctly. Note that all equations and correlations relating to the standard penetration numbers are approximate. Since soil is not homogeneous, a wide variation in the N-value may be obtained in the field. In soil deposits

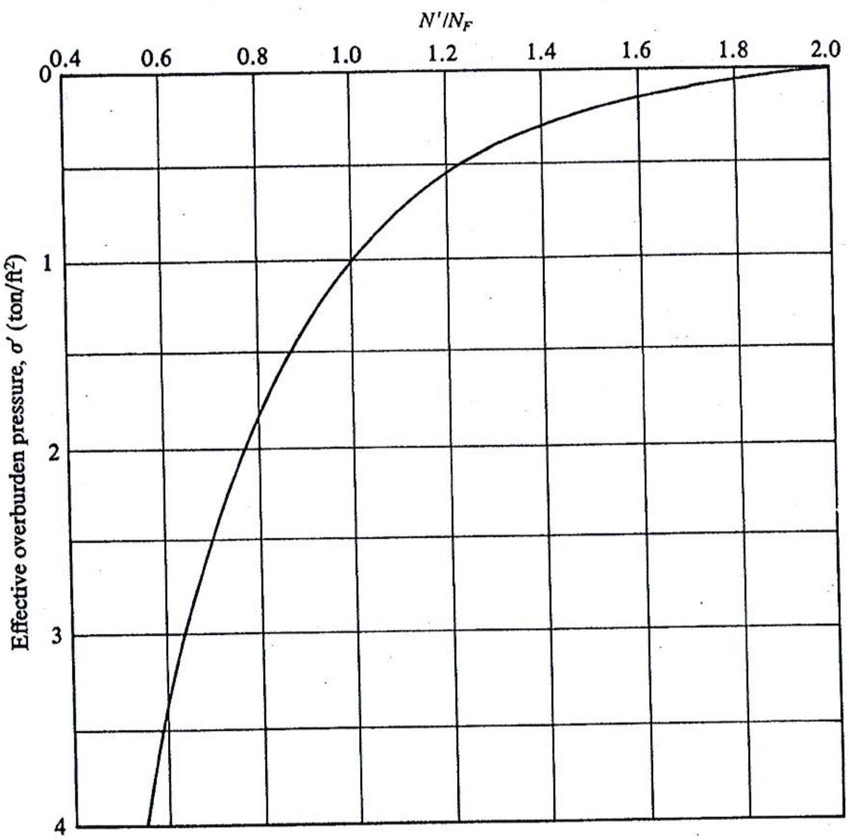


Figure 13.11 Variation of N'/N_F with vertical effective stress, σ' (after Peck, Hanson, and Thornburn, 1974)

Attachment 3

T. William Lambe * Robert V. Whitman (1969)

Soil Mechanics

T. William Lambe • Robert V. Whitman

Massachusetts Institute of Technology

1969

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

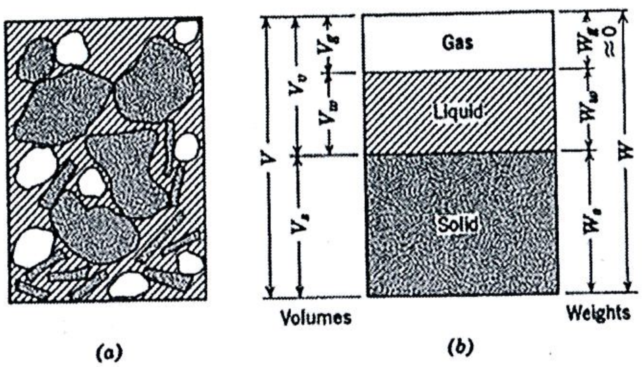


Fig. 3.1 Relationships among soil phases. (*a*) Element of natural soil. (*b*) Element separated into phases.

Volume

Porosity:

$$n = \frac{V_v}{V}$$

Void ratio:

$$e = \frac{V_v}{V_s}$$

Degree of saturation:

$$S = \frac{V_w}{V_v}$$

$$n = \frac{e}{1+e}; \qquad e = \frac{n}{1-n}$$

Weight

Water content:

$$w = \frac{W_w}{W_s}$$

Specific Gravity

Mass:

$$G_m = \frac{\gamma_t}{\gamma_0}$$

Water:

$$G_w = \frac{\gamma_w}{\gamma_0}$$

Solids:

$$G = \frac{\gamma_s}{\gamma_0}$$

γ_0 = Unit weight of water at 4°C $\approx \gamma_w$

Note that $Gw = Se$

Unit Weight

Total:

$$\gamma_t = \frac{W}{V} = \frac{G + Se}{1+e}\gamma_w = \frac{1+w}{1+e}G\gamma_w$$

Solids:

$$\gamma_s = \frac{W_s}{V_s}$$

Water:

$$\gamma_w = \frac{W_w}{V_w}$$

Dry:

$$\gamma_d = \frac{W_s}{V} = \frac{G}{1+e}\gamma_w = \frac{G\gamma_w}{1 + wG/S} = \frac{\gamma_t}{1+w}$$

Submerged (buoyant):

$$\gamma_b = \gamma_t - \gamma_w = \frac{G - 1 - e(1-S)}{1+e}\gamma_w$$

Submerged (saturated soil):

$$\gamma_b = \gamma_t - \gamma_w = \frac{G-1}{1+e}\gamma_w$$

Specific gravity is the unit weight divided by the unit weight of water. Values of specific gravity of solids G for a selected group of minerals[3] are given in Table 3.1.

Table 3.1 Specific Gravities of Minerals

Quartz	2.65
K-Feldspars	2.54–2.57
Na-Ca-Feldspars	2.62–2.76
Calcite	2.72
Dolomite	2.85
Muscovite	2.7–3.1
Biotite	2.8–3.2
Chlorite	2.6–2.9
Pyrophyllite	2.84
Serpentine	2.2–2.7
Kaolinite	2.61[a]
	2.64 ± 0.02
Halloysite (2 H_2O)	2.55
Illite	2.84[a]
	2.60–2.86
Montmorillonite	2.74[a]
	2.75–2.78
Attapulgite	2.30

[a] Calculated from crystal structure.

The expression $Gw = Se$ is useful to check computations of the various relationships.

The student in soil mechanics must understand the meanings of the relationships in Fig. 3.1, convince himself once and for all that they are correct, and add these terms to his active vocabulary. These relationships are basic to most computations in soil mechanics and thus are an essential part of soil mechanics.

Typical Values of Phase Relationships for Granular Soils

Figure 3.2 shows two of the many possible ways that a system of equal-sized spheres can be packed. The dense packings represent the densest possible state for such a system. Looser systems than the simple cubic packing can be obtained by carefully constructing arches within the packing, but the simple cubic packing is the loosest of the stable arrangements. The void ratio and porosity of

[3] Chapter 4 discusses the common soil minerals.

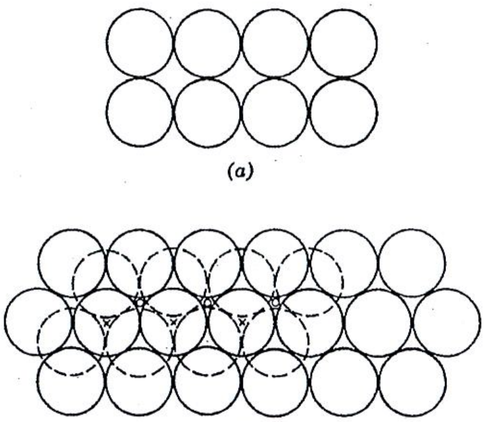


Fig. 3.2 Arrangements of uniform spheres. (*a*) Plan and elevation view: simple cubic packing. (*b*) Plan view: dense packing. Solid circles, first layer; dashed circles, second layer; ∘, location of sphere centers in third layer: face-centered cubic array; ×, location of sphere centers in third layer: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

Table 3.2 Maximum and Minimum Densities for Granular Soils

	Void Ratio		Porosity (%)		Dry Unit Weight (pcf)	
Description	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

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.

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

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

Attachment 4

B.K. Hough, Basic Soils Engineering

B. K. HOUGH, formerly Professor of Civil Engineering at Cornell University and Lehigh University, is presently a consulting engineer with his own consulting firm in Ithaca, N. Y. He has also taught at Massachusetts Institute of Technology. He received his undergraduate and graduate degrees from Massachusetts Institute of Technology. A former student of Professor Terzaghi at M. I. T., he has worked chiefly in soil mechanics ever since, and now has a record of forty years of extensive and varied experience in professional practice, teaching, and research.

BASIC SOILS ENGINEERING

B. K. HOUGH

SECOND EDITION

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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.

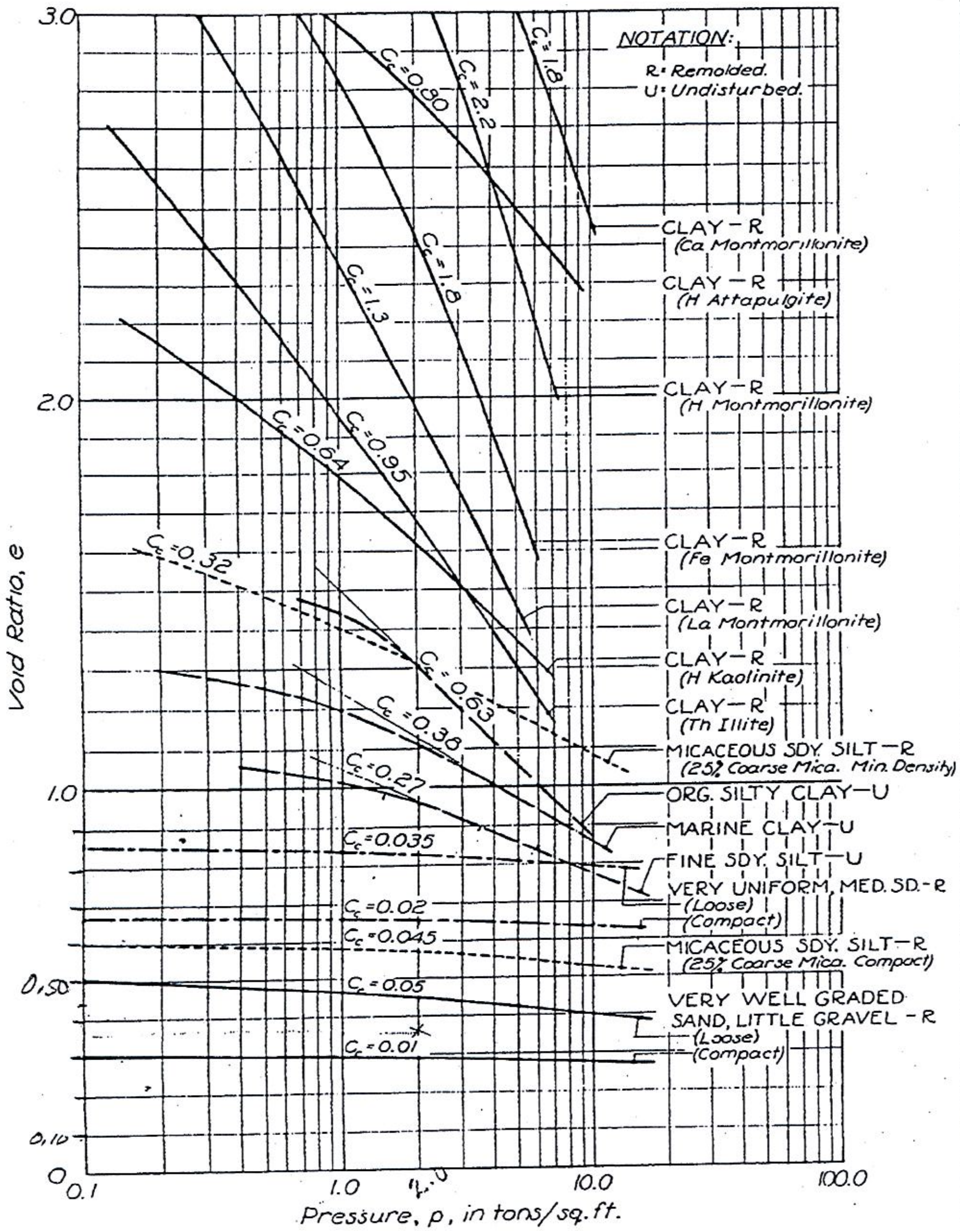


Fig. 5–11. Variation in slope of p-e curves with initial void ratio.

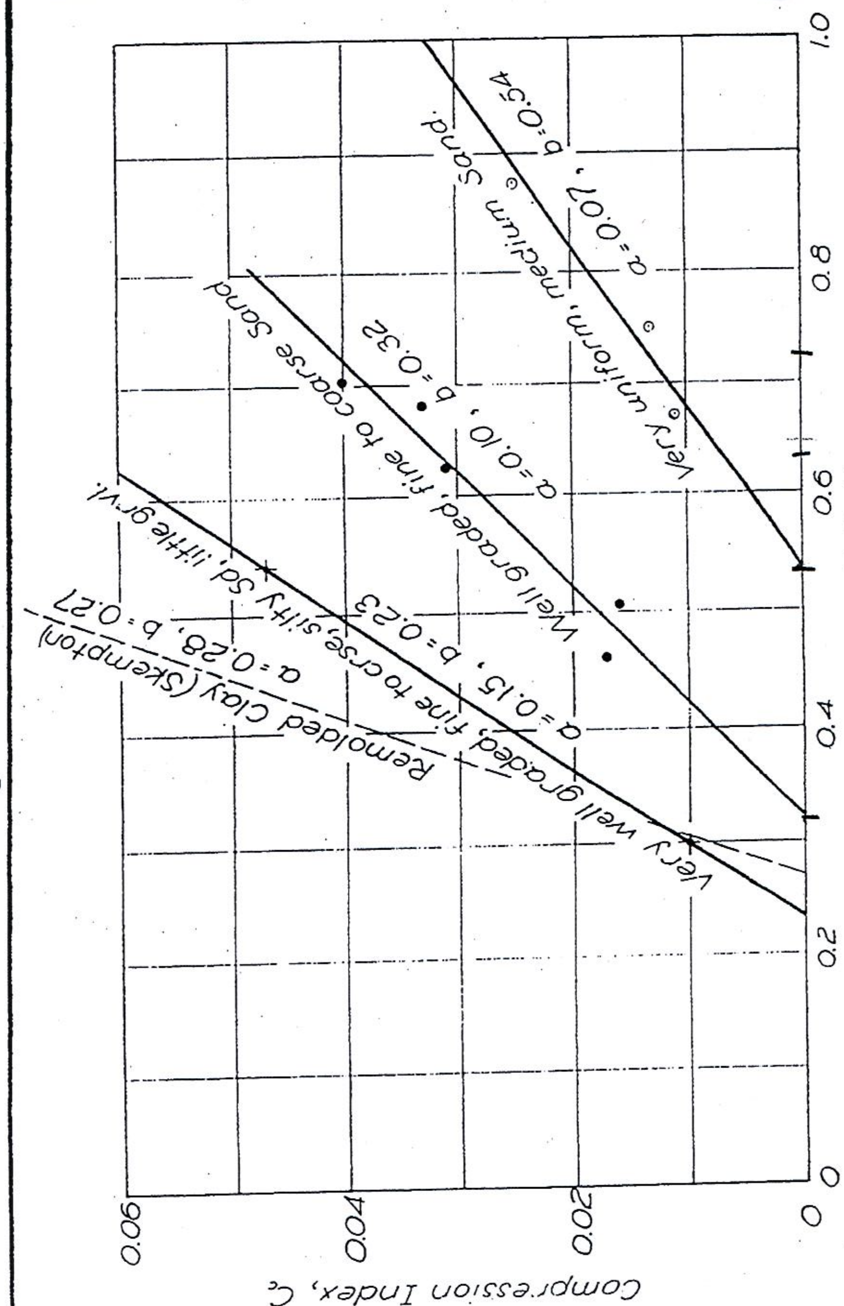


Fig. 12. Variation of C_c with no-load void ratio, e_0.

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	Value of Constant b
Uniform cohesionless material ($C_u \leq 2$)		
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 fine[illegible] 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 [illegible]siderable sums of money to compact both earth fills and natural soil formations, [illegible] to loading.

sion index ...hout 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 ...btain an approximation of the compression index for this material. ... the soil is a clay which is in such a condition that the in-p... ...id ratio and pressure plot at point B, it should be presumed, init...y at least, that it is precompressed and that the field compression diagram

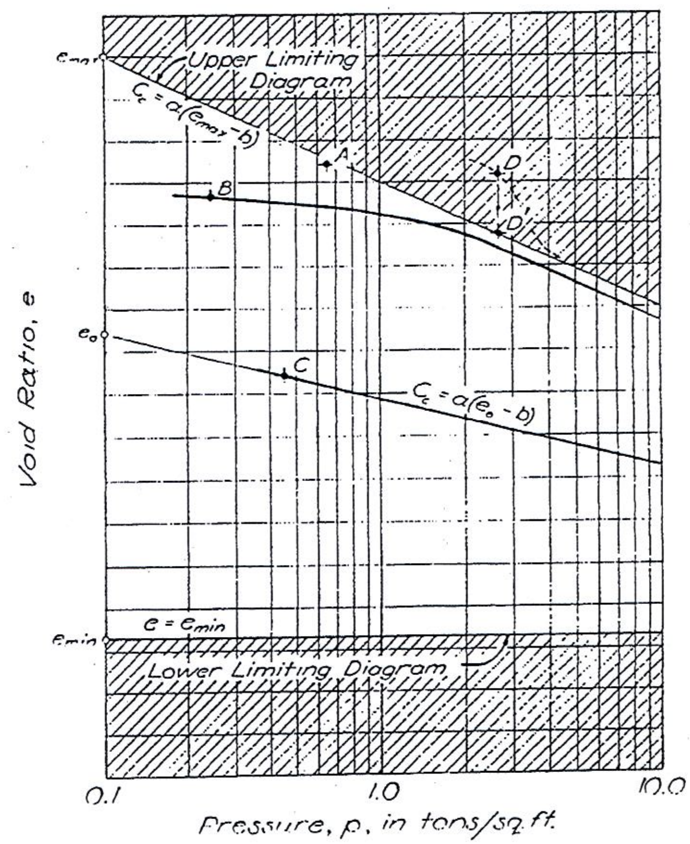


Fig. 5–14. Illustration of procedure for constructing field compression diagrams.

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.

Attachment 5

Principal of Geotechnical Engineering Das (1985)

Principal of Geotechnical Engineering, Das (1985)

The reason for such variation in the e-log p curve is that as time t is increased, the amount of secondary consolidation of the specimen is also increased. This will tend to reduce the void ratio e. Note also that the e-log p curves shown in Figure 8.15 will give slightly different values for the preconsolidation pressure (p_c). The value of p_c will increase with the decrease of t.

The load increment ratio ($\Delta p/p$) also has an influence on the e-log p curves. This was discussed in detail by Leonards and Altschaeffl (1964). Figure 8.16 shows the variation of e with log p for various values of $\Delta p/p$. When $\Delta p/p$ is gradually increased, the e-log p curve gradually moves to the left.

8.7 CALCULATION OF SETTLEMENT FROM ONE-DIMENSIONAL PRIMARY CONSOLIDATION

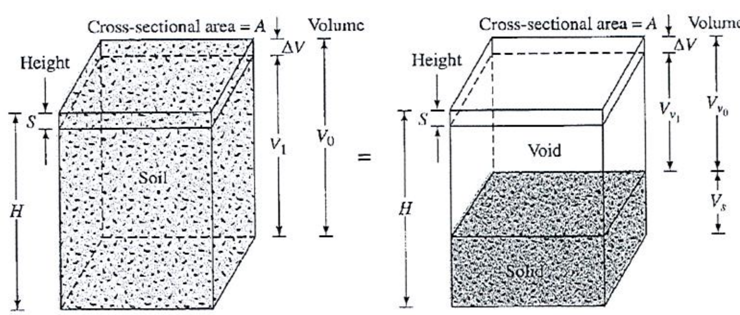
With the knowledge gained from the analysis of consolidation test results, we can now proceed to calculate the probable settlement caused by primary consolidation in the field, assuming one-dimensional consolidation.

Let us consider a saturated clay layer of thickness H and cross-sectional area A under an existing average effective overburden pressure p_O. Because of an increase of pressure, Δp, let the primary settlement be S. Thus, the change in volume (Figure 8.17) can be given by

$$\Delta V = V_0 - V_1 = HA - (H - S)A = SA \tag{8.14}$$

where V_0 and V_1 are the initial and final volumes, respectively. However, the change in the total volume is equal to the change in the volume of voids, ΔV_v. Thus,

$$\Delta V = SA = V_{v0} - V_{v1} = \Delta V_v \tag{8.15}$$



▼ **FIGURE 8.17** Settlement caused by one-dimensional consolidation

where V_{v0} and V_{v1} are the initial and final void volumes, respectively. From the definition of void ratio,

$$\Delta V_v = \Delta e V_s \tag{8.16}$$

where Δe = change of void ratio. But

$$V_s = \frac{V_0}{1 + e_O} = \frac{AH}{1 + e_O} \tag{8.17}$$

where e_O = initial void ratio at volume V_0. Thus, from Eqs. (8.14), (8.15), (8.16), and (8.17),

$$\Delta V = SA = \Delta e V_s = \frac{AH}{1 + e_O} \Delta e$$

or

$$S = H \frac{\Delta e}{1 + e_O} \tag{8.18}$$

For normally consolidated clays that exhibit a linear e-log p (Figure 8.12) relationship,

$$\Delta e = C_c[\log (p_O + \Delta p) - \log p_O] \tag{8.19}$$

where C_c = slope of the e-log p plot and is defined as the compression index. Substitution of Eq. (8.19) in Eq. (8.18) gives

$$\boxed{S = \frac{C_c H}{1 + e_O} \log \left(\frac{p_O + \Delta p}{p_O}\right)} \tag{8.20}$$

For a thicker clay layer, it is more accurate if the layer is divided into a number of sublayers and calculations for settlement are made separately for each sublayer. Thus, the total settlement for the entire layer can be given as

$$S = \sum \left[\frac{C_c H_i}{1 + e_O} \log \left(\frac{p_{O(i)} + \Delta p_{(i)}}{p_{O(i)}}\right)\right]$$

where H_i = thickness of sublayer i
$p_{O(i)}$ = initial average effective overburden pressure for sublayer i
$\Delta p_{(i)}$ = increase of vertical pressure for sublayer i

In overconsolidated clays (Figure 8.13), for $p_O + \Delta p \le p_c$ field e-log p variation will be along the line cb, the slope of which will be approximately equal to that for the laboratory rebound curve. The slope of the rebound curve, C_s, is referred to as the *swell index*, so

$$\Delta e = C_s[\log (p_O + \Delta p) - \log p_O] \tag{8.21}$$

From Eqs. (8.18) and (8.21),

$$\boxed{S = \frac{C_s H}{1 + e_O} \log \left(\frac{p_O + \Delta p}{p_O} \right)}$$

If $p_O + \Delta p > p_c$, then

$$\boxed{S = \frac{C_s H}{1 + e_O} \log \frac{p_c}{p_O} + \frac{C_c H}{1 + e_O} \log \left(\frac{p_O + \Delta p}{p_c} \right)}$$

However, if the e-log p curve is given, it is possible simply to pick Δe (appropriate range of pressures. This figure may be substituted into calculation of settlement, S.

8.8 COMPRESSION INDEX (C_c)

The compression index for the calculation of field settlement causec can be determined by graphic construction (as shown in Figure 8.1 laboratory test results for void ratio and pressure.

Terzaghi and Peck (1967) suggested the following empirical ex pression index:

For undisturbed clays:

$$\boxed{C_c = 0.009(LL - 10)}$$

For remolded clays:

$$C_c = 0.007(LL - 10)$$

where LL = liquid limit, in percent.

In the absence of laboratory consolidation data, Eq. (8.24) is approximate calculation of primary consolidation in the field.

Several other correlations for the compression index are also av have been developed by tests on various clays. Some of these correl Section E.2 (Appendix E).

Attachment 6

T. William Lambe * Robert V. Whitman (1969)

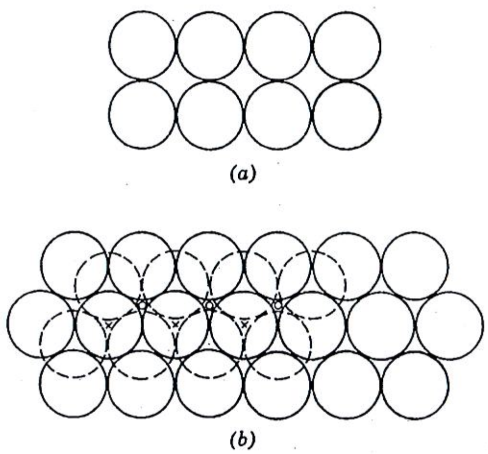


Fig. 3.2 Arrangements of uniform spheres. (*a*) Plan and elevation view: simple cubic packing. (*b*) Plan view: dense packing. Solid circles, first layer; dashed circles, second layer; ∘, location of sphere centers in third layer: face-centered cubic array; ×, location of sphere centers in third layer: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

	Void Ratio		Porosity (%)		Dry Unit Weight (pcf)	
Description	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

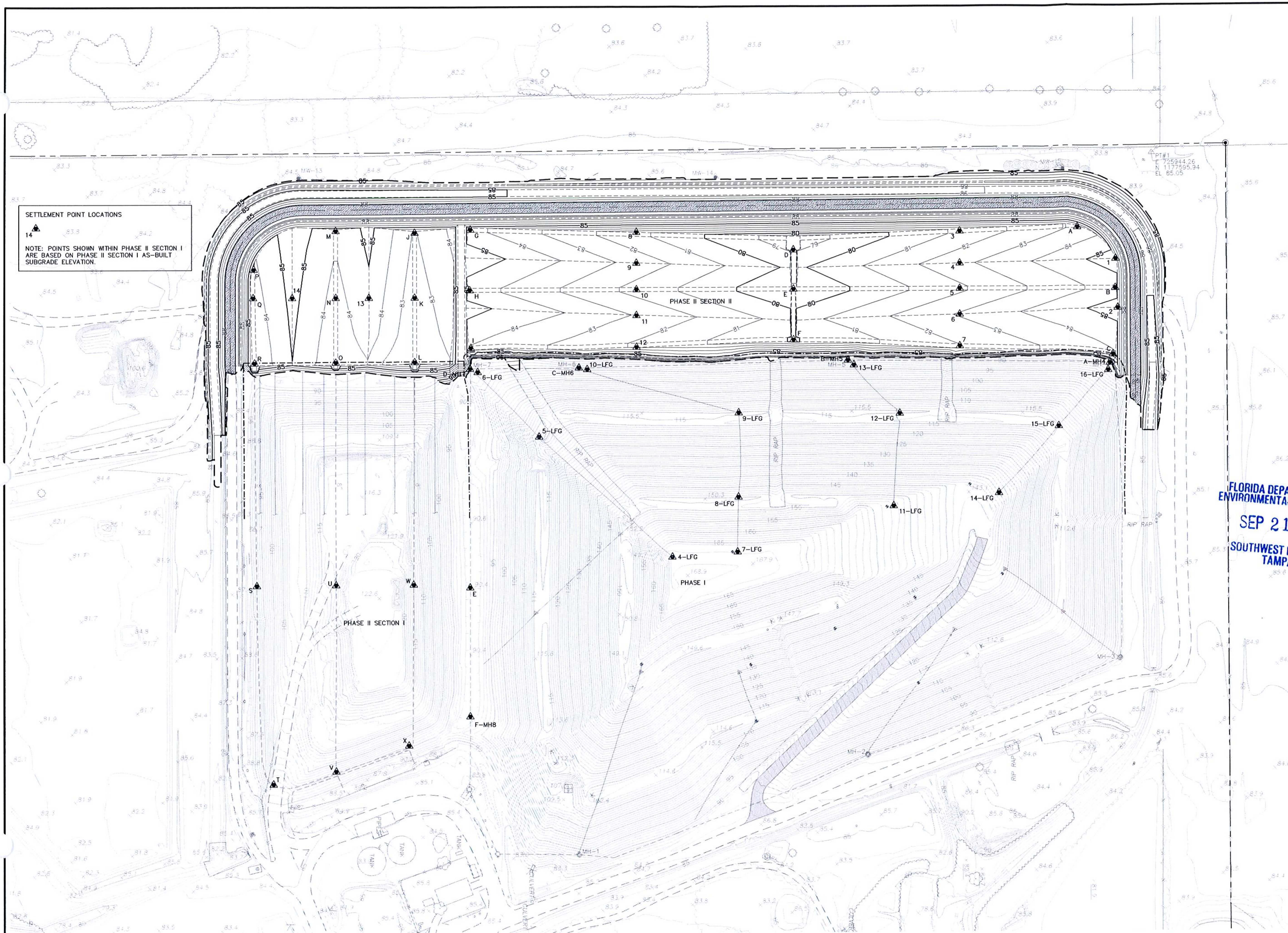
Values of water content for natural granular soils vary from less than 0.1% for air-dry sands to more than 40% for saturated, loose sand.

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

Attachment 7

Settlement Point Locations Drawing



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