I M P O R T A N T

Notice Regarding This PDF File Transmission

If you received this transmission in error, please notify Giles Engineering Associates, Inc., by telephone (1-800-782-0610) or by return e-mail. The information contained in this PDF file is intended solely for the private and confidential use of the designated recipient. If you are not a designated or authorized recipient, further review, dissemination, distribution, or copying of this transmission is strictly prohibited.

To the Authorized Recipient:

This PDF file is an electronic transmission of our final report. An authenticated copy will also be issued to you for your files. The authenticated copy will be affixed with original signatures and/or our corporate seal. While we have taken precautions to assure a complete and secure electronic transmission, please be certain to check this report against the authenticated copy for conformity.

GILES ENGINEERING ASSOCIATES, INC.

Geotechnical Engineering Exploration and Lake Michigan Bluff Stability Analysis

Bender Park
Oakwood Road
Oak Creek, Wisconsin

Prepared for:

Edward E. Gillen, Co.
Milwaukee, Wisconsin

May 25, 2004

Project No. 1G-0309022

GILES
ENGINEERING ASSOCIATES, INC.

GILES
ENGINEERING ASSOCIATES, INC.
GEOTECHNICAL, ENVIRONMENTAL & CONSTRUCTION MATERIALS CONSULTANTS

• Atlanta, GA
• Dallas, TX
• Los Angeles, CA
• Milwaukee, WI
• Orlando, FL
• Washington, D.C.

May 25, 2004

Edward E. Gillen Co.
218 West Becher Street
Milwaukee, WI 53207

Attention: Mr. Gary Jackson

Subject: Geotechnical Engineering Exploration and Lake Michigan Bluff Stability Analysis
Bender Park
Oakwood Road
Oak Creek, Wisconsin
Project No. 1G-0309022

Dear Mr. Jackson:

In accordance with your request and authorization of our Proposal No. 1GP-021005, dated September 25, 2003, we have conducted a Geotechnical Engineering Exploration and Lake Michigan Bluff Stability Analysis for Bender Park in Oak Creek Wisconsin. The conclusions and recommendations developed from our exploration and analysis are discussed in detail in the accompanying report.

We appreciate the opportunity to have been of service on this project. If there are any questions or if we may be of further service, please call.

Very truly yours,

GILES ENGINEERING ASSOCIATES, INC.

Jeffrey Scott Miller

Jeffrey Scott Miller, P.E.
Sr. Project Manager

WISCONSIN
JEFFREY S. MILLER
E-15511
WALES, WIS.
PROFESSIONAL ENGINEER

David M. Maciolek, P.E.
Project Manager

Distribution: Edward E. Gillen Co.
Attention: Mr. Gary Jackson (3)
Milwaukee County Dept. of Parks
Attention: Mr. Jim Ciha (1)

1g0309022-letter/03Geo3/jsm/mgf

N8 W22350 Johnson Drive • Suite A1 • Waukesha, WI 53186
262 / 544-0118 • Fax 262 / 549-5868 • E-Mail milwauke@gilesengr.com

TABLE OF CONTENTS

GEOTECHNICAL ENGINEERING EXPLORATION AND LAKE MICHIGAN BLUFF STABILITY ANALYSIS

BENDER PARK
OAKWOOD ROAD
OAK CREEK, WISCONSIN
PROJECT NO. 1G-0309022

1G0305014toc/03Geo3/jsm

GEOTECHNICAL ENGINEERING EXPLORATION AND LAKE MICHIGAN BLUFF STABILITY ANALYSIS

BENDER PARK
OAKWOOD ROAD
OAK CREEK, WISCONSIN
PROJECT NO. 1G-0309022

1.0 EXECUTIVE SUMMARY

The executive summary is provided solely for overview. Any party who relies on this report must read the full report. The executive summary omits a number of details, any one of which could be crucial to the proper application of this report.

Bluff Stability Analysis and Recommendations

- Evidence of past bluff stability problems is present in the form of barren soil and escarpments on the bluff face.
- With an assumed depressed water level elevation caused by installation of long-term drainage, the bluff stability analyses results indicate that the factor of safety increases by about 4 to 8 percent, relative to the already depressed groundwater levels present due to the dry weather conditions of the last number of years.
- A lowering of and maintaining a low water level within the bluff, development of vegetative growth on the bluff face, maintenance of the bluff toe protection, and elimination of surface water runoff onto the bluff face from the top of the bluff is recommended for stability in the future.
- Horizontally and directionally drilled drains proposed by Giles and the Edward E. Gillen Company to lower the water level and allow drainage for maintaining a lower water level within the bluff on a long-term basis is recommended to be performed.

2.0 SCOPE OF SERVICES

The authorized scope of services performed by us for this project included visual site reconnaissance, subsurface exploration, field and laboratory testing and geotechnical engineering for analysis of the bluff stability, and recommendations for bluff stabilization. This analysis was also conducted to study the effectiveness of an internal water drainage system installation proposed by both Giles and the Edward E. Gillen Company. General comments and other limitations relative to the project are enclosed in Appendix D.

3.0 SITE AND PROJECT DESCRIPTION

The southern portion of Bender Park between Fitzsimmons Road (extended) at the north and Oakwood Road (extended) at the south is about one half mile long and is currently unimproved. Tall grass and brush generally cover the portion west of the bluff with isolated areas of trees generally at the western portions of the park. The bluff face is steep, with immature brush and grass vegetation in some areas, and barren, eroded, and slumped soil in most areas of the face.

Two areas of the bluff were studied for this project. The locations are approximately 600 feet and 2100 feet in distance along the bluff crest and north-northwest of Oakwood Road extended. The two areas are hereafter referred to as the Southern Area and Northern Area, respectively. Topographic information discussed below is shown on the Topographic Map, SE ¼ Section 25 T5N R22E date of mapping January 1988, Southeastern Wisconsin Regional Planning Commission (SEWRPC). This topographic information is the most recently available information for the bluff. The bluff height above Lake Michigan is about 116 feet. Surface topography west of the bluff varies gradually by about 20 feet. No evidence of recent bluff stability problems was present in the ground surface at the top of the bluff. No cracks and no depressions indicative of bluff soil sliding were present in the soil surface, at the time of reconnaissance. The bluff face is at an approximate 1.5 H: 1V slope, at the southern studied area, and about 1.67 H: 1V at the northern studied area. Close observation of the bluff face by walking on the face was not attempted due to the steepness and barren soil condition. The bluff toe has rip-rap protection. Several photographs of the bluff were taken during the original reconnaissance for proposal preparation in August 2003, and are placed in the Giles files for future reference.

Information was obtained from SEWRPC regarding bluff recessional rates and stability in the general area of the property. The information is contained in the SEWRPC Technical Report No. 36, *Lake Michigan Shoreline Recession and Bluff Stability in Southeastern Wisconsin: 1995,* dated December, 1997, and the Community Assistance Planning Report No. 163, *A Lake Michigan Shoreline Erosion Management Plan for Milwaukee County Wisconsin*, dated October 1989. Slope stability calculations performed for the 1989 Report in the southern area of Bender Park resulted in factors of safety of both less than and greater than about 1.0, and therefore this area was characterized as being unstable to marginally stable. Water seepage from the lower portion of the bluff and no shoreline protection present was noted in the 1989 Report. Shoreline recessional data was compiled for the 1997 Report for the period between 1963 and 1995. A recession of between 10 and 400 feet is reported for the 1963 to 1995 time period, which is about 0.3 and 12.5 feet per year average. A beach width ranging from narrow to non-existent was reported for this study area and time frame.

4.0 SUBSURFACE EXPLORATION AND LABORATORY TESTING

Four test borings to characterize the subsoil profile and installation of four piezometers for recording the water level in the soils were performed for this project. The test borings are numbered 1, 1A, 2, and 2A. Test Boring Nos. 1 and 2 were each drilled to a depth of 120± feet below the ground surface, and near the bluff crest. Test Boring Nos. 1A and 2A were each drilled to a depth of 81± feet below the ground surface, and approximately 50 feet west of Test Boring Nos. 1 and 2 respectively. Test Boring Nos. 1 and 2 were drilled with continuous auger core sampling. Test Boring Nos. 1A and 2A were drilled with conventional Standard Penetration Test (ASTM D-1586) sampling. Standard Penetration Test (SPT) sampling was performed in Test Boring No. 1 between 84± and 106± feet in depth where sandy soils were encountered. Due to continuous auger core sampler equipment failure, SPT sampling was performed below 50 feet in depth at Test Boring No. 2.

The piezometers are numbered PZ-1, PZ-1A, PZ-2, and PZ-2A. The piezometers were installed in boreholes located about 5 to 10 feet away from the similarly numbered test borings. The piezometers are pneumatic sensors, placed inside the borehole with the sensor surrounded by a nominal 1 foot long filter sand pack, and with a bentonite clay seal and borehole backfill above the sand pack. The sensors were placed at depths of 80, 60, 75, and 50 feet below the surface at locations PZ-1, PZ-1A, PZ-2, and PZ-2A, respectively. Soil sampling was not performed during borehole drilling for the piezometer installations, except for piezometer PZ-2. At PZ-2, 2 ½ foot SPT sampling was performed between 62 feet in depth and 77 ½ feet, the termination depth, since poor or no soil sample recovery was obtained at nearby Test Boring No. 2.

Two pre-existing observation wells are located about 100 feet north of Test Boring Nos. 2, 2A and piezometers PZ-2 and PZ-2A. The wells are nominal 2 inch diameter standpipe-type wells. The bottom of the wells were determined to be 22± feet and 100 ± feet below the ground surface. Other details of the well construction are not known by Giles. Their bottom depths and periodic measurements of the water levels within the wells were measured by an electric water-level sensor.

The approximate test boring and piezometer locations are indicated on the Boring Location Plan (Figure 1) enclosed in Appendix A. Copies of the Test Boring Logs (Records of Subsurface Exploration) are also enclosed in Appendix A. The elevations shown on the test boring logs were determined by interpolation of the SEWRPC Topography Map.

Laboratory tests performed consist of natural moisture content, in place density, Atterberg Limit index tests, and triaxial shear strength tests. Test results are shown on the Test Boring logs and Figures 2, 3, and 4 enclosed in Appendix A. Field and laboratory test procedures are presented in Appendices B and C, respectively.

The subsurface conditions discussed below were simplified for ease of report interpretation. A more detailed description of the conditions encountered at the test boring locations is provided on the Test Boring Logs enclosed in Appendix B.

Subsoil Conditions

The subsoils encountered at the test boring locations generally consist of silty clay to at least the maximum depths explored. The silty clay is brown in color and has fissures to depths ranging from 10± to 13± feet, and is gray below. Coarse gravel or a cobble was encountered at 25± feet in depth at Test Boring No. 1A. Layers and seams of clay, silt, and very fine sand were encountered between 36± and 93± feet in depth at Test Boring No. 1, and below 42± feet at Test Boring No. 1A. At Test Boring No. 2, layers and seams or lenses of silty clay to clay and silt were encountered between 78± and 114± feet below the surface. Underlying soils encountered at the test borings consist of silty clay with sand and gravel, and sandy clay to the maximum depths explored.

Groundwater Conditions

Free water was encountered at depths of 60± and 45± feet below the surface at Test Boring Nos. 1 and 2, respectively, but did not accumulate after completion of drilling. Free water was not encountered during drilling and did not accumulate after drilling in the other test borings. Piezometric head readings of the piezometers recorded since installation are shown on Table 1 enclosed in Appendix A. Water level recordings of the two standpipe observation wells are also included for reference purposes. The groundwater table level at the time of the last piezometer recording on May 24, 2004 was measured at about El. 653 and El. 641 at PZ-1A and PZ-1, respectively, and at El. 685 and El. 660 at PZ-2A and PZ-2, respectively. A fluctuation in the water table or the development of perched water levels at shallower depths is also anticipated, depending upon the amounts of precipitation, and surface water runoff to the site.

5.0 CONCLUSIONS AND RECOMMENDATIONS

The conditions of the bluff have been evaluated on the basis of the engineering characteristics of the subsurface materials encountered in the test borings. The conclusions of the slope stability analyses, and recommendations for bluff stabilization are discussed in the following sections of this report.

5.1 Groundwater Level Recordings

The groundwater levels within the bluff were measured by determining the piezometeric head which is the pressure from the water level above the pneumatic piezometer sensors placed within the bluff at various elevations. Giles selected pneumatic piezometers for determining the groundwater levels because the response time between groundwater level changes and measurement of the levels in clay is almost instantaneous. This response time is considered to be an advantage in this analysis. The response time is about 42 days or longer for 2 inch diameter standpipe-type observation wells, and were not selected to measure the groundwater levels since a shorter response time is considered necessary to evaluate groundwater level changes.

The groundwater level records are shown on Table 1 enclosed in Appendix A. The initial level readings for piezometers PZ-1 and PZ-1A were obtained a short time after installation and may not be accurate due to the residual water pressures caused by the installation. Subsequent PZ-1 And PZ-1A records and all of the PZ-2 and PZ-2A records are considered valid representations of the groundwater levels at the time of recording.

The groundwater levels recorded indicate a lower groundwater elevation at the piezometers closest to the bluff crest and Lake Michigan. This is considered reasonable since groundwater flow from the higher elevation within in the bluff to the lower lake level elevation is logical.

The groundwater levels measured since piezometer installation are considered to represent a relatively low groundwater level condition. An increase in regional precipitation is generally predicted by the historically cyclical levels of the Great Lakes relative to the recent past near record low Lake Michigan water level. The Lake Michigan water level is expected to rise in the next several years, based on publicly recorded lake level statistics.

The groundwater levels have risen since the recording on December 24, 2003. The most recent recording was performed on May 24, 2004 to compared with the previous recording on May 18, 2003, since moderately heavy rainfall occurred during the time between the dates of the last two recordings. A water level increase of about one foot occurred between May 18, and May 24, 2004 based on the piezometer readings.

5.2 Slope Stability Analyses

Bluff topographic profiles determined in 1988 were used for the analyses, since they are the most recently available topographic information. Some change in the bluff face shape and slope angle most likely has occurred since 1988, as evidenced by the barren, eroded, and slumped soil in most areas of the face. The factors of safety results of the stability analyses are considered representative of the bluff face topography in 1988, and are considered relative to the present conditions.

Slope stability analysis calculations were performed by Giles, and are based on the subsurface conditions and soil properties determined from the test borings and laboratory testing. The bluff heights and horizontal distances from the bluff toe, and the bluff face topography used in the slope stability analyses were obtained from the SEWRPC Topography Map, referenced earlier in this report.

The engineering properties of the subsoils used in the slope stability analysis calculation were determined by field and laboratory tests. The groundwater level elevations relative to the horizontal distance within the bluff was estimated by interpolation between the levels measured at the piezometer locations and the toe of the bluff. The toe of the bluff was used as the groundwater level at the toe location since no water seepage from the bluff face was visually apparent. The calculations were performed with the slope stability analysis computer program STABL5M, developed by Purdue University.

A factor of safety value of less than 1 and greater than 1 was determined by the stability analyses for the bluff slope at the respective Southern and Northern Areas studied, which indicates a possible occurrence of deep rotational slides, and may not be represented on the 1988 topography. The results are shown on Figures 5 and 6 enclosed. This is consistent with the SEWRPC study results of unstable to marginal stability current in the late 1980's.

A large and deep rotational slide is a failure where a large volume of soil, such as 5 to10 or more feet in horizontal width of the ground "breaks off" the top of the crest, and slides down the bluff slope. The potential rotational failure surfaces with a factor of safety values of about 0.87 and 1.06 are shown as the heavy red line on Figures 5 and 6, respectively. Other potential rotational failure surfaces are shown, but indicate higher factor of safety values.

Two other stability calculations were made to evaluate the effect of a lower groundwater level. The results are shown on Figures 7 and 8 enclosed. With a water level depressed below potential rotational failure surfaces, such as by 35 to 50± feet in elevation, the factor of safety increases by about 4 to 8 percent, which indicates an increase in the stability. An increase in the factor of safety is desirable, but is limited due the site geometry and subsoil shear strength.

5.3 Stabilization Recommendations

The bluff is marginally stable to unstable at the current time with the relatively low groundwater levels. However, a lowering of and/or maintaining a low water level within the bluff during the predicted future increased regional precipitation, enabling and maintaining vegetative growth on the bluff face, and elimination of surface water runoff onto the bluff face from the top of the bluff is recommended for increased stability in the future. Bluff toe protection is also beneficial for stability. A 0.3 feet to 12.5 feet per year recessional rate is estimated in the future, based on rates during the time period discussed in the SEWRPC reference. However, this bluff recession rate estimate is based on statistical averaging. Substantially more bluff recession can occur during any one year or event. The actual bluff recession rate can be increased by precipitation and a resulting rise in the water table level in the bluff, and a rise in the level of Lake Michigan. Also, the recession rate can be increased by stabilization work or lack thereof performed on adjacent properties. Fill placed or groins for stabilization constructed from the shoreline outward into the lake can cause detrimental shore erosion, depending upon their positions relative to this property. Massive earth slides on adjacent properties can also increase the bluff recession rate on this property.

Lowering the water level and allowing drainage for maintaining a lower water level within the bluff on a long-term basis is recommended to be performed. A lower water level will increase the stability of the bluff against deep-seated rotational bluff slope failures without massive slope face re-grading. It will also reduce the amount of internal bluff water seepage reaching the bluff face that is one of the causes of bluff face soil erosion and shallow depth instability of the bluff face.

The drainage system is recommended to consist of a number of wick drains that will cost effectively drain to the lake on a long term time frame by gravity without electrically powered pumps. The wick drains are recommended to be installed with horizontal and directional drilling procedures, starting either at the top of the bluff, or from the bluff face near the beach. A further discussion of the drainage system is presented below. Horizontal and directional drilling procedures are recommended so that the water level is intercepted by the drains within the numerous layers and lenses of sand and silt separated by relatively impermeable clay layers and is able to be discharged to the lake.

The location of the interception of the bluff water level is recommended to be west of the potential rotational failure surface. Drilling from the top of the bluff is preferred by Giles over drilling at an angle above horizontal from the bluff face due to several reasons. The reasons are that some excavation and earth grading work on the bluff face would be needed for drilling equipment access, and a possible and undesirable loss of soil (erosion) from within the interior of the bluff could occur during drilling installation.

The drains are planned to consist of a number of synthetic wick drains, bundled together for installation into each drill hole. The wick drains proposed are designed and manufactured specifically for water drainage of fine soils such as sand, silt and clay, similar to the soils encountered at the test boring locations at this site. Bundling a number of the drains together for installation within each drilled drain is recommended to reduce the void space between the soils and the drain and long term loss of soil by movement within the drain drill holes. Discharge of the drains near the beach is recommended to include a non-woven geotextile placed on grade and crushed rock rip-rap for erosion scour protection. Some periodic maintenance of the discharge ends are recommended, consisting of removal of accumulated vegetation debris and eroded soils to maintain free water drainage.

The entire bluff between Oakwood Road and Fitzsimmons Road is recommended to be drained with wicks. However, additional test boring exploration and piezometer installation is recommended on about 600 foot centers along the bluff crest to validate the conditions encountered at the two bluff locations that were currently studied, and to assist in determining the most beneficial wick locations. The piezometers are recommended to determine the initial groundwater level conditions, and to evaluate the affects of the wick drainage for a long time period. For the purposes of this current study, two to three wicks at about a 50 foot spacing at both of the north and south study locations is recommended.

Installation of the drains is recommended to be done as soon as reasonable, while the Lake

Michigan water levels are low. The drains will be effective immediately within the sandy soils upon installation. However, a decrease in the bluff water levels will take a period of time to occur due to the slow permeability of the silt and clay soils. This will allow drainage to be in place when the regional precipitation, Lake Michigan water level, and therefore the internal bluff water levels rise in the future. The success of the wick drainage can be measured by the groundwater level measurements determined by the piezometers.

The installation of the drains is intended to lower the water level and provide a long term condition of drainage and therefore lower water level within the bluff to increase the bluff stability. However, surface erosion and shallow depth slow downward soil movement on the bluff face may still continue. Planting or enabling and maintenance of living tree and brush or other deep rooted vegetation on the bluff face is recommended to reduce the erosion and soil movements.

Grading at the top of the bluff area is recommended to be sloped away from the bluff crest. Water runoff over the crest onto the bluff face from precipitation is recommended to be eliminated.

Other forms of stabilization exist, but are considered to include some possible disadvantages for this site. They consist of massive earth cut of soils on the bluff to reduce the bluff steepness, and/or placement of fill to provide a stabilizing "counterbalance" at the bottom of the bluff, extending into the lake. However, the earth cut and/or landfilling is a significant disturbance. Landfilling extension into Lake Michigan will be limited by local, state and federal regulations. A significant amount of earth cut and/or landfilling is necessary to counter effect the water levels within the bluff that will fluctuate in elevation and therefore result in a destabilizing force.

1g0309022-report/03Geo3/jsm

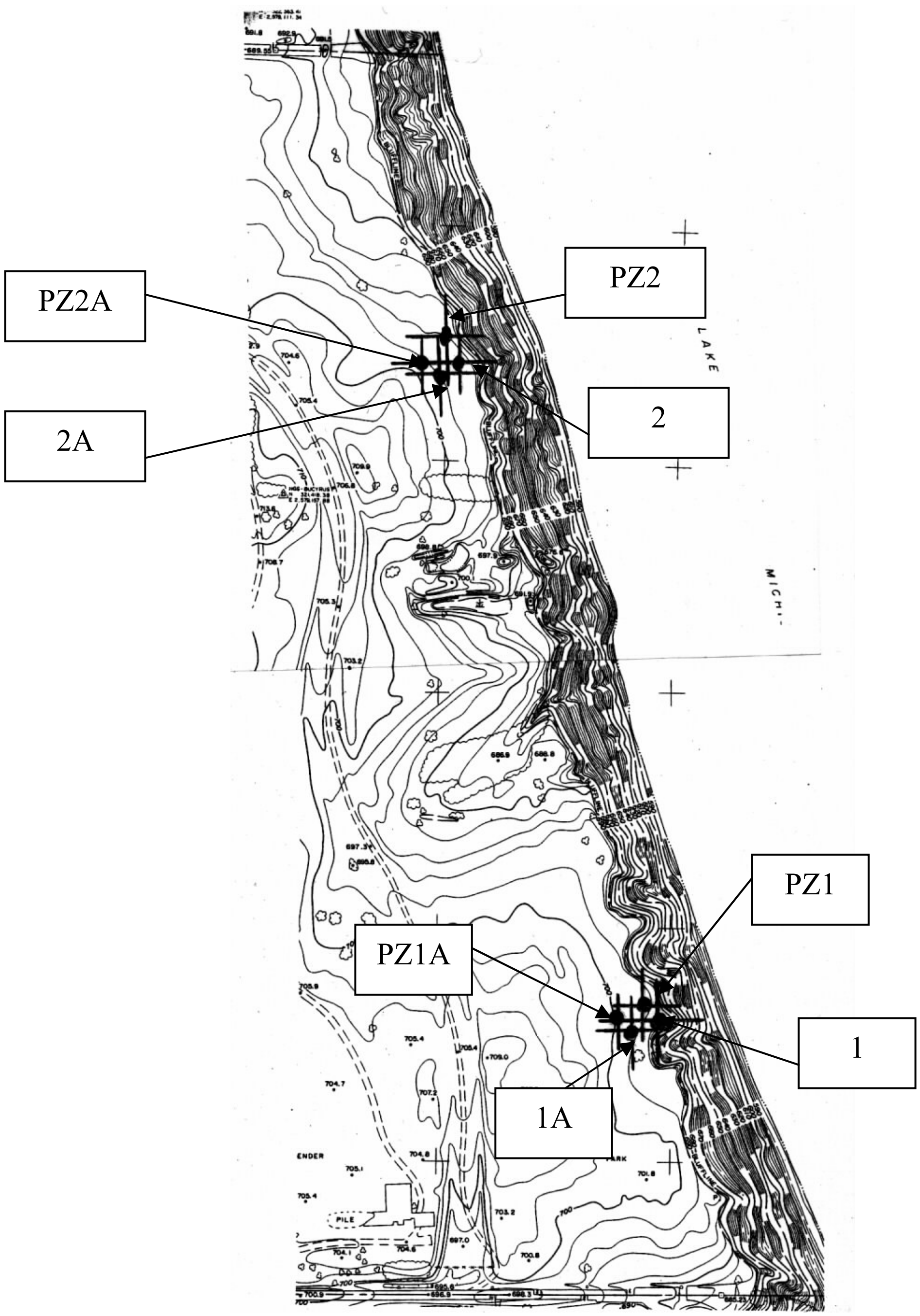
A P P E N D I X   A

FIGURES AND TEST BORING LOGS

The Boring Location Plan contained herein was prepared based upon information supplied by Giles's client, or others, along with Giles's field measurements and observations. The diagram is presented for conceptual purposes only and is intended to assist the reader in report interpretation.

The Test Boring Logs and related information enclosed herein depict the subsurface (soil and water) conditions encountered at the specific boring locations on the date that the exploration was performed. Subsurface conditions may differ between boring locations and within areas of the site that were not explored with test borings. The subsurface conditions may also change at the boring locations over the passage of time.

Notes: (1) This Approximate Boring Location Plan was adapted from the Topographic Map, SE ¼ Sec. 25, T5N, R22E, Milwaukee County, Wisconsin



APPROXIMATE BORING LOCATION PLAN

Approximate Scale
1 inch = 400 feet

N

Bender Park Lake Michigan Bluff
Oak Creek, Wisconsin

Project No. 1G-0309022

GILES ENGINEERING ASSOCIATES, INC.

Giles Engineering Associates, Inc.

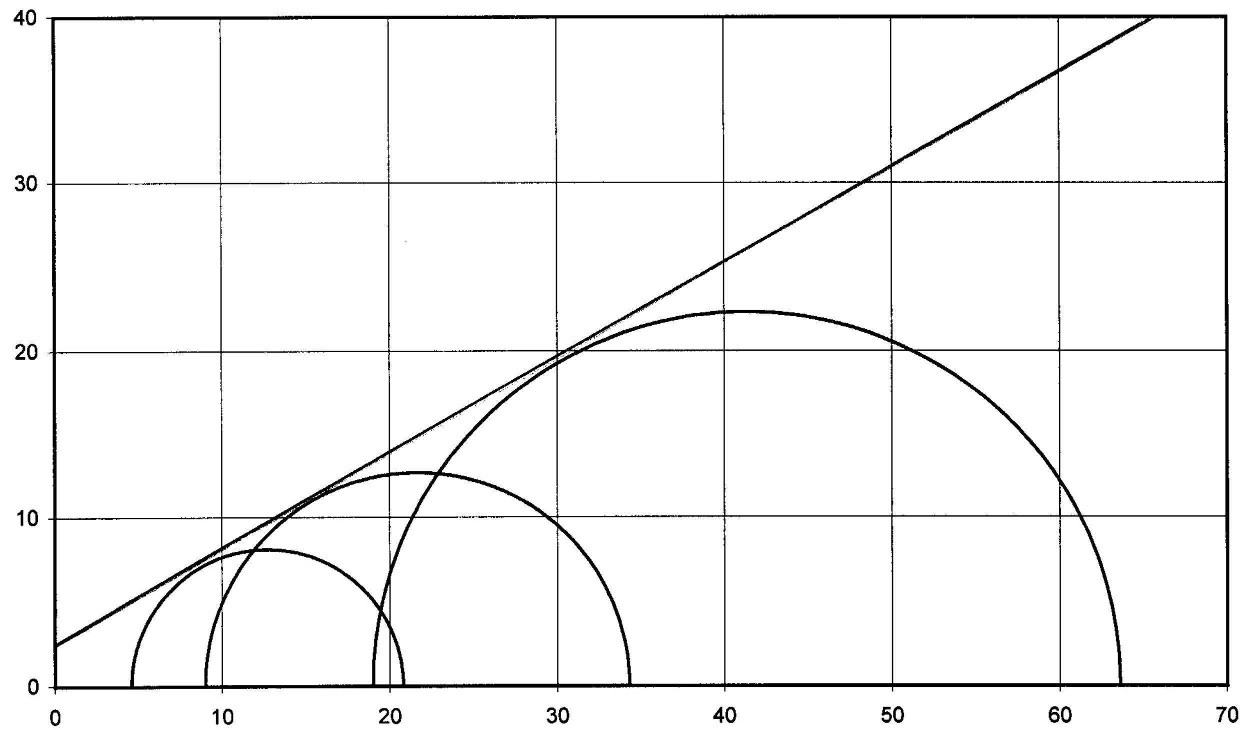
GEOTECHNICAL, ENVIRONMENTAL AND CONSTRUCTION MATERIALS CONSULTANTS

N8 W22350 JOHNSON ROAD, SUITE A1/WAUKESHA, WI 53186/(262) 544-0118/FAX: (262) 549-5868

ATLANTA, GA / DALLAS, TX / ANAHEIM, CA / MADISON, WI / COLUMBIA, MD / SANFORD, FL

SHEAR TEST

Consolidated Undrained (ASTM D4767) (with pore pressure measurement)



Classification	Gray Silty Clay		
Boring No.	1	Sample No. 4 AC	Depth (ft.) 15
		Initial Specimen Properties:	
PROJECT:	Bender Park Bluff Evaluation Oak Creek, WI	Diameter (in.)	2.18
		Height (in.)	4.51
		Moisture Content (%)	15.3
		Natural Density (pcf)	138.3
CLIENT:	Edward E. Gillen Co. Inc.	Dry Density (pcf)	119.9
		LL	31
PROJECT NO.:	1G-0309022	PL	14
		C (psi/psf)	2.16 / 311
FIGURE NO.:	2	PHI (degrees)	30

Giles Engineering Associates, Inc.

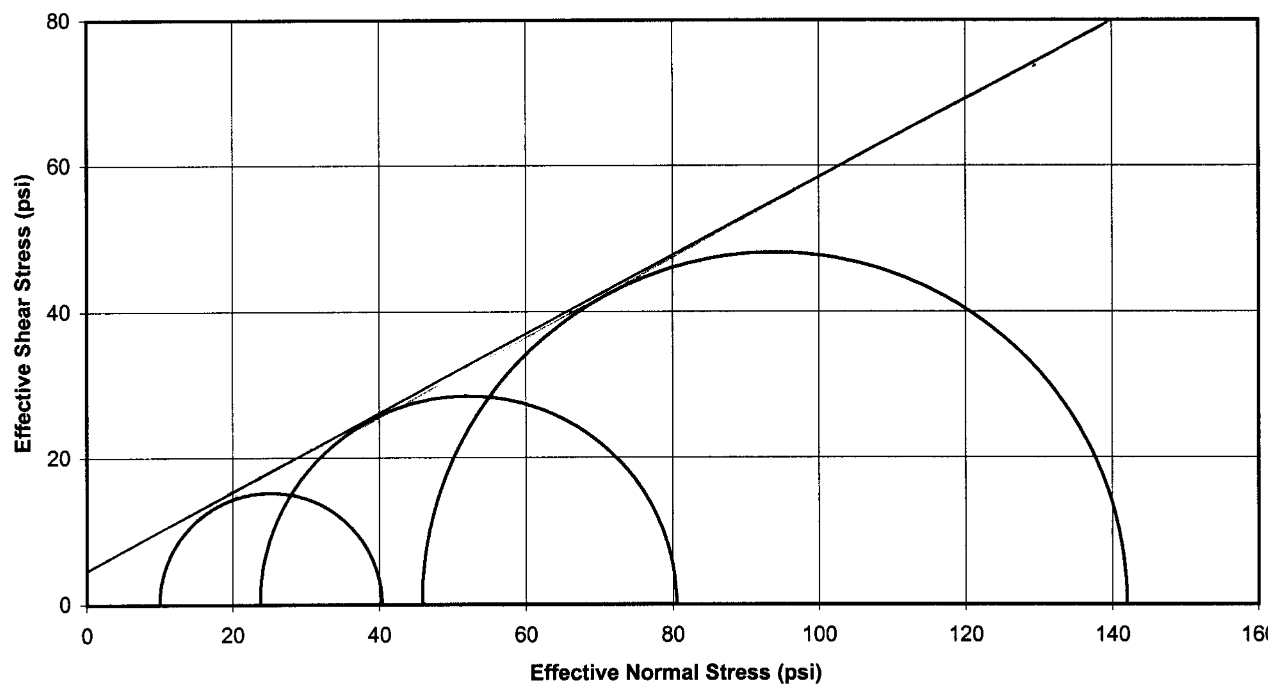
GEOTECHNICAL, ENVIRONMENTAL AND CONSTRUCTION MATERIALS CONSULTANTS

N8 W22350 JOHNSON ROAD, SUITE A1/WAUKESHA, WI 53186/(262) 544-0118/FAX: (262) 549-5868

ATLANTA, GA / DALLAS, TX / ANAHEIM, CA / MADISON, WI / COLUMBIA, MD / SANFORD, FL

SHEAR TEST

Consolidated Undrained (ASTM D4767) (with pore pressure measurement)



Classification	Gray Silty Clay to Clay		
Boring No.	1 Sample No.	13 AC Depth (ft.)	63
		Initial Specimen Properties:	
PROJECT:	Bender Park Bluff Evaluation	Diameter (in.)	2.20
	Oak Creek, WI	Height (in.)	4.75
		Moisture Content (%)	20.5
		Natural Density (pcf)	129.8
CLIENT:	Edward E. Gillen Co. Inc.	Dry Density (pcf)	107.7
		LL	36
PROJECT NO.:	1G-0309022	PL	16
		C (psi/psf)	3.97 / 571
FIGURE NO.:	3	PHI (degrees)	29.5

Giles Engineering Associates, Inc.

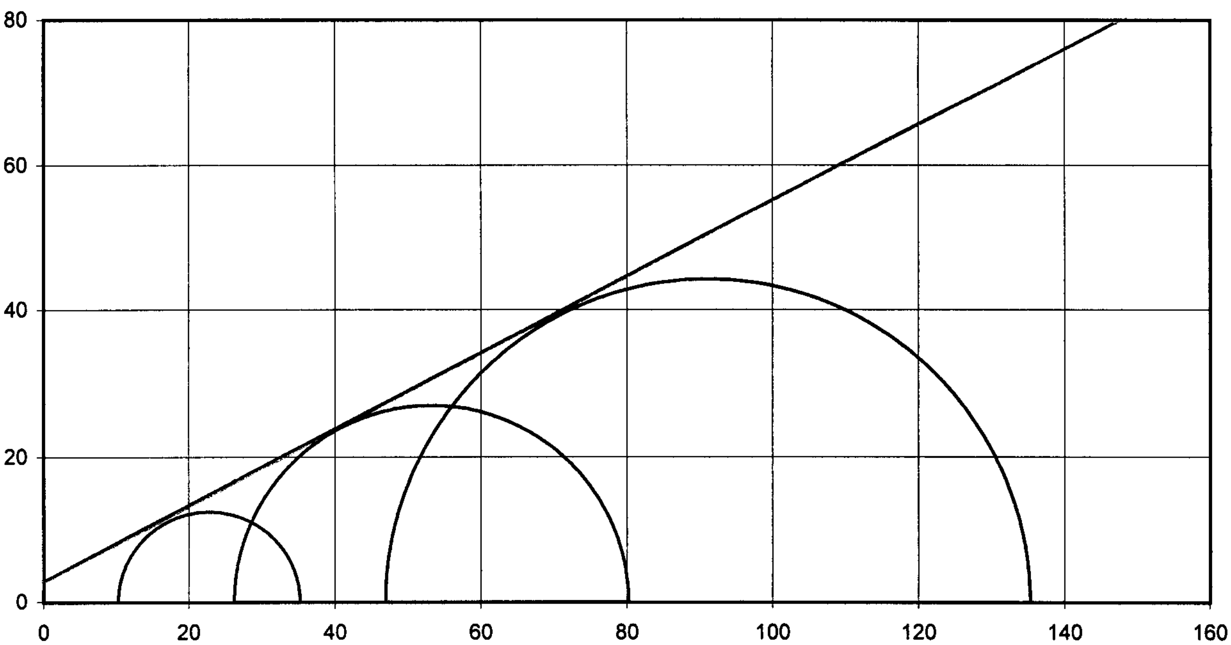
GEOTECHNICAL, ENVIRONMENTAL AND CONSTRUCTION MATERIALS CONSULTANTS

N8 W22350 JOHNSON ROAD, SUITE A1/WAUKESHA, WI 53186/(262) 544-0118/FAX: (262) 549-5868

ATLANTA, GA / DALLAS, TX / ANAHEIM, CA / MADISON, WI / COLUMBIA, MD / SANFORD, FL

SHEAR TEST

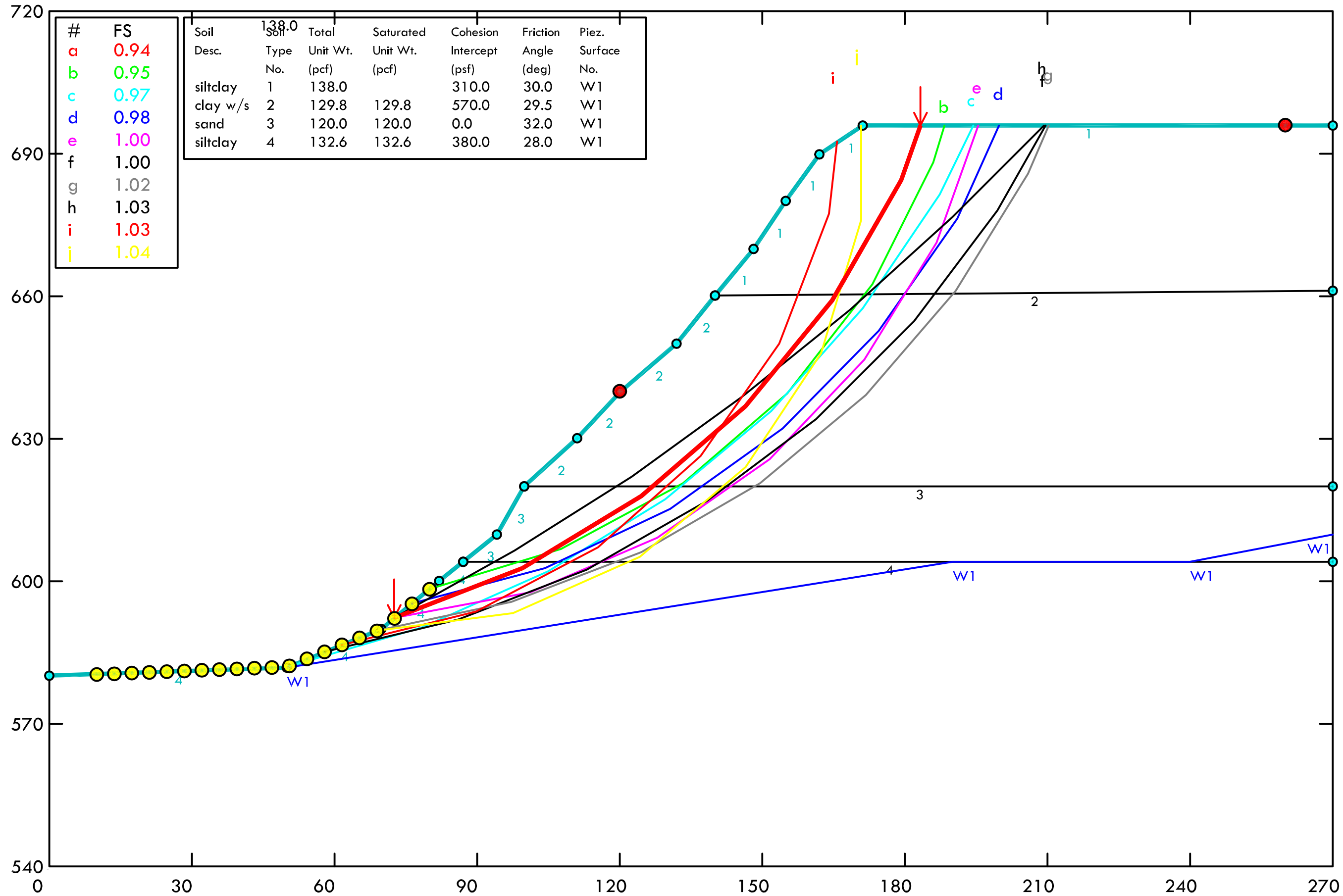
Consolidated Undrained (ASTM D4767) (with pore pressure measurement)



Classification	Gray Silty Clay, trace Gravel		
Boring No.	1 Sample No.	23 AC Depth (ft.)	113.5
		Initial Specimen Properties:	
PROJECT:	Bender Park Bluff Evaluation	Diameter (in.)	2.27
	Oak Creek, WI	Height (in.)	4.57
		Moisture Content (%)	19.6
		Natural Density (pcf)	132.7
CLIENT:	Edward E. Gillen Co. Inc.	Dry Density (pcf)	110.9
		LL	29
PROJECT NO.:	1G-0309022	PL	14
		C (psi/psf)	2.64 / 381
FIGURE NO.:	4	PHI (degrees)	28

Bender Park Bluff Evaluation South Area Depressed Water Level

c:\slope\stedwin279\jobs2004\0309022b.pl2   Run By: Username   5/25/2004   09:20AM



Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
siltclay	1	138.0		310.0	30.0	W1
clay w/s	2	129.8	129.8	570.0	29.5	W1
sand	3	120.0	120.0	0.0	32.0	W1
siltclay	4	132.6	132.6	380.0	28.0	W1

PCSTABL5M/si FSmin=0.94

Safety Factors Are Calculated By The Modified Bishop Method

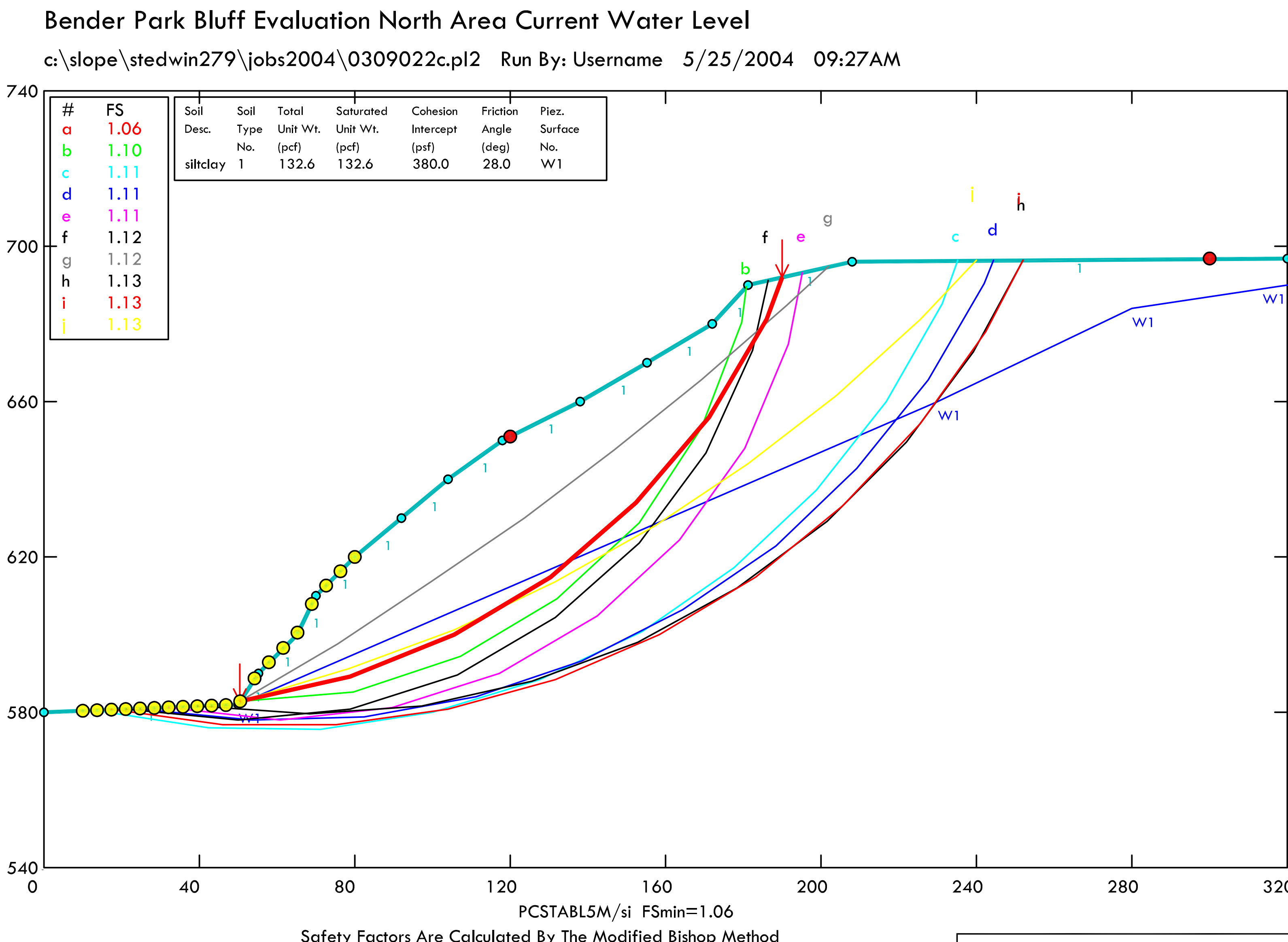
LAKE MICHIGAN BLUFF STABILITY ANALYSIS
BENDER PARK
OAK CREEK, WISCONSIN

GILES ENGINEERING ASSOCIATES, INC.
N8 W22350 JOHNSON RD.; WAUKESHA, WI, 53186
(414)-544-0118

FIGURE 7
BLUFF STABILITY ANALYSIS RESULTS

DESIGNED	DRAWN	APPROVED	SCALE	DATE
JSM	JSM		NONE	5-25-2004

PROJECT NO.: 1G-0309022 | CAD NO.: 0309022_F07



LAKE MICHIGAN BLUFF STABILITY ANALYSIS
BENDER PARK
OAK CREEK, WISCONSIN

GILES ENGINEERING ASSOCIATES, INC.
N8 W22350 JOHNSON RD.; WAUKESHA, WI, 53186
(414)-544-0118

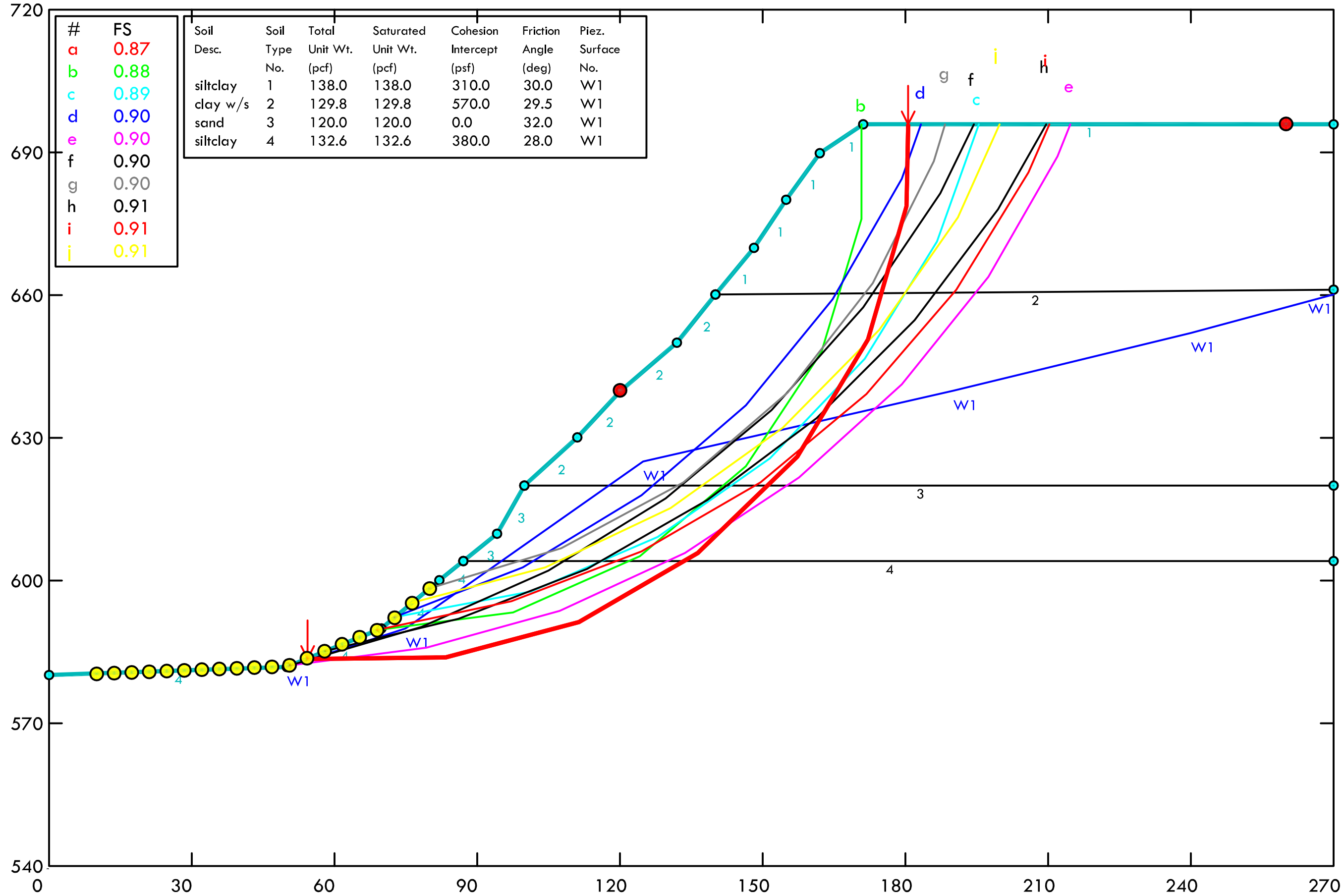
FIGURE 6
BLUFF STABILITY ANALYSIS RESULTS

DESIGNED	DRAWN	APPROVED	SCALE	DATE
JSM	JSM		NONE	5-25-2004

PROJECT NO.: 1G-0309022 | CAD NO.: 0309022_F06

Bender Park Bluff Evaluation South Area Current Water Level

c:\slope\stedwin279\jobs2004\0309022a.pl2 Run By: Username 5/25/2004 09:18AM



PCSTABL5M/si FSmin=0.87

Safety Factors Are Calculated By The Modified Bishop Method

LAKE MICHIGAN BLUFF STABILITY ANALYSIS
BENDER PARK
OAK CREEK, WISCONSIN

GILES ENGINEERING ASSOCIATES, INC.
N8 W22350 JOHNSON RD.; WAUKESHA, WI, 53186
(414)-544-0118

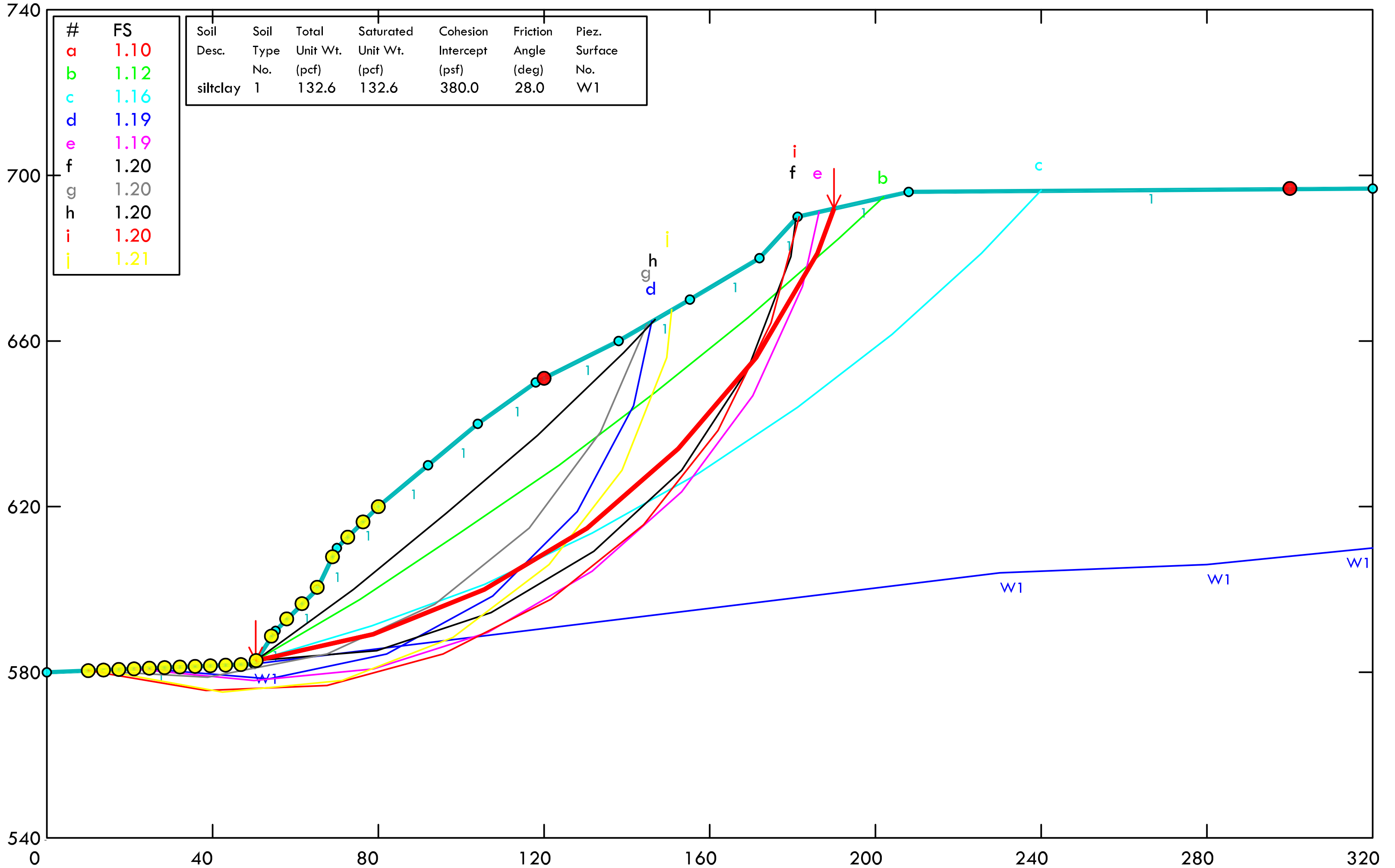
FIGURE 5
BLUFF STABILITY ANALYSIS RESULTS

DESIGNED	DRAWN	APPROVED	SCALE	DATE
JSM	JSM		NONE	5-25-2004

PROJECT NO.: 1G-0309022 | CAD NO.: 0309022_F05

Bender Park Bluff Evaluation North Area Depressed Water Level

c:\slope\stedwin279\jobs2004\0309022d.pl2   Run By: Username   5/25/2004   09:31AM



PCSTABL5M/si FSmin=1.10

Safety Factors Are Calculated By The Modified Bishop Method

LAKE MICHIGAN BLUFF STABILITY ANALYSIS
BENDER PARK
OAK CREEK, WISCONSIN

GILES ENGINEERING ASSOCIATES, INC.
N8 W22350 JOHNSON RD.; WAUKESHA, WI, 53186
(414)-544-0118

FIGURE 8
BLUFF STABILITY ANALYSIS RESULTS

DESIGNED	DRAWN	APPROVED	SCALE	DATE
JSM	JSM		NONE	5-25-2004

PROJECT NO.: 1G-0309022   CAD NO.: 0309022_F08

TABLE 1
LAKE MICHIGAN BLUFF STABILITY ANALYSIS
BENDER PARK, OAK CREEK, WISCONSIN
1G-0309022

Piezometer Record												
Piezometer Location	Ground Surface Elevation	Piezometer Elevation	Pressure (psi) or Depth (feet)					Water Level Elevation				
			10-2-03	11-21-03	12-24-03	5-18-04	5-24-04	10-2-03	11-21-03	12-24-03	5-18-04	5-24-04
PZ-1	697	617	8.01	7.94	8.05	10.12	10.30	635	635	636	640	641
PZ-1A	698	638	1.00	3.84	4.15	6.10	6.40	640	647	648	652	653
PZ-2	696	621	--	14.96	14.79	16.80	17.10	--	656	655	660	660
PZ-2A	698	648	--	12.37	13.37	15.60	15.90	--	677	679	684	685
STS-1	695	--	49.00	--	48.20	46.90	46.70	646	--	647	648	648
STS-2	695	--	8.50		12.90	0.50	1.00	687	--	682	695	694

RECORD OF SUBSURFACE EXPLORATION

BORING NO. & LOCATION: 1	PROJECT: Bender Park Bluff Evaluation	**GILES ENGINEERING ASSOCIATES, INC.**
SURFACE ELEVATION: 697.0	PROJECT LOCATION: Fitzsimmons and Oakwood Roads	**Milwaukee Los Angeles**
COMPLETION DATE: 9/29/03	Oak Creek, Wisconsin	**Madison Dallas Atlanta**
FIELD REPRESENTATIVE: Beauford Jones	GILES PROJECT NUMBER: 1G-0309022	**Washington, D.C. Orlando**

MATERIAL DESCRIPTION	Feet Below Surface	Sample No. & Type	N	q_u (tsf)	q_p (tsf)	q_s (tsf)	W (%)	PID	NOTES
3"± Dark Brown Silty Clay to Clayey Silt and roots (Topsoil)-Damp		1-AC	--						
Brown Silty Clay, trace fine Sand (contains fissures)-Damp	5	2-AC	--						
	10	3-AC	--						
Brown Silty Clay, trace fine Sand and Gravel-Damp							16		Dd = 117.4
Brown to Gray Silty Clay, trace fine Sand and Gravel-Damp to Moist									
Gray Silty Clay-Moist	15	4-AC	--				15		See Figure 2 Dd = 119.9 LL = 31, PL = 14
	20	5-AC	--						(a)
	25	6-ST	--						
	30	7-AC	--				16		Dd = 114.8
Gray Silty Clay, trace fine to coarse Sand and Gravel-Moist	35	8-AC	--						
Gray Silty Clay-Moist	40	9-AC	--						
(thin, damp lens or seam of Gray Silt at 43± feet)									

WATER OBSERVATION DATA	REMARKS
WATER ENCOUNTERED DURING DRILLING: 60.0 ft.	(a) No sample recovery.
WATER LEVEL AFTER REMOVAL: None	
CAVE DEPTH AFTER REMOVAL: None	AC = Auger Core
WATER LEVEL AFTER HOURS:	
CAVE DEPTH AFTER HOURS:	

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between borings. Location of Test Boring is shown on the Boring Location Plan.

RECORD OF SUBSURFACE EXPLORATION

BORING NO. & LOCATION:	PROJECT:
1	Bender Park Bluff Evaluation
SURFACE ELEVATION:	PROJECT LOCATION:
697.0	Fitzsimmons and Oakwood Roads
COMPLETION DATE:	
9/29/03	Oak Creek, Wisconsin
FIELD REPRESENTATIVE:	
Beauford Jones	GILES PROJECT NUMBER: 1G-0309022

GILES ENGINEERING ASSOCIATES, INC.
Milwaukee Los Angeles
Madison Dallas Atlanta
Washington, D.C. Orlando

MATERIAL DESCRIPTION	Feet Below Surface	Sample No. & Type	N	q_u (tsf)	q_p (tsf)	q_s (tsf)	W (%)	PID	NOTES
Gray Silty Clay, trace fine Sand and Gravel-Damp to Moist	45	10-AC	--						
(thin, damp lens or seam of very fine Sand at 47½± feet)									
Gray Silty Clay-Damp to Moist (thin, damp lens or seam of very fine Sand at 49± feet)	50	11-AC	--						
(thin, wet seam or lens of Silt at 53± feet)									
Gray Silty Clay with thin Silt lenses-Moist							16		Dd = 116.9
	55	12-AC	--						
Gray Silt-Wet									
Gray Silty Clay to Clay with thin Silt lenses-Moist	∇ 60	13-AC	--						
Gray Silt, trace Clay-Wet									
Gray Silty Clay to Clay-Moist							21		See Figure 3 Dd = 107.7 LL = 36, PL = 16
Gray fine Sand-Wet	65	14-AC	--						
Gray very fine Sand-Wet									
Gray Silty Clay to Clayey Silt, trace fine Sand-Moist to Wet									
Gray Silty Clay, trace fine to coarse Sand and Gravel-Moist	70	15-AC	--						
	75	16-AC	--						
Gray fine Sand-Wet	80								
Gray very fine Sand, trace Silt-Wet	85	17-SS	15						

WATER OBSERVATION DATA	REMARKS
∇ WATER ENCOUNTERED DURING DRILLING: 60.0 ft.	(a) No sample recovery.
WATER LEVEL AFTER REMOVAL: None	
CAVE DEPTH AFTER REMOVAL: None	AC = Auger Core
WATER LEVEL AFTER HOURS:	
CAVE DEPTH AFTER HOURS:	

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between borings. Location of Test Boring is shown on the Boring Location Plan.

RECORD OF SUBSURFACE EXPLORATION

BORING NO. & LOCATION: 1	PROJECT: Bender Park Bluff Evaluation
SURFACE ELEVATION: 697.0	PROJECT LOCATION: Fitzsimmons and Oakwood Roads
COMPLETION DATE: 9/29/03	Oak Creek, Wisconsin
FIELD REPRESENTATIVE: Beauford Jones	GILES PROJECT NUMBER: 1G-0309022

GILES ENGINEERING ASSOCIATES, INC.
Milwaukee Los Angeles
Madison Dallas Atlanta
Washington, D.C. Orlando

MATERIAL DESCRIPTION	Feet Below Surface	Sample No. & Type	N	q_u (tsf)	q_p (tsf)	q_s (tsf)	W (%)	PID	NOTES
Gray very fine Sand, trace Silt-Wet *(continued)*									
Gray very fine Sand-Wet	90	18-SS	16						
Gray Silty Clay to Clay-Moist	95	19-SS	24						
	100	20-SS	23						
	105	21-SS	17						
		22-AC	--						
Gray Silty Clay, trace to little fine to coarse Sand and Gravel-Moist									
Gray Silty Clay, trace Gravel-Moist	110	23-AC	--						
							20		See Figure 4 Dd = 110.9 LL = 29, PL = 14
	115	24-AC	--						
Boring terminated at 120 feet	120								

WATER OBSERVATION DATA	REMARKS
WATER ENCOUNTERED DURING DRILLING: 60.0 ft.	(a) No sample recovery.
WATER LEVEL AFTER REMOVAL: None	AC = Auger Core
CAVE DEPTH AFTER REMOVAL: None	
WATER LEVEL AFTER HOURS:	
CAVE DEPTH AFTER HOURS:	

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between borings. Location of Test Boring is shown on the Boring Location Plan.

RECORD OF SUBSURFACE EXPLORATION

BORING NO. & LOCATION: 1A	PROJECT: Bender Park Bluff Evaluation	GILES ENGINEERING ASSOCIATES, INC.
SURFACE ELEVATION: 698.0	PROJECT LOCATION: Fitzsimmons and Oakwood Roads	Milwaukee Los Angeles
COMPLETION DATE: 9/30/03	Oak Creek, Wisconsin	Madison Dallas Atlanta
FIELD REPRESENTATIVE: Beauford Jones	GILES PROJECT NUMBER: 1G-0309022	Washington, D.C. Orlando

MATERIAL DESCRIPTION	Feet Below Surface	Sample No. & Type	N	q_u (tsf)	q_p (tsf)	q_s (tsf)	W (%)	PID	NOTES
2"± Brown Silty Clay, trace roots (Topsoil)-Damp		1-SS	10						
Brown Silty Clay, trace fine to coarse Sand and Gravel (contains fissures)-Damp	5	2-SS	27						
Light Brown Silty Clay, trace fine to coarse Sand and Gravel-Damp	10	3-SS	30						
Gray Silty Clay to Clay-Moist	15	4-SS	14						
Gray Silty Clay, trace fine Sand-Moist	20	5-SS	13						
(possibly contains coarse Gravel or Cobbles at 25± feet)	25	6-SS	14						(a)
	30	7-SS	10						
	35	8-SS	13						
	40	9-SS	21						(a)
Gray Silty Clay to Clay with thin seams and lenses of Silt-Moist	45	10-SS	10						
	50	11-SS	12						
	55	12-SS	27						
	60	13-SS	18						

WATER OBSERVATION DATA	REMARKS
WATER ENCOUNTERED DURING DRILLING: None	(a) Poor sample recovery.
WATER LEVEL AFTER REMOVAL: None	
CAVE DEPTH AFTER REMOVAL: 73.0 ft.	
WATER LEVEL AFTER HOURS:	
CAVE DEPTH AFTER HOURS:	

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between borings. Location of Test Boring is shown on the Boring Location Plan.

NORMAL BORING LOGS 1G0309022.GPJ GIL_CORP.GDT 1/7/05

RECORD OF SUBSURFACE EXPLORATION

BORING NO. & LOCATION: 1A	PROJECT: Bender Park Bluff Evaluation
SURFACE ELEVATION: 698.0	PROJECT LOCATION: Fitzsimmons and Oakwood Roads
COMPLETION DATE: 9/30/03	Oak Creek, Wisconsin
FIELD REPRESENTATIVE: Beauford Jones	GILES PROJECT NUMBER: 1G-0309022

GILES ENGINEERING ASSOCIATES, INC.

Milwaukee Los Angeles Madison Dallas Atlanta Washington, D.C. Orlando

MATERIAL DESCRIPTION	Feet Below Surface	Sample No. & Type	N	q_u (tsf)	q_p (tsf)	q_s (tsf)	W (%)	PID	NOTES
Gray Silty Clay to Clay with thin seams and lenses of Silt-Moist *(continued)*	65	14-SS	19						
	70	15-SS	22						
Gray very fine Sandy Silt, trace medium to coarse Sand-Wet	75	16-SS	15						
Gray Silty Clay, trace fine to coarse Sand and Gravel-Moist	80	17-SS	14						

Boring terminated at 81 feet

WATER OBSERVATION DATA	REMARKS
WATER ENCOUNTERED DURING DRILLING: None	(a) Poor sample recovery.
WATER LEVEL AFTER REMOVAL: None	
CAVE DEPTH AFTER REMOVAL: 73.0 ft.	
WATER LEVEL AFTER HOURS:	
CAVE DEPTH AFTER HOURS:	

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between borings. Location of Test Boring is shown on the Boring Location Plan.

RECORD OF SUBSURFACE EXPLORATION

BORING NO. & LOCATION: 2	PROJECT: Bender Park Bluff Evaluation
SURFACE ELEVATION: 696.0	PROJECT LOCATION: Fitzsimmons and Oakwood Roads
COMPLETION DATE: 10/2/03	Oak Creek, Wisconsin
FIELD REPRESENTATIVE: Beauford Jones	GILES PROJECT NUMBER: 1G-0309022

GILES ENGINEERING ASSOCIATES, INC.
Milwaukee Los Angeles
Madison Dallas Atlanta
Washington, D.C. Orlando

MATERIAL DESCRIPTION	Feet Below Surface	Sample No. & Type	N	q_u (tsf)	q_p (tsf)	q_s (tsf)	W (%)	PID	NOTES
12"± Brown Clayey Silt, some roots (Topsoil)-Moist		1-AC	--						
Brown Silty Clay, trace fine Sand (contains fissures)-Damp	5	2-AC	--						
	10	3-AC	--						
Gray Silty Clay, trace Gravel-Damp to Moist	15	4-AC	--						
Gray Silty Clay-Damp to Moist	20	5-AC	--						
	25	6-AC	--						
	30	7-AC	--						
	35	8-AC	--						
	40	9-AC	--						
(Silt and very fine Sand lenses at 45± feet)	▽ 45	10-AC	--						
Gray Silty Clay, trace fine to coarse Sand and Gravel-Moist									
	50	11-AC	--						
	55	12-AC	--						
	60	13-AC	--						

WATER OBSERVATION DATA	REMARKS
▽ WATER ENCOUNTERED DURING DRILLING: 45.0 ft.	(a) No sample recovery.
▼ WATER LEVEL AFTER REMOVAL: None	(b) Poor sample recovery.
CAVE DEPTH AFTER REMOVAL: None	(c) Sample disturbed by drill shoe failure.
▼ WATER LEVEL AFTER HOURS:	AC = Auger Core
CAVE DEPTH AFTER HOURS:	

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between borings. Location of Test Boring is shown on the Boring Location Plan.

RECORD OF SUBSURFACE EXPLORATION

BORING NO. & LOCATION: 2	PROJECT: Bender Park Bluff Evaluation
SURFACE ELEVATION: 696.0	PROJECT LOCATION: Fitzsimmons and Oakwood Roads
COMPLETION DATE: 10/2/03	Oak Creek, Wisconsin
FIELD REPRESENTATIVE: Beauford Jones	GILES PROJECT NUMBER: 1G-0309022

GILES ENGINEERING ASSOCIATES, INC.
Milwaukee Los Angeles
Madison Dallas Atlanta
Washington, D.C. Orlando

MATERIAL DESCRIPTION	Feet Below Surface	Sample No. & Type	N	q_u (tsf)	q_p (tsf)	q_s (tsf)	W (%)	PID	NOTES
Gray Silty Clay, trace fine to coarse Sand and Gravel-Moist *(continued)*		14-AC	--						(a)
	70	15-AC	--						(a)
	75	16-ST	--						(b)
Gray Silty Clay to Clay-Moist	80	17-AC	--						
(failure of drill shoe at 85± feet)	85	18-AC	--						(c)
	90	19-SS	12						
		20-SS	12						
	95	21-SS	14						
Gray Silty Clay to Clay with thin seams and lenses of Silt-Moist		22-SS	20						
		23-SS	16						
	100	24-SS	24						
		25-SS	18						
Gray Silty Clay to Clay-Moist	105	26-SS	12						
(possible drill shoe encountered at 106± to 108± feet)		27-SS	30						(a)
		28-SS	23						(b)
Gray Silty Clay to Clay with thin seams and lenses of Silt-Moist	110	29-SS	18						
Gray Silty Clay to Clayey Silt-Moist		30-SS	21						
Gray Silty Clay, trace fine Sand and Gravel-Moist	115	31-SS	26						
		32-SS	21						
		33-SS	32						
Boring terminated at 120 feet	120								

WATER OBSERVATION DATA	REMARKS
WATER ENCOUNTERED DURING DRILLING: 45.0 ft.	(a) No sample recovery.
WATER LEVEL AFTER REMOVAL: None	(b) Poor sample recovery.
CAVE DEPTH AFTER REMOVAL: None	(c) Sample disturbed by drill shoe failure.
WATER LEVEL AFTER HOURS:	AC = Auger Core
CAVE DEPTH AFTER HOURS:	

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between borings. Location of Test Boring is shown on the Boring Location Plan.

RECORD OF SUBSURFACE EXPLORATION

BORING NO. & LOCATION: 2A	PROJECT: Bender Park Bluff Evaluation
SURFACE ELEVATION: 698.0	PROJECT LOCATION: Fitzsimmons and Oakwood Roads
COMPLETION DATE: 10/6/03	Oak Creek, Wisconsin
FIELD REPRESENTATIVE: Beauford Jones	GILES PROJECT NUMBER: 1G-0309022

GILES ENGINEERING ASSOCIATES, INC.
Milwaukee Los Angeles
Madison Dallas Atlanta
Washington, D.C. Orlando

MATERIAL DESCRIPTION	Feet Below Surface	Sample No. & Type	N	q_u (tsf)	q_p (tsf)	q_s (tsf)	W (%)	PID	NOTES
3"± Dark Brown Clayey Silt and roots (Topsoil)-Damp		1-SS	12						
Brown Clayey Silt to Silty Clay (contains fissures)-Damp	5	2-SS	16						
Brown slightly mottled Orange-Brown Clayey Silt (contains fine Sand and Silt seams)-Moist to Wet									
Brown Clayey Silt to Silt-Wet	10	3-SS	14						
Gray Clayey Silt to Silt-Wet									
Gray Clayey Silt, some fine to coarse Sand-Moist to Wet	15	4-SS	15						
	20	5-SS	9						(a)
Gray Silty Clay, little to some fine to coarse Sand and Gravel-Moist	25	6-SS	10						
Gray Silty Clay, trace fine to coarse Sand and Gravel-Moist	30	7-SS	14						
	35	8-SS	19						
	40	9-SS	14						
	45	10-SS	16						
	50	11-SS	11						
	55	12-SS	18						
	60	13-SS	18						

WATER OBSERVATION DATA	REMARKS
WATER ENCOUNTERED DURING DRILLING: None	(a) Poor sample recovery.
WATER LEVEL AFTER REMOVAL: None	
CAVE DEPTH AFTER REMOVAL: None	
WATER LEVEL AFTER HOURS:	
CAVE DEPTH AFTER HOURS:	

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between borings. Location of Test Boring is shown on the Boring Location Plan.

RECORD OF SUBSURFACE EXPLORATION

BORING NO. & LOCATION: 2A	PROJECT: Bender Park Bluff Evaluation	**GILES ENGINEERING ASSOCIATES, INC.**
SURFACE ELEVATION: 698.0	PROJECT LOCATION: Fitzsimmons and Oakwood Roads	Milwaukee Los Angeles
COMPLETION DATE: 10/6/03	Oak Creek, Wisconsin	Madison Dallas Atlanta
FIELD REPRESENTATIVE: Beauford Jones	GILES PROJECT NUMBER: 1G-0309022	Washington, D.C. Orlando

MATERIAL DESCRIPTION	Feet Below Surface	Sample No. & Type	N	q_u (tsf)	q_p (tsf)	q_s (tsf)	W (%)	PID	NOTES
Gray Silty Clay, trace fine to coarse Sand and Gravel-Moist *(continued)*	65	14-SS	18						
	70	15-SS	20						
	75	16-SS	13						
	80	17-SS	10						

Boring terminated at 81 feet

WATER OBSERVATION DATA	REMARKS
WATER ENCOUNTERED DURING DRILLING: None	(a) Poor sample recovery.
WATER LEVEL AFTER REMOVAL: None	
CAVE DEPTH AFTER REMOVAL: None	
WATER LEVEL AFTER HOURS:	
CAVE DEPTH AFTER HOURS:	

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between borings. Location of Test Boring is shown on the Boring Location Plan.

RECORD OF SUBSURFACE EXPLORATION

BORING NO. & LOCATION: PZ-2	PROJECT: Bender Park Bluff Evaluation
SURFACE ELEVATION: 696.0	PROJECT LOCATION: Fitzsimmons and Oakwood Roads
COMPLETION DATE: 10/7/03	Oak Creek, Wisconsin
FIELD REPRESENTATIVE: Beauford Jones	GILES PROJECT NUMBER: 1G-0309022

GILES ENGINEERING ASSOCIATES, INC.
Milwaukee Los Angeles
Madison Dallas Atlanta
Washington, D.C. Orlando

MATERIAL DESCRIPTION	Feet Below Surface	Sample No. & Type	N	q_u (tsf)	q_p (tsf)	q_s (tsf)	W (%)	PID	NOTES
No sampling from ground surface to 62± feet									
	5								
	10								
	15								
	20								
	25								
	30								
	35								
	40								
	45								
	50								
	55								
	60								

WATER OBSERVATION DATA	REMARKS
WATER ENCOUNTERED DURING DRILLING: None	NOTE: Pneumatic piezometer set at 75± feet in depth.
WATER LEVEL AFTER REMOVAL: None	
CAVE DEPTH AFTER REMOVAL: None	
WATER LEVEL AFTER HOURS:	
CAVE DEPTH AFTER HOURS:	

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between borings. Location of Test Boring is shown on the Boring Location Plan.

RECORD OF SUBSURFACE EXPLORATION

BORING NO. & LOCATION: PZ-2	PROJECT: Bender Park Bluff Evaluation	GILES ENGINEERING ASSOCIATES, INC.
SURFACE ELEVATION: 696.0	PROJECT LOCATION: Fitzsimmons and Oakwood Roads	Milwaukee Los Angeles
COMPLETION DATE: 10/7/03	Oak Creek, Wisconsin	Madison Dallas Atlanta
FIELD REPRESENTATIVE: Beauford Jones	GILES PROJECT NUMBER: 1G-0309022	Washington, D.C. Orlando

MATERIAL DESCRIPTION	Feet Below Surface	Sample No. & Type	N	q_u (tsf)	q_p (tsf)	q_s (tsf)	W (%)	PID	NOTES
Gray Silty Clay, trace fine to coarse Sand and Gravel-Moist		1-SS	8						
	65	2-SS	12						
		3-SS	14						
		4-SS	13						
	70	5-SS	12						
		6-SS	15						
	75	7-SS	10						
		8-SS	14						

Boring terminated at 77½ feet

WATER OBSERVATION DATA	REMARKS
WATER ENCOUNTERED DURING DRILLING: None	NOTE: Pneumatic piezometer set at 75± feet in depth.
WATER LEVEL AFTER REMOVAL: None	
CAVE DEPTH AFTER REMOVAL: None	
WATER LEVEL AFTER HOURS:	
CAVE DEPTH AFTER HOURS:	

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between borings. Location of Test Boring is shown on the Boring Location Plan.

A P P E N D I X B

FIELD PROCEDURES

The field operations were conducted in general accordance with the procedures recommended by the American Society for Testing and Materials (ASTM) designation D 420 entitled "Standard Guide for Sampling Soil and Rock" and/or other relevant specifications. Soil samples were preserved and transported to Giles's laboratory in general accordance with the procedures recommended by ASTM designation D 4220 entitled "Standard Practice for Preserving and Transporting Soil Samples." Brief descriptions of the sampling, testing and field procedures commonly performed by Giles are provided herein.

GENERAL FIELD PROCEDURES

Test Boring Elevations

The ground surface elevations reported on the Test Boring Logs are referenced to the assumed benchmark shown on the Boring Location Plan (Figure 1). Unless otherwise noted, the elevations were determined with a conventional hand-level and are accurate to within about 1 foot.

Test Boring Locations

The test borings were located on-site based on the existing site features and/or apparent property lines. Dimensions illustrating the approximate boring locations are reported on the Boring Location Plan (Figure 1).

Water Level Measurement

The water levels reported on the Test Boring Logs represent the depth of "free" water encountered during drilling and/or after the drilling tools were removed from the borehole. Water levels measured within a granular (sand and gravel) soil profile are typically indicative of the water table elevation. It is usually not possible to accurately identify the water table elevation within cohesive (clayey) soils, since the rate of seepage is slow. The water table elevation within cohesive soils must therefore be determined over a period of time with groundwater observation wells.

It must be recognized that the water table may fluctuate seasonally and during periods of heavy precipitation. Depending on the subsurface conditions, water may also become perched above the water table, especially during wet periods.

Borehole Backfilling Procedures

Each borehole was backfilled upon completion of the field operations. If potential contamination was encountered, and/or if required by state or local regulations, boreholes were backfilled with an "impervious" material (such as bentonite slurry). Borings that penetrated pavements, sidewalks, etc. were "capped" with Portland Cement concrete, asphaltic concrete, or a similar surface material. It must, however, be recognized that the backfill material may settle, and the surface cap may subside, over a period of time. Further backfilling and/or re-surfacing by Giles' client or the property owner may be required.

<u>FIELD SAMPLING AND TESTING PROCEDURES</u>

<u>Auger Sampling (AU)</u>

Soil samples are removed from the auger flights as an auger is withdrawn above the ground surface. Such samples are used to determine general soil types and identify approximately soil stratifications. Auger samples are highly disturbed and are therefore not typically used for geotechnical strength testing.

<u>Split-Barrel Sampling (SS) – (ASTM D-1586)</u>

A split-barrel sampler with a 2-inch outside diameter is driven into the subsoil with a 140-pound hammer, free-falling a vertical distance of 30 inches. The summation of hammer-blows required to drive the sampler the final 12 inches of an 18-inch sample interval is defined as the "Standard Penetration Resistance" or "N-value." The N-value is representative of the soils' resistance to penetration. The N-value is therefore an index of the relative density of granular soils and the comparative consistency of cohesive soils. A soil sample is collected from each SPT interval.

<u>Shelby Tube Sampling (ST) – (ASTM D-1587)</u>

A relatively undisturbed soil sample is collected by hydraulically advancing a thin-walled Shelby Tube sampler into a soil mass. Shelby Tubes have a sharp cutting edge and are commonly 2 to 5 inches in diameter. Unless otherwise noted, Giles uses 3-inch diameter tubes.

<u>Bulk Sample (BS)</u>

A relatively large volume of soil is collected with a shovel or other manually-operated tool. The sample is typically transported to Giles' materials laboratory in a sealed bag or bucket.

<u>Dynamic Cone Penetration Test (DC) – (ASTM STP 399)</u>

This test is conducted by driving a 1.5-inch-diameter cone into the subsoil using a 15-pound steel ring (hammer), free-falling a vertical distance of 20 inches. The number of hammer-blows required to drive the cone 1¾ inches is an indication of the soil strength and density, and is defined as "N." The Dynamic Cone Penetration test is commonly conducted in hand auger borings, test pits and within excavated trenches.

-Continued-

Ring-Lined Barrel Sampling – (ASTM D 3550)

In this procedure, a ring-lined barrel sampler is used to collect soil samples for classification and laboratory testing. This method provides samples that fit directly into laboratory test instruments without additional handling/disturbance.

Sampling and Testing Procedures

The field testing and sampling operations were conducted in general accordance with the procedures recommended by the American Society for Testing and Materials (ASTM) and/or other relevant specifications. Results of the field testing (i.e. N-values) are reported on the Test Boring Logs. Explanations of the terms and symbols shown on the logs are provided on the appendix enclosure entitled "General Notes."

A P P E N D I X   C

LABORATORY TESTING AND CLASSIFICATION

The laboratory testing was conducted under the supervision of a geotechnical engineer in general accordance with the procedures recommended by the American Society for Testing and Materials (ASTM) and/or other relevant specification. Brief descriptions of laboratory tests commonly performed by Giles are provided herein.

LABORATORY TESTING AND CLASSIFICATION

In this procedure, soil samples are 'scanned' in Giles' analytical laboratory using a Photoionization Detector (PID). The instrument is equipped with an 11.7 eV lamp calibrated to a Benzene Standard and is capable of detecting a minute concentration of ***certain*** *Volatile Organic Compound (VOC) vapors, such as those commonly associated with petroleum products and some solvents. Results of the PID analysis are expressed in HNu (manufacturer's) units rather than actual concentration.*

Moisture Content (w) (ASTM D2216)

Moisture content is defined as the ratio of the weight of water contained within a soil sample to the weight of the dry solids within the sample. Moisture content is expressed as a percentage.

Unconfined Compressive Strength (qu) (ASTM D2166)

An axial load is applied at a uniform rate to a cylindrical soil sample. The unconfined compressive strength is the maximum stress obtained or the stress when 15% axial strain is reached, whichever occurs first.

Calibrated Penetrometer Resistance (qp)

The small, cylindrical tip of a hand-held penetrometer is pressed into a soil sample to a prescribed depth to measure the soils capacity to resist penetration. This test is used to evaluate unconfined compressive strength.

Vane-Shear Strength (qs)

The blades of a vane are inserted into the flat surface of a soil sample and the vane is rotated until failure occurs. The maximum shear resistance measured immediately prior to failure is taken as the vane-shear strength.

Loss-On-Ignition (ASTM D2974; Method C)

The Loss-On-Ignition (L.O.I.) test is used to determine the organic content of a soil sample. This procedure is conducted by heating a dry soil sample to 440ºC in order to burn-off or "ash" organic matter present within the sample. The L.O.I. value is the ratio of the weight lost due to ignition compared to the initial weight of the dry sample. L.O.I. is expressed as a percentage.

Particle Size Distribution (ASTM D 421, D 422, and D 1140)

This test is performed to determine the distribution of specific particle sizes (diameters) within a soil sample. The distribution of coarse-grained soil particles (sand and gravel) is determined from a "sieve analysis," which is conducted by passing the sample through a series of nested sieves. The distribution of fine-grained soil particles (silt and clay) is determined from a "hydrometer analysis," which is based on the sedimentation of particles suspended in water.

Consolidation Test (ASTM D 2435)

In this procedure, a series of cumulative vertical loads are applied to a small, laterally confined soil sample. During each load increment, vertical compression (consolidation) of the sample is measured over a period of time. Results of this test are used to estimated settlement and time rate of settlement.

Classification of Samples

Each soil sample was visually-manually classified, based on texture and plasticity, in general accordance with the Unified Soil Classification System (ASTM D-2488-75). The classifications are reported on the Test Boring Logs.

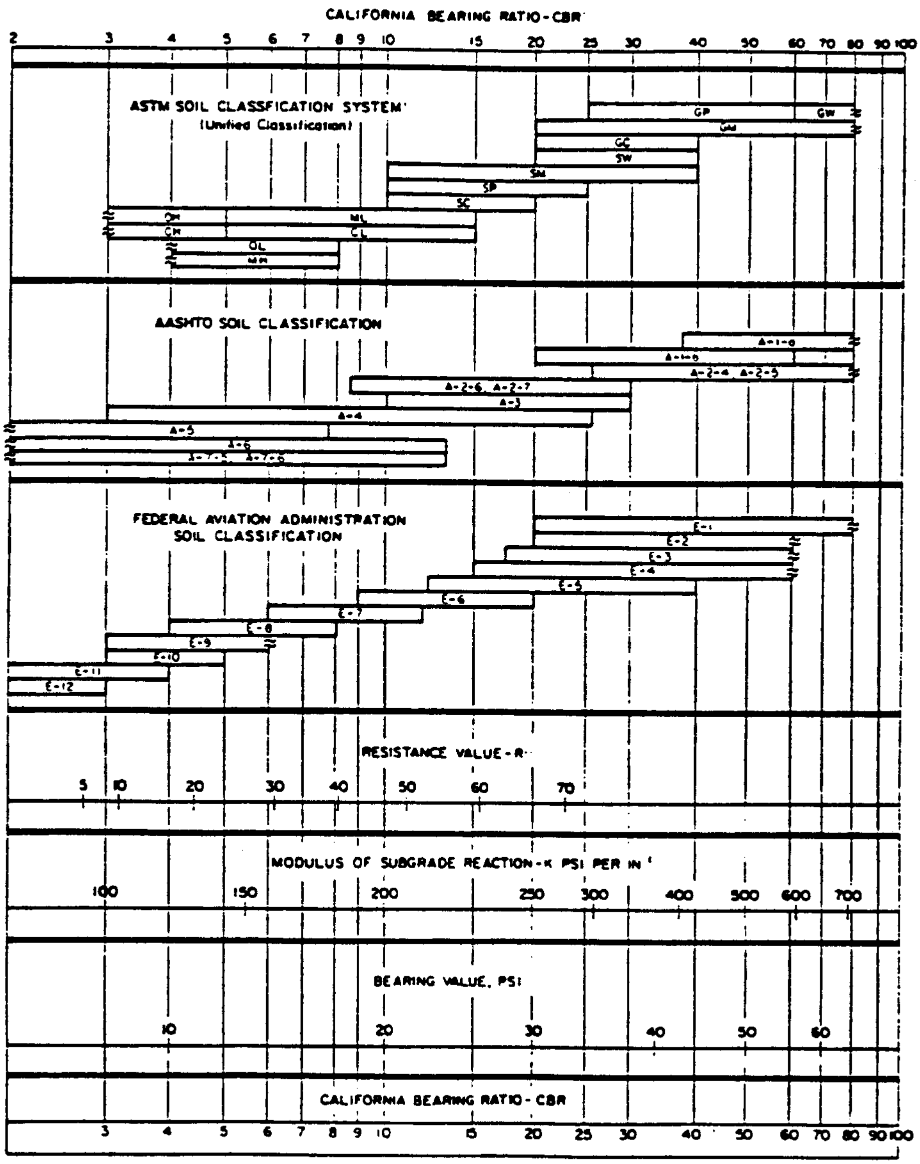
Laboratory Testing

The laboratory testing operations were conducted in general accordance with the procedures recommended by the American Society for Testing and Materials (ASTM) and/or other relevant specifications. Results of the laboratory tests are provided on the Test Boring Logs or other appendix enclosures. Explanation of the terms and symbols used on the logs is provided on the appendix enclosure entitled "General Notes."

California Bearing Ratio (CBR) Test ASTM D-1833

The CBR test is used for evaluation of a soil subgrade for pavement design. The test consists of measuring the force required for a 3-square-inch cylindrical piston to penetrate 0.1 or 0.2 inches into a compacted soil sample. The result is expressed as a percent of force required to penetrate a standard compacted crushed stone.

Unless a CBR test has been specifically requested by the client or heavy traffic loads are expected, the CBR is estimated from published charts, based on soil classification and strength characteristics. A typical correlation chart is indicated below.



A P P E N D I X D

GENERAL INFORMATION AND IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL ENGINEERING REPORT

GUIDE SPECIFICATIONS FOR SUBGRADE AND GRADE PREPARATION FOR FILL, FOUNDATION, FLOOR SLAB AND PAVEMENT SUPPORT; AND SELECTION, PLACEMENT AND COMPACTION OF FILL SOILS USING STANDARD PROCTOR PROCEDURES

1. Construction monitoring and testing of subgrades for fill, foundation, floor slab and pavement; and fill selection, placement and compaction shall be performed by an experienced soils engineer and/or his representatives.

2. All compaction fill, subgrades, and grades shall be (a) underlain by suitable bearing material, (b) free of all organic, frozen, or other deleterious material, and (c) observed, tested and approved by qualified engineering personnel representing an experienced soils engineer. Preparation of subgrades after stripping vegetation, organic or other unsuitable materials shall consist of (a) proofrolling to detect soil, wet, yielding soils or other unstable materials that must be undercut, (b) scarifying top 6 to 8 inches, (c) moisture conditioning the soils as required, and (d) recompaction to same minimum in-situ density required for similar materials indicated under Item 5. Note: Compaction requirements for pavement subgrade are higher than other areas. Weather and construction equipment may damage compacted fill surface and reworking and retesting may be necessary to assure proper performance.

3. In overexcavation and fill areas, the compacted fill must extend (a) a minimum 1 foot lateral distance beyond the exterior edge of the foundation at bearing grade or pavement at subgrade and down to compacted fill subgrade on a maximum 0.5 (H): 1(V) slope, (b) 1 foot above footing grade outside the building, and (c) to floor subgrade inside the building. Fill shall be placed and compacted on a 5 (H):1 (V) slope or must be stepped or benched as required to flatten if not specifically approved by qualified personnel under the direction of an experienced soil engineer.

4. The compacted fill materials shall be free of deleterious, organic, or frozen matter, shall contain no chemicals that may result in the material being classified as “contaminated,” and shall be low-expansive with a maximum Liquid Limit (ASTM D-423) and Plasticity Index (ASTM D-424) of 30 and 15, respectively, unless specifically tested and found to have low expansive properties and approved by an experienced soils engineer. The top 12 inches of compacted fill should have a maximum 3-inch-particle diameter and all underlying compacted fill a maximum 6-inch diameter unless specifically approved by an experienced soils engineer. All fill material must be tested and approved under the direction of an experienced soils engineer prior to placement. If the fill is to provide non-frost susceptible characteristics, it must be classified as a clean GW, GP, SW or SP per Unified Soil Classification System (ASTM D-2487).

5. For structural fill depths less than 20 feet, the density of the structural compacted fill and scarified subgrade and grades shall not be less than 95 percent of the maximum dry density as determined by Standard Proctor (ASTM D-698) with the exception of the top 12 inches of pavement subgrade which shall have a minimum in-situ density of 100 percent of maximum dry density, or 5 percent higher than underlying fill materials. Where the structural fill depth is greater than 20 feet, the portions below 20 feet should have a minimum in-place density of 100 percent of its maximum dry density of 5 percent greater than the top 20 feet. The moisture content of cohesive soil shall not vary by more than –1 to +3 percent and granular soil ±3 percent of the optimum when placed and compacted or recompacted, unless specifically recommended/approved by the soils engineer monitoring the placement and compaction. Cohesive soils with moderate to high expansive potentials (PI >15) should, however, be placed, compacted and maintained prior to construction at a moisture content of 3±1 percent above optimum moisture content to limit future heave. The fill shall be placed in layers with a maximum loose thickness of 8 inches for foundations and 10 inches for floor slabs and pavements, unless specifically approved by the soils engineer taking into consideration the type of materials and compaction equipment being used. The compaction equipment should consist of suitable mechanical equipment specifically designed for soil compaction. Bulldozers or similar tracked vehicles are typically not suitable for compaction.

6. Excavation, filing, subgrade and grade preparation shall be performed in a manner and sequence that will provide drainage at all times and proper control of erosion. Precipitation, springs, and seepage water encountered shall be pumped or drained to provide a suitable working platform. Springs or water seepage encountered during grading/foundation construction must be called to the soil engineer’s attention immediately for possible construction procedure revision or inclusion of an underdrain system.

7. Non-structural fill adjacent to structural fill should typically be placed in unison to provide lateral support. Backfill along walls must be placed and compacted with care to ensure excessive unbalanced lateral pressures do not develop. The type of fill material placed adjacent to below grade walls (i.e. basement walls and retaining walls) must be properly tested and approved by an experienced soils engineer with consideration for the lateral pressure used in the wall design.

8. Wherever, in the opinion of the soils engineer or the Owner’s Representatives, an unstable condition is being created either by cutting or filling, the work shall not proceed into that are until an appropriate geotechnical exploration and analysis has been performed and the grading plan revised, if found necessary.

GENERAL COMMENTS

The soil samples obtained during the subsurface exploration will be retained for a period of thirty days. If no instructions are received, they will be disposed of at that time.

This report has been prepared exclusively for the client in order to aid in the evaluation of this property and to assist the architects and engineers in the design and preparation of the project plans and specifications. Copies of this report may be provided to contractor(s), with contract documents, to disclose information relative to this project.. The report, however, has not been prepared to serve as the plans and specifications for actual construction without the appropriate interpretation by the project architect, structural engineer, and/or civil engineer. Reproduction and distribution of this report must be authorized by the client and *Giles*.

This report has been based on assumed conditions/characteristics of the proposed development where specific information was not available. It is recommended that the architect, civil engineer and structural engineer along with any other design professionals involved in this project carefully review these assumptions to ensure they are consistent with the actual planned development. When discrepancies exist, they should be brought to our attention to ensure they do not affect the conclusions and recommendations provided herein have been correctly interpreted.

The analysis of this site was based on a subsoil profile interpolated from a limited subsurface exploration. If the actual conditions encountered during construction vary from those indicated by the borings, Giles must be contacted immediately to determine if the conditions alter the recommendations contained herein.

The conclusions and recommendations presented in this report have been promulgated in accordance with generally accepted professional engineering practices in the field of geotechnical engineering. No other warranty is either expressed or implied.

CHARACTERISTICS AND RATINGS OF UNIFIED SOIL SYSTEM CLASSES FOR SOIL CONSTRUCTION *

Class	Compaction Characteristics	Max. Dry Density Standard Proctor (pcf)	Compressibility and Expansion	Drainage and Permeability	Value as an Embankment Material	Value as Subgrade When Not Subject to Frost	Value as Base Course	Value as Temporary Pavement	
								With Dust Palliative	With Bituminous Treatment
GW	Good: tractor, rubber-tired, steel wheel or vibratory roller	125-135	Almost none	Good drainage, pervious	Very stable	Excellent	Good	Fair to Poor	Excellent
GP	Good: tractor, rubber-tired, steel wheel or vibratory roller	115-125	Almost none	Good drainage, pervious	Reasonably stable	Excellent to good	Poor to fair	Poor	
GM	Good: rubber-tired or light sheepsfoot roller	120-135	Slight	Poor drainage, semipervious	Reasonably stable	Excellent to good	Fair to poor	Poor	Poor to fair
GC	Good to fair: rubber-tired or sheepsfoot roller	115-130	Slight	Poor drainage, impervious	Reasonably stable	Good	Good to fair **	Excellent	Excellent
SW	Good: tractor, rubber-tired or vibratory roller	110-130	Almost none	Good drainage, pervious	Very stable	Good	Fair to poor	Fair to poor	Good
SP	Good: tractor, rubber-tired or vibratory roller	100-120	Almost none	Good drainage, pervious	Reasonably stable when dense	Good to fair	Poor	Poor	Poor to fair
SM	Good: rubber-tired or sheepsfoot roller	110-125	Slight	Poor drainage, impervious	Reasonably stable when dense	Good to fair	Poor	Poor	Poor to fair
SC	Good to fair: rubber-tired or sheepsfoot roller	105-125	Slight to medium	Poor drainage, impervious	Reasonably stable	Good to fair	Fair to poor	Excellent	Excellent
ML	Good to poor: rubber-tired or sheepsfoot roller	95-120	Slight to medium	Poor drainage, impervious	Poor stability, high density required	Fair to poor	Not suitable	Poor	Poor
CL	Good to fair: sheepsfoot or rubber-tired roller	95-120	Medium	No drainage, impervious	Good stability	Fair to poor	Not suitable	Poor	Poor
OL	Fair to poor: sheepsfoot or rubber-tired roller	80-100	Medium to high	Poor drainage, impervious	Unstable, should not be used	Poor	Not suitable	Not suitable	Not suitable
MH	Fair to poor: sheepsfoot or rubber-tired roller	70-95	High	Poor drainage, impervious	Poor stability, should not be used	Poor	Not suitable	Very poor	Not suitable
CH	Fair to poor: sheepsfoot roller	80-105	Very high	No drainage, impervious	Fair stability, may soften on expansion	Poor to very poor	Not suitable	Very poor	Not suitable
OH	Fair to poor: sheepsfoot roller	65-100	High	No drainage, impervious	Unstable, should not be used	Very poor	Not suitable	Not suitable	Not suitable
Pt	Not suitable		Very high	Fair to poor drainage	Should not be used	Not suitable	Not suitable	Not suitable	Not suitable

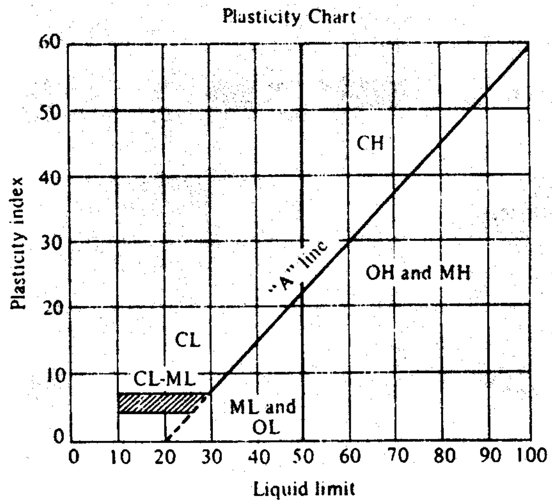
* "The Unified Classification: Appendix A - Characteristics of Soil, Groups Pertaining to Roads and Airfields, and Appendix B - Characteristics of Soil Groups Pertaining to Embankments and Foundations," Technical Memorandum 357, U.S. Waterways Ixperiment Station, Vicksburg, 1953.

** Not suitable if subject to frost.

UNIFIED SOIL CLASSIFICATION SYSTEM. (ASTM D-2487)

Major Divisions			Group Symbols	Typical Names	Laboratory Classification Criteria		
Coarse-grained soils (More than half of material is larger than No. 200 sieve size)	Gravels (More than half of coarse fraction is larger than No. 4 sieve size)	Clean gravels (Little or no fines)	GW	Well-graded gravels, gravel-sand mixtures, little or no fines	Determine percentages of sand and gravel from grain-size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve size), coarse-grained soils are classified as follows: Less than 5 per cent: GW, GP, SW, SP; More than 12 per cent: GM, GC, SM, SC; 5 to 12 per cent: *Borderline* cases requiring dual symbols[b]	$C_u = \dfrac{D_{60}}{D_{10}}$ greater than 4; $C_c = \dfrac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	
			GP	Poorly graded gravels, gravel-sand mixtures, little or no fines		Not meeting all gradation requirements for GW	
		Gravels with fines (Appreciable amount of fines)	GM[a] d / u	Silty gravels, gravel-sand-silt mixtures		Atterberg limits below "A" line or P.I. less than 4	Above "A" line with P.I. between 4 and 7 are *borderline* cases requiring use of dual symbols
			GC	Clayey gravels, gravel-sand-clay mixtures		Atterberg limits below "A" line with P.I. greater than 7	
	Sands (More than half of coarse fraction is smaller than No. 4 sieve size)	Clean sands (Little or no fines)	SW	Well-graded sands, gravelly sands, little or no fines		$C_u = \dfrac{D_{60}}{D_{10}}$ greater than 6; $C_c = \dfrac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	
			SP	Poorly graded sands, gravelly sands, little or no fines		Not meeting all gradation requirements for SW	
		Sands with fines (Appreciable amount of fines)	SM[a] d / u	Silty sands, sand-silt mixtures		Atterberg limits above "A" line or P.I. less than 4	Limits plotting in hatched zone with P.I. between 4 and 7 are *borderline* cases requiring use of dual symbols
			SC	Clayey sands, sand-clay mixtures		Atterberg limits above "A" line with P.I. greater than 7	
Fine-grained soils (More than half material is smaller than No. 200 sieve)	Silts and clays (Liquid limit less than 50)		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts with slight plasticity			
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays			
			OL	Organic silts and organic silty clays of low plasticity			
	Silts and clays (Liquid limit greater than 50)		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts			
			CH	Inorganic clays of high plasticity, fat clays			
			OH	Organic clays of medium to high plasticity, organic silts			
	Highly organic soils		Pt	Peat and other highly organic soils			

Plasticity Chart



[a]Division of GM and SM groups into subdivisions of d and u are for roads and airfields only. Subdivision is based on Atterberg limits; suffix d used when L.L. is 28 or less and the P.I. is 6 or less; the suffix u used when L.L. is greater than 28.
[b]Borderline classifications, used for soils possessing characteristics of two groups, are designated by combinations of group symbols. For example: GW-GC, well-graded gravel-sand mixture with clay binder.

GENERAL NOTES

SAMPLE IDENTIFICATION

All samples are visually classified in general accordance with the Unified Soil Classification System (ASTM D-2487-75 or D-2488-75)

DESCRIPTIVE TERM (% BY DRY WEIGHT)

Trace:	1-10%
Little:	11-20%
Some:	21-35%
And/Adjective	36-50%

PARTICLE SIZE (DIAMETER)

Boulders:	8 in and larger
Cobbles:	3 in to 8 in
Gravel:	coarse - ¾ to 3 in
	fine - No. 4 (4.76 mm) to ¾ in
Sand:	coarse - No. 4 (4.76 mm) to No. 10 (2.0 mm)
	medium - No. 10 (2.0 mm) to No. 40 (0.42 mm)
	fine - No. 40 (0.42 mm) to No. 200 (0.074 mm)
Silt:	No. 200 (0.074 mm) and smaller (Non-plastic)
Clay:	No. 200 (0.074 mm) and smaller (Plastic)

SOIL PROPERTY SYMBOLS

Dd:	Dry Density (pcf)
LL:	Liquid Limit, percent
PL:	Plastic Limit, percent
PI:	Plasticity Index (LL-PL)
LOI:	Loss on Ignition, percent
Gs:	Specific Gravity
K:	Coefficient of Permeability
w:	Moisture content, percent
qp:	Calibrated Penetrometer Resistance, tsf
qs:	Vane-Shear Strength, tsf
qu:	Unconfined Compressive Strength, tsf
qc:	Static Cone Penetrometer Resistance Correlated to Unconfined Compressive Strength, tsf
PID:	Results of vapor analysis conducted on representative samples utilizing a Photoionization Detector calibrated to a benzene standard. Results expressed in HNU-units (BDL=Below Detection Limits)
N:	Penetration Resistance per 6 inch interval, or fraction thereof, for a standard 2 inch O.D. (1⅜ inch I.D.) split spoon sampler driven with a 140 pound weight free-falling 30 inches. Performed in general accordance with Standard Penetration Test Specifications (ASTM D-1586). N in blows per foot equals sum of N values where plus sign is shown
Nc:	Penetration Resistance per 1¾ inches of Dynamic Cone Penetrometer. Approximately equivalent to Standard Penetration Test N-Value in blows per foot.
Nr:	Penetration Resistance per 6 inch interval, or fraction thereof, for California Ring Sampler driven with a 140 pound weight free-falling 30 inches per ASTM D-3550. Not equivalent to Standard Penetration Test N-Value.

DRILLING AND SAMPLING SYMBOLS

SS:	Split-Spoon
ST:	Shelby Tube - 3" O.D. (except where noted)
CS:	3" O.D. California Ring Sampler
DC:	Dynamic Cone Penetrometer per ASTM Special Technical Publication No. 399
AU:	Auger Sample
DB:	Diamond Bit
CB:	Carbide Bit
WS:	Wash Sample
RB:	Rock-Roller Bit
BS:	Bulk Sample
Note:	Depth intervals for sampling shown on Record of Subsurface Exploration are not indicative of sample recovery, but position where sampling initiated

SOIL STRENGTH CHARACTERISTICS

COHESIVE (CLAYEY) SOILS

COMPARATIVE CONSISTENCY	BLOWS PER FOOT (N)	UNCONFINED COMPRESSIVE STRENGTH (TSF)
Very Soft	0-2	0-0.25
Soft	3-4	0.25-0.50
Medium Stiff	5-8	0.50-1.00
Stiff	9-15	1.00-2.00
Very Stiff	16-30	2.00-4.00
Hard	31+	4.00+

NON-COHESIVE (GRANULAR) SOILS

RELATIVE DENSITY	BLOWS PER FOOT (N)
Very Loose	0-4
Loose	5-10
Firm	11-30
Dense	31-50
Very Dense	51+

DEGREE OF PLASTICITY	PI
None to Slight	0-4
Slight	5-10
Medium	11-30
High to Very High	31+

DEGREE OF EXPANSIVE POTENTIAL	PI
Low	0-15
Medium	15-25
High	25+

Important Information About Your
Geotechnical Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims and disputes.

The following information is provided to help you manage your risks.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared *solely* for the client. *No one except you* should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one – not even you –* should apply the report for any purpose or project except the one originally contemplated.

A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors

Geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, *do not rely on a geotechnical engineering report* that was:

- Not prepared for you,
- Not prepared for your project,
- Not prepared for the specific site explored, or
- Completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotech-nical engineering report include those that affect:

- The function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light industrial plant to a refrigerated warehouse,
- Elevation, configuration, location, orientation, or weight of the proposed structure,
- Composition of the design team, or
- Project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes – even minor ones – and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical engineering report is based on conditions that existed at the time the study was performed. *Do not rely on a geotechnical engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions *only* at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgement to render an *opinion* about subsurface conditions throughout the site. Actual subsurface conditions may differ – sometimes significantly – from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observations is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final,* because geotechnical engineers develop them principally from judgement and opinion. Geotechnical engineers can finalize their recommendations only by observing actual subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.*

A Geotechnical Engineering Report Is Subject To Misinterpretation

Other design team members' misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observations.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure contractors have sufficient time* to perform additional study. Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce such risks, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely*. Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures*. If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. *Do not rely on an environmental report prepared for someone else.*

Rely on Your Geotechnical Engineer for Additional Assistance

Membership in ASFE exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit to everyone involved with a construction project. Confer with an ASFE-member geotechnical engineer for more information.

ASFE

8811 Colesville Road/Suite G106, Silver Spring, MD 20910
Telephone: 301/565-2733 Facsimile: 301/589-2017
e-mail: **info@asfe.org** www.asfe.org