



September 23, 2010
Project No.: CPGT-10-0024

TO: **Florida Department of Environmental Protection**
Northeast District Office
7825 Baymeadows Way, Suite 200B
Jacksonville, Florida 32256-7590

ATTN: Mr. Emerson C. Raulerson, P.E.
Solid Waste Section Supervisor

SUBJECT: **Response to First Request for Additional Information**
Seminole Generating Station- Increment 1 Buildout
Putnam County, Florida
WACS I.D. Number: NED/54/94790
Site Certification Number: PA78-10

Dear Mr. Raulerson:

The following is our response to comments from the Florida Department of Environmental Protection regarding the above referenced permit application. The responses provided herein address comments related to soil and groundwater conditions and assumptions for conducting geotechnical investigations of the proposed landfill.

Comment No. 17

The application (e.g., Drawing C-6 and C-9, etc.) indicate that a combination of a geocomposite and a textured HDPE geomembrane liner will be used for liner system construction. However, DEP is concerned with this approach, especially down the side slope of the existing landfill, because textured liners appear to provide very little frictional resistance if in direct contact with geonets. Please address this matter. Also address the various interfaces that will be created along the subject existing landfill slope by the proposed expansion, and demonstrate that sliding will not occur along those various interfaces.

Response

We have reviewed the referenced details of the proposed liner system designed by PBS&J. The proposed liner will consist of a geocomposite consisting of an upper nonwoven geotextile, a lower woven geotextile and a layer of geonet between the two fabrics. The various layers of the

composite are bonded and will therefore act as a single layer. The lower woven fabric will be in contact with the textured HDPE liner. The critical slip surface will be the interface between these two surfaces. Based on site specific test data performed by Ardaman and Associates, the friction angle between the two surfaces is 23.1 degrees for woven geotextiles and 24.7 degrees for double-sided geocomposite. We have conservatively selected 23 degrees for the analysis. Excerpts from the Ardaman report (titled Seminole Electric Cooperative, Inc., CQA Report for SGS Landfill Liner System) are provided in **Attachment A**.

Comment No. 18

The slope stability analysis provided in Section 5 (Attachment C) does not appear to account for the liner system (i.e., textured geomembrane, geocomposite, etc.) that will underlie the waste, nor the liner system that will be installed along the side slope of the existing landfill. Please address, and provide the aforementioned slope stability analysis.

Response

The slope stability analysis has been revised to include the textured geomembrane and geocomposite liner system. As described in Comment 17, the interface between the two surfaces will be the critical failure surface and was assigned the referenced friction angle. The results of the revised stability analyses are presented in **Attachment B** and indicate a minimum factor of safety of 1.5. The results indicate that failure along the slope of the existing liner is not a concern due to the geometry of the landfill build-out. Essentially, this transition slope will be located far back from the limits of the build-out slopes, and will therefore not be of concern. This assumes that the landfill material will be placed uniformly over the build-out area and that large amounts of localized filling against the existing slope will not occur.

Comment No. 19

Please note that the slope stability analysis provided in Section 5 does not appear to analyze the maximum final slopes with a final cover system. Please provide a slope stability analysis of the maximum and critical slopes (and address how the critical slopes were selected) that will account for the proposed final cover system.

Response

A slope stability analysis accounting for the cover system will be conducted as part of a subsequent submittal for approval of the landfill closure plan.

Comment No. 20

Please explain and justify how the applicant determined the friction angles for the various materials that will comprise and underlie the proposed landfill (i.e., weak pozzocrete, weak fly ash, soft clay, etc.) and which are utilized in conducting the slope stability analysis provided in Section 5.

Response

The friction angles utilized in the slope stability analysis were estimated based on the material composition, relative densities, fines content, Atterberg Limits and compressive strength. For the landfill material, it was conservatively assumed that the existing landfill and proposed buildout consists of the densified fly ash layer (Stratum 1D). In reality, significant portions of the existing landfill consist of cemented pozzocrete materials (Strata 1A through 1C) which have higher strength characteristics, including friction angle. The friction angle chosen to represent the existing and proposed landfill material is 32 degrees. This value was chosen based on the referenced McLaren and DiGioia study (1987), which presents a mean value of 34 degrees for Class F fly ash. The value was further reduced to be conservative. Class F fly ash most closely describes the stratum 1D materials at this landfill. For your reference, a copy of the study is provided in **Attachment C**.

For the underlying natural soils, the selected friction angles were based on published and well established empirical data, correlated to fines content, relative density and plasticity. The granular fine sands were found to be in a loose to medium dense condition throughout the property. The friction angle value of 28 degrees assigned for the sands conservatively assumes loose conditions. Recommended empirical values for loose sands range from 28 to 30 degrees. The underlying soft, stiff and consolidated clays were assigned friction angles only for analysis of immediate, or end of construction, stability. For clays, the friction angle is primarily dependent on the relative density and plasticity of the soil. For soft clays in the tested plasticity range (25 to 29%) friction angles of 25 to 30 degrees are recommended by Lambe and Whitman, and a value of 22 was utilized in the analysis. For the stiff clays in the tested plasticity range (15 to 17%), friction angles of 29 to 31 are recommended, and a value of 25 was utilized. For very dense consolidated clays recommended values range from 30 to 35 degrees, and a value of 25 degrees was utilized. Excerpts of the Lambe and Whitman empirical data is provided in **Attachment D**. Friction angle was neglected for the clay soils when conducting the longterm stability analysis.

Comment No. 21

According to Attachment A of Section 5, a friction angle of 28 degrees for the waste fill material was utilized in conducting the bearing capacity analysis. According to page 12 (paragraph 6) and the table provided on page 13 both of Tab Section 5, the fly ash fill material strength parameters are based on recommendations of McLaren and DiGioia (1987), etc. Please justify

that those aforementioned strength parameter recommendations are applicable to the subject Facility's fly ash, or provide laboratory analysis of the Facility's fly ash, etc. and revise the bearing capacity analysis, etc. accordingly.

Response

The friction angle associated with the bearing capacity analysis represents the natural soils below the landfill and not the landfill material itself. The bearing weight of the landfill material on the underlying natural soils is the mechanism analyzed for bearing capacity analysis.

Closure

We appreciate the opportunity to provide our services on this project and trust that this information is sufficient for your needs. Should you have any questions or comments concerning this data, please feel free to contact the undersigned.

ANDREYEV ENGINEERING, INC.

A handwritten signature in black ink, appearing to read "T. Scott Cavin".

T. Scott Cavin, P.E.
Vice President
FL Registration No. 48125

ATTACHMENT A

**Excerpts from Ardaman report
(titled Seminole Electric Cooperative, Inc., CQA Report for SGS
Landfill Liner System)**

3.5 **Manufacturer's Test Data for Geotextile**

The liner system design calls for the gravel surrounding the pipes in the leachate collection and leak detection systems to be wrapped with a geotextile fabric to limit intrusion of the drainage sand into the gravel and pipes. A geotextile is also required under the Fabriform channels. The geotextile provided was a 12-ounce per square yard nonwoven polypropylene product. Table 9 presents mass per unit area, thickness, grab tensile strength, grab elongation, apparent opening size, and permeability test data for this geotextile. The product data sheet in Appendix 2 and the table indicate that this product meets the specification requirements.

3.6 **Other Contractor's Submittals**

In addition to the product data and test results submitted, Comanco provided proposed panel layout plans for the GCL and the primary and secondary HDPE geomembrane liners, certificate of acceptance of liner subgrade, shop drawings for the fabricated HDPE pipe fittings, a shop drawing for the HDPE leachate sump, a rebar plan for the concrete pad under the leachate sump, and concrete mix designs for the concrete pads for the sump and cleanout and the Fabriform channels. The Comanco submittals were reviewed by Ardaman, modified by Comanco in response to our comments if necessary, and recommended for acceptance by SECI. A sample of slotted 6-inch diameter HDPE pipe proposed for construction of the leachate collection and leak detection systems was submitted to Ardaman for approval prior to construction and found to conform to the specifications.

4.0 **LABORATORY CQA CONFORMANCE TESTING**

The laboratory conformance tests performed by Ardaman and the corresponding ASTM test methods and project specification requirements for the components of the bottom liner systems are summarized in Table 10. Results of conformance testing are presented in the following sections.

4.1 **Interface Friction between Geosynthetic Components of Liner System**

The project specifications required all components of the bottom liner system to have a minimum peak interface friction angle of 12°, as determined in accordance with ASTM D5321 or ASTM D6243 (for GCL interfaces). The interfaces between the geosynthetic components placed on the 3H:1V side slopes of the perimeter dike are required to have a peak friction angle of at least 22°. Prior to the start of liner system construction, Ardaman obtained samples of the proposed geosynthetic products from GSE. Direct shear tests were performed for the following material interfaces: between the 60-mil textured HDPE and the GCL, against both the woven and nonwoven geotextile sides; between the 60-mil textured HDPE and the geonet; and between the 60-mil textured HDPE and the double-sided geocomposite. The tests were performed under normal stresses of approximately 4, 8, and 14 psi, under a maximum displacement rate of 0.005 inches per minute.

The laboratory testing results are presented in Table 11. The overall interface friction angles for the geosynthetic interfaces are as follows:

- textured HDPE geomembrane and GCL woven geotextile = 23.1°
- textured HDPE geomembrane and GCL nonwoven geotextile = 31.1°
- textured HDPE geomembrane and geonet = 13.9°
- textured HDPE geomembrane and double-sided geocomposite = 24.7°

These values are consistent with published data and testing performed by Ardaman for other projects. The peak interface friction angles of all tested material interfaces exceeded the minimum required by the project specifications.

4.2 Geosynthetic Clay Liner

The bottom component of the liner system is a GCL, which is a geosynthetic product consisting of a thin layer of bentonite encapsulated between two layers of geotextile. A reinforced, needle-punched GCL product was specified. Comanco selected the GSE BentoLiner NSL product as the GCL component of the double liner system. BentoLiner NSL is a reinforced, needle-punched GCL that has a bentonite layer encapsulated between a woven and a nonwoven geotextile.

The project specifications required sampling and CQA testing of the GCL product at a frequency of one test per 100,000 square feet, which translated to a minimum of seven samples. Conformance testing of the GCL products included bentonite mass per unit area, index flux, hydraulic conductivity, and peel strength.

The ASTM test methods and the results of conformance testing for eight samples of the BentoLiner NSL product are presented in Table 12. As shown in the table, all eight samples met the bentonite mass per unit area and peel strength requirements, and the seven samples tested for index flux and hydraulic conductivity met the requirements stipulated in the project specifications.

4.3 HDPE Geomembrane Liner

The 60-mil textured HDPE geomembrane used on this project was manufactured by GSE. To verify conformance of the textured HDPE geomembrane with the material property requirements stipulated in the project specifications, laboratory CQA tests were performed by Ardaman on samples of the HDPE geomembrane prior to shipments of the liner rolls to the project site. The samples were shipped to Ardaman by GSE in accordance with procedures agreed upon between Ardaman and Comanco.

The conformance tests required as part of the laboratory CQA testing program included determinations of thickness, density, carbon black content, tear resistance, and tensile properties (i.e., strength and elongation at yield, and strength and elongation at break). The project specifications required determination of average and minimum core thickness at a minimum frequency of two tests per acre and performance of other conformance tests at a frequency of one test per acre. For approximately 39 acres of HDPE geomembrane, this frequency translated to 78 samples for thickness and 39 samples for other conformance tests.

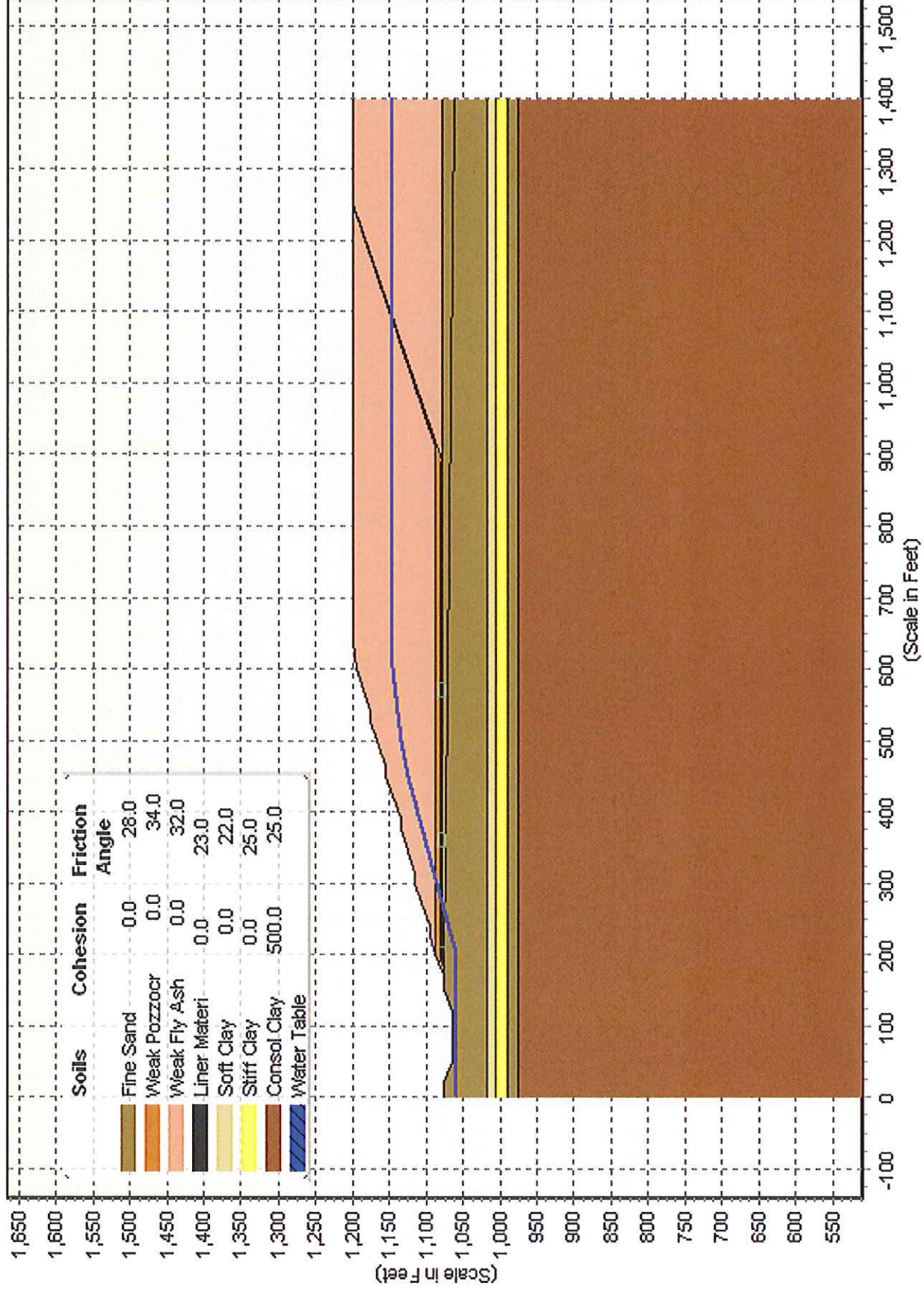
Samples from 78 rolls of textured HDPE geomembrane liner were selected by Ardaman for conformance testing. These samples were designated CT-T-1 through CT-T-78. The ASTM test methods and conformance test results for the HDPE geomembrane liner are summarized in Table 13. The material property requirements stipulated in the project specifications and the liner roll numbers are also shown in the table.

As shown in Table 13, with the exception of Sample Number CT-T-23 from Roll Number 102136802, which failed the minimum elongation at break requirement (≥ 150 percent), all other measured properties of the HDPE liner samples complied with the material property requirements stipulated in the project specifications. Sample Number CT-T-23 had a mean elongation at break of 146 percent in the transverse direction. Retesting of another sample from this roll yielded a mean elongation at break of 423 percent. Therefore, this roll was accepted for construction.

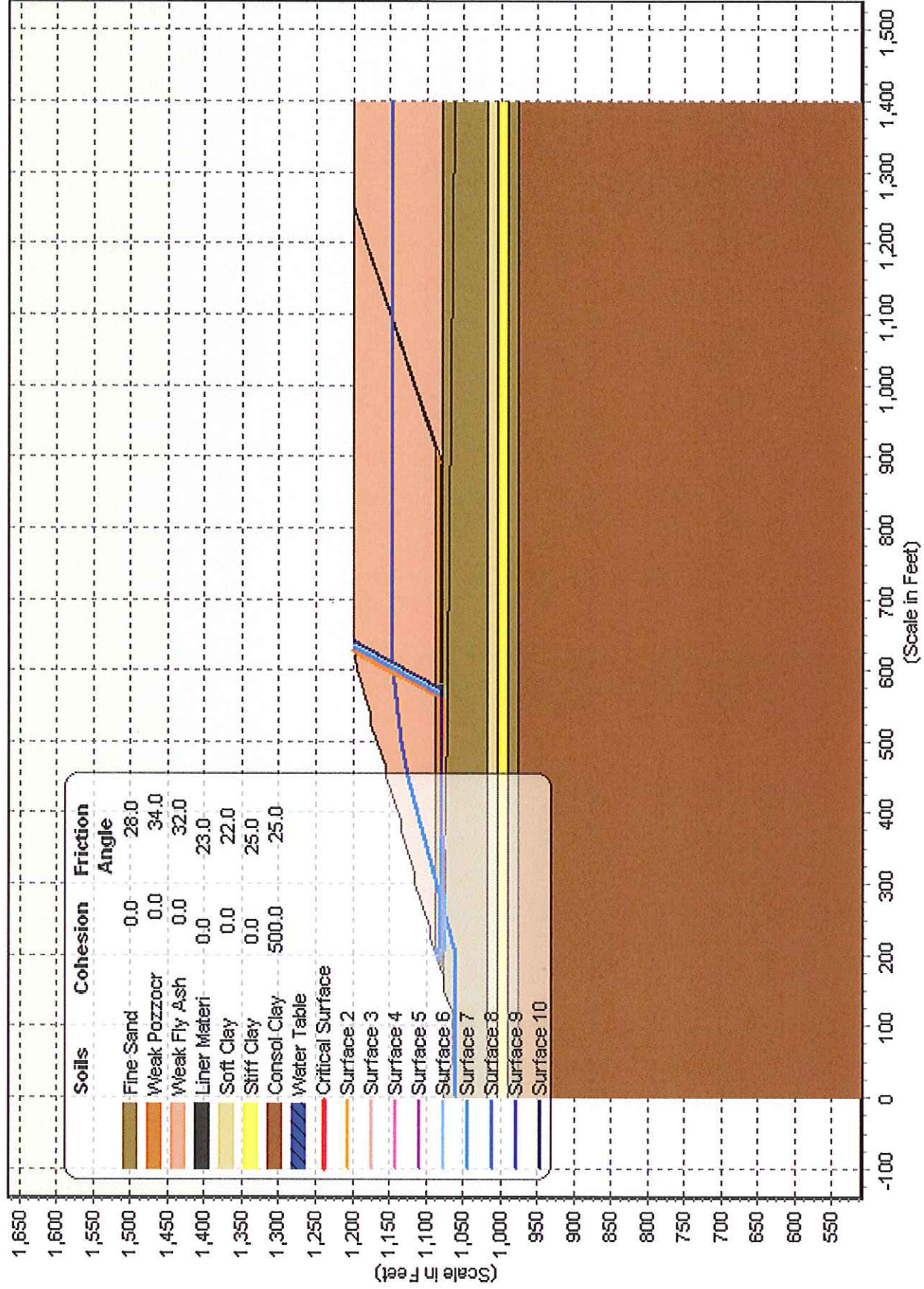
ATTACHMENT B

Slope Stability Analysis Results Revised to Include Bottom and Side Slope Liner

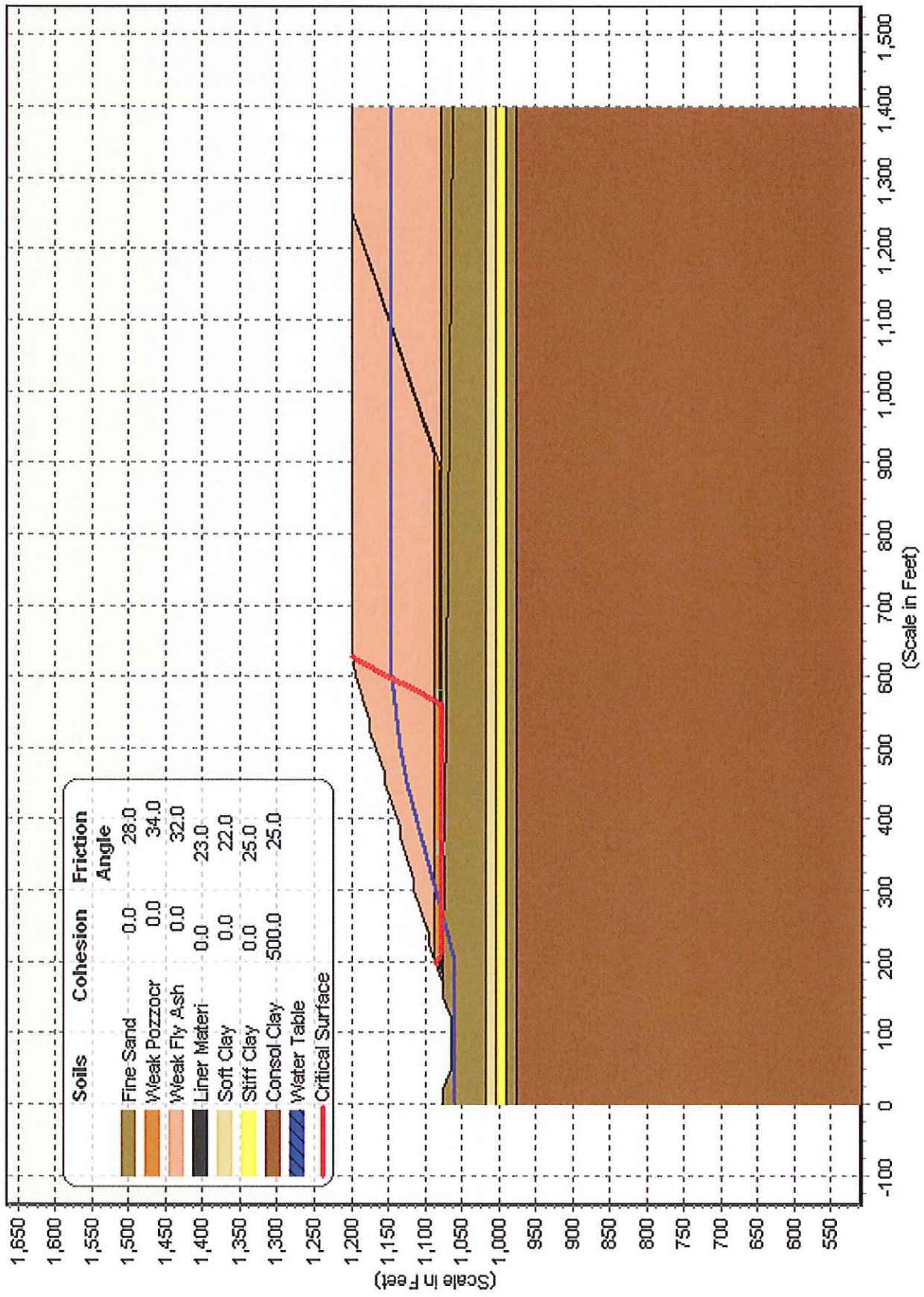
SELE- Section 1 -1 (Liner Interface) (ESSA)



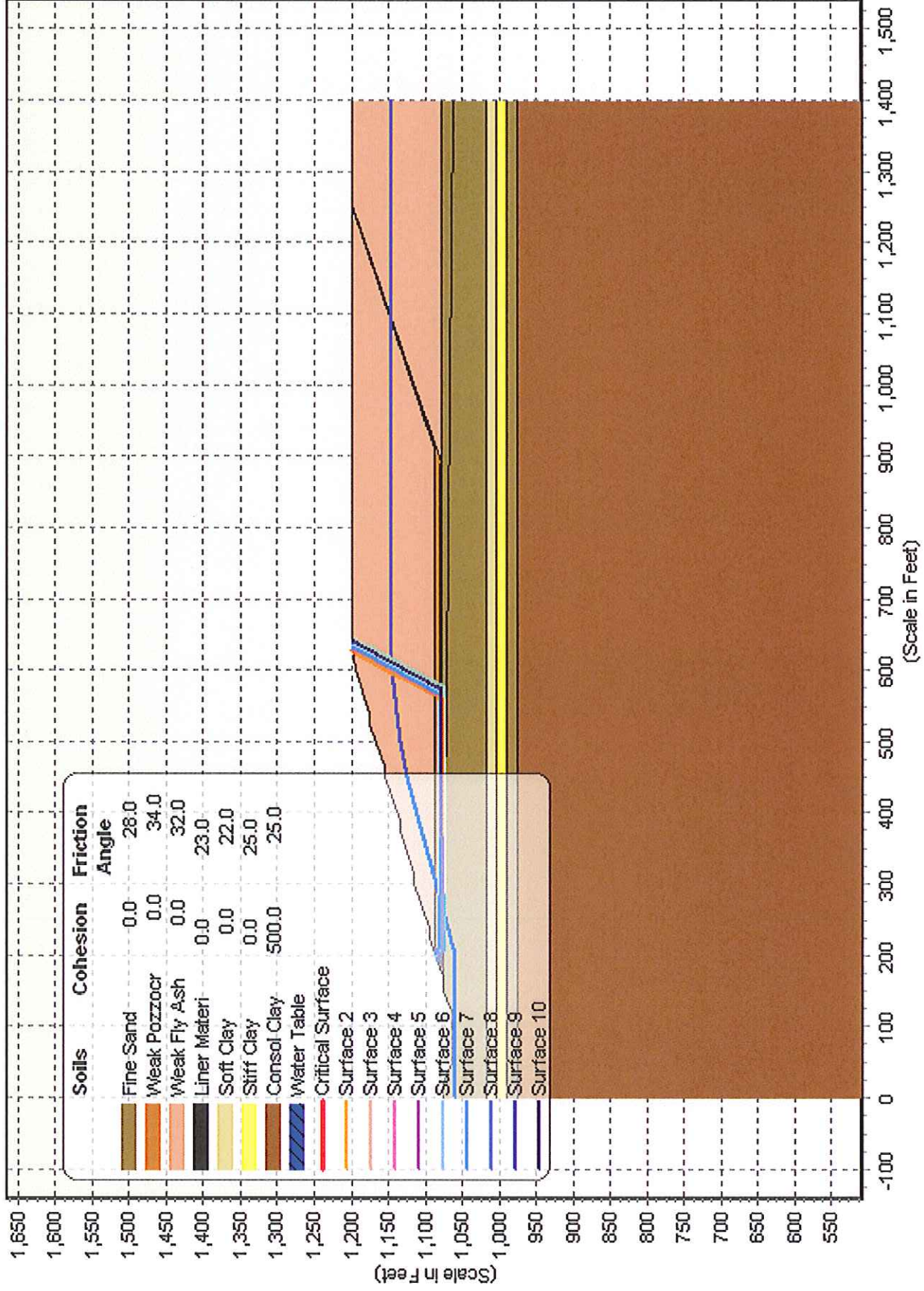
SELE- Section 1 -1 (Liner Interface) (ESSA) - FS Min- Janbu = 1.464



SELE- Section 1 -1 (Liner Interface) (ESSA) - FS Min- Janbu = 1.464



SELE- Section 1 -1 (Liner Interface) (ESSA) - FS Min- Janbu = 1.464



result.out
 ** STABL for WINDOWS **
 by
 Geotechnical Software Solutions

1

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

Run Date: 9/23/10
 Time of Run: 15:00
 Run By: Shawkat Ali
 Input Data Filename: run.in
 Output Filename: result.out
 Unit: U.S.C.
 Plotted Output Filename: result.plt

PROBLEM DESCRIPTION SELE- Section 1 -1 (Liner Interface) (ES
 SA)

BOUNDARY COORDINATES

20 Top Boundaries
 31 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	0.00	1075.00	20.00	1075.00	1
2	20.00	1075.00	50.00	1065.00	1
3	50.00	1065.00	120.00	1065.00	1
4	120.00	1065.00	150.00	1075.00	1
5	150.00	1075.00	165.00	1075.00	1
6	165.00	1075.00	171.00	1077.00	1
7	171.00	1077.00	201.00	1087.00	5
8	201.00	1087.00	225.00	1095.00	6
9	225.00	1095.00	240.00	1095.00	6
10	240.00	1095.00	300.00	1115.00	6
11	300.00	1115.00	315.00	1115.00	6
12	315.00	1115.00	375.00	1135.00	6
13	375.00	1135.00	390.00	1135.00	6
14	390.00	1135.00	450.00	1155.00	6
15	450.00	1155.00	465.00	1155.00	6
16	465.00	1155.00	525.00	1175.00	6
17	525.00	1175.00	540.00	1175.00	6
18	540.00	1175.00	600.00	1195.00	6
19	600.00	1195.00	640.00	1200.00	6
20	640.00	1200.00	1400.00	1200.00	6
21	201.00	1087.00	911.00	1087.00	5
22	911.00	1087.00	1250.00	1200.00	7
23	190.00	1080.00	890.00	1080.00	7
24	185.00	1078.00	890.00	1078.00	1
25	890.00	1078.00	1252.00	1200.00	6
26	890.00	1078.00	1400.00	1078.00	1
27	171.00	1077.00	1400.00	1062.00	1

result.out

28	0.00	1017.00	1400.00	1017.00	2
29	0.00	1005.00	1400.00	1005.00	3
30	0.00	990.00	1400.00	990.00	1
31	0.00	975.00	1400.00	975.00	4

1

ISOTROPIC SOIL PARAMETERS

7 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	110.0	115.0	0.0	28.0	0.00	0.0	1
2	100.0	110.0	0.0	22.0	0.00	0.0	1
3	110.0	120.0	0.0	25.0	0.00	0.0	1
4	115.0	120.0	500.0	25.0	0.00	0.0	1
5	80.0	85.0	0.0	34.0	0.00	0.0	1
6	75.0	80.0	0.0	32.0	0.00	0.0	1
7	110.0	120.0	0.0	23.0	0.00	0.0	1

1

1 PIEZOMETRIC SURFACE(S) HAVE BEEN SPECIFIED

Unit Weight of Water = 62.40

Piezometric Surface No. 1 Specified by 6 Coordinate Points

Point No.	X-Water (ft)	Y-Water (ft)
1	0.00	1060.00
2	200.00	1060.00
3	450.00	1125.00
4	500.00	1135.00
5	600.00	1145.00
6	1400.00	1145.00

1

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

The Active And Passive Portions Of The Sliding Surfaces Are Generated According To The Rankine Theory.

200 Trial Surfaces Have Been Generated.

3 Boxes Specified For Generation Of Central Block Base

result.out

Length Of Line Segments For Active And Passive Portions Of
Sliding Block Is 10.0

Box No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Height (ft)
1	210.00	1079.00	210.00	1079.00	10.00
2	350.00	1079.00	370.00	1079.00	10.00
3	560.00	1079.00	580.00	1079.00	10.00

**** ERROR - BK12 ****

1

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

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

Failure Surface Specified By 21 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	197.16	1085.72
2	199.10	1084.69
3	207.92	1080.00
4	210.00	1078.63
5	369.00	1078.88
6	560.49	1078.61
7	561.41	1080.00
8	565.14	1087.00
9	569.98	1095.75
10	574.83	1104.49
11	579.68	1113.24
12	584.53	1121.98
13	589.38	1130.73
14	594.22	1139.48
15	599.07	1148.22
16	603.92	1156.97
17	608.77	1165.72
18	613.62	1174.46
19	618.46	1183.21
20	623.31	1191.95
21	626.86	1198.36

*** 1.464 ***

Individual data on the 37 slices

Water Water

Earthquake

Slice No.	Width (ft)	Weight (lbs)	result.out						
			Force Top (lbs)	Force Bot (lbs)	Force Norm (lbs)	Force Tan (lbs)	Force Hor (lbs)	Force Ver (lbs)	Surcharge Load (lbs)
1	1.9	129.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
2	1.9	380.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3	6.9	3457.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0
4	2.1	1732.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0
5	15.0	16834.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0
6	15.0	19607.7	0.0	0.0	0.0	0.0	0.0	0.0	0.0
7	32.0	54535.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0
8	4.9	10634.7	0.0	187.9	0.0	0.0	0.0	0.0	0.0
9	23.1	58518.6	0.0	5925.6	0.0	0.0	0.0	0.0	0.0
10	3.8	10917.9	0.0	1796.1	0.0	0.0	0.0	0.0	0.0
11	11.2	31751.4	0.0	6514.7	0.0	0.0	0.0	0.0	0.0
12	54.0	192032.4	0.0	59002.6	0.0	0.0	0.0	0.0	0.0
13	6.0	26057.3	0.0	9377.3	0.0	0.0	0.0	0.0	0.0
14	15.0	66444.4	0.0	25913.8	0.0	0.0	0.0	0.0	0.0
15	60.0	313985.8	0.0	*****	0.0	0.0	0.0	0.0	0.0
16	15.0	90560.5	0.0	43830.4	0.0	0.0	0.0	0.0	0.0
17	35.0	227624.6	0.0	*****	0.0	0.0	0.0	0.0	0.0
18	25.0	182149.6	0.0	89368.0	0.0	0.0	0.0	0.0	0.0
19	15.0	114176.6	0.0	55509.5	0.0	0.0	0.0	0.0	0.0
20	20.5	161488.2	0.0	78130.8	0.0	0.0	0.0	0.0	0.0
21	0.9	7420.4	0.0	6390.8	0.0	0.0	0.0	0.0	0.0
22	3.7	28843.8	0.0	28466.0	0.0	0.0	0.0	0.0	0.0
23	4.8	34962.8	0.0	31283.4	0.0	0.0	0.0	0.0	0.0
24	4.8	32170.9	0.0	26153.6	0.0	0.0	0.0	0.0	0.0
25	4.8	29379.1	0.0	21024.1	0.0	0.0	0.0	0.0	0.0
26	4.8	26587.2	0.0	15894.4	0.0	0.0	0.0	0.0	0.0
27	4.8	23795.4	0.0	10764.9	0.0	0.0	0.0	0.0	0.0
28	4.8	21003.5	0.0	5635.2	0.0	0.0	0.0	0.0	0.0
29	2.9	11236.6	0.0	919.0	0.0	0.0	0.0	0.0	0.0
30	1.9	6991.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0
31	0.9	3186.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0
32	3.9	12292.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0
33	4.8	12526.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0
34	4.8	9566.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0
35	4.8	6606.7	0.0	0.0	0.0	0.0	0.0	0.0	0.0
36	4.8	3646.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0
37	3.5	793.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0

Failure Surface Specified By 20 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	198.49	1086.16
2	201.17	1084.74
3	210.00	1080.05
4	359.74	1078.18
5	573.28	1078.97
6	573.96	1080.00
7	577.68	1087.00
8	582.53	1095.75
9	587.38	1104.49
10	592.22	1113.24
11	597.07	1121.98
12	601.92	1130.73
13	606.77	1139.48
14	611.62	1148.22
15	616.46	1156.97
16	621.31	1165.72

		result.out
17	626.16	1174.46
18	631.01	1183.21
19	635.86	1191.95
20	640.32	1200.00

*** 1.467 ***

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Failure Surface Specified By 22 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	196.40	1085.47
2	197.85	1084.69
3	206.68	1080.00
4	209.70	1078.00
5	210.00	1077.82
6	363.05	1078.86
7	565.37	1078.53
8	566.34	1080.00
9	570.07	1087.00
10	574.91	1095.75
11	579.76	1104.49
12	584.61	1113.24
13	589.46	1121.98
14	594.31	1130.73
15	599.15	1139.48
16	604.00	1148.22
17	608.85	1156.97
18	613.70	1165.72
19	618.55	1174.46
20	623.39	1183.21
21	628.24	1191.95
22	632.16	1199.02

*** 1.475 ***

Failure Surface Specified By 22 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	193.97	1084.66
2	202.73	1080.00
3	205.75	1078.00
4	208.17	1076.55
5	210.00	1075.45
6	362.47	1078.88
7	564.81	1079.71
8	565.00	1080.00
9	568.72	1087.00
10	573.57	1095.75

		result.out
11	578.42	1104.49
12	583.27	1113.24
13	588.12	1121.98
14	592.97	1130.73
15	597.81	1139.48
16	602.66	1148.22
17	607.51	1156.97
18	612.36	1165.72
19	617.21	1174.46
20	622.05	1183.21
21	626.90	1191.95
22	630.72	1198.84

*** 1.527 ***

1

Failure Surface Specified By 23 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	194.39	1084.80
2	194.58	1084.69
3	203.41	1080.00
4	206.43	1078.00
5	208.87	1076.54
6	210.00	1075.86
7	364.76	1078.78
8	564.16	1078.74
9	564.99	1080.00
10	568.71	1087.00
11	573.56	1095.75
12	578.40	1104.49
13	583.25	1113.24
14	588.10	1121.98
15	592.95	1130.73
16	597.80	1139.48
17	602.65	1148.22
18	607.49	1156.97
19	612.34	1165.72
20	617.19	1174.46
21	622.04	1183.21
22	626.89	1191.95
23	630.70	1198.84

*** 1.528 ***

Failure Surface Specified By 22 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	197.02	1085.67

		result.out
2	198.86	1084.69
3	207.69	1080.00
4	210.00	1078.47
5	359.98	1078.66
6	568.93	1077.71
7	569.10	1078.00
8	570.43	1080.00
9	574.15	1087.00
10	579.00	1095.75
11	583.84	1104.49
12	588.69	1113.24
13	593.54	1121.98
14	598.39	1130.73
15	603.24	1139.48
16	608.09	1148.22
17	612.93	1156.97
18	617.78	1165.72
19	622.63	1174.46
20	627.48	1183.21
21	632.33	1191.95
22	636.55	1199.57

*** 1.537 ***

1

Failure Surface Specified By 21 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	197.89	1085.96
2	200.28	1084.69
3	209.11	1080.00
4	210.00	1079.41
5	353.16	1077.94
6	565.66	1078.06
7	566.94	1080.00
8	570.66	1087.00
9	575.51	1095.75
10	580.35	1104.49
11	585.20	1113.24
12	590.05	1121.98
13	594.90	1130.73
14	599.75	1139.48
15	604.60	1148.22
16	609.44	1156.97
17	614.29	1165.72
18	619.14	1174.46
19	623.99	1183.21
20	628.84	1191.95
21	632.80	1199.10

*** 1.540 ***

result.out

Failure Surface Specified By 22 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	192.62	1084.21
2	200.53	1080.00
3	203.55	1078.00
4	205.93	1076.57
5	210.00	1074.13
6	361.08	1078.49
7	575.03	1079.39
8	575.43	1080.00
9	579.15	1087.00
10	584.00	1095.75
11	588.85	1104.49
12	593.70	1113.24
13	598.55	1121.98
14	603.40	1130.73
15	608.24	1139.48
16	613.09	1148.22
17	617.94	1156.97
18	622.79	1165.72
19	627.64	1174.46
20	632.48	1183.21
21	637.33	1191.95
22	641.79	1200.00

*** 1.549 ***

1

Failure Surface Specified By 20 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	201.38	1087.13
2	201.62	1087.00
3	210.00	1082.54
4	360.43	1078.56
5	573.91	1079.64
6	574.15	1080.00
7	577.88	1087.00
8	582.72	1095.75
9	587.57	1104.49
10	592.42	1113.24
11	597.27	1121.98
12	602.12	1130.73
13	606.96	1139.48
14	611.81	1148.22
15	616.66	1156.97
16	621.51	1165.72
17	626.36	1174.46
18	631.20	1183.21
19	636.05	1191.95
20	640.51	1200.00

*** 1.552 ***

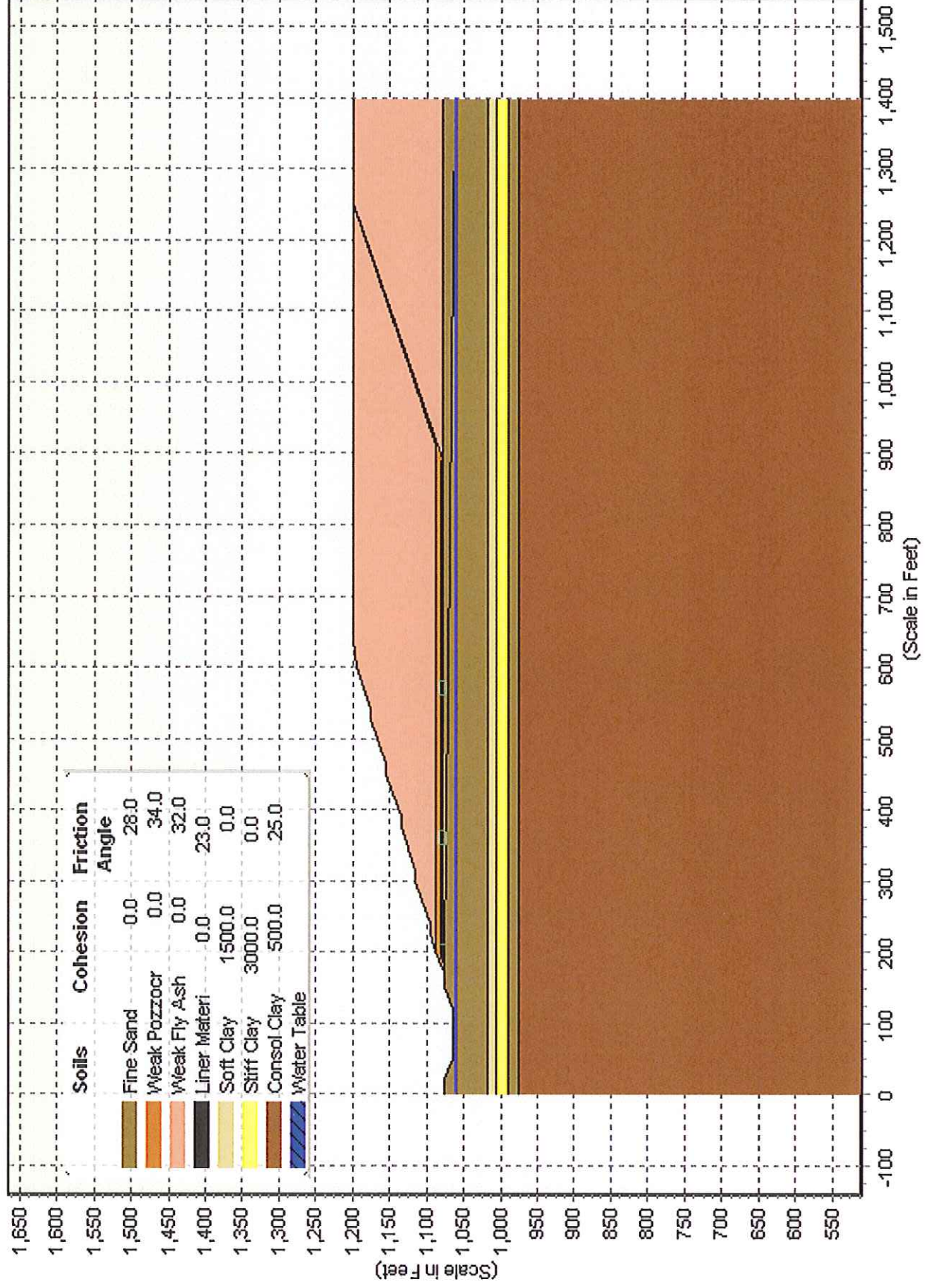
Point No.	X-Surf (ft)	Y-Surf (ft)
1	193.33	1084.44
2	201.69	1080.00
3	204.71	1078.00
4	207.11	1076.56
5	210.00	1074.82
6	366.79	1078.27
7	575.01	1079.53
8	575.32	1080.00
9	579.04	1087.00
10	583.89	1095.75
11	588.74	1104.49
12	593.59	1113.24
13	598.44	1121.98
14	603.28	1130.73
15	608.13	1139.48
16	612.98	1148.22
17	617.83	1156.97
18	622.68	1165.72
19	627.52	1174.46
20	632.37	1183.21
21	637.22	1191.95
22	641.68	1200.00

*** 1.552 ***

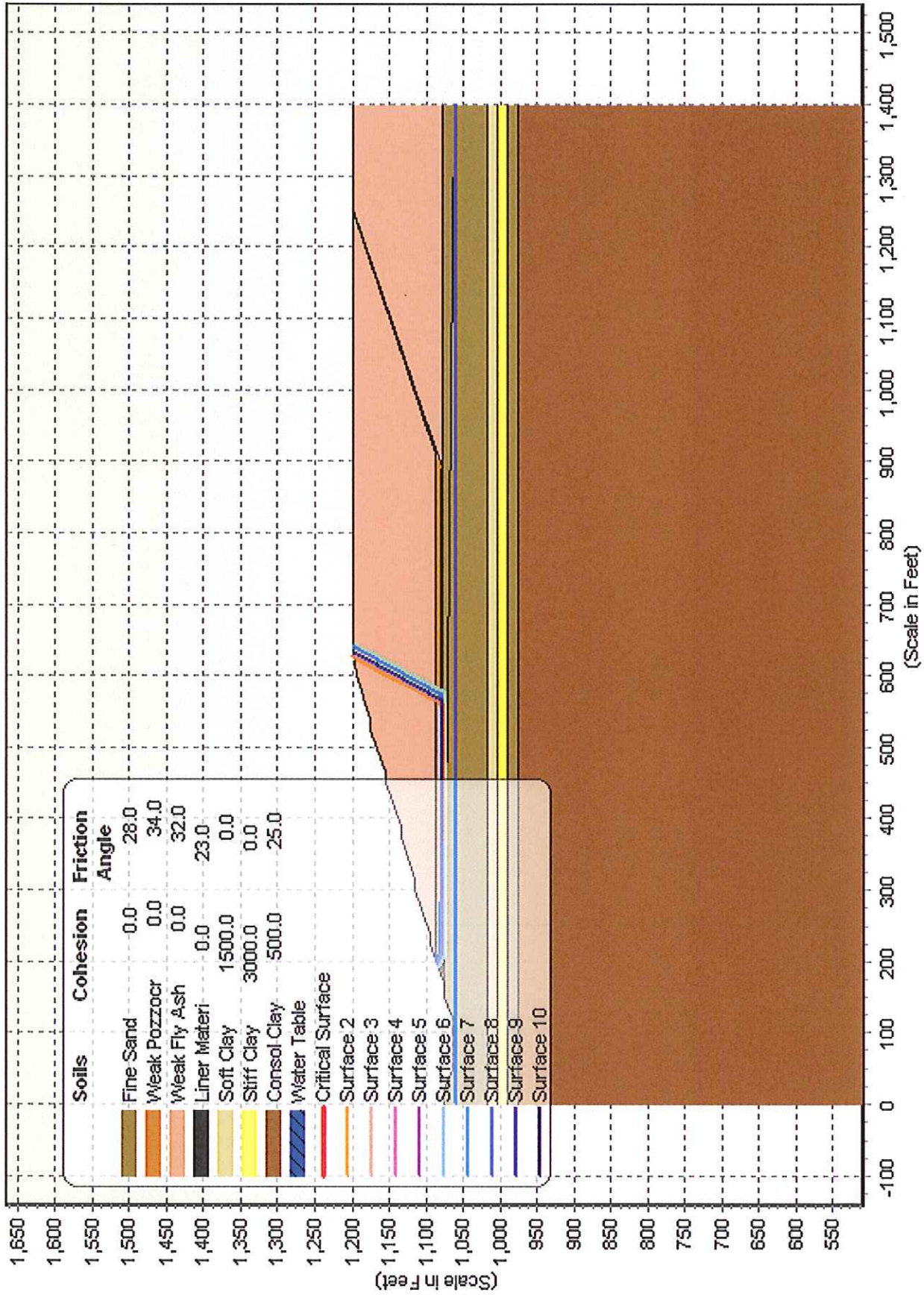
	Y	A	X	I	S	F	T
	0.00	236.50	473.00	709.50	946.00	1182.50	
X	0.00	+	+	+	+	+	+
	-						*
	-						*
	-						**
	-						W*
236.50	+						*
	-						*
	-						.1
	-						*
	-						W*
A	473.00	+					*
	-						W
	-						*
	-						.1
	-						.21w1

		-	result.out		
		-			.21
X	709.50	+			
		-			
		-			
		-			
		-			
I	946.00	+		*	
		-			
		-			
S	1182.50	+			
		-			
		-			
		-			
	1419.00	+		*** ** W	
		-			
		-			
F	1655.50	+			
		-			
		-			
		-			
T	1892.00	+			

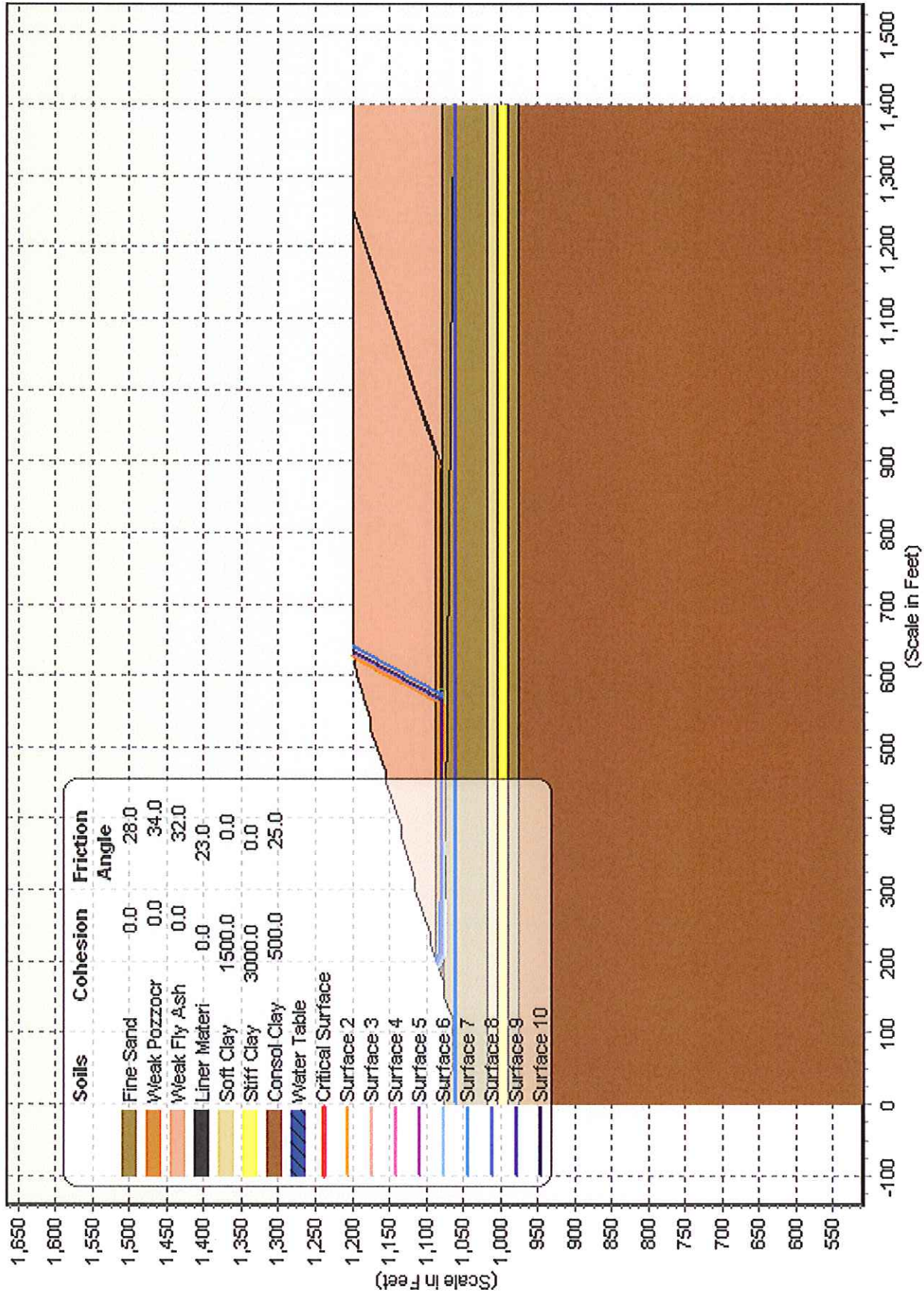
SELE- Section 1 -1 (Liner Interface) (USSA)



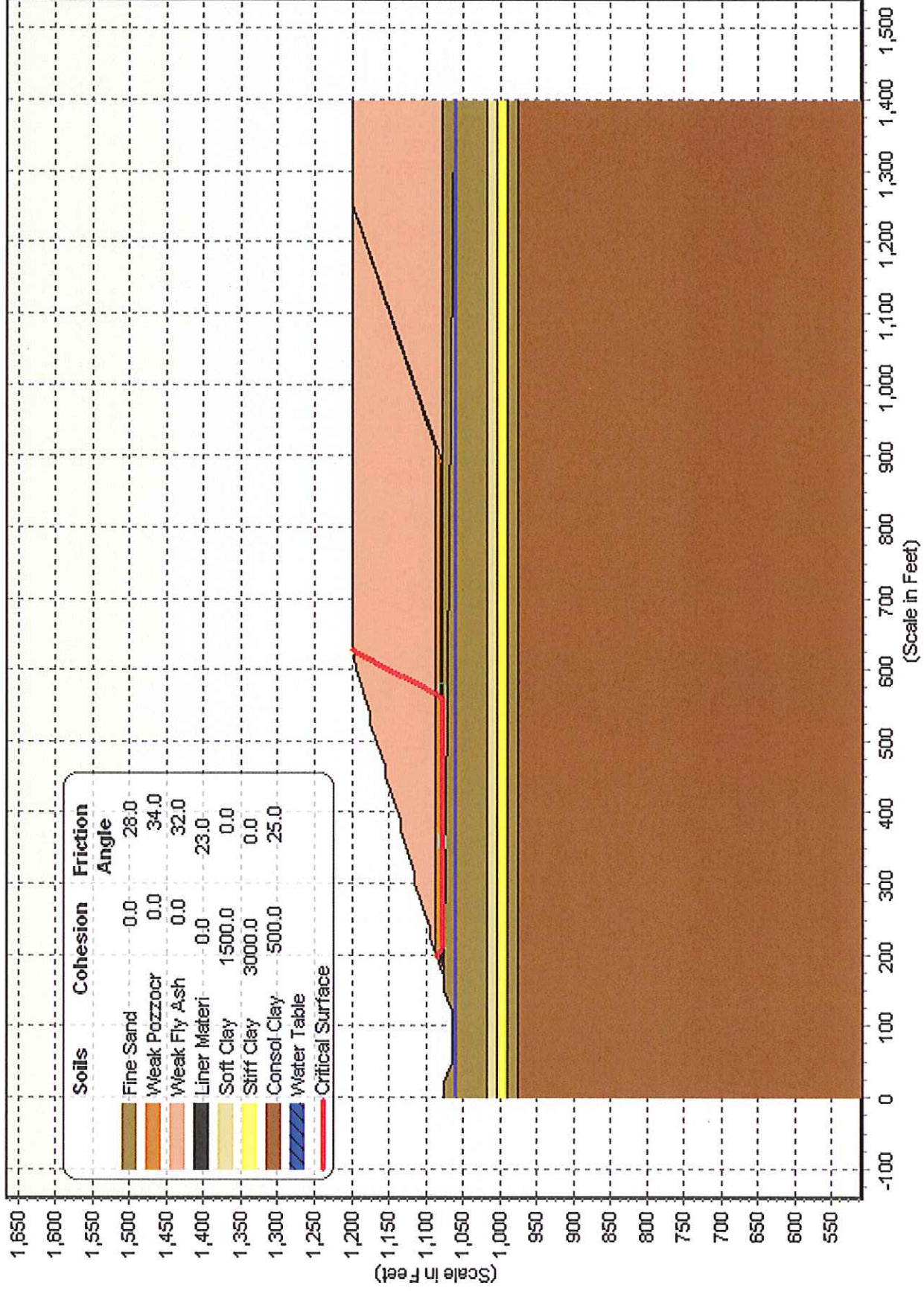
SELE- Section 1 -1 (Liner Interface) (USSA) - FS Min- Janbu = 2.386



SELE- Section 1 -1 (Liner Interface) (USSA) - FS Min- Janbu = 2.386



SELE- Section 1 -1 (Liner Interface) (USSA) - FS Min- Janbu = 2.386



result.out
 ** STABL for WINDOWS **
 by
 Geotechnical Software Solutions

1

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

Run Date: 9/23/10
 Time of Run: 14:00
 Run By: Shawkat Ali
 Input Data Filename: run.in
 Output Filename: result.out
 Unit: U.S.C.
 Plotted Output Filename: result.plt

PROBLEM DESCRIPTION SELE- Section 1 -1 (Liner Interface) (US
 SA)

BOUNDARY COORDINATES

20 Top Boundaries
 31 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	0.00	1075.00	20.00	1075.00	1
2	20.00	1075.00	50.00	1065.00	1
3	50.00	1065.00	120.00	1065.00	1
4	120.00	1065.00	150.00	1075.00	1
5	150.00	1075.00	165.00	1075.00	1
6	165.00	1075.00	171.00	1077.00	1
7	171.00	1077.00	201.00	1087.00	5
8	201.00	1087.00	225.00	1095.00	6
9	225.00	1095.00	240.00	1095.00	6
10	240.00	1095.00	300.00	1115.00	6
11	300.00	1115.00	315.00	1115.00	6
12	315.00	1115.00	375.00	1135.00	6
13	375.00	1135.00	390.00	1135.00	6
14	390.00	1135.00	450.00	1155.00	6
15	450.00	1155.00	465.00	1155.00	6
16	465.00	1155.00	525.00	1175.00	6
17	525.00	1175.00	540.00	1175.00	6
18	540.00	1175.00	600.00	1195.00	6
19	600.00	1195.00	640.00	1200.00	6
20	640.00	1200.00	1400.00	1200.00	6
21	201.00	1087.00	911.00	1087.00	5
22	911.00	1087.00	1250.00	1200.00	7
23	190.00	1080.00	890.00	1080.00	7
24	185.00	1078.00	890.00	1078.00	1
25	890.00	1078.00	1252.00	1200.00	6
26	890.00	1078.00	1400.00	1078.00	1
27	171.00	1077.00	1400.00	1062.00	1

			result.out		
28	0.00	1017.00	1400.00	1017.00	2
29	0.00	1005.00	1400.00	1005.00	3
30	0.00	990.00	1400.00	990.00	1
31	0.00	975.00	1400.00	975.00	4

1

ISOTROPIC SOIL PARAMETERS

7 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	110.0	115.0	0.0	28.0	0.00	0.0	1
2	100.0	110.0	1500.0	0.0	0.00	0.0	1
3	110.0	120.0	3000.0	0.0	0.00	0.0	1
4	115.0	120.0	500.0	25.0	0.00	0.0	1
5	80.0	85.0	0.0	34.0	0.00	0.0	1
6	75.0	80.0	0.0	32.0	0.00	0.0	1
7	110.0	120.0	0.0	23.0	0.00	0.0	1

1

1 PIEZOMETRIC SURFACE(S) HAVE BEEN SPECIFIED

Unit Weight of Water = 62.40

Piezometric Surface No. 1 Specified by 5 Coordinate Points

Point No.	X-Water (ft)	Y-Water (ft)
1	0.00	1060.00
2	300.00	1060.00
3	500.00	1060.00
4	650.00	1060.00
5	1400.00	1060.00

1

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

The Active And Passive Portions Of The Sliding Surfaces Are Generated According To The Rankine Theory.

200 Trial Surfaces Have Been Generated.

3 Boxes Specified For Generation Of Central Block Base

result.out

Length Of Line Segments For Active And Passive Portions Of
Sliding Block Is 10.0

Box No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Height (ft)
1	210.00	1079.00	210.00	1079.00	10.00
2	350.00	1079.00	370.00	1079.00	10.00
3	560.00	1079.00	580.00	1079.00	10.00

**** ERROR - BK12 ****

1

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

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

Failure Surface Specified By 21 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	197.16	1085.72
2	199.10	1084.69
3	207.92	1080.00
4	210.00	1078.63
5	369.00	1078.88
6	560.49	1078.61
7	561.41	1080.00
8	565.14	1087.00
9	569.98	1095.75
10	574.83	1104.49
11	579.68	1113.24
12	584.53	1121.98
13	589.38	1130.73
14	594.22	1139.48
15	599.07	1148.22
16	603.92	1156.97
17	608.77	1165.72
18	613.62	1174.46
19	618.46	1183.21
20	623.31	1191.95
21	626.86	1198.36

*** 2.386 ***

Individual data on the 32 slices

Water Force	Water Force	Force	Force	Earthquake Force	Surcharge
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Page 3

Slice No.	width (ft)	weight (lbs)	result.out						
			Top (lbs)	Bot (lbs)	Norm (lbs)	Tan (lbs)	Hor (lbs)	Ver (lbs)	Load (lbs)
1	1.9	129.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
2	1.9	380.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3	6.9	3457.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0
4	2.1	1732.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0
5	15.0	16834.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0
6	15.0	19607.7	0.0	0.0	0.0	0.0	0.0	0.0	0.0
7	60.0	123039.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0
8	15.0	41911.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0
9	54.0	186990.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0
10	6.0	25272.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0
11	15.0	64282.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0
12	60.0	302474.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0
13	15.0	86955.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
14	60.0	393165.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
15	15.0	109627.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0
16	20.5	155086.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0
17	0.9	7134.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0
18	3.7	27772.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0
19	4.8	33747.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0
20	4.8	31154.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0
21	4.8	28562.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0
22	4.8	25969.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0
23	4.8	23377.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0
24	4.8	20784.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0
25	4.8	18192.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0
26	0.9	3186.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0
27	3.9	12292.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0
28	4.8	12526.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0
29	4.8	9566.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0
30	4.8	6606.7	0.0	0.0	0.0	0.0	0.0	0.0	0.0
31	4.8	3646.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0
32	3.5	793.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0

Failure Surface Specified By 22 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	196.40	1085.47
2	197.85	1084.69
3	206.68	1080.00
4	209.70	1078.00
5	210.00	1077.82
6	363.05	1078.86
7	565.37	1078.53
8	566.34	1080.00
9	570.07	1087.00
10	574.91	1095.75
11	579.76	1104.49
12	584.61	1113.24
13	589.46	1121.98
14	594.31	1130.73
15	599.15	1139.48
16	604.00	1148.22
17	608.85	1156.97
18	613.70	1165.72
19	618.55	1174.46
20	623.39	1183.21
21	628.24	1191.95
22	632.16	1199.02

result.out

*** 2.399 ***

1

Failure Surface Specified By 20 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	198.49	1086.16
2	201.17	1084.74
3	210.00	1080.05
4	359.74	1078.18
5	573.28	1078.97
6	573.96	1080.00
7	577.68	1087.00
8	582.53	1095.75
9	587.38	1104.49
10	592.22	1113.24
11	597.07	1121.98
12	601.92	1130.73
13	606.77	1139.48
14	611.62	1148.22
15	616.46	1156.97
16	621.31	1165.72
17	626.16	1174.46
18	631.01	1183.21
19	635.86	1191.95
20	640.32	1200.00

*** 2.400 ***

Failure Surface Specified By 22 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	193.97	1084.66
2	202.73	1080.00
3	205.75	1078.00
4	208.17	1076.55
5	210.00	1075.45
6	362.47	1078.88
7	564.81	1079.71
8	565.00	1080.00
9	568.72	1087.00
10	573.57	1095.75
11	578.42	1104.49
12	583.27	1113.24
13	588.12	1121.98
14	592.97	1130.73
15	597.81	1139.48
16	602.66	1148.22

		result.out
17	607.51	1156.97
18	612.36	1165.72
19	617.21	1174.46
20	622.05	1183.21
21	626.90	1191.95
22	630.72	1198.84

*** 2.442 ***

1

Failure Surface Specified By 23 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	194.39	1084.80
2	194.58	1084.69
3	203.41	1080.00
4	206.43	1078.00
5	208.87	1076.54
6	210.00	1075.86
7	364.76	1078.78
8	564.16	1078.74
9	564.99	1080.00
10	568.71	1087.00
11	573.56	1095.75
12	578.40	1104.49
13	583.25	1113.24
14	588.10	1121.98
15	592.95	1130.73
16	597.80	1139.48
17	602.65	1148.22
18	607.49	1156.97
19	612.34	1165.72
20	617.19	1174.46
21	622.04	1183.21
22	626.89	1191.95
23	630.70	1198.84

*** 2.450 ***

Failure Surface Specified By 22 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	192.62	1084.21
2	200.53	1080.00
3	203.55	1078.00
4	205.93	1076.57
5	210.00	1074.13
6	361.08	1078.49
7	575.03	1079.39

		result.out
8	575.43	1080.00
9	579.15	1087.00
10	584.00	1095.75
11	588.85	1104.49
12	593.70	1113.24
13	598.55	1121.98
14	603.40	1130.73
15	608.24	1139.48
16	613.09	1148.22
17	617.94	1156.97
18	622.79	1165.72
19	627.64	1174.46
20	632.48	1183.21
21	637.33	1191.95
22	641.79	1200.00

*** 2.477 ***

1

Failure Surface Specified By 22 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	193.33	1084.44
2	201.69	1080.00
3	204.71	1078.00
4	207.11	1076.56
5	210.00	1074.82
6	366.79	1078.27
7	575.01	1079.53
8	575.32	1080.00
9	579.04	1087.00
10	583.89	1095.75
11	588.74	1104.49
12	593.59	1113.24
13	598.44	1121.98
14	603.28	1130.73
15	608.13	1139.48
16	612.98	1148.22
17	617.83	1156.97
18	622.68	1165.72
19	627.52	1174.46
20	632.37	1183.21
21	637.22	1191.95
22	641.68	1200.00

*** 2.483 ***

Failure Surface Specified By 20 Coordinate Points

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

result.out

1	201.38	1087.13
2	201.62	1087.00
3	210.00	1082.54
4	360.43	1078.56
5	573.91	1079.64
6	574.15	1080.00
7	577.88	1087.00
8	582.72	1095.75
9	587.57	1104.49
10	592.42	1113.24
11	597.27	1121.98
12	602.12	1130.73
13	606.96	1139.48
14	611.81	1148.22
15	616.66	1156.97
16	621.51	1165.72
17	626.36	1174.46
18	631.20	1183.21
19	636.05	1191.95
20	640.51	1200.00

*** 2.486 ***

1

Failure surface Specified By 20 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	201.57	1087.19
2	201.92	1087.00
3	210.00	1082.70
4	358.34	1079.14
5	567.30	1079.60
6	567.57	1080.00
7	571.29	1087.00
8	576.14	1095.75
9	580.98	1104.49
10	585.83	1113.24
11	590.68	1121.98
12	595.53	1130.73
13	600.38	1139.48
14	605.23	1148.22
15	610.07	1156.97
16	614.92	1165.72
17	619.77	1174.46
18	624.62	1183.21
19	629.47	1191.95
20	633.47	1199.18

*** 2.506 ***

Failure surface Specified By 21 Coordinate Points
Page 8

```
result.out
```

Point No.	X-Surf (ft)	Y-Surf (ft)
1	197.89	1085.96
2	200.28	1084.69
3	209.11	1080.00
4	210.00	1079.41
5	353.16	1077.94
6	565.66	1078.06
7	566.94	1080.00
8	570.66	1087.00
9	575.51	1095.75
10	580.35	1104.49
11	585.20	1113.24
12	590.05	1121.98
13	594.90	1130.73
14	599.75	1139.48
15	604.60	1148.22
16	609.44	1156.97
17	614.29	1165.72
18	619.14	1174.46
19	623.99	1183.21
20	628.84	1191.95
21	632.80	1199.10

*** 2.517 ***

1

	Y	A	X	I	S	F	T
	0.00	236.50	473.00	709.50	946.00	1182.50	
X	0.00	+	+	+	+	***	*
		-	-	-	-	-	-
		-	-	-	-	-	-
		-	-	-	-	*	*
		-	-	-	-	**	
	236.50	+				4*	*
		-	-	-	-	-	-
		-	-	-	-	W	*
		-	-	-	-	.1	*
		-	-	-	-		*
A	473.00	+					*
		-	-	-	-	W	*
		-	-	-	-	.1	*
		-	-	-	-	.2111	
		-	-	-	-	W	.21
X	709.50	+					
		-	-	-	-	-	-
		-	-	-	-	-	-
		-	-	-	-	*	

result.out

I	946.00	+
		-
		-
		-
		-
S	1182.50	+
		-
		-
		-
		-
	1419.00	+
		-
		-
		-
		-
F	1655.50	+
		-
		-
		-
		-
T	1892.00	+

*** **

ATTACHMENT C

McLaren and DiGioia Study (1987),

GEOTECHNICAL PRACTICE FOR WASTE DISPOSAL '87

Proceedings of a Specialty Conference
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THE TYPICAL ENGINEERING PROPERTIES OF FLY ASH

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Abstract: Knowledge of key physical and engineering properties of coal fly ash is needed in the planning, design, and construction of both disposal and utilization projects. While considerable data concerning the properties of fly ash has been generated, it is not easily accessible. Engineers working with fly ash are faced with not having information on what values and range of values are typical for physical and engineering properties.

To provide a better understanding of the typical values and range of values for key fly ash physical and engineering properties, a data base of values was compiled from published and unpublished sources. Data on 131 Class F fly ash samples and 26 Class C fly ash samples were included. The data base was analyzed to determine typical values and the range of values for key physical and engineering parameters.

This paper presents the typical values. In some cases the typical values appear unusual. These anomalies are noted but are not explained. Also included is a review of the published literature used in compiling the data base. The values presented are useful for conceptual design and planning of projects using fly ash. The ranges of typical values are useful in screening other fly ash data to determine whether the values are likely to be unusual.

Introduction

Fly ash is the unburned residue resulting from the combustion of coal in utility and industrial boilers. It is composed of the noncombustible mineral matter in the coal and of any carbon remaining due to incomplete combustion. Generally, about 80 percent of the uncombusted material is carried from the boiler by stack gases as fly ash and is collected by the air pollution control equipment. Fly ash is a fine, silt-size material consisting largely of spherical, sometimes hollow, glassy particles.

In 1985 approximately 50 million tons of fly ash were produced by the electric utility industry in the United States. An additional

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5 to 10 million tons are estimated to have been produced by commercial and industrial boilers. Coal fly ash is the fourth largest mineral produced in the country.

About 20 percent of the fly ash produced is used as a construction material. Fly ash is a pozzolan and a considerable amount is used as a concrete additive. It is also used in combination with lime, portland cement and aggregate as road base, grout, and as a backfill. Fly ash is also useful as a soil amendment for land reclamation and for subgrade stabilization. Large volumes have also been used as structural fill.

In spite of its useful properties, over 70 percent of the fly ash produced in the United States in 1985 was placed in storage or disposal sites. It is estimated that over a billion tons have been stockpiled. Stockpiled fly ash is deposited in one of two ways. Fly ash is often sluiced to ponds where it is allowed to settle. This method of handling is termed wet disposal. Wet disposal has been the most common disposal method in the past. However, due to environmental concerns and other factors, wet disposal is becoming less common.

The alternate method of handling fly ash is to add sufficient water to moisten it so that it can be handled like soil. This procedure is termed conditioning. Placement of conditioned fly ash as fill in stockpiles is referred to as dry disposal.

There are two general types of fly ash. Class F fly ash is produced by burning bituminous or anthracite coal. Class C fly ash is produced by burning subbituminous coal or lignite. The American Society for Testing and Materials has standards for Class F and C fly ash when it is used in concrete. However, the terms Class C and F are also generally applied in describing fly ash based on the type of coal, regardless of whether the ash meets ASTM standards for use in concrete. A significant amount of fly ash does not comply with the standards for use in concrete.

Class F fly ash is the most common. It has pozzolanic properties but is not appreciably self-cementing. Because of the geographic distribution of coal types, Class F fly ash is principally produced in the eastern United States.

Although less common, the production of Class C fly ash has increased substantially in recent years along with the use of subbituminous coal and lignite. Most Class C fly ash is produced in the western United States. Class C fly ash is self-cementing due to the presence of lime or other chemical compounds.

The handling, use, and disposal of large volumes of fly ash are formidable engineering problems. This paper focuses on those physical and engineering properties of particular interest to engineers dealing with these problems. Knowledge of the chemical properties of fly ash is also of interest to those concerned with the environmental effects of the handling and disposal of fly ash. However, this paper does not

discuss the chemical properties of fly ash except as they relate to physical and engineering properties.

State of Knowledge

Although fly ash has been produced in large quantities for decades, the amount of published and unpublished information available on its properties is relatively small. Much of the work that has been done with fly ash has been limited to a single fly ash source for a specific project. As such, the information developed is not of general interest and is seldom published. Other studies dealing with fly ash from several sources are of general interest and have been published. However, due to budget and time constraints most of these studies are limited to fly ash from only a few sources. In either case, most studies deal with only those properties pertinent to the specific scope of a particular program. Thus, the data that is available often is incomplete or limited.

In many other cases data is available only on fly ash that has been mixed with other materials for utilization or disposal. These materials include portland cement, lime, soil, aggregate and FGD sludge. In many of these cases data on the ash alone was not determined or presented. Other data is available on fly ash from British or other foreign sources. Because of differences in geology and testing procedures, the direct applicability of this data to U.S. cases is sometimes questioned.

All of these factors contribute to the fact that the available information on the properties of fly ash is limited. This situation makes both conceptual design and evaluation of test results difficult. In many cases design engineering with fly ash proceeds as though the fly ash were a totally new material rather than one that is very common.

Published Data

Over a thousand articles, papers and reports have been published in the last 20 years dealing with one aspect or another of fly ash handling, utilization or disposal. While most of these works do not contain significant data on the physical and engineering properties of fly ash, there are some notable exceptions.

One study collected and tested fly ash samples from all of the major power plants in Kentucky (Rose et al. 1982). A total of 25 samples from 18 separate plants were included in the study. Grain size analyses, specific gravity and compaction characteristics were determined in order to inventory ash properties for potential by-product utilization.

A total of 13 separate sources in Pennsylvania, Ohio, West Virginia and other states were sampled and tested as part of a study of using fly ash to stabilize coal refuse for use as roadbase (Anderson et al 1982). Samples were tested for grain size, specific gravity, compaction characteristics and other properties.

Samples of 5 Michigan fly ashes were tested for compaction characteristics, permeability, shear strength and compressibility (Gray and Lin 1972). Two Illinois fly ashes were tested for compaction characteristics, compressibility, shear strength and grain size in a study of an ash disposal pond (Cunningham et al 1977). Data on four fly ashes from two Wisconsin power plants were collected and published in another study (Sikes 1973).

In a study of stabilized fly ash road base for the National Ash Association, the compaction behavior of 6 fly ash samples from the eastern U.S. were determined (GAI Consultants 1976). Another study sampled and tested 10 separate sources of Class C fly ash (Bahor et al. 1981). Specific gravity, grain size distributions, permeabilities, shear strengths and compaction characteristics were determined. The purpose of this study was to establish a limited data base of Class C fly ash properties. The physical and engineering properties of several other Class C fly ashes were determined in conjunction with utilization projects (Joshi 1978). Class C fly ashes from both the western U.S. and Canada were included in this work.

Other data on physical and engineering properties of interest for a limited number of samples were published by several investigators. Appendix I contains 12 other references in addition to those cited above.

Data Base

Using data from the above cited published sources, a data base was compiled with data from 86 separate samples. The data base was expanded by including additional data that was unpublished. This data was obtained from the files of utilities and consultants. An additional 71 samples were added in this way. The total data base included data on 157 samples from 98 separate power plants in the U.S. and Canada. The samples were divided into two groups. Those which had only minor amounts of lime and had no significant hardening properties were classified as Class F. The remaining samples which were obtained from plants burning subbituminous coal or lignite, or which had self-hardening properties, were classified as Class C.

A total of 131 samples representing 74 different plants were included in the data base for Class F fly ash. The sources were primarily from the eastern and midwestern U.S. Fewer samples were available for Class C fly ash. A total of 26 samples were included from 24 separate sources.

The data base was analyzed to determine the distribution of values for key parameters of interest to geotechnical engineers. The parameters evaluated included specific gravity, grain size, compaction characteristics, permeability, shear strength and compressibility. The distributions, the means, and the standard deviations were determined for these parameters. The coefficient of variation, the standard deviation as a percentage of the mean, was also determined. Other characteristics of particular interest were also noted.

Specific Gravity

Specific gravity is one parameter that is frequently determined in analyzing fly ash for chemical properties and for use as an additive in concrete. It was therefore commonly reported for samples included in the data base. It is also of interest in understanding the engineering properties of fly ash.

A total of 98 samples of Class F ash had specific gravity values available. The mean value was 2.40 and the distribution of values is shown in Figure 1. The coefficient of variation was 6.0% and the standard deviation 0.15. For Class C fly ash, the specific gravity of 17 samples were available. The mean value was 2.5, the coefficient of variation 6.8% and the standard deviation was 0.17. The specific gravity of fly ash is less than most soils, which range from about 2.6 to 2.8. Therefore fly ash fills tend to be less dense than those constructed of natural soils. This reduction in density can be an advantage in some applications where fly ash is used as a structural fill.

Grain Size Distribution

The grain size distribution of fly ash provides considerable insight into its properties and behavior. Fly ash is typically a relatively uniform, silt-sized material. Grain size data were available for 98 samples of Class F fly ash. The mean coefficient of uniformity of these samples was 5.94. The standard deviation was 3.6, and the coefficient of variation 60.8%. The distribution of values is shown on Figure 2.

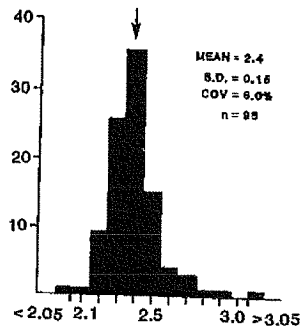


FIG. 1 - Distribution of Values for Specific Gravity of Class F Fly Ash

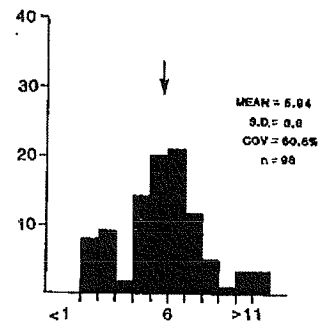


FIG. 2 - Distribution of Values for Coefficient of Uniformity of Class F Fly Ash

Figure 3 shows the distribution of typical values for the grain size of Class F fly ash. The D85, D50 and D15 size distributions are shown. The mean value, and the range of values plus or minus one standard deviation are also shown. There were 84 samples available for each of these values. Table 1 presents the mean, standard deviation and coefficient of variation for the D85, D50 and D15 particle sizes.

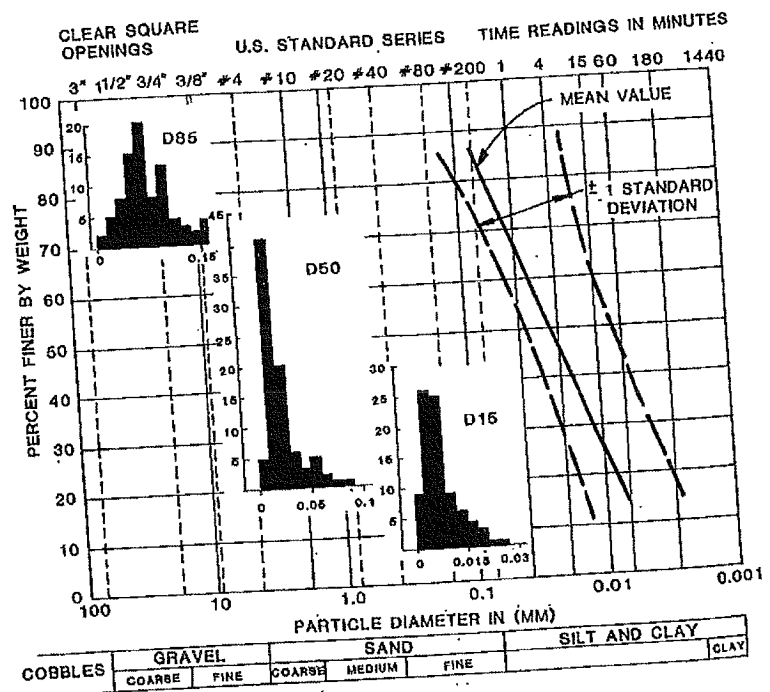


FIG. 3 - Distribution of Typical Values for the Grain Size of Class F Fly Ash

Grain size data for 17 Class C samples were reviewed. The sizes were similar to those for Class F ash. The coefficient of uniformity for samples available had a mean value of 7.19. The standard deviation was 3.8 and the coefficient of variation 52.9%. Information on the values for D85, D50 and D15 particle sizes is presented on Table 1.

TABLE 1. - Summary of Grain Size Analyses for Fly Ash

Grain Size in millimeters (1)	Mean Value (2)	Standard Deviation (3)	Coefficient of Variation (4)	Number of Samples (5)
(a) Class F Fly Ash				
D 85	0.079	0.063	80.0%	84
D 50	0.023	0.015	66.9%	84
D15	0.0075	0.0048	64.8%	84
(b) Class C Fly Ash				
D 85	0.063	0.020	31.7%	17
D 50	0.022	0.011	50.0%	17
D 15	0.0084	0.0082	97.6%	17

As shown above, fly ash is a relatively fine, uniform material. In sampling ash that has been stockpiled in fills and particularly in ponds it is not unusual to obtain samples that are substantially more coarse than the typical values cited above. This situation usually results from bottom ash being mixed with the fly ash or, in the case of ponds, from particle segregation during deposition in the pond.

Compaction Behavior

The compaction behavior of fly ash, as determined by the maximum dry density and optimum moisture content, was the test most commonly made on the samples located. Compaction test results were located for a total of 113 Class F samples and 25 Class C samples. Due to the large volumes of ash placed as compacted fill in dry disposal sites and its use as structural fill this is not surprising.

Most of the samples located were tested by either Modified or Standard Proctor methods. In a few cases some other procedure was used, or the test method was not available. Initially all of the results were compared, regardless of the test procedure used.

For the 113 Class F samples, the mean maximum dry density was 83.4 pounds per cubic foot. The standard deviation of the maximum dry density values was 13.0 pounds per cubic foot for a coefficient of variation of 15.6%. The corresponding optimum moisture contents for 110 of the samples were available. The mean value was 25.3%, and the standard deviation was 10.2%. The coefficient of variation of optimum moisture content values was 40.1%. The distribution of values for maximum dry density and optimum moisture contents for Class F fly ash are shown on Figures 4 and 5.

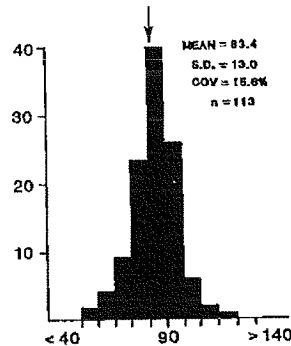


FIG. 4 - Distribution of Typical Values of Maximum Dry Density for Class F Fly Ash

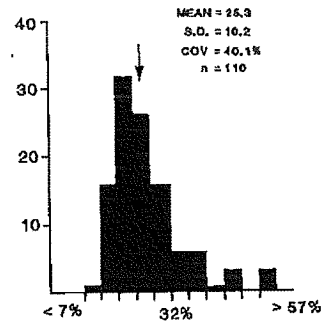


FIG. 5 - Distribution of Typical Values of Optimum Moisture for Class F Fly Ash

Perhaps a better way to look at the range of typical values is a plot of maximum dry density versus optimum moisture content for the samples available. Figure 6 shows this relationship for 110 samples. Most of the values fall within a relatively narrow band. As expected, the maximum dry density is inversely proportional to the optimum moisture content.

For the 25 Class C samples, the mean maximum dry density was 94.0 pounds per cubic foot with a standard deviation and coefficient of variation of 13.5 pounds per cubic foot and 14.4%, respectively. The optimum moisture content mean value was 19.8% with a standard deviation and coefficient of variation of 7.7% and 38.9%, respectively.

To determine the effect of the compaction test method, the Class F samples were separated by test type. Standard Proctor methods were used for 66 samples. The mean maximum dry density was 85.2 pounds per cubic foot and the mean optimum moisture content was 24.2%. For samples tested using Modified Proctor methods, the mean maximum dry density was 81.3 pounds per cubic foot and the mean optimum moisture content was 26.6%. There were 43 Modified Proctor samples included.

The reason that the mean value of maximum dry density for Standard Proctor samples is higher than that for Modified Proctor was not determined. Plots of maximum dry density versus optimum moisture content are denoted by test type on Figure 6. The plots appear to coincide rather than be parallel lines as might be expected if there were a consistent difference in the results of the two test methods. Therefore it is likely that the difference in mean values for maximum dry density is a result of the limited available data and is not real.

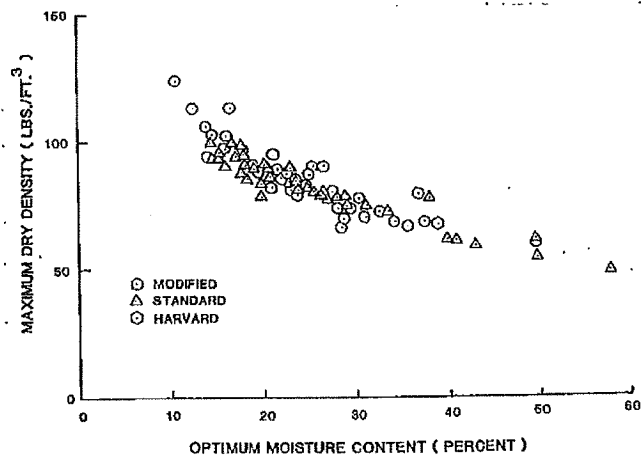


FIG. 6 - Maximum Dry Density Versus Optimum Moisture Content for Class F Fly Ash

While the dry density and compaction behavior is of interest in testing and quality control, the wet density is needed in conceptual design of fly ash fills. Calculation of settlement, bearing capacity and slope stability are all determined using wet density values. The mean value of wet density for 110 Class F samples was 103.4 pounds per cubic foot. The standard deviation was 10.1 pounds per cubic foot and the coefficient of variation 9.7%. The distribution of values for Class F fly ash are shown on Figure 7.

For Class C fly ash, 25 samples were evaluated. The mean maximum wet density was 111.7 pounds per cubic foot. The standard deviation was 10.3 pounds per cubic foot and the coefficient of variation 9.2%. Modified, Standard and other compaction procedures were included in determining wet density values for both classes of fly ash.

Permeability

Permeability values for 41 Class F samples were located. Since permeability values are often compared on an order of magnitude basis, the base 10 logarithms of the permeability values for the samples were compared. For example, the base 10 logarithm of a permeability of 1.0×10^{-5} cm/sec is -5.00, and that of 1.0×10^{-6} cm/sec is -6.00. The distribution of values is shown on Figure 8. The mean value of the 41 samples was -4.88. This value is equivalent to a permeability of 1.32×10^{-5} cm/sec. The standard deviation of the logarithmic values was 0.46, about 1/2 an order of magnitude. The coefficient of variation of the logarithmic values was 9.5%.

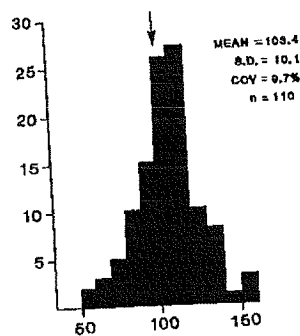


FIG. 7 - Distribution of Values for Maximum Wet Density for Class F Fly Ash.

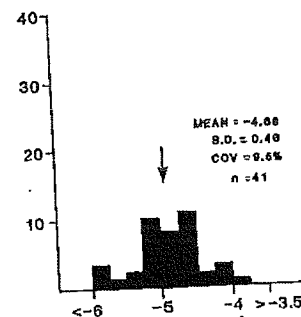


FIG. 8 - Distribution of Values for Log(10) of Permeability of Class F Fly Ash.

For Class C fly ash samples, 10 permeability test results were located. The mean value of the logarithmic values was -4.95 , equivalent to a permeability of 1.13×10^{-5} cm/sec. The standard deviation of the logarithmic values was 0.60 and the coefficient of variation was 12.1%.

The data base was analyzed in order to see if any effect of compaction on permeability could be determined. The results for undisturbed Class F samples obtained from ponds or poorly compacted fills were compared to those for the entire data base. The mean of the logarithm of the permeability for 10 undisturbed Class F samples was -4.73 , equivalent to a permeability of 1.8×10^{-5} cm/sec. This value represents an increase on permeability of 40% over that obtained for the entire data base. However, it is not significantly greater on an order of magnitude basis.

For Class C fly ash, reductions in permeability occur if the material is exposed to moisture and allowed to cure. This phenomenon has been observed for several of the samples included in the data base (Bahor et al. 1981; Gray and Lin 1972). Reductions in permeability of an order of magnitude have been observed for some Class C fly ashes.

Shear Strength

The shear strength of Class F fly ash is primarily the result of internal friction. Class F fly ash is non-cohesive and while it may appear to be cohesive in a partially saturated state, this effect is completely lost when the material is either dried or saturated. In contrast to this behavior, Class C fly ash can develop considerable cohesive shear strength due to cementitious reactions. This cohesion is the dominant shear strength of Class C fly ashes.

Shear strength test results were located for 51 Class F fly ash samples. Test procedures were primarily direct shear or consolidated drained triaxial tests. The friction angle and, in many cases, cohesion values were reported. The cohesion results ranged from zero to a few hundred pounds per square foot. For the purposes of this analysis the cohesion values were considered to be due to apparent cohesion, particle interlocking or other factors and were not considered. The mean value for friction angle was 34.0 degrees. The standard deviation was 3.3 degrees and the coefficient of variation was 9.8%. The distribution of values is shown on Figure 9.

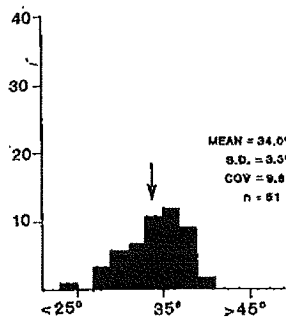


FIG. 9 - Distribution of Values for Friction Angle for Class F Fly Ash

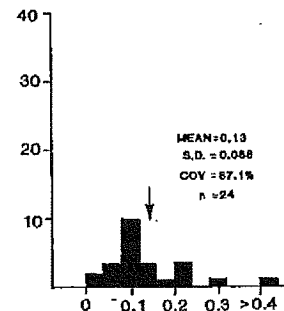


FIG. 10 - Distribution of Values for Compression Index for Class F Fly Ash

Samples from ash ponds and poorly compacted fills were evaluated separately in order to determine if any effect of sample compaction was evident in the data base. There were 12 samples of Class F fly ash that were uncompacted or poorly compacted. The mean friction angle for these samples was 32.8 degrees, slightly less than that of the entire data base. Previous tests have shown that the shear strength of fly ash from a single source is dependent upon the degree of compaction (DiGioia and Nuzzo 1972). The relatively small decrease in shear strength observed for lower density samples in the data base is believed to be due to the fact that fly ash is easily compacted with only moderate effort. Thus, even poorly compacted fills are relatively dense.

Class C fly ashes may gain considerable cohesive strength when exposed to moisture and allowed to cure. There is considerable variation among the fly ashes included in the data base. Test results indicate that cohesion may range up to several hundred pounds per square inch for some Class C fly ashes (Bahor et al. 1981; Chae and Snyder 1977; Gray and Lin 1972; Joshi 1978).

Compressibility

Compression index, C_c , values were determined for 24 of the Class F samples in the data base. The mean value was 0.13 with a standard deviation of 0.088, and a coefficient of variation of 67.1%. The distribution of values is shown on Figure 10.

Ten of the samples for which compressibility data were available were Shelby tube samples from ponded ash or poorly compacted fills. These samples were compared to the entire data base to determine if any difference due to compaction was evident. The mean value for C_c for the uncompacted samples was 0.16, about 25% greater than that for the total data base.

Little data is available on the compressibility of Class C fly ash. Tests on stabilized and Class C fly ashes (Gray and Lin 1972) have demonstrated that the compressibility of these materials is reduced as the material cures.

The compressibility of actual fly ash fills is not well understood, but the limited data that is available show that they are less compressible than indicated by the C_c values determined in laboratory tests (Cragg 1985; Leonards and Bailey 1982; Seals et al. 1977). In those cases where fly ash has been used as structural fill, the general performance has been excellent.

Summary

A data base of key physical and engineering properties was assembled to assess the typical values and range of values of fly ash from U.S. and Canadian sources. Table 2 presents a summary of the data for Class F fly ash. Table 3 summarizes the data for Class C fly ash.

A review of the data shows that fly ash is a relatively uniform, silt-sized material with a specific gravity slightly lower than most natural soils. Compaction of fly ash is moisture dependent, but the range of optimum moisture contents is greater than that of natural silts and silty clays. The maximum dry and wet densities of compacted fly ash are somewhat less than typical values for natural soils. This property makes it useful in light-weight structural fills.

Class F fly ash has moderate frictional shear strength when drained and Class C fly ash can develop significant cohesive strength when moistened and cured. Class F fly ash has a low permeability, but is not impervious. Class C fly ash has similar permeability to Class F fly ash and will become less permeable when moistened and allowed to cure. Laboratory tests on Class F fly ash indicate that it is a moderately compressible material. Limited field data indicate that fly ash is less compressible than suggested by laboratory tests.

The data presented show that the typical values for most of the parameters investigated are within a relatively narrow range of values. The typical values determined and the range of values should

be useful to geotechnical engineers for conceptual design and for evaluating specific test results to determine if those results are representative or may be suspect. However, actual properties of a particular fly ash may vary considerably from these typical values due to changes in fuel, combustion processes and equipment, contamination with other materials, segregation in handling or other factors. Therefore for actual design purposes there is no substitute for data on the actual fly ash being used.

TABLE 2. - Summary of Results for Class F Fly Ash

Property (1)	Mean Value (2)	Standard Deviation (3)	Coefficient of Variation	Comments (5)
Specific Gravity	2.4	0.15	6.0%	
Coefficient of Uniformity	5.49	3.6	60.8%	Also See Table 1
Maximum Dry Density in pounds per cubic foot	83.4	13.0	15.6%	Also See Figure 6
Optimum Moisture Content - %	25.3	10.2	40.1%	Also See Figure 6
Maximum Wet Density in pounds per cubic foot	103.4	10.1	9.7%	
Log (10) of Permeability	-4.88	0.46	9.5%	Mean Value Equals 1.32×10^{-5} cm/sec
Effective Friction Angle in degrees	34.0	3.3	9.8%	
Compression Index	0.13	0.088	67.1%	

TABLE 3. Summary of Results for Class C Fly Ash

Property (1)	Mean Value (2)	Standard Deviation (3)	Coefficient of Variation	Comments (5)
Specific Gravity	2.5	0.17	6.8%	
Coefficient of Uniformity	7.19	3.8	52.9%	Also See Table 1
Maximum Dry Density in pounds per cubic foot	94.0	13.5	14.4%	
Optimum Moisture Content - %	19.8	7.7	38.9%	
Maximum Wet Density in pounds per cubic foot	111.7	10.3	9.2%	
Log (10) of Permeability	-4.95	0.60	12.1%	Mean Value Equals 1.13×10^{-5} cm/sec

Appendix I.- References

- Anderson, R.B., DiGioia, A.M. Jr., and Head, W.J. (1982). "Coal Refuse - Fly Ash Mixtures as Base-Course Material," presented at the Sixth International Ash Utilization Symposium, Reno, Nevada, DPE/MEFC/82-52 V1, 137-163.
- Bahor, M.P., et al. (1981). "Coal Ash Disposal Manual, Second Edition," Electric Power Research Institute Report CS2049.
- Chae, Y.S., and Snyder, J.L. (1977). "Vibratory Compaction of Fly Ash," presented at the ASCE Specialty Conference on Geotechnical Practice for Disposal of Solid Waste Materials, Ann Arbor, Michigan, 41-62.
- Cragg, C. (1985). "Fly Ash Speeds Bridge Construction," presented at the Seventh International Ash Utilization Symposium, Orlando, Florida, DOE/MEFC-85/6018 V2, 487-501.
- Cunningham, J.A., Lukas, R.G., and Anderson, T.C. (1977). "Impoundment of Fly Ash and Slag - A Case Study," presented at the ASCE Specialty Conference on Geotechnical Practice for Disposal of Solid Waste Materials, Ann Arbor, Michigan, 227-245.
- DiGioia, A.M. Jr., and Nuzzo, W.L. (1972). "Fly Ash as Structural Fill," J. of Power Div., ASCE, 98(1), 77-92.
- GAI Consultants, Inc. (1976). "Guide for the Design and Construction of Cement-Stabilized Fly Ash Pavements," National Ash Association, Washington D.C.

- Gray, D.H., and Lin, Y.K. (1972). "Engineering Properties of Compacted Fly Ash," J. of the Soil Mechanics and Foundation Div., ASCE, 98(4), 361-380.
- Head, H.J., and Seals, R.K. (1979). "Design and Construction of Experimental Haul Road and Parking Lot Facilities Utilizing Power Plant Ash," presented at the Fifth International Ash Utilization Symposium, Atlanta, Georgia, METC/SP-79/10 V2, 679-693.
- Joshi, R.C. (1978). "Structural Fills Using Fly Ash," presented at the Ash Management Conference, Texas A&M University, College Station, Texas.
- Kinder, D.L. and Morrison, R.E. (1979). "An Engineering Approach for Using Fly Ash in a Structural Fill," presented at the Fifth International Ash Utilization Symposium, Atlanta, Georgia METC/SP-70/10 V2, 713-724.
- Lamb, D.W., Herrmann, W., and Pizzella, J. (1976). "Fly Ash as Construction Material for Water Retaining Structures," presented at the Fourth International Ash Utilization Symposium, St. Louis, Missouri, MERC/SP-76/4, 369-379.
- Leonards, G.A., and Bailey, B. (1982). "Use of Pulverized Coal Ash as Structural Fill," Journal of Geotechnical Division, ASCE, 108(4), 517-531.
- Ng, S.Y. (1979). "Experience with Geotechnical Methods for Design of Fly Ash Embankments and Structural Fills," presented at the Fifth International Ash Utilization Symposium, Atlanta, Georgia, METC/SP-79/10 V2, 725-747.
- Rose, J.L., Chiang, L., and Lowe, J.A. (1982). "An Engineering Evaluation of Blended Coal By-Product Materials for Stabilized Base and Highway Construction Applications," presented at the Sixth International Utilization Symposium, Reno, Nevada, DOE/METC/82-52 VI, 314-357.
- Santhanam, C.J., and Ullrich, C.R. (1979). "Characterization of Flue Gas Cleaning (FGC) Wastes from Coal Combustion," presented at the Fifth International Ash Utilization Symposium, Atlanta, Georgia, METC/SP-79/10 V1, 405-447.
- Seals, R.K., Moulton, L.K., and Kinder, D.L. (1977). "In Situ Testing of a Compacted Fly Ash Fill," presented at the ASCE Specialty Conference on Geotechnical Practice for Disposal of Solid Waste Materials, Ann Arbor, Michigan, 493-516.
- Sikes, P.G., and Kolbeck, H.J. (1973). "Disposal and Uses of Power Plant Ash in Urban Area," Journal of Power Division, ASCE, 99(1), 217-234.
- Srinivasan, C., Beckwith, G.H., and Burke, H.H. (1977). "Geotechnical Investigations of Power Plant Wastes," presented at the ASCE Specialty Conference on Geotechnical Practice for Disposal of Solid Waste Materials, Ann Arbor, Michigan, 169-187.
- Thorton, S.I., Parker, D.G., and Yang, S.N. (1979). "Delayed Compaction of High Calcium Fly Ash Stabilized Soil," presented at the Fifth International Ash Utilization Symposium, Atlanta, Georgia, METC/SP-79/10 V2, 748-757.

ATTACHMENT D

Excerpts from Lambe and Whitman 1969)

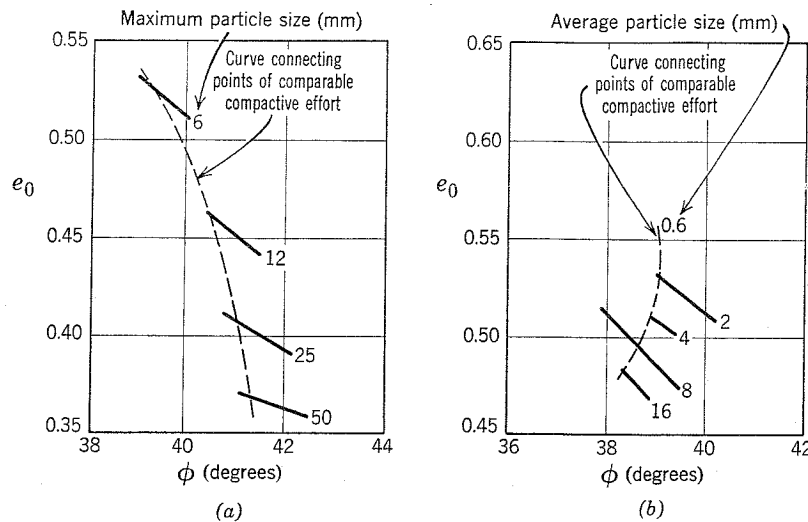


Fig. 11.12 Effect of particle size and gradation on friction angle. (a) Soils with same minimum particle size (0.5 mm). (b) Soils with same uniformity coefficient. Data from Leslie (1963).

particle size increases the load per particle, and hence crushing begins at a smaller confining stress. Spurred by the increasing popularity of rockfill dams, several laboratories have constructed triaxial testing systems which can accommodate specimens as large as 12 in. in diameter. An apparatus that can test specimens 3.7 ft in diameter and 8.2 ft long has been constructed in Mexico (Marsal, 1963).

Grading of the Sand

Figure 11.12a shows data for four soils all having the same minimum particle size but different maximum particle sizes. For comparable compactive efforts, the better graded sand has both a smaller initial void ratio and a larger friction angle. It is apparent that a better distribution of particle sizes produces a better interlocking. This trend is also shown by the data in Table 11.2, and is further confirmed by a series of tests reported by Holtz and Gibbs (1956).

In many soils, a few particles of relatively large size make up a large fraction of the total weight of the soil.

Table 11.2 Effect of Angularity and Grading on Peak Friction Angle

Shape and Grading	Loose	Dense
Rounded, uniform	30°	37°
Rounded, well graded	34°	40°
Angular, uniform	35°	43°
Angular, well graded	39°	45°

From Sowers and Sowers, 1951.

If these particles are numerous enough so that they interlock with each other, it is important that these large particles be present in the test specimen. However, if these larger particles are just embedded in a matrix of much smaller particles so that the shearing takes place through the matrix, then the large particles can safely be omitted from the specimen. Unfortunately, the profession still is lacking definitive guides as to what constitutes a satisfactory test upon a gravelly soil.

A well-graded soil experiences less breakdown than a uniform soil of the same particle size, since in a well-graded soil there are many interparticle contacts and the load per contact is thus less than in the uniform soil. Figure 11.13 illustrates that the better graded soil suffers less decrease in ϕ with increasing confining pressure.

Angularity of Particles

It would be expected that angular particles would interlock more thoroughly than rounded particles, and hence that sands composed of angular particles would have the larger friction angle. The data for peak friction angle presented in Table 11.2 confirm this prediction. Even when a sand is strained to its ultimate condition, so that no further volume change is taking place and the sand is in a loose condition, the sand with the angular particles has the greater friction angle. In gravels, the effect of angularity is less because of particle crushing.

Mineral Type

Unless a sand contains mica, it makes little difference whether the sand is composed primarily of quartz, one of the feldspars, etc. A micaceous sand will often have a large void ratio, and hence little interlocking and a low

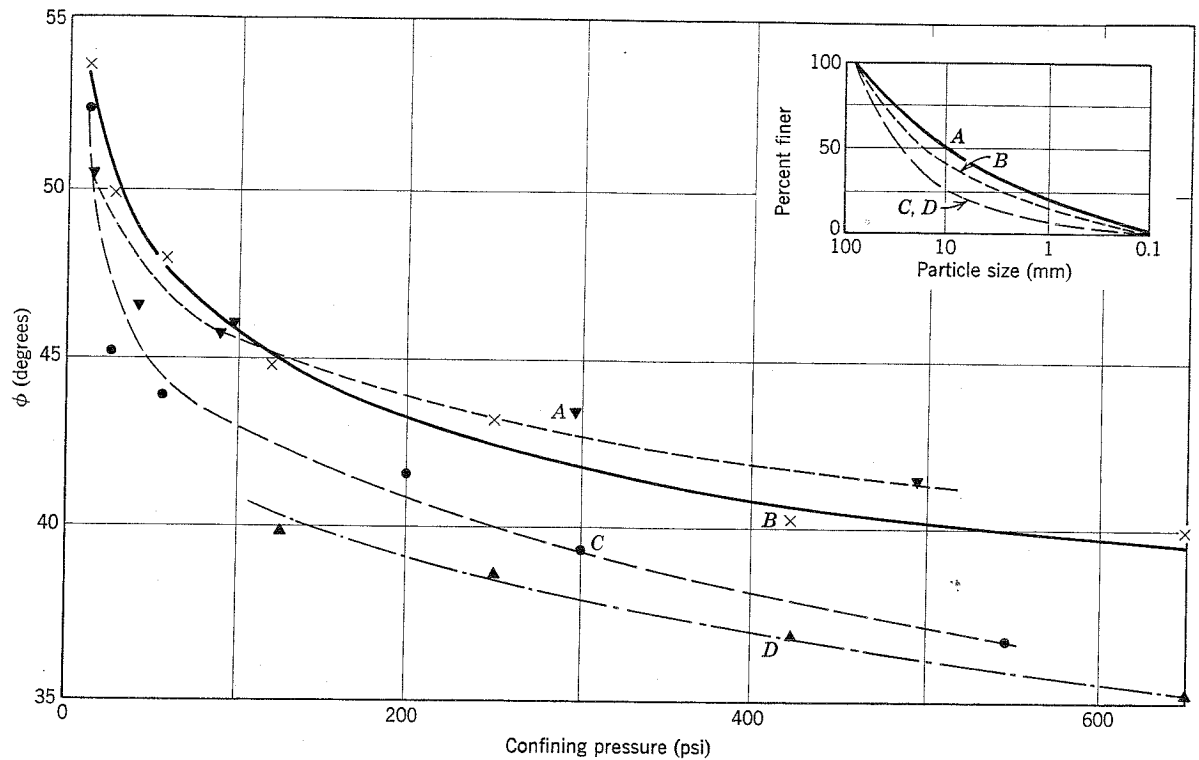


Fig. 11.13 Friction angle versus confining pressure (data from Leslie 1963).

friction angle. The smaller value of ϕ_μ for mica compared to that of quartz has relatively little to do with this result.

Tests (Horn and Deere, 1962) have been carried out using powdered mica with care taken to have the mica flakes oriented nearly parallel. The result was a friction angle (ϕ_{cv}) of 16° , compared to $\phi_\mu = 13\frac{1}{2}^\circ$. There is some small amount of interlocking in such a case.

Where particles of gravel are an important constituent of soil, the origin of the gravel particles can have an important effect. If the gravel particles are relatively soft, crushing of these particles will minimize the interlocking effect and decrease the friction angle as compared to a comparable soil with hard gravel particles.

Summary

The composition of a granular soil can have an important influence upon its friction angle, indirectly by influencing e_0 and directly by influencing the amount of interlocking that occurs for a given e_0 . Table 11.3 provides a summary of data that can be used for preliminary design. However, for final design of an embankment, the actual soil should be tested using the void ratio and stress system that will exist in the field.

11.5 DETERMINATION OF *IN SITU* FRICTION ANGLE

The results presented in the foregoing sections have emphasized the predominant role of the degree of inter-

locking upon magnitude of the friction angle. Thus, if we wish to determine the friction angle of a sand *in situ*, it is not enough to find the nature and shape of the particles composing the sand. It is essential to know how tightly together these particles are packed in their natural state.

It is extremely difficult to obtain samples of a sand without changing the porosity. Thus, except for

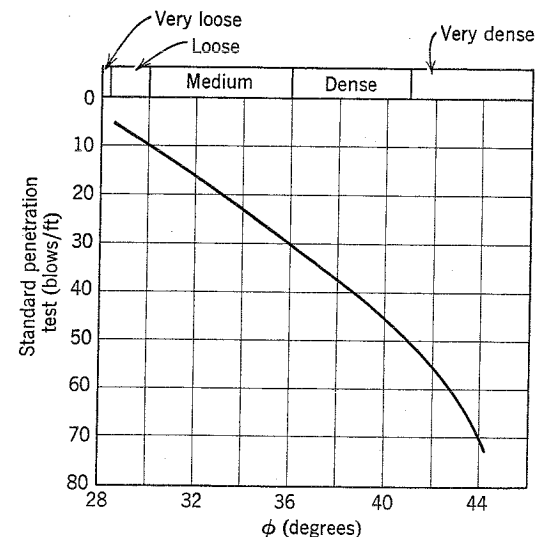


Fig. 11.14 Correlation between friction angle and penetration resistance (From Peck, Hanson, and Thornburn, 1953).

Table 11.3 Summary of Friction Angle Data for Use in Preliminary Design

Classification	Friction Angles							
	Slope Angle of Repose		At Ultimate Strength		At Peak Strength			
					Medium Dense		Dense	
	$i(^{\circ})$	Slope (vert. to hor.)	$\phi_{cv}(^{\circ})$	$\tan \phi_{cv}$	$\phi(^{\circ})$	$\tan \phi$	$\phi(^{\circ})$	$\tan \phi$
Silt (nonplastic)	26	1 on 2	26	0.488	28	0.532	30	0.577
	to		to		to		to	
	30	1 on 1.75	30	0.577	32	0.625	34	0.675
Uniform fine to medium sand	26	1 on 2	26	0.488	30	0.577	32	0.675
	to		to		to		to	
	30	1 on 1.75	30	0.577	34	0.675	36	0.726
Well-graded sand	30	1 on 1.75	30	0.577	34	0.675	38	0.839
	to		to		to		to	
	34	1 on 1.50	34	0.675	40	0.839	46	1.030
Sand and gravel	32	1 on 1.60	32	0.625	36	0.726	40	0.900
	to		to		to		to	
	36	1 on 1.40	36	0.726	42	0.900	48	1.110

From B. K. Hough, *Basic Soils Engineering*. Copyright © 1957, The Ronald Press Company, New York.

Note. Within each range, assign lower values if particles are well rounded or if there is significant soft shale or mica content, higher values for hard, angular particles. Use lower values for high normal pressures than for moderate normal pressure.

problems involving man-made fills, it is difficult to either measure or estimate the friction angle of a sand on the basis of laboratory tests alone. For these reasons, extensive use is made in practice of correlations between the friction angle of a sand and the resistance of the natural sand deposit to penetration.

Figure 11.14 shows an empirical correlation between the resistance offered to the standard penetration spoon (Chapter 7) and the friction angle. Inevitably, any such correlation is crude. The actual friction angle may deviate by $\pm 3^{\circ}$ or more from the value given by the curve. The given relation is intended to apply for depths of overburden up to 40 ft, and is conservative for greater depths.

11.6 SUMMARY OF MAIN POINTS

1. The strength of soil can be represented by a Mohr envelope, which is a plot of τ_{ff} versus σ_{ff} . Generally the Mohr envelope of a granular soil is curved. For stresses less than 100 psi, the envelope usually is almost straight so that

$$\tau_{ff} = \sigma_{ff} \tan \phi$$

where ϕ is the friction angle corresponding to the peak point of the stress-strain curve.

2. The value of ϕ for any soil depends upon ϕ_{μ} and upon the amount of interlocking; i.e., the initial void ratio and σ_{ff} .
3. Where sand is being subjected to very large strains, ϕ_{cv} should be used in the failure law. Unless the sand is very loose, ϕ_{cv} will be less than ϕ . Where the sand is sliding over the surface of a structure, the friction angle will vary from ϕ_{μ} to ϕ_{cv} , depending on the smoothness of the surface.
4. A knowledge of the effect of composition helps guide the selection of materials to be used in man-made fills
5. Materials to be used in man-made fills should be tested using the actual range of confining pressures which will be encountered in the fill.
6. For many practical problems, the friction angle of an *in situ* sand deposit can be determined by indirect means, such as the standard penetration test.

PROBLEMS

- 11.1 Given the following triaxial test data, plot the results (a) in a Mohr diagram and (b) in a p - q diagram, and determine ϕ by each method.

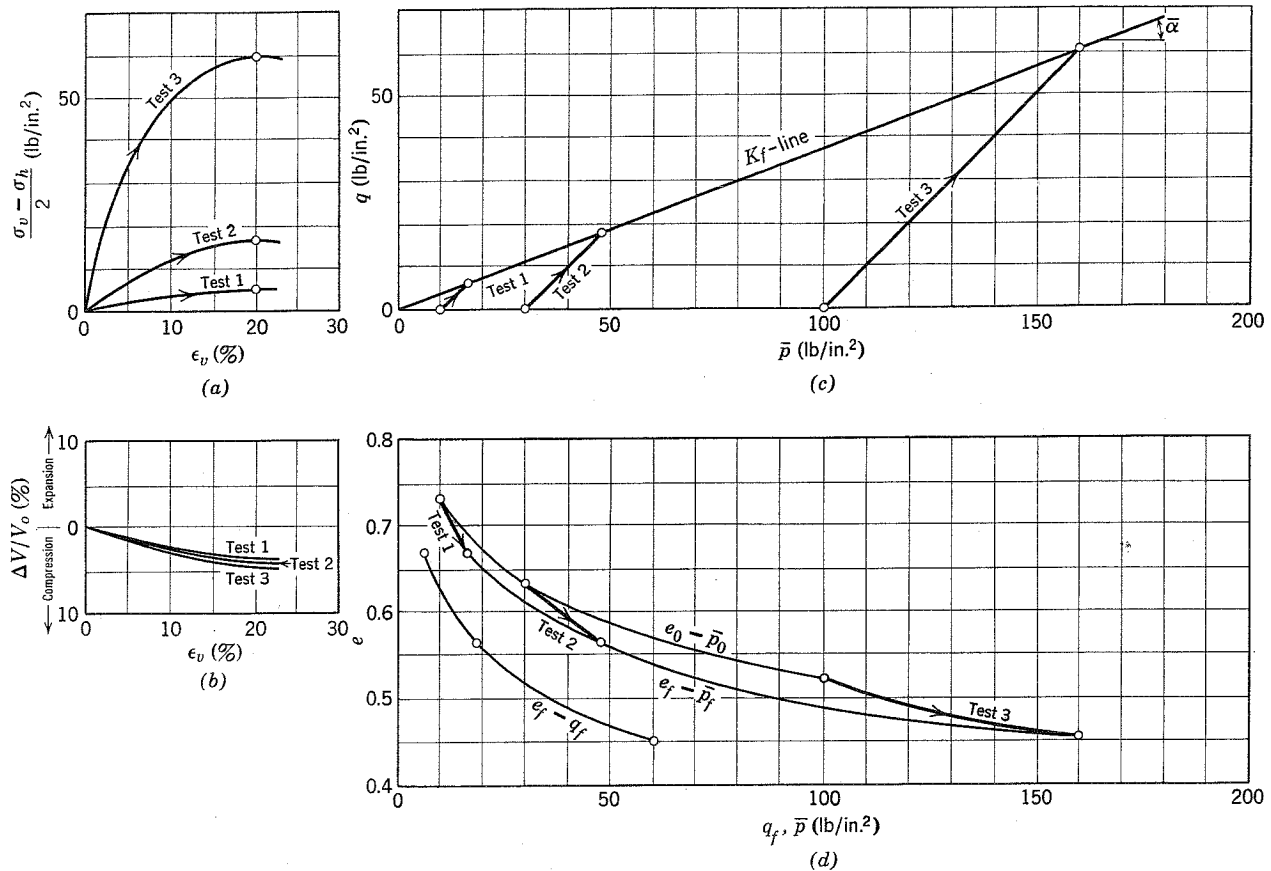


Fig. 21.1 Test results on normally consolidated Weald clay.

Figure 21.2 shows stress-strain and stress path data for normally consolidated Weald clay presented in a normalized fashion. Since the K_f -line is straight, the $\bar{p}$ - q plot can be accurately normalized. The stress-strain data can only be approximated by normalized curves.

Stress-Volume Relationship

From Fig. 21.1 we can see that the test data from the drained tests on the normally consolidated Weald clay indicate that $e_0 - \bar{p}_0$, $e_f - \bar{p}_f$, and $e_f - q_f$ are smooth plots. Also shown in this figure are lines indicating how the void ratio changes during the drained tests. The stress-volume data in Fig. 21.1d are replotted in Fig. 21.3 in a more convenient form. Water contents are used in place of void ratios since they are more convenient to determine, and for a saturated soil, water content and void ratio are uniquely related. The stress data are plotted on a log scale since it has been found empirically that void ratio or water content versus the various stresses form approximately straight lines which are parallel to one another for normally consolidated clay.

From the data plots in Fig. 21.1d or 21.3, we see that there is a three-way relationship among shear strength, effective stress at failure, and water content or void ratio

at failure. In Part V we shall see that the same $q_f - \bar{p}_f - w_f$ relation still applies to Weald clay when the clay is sheared without permitting water to escape.

Any factor that alters the void ratio, i.e. the density of the skeleton, should alter the drained strength, which should also be accompanied by a change in the effective stresses existing at failure. In a sense, a change in effective stress can produce a material with a different density and hence a different strength. It is difficult to say which factor—density or effective stress—controls strength: all three of these factors are interrelated. However, it is more useful to use *effective stress* as the primary controlling variable.

Other Loading Conditions

There is an infinite number of possible stress paths for shearing a sample of clay. Figure 9.8 showed four convenient stress paths for the triaxial tests. In general, the strength parameters of normally consolidated soil are approximately independent of the stress path to failure. As a first and a very good general approximation, we can take q_f versus $\bar{p}_f$ for normally consolidated clay as a unique straight line. The value of q_f , of course, depends very much on the stress path of a loading since $\bar{p}_f$ is a

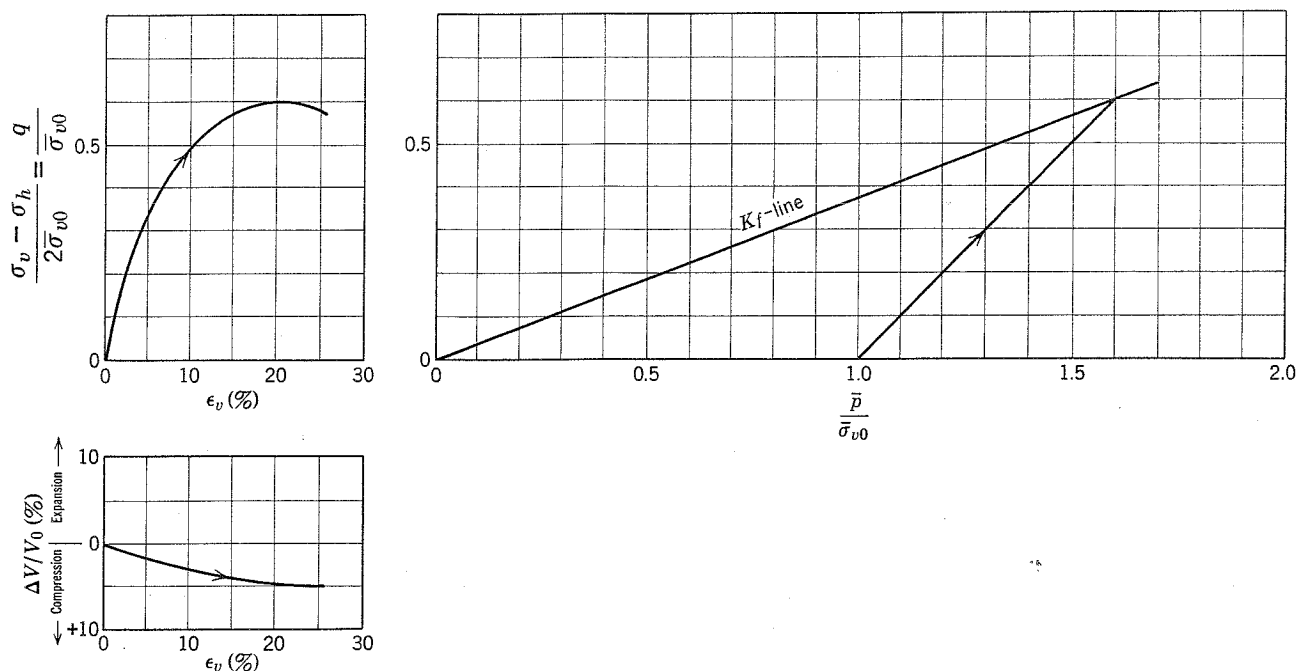


Fig. 21.2 Normalized test results.

function of loading stress path. Examples 21.1 to 21.3 illustrate the estimation of q_f for various loading conditions.

Comparison of Clay and Sand

The preceding discussion illustrates that the behavior of normally consolidated clay during drained shear is essentially similar to that of sand, or more particularly, loose sand. That is, for both classes of soil the failure law is

$$\tau_{ff} = \bar{\sigma}_{ff} \tan \bar{\phi}$$

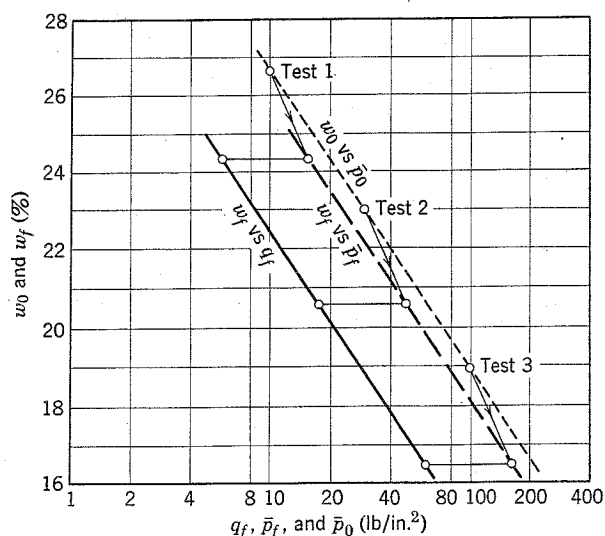


Fig. 21.3 Test results on normally consolidated Weald clay.

Typical Values of $\bar{\phi}$ for Normally Consolidated Clays

Figure 21.4 gives a good indication of typical values of $\bar{\phi}$ for soil. Although there is considerable scatter, there is a definite trend toward decreasing $\bar{\phi}$ with increasing plasticity. It is difficult to determine the effective stress-strength relationship for an extremely plastic clay such as montmorillonite or the Mexico City clay because of the extremely long time required for dissipation of excess pore pressures during the test. Hence the magnitude of $\bar{\phi}$ for very plastic soils is uncertain.

21.2 OVERCONSOLIDATED CLAY

Now let us take several specimens of remolded Weald clay and consolidate them to 120 lb/in.² This stress (represented by $\bar{p}_m$ for maximum past $\bar{p}$) is greater than any effective stress to which the specimens have been subjected since remolding. Now the chamber pressure acting on each specimen is reduced to values $\bar{p}_0$ ranging from 5 to 70 lb/in.², and water is permitted to flow into the specimens so that they come to equilibrium under these reduced effective stresses. The overconsolidation ratio OCR (defined as $\text{OCR} = \bar{p}_m/\bar{p}_0$; in one-dimensional compression, OCR is $\bar{\sigma}_{vm}/\bar{\sigma}_{v0}$) is a convenient way to characterize the condition of these specimens. The values of OCR for our specimens range from 1.7 to 24.

Strength Versus Effective Stress

Figure 21.5 shows the stress paths for these tests. As long as $\bar{p}_f > 0.5\bar{p}_m$, the q_f versus $\bar{p}_f$ points fall along the

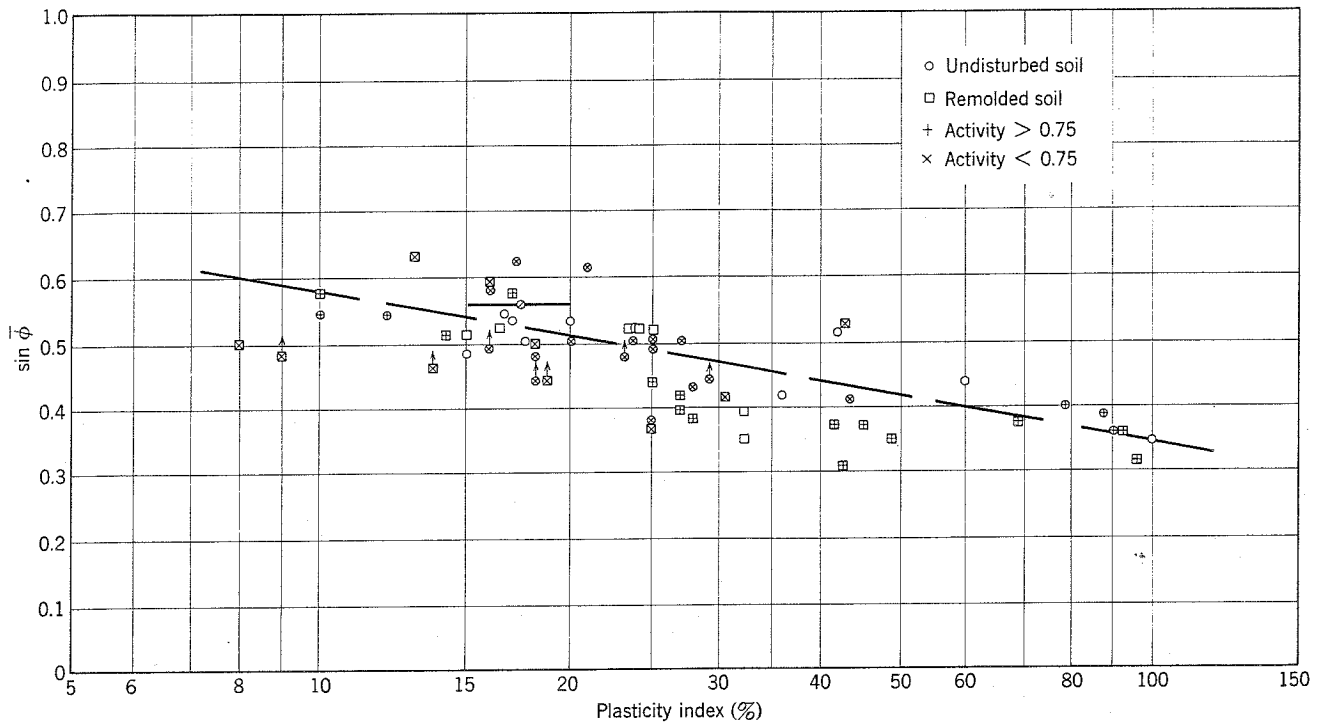


Fig. 21.4 Relationship between $\sin \phi$ and plasticity index for normally consolidated soils (From Kenney, 1959).

same line obtained from tests on normally consolidated specimens. For lower $\bar{p}_f$ the points fall above the relation from normally consolidated tests. Thus preconsolidation affects the effective stress-strength relation and tends to make the sample stronger at a given $\bar{p}_f$. This preconsolidation effect is difficult to see when results are plotted to the scale of Fig. 21.5a, hence the portion of this plot near the origin is magnified in the lower portion of the figure.

► Example 21.1

A specimen of Weald clay is consolidated to 100 lb/in.², and is then failed by decreasing $\bar{\sigma}_3$ while $\bar{\sigma}_1$ is held constant. Find q_f , $\bar{p}_f$, and w_f .

Solution. On part (c) of Fig. 21.1 draw the effective stress

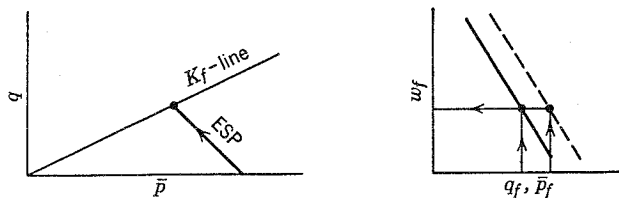
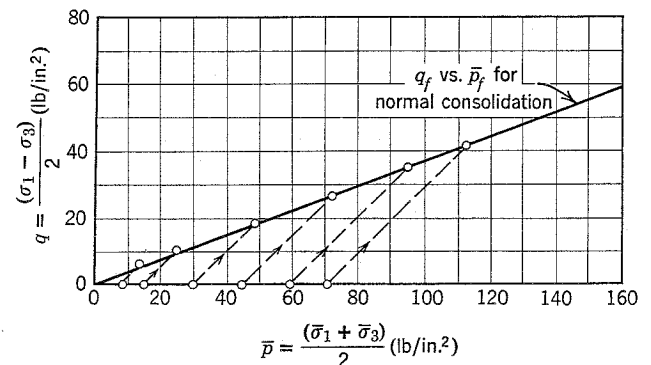


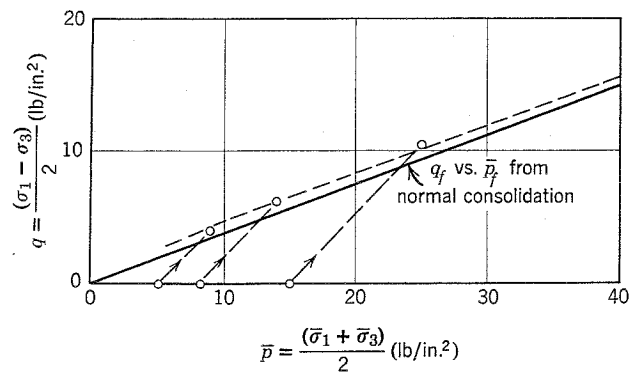
Fig. E21.1

path for this loading until it intersects the q_f - $\bar{p}_f$ relation (see Fig. E21.1). The point of intersection gives q_f and $\bar{p}_f$. Then go to Fig. 21.3, enter with either q_f or $\bar{p}_f$, and read w_f .

Answers. $q_f = 27$ lb/in.², $\bar{p}_f = 73$ lb/in.², $w_f = 19.2\%$. Note that w increased slightly during shear. ◀



(a)



(b)

Fig. 21.5 Results of CD tests on overconsolidated Weald clay. $\bar{p}_m = 120$ lb/in.²

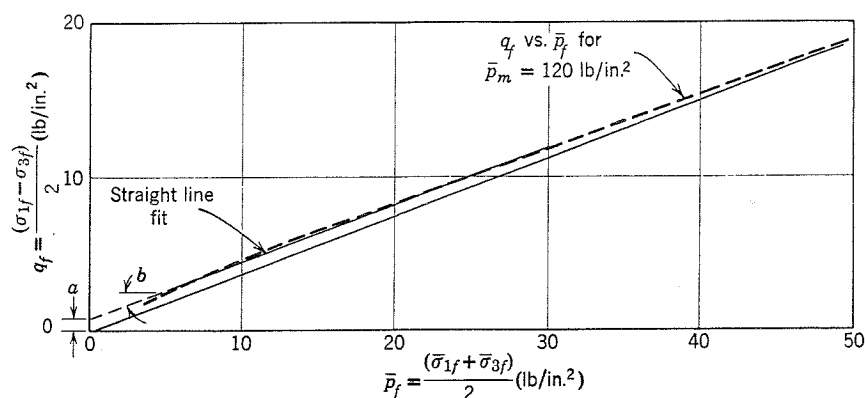


Fig. 21.6 Straight line fit to curved Mohr envelope for overconsolidated Weald clay.

$$a = 1 \text{ lb/in.}^2 \quad b = 20.3^\circ$$

$$\sin \bar{\phi} = \tan b \quad \bar{c} = a / \cos \bar{\phi}$$

$$\bar{\phi} = 21.5^\circ \quad \bar{c} = 150 \text{ lb/ft}^2$$

► Example 21.2

A specimen is consolidated to 60 lb/in.², and is then failed by increasing $\bar{\sigma}_1$ and decreasing $\bar{\sigma}_3$ in such a way that $\bar{p}$ remains constant. Find q_f , $\bar{p}_f$, and w_f .

Solution. For this case the effective stress path is a vertical line (see Fig. E21.2) and $\bar{p}_f = 60 \text{ lb/in.}^2$. Next turn to Fig. 21.3 to find the answers.

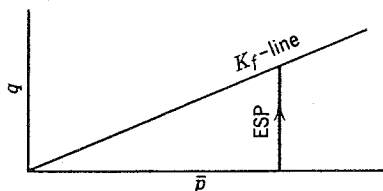


Fig. E21.2

Answers. $q_f = 22.5 \text{ lb/in.}^2$, $w_f = 19.9\%$. Note that w decreased during shear. (Incidentally, actual test data for this loading gave $q_f = 22.5 \text{ lb/in.}^2$ and $w_f = 19.6\%$.) ◀

The effective stress-strength relation for overconsolidated Weald clay (see Fig. 21.5) has been reproduced in Fig. 21.6. This relation is not a straight line. For calculation purposes, it is usually desirable to replace the curved relation with a straight line, thus expressing strength as

$$\tau_{ff} = \bar{c} + \bar{\sigma}_{ff} \tan \bar{\phi}$$

This straight line is drawn to provide the best fit to the actual curve over the stress range of interest. Such a straight line, when extended to the ordinate, generally produces a *cohesion intercept* even though the effective stress-strength relation itself may curve back through the origin. For the particular straight-line fit shown in Fig. 21.6, the strength parameters for Weald clay are

► Example 21.3

A specimen is consolidated to 30 lb/in.², and is then failed by increasing $\bar{\sigma}_1$ and $\bar{\sigma}_3$, such that $\Delta \bar{\sigma}_3 = \frac{1}{3} \Delta \bar{\sigma}_1$. Find $\bar{p}_f$, q_f , and w_f .

Solution. From the given data:

$$\left. \begin{aligned} \Delta \bar{p} &= \frac{\Delta \bar{\sigma}_1 + \Delta \bar{\sigma}_3}{2} = \frac{2}{3} \Delta \bar{\sigma}_1 \\ \Delta q &= \frac{\Delta \bar{\sigma}_1 - \Delta \bar{\sigma}_3}{2} = \frac{1}{3} \Delta \bar{\sigma}_1 \end{aligned} \right\} \quad \Delta q = \frac{1}{2} \Delta \bar{p}$$

The effective stress path rises at a slope of 2 on 1 to the right (see Fig. E21.3). The intersection with q_f versus $\bar{p}_f$ (Fig. 21.1c) gives q_f and $\bar{p}_f$. w_f is then read from Fig. 21.3.

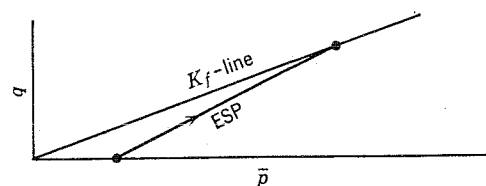


Fig. E21.3

Answers. $q_f = 43.5 \text{ lb/in.}^2$, $\bar{p}_f = 117 \text{ lb/in.}^2$, $w_f = 17.4\%$. Note that there has been a large decrease in water content during shear (from 23% to 17.4%), and a very large axial strain would be necessary in order to achieve failure. ◀

$\bar{\phi} = 21.5^\circ$ and $\bar{c} = 150 \text{ lb/ft}^2$. This situation is similar to that experienced with dense sands.

Stress-Volume Relationships

The relationships between $\bar{p}_0$ and w_0 , $\bar{p}_f$ and q_f and w_f are shown in Fig. 21.7. For convenience in separating the relationships, they are given in two diagrams. The straight portions without data points are the relations for normally consolidated specimens.

The effect of preconsolidation is seen in all of these relations. Thus the w_f versus q_f and w_f versus $\bar{p}_f$ relations are different for overconsolidated specimens than for the normally consolidated specimens: for a given $\bar{p}_f$ the water content is smaller (corresponding to the increased strength) in the overconsolidated specimen. However, the relations for the failure condition are affected much less by preconsolidation than is the w_0 versus $\bar{p}_0$ relation. Thus shearing action tends to destroy the effects of preconsolidation but does not quite succeed.

Figure 21.7 is especially useful for illustrating the changes in water content during shear. Some of the specimens increased in water content during shear, whereas others decreased in water content. For example, the specimen that was rebounded to 15 lb/in.² increased in water content from 20.5 to 21.3%; i.e., water was sucked into the specimen during the shear part of the test. On the other hand, the specimen that was rebounded to 60 lb/in.² decreased in water content during shear from 18.9 to 18.3%. A sample with an overconsolidation ratio of about 4 would not have any net volume change during the shear process.

The q_f - $\bar{p}_f$ - w_f Relationship

We have now seen that a soil does not possess a truly unique relationship among shear strength, effective stress at failure, and water content at failure—this relationship is affected by the degree of preconsolidation.¹

This will be the first of several instances in which we shall see that the q_f - $\bar{p}_f$ - w_f relation is only approximately unique. Actually, the concept of such a relation is qualitative rather than quantitative. It reminds us that the three quantities are closely interrelated and helps us to envision how changes in either $\bar{p}_f$ or w_f will affect strength and each other.

Having determined this relationship for a given preconsolidation, we can use it to find the strength for any type of loading. Examples corresponding to those for normally consolidated soil will be found among the problems.

Comparison of Clay and Sand

The results presented here again emphasize the similarities in the strength behavior of sand and clay. For both types of soils, the shear strength is strongly dependent on the effective stress existing at the time of failure. Furthermore, in both types of soil, the shear strength is also dependent on the initial void ratio before shear, i.e., the degree to which the soil has been “pre-densified” by some past action. In a general way, dense sands correspond to heavily overconsolidated clays,

whereas loose sands correspond to normally consolidated clays. Note that heavily overconsolidated clays and dense sands both have curved failure envelopes.

Stress by itself is not usually as effective in densifying a sand as it is in densifying a clay. In order to achieve a significant densification of a loose sand, it generally is necessary to apply many cycles of loading and unloading (or vibration). Thus, in detail, we have one difference between the strength behavior of clay and sand: an application and removal of stress (slowly, so that drainage of water can occur) will improve the strength of clay at some given effective stress, but will not significantly alter the strength of the sand.

There is one additional difference between the drained strength behavior of sand and clay: whereas the Mohr envelope for a dense sand does pass through the origin, that for heavily overconsolidated clay generally passes

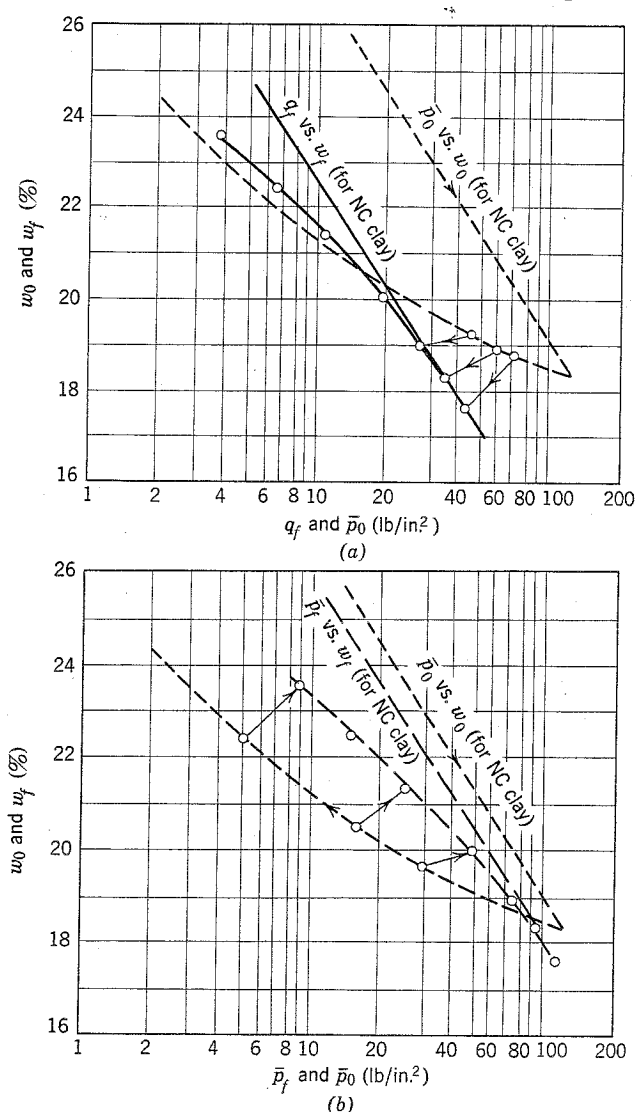


Fig. 21.7 Stress-volume relations for overconsolidated Weald clay. $\bar{p}_m = 120$ lb/in.²

¹ For remolded Weald clay there is such a unique relationship for any given $\bar{p}_m$; i.e., there is a family of such relationships with $\bar{p}_m$ as a parameter and with the relation for normally consolidated clays as an envelope.

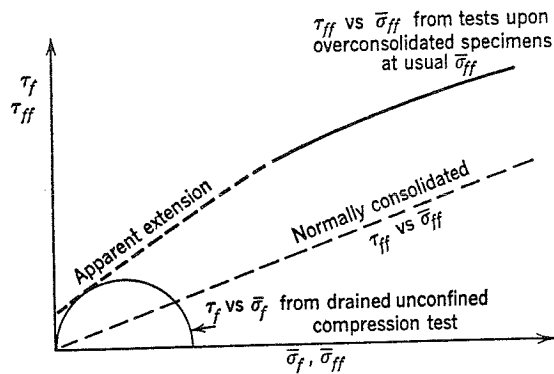


Fig. 21.8 Extension of curved Mohr envelope toward origin.

somewhat above the origin. This actual cohesion intercept presumably results from the formation of bonds among clay particles. An important question is: How close does the envelope pass to the origin; i.e., how large is q_f when $\bar{p}_f = 0$? A preliminary investigation using submerged, unjacketed specimens (so that $\bar{\sigma}_3 = 0$) has given the following results for two remolded clays with $\bar{p}_m = 6 \text{ kg/cm}^2$ (see Fig. 21.8).

	$\bar{c}$ for straight line fit for $\bar{\sigma}_{ff}$ between 1 and 6 kg/cm^2	Actual $\bar{c}$
Clay A	0.1 kg/cm^2	0.050 kg/cm^2
Clay B	0.2 kg/cm^2	0.085 kg/cm^2

Typical Values of $\bar{c}$ and $\bar{\phi}$ for Overconsolidated Clays

The magnitude of $\bar{c}$ and $\bar{\phi}$ for a given clay depends on how large the preconsolidation stress has been, how long the clay has been under the preconsolidation stress, etc. The effect of preconsolidation can best be illustrated by data for a compacted soil (Fig. 21.9) where the compaction effort supplies the preconsolidation.

The $\bar{c}$ and $\bar{\phi}$ for a given soil also depend on the stress

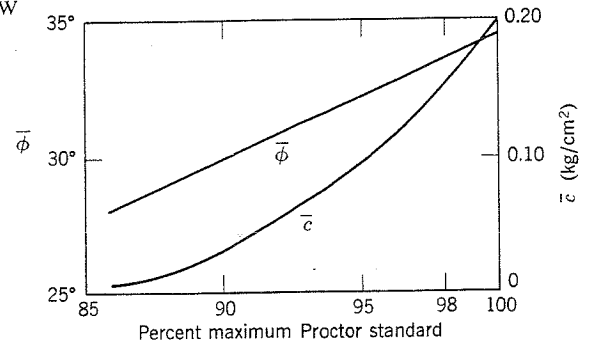


Fig. 21.9 Effect of compactive effort on strength parameters of clayey sand (after Moretto et al., 1963).

range over which a straight line fit is made to the curved Mohr envelope. Thus:

1. When effective stress is a large fraction of preconsolidation stress—when there is low OCR— $\bar{\phi}$ will be slightly less than for normally consolidated clay, while $\bar{c}$ will depend on magnitude of preconsolidation stress (void ratio).
2. When effective stress is very small compared to preconsolidation stress—when there is high OCR— $\bar{c}$ will be relatively small and $\bar{\phi}$ will depend on magnitude of preconsolidation stress (void ratio).

Figure 21.10 illustrates (to an extreme degree) the way in which $\bar{c}$ and $\bar{\phi}$ can vary with the stress range.

The Mohr envelope for an overconsolidated soil seldom is as curved as that in Fig. 21.10 (Fig. 21.5 is more typical). Usually, the $\bar{\phi}$ from a straight-line fit to an overconsolidated clay is about equal to that for normally consolidated specimens. Moreover, the effects of preconsolidation on strength are important numerically only at small effective stresses; at large effective stresses the envelopes for normally and overconsolidated specimens of a clay tend to merge. Thus the major question with regard to overconsolidated clay lies in choosing the appropriate value of $\bar{c}$.

Values of $\bar{c}$ ranging from 100 to 500 lb/ft^2 are often used

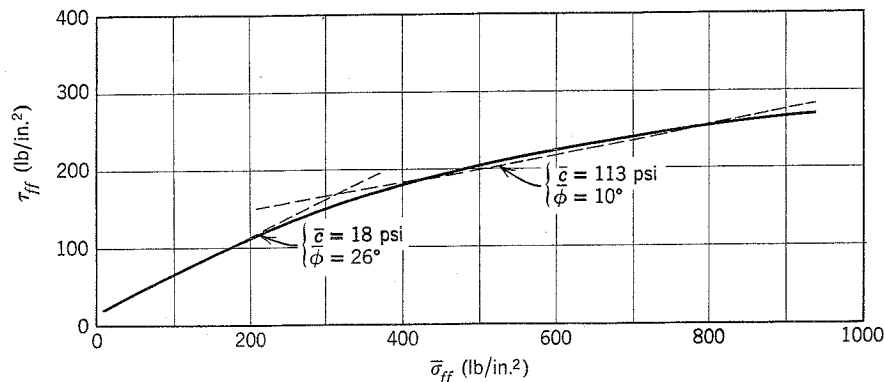


Fig. 21.10 Strength envelope for unweathered London clay (From Bishop, et al., 1965).