SCS ENGINEERS







Initial Closure Plan

Coal Combustion Residue Landfill
Lansing Generating Station

Lansing, Iowa

Prepared for:

Interstate Power and Light Company
Lansing Generating Station
2320 Power Plant Drive
Lansing, Iowa 52151-7539

Prepared by:

SCS ENGINEERS
2830 Dairy Drive
Madison, Wisconsin 53718-6751
(608) 224-2830

September 2016
File No. 25216109.00

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September 2016
File No. 25216109.00

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PE CERTIFICATION

LICENSED PROFESSIONAL ENGINEER
ERIC J. NELSON
23136
IOWA
9/21/16

I, Eric J. Nelson, hereby certify the following:

- This Initial Closure Plan meets the requirements of 40 CFR 257.102(b)
- The final cover system described in this Initial Closure Plan meets the design requirements in 40 CFR 257.102(d)(3)

The Initial Closure Plan was prepared by me or under my direct supervision, and that I am a duly licensed Professional Engineer under the laws of the State of Iowa.

9/21/2016

(signature) (date)

ERIC J. NELSON

(printed or typed name)

License number 23136

My license renewal date is December 31, 2016.

Pages or sheets covered by this seal:
SEPTEMBER 2016 INITIAL CLOSURE PLAN
IPL LANSING GENERATING STATION

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1.0 INTRODUCTION AND PROJECT SUMMARY

On behalf of Interstate Power and Light Company (IPL), SCS Engineers (SCS) has prepared this Initial Closure Plan for the Lansing Generating Station (LAN) Coal Combustion Residue (CCR) Landfill as required by 40 CFR 257.102(b).

40 CFR 257.102(b) *"Written closure plan—(1) Content of the plan. The owner or operator of a CCR unit must prepare a written closure plan that describes the steps necessary to close the CCR unit at any point during the active life of the CCR unit consistent with recognized and generally accepted good engineering practices. The written closure plan must include, at a minimum, the information specified in paragraphs (b)(1)(i) through (vi) of this section."*

The LAN CCR Landfill includes an active CCR landfill, which currently consists of a single CCR unit. The CCR unit has received CCR both before and after the effective date of the CCR Rule.

The CCR unit has been partially closed by leaving the CCR in place and capping it. Phase 1 was capped in 2014 and Phase 2 was capped in 2015 with a final cover system meeting 40 CFR 257.102(d) (3), as described in **Section 3.0**. The remaining open areas of the CCR unit will receive the same or equivalent final cover when final waste grades are achieved.

Figure 1 shows the site location. **Figure 2** shows the closure areas. A detail of the final cover system is also included on **Figure 2**.

2.0 PROPOSED CLOSURE PLAN NARRATIVE

40 CFR 257.102(b)(1)(i) *"A narrative description of how the CCR unit will be closed in accordance with this section."*

When CCR placement is completed in the CCR unit, or if early closure is required, the unit will be closed by covering the CCR with the final cover system described in **Section 3.0**. Prior to final cover system construction, the CCR surfaces will be graded and compacted to establish a firm subgrade for final cover construction. Based on the currently constructed portions of the CCR unit, it is estimated that the final cover will be placed in the area shown on **Figure 2** by the end of 2019. Actual closure sequencing and timeframes are dependent on CCR generation rates. A detailed closure schedule is presented in **Appendix B**.

The initiation of closure activities will commence no later than 30 days after the final receipt of CCR as required by 40 CFR 257.102(e)(1) or in accordance with 40 CFR 257.102(e)(2).

3.0 FINAL COVER SYSTEM AND PERFORMANCE

40 CFR 257.102(b)(1)(iii). *"If closure of the CCR unit will be accomplished by leaving CCR in place, a description of the final cover system, designed in accordance with paragraph (d) of this section, and the methods and procedures to be used to install the final cover. The closure plan must also discuss how the final cover system will achieve the performance standards specified in paragraph (d) of this section."*

"(d) Closure performance standard when leaving CCR in place.
(1) The owner or operator of a CCR unit must ensure that, at a minimum, the CCR unit is closed in a manner that will:

(i) *Control, minimize or eliminate, to the maximum extent feasible, post-closure infiltration of liquids into the waste and releases of CCR, leachate, or contaminated run-off to the ground or surface waters or to the atmosphere;*

The final cover system design will minimize or eliminate infiltration, as further described below.

(ii) *Preclude the probability of future impoundment of water, sediment, or slurry;*

The final cover system will meet these criteria, as further described below.

(iii) *Include measures that provide for major slope stability to prevent the sloughing or movement of the final cover system during the closure and post-closure care period;*

The final cover system is designed to provide slope stability and to prevent sloughing or movement during the closure and post-closure care period. Stability of the final cover system was assessed as part of the Iowa Department of Natural Resources (IDNR) landfill permitting process and is further addressed below.

(iv) *Minimize the need for further maintenance of the CCR unit; and*

Maintenance of the final cover will be minimized by the establishment of vegetative cover and the erosion control systems, which are further described below.

(v) *Be completed in the shortest amount of time consistent with recognized and generally accepted good engineering practices."*

All closure activities for the CCR unit will be completed within 6 months, as stated in **Section 7.0** below.

"(2) Drainage and stabilization of CCR surface impoundments."

This does not apply to this CCR unit.

"(3) Final cover system"

The Phase 1 and 2 final cover systems (see **Figure 2** for detail) in place (and planned for the remaining area of the landfill), is as follows from the bottom up:

- Two feet of clay, compacted to $1x10^{-7}$ cm/sec permeability
- Six inches of un-compacted rooting zone material
- Six inches of topsoil

This final cover meets the minimum requirements of 40 CFR 257.102(d)(3)(i)(A) through (D) as follows:

- Per 257.102(d)(3)(i)(A), the permeability of the final cover system is less than or equal to the permeability of the bottom liner system and is no greater than $1x10^{-5}$ cm/sec that is required by the rule. The Lansing CCR landfill infiltration layer consists of 2-feet compacted clay with a maximum permeability of 1×10^{-7} cm/sec. This CCR unit does not have an engineered liner.

- Per 257.102(d)(3)(i)(B), the final cover system includes 3 feet of soil, which is greater than the 18 inches of earthen material required to minimize infiltration in the Rule.

- Per 257.102(d)(3)(i)(C), erosion of the final cover system is minimized with a vegetative support layer consisting of 6 inches of uncompacted rooting zone material and 6 inches of topsoil. This provides more than the required 6-inch thickness for plant growth.

  Also this final cover system limits infiltration while promoting surface water runoff in a controlled manner to minimize erosion and promote stability. The surface of 12 inches of soil supports vegetation that assists with erosion control.

  In addition, the surface has intermediate drainage swales to reduce the flow lengths down the final cover slope, also aiding in erosion control (see **Figure 2**). Where needed, the intermediate drainage swales are connected to rock chutes to control storm water runoff and prevent erosion of the final cover.

- Per 257.102(d)(3)(i)(D), the design of the final cover system minimizes disruptions to the final cover system. Stability of the final cover system was assessed as part of the IDNR landfill permitting process. The stability calculations are included in **Appendix A**.

  The design of the final cover system accommodates settling and subsidence of the CCR fill below the cover. Based on the subsurface investigation and CCR test results presented in **Appendix A**, the CCR has been and will continue to consolidate and gain strength as filling progresses prior to final cover placement. Filling has occurred gradually over a period of more than 15 years so the CCR strength has increased since the investigation and testing in 2000. The final cover system is designed with a maximum slope of 25 percent (4 horizontal to 1 vertical). Because the final cover has a relatively large positive slope and the CCR has been gaining strength over time, the final cover is expected to easily accommodate the remaining relatively minor settlement potential of the CCR fill when fill placement ends and the landfill is closed.

All final cover materials will be tested to confirm they meet the required specifications and construction will be overseen and documented by a licensed engineer. Clay material placement will be tested for compaction, permeability, and thickness. Rooting zone and topsoil layers will be checked for thickness. All areas will be restored after final cover is placed. Vegetation will be monitored and maintained.

4.0 MAXIMUM INVENTORY OF CCR

40 CFR 257.102(b)(1)(iv). *"An estimate of the maximum inventory of CCR ever on-site over the active life of the CCR unit."*

The permitted maximum volume of CCR in the CCR unit is 485,000 cubic yards. The estimated maximum inventory of CCR ever on the CCR landfill site over the active life of the CCR unit is based on the design capacity of the CCR unit. The volume is taken from the IDNR approved 2001 Permit Application.

5.0 LARGEST AREA OF CCR UNIT REQUIRING FINAL COVER

40 CFR 257.102(b)(1)(v). *"An estimate of the largest area of the CCR unit ever requiring a final cover as required by paragraph (d) of this section at any time during the CCR unit's active life."*

The largest area of the CCR unit requiring final cover is the 9 acre area that is currently open, as shown on **Figure 2**.

6.0 SCHEDULE OF SEQUENTIAL CLOSURE ACTIVITIES

40 CFR 257.102(b)(1)(vi). *"A schedule for completing all activities necessary to satisfy the closure criteria in this section, including an estimate of the year in which all closure activities for the CCR unit will be completed."*

The existing CCR unit is expected to reach capacity in 2019. That closure date is based on the site life calculated from the design capacity calculations and anticipated disposal rates. At that time, all existing CCR unit areas will be closed, unless new adjacent CCR units are constructed allowing for the overlay of additional CCR onto the existing unit. The preliminary schedule for closure of the existing CCR unit is in **Appendix B**.

7.0 COMPLETION OF CLOSURE ACTIVITIES

<u>40 CFR 257.102((f)(1)(i).</u> *"For existing and new CCR landfills and any lateral expansion of a CCR landfill, within six months of commencing closure activities."*

As shown on the enclosed schedule, closure of the remaining open area of the CCR unit will be completed within 6 months of commencing closure activities.

<u>40 CFR 257.102(f)(3)</u>. *"Upon completion, the owner or operator of the CCR unit must obtain a certification from a qualified professional engineer verifying that closure has been completed in accordance with the closure plan specified in paragraph (b) of this section and the requirements of this section."*

A qualified licensed engineer will oversee the final cover construction. The engineer will verify final cover materials and methods, and oversee material testing. At the end of construction, the engineer will provide a report summarizing and documenting construction, and will certify compliance with the requirements.

8.0 CERTIFICATION

<u>40 CFR 257.102(b)(4)</u> *"The owner or operator of the CCR unit must obtain a written certification from a qualified professional engineer that the initial and any amendment of the written closure plan meets the requirement of this section."*

Eric Nelson, PE, a licensed professional engineer in the State of Iowa, has overseen the preparation of this Initial Closure Plan. A certification statement is provided as **page iii** of this plan.

<u>40 CFR 257.102(d)(3)(iii).</u> *"The owner or operator of the CCR unit must obtain a written certification from a qualified professional engineer that the design of the final cover system meets the requirement of this section."*

Eric Nelson, PE, a licensed professional engineer in the State of Iowa has reviewed the final cover design and certifies that the design meets the requirements of 40 CFR 257.102(d). The certification statement is provided on **page iii** of this plan.

9.0 RECORDKEEPING AND REPORTING

<u>40 CFR 257.102(b)(vi)(2)(iii).</u> *"The owner or operator has completed the written closure plan when the plan, including the certification required by paragraph (b)(4) of this section, has been placed in the facility's operating record as required by Section 257.105(i)(4)."*

The Closure Plan will be placed in the facility's operating record and on Alliant Energy's CCR Rule Compliance Data and Information website.

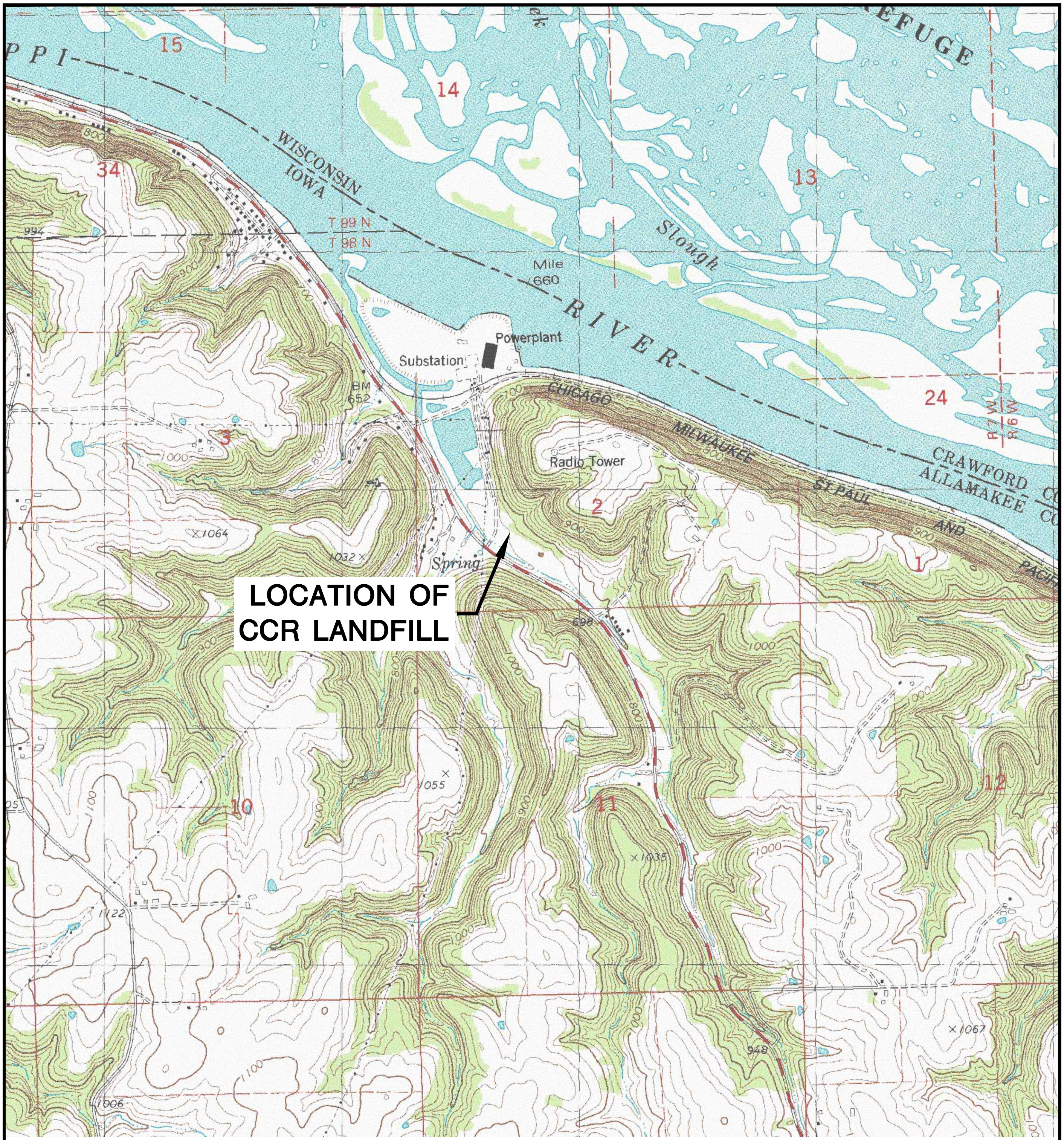
Amendments to the written closure plan will be done when there is a change in the operation of the CCR unit that affects the plan or when unanticipated events warrant revision to the written closure plan as required by 40 CFR 257.102(b)(3).

IPL will provide notification as follows:

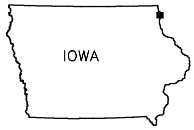
- Intent to initiate closure.
- Closure completion.
- Availability of the written closure plan and any amendments.

All notifications will be placed in the facility's operating record and on the website per 40 CFR 257.105(i), 257.106(i), 257.107(i).

FIGURES



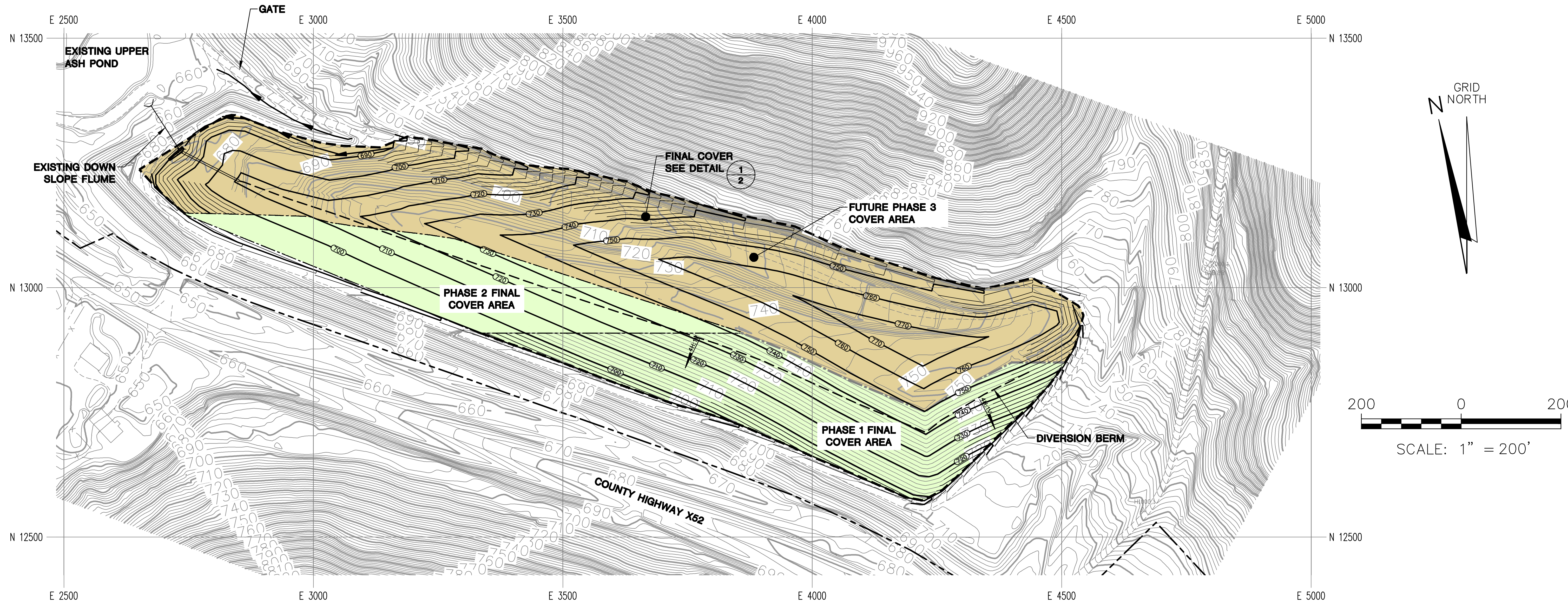
LANSING QUADRANGLE
IOWA–ALLAMAKEE CO.
7.5 MINUTE SERIES (TOPOGRAPHIC)
SW/4 FERRYVILLE 15' QUADRANGLE
1983
SCALE: 1" = 2,000'



CLIENT	SITE	
INTERSTATE POWER AND LIGHT CO. 2320 POWER PLANT DRIVE LANSING, IA 52151–9733	INITIAL CLOSURE PLAN COAL COMBUSTION RESIDUE LANDFILL LANSING, IOWA	SITE LOCATION MAP

				ENGINEER	
PROJECT NO.	25216109.00	DRAWN BY:	AHB	SCS ENGINEERS 2830 DAIRY DRIVE MADISON, WI 53718–6751 PHONE: (608) 224–2830	FIGURE 1
DRAWN:	05/01/13	CHECKED BY:	LB		
REVISED:	09/08/16	APPROVED BY:	EN 09/21/16		

I:\25216109.00\Drawings\Initial Closure Plan\Figure1.dwg, 9/21/2016 9:52:27 AM

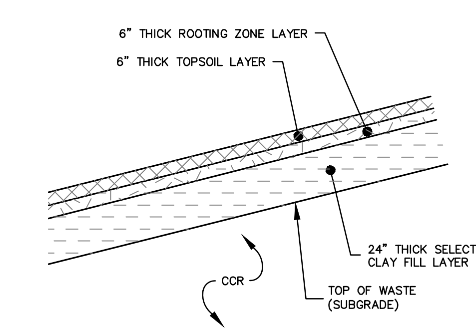


LEGEND

APPROXIMATE PROPERTY LINE	
APPROVED LIMITS OF WASTE	
PROPOSED PHASE LIMIT	
LIMITS OF PHASE 1 FINAL COVER	
LIMITS OF PHASE 2 FINAL COVER	
760	EXISTING 10' CONTOUR
EXISTING 2' CONTOUR	
EXISTING PERIMETER/UNPAVED ROAD	
EXISTING RIPRAP	
700	PROPOSED FINAL GRADE (10' CONTOUR)
	PROPOSED FINAL GRADE (2' CONTOUR)
4H:1V	SLOPE AND DIRECTION
	CLOSED AREA
	AREAS TO BE CLOSED

NOTES:

1. TOPOGRAPHIC SURVEY OF EXISTING LANDFILL GRADES AREA FOR THE PHASE 2 FINAL COVER WAS COMPLETED BY MOHN SURVEYING ON MARCH 12, 2015.
2. TOPOGRAPHIC SURVEY OF EXISTING LANDFILL GRADES WITHIN THE EXISTING FINAL COVER WAS COMPLETED BY MOHN SURVEYING IN APRIL 2015.
3. THE REMAINING TOPOGRAPHIC SURVEY OF EXISTING LANDFILL GRADES WITHIN LIMITS OF WASTE WAS COMPLETED BY SCS ENGINEERS ON JUNE 2–3, 2014.
4. EXISTING GRADES OUTSIDE THE 2014–2015 TOPOGRAPHIC SURVEY LIMITS ARE BASED ON KBM, INC.'S AERIAL SURVEY COMPLETED APRIL 18. 2001.
5. PERIMETER ROAD LOCATION BASED ON 2002 SURVEY AND UPDATED WITH AS-BUILT LOCATION AROUND LIMITS OF PHASE 1 FINAL COVER AREA.



1/2 **FINAL COVER SYSTEM**

SCALE: 1"=4'

PROJECT NO.	25216109.00	DRAWN BY:	AHB
DRAWN:	08/22/16	CHECKED BY:	DN
REVISED:	09/08/16	APPROVED BY:	EN 09/21/16

ENGINEER: SCS ENGINEERS, 2830 DAIRY DRIVE MADISON, WI 53718–6751, PHONE: (608) 224–2830

CLIENT: INTERSTATE POWER AND LIGHT CO., 2320 POWER PLANT DRIVE, LANSING, IA 52151–9733

SITE: INITIAL CLOSURE PLAN, COAL COMBUSTION RESIDUE LANDFILL, LANSING, IOWA

INITIAL CLOSURE PLAN

FIGURE 2

I:\25216109.00\Drawings\Initial Closure Plan\Phase 2 Waste Grades.dwg, 9/21/2016 9:53:11 AM

APPENDIX A

Stability Calculations

MH7 **FILE COPY**

~~1913~~
1792

CGC, Inc.

Construction • Geotechnical
Consulting Engineering/Testing

RECEIVED
JUL 23 2001

July 20, 2001
C20207

Ms. Sherren Clark
BT^2, Inc.
2830 Dairy Drive
Madison, WI 53718

Re: Supplemental Report
Fly Ash Landfill Expansion
Alliant Energy Site
Lansing, Iowa

Dear Ms. Clark:

CGC, Inc. was asked for our opinion regarding the overall stability of the proposed fly ash landfill if the fill height was increased by approximately 10 ft to near EL 776. Exterior slopes are to remain at 4H:1V. Relevant information is attached which was sent to us by BT^2.

Based on our original report dated January 2, 2001, our slope stability analysis for the ash landfill at a 4H:1V slope to EL 766$^{\pm}$ indicated safety factors well above values considered acceptable for this situation (i.e., 1.55 to 2.32). In our opinion, the proposed changes in elevation will have little (if any) effect on the stability analysis results. As such, it is our opinion that resulting safety factors will remain acceptable and risks of movement will remain low.

* * * *

We trust this letter addresses your present needs. Please call if questions.

Sincerely,

CGC, Inc.

Michael N. Schultz

Michael N. Schultz, P.E.
Principal/Consulting Professional

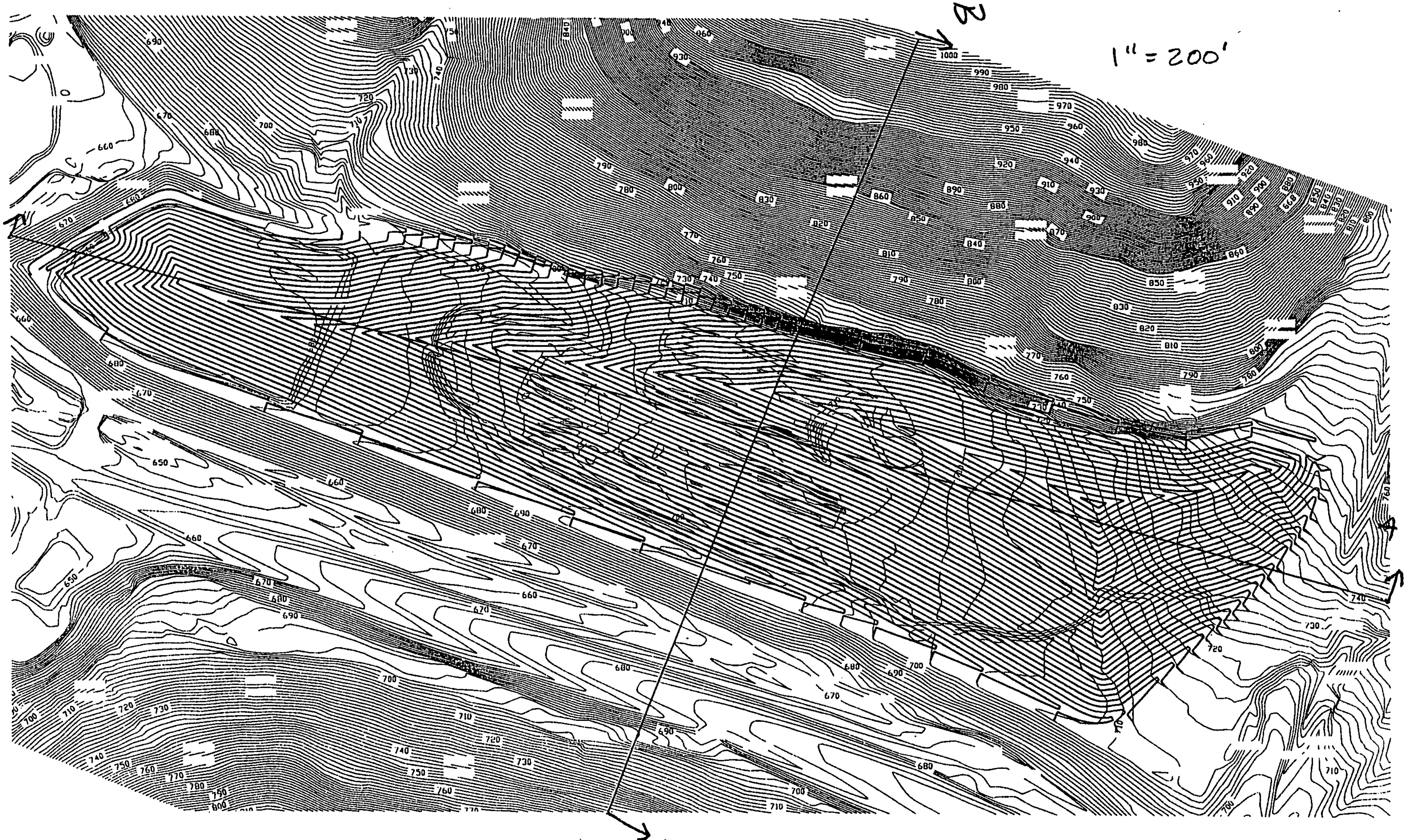
William W. Wuellner / MNS

William W. Wuellner, P.E.
Senior Geotechnical Engineer

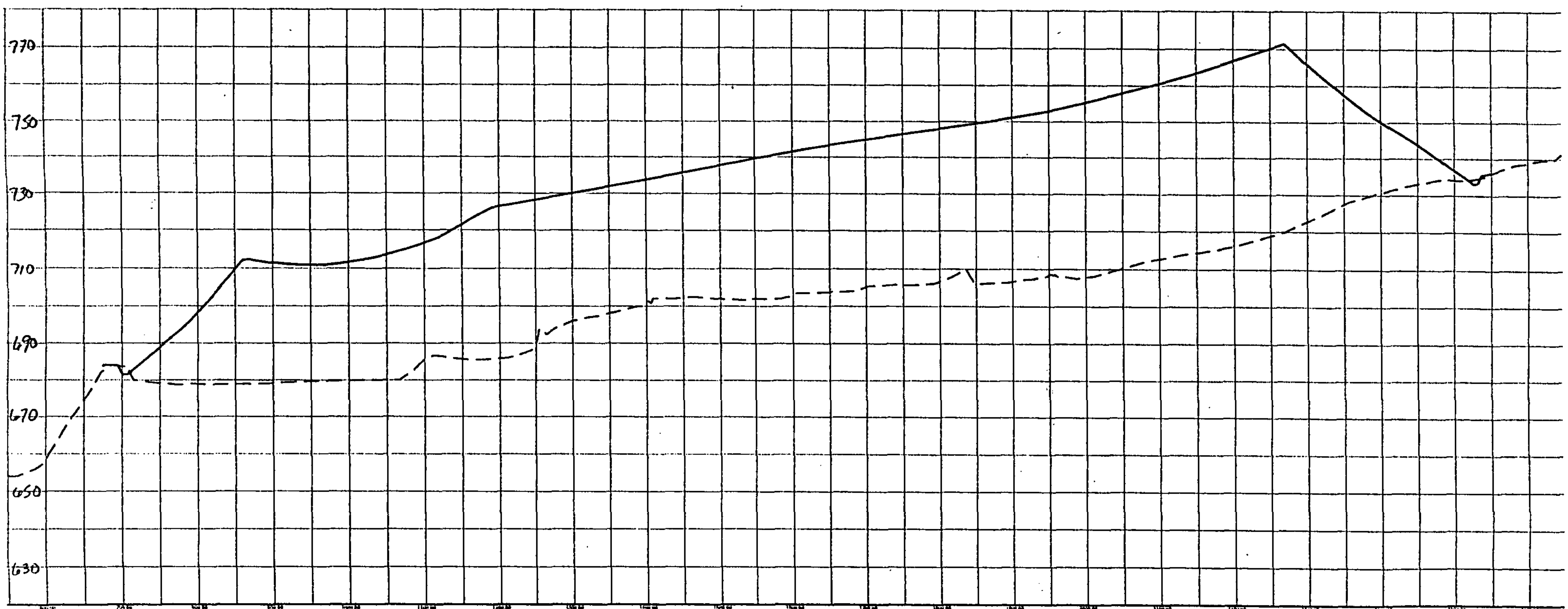
Encl: As stated

D:\Julie\July01\20207.mns

3011 Perry Street, Madison, WI 53713
Telephone: 608/288-4100
FAX: 608/288-7887


1" = 200'
A
A
B
B

A - A



1" = 200' H

1" = 20' V


760.00
750.00
740.00
730.00
720.00
710.00
700.00
690.00
680.00
670.00
660.00
650.00
640.00
630.00
0.00
100.00
200.00
300.00
400.00
500.00
600.00
700.00
800.00
900.00
1000.00
B - B

1" = 100' H

1792

FILE COPY

Environmental Engineering and Science

January 2, 2001

Ms. Linda Lynch
Alliant Energy
222 W. Washington Ave.
P.O. Box 192
Madison, WI 53701-0192

Mr. Ted Shonts
Alliant Energy
2320 Power Plant Drive
Lansing, IA 52151-7539

SUBJECT: Ash Disposal Area Stability Evaluation
Alliant Energy - Lansing Power Station
BT² Project #1792

Dear Ms. Lynch and Mr. Shonts:

This report provides the results of a slope stability evaluation for the proposed expansion of the ash disposal area at the Lansing Power Station. Slope stability had been identified as a potential barrier to vertical expansion of the ash disposal area in previous analysis performed by Terracon Consultants, Inc. Terracon noted the apparent very loose/soft condition of the existing ash fill based on conventional borings using the standard penetration test. In BT²'s Ash Fill Options Evaluation report, dated August 22, 2000, we indicated that vertical expansion of the ash disposal area was potentially feasible and could provide cost-effective ash disposal by filling over the existing plateau area without raising the height of the existing perimeter soil berm. To evaluate this option further, we recommended additional borings, field cone penetration testing of the ash fill to assess its strength and settlement characteristics, geotechnical laboratory testing, and analysis of the slope stability of the proposed expansion.

The stability analysis was performed by CGC, Inc., of Madison, Wisconsin, under subcontract to BT². CGC's report is attached to this letter.

Description of the Proposed Expansion

The stability analysis was performed based on the preliminary design for the disposal area expansion that was outlined in the previous Ash Fill Options Evaluation report. A map and two cross sections showing the proposed design are attached as **Figures 1** through **3**. The key design and operations assumptions that were incorporated into the analysis are based on the following project description.

For the proposed vertical expansion of the existing ash disposal area, ash fill will be placed over the existing ash in the plateau area. Construction of the expansion will involve preparing the site for ash filling, constructing surface water drainage controls, dredging and dewatering the ash, hauling and placing the ash, and constructing a final cover. Unlike the existing ash disposal area, construction of the vertical expansion will not involve construction of perimeter berms with relatively steep exterior side slopes, to be filled with ash. Instead, ash will be placed within the limits of the existing berms, at a maximum slope of 4 horizontal to 1 vertical (4H:1V).

The two attached cross sections (**Figures 2** and **3**) show the proposed expansion with 4H:1V slopes on both the berm side and the bluff side of the disposal area. A possible additional expansion area is also shown, based on filling against the existing bluff. With the additional expansion option, ash would be placed at a 4H:1V slope up to approximately the center of the existing disposal area, then at a 20H:1V slope up to the bluff. For the stability analysis, CGC made the conservative assumption that the additional expansion area would be filled.

Prior to placing ash in the plateau area, vegetated areas will be cleared and grubbed and any existing cover soils will be removed and stockpiled for reuse in the new final cover. The existing ash stock piles will be leveled and compacted prior to placement of new ash. In addition, berms and other stormwater diversion structures will be constructed to divert water away from active fill areas. We assume that ash will be placed to a maximum height of approximately 40 feet above the existing elevation of the plateau area. Following the placement of the ash, a final cover consisting of 2 feet of compacted soil, 6 inches of rooting zone soil, and 6 inches of top soil will be placed, along with seed, fertilizer, and mulch. The cover could be constructed over several years as phases of the landfill expansion are filled to final grades.

We assume that the ash will be dredged and dewatered on-site near the ash sluice pond. We also assume that ash dredging, dewatering, hauling, and placement will occur over a 10-year period.

Stability Analysis

The stability analysis for the proposed vertical expansion of the ash disposal area included the following tasks:

- Additional borings in the perimeter soil berm (5) and one boring in the ash fill;
- Installation of a water table monitoring well in the berm;
- Cone penetration tests in the ash (4) and one test in the soil berm;
- Geotechnical laboratory testing; and
- Slope stability analysis (3 sections).

The results of the stability analysis indicate that vertical expansion of the ash disposal area is geotechnically feasible. For the proposed design, the analysis indicated safety factors ranging from 1.55 to 2.32, based on varying sets of assumed soil parameters. Minimum acceptable safety factors for a project of this type are in the range of 1.3 to 1.5. The only scenario that yielded a safety factor of less than 2 was based on the results from a boring near the south end of the berm, where some soft soils were encountered in the berm.

The details of the analysis methods and results are presented in the attached report prepared by CGC.

Recommendations

If Alliant chooses to move forward with the development of the proposed expansion of the ash disposal area, the next step in the process will be to obtain IDNR approval. To complete the permitting process, we anticipate that the following steps will need to be implemented:

- Obtain current topography of the plateau area.
- Locate existing monitoring wells and install new monitoring wells (assume two new wells).
- Collect hydrogeologic data and groundwater quality data.
- Evaluate operational options for ash dredging, dewatering, and hauling/placement.
- Develop design/permit drawings and specifications and perform associated calculations.
- Prepare feasibility report presenting data collected and analysis performed with updated construction cost estimate.
- Submit permit application to IDNR.

The estimated cost for these tasks in our August 2000 Ash Fill Options Evaluation report was $37,400.

It may also be beneficial to discuss the potential expansion with the IDNR and obtain clarification and approval for the scope of work to be performed for the permit application.

If you have any questions concerning this report, please call us at 608-224-2830. We appreciate the opportunity to work with you on this project.

Sincerely,
BT², Inc.

Sherren Clark, P.G., P.E.
Project Manager

Debra Nelson

Debra Nelson, P.E.
Senior Engineer

Attachments: Figures 1-3
Appendix - CGC Report

cc: Mike Schultz, CGC

I:\1792\Reports\0012stability_rpt.wpd

CGC, Inc.

Construction • Geotechnical
Consulting Engineering/Testing

January 2, 2001
C20207

Ms. Sherren Clark
BT^2, Inc.
2830 Dairy Drive
Madison, WI 53718

Re: Subsurface Investigation
Fly Ash Landfill Expansion
Alliant Energy Site
Lansing, Iowa

Dear Ms. Clark:

CGC, Inc. has completed the geotechnical investigation for the potential expansion to the fly ash landfill at the Alliant Energy site in Lansing, Iowa. This report presents the findings of the exploration program consisting of Standard Penetration Test (SPT) borings , Cone Penetration Test (CPT) probes, field density tests and laboratory tests. The report also provides slope stability analyses for the proposed vertical expansion to the landfill. CGC's analysis and report were performed under subcontract to BT^2.

PROJECT DESCRIPTION

Our understanding of the potential landfill expansion is as follows. The landfill expansion option involves increasing the capacity of the existing landfill by vertically expanding the present fly ash surface about 40 ft above the existing plateau area. The plateau area was created by placing ash within a basin created by the construction of an earthen dike along the west and north edges of the landfill area. Expansion will establish a new fly ash landfill height at about EL 743 using 4H:1V exterior slopes and a 20H:1V slope extending from the peak to the original bluff slope. A final cover measuring about 3 ft thick will be placed over the ash, with berms and diversion ditches also to be constructed to control surface water runoff. Fly ash will be dredged and dewatered on site prior to placement, with placement to be done over a 10-year period using truck hauling and dozer spreading/compaction.

INVESTIGATION

The subsurface conditions of the existing plateau area were investigated by drilling six SPT borings on the present fly ash surface or perimeter dike. Five CPT probes were also conducted on the ash or dike until probe refusal occurred. Locations of the SPT and CPT borings/probes are presented in Attachment B. A sixth location was planned (CPT-5), but could not be conducted because access to the area was prevented by snow.

The SPT borings were drilled by Boart-Longyear (under subcontract to BT^2) on November 27 and 28, 2000. The boring logs are presented in Attachment D. The CPT probes were conducted by Stratigraphics on November 21, 2000, with that data presented in Attachment C. A monitoring well (MW-10) was also installed in SPT-2 by Boart-Longyear to a depth of 29 ft. Additional details regarding drilling and sampling are described in Attachment A.

3011 Perry Street, Madison, WI 53713
Telephone: 608/288-4100
FAX: 608/288-7887

CGC, Inc.

The SPT soil borings and CPT probes reveal that fly ash extends to depths averaging near 35 ft in the northern portion of the basin. The ash thickness tapers off going toward the south. The ash is generally a mix of loose to medium dense sand size particles and/or soft to stiff silt and clay size particles. It is underlain by a weathered rock zone followed by more competent dolomite. The confining berm to the west is generally comprised of medium dense sands and relatively stiff clays. As an exception, the dike near CPT-6 has a tendency to be softer and less dense.

Additional soil borings were conducted by Terracon as part of a study done in 1996. That information is contained in Attachment E of this submittal. Conditions were similar, with the fly ash depths extending to 44 ft in one of their borings.

Free standing groundwater was generally not encountered in the SPT borings or well MW-10. The CPT data suggests a perched condition on the surface of the weathered bedrock/dolomite (refer to "generated pore pressure" column on "CPTU-EC log with Lithologic Evaluation" data sheet for each CPT probe in Attachment C).

LABORATORY TESTING

A sample of the fly ash was obtained by CGC in conjunction with CPT activities on November 21, 2000. It was obtained from on-site stockpiles and appeared to have a grain size distribution that was representative of some of the finest (i.e., least coarse) material on site. This material was selected because it is more susceptible to slope stability failure than the coarse-grained ash. Atterberg limits and grain size/hydrometer tests were performed on that sample by CGC, with those results presented in Attachment F. The results indicate that the tested sample has soil properties that would classify it as a silt.

Two sand cone field density tests on similar ash were conducted by CGC in the field on November 21 and revealed a wet density of 82 pcf for both tests.

Samples of the fly ash from the stockpiles were submitted to the UW Madison geotechnical laboratory for triaxial testing to evaluate shear strength parameters for implementation during slope stability modeling. A series of three unconsolidated-undrained (UU) tests were conducted on ash samples compacted to 82 pcf at moisture contents of 25%, 35% and 45% to simulate anticipated field conditions in the short term. Two additional consolidated-undrained (CU) tests with pore pressure measurements were also done to simulate long term conditions. The results of these tests are presented in Attachment F. Strength test results from the UU and CU laboratory testing found in ~~Appendix~~ Attachment F correlate well with data obtained from the CPT probes for the in-place ashes depicted on ~~Appendix D~~ Attachment C data sheets labeled "Evaluated Properties Using Global Database" under the drained friction angle and undrained shear strength columns.

DISCUSSION AND RECOMMENDATIONS

Based on the laboratory testing and field analysis, a series of cross-sections of the proposed expansion area were evaluated from a slope stability viewpoint. The slope stability evaluation revealed that the proposed expansion is feasible because resulting safety factors against movement exceed typical acceptable levels. The following paragraphs present the stability analysis results, along with soil parameters used in the conceptual design. Important information about the limitations of this report is presented in Attachment H.

Incorporating soil parameters determined from CPT, SPT, field density and laboratory testing programs, CGC performed a slope stability analysis using the computer program STABL5. The program uses the Modified Bishop Method of analysis to calculate factors of safety against sliding along various semicircular arcs, accounting for soil loads, soil shear strength, water levels and other factors.

Key assumptions used in these analyses include the following:

- Soil profile: A soil profile consisting of a composite of the SPT and CPT borings was developed by roughly averaging existing ash depths and natural layer thicknesses. We analyzed for the full expansion option that includes a 20H:1V slope extending from the initial peak at EL 743 to the original bluff slope. This configuration would be more critical from a slope stability point of view than just the initial phase of the vertical expansion. The assumed soil profile is indicated in the figures in Attachment G.

- Water level: Based on water levels encountered in the recent borings, the slope was modeled with groundwater at the base of the existing ash fill.

- Ash Shear Strength Parameters: Because the ash will be placed in the landfill at a relatively slow rate and the ash is moderately permeable, both the existing and future ash fill is expected to develop its shear strength primarily from frictional resistance. Using parameters determined from CU shear strength testing which correlated well with in-situ CPT data, we have conservatively modeled the fly ash as material with a friction angle of 29° and zero cohesion. (Note that the triaxial laboratory testing and CPT probe strength data suggests friction angles as great as approximately 42° on the average could be considered for modeling).

- Potential Weak Zone in Earth Berm: To model the zone of the existing embankment near CPT/SPT 6, where somewhat loose/soft conditions were noted, we used lower strength soil parameters for the earth berm in several analyses. Because the berm fill at this location is a mixture of clay and sand, the analyses were conducted assuming the fill would behave as both a frictional and cohesive material. Shear strength parameters were estimated based on correlations with SPT blow count values and pocket penetrometer readings.

- Failure Plane Analysis: To fully evaluate various modes of failure, parameters in the STABL program were modified to force the potential slip circles through critical sections of the slope. This effort was necessary to check that potential failure surfaces with the lowest factors of safety had been identified. The two modes are identified as "failure through ash slope" and "failure through earth berm".

Out of hundreds of trial arcs of varying radii and centers, the ten arcs with the lowest factors of safety for each condition are shown in Figures G-1 through G-6 in Attachment G. The minimum factors of safety for the proposed slope are summarized in the following table:

TABLE 1

ESTIMATED MINIMUM FACTORS OF SAFETY FOR THE PROPOSED ASH LANDFILL SLOPE

	Typical Berm Strength	"Weak" Berm Parameters
Failure through Ash Slope	2.32	2.32
Failure through Earth Berm	2.31	1.55 - 1.59

Note that a factor of safety of about 1.0 or less indicates incipient slope failure or a high risk of movement.

From this analysis we conclude that the calculated factor of safety for the proposed ash landfill slope is well above the minimum factor of safety of 1.3 to 1.5 desired in this case (Sowers and Sowers, 1970).

*****

We trust this report addresses your present needs. General limitations regarding the conclusions and opinions presented in this report are discussed in Attachment H. If you have any questions, please contact us.

Sincerely,

CGC, INC.

Michael N. Schultz, P.E.
Principal/Consulting Professional

William W. Wuellner, P.E.
Senior Geotechnical Engineer

Encl:	Attachment A -	Field Investigation
	Attachment B -	Soil Boring Location Map
	Attachment C -	CPT Probe Report
	Attachment D -	Log of Test Borings (Boart Longyear)
		Well Detail
		Log of Test Boring-General Notes
		Unified Soil Classification System
		Abandonment Forms
	Attachment E -	Previous Terracon Report
	Attachment F -	Laboratory Test Results
	Attachment G -	Slope Stability Analyses
		• Figures G-1 through G-6
	Attachment H -	Document Qualifications

Reference: Sowers and Sowers, *Introductory Soil Mechanics and Foundations*, 1970, pg 517.

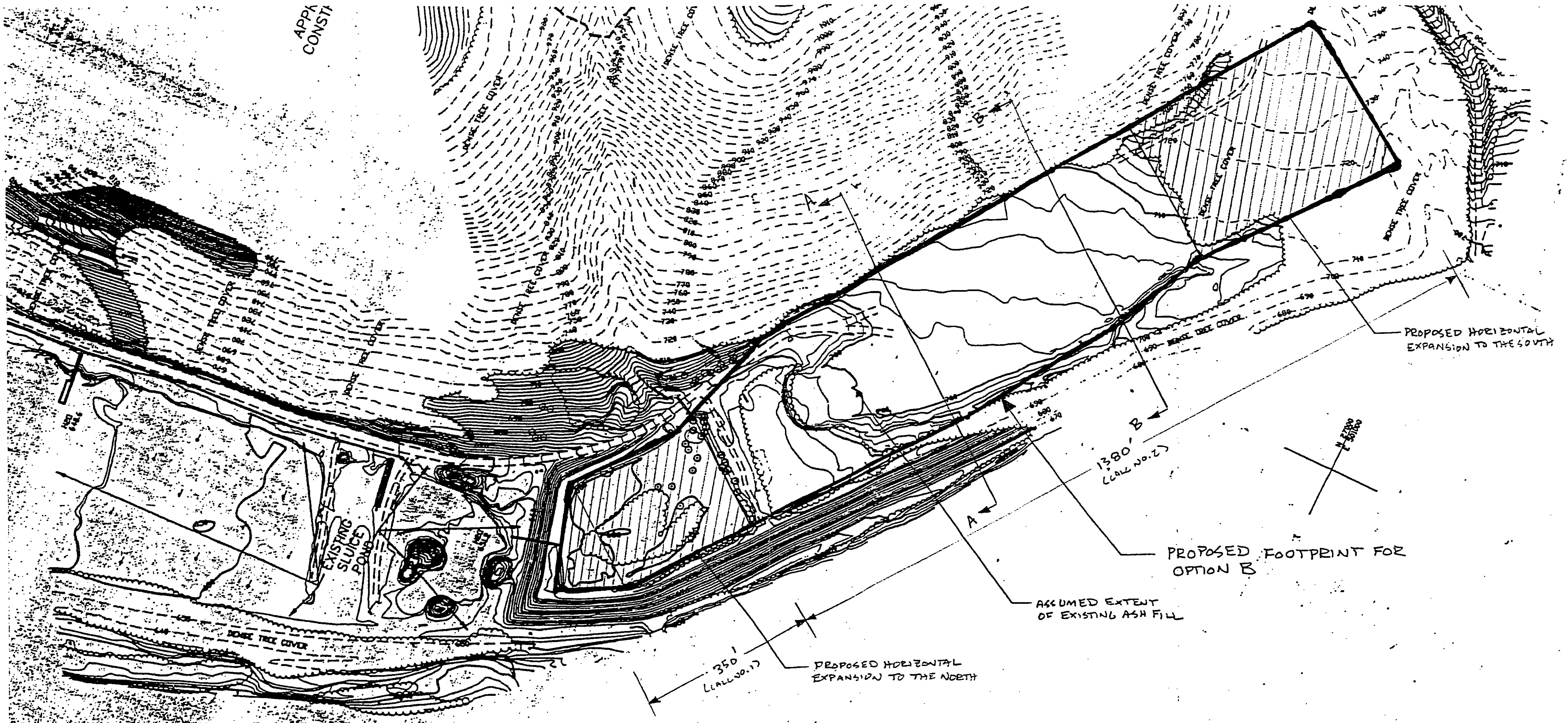
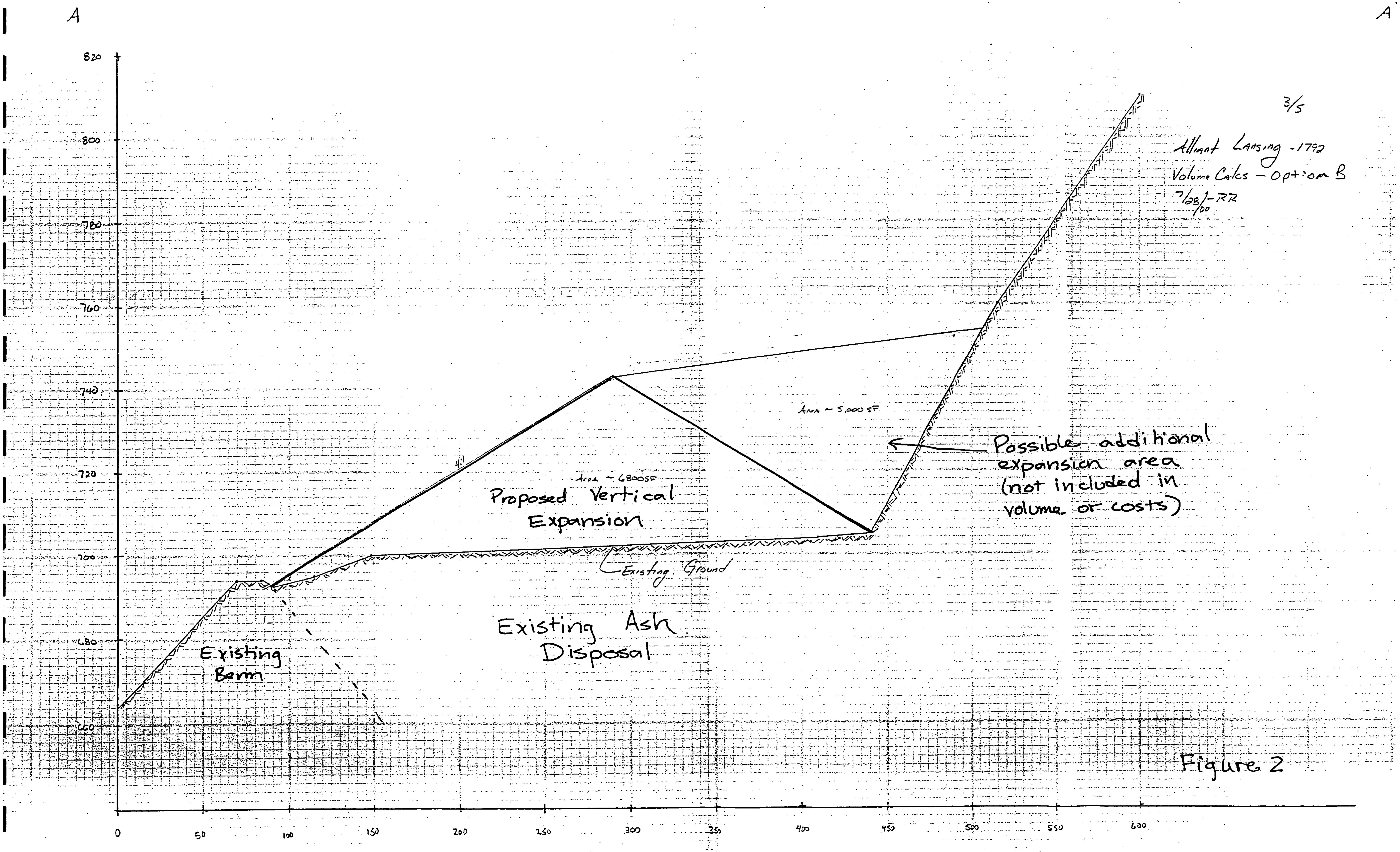


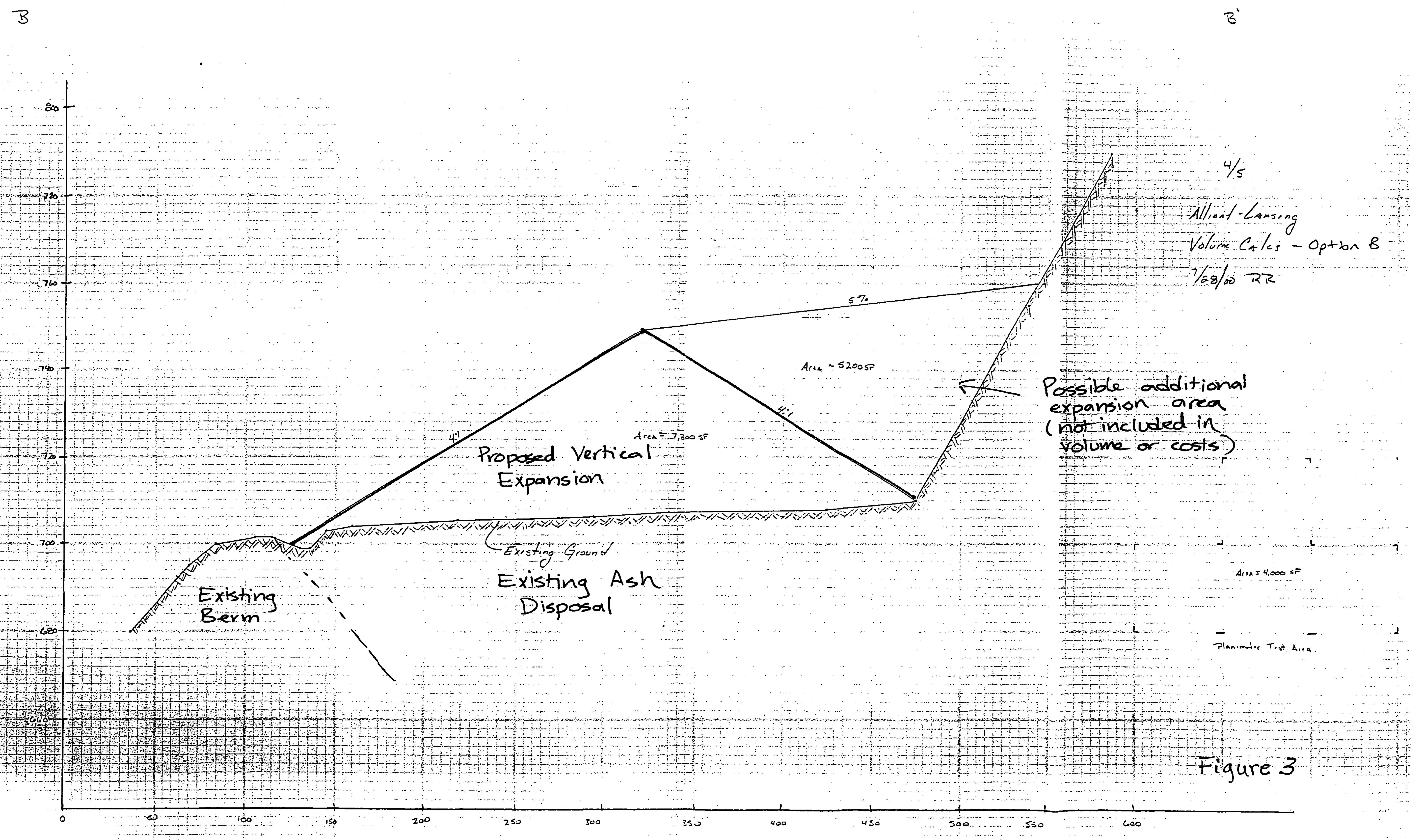
Figure 1

APPROXIMATE SCALE 1" = 200'



A
A'
3/5
Alliant Lansing -1792
Volume Calcs – Option B
7/28/00 – RR
820
800
780
760
740
720
700
680
660
0
50
100
150
200
250
300
350
400
450
500
550
600
4:1
Area ~ 6800SF
Proposed Vertical Expansion
Area ~ 5,000 SF
Possible additional expansion area (not included in volume or costs)
Existing Ground
Existing Berm
Existing Ash Disposal

Figure 2



B
B'
4/5
Alliant-Lansing
Volume Calcs – Option B
7/28/00 RR
800
780
760
740
720
700
680
660
0
50
100
150
200
250
300
350
400
450
500
550
600
5%
4:1
4:1
Area ~ 5200 SF
Area = 7,200 SF
Proposed Vertical Expansion
Possible additional expansion area (not included in volume or costs)
Existing Ground
Existing Ash Disposal
Existing Berm
Area = 4,000 SF
Planimeter Test Area

Figure 3

ATTACHMENT A

FIELD INVESTIGATION

ATTACHMENT A

FIELD INVESTIGATION

Six Standard Penetration Test (SPT) soil borings were drilled on November 27 and 28, 2000 at locations and depths selected by BT^2 in consultation with CGC. The soil borings were drilled by Boart Longyear using a truck-mounted, rotary drill rig equipped with hollow-stem augers. In addition, piezometric cone penetration test (CPT) soundings were performed by Stratigraphics to depths up to about 45 ft. Refusal occurred in each CPT probe hole on assumed bedrock. The boring/sounding locations were staked in the field by taping from existing site features by BT^2. The locations of the borings and soundings are shown on the Boring Location Map presented in Attachment B. Borehole locations and ground surface elevations were surveyed after the borings were completed. Note that a sixth CPT location (CPT-5) could not be conducted because of access problems caused by snow.

Soil samples were obtained at 2.5 foot intervals for the SPT borings. The soil samples were obtained in general accordance with specifications for standard penetration testing, ASTM D 1586. The specific procedures used for drilling and sampling are described below.

1. Boring Procedures Between Samples

   The boring is extended downward, between samples, by a hollow-stem auger.

2. Standard Penetration Test and Split-Barrel Sampling of Soils
   (ASTM Designation: D 1586)

   This method consists of driving a 2-inch outside diameter split barrel sampler using a 140-pound weight falling freely through a distance of 30 inches. The sampler is first seated 6 inches into the material to be sampled and then driven 12 inches. The number of blows required to drive the sampler the final 12 inches is recorded on the log of borings and is known as the Standard Penetration Resistance. Recovered samples are first classified as to texture by the driller.

Detailed procedures for the CPT soundings are included in Stratigraphics' report in Attachment ~~D~~ C.

During the field exploration, the driller visually classified the soil and prepared a field log. *Field screening of the samples for possible environmental contaminants was not conducted by the drillers, as environmental site assessment activities were not part of CGC's work scope.* Water level observations were made in each boring during and after drilling and are shown at the bottom of each boring log. Upon completion of drilling, the boreholes were backfilled with bentonite, and the soil samples were delivered to our laboratory for visual classification and laboratory testing. The soils were visually classified by a CGC geotechnical engineer using the Unified Soil Classification System. The final logs typed and a description of the Unified Soil Classification System are presented in Attachment ~~C~~ D.

ATTACHMENT B

SOIL BORING LOCATION MAP

ATTACHMENT C

CPT PROBE REPORT

PIEZOMETRIC CONE PENETRATION TESTING WITH SOIL ELECTRICAL CONDUCTIVITY MEASUREMENTS AND CPT-EMOD (LOAD/SETTLEMENT) TESTING ALLIANT POWER PLANT FLYASH LANDFILL LANSING, IOWA

Prepared for:

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November, 2000
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FIGURES AND TABULATIONS

1.0 INTRODUCTION

STRATIGRAPHICS, The Geotechnical Data Acquisition Corporation, performed cone penetrometer exploration at the Alliant Power Plant Flyash Landfill Site near Lansing, Iowa. Piezometric Cone Penetration Test with soil Electrical Conductivity measurement (CPTU-EC) soundings, with CPTU dissipation and CPT-EMOD load/settlement tests, were performed to provide data on geotechnical properties of subsurface soils for BT2, Inc. All work was performed under the direction of Mr. Michael Shultz of CGC, Inc.

The work was performed on November 21, 2000. Five CPT soundings were completed to depths ranging from 28.3 to 49.8 ft, for a total of 205.4 ft of data. Three CPT-EMOD tests and three CPTU dissipation tests were performed for evaluation of soil deformation properties.

This report includes CPTU-EC sounding logs and tabulations of recorded data and correlated geotechnical parameters. CPT-EMOD data are presented in graphical and tabular form for each test. CPTU dissipation data are presented in graphical form, and are summarized in a table. Digital data are presented for each CPTU-EC sounding on the attached data disk, along with JPEG images of the logs. Details of penetrometer exploration techniques are included in the main body of the report, both for this study and for penetrometer uses in general. A statement of limitations is presented in Section 9 of the main body of this report. Recommended practices when using CPT for design are presented in Appendix A, Section 4.0.

2.0 PENETROMETER EQUIPMENT AND DATA ACQUISITION

2.1 Procedure The Cone Penetration Test (CPT) consists of smoothly and continuously pushing a small diameter, instrumented probe (penetrometer) deep into the ground while a computer data acquisition system displays and records the soil response to penetration (Figure 1). In geotechnical terms, the CPT penetrometer models a foundation pile under plunging failure load conditions. CPT data are used to develop continuous, high resolution profiles of in situ soil conditions rapidly, accurately and economically.

The soil resistance to penetration, acting on the tip and along the sides of the penetrometer, is measured during CPT. CPT soil resistance measurements are accurate and highly repeatable. The measurements can be used for the evaluation of stratigraphy and various geotechnical parameters. Performance of CPT is specified by ASTM Standard D3441.

A pressure transducer is added to the CPT penetrometer to acquire hydrogeologic data (Saines and others, 1989) and is called a Piezometric Cone Penetration Test (CPTU). A soil electrical conductivity sensor is added to the penetrometer (CPTU-EC) to acquire qualitative moisture information in vadose zone soils, and general groundwater quality data (Strutynsky and others, 1991, 1998). Penetrometer groundwater, soil, and soil gas samplers are used for direct sampling (Strutynsky and Sainey, 1990, Strutynsky and others, 1998). Recent advances in penetrometer instrumentation include a natural gamma sensor, induced UV fluorescence for detection of hydrocarbons and other compounds, and shear wave velocity and stress controlled testing for low and high strain soil deformation evaluation.

The penetrometer is mounted at the end of a string of sounding rods. A hydraulic ram is used to push the penetrometer and rod string into the ground at a constant rate of 4 ft per minute. Electronic signals from the downhole sensors are transmitted by a cable, strung through the sounding rods, to the computer data acquisition system. Measurements are displayed and recorded for immediate definition of subsurface conditions. Downhole equipment can be automatically steam cleaned during retrieval at the end of a test. Open hole can be grouted using a bentonite clay grout.

Large 3 axle trucks are used to carry the 2 penetrometer systems used by STRATIGRAPHICS. Truck weight and ballast serve to counteract the thrust of the hydraulic ram. The enclosed rig work area allows all-weather operations. Computers, samplers, electrical power, lighting, compressed air, steam cleaner, grout pump, and water tank are all included on each rig, providing for self-contained operations. Other systems for mounting on drill rigs can be used in areas with poor access or for overseas projects.

Lightning detection systems are mounted on the rigs to monitor dangerous weather conditions that can affect safety and productivity. Differential, carrier phase, post processed Global Positioning Systems (GPS) are also mounted on the rigs to allow surveying exploration points to an accuracy of about 5 to 15 cm.

No borehole is required during exploration because penetrometers are directly thrust into the soil from the ground surface. Pressures of over 3 million pounds per square foot can be applied to the tip of the penetrometer for penetration of most soils finer than medium gravel. Asphalt pavements up to 6 inches thick can usually be penetrated by penetrometer methods without predrilling. Site disturbance is reduced since no borehole cuttings or drilling fluids are generated during penetrometer operations. Personnel exposure to possibly contaminated soil is significantly less than exposures during drilling and sampling operations. Penetrometer downhole equipment can be decontaminated during retrieval.

Four hundred to thirteen hundred feet of CPT (with no time dependent piezometric, gamma or shear wave velocity measurements) can be performed in one day, depending on site access. Depths of more than 200 ft can be achieved, depending on site stratigraphy. Where soils are exceptionally dense or gravelly, an uninstrumented prepunch tool can be used to probe the subsurface. Information obtained using the prepunch tool can be similar to that obtained during mechanical (Dutch) cone testing, especially where friction on the sounding rods is minimal. Dynamic driving can be used in gravelly soils.

2.1.1 Signal Conditioning and Recording CPT data are acquired using a 16 bit (resolution of 1 part in 32,768) analog to digital data logger and PC field computer. Sounding logs are graphically displayed and printed for immediate evaluation of subsurface conditions. Data are recorded on disk for data processing and archiving.

2.2 Soil Shear Resistance Measurements The soil penetration resistance is measured on the tip and along the sides of the CPT penetrometer (Figure 1). The conical tip of the penetrometer has a projected cross-sectional area of 15 square centimeters (2.3 square inches), and a diameter of 1.7 inches. The cone tip resistance reflects the deep bearing capacity of the soil. The friction between the soil and the penetrometer is measured along a cylindrical sleeve mounted behind the cone tip. The friction sleeve has a surface area of 200 square centimeters (31.0 square inches), a length of 5.8 inches, and a diameter slightly larger than the cone tip. Two strain gage loadcells are used to measure the tip and friction sleeve resistances (Strutynsky and others, 1985). The tip measurement has a layer resolution of about 2 to 4 inches, and the friction sleeve about 6 inches.

2.3 Piezometric Measurements A fluid pressure transducer is used to measure the soil pore water pressure response to penetration. The CPTU piezometric measurement has a layer resolution of about 1 inch. The advance of the penetrometer causes volumetric distortion of the soil, which generates a local water pressure field. These generated pressures dissipate almost instantaneously in soils of high permeability, so equilibrium water pressures are measured during CPTU in coarse sand and gravel. In medium or low permeability soils, the generated water pressure field is sustained for a lengthy period of time (Saines and others, 1989).

The dissipation of generated water pressure can be recorded during pauses in the penetration process. If the pauses are long enough for all generated water pressures to dissipate, potentiometric surface measurements can be obtained at multiple depths in a single CPTU sounding. The dissipation test is also used to estimate soil hydraulic conductivity and consolidation characteristics.

2.3.1 Piezometer Saturation The CPTU piezometer filter is saturated with an incompressible liquid so that instantaneous responses (zero lag time) can be achieved during testing. High saturation levels are indicated by sharp responses at interfaces and immediate regeneration of excess water pressure after pauses in penetration. Low saturation levels leading to poor measurements can be caused by inadequate equipment preparation, soil suction, or filter damage on coarse soil particles. Clogging of piezometric filters can also lead to poor results. Loss of filter saturation or clogged filters are beyond the control of the operator. Thus, CPTU piezometric measurements can be less repeatable than CPT tip and friction sleeve resistance measurements.

2.4 Electrical Conductivity and Thermal Measurements A CPTU-EC penetrometer including tip, sleeve, piezometric, temperature, and electrical conductivity (EC) sensors can be used to simultaneously acquire geotechnical, hydrogeological and qualitative geochemical information. Soil EC is measured using a two electrode array, energized with a 3 kHz signal, mounted on the penetrometer tip. The EC measurement has a resolution of about 0.75 inches. A thermal sensor can also be mounted inside the penetrometer. Significant frictional heating occurs when penetrating sandy soils. During pauses, the generated heat will dissipate and the penetrometer will reach thermal equilibrium with the soil. This allows a soil temperature profile to be acquired.

2.5 Natural Gamma Measurements A CPTU-ECG penetrometer incorporating cone, friction, piezometric, soil electrical conductivity and natural gamma (G) sensors can be used to simultaneously acquire geotechnical, hydrogeological, qualitative geochemical and radiological information. Gamma measurements can be used to enhance lithologic interpretation. Radionuclide contamination may also be detected using gamma logging.

2.6 UV Fluorescence A CPTU-EC-UVF penetrometer incorporating cone, friction, piezometric, soil electrical conductivity, and UV Fluorescence (UVF) sensors can be used to simultaneously acquire geotechnical, hydrogeological, and qualitative geochemical information. The UVF system consists of a sapphire window in the penetrometer, a UV excitation light source, and photodiode light detectors. UV light is transmitted through the window into the adjacent soil. If the soil contains compounds, such as petroleum hydrocarbons, that fluoresce, the resulting light is detected and recorded.

The UV light source is bandpass filtered to provide an excitation wavelength of 254 nm. The photodiode sensors are longpass filtered to monitor resulting fluorescent light emissions above 290 nm. Future improvements to the UVF module may include a photonic sensor array, filtered at several different wavelengths to allow some compound differentiation capabilities.

2.7 CPT Shear Wave Velocity Measurements A geophone module is attached to the penetrometer and is deployed similarly to other penetrometer sensors. A main advantage of pushed in geophones is that they have superior coupling to the soil, resulting in much better definition of wave arrival, as compared to borehole deployed geophones. The shear wave system consists of a pair of downhole geophones, an uphole wave source and timing trigger, signal conditioning and A/D, signal enhancement and acquisition software, and the PC data acquisition computer.

The CPT shear wave velocity data acquisition procedure is as follows: 1) the geophone module is pushed to the required depth; 2) the data file/signal conditioning is initialized; 3) a hammer/timing trigger is used as an uphole wave source (in a polarized mode); and 4) the output of the geophone pair is recorded as a function of time after the initial trigger. The data acquisition software allows signal stacking to enhance the picking of wave arrival times. The procedure is repeated at multiple depths, to allow calculation of interval wave velocities between adjacent tests.

2.8 CPT-EMOD measurements The standard CPT procedure is conducted as a constant rate of strain test, resulting in a continuous measurement of soil ultimate bearing and frictional strength. By conducting CPT under monotonically increasing stress conditions, soil deformation properties can be evaluated. The CPT-EMOD test is conducted during short pauses in the continuous push process. Load/settlement data are analyzed using elastic theory, as might be done for a plate load test, for evaluation of Young's Modulus at various stress levels.

2.9 Penetrometer Geometry The CPT penetrometer external geometry is specified by ASTM standards. Differences in penetrometer internal design can lead to some variability in response between penetrometers of different manufacture, especially in very soft clays. The CPTU measurement of generated water pressure depends on external filter geometry. Measurements of equilibrium water pressures after pauses in the penetration process are not sensitive to geometry, and reflect undisturbed conditions. CPTU piezometric filters are typically mounted on either the cone tip (U1 position) or just ahead of the friction sleeve (U2 position). Each position has advantages and disadvantages. Measurements taken with the cone tip U1 filter are at a maximum and show high resolution of thin soil seams. The cone tip U1 filter is prone to damage on coarse soil particles.

Negative pressures are often measured in dense, silty or clayey sands and hard clays when using the U2 friction sleeve filter. These low pressures are probably caused by soil elastic rebound (expansion) as the soil moves from the intensely loaded region beneath the cone tip to the less loaded region next to the friction sleeve. Soil expansion can induce large suction forces on the U2 friction sleeve filter, which can result in decreased filter saturation levels.

Site characteristics and data usage determine which piezometric filter geometry is appropriate. The piezometric filter is placed at the U2 friction sleeve position on the CPTU-EC penetrometer. Generally good results can be obtained using this geometry when proper preparation techniques are followed.

2.10 Downhole Equipment Decontamination and Open Hole Grouting The rod string is retrieved through a rodwasher mounted on the hydraulic ram assembly. High pressure hot water is sprayed from internal nozzles to clean the rod string. Wash water (½ gallon per 10 ft of rod) can be captured for disposal.

The STRATIGRAPHICS grouting system can be used to seal open hole. As penetrometers are being advanced, bentonite grout is pumped into the annular space formed between the smaller diameter sounding rods and the larger diameter penetrometer. A bypass is opened and additional grout is pumped to seal the hole during rod string retrieval. About 3/4 gallons of grout are required to seal 10 ft of open hole.

Pressure grouting during sounding advance can control cross-contamination between different strata. The grout decreases the contact of downhole equipment with contaminated soil. The grout also can decrease friction on the sounding rods, which may allow deeper penetration. Grout levels are checked after sounding completion, and additional grout can be added to account for penetration of grout into permeable strata.

3.0 PENETROMETER SAMPLING EQUIPMENT

Groundwater, soil gas, and soil samplers are deployed in the same manner as CPTU-EC penetrometers. Good sample isolation is achieved because no open hole exists during penetrometer operations.

3.1 Groundwater Sampler The STRATIGRAPHICS groundwater sampler is a shielded wellpoint sampler of heavy construction. The shield prevents sampler contamination while penetrating soils above the sampling depth. After shield retraction, groundwater flows under in situ pressure conditions, through a 20 inch long screen, into the 350 ml sample barrel. The sampler is retrieved to pour off the sample and for decontamination. Small diameter pumps can be used with the sampler to acquire large volumes of sample. This sampler can be deployed in any soil capable of being penetrated by the CPTU-EC penetrometer (Strutynsky and others, 1998).

A pressure transducer can be placed inside the sampler barrel. This allows the measurement of sample inflow rate. Analysis of inflow data using rising head slug test methods can provide a means of estimating soil hydraulic conductivities. If equilibrium conditions are reached, a measurement of the static water pressure head is obtained during groundwater sampling.

3.2 Soil Gas Sampler The STRATIGRAPHICS soil gas sampler is a shielded screen sampler, similar to the groundwater sampler. The shield is opened by pulling back the rod string during sampling, and soil gases are then extracted. The shield can be closed, and the rod string advanced to another depth, allowing multiple samples during a single rod trip. Soil gasses are extracted from the rod string. A vacuum box can be used to inflate Tedlar bags for off site analysis. Portable analytical equipment can be used to allow immediate analysis. The sampler, rod string and tubing are purged before sampling.

3.3 Soil Samplers Fixed piston samplers can be used to obtain soil samples during penetrometer exploration. The STRATIGRAPHICS and MOSTAP 2-meter samplers are deployed similarly to a penetrometer. A piston, locked into the tip of the barrel to prevent soil from entering the sampler prematurely, is released at the top of the sampling interval, and the barrel is then advanced. Soil enters the barrel and is retained by a core catcher. The sampler is retrieved to remove the sample and for sampler decontamination.

The MOSTAP Sampler is used to obtain 1 inch diameter samples as long as 2 meters (78 inches). This sampler incorporates a PVC liner and a nylon stocking to allow retrieval of such a long sample. As the sample enters the sampler, it is encased in the nylon stocking. The stocking lessens soil friction around on the sample as it enters the PVC liner. At the end of the 2 meter run, the sampler is rotated to twist the stocking, helping retain the sample. This sampler can only be used in softer soils.

4.0 PIEZOMETER INSTALLATION TECHNIQUES

Penetrometer methods can be used to install piezometers for water level measurements, slug testing, groundwater sampling, and for remediation activities, such as sparging and soil vapor extraction (SVE). Various installation techniques are available (Saines and others, 1989). Proprietary, low volume change piezometers also can be installed using penetrometer equipment. These piezometers are often used for long term water pressure measurements during geotechnical projects.

PVC piezometers are most often installed using a steel casing pushed to depth. The steel casing is sealed with an expendable tip, which prevents soil from entering the casing during deployment. The PVC screen and risers are lowered into the casing. The steel casing is then withdrawn, leaving the expendable tip and PVC piezometer in place.

5.0 DATA REDUCTION

Test data are monitored as the soundings are performed. Data are recorded on hard disk and may consist of: depth, time, tip and sleeve resistance, generated water pressure, EC, UVF, temperature and natural gamma. Data are processed in-house for final reporting. Before final reporting, data pass a quality control review. Routine checking of proper equipment performance is conducted in the field. Office review helps assure that data quality is maintained throughout the study.

Several parameters can be computed to enhance data correlation:

friction ratio, FR (in %):

$$FR = fs/qc * 100 \qquad \text{(Eq. 1); and}$$

pore pressure ratio, Bq (dimensionless):

$$Bq = (U-Ue)/(qc-Sv) \qquad \text{(Eq. 2);}$$

where: fs is the measured friction sleeve resistance, in TSF;
qc is the measured cone end bearing resistance, in TSF;
U is the measured generated pore water pressure, in TSF;
Ue is the measured or estimated equilibrium pore water pressure, in TSF; and
Sv is the total soil overburden pressure, in TSF.

Measured data and correlated parameters are presented in a graphical sounding log format for each sounding; numerical data are typically tabulated at 0.5 ft intervals. Tabulated digital data are attached on disk.

CPTU dissipation test data are recorded as a function of time during pauses in the penetration process. The CPTU dissipation data are normalized using the following equation:

normalized dissipation level, U* (dimensionless):

(Ut - Ue) / (U0 - Ue) (Eq. 3);

where: Ut is the excess pore water pressure at time t, in TSF;

Ue is the measured or estimated equilibrium, undisturbed pore water pressure (in situ pore water pressure before penetrometer insertion), in TSF; and

U0 is the excess pore water pressure at time equal to zero, at the start of the dissipation test, in TSF

The normalized dissipation level is plotted versus log scale time. In uniform soils, the plot takes the shape of a reverse S-curve, beginning at 1.0 at zero time (at the instant the penetration process is stopped) and falling to 0.0 when equilibrium pressures are achieved. Boundary effects in interbedded deposits can cause deviation from this ideal.

An estimate of the horizontal coefficient of soil consolidation can be calculated (Baligh and Levadoux, 1980) using:

Ch (in cm**2/sec) = (r**2*T)/t (Eq. 4a).

Estimates of soil hydraulic conductivity in the horizontal direction can be calculated using:

kh (in cm/s) = ((r**2*T)/t)*RR*(Gw/(2.3*Sv')) (Eq. 4b);

where: r is the penetrometer radial dimension at the plane of the piezometric filter, equal to 2.2 cm for the friction sleeve filter and 1.9 cm for the cone tip filter;

T is a dimensionless time factor at the 50% normalized dissipation level, equal to 5.5 for the friction sleeve filter and 3.8 for the cone tip filter;

t is the measured time, in seconds, at which the normalized dissipation level is 50%;

RR is a dimensionless soil compressibility parameter;

Gw is the unit weight of water, in kg/cm***3; and

Sv' is the effective soil vertical overburden pressure, in kg/cm**2.

Dissipation test data can be presented in graphical plots and are summarized in tabular form.

6.0 GENERAL DATA EVALUATION

<u>6.1 Sounding Log</u> The CPTU-EC sounding logs provide high resolution information on subsurface conditions. Soil layering is often highly apparent. Soil relative strength and saturation levels can also be evaluated. Zones of anomalous soil electrical conductivity can be identified. Lateral continuity of conditions can be developed by comparing adjacent sounding. Digital CPTU-EC data files can be used in two and three dimensional data visualization, CAD or GIS software programs.

<u>6.2 Soil Type Classification</u> Correlations between penetrometer data and soil classification have been developed from geotechnical bearing capacity theory and a relational database on adjacent CPT soundings and drilled boreholes (Douglas and Olsen, 1981). A CPT soil classification chart based on cone tip resistance and friction ratio is presented in Appendix A.

The CPT tip resistance increases exponentially with soil grain size. For example, tip resistance in dense sands ranges from about 100 to 400 tons per square foot (TSF), while tip resistance in a stiff clay ranges from about 5 to 15 TSF. The friction ratio (Section 5.0) is also used for indication of soil type. The friction ratio increases with the fines content and compressibility of a soil. The friction ratio is less than about 1% in a sand and greater than about 3% in a clay.

Correlated CPT soil classifications reflect the soil shear resistance to penetration. Soil shear resistance is not entirely controlled by grain size distribution. However, correlated CPT classifications generally agree with classifications based on grain size distribution methods, such as the Unified Soil Classification System (USCS).

The generated water pressure measurement may be useful for classification of saturated soils. Penetration of coarse sand and gravel occurs under drained loading conditions, and equilibrium pressures are measured during penetration. The pore pressure ratio (Section 5.0) is zero in high permeability soils.

For saturated soils of permeability less than about 1*10E-2 cm/sec, undrained loading with significant excess water pressure generation occurs during CPTU. Positive excess water pressures are generally measured during penetration of silt or clay soils when using either the cone tip or friction sleeve filter penetrometer (Section 2.7). Pore pressure ratios of fine grained soils typically range from about 0.4 to 1.0.

Positive excess water pressures are also usually measured in dense, silty or clayey sands when using the cone tip filter penetrometer, with pore pressure ratios from about 0 to 0.3. Due to geometric effects (Section 2.7), negative pressures are usually measured in dense, silty or clayey sands, sandy silts, or hard sandy clays with the friction sleeve filter penetrometer. Thus, it is important to note the type of piezometer in use. The CPTU-EC penetrometer uses a friction sleeve piezometric filter.

6.3 Potentiometric Surfaces Equilibrium water pressures are measured during penetrometer advance in saturated, coarse sands and gravels. Measurements of equilibrium water pressures can be obtained during CPTU in lower permeability soils by pausing during penetration and allowing generated water pressures to dissipate to equilibrium conditions.

6.4 Soil Saturation Soil saturation often can be evaluated using the CPTU sounding log. Atmospheric (zero) pressure is measured during CPTU in unsaturated soils. Hydrostatic pressures are measured in saturated, high permeability soils. Significant water pressures are generated in saturated, low permeability soils due to penetrometer advance. Decreased levels of water pressure generation can be indicative of partially saturated soils. Decreased water pressure generation also may occur in organic soils due to the high compressibility of organic soil particles and the presence of biogenic gases, such as methane and hydrogen sulfide.

6.5 Soil Hydraulic Conductivity Excess water pressures are generated by penetrometer advance in saturated soils with permeability of less than about 1*10E-2 cm/sec. These pressures can be allowed to dissipate during pauses in the penetration process. The CPTU dissipation test is similar to a falling head slug test and can be used to estimate soil hydraulic conductivity in the horizontal direction. Very high water pressures are typically generated in low permeability soils by penetrometer advance. The large water pressure changes require soil compressibility (storage) effects to be included in analyses. The CPTU tip resistance provides an index of soil compressibility for these computations (Section 5.0).

6.6 Soil Electrical Conductivity Behavior Soil electrical conductivity (EC) is controlled by the conductance of both the soil particles and soil pore fluids. The ratio between pore fluid and soil-pore fluid electrical conductivity is termed the formation factor (Archie, 1942). Clays can be electrically conductive due to adsorbed water and ionic electrical charges on the clay platelets. Clay EC depends on mineralogy, porosity and pore fluid characteristics. Sand grains are typically non-conductive, so granular soil conductance is primarily dependent on the conductance of pore fluids and the sand's porosity. The following factors affect granular soil EC: **Pore fluids** Pore fluids play a major role in sand EC. A dry sand has low conductance since both the sand grains and the air in the pore space have very low EC. Sands saturated with conductive liquids, such as brine or landfill leachates, have high electrical conductivity. Hydrocarbons typically decrease EC because of their low conductance. **Saturation** Soil saturation has a pronounced effect on soil EC, as conductance increases with water saturation. Low saturation is typically associated with low EC. **Porosity** The low porosity of a dense sand results in less pore fluid available for electrical conductance and thus lower EC; the high porosity of a loose sand is associated with higher EC. Formation factors vary as an inverse function of porosity, from about 3 at high porosity to about 4.5 at low porosity. **Clay content** The addition of as little as 5% clay to a sand can significantly increase soil electrical conductance (Windle, 1977). **Gravel Interference** The high resolution of the STRATIGRAPHICS CPTU-EC electrode array makes measurements sensitive to soil grain size. Two behaviors can occur when penetrating gravelly soils. One can occur when a large particle is crushed against an electrode, masking it from the pore fluids, which results in very low EC values. This can result in false positive hydrocarbon interpretations. An opposite behavior is observed in gravel deposits which contain few fine grained, intersticial soils. The resolution of the EC measurement is so high that electrical conductance paths are often entirely within the pore fluid of the coarse grained soil. In this situation, high EC values are measured, more closely reflecting pore fluid EC, rather than soil EC.

6.7 EC Evaluation EC data are evaluated in conjunction with piezometric data and soil types for qualitative geochemical characteristics. Anomalous zones possibly indicative of contaminants can be directly sampled for quantitative chemical analysis.

Vadose Zone Low or zero EC values are measured in dry sandy soils. Increased EC in sands above the water table may indicate moisture infiltration. Low EC data in silty or clayey soils can be anomalous as fine grained soils often retain significant amounts of moisture within their pore spaces, creating good conditions for electrical conductance. Thus, low EC values in silty or clayey soils in the vadose zone may indicate hydrocarbon contamination. Elevated EC values in the vadose zone may be associated with road deicing salts, buried metals and rusted metal objects, flyash and cinders, among others.

Saturated Soils Low EC values in saturated soils can be indicative of anomalous geochemistry. In particular, depressed EC zones immediately at the water table may be associated with floating (LNAPL) compounds. Very low EC zones at interfaces between aquifers and aquitards may be associated with either LNAPL or DNAPL compounds. Gravel interference must be considered when evaluating depressed EC zones in saturated soils.

Elevated EC values in saturated soils can be due to increased soil clay content or to increased dissolved salts in the ground water. Increased clay contents are evaluated based on the CPTU-EC piezometric data and soil type information. Zones of elevated EC immediately above an aquiclude may be associated with brines (Strutynsky and others, 1998).

6.8 UV Fluorescence Behavior Fluorimetry (measurement of fluorescence) has been used for many years for the detection and identification of various compounds and minerals. An excitation light of short wavelength is used to expose the specimen. If fluorescent compounds or minerals are present, light of longer wavelength, as compared to the excitation wavelength, will be emitted from the specimen. This resulting light can be monitored for intensity and spectral distribution.

Compounds that fluoresce include a wide range of hydrocarbon and other organic compounds. Heavy hydrocarbons (e.g. fuel oil and coal tars) fluoresce at relatively long wavelength excitation. As excitation wavelength decreases below about 300 nm, fluorescence from lighter hydrocarbons (e.g. jet fuel and gasoline) is observed. In addition to hydrocarbons, other compounds and minerals, such as fluorites and other carbonates, also exhibit fluorescence. Compounds that fluoresce include dyes and optical brighteners. Dyes and brighteners can be found in paints, detergents, antifreeze compounds, some food additives and cosmetics, among others. UVF response will be affected by the presence of any such compounds.

6.9 CPT-SPT Correlation Since most geoscientists are familiar with drilling and split spoon sampling, CPT data have been correlated with SPT blowcount N-values. The SPT N-value is defined by ASTM to be the number of blows of a 140 lb hammer, dropped 30 inches, required to drive a 2 inch outside diameter sampler 12 inches into the bottom of the borehole, after an initial seating drive of 6 inches. Correlations of CPT to the crude SPT have been based on numerical modeling of the two penetration processes and on side by side comparisons (Douglas and others, 1981). Additional details on CPT-SPT correlations are included in Appendix A.

7.0 GEOTECHNICAL DATA CORRELATION

CPT data have been correlated with soil type, drained friction angle, undrained shear strength, relative density and SPT blowcounts, among others. A correlation scheme including tip resistance and friction ratio has generally proved most useful for evaluating CPT data. Correlation of CPT data with other parameters has been developed using: 1) comparisons between CPT data and results of other in situ and laboratory tests in adjacent boreholes; 2) CPT testing on large scale soil samples of known composition; and 3) geotechnical bearing capacity and cavity expansion theory. Site specific information can be used to fine tune correlations. Additional information on correlation techniques, including overburden pressure normalization, test drainage conditions and recommended practices, is presented in Appendix A.

8.0 PROGRAM RESULTS

Acquired data are presented following the report text and consist of: 1) sounding logs with lithologic evaluation; 2) data presentation sounding logs; 3) tabulations of correlated geotechnical parameters, including soil classifications; and 4) CPT-EMOD and CPTU dissipation test results. It should be noted that the computerized correlations of soil types and other geotechnical properties were generated using a global rather than a site specific data base. Use of site specific data was beyond the scope of this study.

9.0 STATEMENT OF LIMITATIONS

Subsurface information was gathered only at the sounding locations. Extrapolation of sounding data to develop stratigraphic continuity is conjectural. Actual site conditions between sounding locations may differ.

Computer correlation of penetrometer data with other parameters was performed using generalized charts rather than on site specific information. Site specific correlation work based on results of detailed laboratory testing was beyond the scope of this study. Evaluation of soil saturation and potentiometric surfaces is only representative of conditions encountered during the field program. Seasonal variation must be expected.

Data gathering for this study was attempted to be performed in general accordance with accepted procedures and practices. Correlation of penetrometer data with other parameters is empirical and should not be considered as the exact equivalent of laboratory testing. STRATIGRAPHICS shall not be responsible for another's interpretation of the information obtained for this study.

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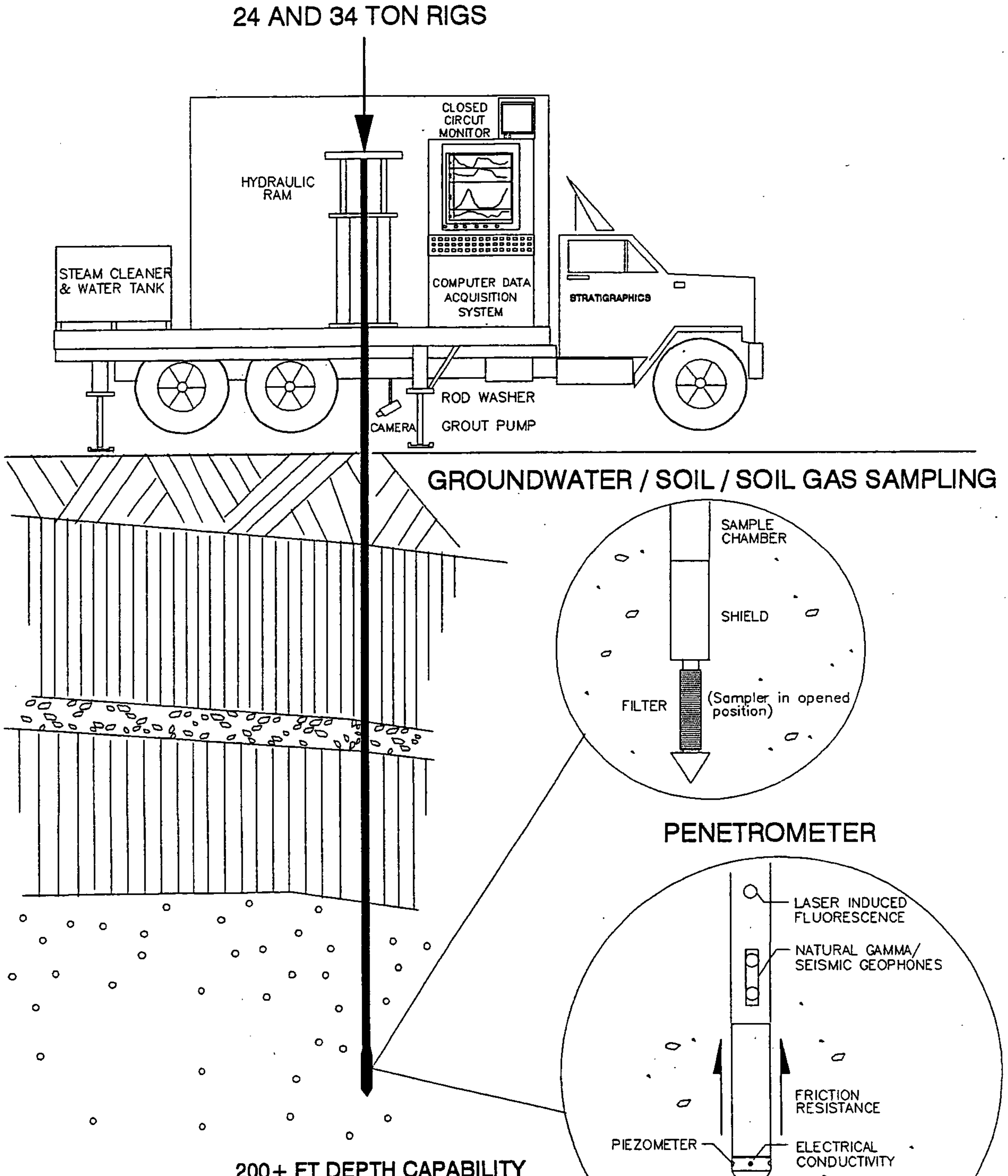

PENETROMETER EXPLORATION SYSTEM

STRATIGRAPHICS

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TABLE 1
STRATIGRAPHICS
SUMMARY OF CPTU-EC SOUNDINGS
ALLIANT POWER PLANT FLYASH SITE
EAST LANSING, IOWA

SOUNDING NUMBER	DATE PERFORMED	SOUNDING TYPE	SOUNDING DEPTH (feet)	COMMENTS
CP-001	11/21/00	CPTU-EC-EMOD	45.8	
CP-002	11/21/00	CPTU-EC	28.3	
CP-003	11/21/00	CPTU-EC	49.8	
CP-004	11/21/00	CPTU-EC	43.4	
CP-006	11/21/00	CPT-EC	38.1	

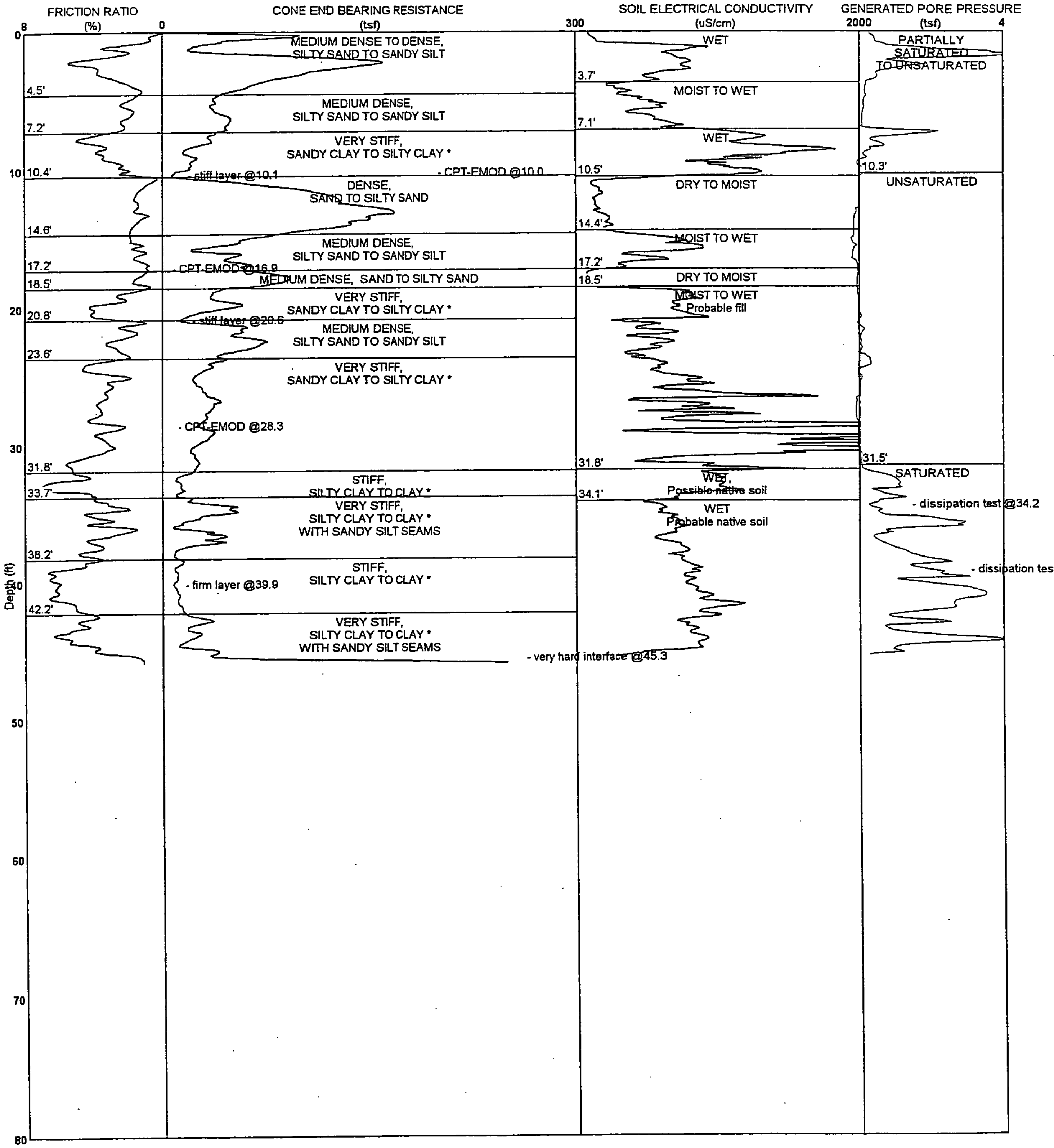
TABLE 2
STRATIGRAPHICS
SUMMARY OF CPTU-EC DISSIPATION TEST DATA
ALLIANT POWER PLANT FLYASH SITE
EAST LANSING, IOWA

SOUNDING NUMBER	DEPTH (ft)	SOIL TYPE AT DISSIPATION DEPTH	t50 (sec)	ESTIMATED SOIL HORIZONTAL HYDRAULIC CONDUCTIVITY kh (cm/sec)	ESTIMATED HORIZONTAL COEFFICIENT OF CONSOLIDATION IN OVERCONSOLIDATED RANGE* Ch(oc) (cm**2/sec)	COMMENT
CP-001	34.2	Sandy silt	4.5	2E-05	6E+00	
	39.0	Clayey silt	5.5	1E-05	5E+00	
CP-02	13.9	Clayey sand	7	2E-04	4E+00	May be partially saturated soil

*1. Estimates of the vertical coefficient of consolidation, in the normally consolidated range, can be estimated using:
Cv(nc)= RR(probe)/CR *(kv/kh)*Ch(oc) from Baligh and Levadoux, 1980 (see Appendix B of this report)

NOTE: All dissipation tests must be performed in lower hydraulic conductivity (less than about 1E-2 cm/s) soil layers and strata, as CPTU-EC generated soil pore water pressures in more conductive soils dissipate faster than the response time of the sensors and data acquisition system. As such, this summary of test results is necessarily biased towards lower conductivity layers at the Site, and must not be considered as representative of the entire soil profile. Inspection of the continuous CPTU-EC sounding logs will indicate the relative frequency of lower and higher hydraulic conductivity soil layers at the Site.

CPTU-EC LOG WITH LITHOLOGIC EVALUATION



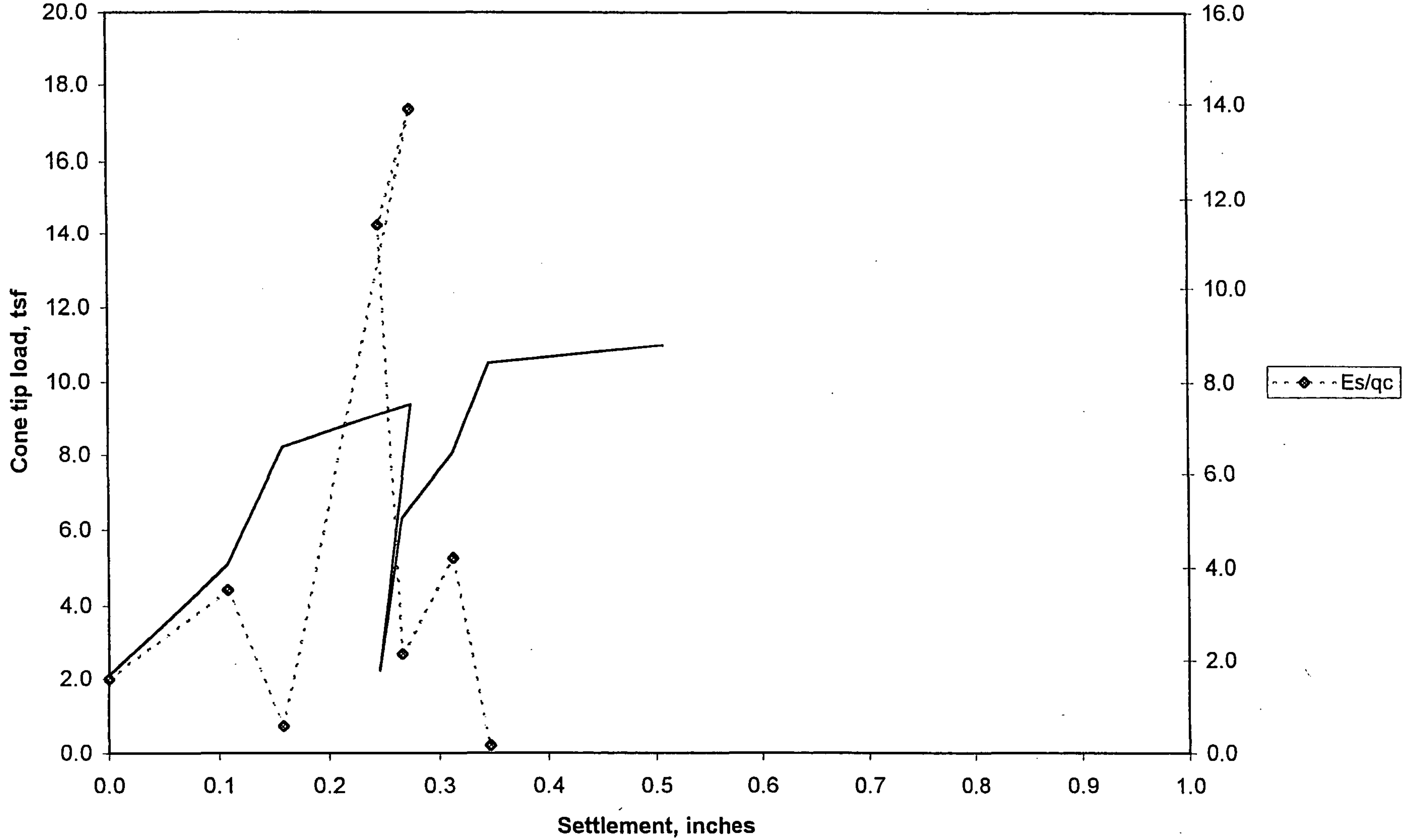
CPT-1

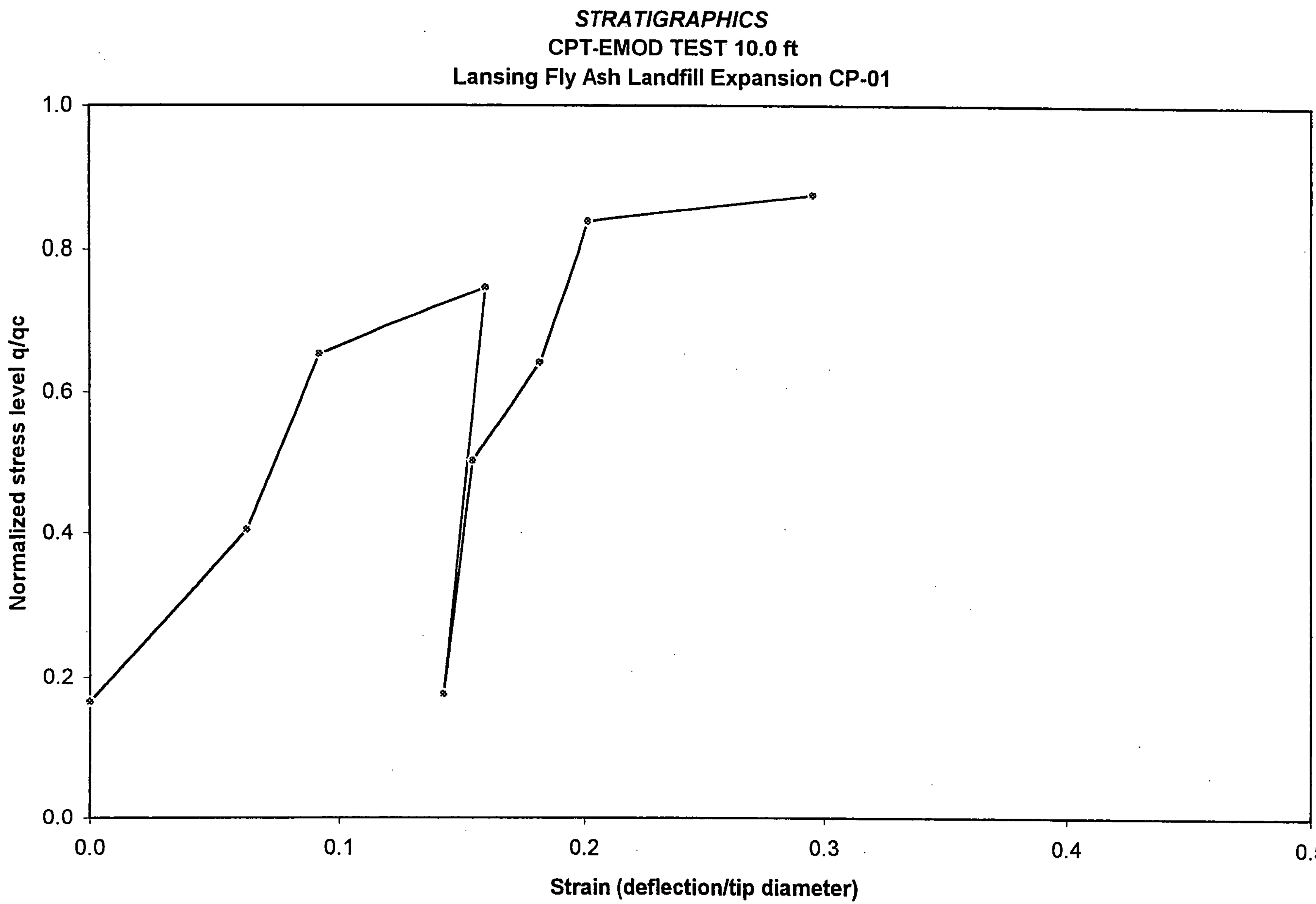
STRATIGRAPHICS
Cone tip 15 sq cm, friction sleeve 200 sq cm
Project: Lansing Fly Ash Landfill Expansion
Project Number: 00-120-160
Sounding Number:
Test Depth: 10.0 ft   **qc:** 13 tsf   **Poisson's Ratio:** 0.35

TIP LOAD (lbs)	TOTAL LOAD (lbs)	TIP STRESS q (tsf)	FRICTION STRESS fs (tsf)	ROD DEFLECTION (in)	ROD ELASTIC DEFORMATION (in)	CORR ROD DEFLECTION (in)	LOAD SETTLEMENT SLOPE (tsf/ft)	ELASTIC MODULUS (tsf)	E/qc	AVG E/qc	NORMALIZED STRESS q/qc	NORMALIZED STRAIN def/dia
67	138	2.094	0.163	-0.0438	0.00008	0.000	333	20	1.6	1.6	0.17	0.0000
163	222	5.094	0.136	0.0644	0.00020	0.108	743	45	3.5	3.5	0.40	0.0628
263	318	8.219	0.127	0.1150	0.00033	0.158	122	7	0.6	0.6	0.65	0.0921
301	370	9.406	0.159	0.2318	0.00038	0.275	2913	175	13.9	13.9	0.75	0.1600
71	66	2.219	-0.012	0.2019	0.00009	0.246	2392	144	11.4	11.4	0.18	0.1428
202	285	6.313	0.191	0.2226	0.00025	0.266	448	27	2.1	2.1	0.50	0.1547
258	302	8.063	0.101	0.2695	0.00032	0.313	882	53	4.2	4.2	0.64	0.1820
337	399	10.531	0.143	0.3032	0.00042	0.347	35	2	0.2	0.2	0.84	0.2015
352	416	11.000	0.147	0.4643	0.00044	0.508					0.87	0.2952

E25/qc	**2.0**	
E50/qc	**2.0**	
Emax/qc	**3.5**	**E40**
Eunload/qc	**13.9**	
Ereload/qc	**11.4**	

STRATIGRAPHICS
CPT-EMOD TEST 10.0 ft
Lansing Fly Ash Landfill Expansion CP-01




STRATIGRAPHICS
CPT-EMOD TEST 10.0 ft
Lansing Fly Ash Landfill Expansion CP-01
Normalized stress level q/qc
1.0
0.8
0.6
0.4
0.2
0.0
0.0
0.1
0.2
0.3
0.4
0.5
Strain (deflection/tip diameter)

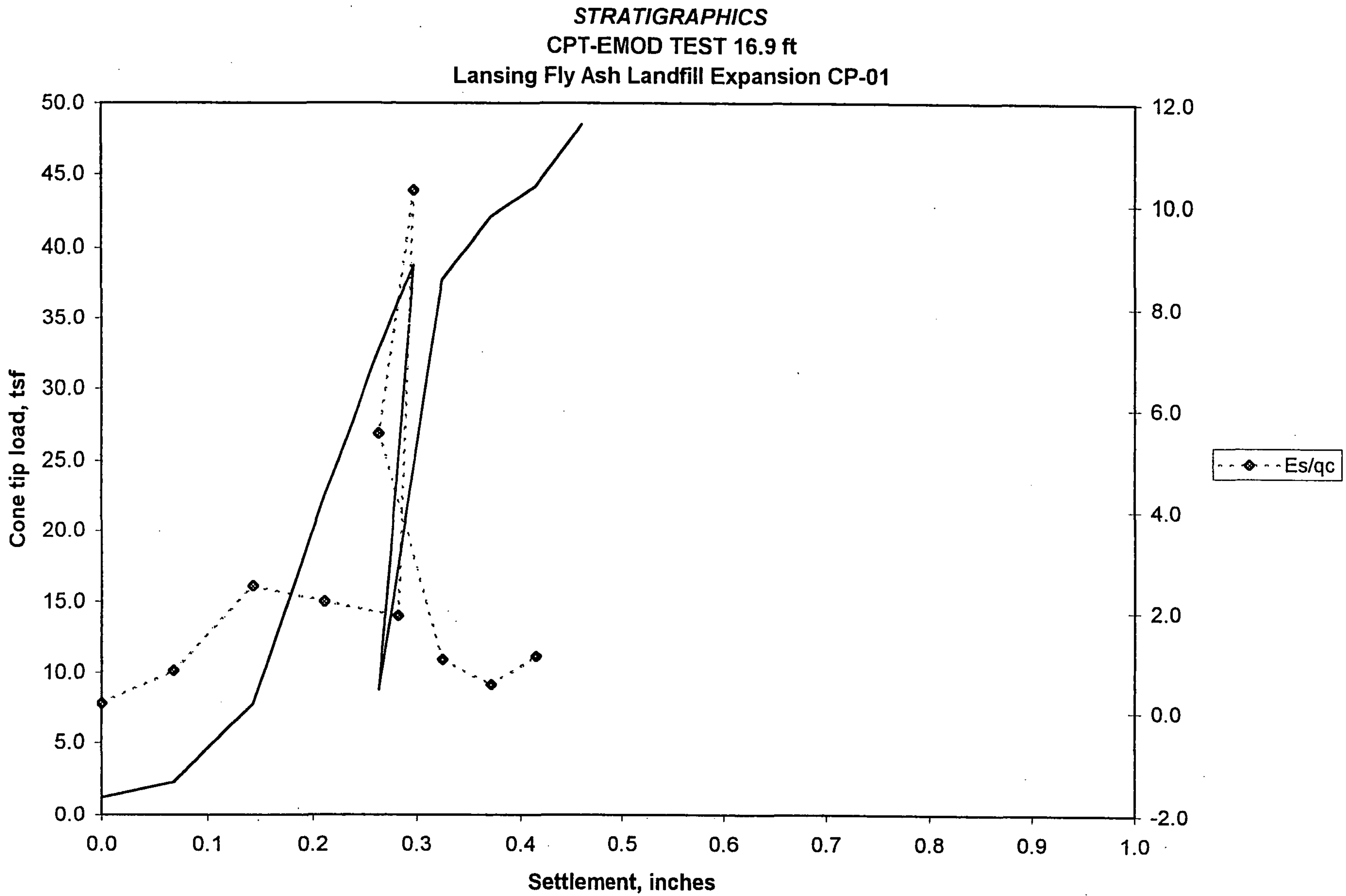
STRATIGRAPHICS

Cone tip 15 sq cm, friction sleeve 200 sq cm

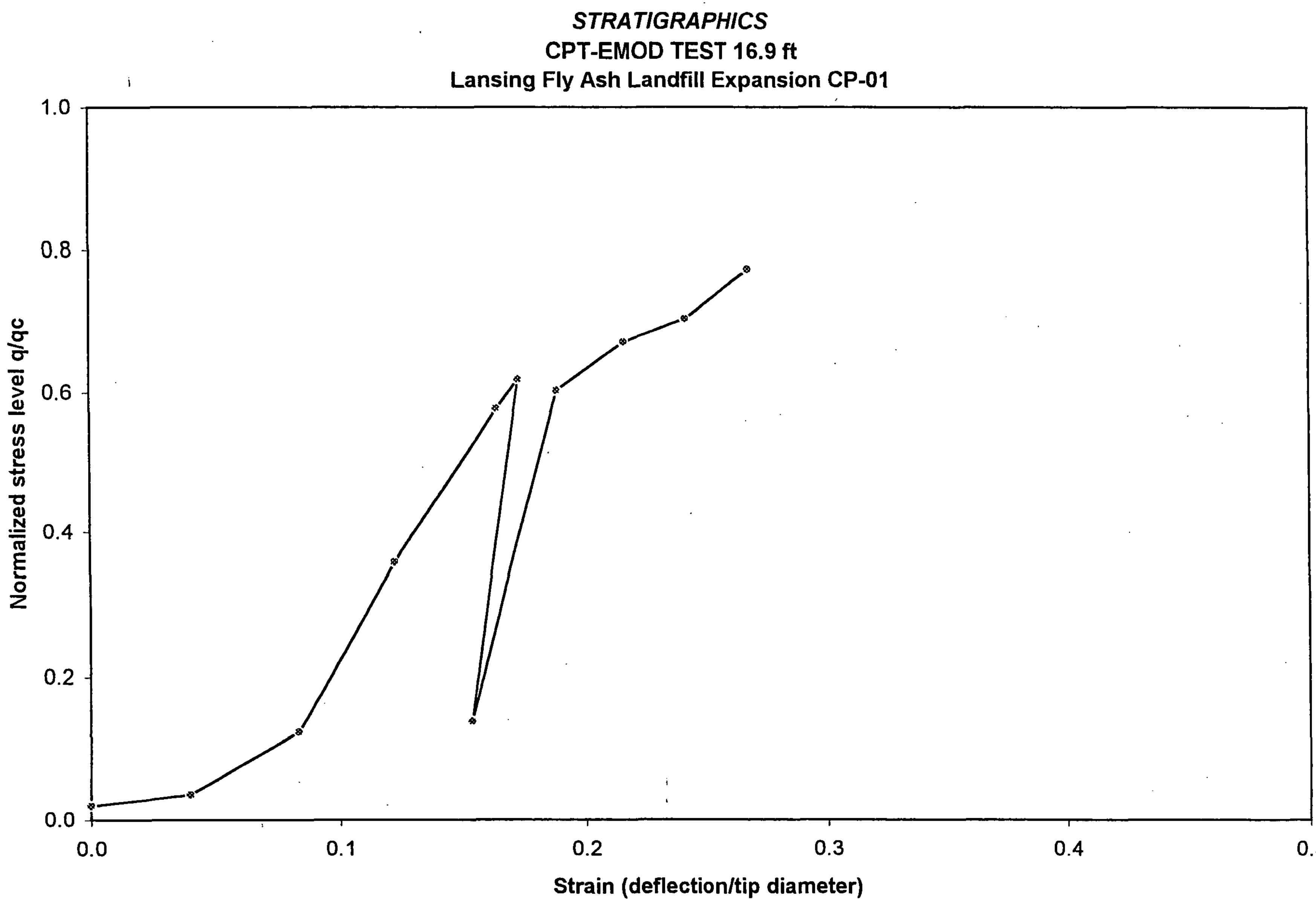
Project: **Lansing Fly Ash Landfill Expansion**
Project Number: **00-120-160**
Sounding Number:
Test Depth: **16.9 ft** **qc:** **63 tsf** **Poisson's Ratio:** **0.35**

TIP LOAD (lbs)	TOTAL LOAD (lbs)	TIP STRESS q (tsf)	FRICTION STRESS fs (tsf)	ROD DEFLECTION (in)	ROD ELASTIC DEFORMATION (in)	CORR ROD DEFLECTION (in)	LOAD SETTLEMENT SLOPE (tsf/ft)	ELASTIC MODULUS (tsf)	E/qc	AVG E/qc	NORMALIZED STRESS q/qc	NORMALIZED STRAIN def/dia
40	192	1.250	0.350	-0.0425	0.00007	0.000	176	11	0.2	0.2	0.02	0.0000
72	242	2.250	0.391	0.0256	0.00012	0.068	871	52	0.8	0.8	0.04	0.0395
247	427	7.719	0.414	0.1012	0.00042	0.143	2620	157	2.5	2.5	0.12	0.0833
718	882	22.438	0.377	0.1694	0.00121	0.211	2303	138	2.2	2.2	0.36	0.1225
1158	1297	36.188	0.320	0.2418	0.00195	0.282	2002	120	1.9	1.9	0.58	0.1642
1240	1371	38.750	0.301	0.2573	0.00209	0.298	10743	645	10.3	10.3	0.62	0.1731
278	268	8.688	-0.023	0.2221	0.00047	0.264	5777	347	5.5	5.5	0.14	0.1536
1209	1361	37.781	0.350	0.2841	0.00203	0.325	1096	66	1.0	1.0	0.60	0.1887
1346	1467	42.063	0.278	0.3312	0.00227	0.371	567	34	0.5	0.5	0.67	0.2160
1412	1541	44.125	0.297	0.3750	0.00238	0.415	1168	70	1.1	1.1	0.70	0.2414
1552	1670	48.500	0.271	0.4202	0.00261	0.460					0.77	0.2675

E25/qc	**2.4**	
E50/qc	**2.0**	
Emax/qc	**2.5**	**E10**
Eunload/qc	**10.3**	
Ereload/qc	**5.5**	



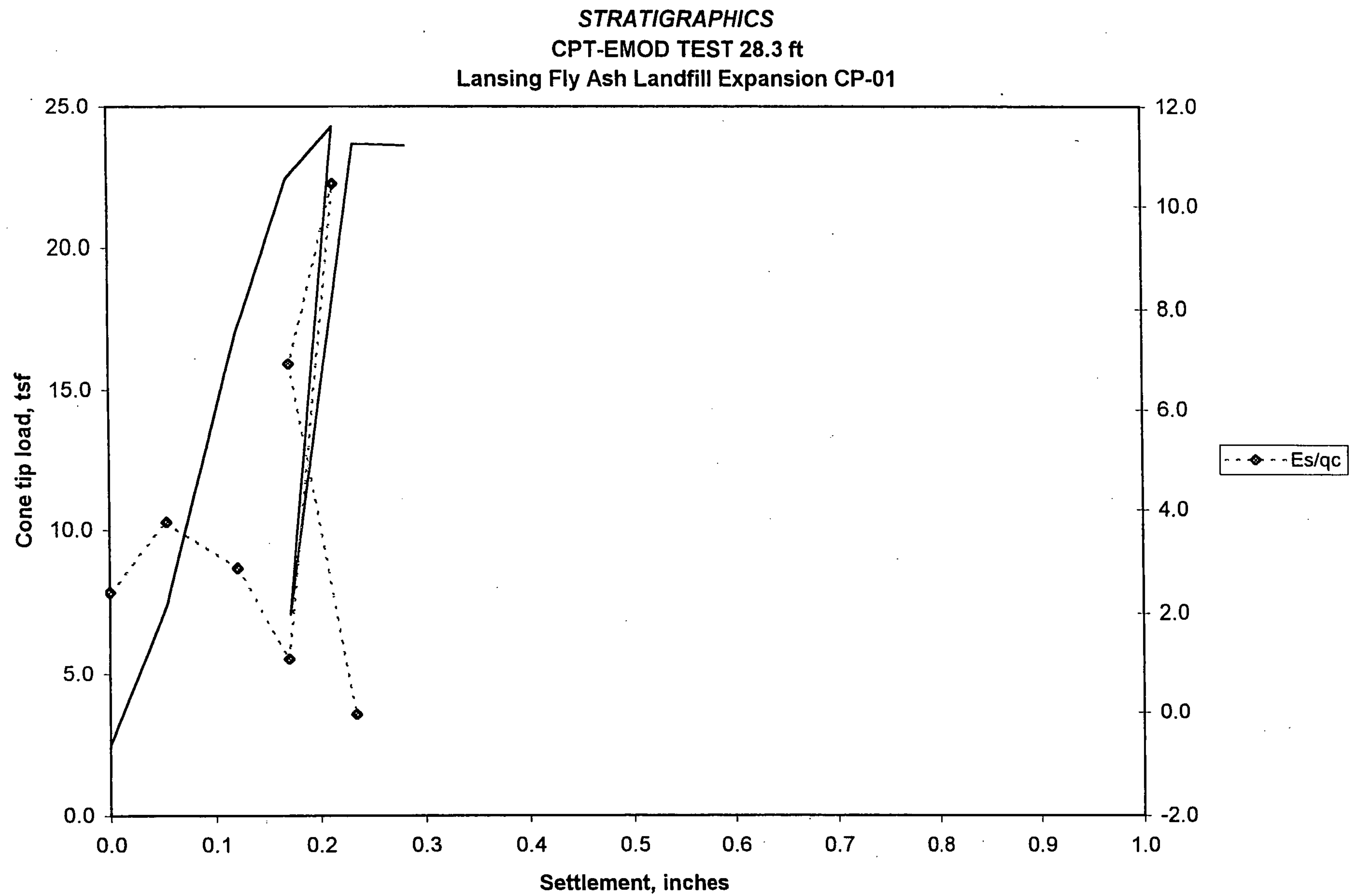
STRATIGRAPHICS
CPT-EMOD TEST 16.9 ft
Lansing Fly Ash Landfill Expansion CP-01
Cone tip load, tsf
Settlement, inches
Es/qc
0.0
5.0
10.0
15.0
20.0
25.0
30.0
35.0
40.0
45.0
50.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
-2.0
2.0
4.0
6.0
8.0
10.0
12.0


STRATIGRAPHICS
CPT-EMOD TEST 16.9 ft
Lansing Fly Ash Landfill Expansion CP-01
Normalized stress level q/qc
1.0
0.8
0.6
0.4
0.2
0.0
0.0
0.1
0.2
0.3
0.4
0.5
Strain (deflection/tip diameter)

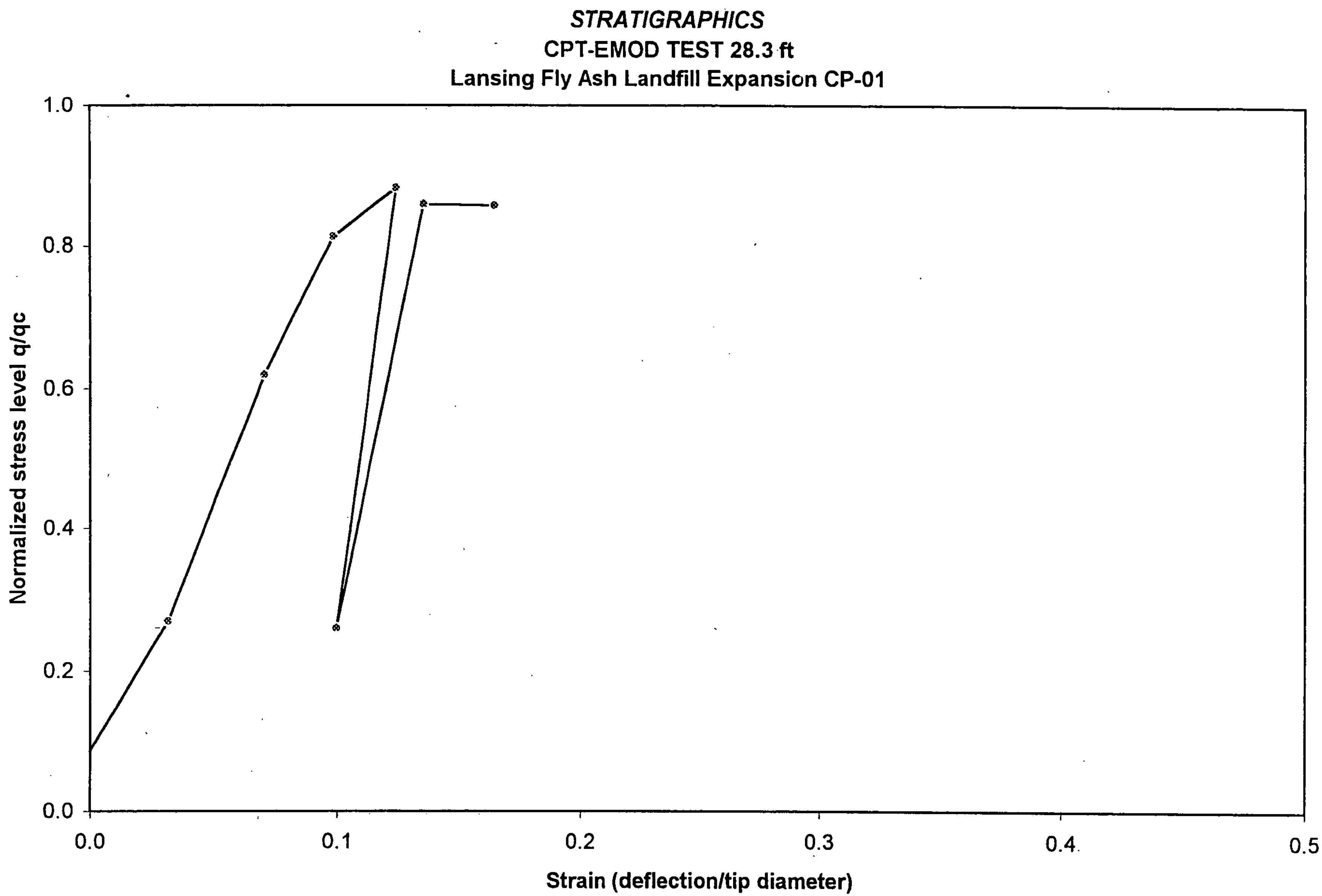
STRATIGRAPHICS
Cone tip 15 sq cm, friction sleeve 200 sq cm
Project: Lansing Fly Ash Landfill Expansion
Project Number: 00-120-160
Sounding Number:
Test Depth: 28.3 ft **qc: 28 tsf** **Poisson's Ratio: 0.40**

TIP LOAD (lbs)	TOTAL LOAD (lbs)	TIP STRESS q (tsf)	FRICTION STRESS fs (tsf)	ROD DEFLECTION (in)	ROD ELASTIC DEFORMATION (in)	CORR ROD DEFLECTION (in)	LOAD SETTLEMENT SLOPE (tsf/ft)	ELASTIC MODULUS (tsf)	E/qc	AVG E/qc	NORMALIZED STRESS q/qc	NORMALIZED STRAIN def/dia
78	271	2.438	0.444	-0.0321	0.00019	0.000	1102	66	2.4	2.4	0.09	-0.0001
237	416	7.406	0.412	0.0224	0.00057	0.054	1725	104	3.8	3.8	0.27	0.0314
546	743	17.063	0.453	0.0903	0.00131	0.121	1317	79	2.9	2.9	0.62	0.0704
717	902	22.406	0.426	0.1394	0.00172	0.170	507	30	1.1	1.1	0.81	0.0987
777	969	24.281	0.442	0.1839	0.00186	0.214	4801	288	10.4	10.4	0.88	0.1245
228	220	7.125	-0.018	0.1397	0.00055	0.171	3172	190	6.9	6.9	0.26	0.0996
757	921	23.656	0.377	0.2035	0.00181	0.234	-15	-1	0.0	0.0	0.86	0.1359
755	940	23.594	0.426	0.2530	0.00181	0.283					0.86	0.1647

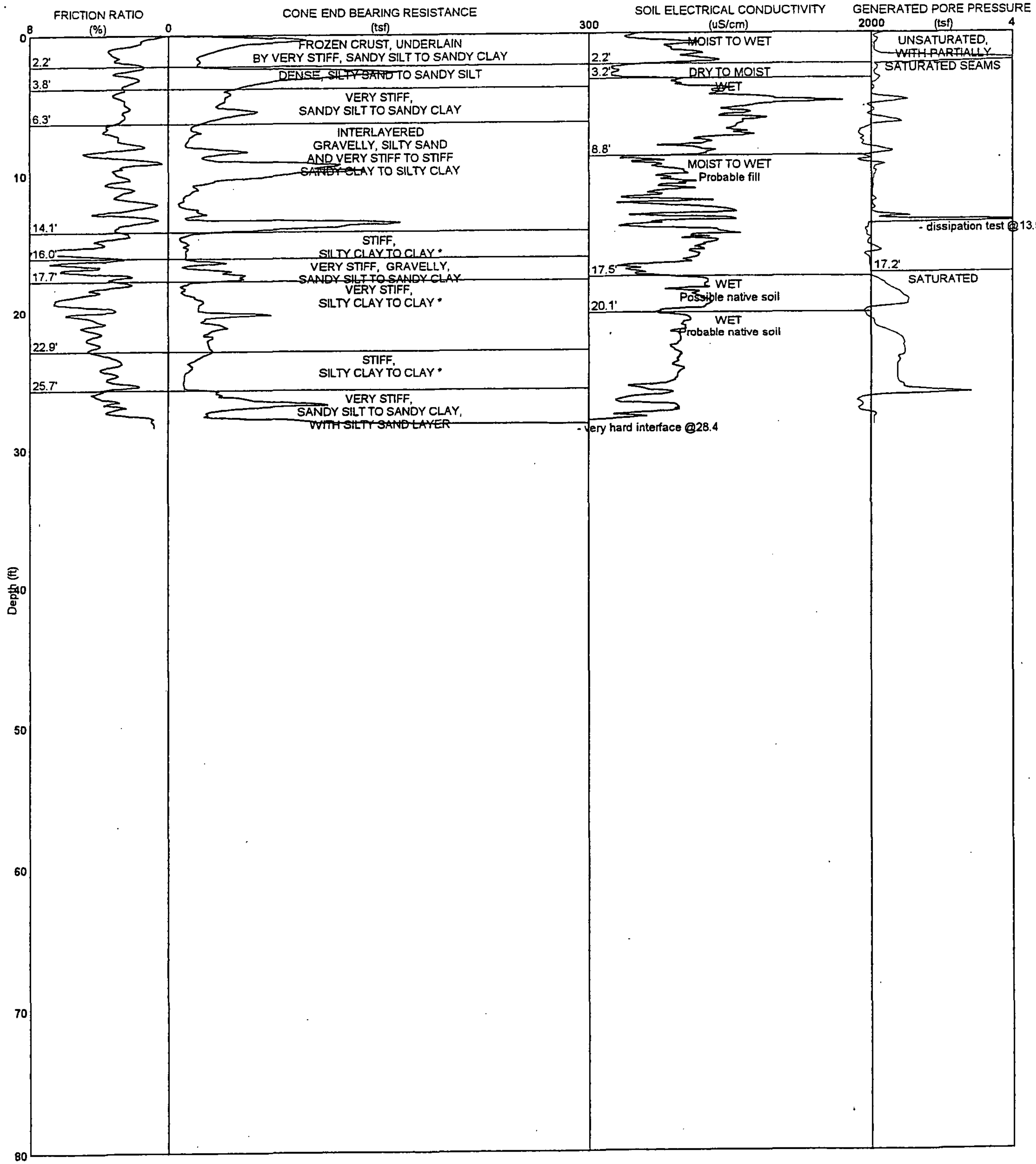
E25/qc	**3.8**	
E50/qc	**3.0**	
Emax/qc	**3.8**	**E25**
Eunload	**10.4**	
Ereload/qc	**6.9**	



STRATIGRAPHICS
CPT-EMOD TEST 28.3 ft
Lansing Fly Ash Landfill Expansion CP-01
Cone tip load, tsf
Settlement, inches
Es/qc
25.0
20.0
15.0
10.0
5.0
0.0
12.0
10.0
8.0
6.0
4.0
2.0
0.0
-2.0
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0


STRATIGRAPHICS
CPT-EMOD TEST 28.3 ft
Lansing Fly Ash Landfill Expansion CP-01
Normalized stress level q/qc
Strain (deflection/tip diameter)
1.0
0.8
0.6
0.4
0.2
0.0
0.0
0.1
0.2
0.3
0.4
0.5

CPTU-EC LOG WITH LITHOLOGIC EVALUATION



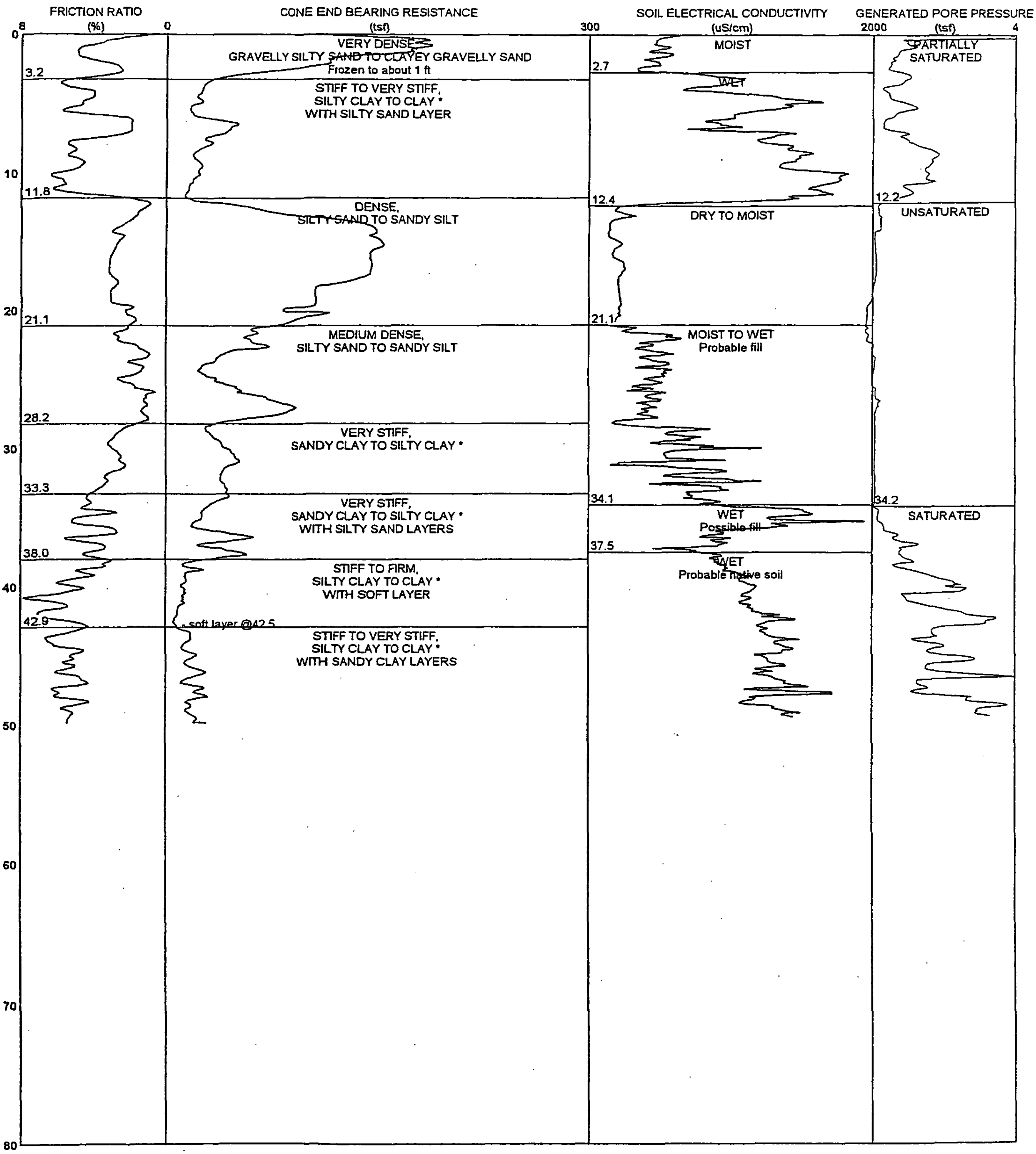
CPT-2

PROJECT NAME:Lansing Fly Ash Landfill Expansion
PROJECT NUMBER:00-120-160

STRATIGRAPHICS

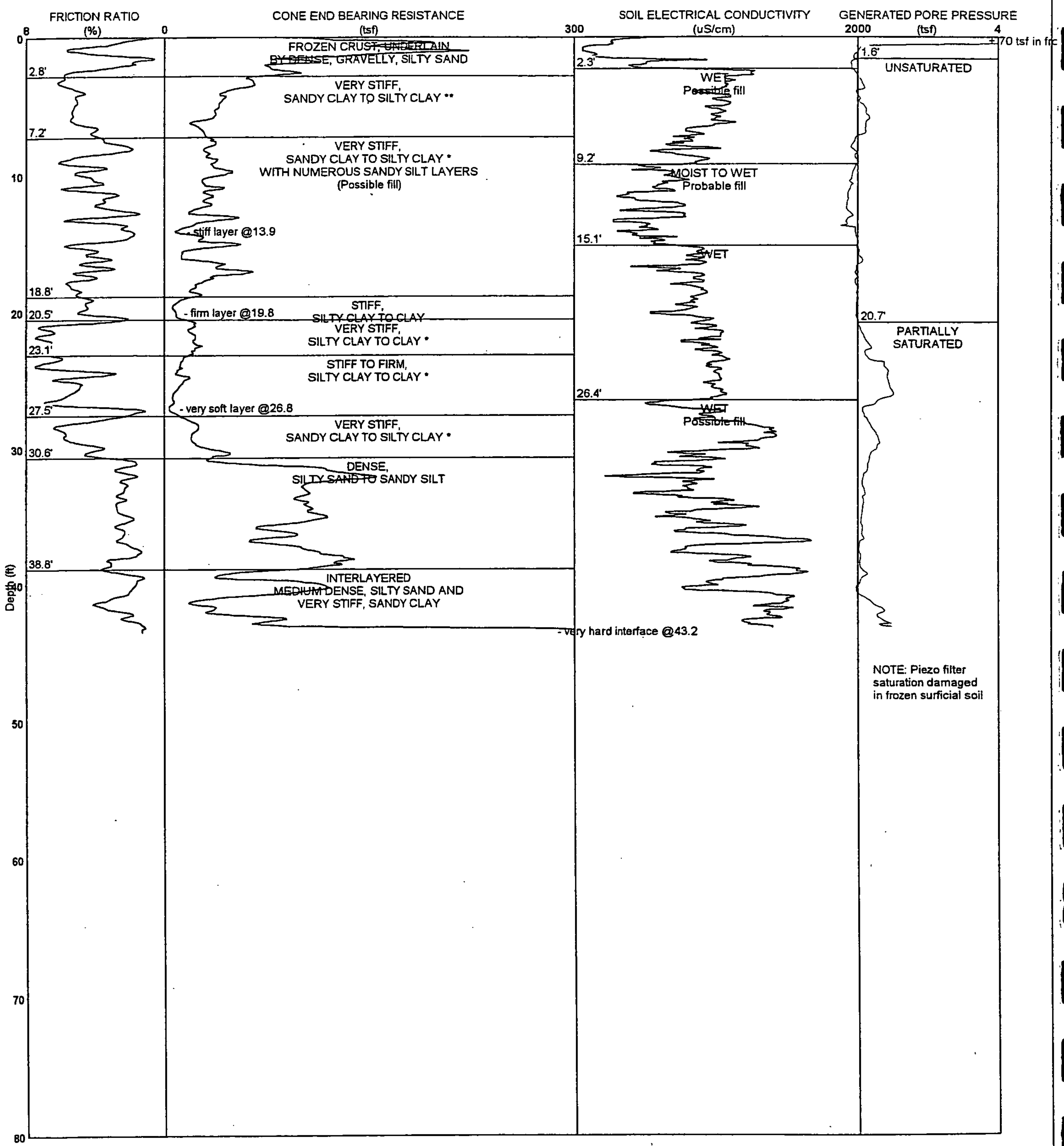
R2DATE:11-21-2000 TIME:11:49:30.88
SOUNDING NUMBER:CP-02

CPTU-EC LOG WITH LITHOLOGIC EVALUATION



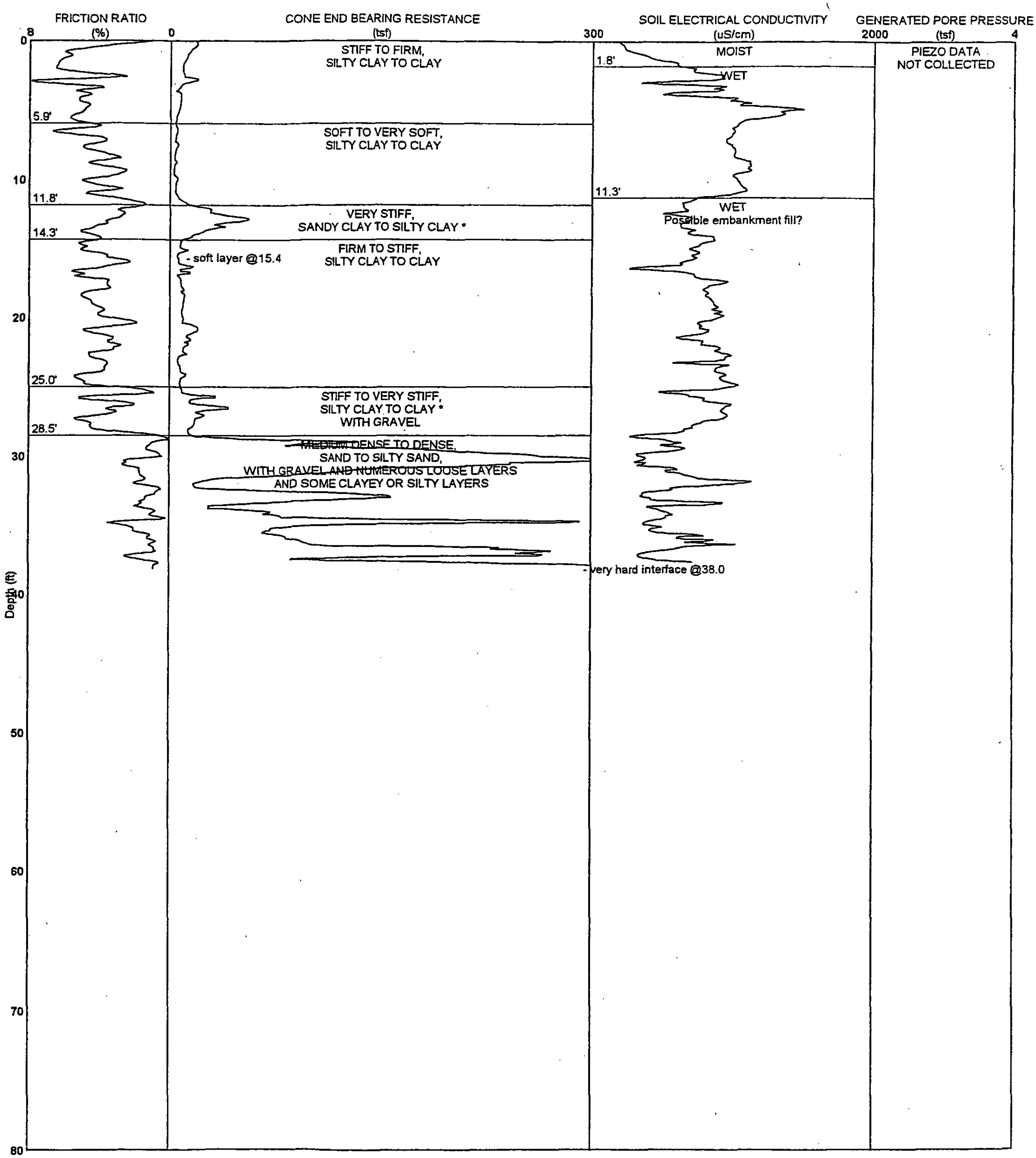
CPT-3

CPTU-EC LOG WITH LITHOLOGIC EVALUATION



CPT-4

CPTU-EC LOG WITH LITHOLOGIC EVALUATION



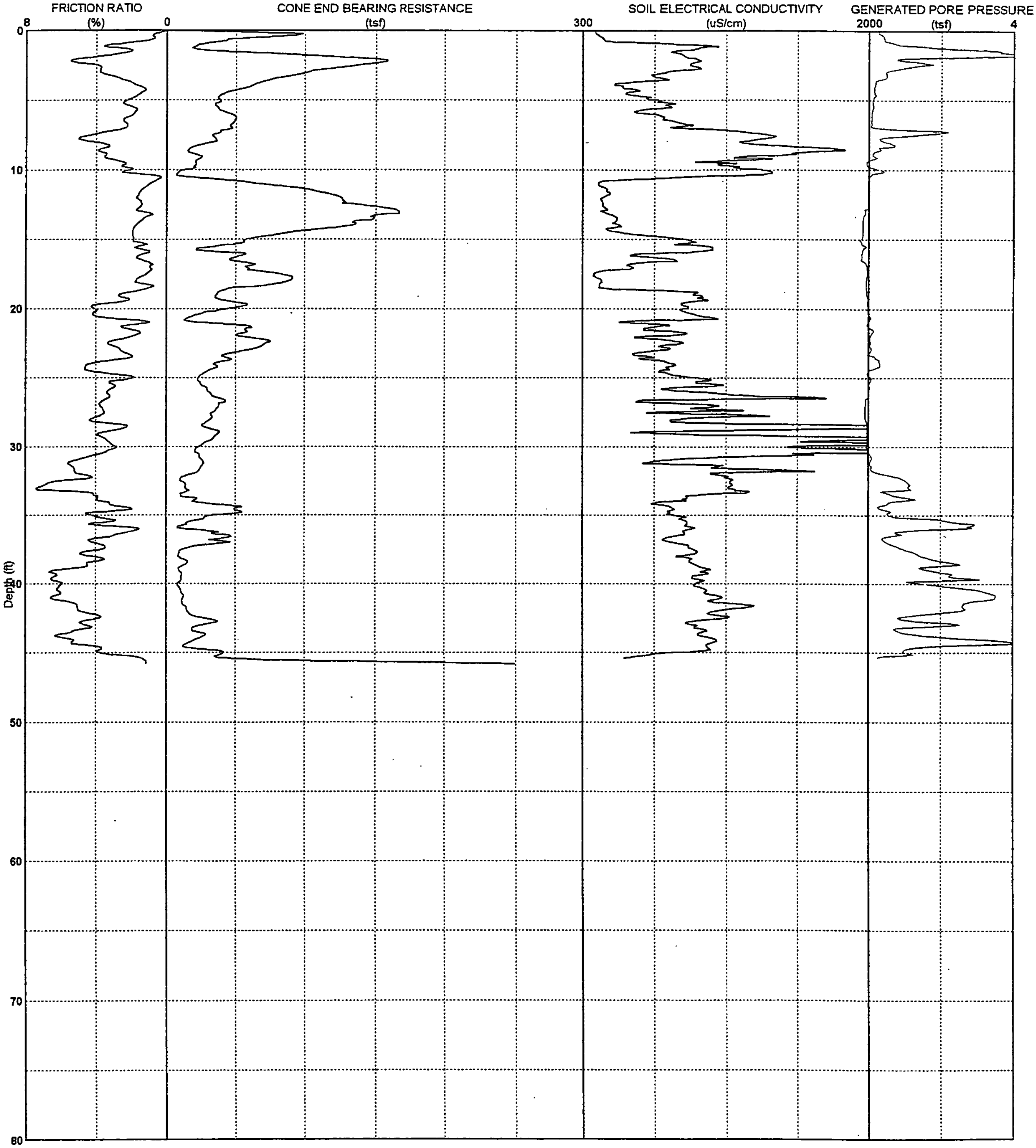
CPT-6

PROJECT NAME:Lansing Fly Ash Landfill Expansion
PROJECT NUMBER:00-120-160

STRATIGRAPHICS

R2DATE:11-21-2000 TIME:14:30:30.51
SOUNDING NUMBER:CP-06

CPTU-EC LOG



PROJECT NAME:Lansing Fly Ash Landfill Expansion
PROJECT NUMBER:00-120-160

STRATIGRAPHICS

R2DATE:11-21-2000 TIME:12:47:07.01
SOUNDING NUMBER:CP-01

STRATIGRAPHICS Evaluated Properties Using Global Database

PROJECT NAME:Lansing Fly Ash Landfill Expansion
PROJECT NUMBER:00-120-160
R2DATE:11-21-2000 TIME:12:47:07.01
SOUNDING NUMBER:CP-01

Depth (ft)	Cone (tsf)	Norm Cone (tsf)	Friction (tsf)	Averaged Friction Ratio (%)	Soil Conductivity (uS/cm)	Evaluated Soil Type	Drained Friction Angle (deg)	Relative Density (%)	Nc	Undrained Shear Strength (ksf)	Undrained Large Strain Shear Strength (ksf)	SPT (N)	NORM SPT (N1')
1.0	24.0	38.7	1.18	3.9	1134	Stiff, Sandy clay to silty clay *			25	1.92	2.36	12 - 19	20 - 30
1.5	41.9	63.7	1.79	2.0	623	Dense, Silty sand to sandy silt	36-37	60-80				13 - 20	20 - 30
2.0	140.6	205.3	7.18	4.8	783	Hard, Gravelly Sandy clay to hardpan **			33	8.51	14.36	+ 68	+ 100
2.5	138.6	195.6	5.74	3.9	747	Hard, Gravelly clayey sand to gravelly sandy silt			33	8.39	11.48	+ 71	+ 100
3.0	102.7	141.0	4.43	3.7	579	Hard, Gravelly clayey sand to gravelly sandy silt			33	6.21	8.86	+ 73	+ 100
3.5	76.9	102.9	2.30	2.6	586	Very dense, Silty sand to sandy silt	36-37	80-100				30 - 45	40 - 60
4.0	61.8	80.9	1.06	1.6	282	Medium dense, Silty sand to sandy silt	37-40	40-60				15 - 23	20 - 30
4.5	43.0	55.2	0.75	1.5	310	Medium dense, Silty sand to sandy silt	36-37	40-60				08 - 12	10 - 15
5.0	38.7	48.8	0.86	2.2	506	Medium dense, Silty sand to sandy silt	27-31	40-60				12 - 16	15 - 20
5.5	37.1	46.1	0.78	2.0	608	Medium dense, Silty sand to sandy silt	27-31	40-60				08 - 12	10 - 15
6.0	46.3	56.5	0.91	1.9	465	Medium dense, Silty sand to sandy silt	36-37	40-60				12 - 16	15 - 20
6.5	49.3	59.4	1.22	2.5	615	Dense, Silty sand to sandy silt	27-31	60-80				17 - 25	20 - 30
7.0	47.2	56.1	1.12	2.4	666	Dense, Silty sand to sandy silt	27-31	60-80				17 - 25	20 - 30
7.5	33.5	39.3	1.69	4.3	1302	Very stiff, Silty clay to clay *			25	2.64	3.38	17 - 26	20 - 30
8.0	34.1	39.5	1.39	4.1	1096	Very stiff, Sandy clay to silty clay *			25	2.69	2.78	17 - 26	20 - 30
8.5	16.2	18.6	0.87	3.7	1777	Very stiff, Silty clay to clay *			15	2.10	1.74	05 - 09	06 - 10
9.0	24.4	27.7	0.81	3.5	1267	Very stiff, Sandy clay to silty clay *			20	2.39	1.61	09 - 13	10 - 15
9.5	21.0	23.6	0.53	2.4	999	Very stiff, Sandy silt to sandy clay			20	2.05	1.06	05 - 09	06 - 10
10.0	12.6	14.0	0.35	2.0	1258	Stiff, Sandy silt to clayey silt			15	1.60	0.70	02 - 04	02 - 04
10.5	15.0	16.5	0.12	0.4	730	Very loose, Sensitive fine grained soil	31-36	0-20				00 - 02	00 - 02
11.0	67.5	73.6	0.88	1.0	109	Medium dense, Sand to silty sand	37-40	40-60				14 - 18	15 - 20
11.5	107.6	116.2	1.75	1.5	169	Dense, Sand to silty sand	40-42	60-80				28 - 37	30 - 40
12.0	126.5	135.5	2.26	1.7	139	Dense, Silty sand to sandy silt	40-42	60-80				37 - 56	40 - 60
12.5	137.7	146.3	2.18	1.4	142	Dense, Sand to silty sand	40-42	60-80				38 - 56	40 - 60
13.0	166.3	175.2	2.41	1.4	141	Dense, Sand to silty sand	40-42	60-80				38 - 57	40 - 60
13.5	149.1	156.0	2.09	1.3	240	Dense, Sand to silty sand	40-42	60-80				38 - 57	40 - 60
14.0	133.0	138.0	2.46	1.8	264	Dense, Silty sand to sandy silt	40-42	60-80				39 - 58	40 - 60
14.5	90.3	93.0	2.17	2.0	253	Dense, Silty sand to sandy silt	37-40	60-80				29 - 39	30 - 40
15.0	58.9	60.2	1.38	1.9	700	Medium dense, Silty sand to sandy silt	36-37	40-60				15 - 20	15 - 20
15.5	35.7	36.2	0.84	1.6	859	Medium dense, Silty sand to sandy silt	27-31	40-60				06 - 10	06 - 10
16.0	56.6	57.1	0.60	1.2	535	Medium dense, Silty sand to sandy silt	37-40	40-60				10 - 15	10 - 15
16.5	46.5	46.6	0.73	1.4	642	Medium dense, Silty sand to sandy silt	36-37	40-60				10 - 15	10 - 15
17.0	58.1	57.8	0.68	1.0	356	Medium dense, Sand to silty sand	37-40	40-60				10 - 15	10 - 15
17.5	83.0	82.1	1.05	1.2	77	Medium dense, Sand to silty sand	37-40	40-60				20 - 30	20 - 30
18.0	88.7	87.2	1.56	1.8	133	Dense, Silty sand to sandy silt	37-40	60-80				20 - 31	20 - 30
18.5	47.2	46.1	0.75	1.0	136	Loose, Silty sand to sandy silt	36-37	20-40				06 - 10	06 - 10
19.0	36.0	35.0	1.11	2.7	792	Very stiff, Sandy silt to sandy clay			20	3.49	2.22	10 - 15	10 - 15
19.5	48.5	46.9	1.41	2.7	726	Very stiff, Sandy silt to sandy clay			25	3.79	2.83	16 - 21	15 - 20
20.0	44.6	42.8	2.11	4.0	688	Very stiff, Sandy clay to silty clay *			25	3.47	4.22	21 - 31	20 - 30
20.5	19.6	18.7	1.22	4.0	806	Very stiff, Silty clay to clay *			15	2.45	2.44	06 - 10	06 - 10
21.0	25.9	24.6	0.41	1.0	309	Loose, Silty sand to sandy silt	31-36	20-40				04 - 06	04 - 06
21.5	57.3	54.1	1.13	2.0	434	Medium dense, Silty sand to sandy silt	36-37	40-60				16 - 21	15 - 20
22.0	53.2	50.0	1.35	2.0	427	Medium dense, Silty sand to sandy silt	27-31	40-60				16 - 21	15 - 20
22.5	72.7	67.9	2.31	3.3	682	Hard, Sandy silt to sandy clay			25	5.70	4.61	32 - 43	30 - 40
23.0	59.1	54.9	1.73	2.6	552	Hard, Sandy silt to sandy clay			25	4.62	3.46	22 - 32	20 - 30
23.5	42.3	39.1	0.89	2.0	486	Medium dense, Silty sand to sandy silt	27-31	40-60				11 - 16	10 - 15
24.0	33.9	31.2	1.81	4.4	635	Very stiff, Silty clay to clay *			25	2.59	3.62	16 - 22	15 - 20
24.5	30.2	27.7	1.52	4.3	541	Very stiff, Silty clay to clay *			20	2.87	3.04	16 - 22	15 - 20
25.0	23.0	20.9	0.45	1.9	783	Very stiff, Sandy silt to clayey silt			15	2.86	0.90	04 - 07	04 - 06

* Indicates lightly overconsolidated soil
** Indicates heavily overconsolidated or cemented soil

Mixed soils containing both granular and fine grained particles (e.g. clayey sands) may undergo partial drained failure during CPT.
Both undrained and drained parameters can be estimated for these soils.

Structure rate of loading should be considered in choosing which strength parameters to use for design.
Drained and undrained parameters must not be combined as such combination will result in significant overprediction of in situ shear strength.

STRATIGRAPHICS Evaluated Properties Using Global Database

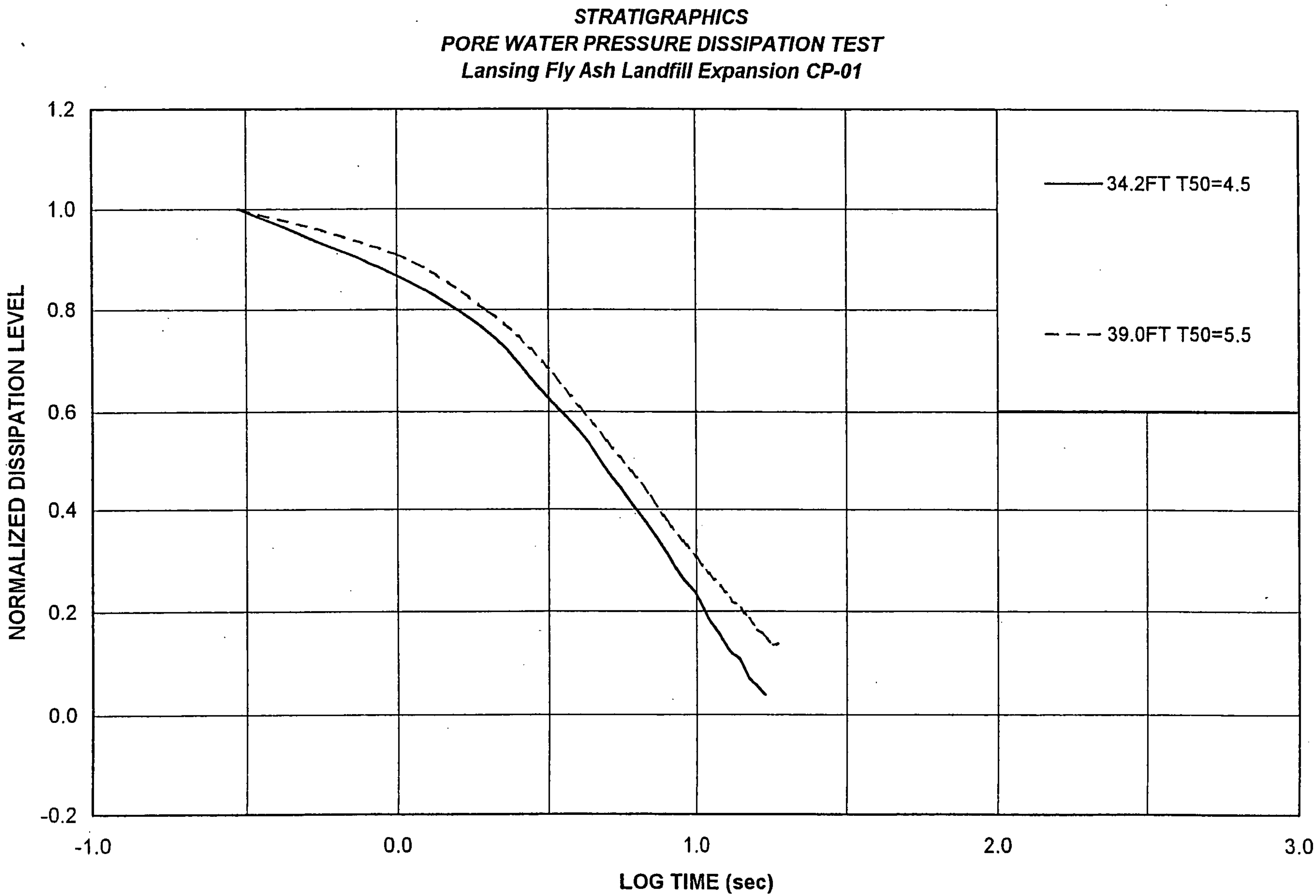
PROJECT NAME:Lansing Fly Ash Landfill Expansion
PROJECT NUMBER:00-120-160
R2DATE:11-21-2000 TIME:12:47:07.01
SOUNDING NUMBER:CP-01

Depth (ft)	Cone (tsf)	Norm Cone (tsf)	Friction (tsf)	Averaged Friction Ratio (%)	Soil Conductivity (uS/cm)	Evaluated Soil Type	Drained Friction Angle (deg)	Relative Density (%)	Nc	Undrained Shear Strength (ksf)	Undrained Large Strain Shear Strength (ksf)	SPT (N)	NORM SPT (N1')
25.5	25.1	22.8	0.82	3.0	955	Very stiff, Sandy clay to silty clay *			20	2.36	1.64	07 - 11	06 - 10
26.0	30.5	27.6	1.14	3.4	753	Very stiff, Sandy clay to silty clay *			20	2.89	2.28	11 - 17	10 - 15
26.5	39.0	35.1	1.33	3.3	1486	Very stiff, Sandy clay to silty clay *			25	2.99	2.66	17 - 22	15 - 20
27.0	38.2	34.2	1.52	3.8	928	Very stiff, Sandy clay to silty clay *			25	2.93	3.05	17 - 22	15 - 20
27.5	33.8	30.1	1.26	3.5	518	Very stiff, Sandy clay to silty clay *			20	3.21	2.53	11 - 17	10 - 15
28.0	32.3	28.7	1.42	4.3	678	Very stiff, Silty clay to clay *			20	3.06	2.84	17 - 23	15 - 20
28.5	26.0	23.0	0.73	2.2	3170	Very stiff, Sandy silt to sandy clay			20	2.43	1.47	05 - 07	04 - 06
29.0	38.2	33.6	1.36	3.7	437	Very stiff, Sandy clay to silty clay *			25	2.92	2.71	17 - 23	15 - 20
29.5	32.9	28.8	1.19	3.4	1846	Very stiff, Sandy clay to silty clay *			20	3.11	2.38	11 - 17	10 - 15
30.0	23.4	20.4	0.74	2.8	1438	Very stiff, Sandy clay to silty clay *			15	2.88	1.48	07 - 11	06 - 10
30.5	23.1	20.1	0.89	3.7	1509	Very stiff, Silty clay to clay *			20	2.13	1.78	07 - 12	06 - 10
31.0	26.2	22.7	1.38	5.2	563	Very stiff, Silty clay to clay *			20	2.44	2.76	17 - 23	15 - 20
31.5	24.9	21.5	1.38	5.3	906	Very stiff, Silty clay to clay *			20	2.31	2.77	12 - 17	10 - 15
32.0	18.1	15.6	1.10	4.8	893	Very stiff, Silty clay to clay *			15	2.16	2.20	07 - 12	06 - 10
32.5	10.2	8.8	0.83	6.2	1038	Stiff, Silty clay to clay *			12	1.38	1.67	05 - 07	04 - 06
33.0	13.6	11.7	1.01	7.4	1018	Stiff, Silty clay to clay *			14	1.66	2.03	07 - 12	06 - 10
33.5	10.2	8.8	0.65	4.1	822	Stiff, Silty clay to clay			10	1.64	1.29	02 - 05	02 - 04
34.0	18.8	16.0	1.29	3.5	608	Very stiff, Silty clay to clay *			15	2.23	2.58	05 - 07	04 - 06
34.5	51.1	43.6	0.97	2.0	623	Medium dense, Silty sand to sandy silt	27-31	40-60				12 - 18	10 - 15
35.0	36.2	30.9	2.07	4.2	619	Very stiff, Silty clay to clay *			25	2.73	4.14	18 - 23	15 - 20
35.5	15.7	13.4	0.81	3.4	720	Stiff, Silty clay to clay *			15	1.81	1.62	05 - 07	04 - 06
36.0	13.1	11.1	0.39	1.5	756	Stiff, Sandy silt to clayey silt			15	1.46	0.78	00 - 02	00 - 02
36.5	46.3	39.2	1.22	3.0	654	Very stiff, Sandy silt to sandy clay			25	3.53	2.44	18 - 24	15 - 20
37.0	44.5	37.6	1.73	4.1	631	Very stiff, Sandy clay to silty clay *			25	3.38	3.46	24 - 35	20 - 30
37.5	11.7	9.8	0.88	3.7	773	Stiff, Silty clay to clay			15	1.25	1.75	02 - 05	02 - 04
38.0	8.5	7.1	0.52	4.2	651	Stiff, Silty clay to clay			10	1.24	1.04	00 - 02	00 - 02
38.5	14.9	12.5	0.59	4.5	759	Stiff, Silty clay to clay *			15	1.68	1.19	05 - 07	04 - 06
39.0	8.8	7.4	0.70	6.3	870	Stiff, Silty clay to clay *			12	1.08	1.40	05 - 07	04 - 06
39.5	10.5	8.8	0.68	6.5	759	Stiff, Silty clay to clay *			12	1.35	1.35	05 - 07	04 - 06
40.0	7.9	6.7	0.58	6.0	782	Firm, Clay			12	0.93	1.16	05 - 07	04 - 06
40.5	10.5	8.8	0.72	6.2	859	Stiff, Silty clay to clay *			12	1.35	1.44	05 - 07	04 - 06
41.0	12.1	10.1	0.77	6.6	970	Stiff, Silty clay to clay *			14	1.38	1.54	07 - 12	06 - 10
41.5	11.9	9.9	0.71	5.1	1112	Stiff, Silty clay to clay *			15	1.25	1.41	05 - 07	04 - 06
42.0	15.4	12.8	0.93	4.9	926	Stiff, Silty clay to clay *			15	1.72	1.86	07 - 12	06 - 10
42.5	27.2	22.6	1.21	3.8	1004	Very stiff, Silty clay to clay *			20	2.47	2.42	12 - 18	10 - 15
43.0	25.6	21.2	1.46	4.6	772	Very stiff, Silty clay to clay *			20	2.30	2.93	12 - 18	10 - 15
43.5	22.4	18.5	1.37	5.4	834	Stiff, Silty clay to clay *			20	1.98	2.74	12 - 18	10 - 15
44.0	22.6	18.6	1.40	5.5	908	Stiff, Silty clay to clay *			20	1.99	2.80	12 - 18	10 - 15
44.5	12.2	10.1	1.05	4.3	855	Stiff, Silty clay to clay *			15	1.28	2.10	05 - 07	04 - 06
45.0	41.4	34.1	1.62	3.8	666	Very stiff, Sandy clay to silty clay *			25	3.10	3.24	18 - 24	15 - 20
45.5	64.4	52.9	1.86	1.3	N/A	Medium dense, Silty sand to sandy silt	36-37	40-60				12 - 18	10 - 15

* Indicates lightly overconsolidated soil
** Indicates heavily overconsolidated or cemented soil

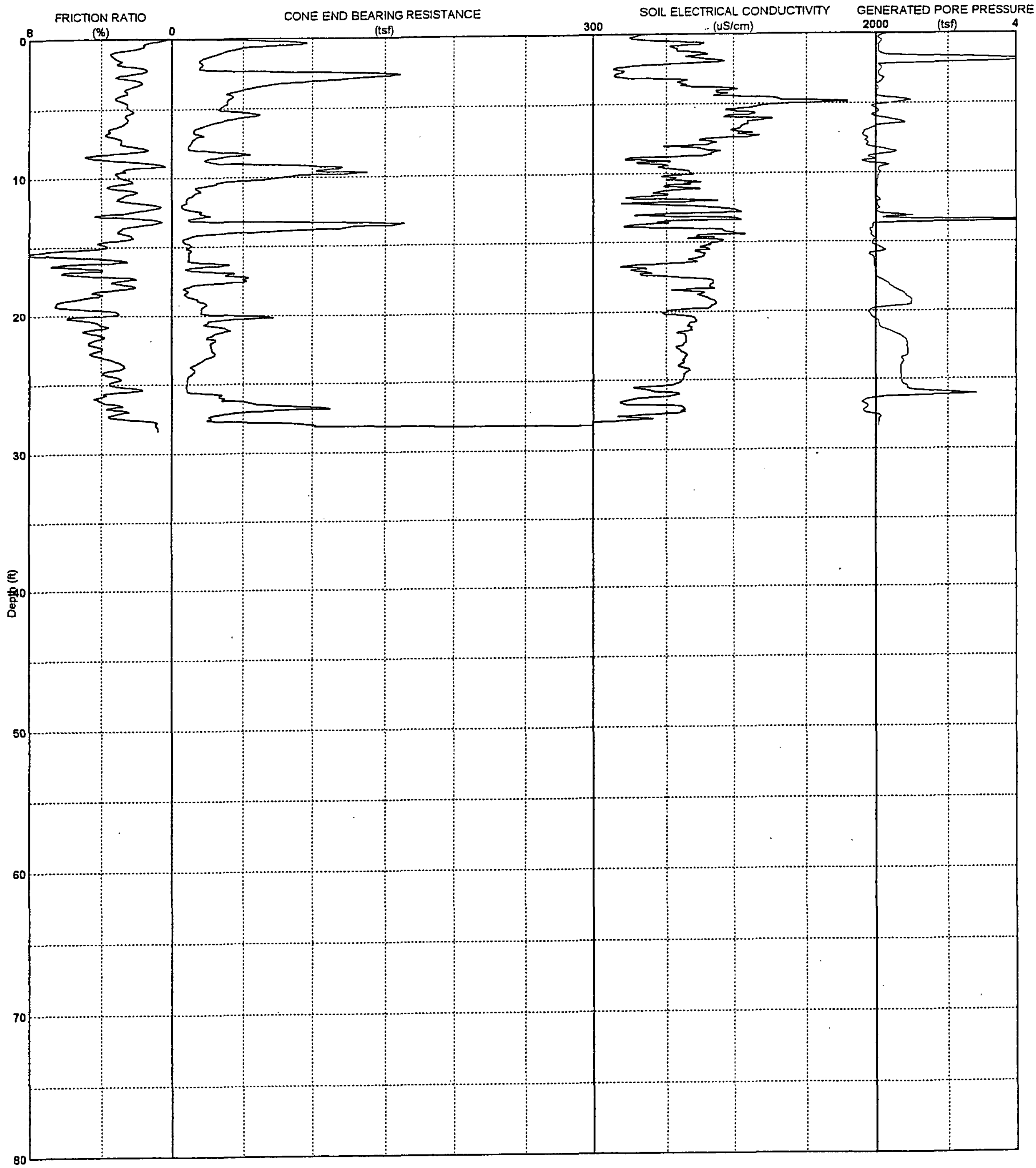
Mixed soils containing both granular and fine grained particles (e.g. clayey sands) may undergo partial drained failure during CPT.
Both undrained and drained parameters can be estimated for these soils.

Structure rate of loading should be considered in choosing which strength parameters to use for design.
Drained and undrained parameters must not be combined as such combination will result in significant overprediction of in situ shear strength.



STRATIGRAPHICS
PORE WATER PRESSURE DISSIPATION TEST
Lansing Fly Ash Landfill Expansion CP-01
34.2FT T50=4.5
39.0FT T50=5.5
NORMALIZED DISSIPATION LEVEL
1.2
1.0
0.8
0.6
0.4
0.2
0.0
-0.2
-1.0
0.0
1.0
2.0
3.0
LOG TIME (sec)

CPTU-EC LOG



PROJECT NAME:Lansing Fly Ash Landfill Expansion
PROJECT NUMBER:00-120-160

STRATIGRAPHICS

R2DATE:11-21-2000 TIME:11:49:30.88
SOUNDING NUMBER:CP-02

STRATIGRAPHICS Evaluated Properties Using Global Database

PROJECT NAME:Lansing Fly Ash Landfill Expansion
PROJECT NUMBER:00-120-160
R2DATE:11-21-2000 TIME:11:49:30.88
SOUNDING NUMBER:CP-02

Depth (ft)	Cone (tsf)	Norm Cone (tsf)	Friction (tsf)	Averaged Friction Ratio (%)	Soil Conductivity (uS/cm)	Evaluated Soil Type	Drained Friction Angle (deg)	Relative Density (%)	Nc	Undrained Shear Strength (ksf)	Undrained Large Strain Shear Strength (ksf)	SPT (N)	NORM SPT (N1')
1.0	34.7	55.9	1.66	3.6	567	Very stiff, Sandy clay to silty clay *			25	2.77	3.32	12 - 19	20 - 30
1.5	20.4	31.1	0.82	3.0	740	Very stiff, Sandy clay to silty clay *			20	2.03	1.64	07 - 10	10 - 15
2.0	20.2	29.4	0.95	2.1	893	Very stiff, Sandy silt to sandy clay			20	2.00	1.90	04 - 07	06 - 10
2.5	130.2	183.8	2.94	2.1	150	Very dense, Gravelly silty sand to clayey gravelly s	40-42	80-100				42 - 70	60 - 99
3.0	112.7	154.6	2.53	1.9	155	Dense, Silty sand to sandy silt	40-42	60-80				29 - 44	40 - 60
3.5	59.8	80.0	1.90	2.4	611	Dense, Silty sand to sandy silt	36-37	60-80				22 - 30	30 - 40
4.0	38.6	50.6	1.28	2.7	973	Very stiff, Sandy silt to sandy clay			25	3.07	2.56	11 - 15	15 - 20
4.5	39.2	50.3	1.12	2.7	1103	Very stiff, Sandy silt to sandy clay			25	3.11	2.24	16 - 23	20 - 30
5.0	33.2	41.9	1.11	2.4	1438	Very stiff, Sandy silt to sandy clay			25	2.63	2.21	08 - 12	10 - 15
5.5	60.9	75.6	1.31	2.6	1000	Dense, Silty sand to sandy silt	27-31	60-80				24 - 32	30 - 40
6.0	31.4	38.4	1.18	2.6	1178	Very stiff, Sandy silt to sandy clay			25	2.48	2.36	08 - 12	10 - 15
6.5	16.0	19.3	0.84	3.5	1071	Very stiff, Sandy clay to silty clay *			15	2.08	1.67	05 - 08	06 - 10
7.0	20.1	23.9	0.59	3.4	1087	Stiff, Sandy clay to silty clay *			20	1.97	1.18	05 - 08	06 - 10
7.5	13.3	15.6	0.45	2.9	794	Stiff, Sandy clay to silty clay *			15	1.72	0.89	03 - 05	04 - 06
8.0	17.1	19.8	0.50	1.4	660	Loose, Silty sand to sandy silt	27-31	20-40				03 - 05	04 - 06
8.5	36.8	42.2	1.84	4.7	859	Very stiff, Silty clay to clay *			25	2.91	3.68	17 - 26	20 - 30
9.0	45.2	51.2	0.52	0.7	300	Loose, Sand to silty sand	37-40	20-40				05 - 09	06 - 10
9.5	104.4	117.2	2.45	2.7	462	Very dense, Silty sand to sandy silt	36-37	80-100				36 - 53	40 - 60
10.0	78.9	87.6	3.03	2.8	686	Hard, Sandy silt to sandy clay			30	5.22	6.06	36 - 54	40 - 60
10.5	30.5	33.5	1.73	3.1	542	Very stiff, Sandy clay to silty clay *			20	2.98	3.46	09 - 14	10 - 15
11.0	19.9	21.7	0.39	2.0	553	Very stiff, Sandy silt to sandy clay			15	2.57	0.78	04 - 06	04 - 06
11.5	10.7	11.6	0.45	3.0	488	Stiff, Clayey silt to silty clay			15	1.34	0.91	02 - 04	02 - 04
12.0	6.2	6.7	0.04	0.8	880	Stiff, Sandy silt to clayey silt			10	1.10	0.08	00 - 02	00 - 02
12.5	19.9	21.2	0.46	2.3	845	Very stiff, Sandy silt to sandy clay			15	2.56	0.91	04 - 06	04 - 06
13.0	12.8	13.5	0.92	1.3	352	Stiff, Sandy silt to clayey silt			15	1.61	1.85	00 - 02	00 - 02
13.5	158.0	165.2	2.20	2.0	461	Very dense, Silty sand to sandy silt	40-42	80-100				57 - 95	60 - 99
14.0	59.6	61.9	3.38	3.0	698	Hard, Sandy silt to sandy clay			25	4.70	6.76	19 - 29	20 - 30
14.5	8.0	8.2	0.48	2.6	733	Stiff, Clayey silt to silty clay			10	1.42	0.96	00 - 02	00 - 02
15.0	13.1	13.4	0.44	3.7	889	Stiff, Silty clay to clay *			15	1.62	0.88	04 - 06	04 - 06
15.5	11.8	12.0	1.01	8.1	815	Stiff, Silty clay to clay *			14	1.55	2.03	10 - 15	10 - 15
16.0	10.9	10.9	0.61	2.6	776	Stiff, Clayey silt to silty clay			15	1.32	1.22	00 - 02	00 - 02
16.5	18.8	18.8	1.39	5.6	577	Stiff, Silty clay to clay *			20	1.78	2.79	10 - 15	10 - 15
17.0	42.2	42.0	2.84	6.0	274	Very stiff, Sandy clay to silty clay **			25	3.29	5.67	40 - 60	40 - 60
17.5	52.1	51.7	1.54	3.0	571	Hard, Sandy silt to sandy clay			25	4.08	3.07	20 - 30	20 - 30
18.0	9.5	9.4	0.60	2.2	836	Stiff, Clayey silt to silty clay			10	1.69	1.20	00 - 02	00 - 02
18.5	9.1	9.0	0.54	3.9	578	Stiff, Silty clay to clay			15	1.07	1.08	02 - 04	02 - 04
19.0	21.1	20.8	1.49	6.4	799	Very stiff, Silty clay to clay *			18	2.22	2.97	15 - 20	15 - 20
19.5	20.7	20.3	1.22	5.0	860	Stiff, Silty clay to clay *			20	1.95	2.44	10 - 15	10 - 15
20.0	44.0	43.0	1.61	3.8	509	Very stiff, Sandy clay to silty clay *			25	3.42	3.22	20 - 31	20 - 30
20.5	25.5	24.9	2.10	4.4	711	Very stiff, Silty clay to clay *			20	2.43	4.19	10 - 15	10 - 15
21.0	37.4	36.3	1.40	4.0	658	Very stiff, Sandy clay to silty clay *			25	2.89	2.80	15 - 21	15 - 20
21.5	28.6	27.7	1.43	4.0	604	Very stiff, Silty clay to clay *			20	2.73	2.85	10 - 15	10 - 15
22.0	29.2	28.2	1.30	4.7	643	Very stiff, Silty clay to clay *			20	2.79	2.60	16 - 21	15 - 20
22.5	27.7	26.7	1.12	3.9	625	Very stiff, Silty clay to clay *			20	2.63	2.24	10 - 16	10 - 15
23.0	29.0	27.8	1.21	4.2	617	Very stiff, Silty clay to clay *			20	2.76	2.43	16 - 21	15 - 20
23.5	18.5	17.8	0.64	2.9	631	Very stiff, Sandy clay to silty clay *			15	2.28	1.29	04 - 06	04 - 06
24.0	16.3	15.6	0.50	3.3	586	Stiff, Sandy clay to silty clay *			15	1.98	1.00	04 - 06	04 - 06
24.5	11.1	10.5	0.43	3.1	642	Stiff, Silty clay to clay			15	1.28	0.87	02 - 04	02 - 04
25.0	10.4	9.9	0.38	3.5	612	Stiff, Silty clay to clay			15	1.19	0.76	02 - 04	02 - 04

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Mixed soils containing both granular and fine grained particles (e.g. clayey sands) may undergo partial drained failure during CPT.
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Structure rate of loading should be considered in choosing which strength parameters to use for design.
Drained and undrained parameters must not be combined as such combination will result in significant overprediction of in situ shear strength.

STRATIGRAPHICS Evaluated Properties Using Global Database

PROJECT NAME:Lansing Fly Ash Landfill Expansion
PROJECT NUMBER:00-120-160
R2DATE:11-21-2000 TIME:11:49:30.88
SOUNDING NUMBER:CP-02

Depth (ft)	Cone (tsf)	Norm Cone (tsf)	Friction (tsf)	Averaged Friction Ratio (%)	Soil Conductivity (uS/cm)	Evaluated Soil Type	Drained Friction Angle (deg)	Relative Density (%)	Nc	Undrained Shear Strength (ksf)	Undrained Large Strain Shear Strength (ksf)	SPT (N)	NORM SPT (N1')
25.5	10.7	10.2	0.49	2.3	326	Stiff, Clayey silt to silty clay			15	1.23	0.98	00 - 02	00 - 02
26.0	35.5	33.6	2.03	4.4	592	Very stiff, Silty clay to clay *			25	2.72	4.06	16 - 21	15 - 20
26.5	63.7	60.2	2.30	2.9	214	Hard, Sandy silt to sandy clay			25	4.97	4.59	21 - 32	20 - 30
27.0	58.6	55.2	2.33	2.6	627	Hard, Sandy silt to sandy clay			25	4.56	4.66	21 - 32	20 - 30
27.5	27.3	25.6	1.47	2.5	333	Very stiff, Sandy silt to sandy clay			20	2.56	2.93	06 - 11	06 - 10
28.0	102.3	95.8	2.54	0.9	-195131	Medium dense, Sand to silty sand	40-42	40-60				21 - 32	20 - 30

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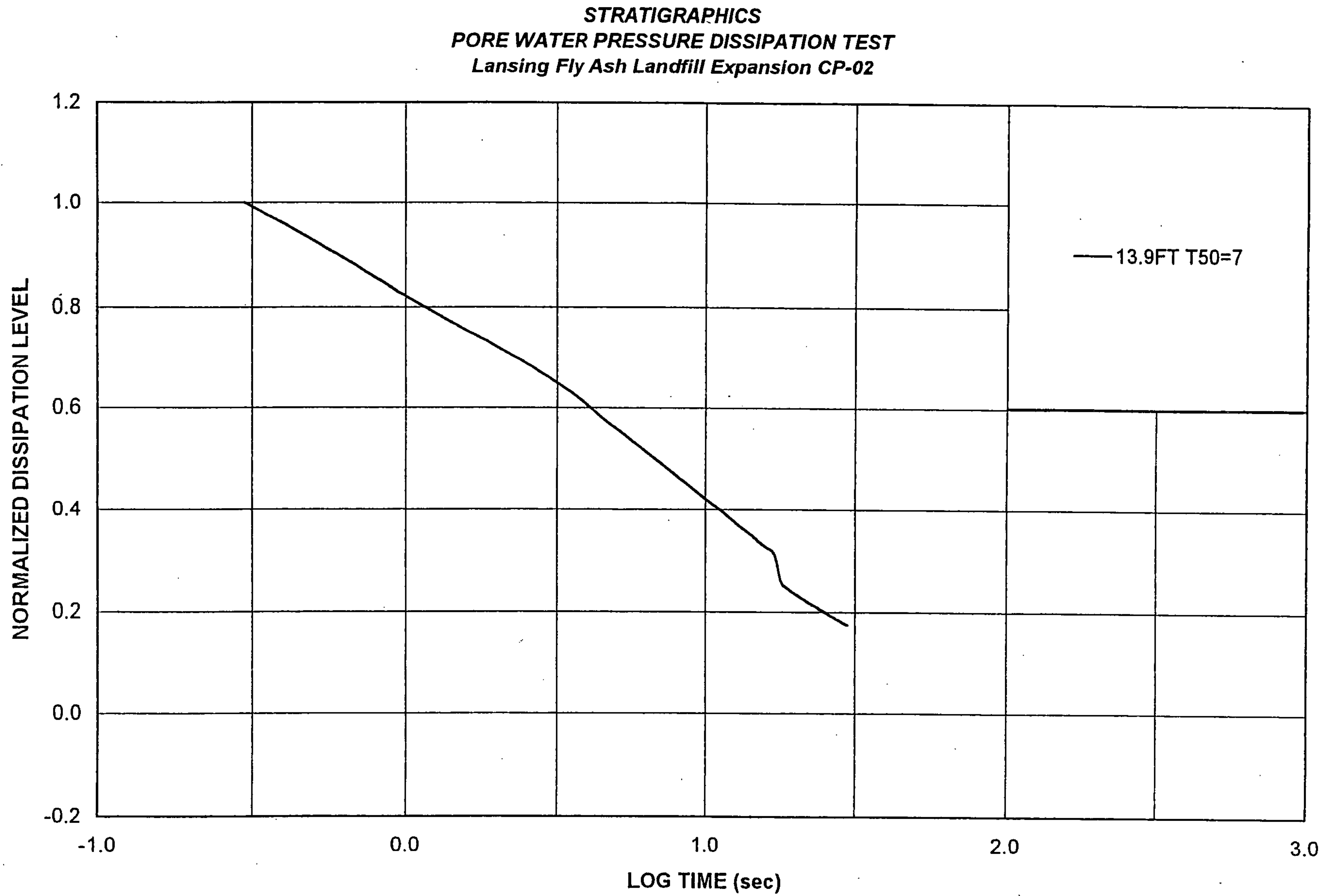
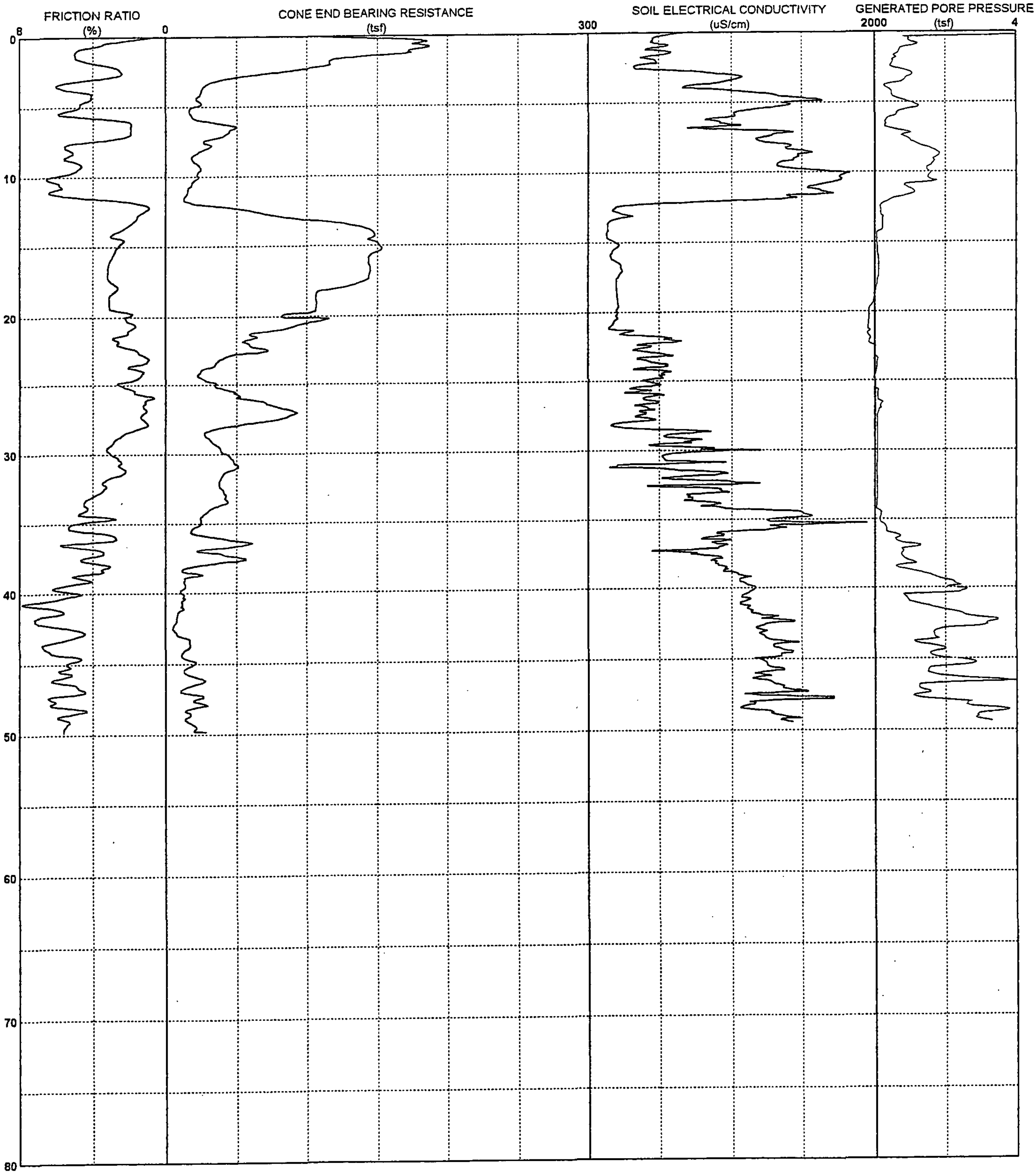

STRATIGRAPHICS
PORE WATER PRESSURE DISSIPATION TEST
Lansing Fly Ash Landfill Expansion CP-02
13.9FT T50=7
NORMALIZED DISSIPATION LEVEL
1.2
1.0
0.8
0.6
0.4
0.2
0.0
-0.2
-1.0
0.0
1.0
2.0
3.0
LOG TIME (sec)

CPTU-EC LOG



PROJECT NAME:Lansing Fly Ash Landfill Expansion
PROJECT NUMBER:00-120-160

STRATIGRAPHICS

R2DATE:11-21-2000 TIME:08:52:13.51
SOUNDING NUMBER:CP-03a

STRATIGRAPHICS Evaluated Properties Using Global Database

PROJECT NAME:Lansing Fly Ash Landfill Expansion
PROJECT NUMBER:00-120-160
R2DATE:11-21-2000 TIME:08:52:13.51
SOUNDING NUMBER:CP-03a

Depth (ft)	Cone (tsf)	Norm Cone (tsf)	Friction (tsf)	Averaged Friction Ratio (%)	Soil Conductivity (uS/cm)	Evaluated Soil Type	Drained Friction Angle (deg)	Relative Density (%)	Nc	Undrained Shear Strength (ksf)	Undrained Large Strain Shear Strength (ksf)	SPT (N)	NORM SPT (N1')
1.0	168.3	271.2	9.13	5.0	432	Hard, Gravelly Sandy clay to hardpan **			33	10.20	18.25	+ 62	+ 100
1.5	135.3	206.0	7.95	4.9	551	Hard, Gravelly Sandy clay to hardpan **			33	8.19	15.91	+ 66	+ 100
2.0	115.6	168.8	4.15	3.5	476	Hard, Gravelly clayey sand to gravelly sandy silt			33	7.00	8.30	+ 68	+ 100
2.5	87.3	123.3	2.63	2.5	335	Very dense, Silty sand to sandy silt	37-40	80-100				28 - 42	40 - 60
3.0	43.9	60.3	2.42	3.9	944	Very stiff, Sandy clay to silty clay *			25	3.50	4.85	22 - 29	30 - 40
3.5	27.5	36.8	2.03	6.0	961	Very stiff, Sandy clay to silty clay **			25	2.18	4.06	22 - 30	30 - 40
4.0	24.8	32.4	1.04	4.2	761	Stiff, Silty clay to clay *			25	1.98	2.08	11 - 15	15 - 20
4.5	21.8	28.0	0.96	4.1	1328	Very stiff, Silty clay to clay *			20	2.15	1.93	12 - 16	15 - 20
5.0	20.7	26.1	1.10	4.7	1396	Very stiff, Silty clay to clay *			20	2.04	2.20	12 - 16	15 - 20
5.5	16.9	20.9	1.17	5.8	1169	Stiff, Silty clay to clay *			20	1.65	2.34	12 - 16	15 - 20
6.0	26.4	32.2	0.83	2.3	1020	Very stiff, Sandy silt to sandy clay			20	2.60	1.66	05 - 08	06 - 10
6.5	49.4	59.5	0.93	2.0	935	Medium dense, Silty sand to sandy silt	36-37	40-60				12 - 17	15 - 20
7.0	41.8	49.7	0.90	2.0	1199	Medium dense, Silty sand to sandy silt	27-31	40-60				13 - 17	15 - 20
7.5	26.6	31.2	1.52	4.4	1226	Very stiff, Silty clay to clay *			25	2.09	3.04	13 - 17	15 - 20
8.0	29.0	33.6	1.60	5.4	1411	Very stiff, Silty clay to clay *			25	2.28	3.20	17 - 26	20 - 30
8.5	19.5	22.3	1.25	5.3	1539	Stiff, Silty clay to clay *			20	1.90	2.50	13 - 17	15 - 20
9.0	20.8	23.6	1.10	4.8	1458	Very stiff, Silty clay to clay *			20	2.03	2.20	09 - 13	10 - 15
9.5	24.7	27.7	1.13	4.8	1334	Very stiff, Silty clay to clay *			20	2.41	2.26	13 - 18	15 - 20
10.0	22.9	25.4	1.48	6.3	1787	Very stiff, Silty clay to clay *			20	2.23	2.96	18 - 27	20 - 30
10.5	18.3	20.2	1.21	5.8	1770	Stiff, Silty clay to clay *			20	1.77	2.41	14 - 18	15 - 20
11.0	15.7	17.1	1.06	6.2	1575	Very stiff, Silty clay to clay *			15	2.00	2.12	09 - 14	10 - 15
11.5	13.1	14.2	0.71	4.6	1704	Stiff, Silty clay to clay *			15	1.66	1.42	06 - 09	08 - 10
12.0	20.4	21.9	0.54	1.3	1202	Loose, Silty sand to sandy silt	27-31	20-40				04 - 06	04 - 06
12.5	60.1	63.9	0.85	1.2	203	Medium dense, Silty sand to sandy silt	37-40	40-60				14 - 19	15 - 20
13.0	87.5	92.2	1.77	1.6	258	Dense, Silty sand to sandy silt	37-40	60-80				19 - 28	20 - 30
13.5	130.0	135.9	2.96	2.1	157	Dense, Silty sand to sandy silt	37-40	60-80				38 - 57	40 - 60
14.0	147.0	152.5	4.12	2.8	139	Very dense, Gravelly silty sand to clayey gravelly s	36-37	+100				58 - 95	60 - 99
14.5	145.3	149.7	3.42	2.4	134	Very dense, Gravelly silty sand to clayey gravelly s	37-40	80-100				58 - 96	60 - 99
15.0	151.8	155.2	4.10	2.7	204	Very dense, Gravelly silty sand to clayey gravelly s	37-40	80-100				59 - 97	60 - 99
15.5	148.7	151.0	4.38	2.9	187	Very dense, Gravelly silty sand to clayey gravelly s	36-37	+100				+ 98	+ 100
16.0	143.9	145.2	4.49	3.1	164	Hard, Gravelly clayey sand to gravelly sandy silt			33	8.66	8.98	+ 99	+ 100
16.5	144.9	145.2	4.63	3.2	228	Hard, Gravelly clayey sand to gravelly sandy silt			33	8.72	9.25	+ 100	+ 100
17.0	144.6	144.0	4.67	3.2	250	Hard, Gravelly clayey sand to gravelly sandy silt			33	8.70	9.34	+ 100	+ 100
17.5	141.5	140.0	4.37	3.0	214	Very dense, Gravelly silty sand to clayey gravelly s	36-37	+100				61 - 100	60 - 99
18.0	125.9	123.8	3.61	2.7	203	Very dense, Gravelly silty sand to clayey gravelly s	36-37	80-100				41 - 61	40 - 60
18.5	106.9	104.5	3.47	3.1	207	Hard, Gravelly clayey sand to gravelly sandy silt			30	7.05	6.93	41 - 61	40 - 60
19.0	107.0	104.0	3.34	3.1	212	Hard, Gravelly clayey sand to gravelly sandy silt			30	7.06	6.68	41 - 62	40 - 60
19.5	106.7	103.0	3.22	2.9	219	Very dense, Silty sand to sandy silt	36-37	80-100				41 - 62	40 - 60
20.0	82.7	79.4	2.38	2.3	210	Dense, Silty sand to sandy silt	36-37	60-80				21 - 31	20 - 30
20.5	100.2	95.7	1.89	1.9	202	Dense, Silty sand to sandy silt	37-40	60-80				31 - 42	30 - 40
21.0	81.0	76.9	1.84	2.0	157	Dense, Silty sand to sandy silt	36-37	60-80				21 - 32	20 - 30
21.5	62.9	59.4	1.78	2.7	238	Hard, Sandy silt to sandy clay			25	4.93	3.57	21 - 32	20 - 30
22.0	56.4	53.0	1.65	2.7	591	Hard, Sandy silt to sandy clay			25	4.41	3.30	21 - 32	20 - 30
22.5	71.0	66.4	1.05	1.7	448	Medium dense, Silty sand to sandy silt	36-37	40-60				21 - 32	20 - 30
23.0	43.4	40.4	0.65	1.1	479	Loose, Silty sand to sandy silt	36-37	20-40				08 - 11	06 - 10
23.5	35.2	32.6	0.42	1.3	403	Loose, Silty sand to sandy silt	27-31	20-40				06 - 11	06 - 10
24.0	24.9	22.9	0.41	1.3	532	Loose, Silty sand to sandy silt	27-31	20-40				04 - 07	04 - 06
24.5	24.7	22.6	0.46	1.5	524	Loose, Silty sand to sandy silt	27-31	20-40				04 - 07	04 - 06
25.0	35.0	31.9	1.03	2.6	405	Very stiff, Sandy silt to sandy clay			20	3.35	2.06	11 - 16	10 - 15

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STRATIGRAPHICS Evaluated Properties Using Global Database

PROJECT NAME:Lansing Fly Ash Landfill Expansion
PROJECT NUMBER:00-120-160
R2DATE:11-21-2000 TIME:08:52:13.51
SOUNDING NUMBER:CP-03a

Depth (ft)	Cone (tsf)	Norm Cone (tsf)	Friction (tsf)	Averaged Friction Ratio (%)	Soil Conductivity (uS/cm)	Evaluated Soil Type	Drained Friction Angle (deg)	Relative Density (%)	Nc	Undrained Shear Strength (ksf)	Undrained Large Strain Shear Strength (ksf)	SPT (N)	NORM SPT (N1')
25.5	48.2	43.8	0.91	1.8	315	Medium dense, Silty sand to sandy silt	27-31	40-60				11 - 17	10 - 15
26.0	53.2	48.0	0.53	0.8	535	Loose, Sand to silty sand	37-40	20-40				07 - 11	06 - 10
26.5	74.6	67.1	1.12	1.3	500	Medium dense, Silty sand to sandy silt	37-40	40-60				17 - 22	15 - 20
27.0	92.4	82.8	1.08	1.2	449	Medium dense, Sand to silty sand	37-40	40-60				22 - 34	20 - 30
27.5	80.2	71.5	1.10	1.3	348	Medium dense, Silty sand to sandy silt	37-40	40-60				17 - 22	15 - 20
28.0	45.4	40.3	0.70	1.1	224	Loose, Silty sand to sandy silt	36-37	20-40				07 - 11	06 - 10
28.5	27.4	24.2	0.79	2.3	529	Very stiff, Sandy silt to sandy clay			20	2.57	1.59	07 - 11	06 - 10
29.0	31.7	27.9	0.98	2.8	542	Very stiff, Sandy clay to silty clay *			20	3.00	1.95	07 - 11	06 - 10
29.5	37.8	33.1	1.26	3.2	550	Very stiff, Sandy clay to silty clay *			20	3.60	2.51	11 - 17	10 - 15
30.0	42.6	37.2	1.38	3.0	1214	Very stiff, Sandy silt to sandy clay			25	3.26	2.76	17 - 23	15 - 20
30.5	47.9	41.6	1.25	2.5	526	Very stiff, Sandy silt to sandy clay			25	3.68	2.51	17 - 23	15 - 20
31.0	50.6	43.8	1.23	2.5	382	Very stiff, Sandy silt to sandy clay			25	3.90	2.45	17 - 23	15 - 20
31.5	40.5	34.9	1.13	2.6	788	Very stiff, Sandy silt to sandy clay			20	3.86	2.27	12 - 17	10 - 15
32.0	37.7	32.4	1.35	3.5	520	Very stiff, Sandy clay to silty clay *			20	3.58	2.70	17 - 23	15 - 20
32.5	39.4	33.7	1.40	3.4	879	Very stiff, Sandy clay to silty clay *			25	3.00	2.80	18 - 23	15 - 20
33.0	42.7	36.4	1.68	3.9	962	Very stiff, Sandy clay to silty clay *			25	3.26	3.36	18 - 23	15 - 20
33.5	43.8	37.1	1.90	4.5	725	Very stiff, Silty clay to clay *			25	3.34	3.81	24 - 35	20 - 30
34.0	31.2	26.3	1.58	4.3	783	Very stiff, Silty clay to clay *			20	2.91	3.16	12 - 18	10 - 15
34.5	25.6	21.6	0.96	3.5	1509	Very stiff, Sandy clay to silty clay *			20	2.35	1.92	07 - 12	06 - 10
35.0	24.9	20.9	1.25	5.0	1285	Very stiff, Silty clay to clay *			20	2.28	2.51	12 - 18	10 - 15
35.5	18.0	15.2	1.11	4.6	1333	Very stiff, Silty clay to clay *			15	2.12	2.21	07 - 12	06 - 10
36.0	36.7	30.8	1.40	2.8	893	Very stiff, Sandy silt to sandy clay			20	3.45	2.80	12 - 18	10 - 15
36.5	58.2	48.7	3.04	5.7	982	Hard, Sandy clay to silty clay **			25	4.48	6.09	48 - 72	40 - 60
37.0	22.6	18.9	1.38	3.5	908	Very stiff, Sandy clay to silty clay *			15	2.72	2.76	07 - 12	06 - 10
37.5	52.4	43.7	2.09	4.4	778	Hard, Silty clay to clay *			25	4.01	4.18	24 - 36	20 - 30
38.0	29.1	24.2	1.49	3.2	895	Very stiff, Sandy clay to silty clay *			20	2.68	2.98	07 - 12	06 - 10
38.5	12.5	10.4	0.61	3.5	963	Stiff, Silty clay to clay			15	1.36	1.22	02 - 05	02 - 04
39.0	13.5	11.2	0.81	4.4	1050	Stiff, Silty clay to clay *			15	1.49	1.62	05 - 07	04 - 06
39.5	13.8	11.5	0.77	5.7	1062	Stiff, Silty clay to clay *			15	1.53	1.54	07 - 12	06 - 10
40.0	11.3	9.4	0.57	4.7	1182	Stiff, Silty clay to clay *			15	1.19	1.14	05 - 07	04 - 06
40.5	12.9	10.6	0.76	6.2	1089	Stiff, Silty clay to clay *			14	1.49	1.52	07 - 12	06 - 10
41.0	11.5	9.5	0.77	6.6	1078	Stiff, Silty clay to clay *			14	1.29	1.54	07 - 12	06 - 10
41.5	8.3	6.8	0.61	5.9	1136	Firm, Clay			12	0.96	1.22	05 - 07	04 - 06
42.0	7.9	6.5	0.55	7.1	1343	Firm, Clay			12	0.89	1.10	05 - 07	04 - 06
42.5	5.0	4.1	0.35	5.2	1287	Soft, Clay			10	0.50	0.70	00 - 02	00 - 02
43.0	8.7	7.1	0.67	4.7	1245	Stiff, Clay			10	1.22	1.34	02 - 05	02 - 04
43.5	16.9	13.8	1.04	6.3	1254	Very stiff, Silty clay to clay *			14	2.04	2.09	07 - 12	06 - 10
44.0	14.3	11.7	1.08	6.5	1311	Stiff, Silty clay to clay *			14	1.66	2.16	07 - 12	06 - 10
44.5	11.3	9.2	0.80	4.8	1443	Stiff, Silty clay to clay *			15	1.16	1.60	05 - 07	04 - 06
45.0	20.5	16.7	0.97	5.5	1195	Very stiff, Silty clay to clay *			15	2.38	1.94	12 - 18	10 - 15
45.5	12.6	10.3	1.02	5.8	1276	Stiff, Silty clay to clay *			15	1.32	2.03	05 - 07	04 - 06
46.0	23.6	19.1	1.44	5.9	1173	Very stiff, Silty clay to clay *			20	2.08	2.88	12 - 18	10 - 15
46.5	19.2	15.5	1.18	4.9	1268	Very stiff, Silty clay to clay *			15	2.19	2.36	07 - 12	06 - 10
47.0	11.7	9.5	0.92	4.4	1374	Stiff, Silty clay to clay *			15	1.19	1.84	05 - 07	04 - 06
47.5	21.1	17.0	1.50	6.3	1178	Very stiff, Silty clay to clay *			15	2.44	2.99	12 - 19	10 - 15
48.0	25.5	20.5	1.42	5.9	1132	Very stiff, Silty clay to clay *			20	2.26	2.84	19 - 25	15 - 20
48.5	17.1	13.8	0.88	5.0	1111	Stiff, Silty clay to clay *			15	1.90	1.77	07 - 12	06 - 10
49.0	14.3	11.5	0.97	5.4	1383	Stiff, Silty clay to clay *			15	1.51	1.93	07 - 12	06 - 10
49.5	21.6	17.3	1.24	5.6	N/A	Very stiff, Silty clay to clay *			15	2.48	2.49	12 - 19	10 - 15

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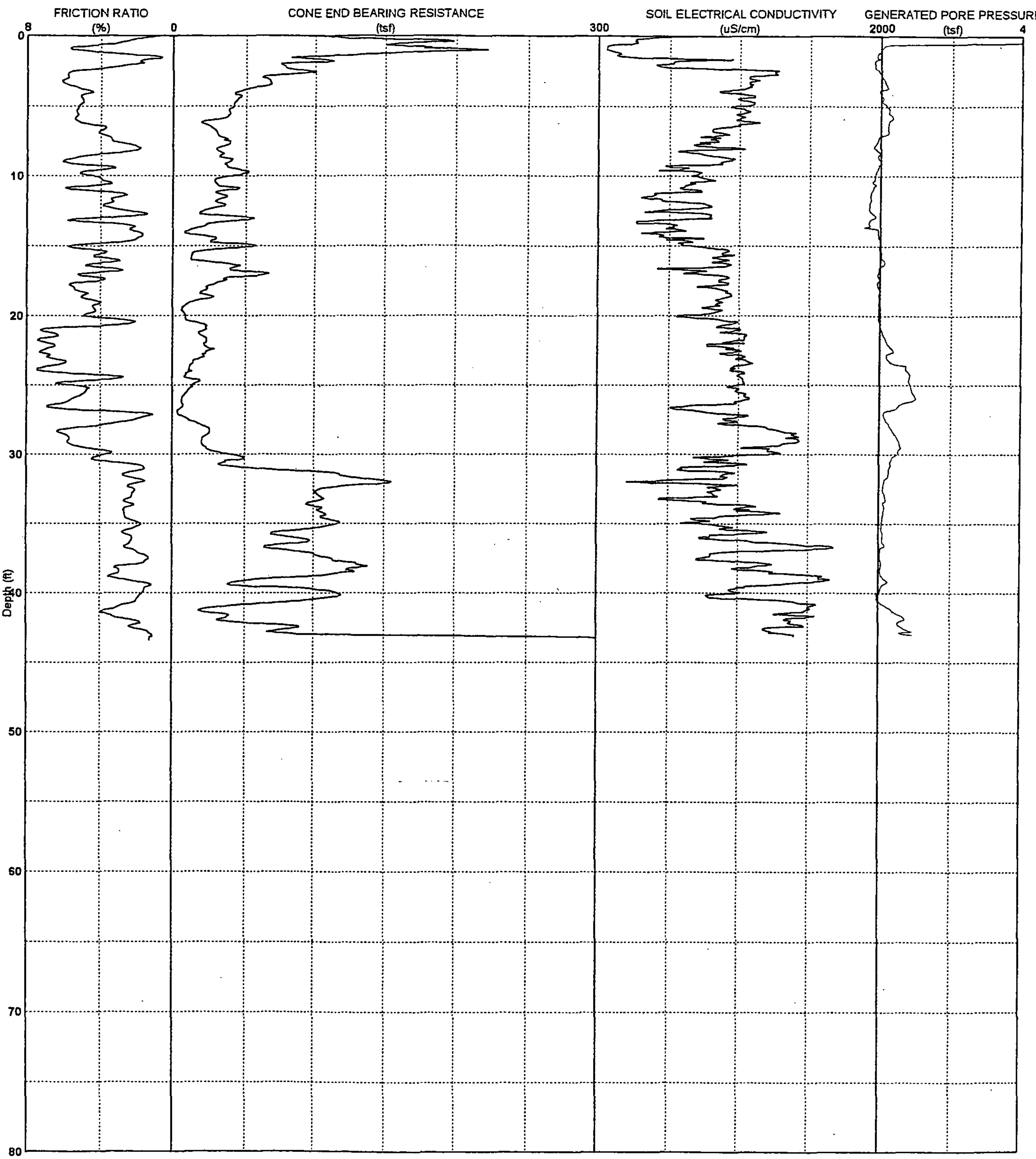
Drained and undrained parameters must not be combined as such combination will result in significant overprediction of in situ shear strength.

CPTU-EC LOG



PROJECT NAME:Lansing Fly Ash Landfill Expansion
PROJECT NUMBER:00-120-160

STRATIGRAPHICS

R2DATE:11-21-2000 TIME:10:30:53.77
SOUNDING NUMBER:CP-04

STRATIGRAPHICS Evaluated Properties Using Global Database

PROJECT NAME:Lansing Fly Ash Landfill Expansion
PROJECT NUMBER:00-120-160
R2DATE:11-21-2000 TIME:10:30:53.77
SOUNDING NUMBER:CP-04

Depth (ft)	Cone (tsf)	Norm Cone (tsf)	Friction (tsf)	Averaged Friction Ratio (%)	Soil Conductivity (uS/cm)	Evaluated Soil Type	Drained Friction Angle (deg)	Relative Density (%)	Nc	Undrained Shear Strength (ksf)	Undrained Large Strain Shear Strength (ksf)	SPT (N)	NORM SPT (N1')
1.0	212.8	342.9	6.23	4.9	61	Hard, Gravelly Sandy clay to hardpan **			33	12.90	12.47	+ 62	+ 100
1.5	88.7	135.1	0.99	0.7	242	Medium dense, Sand to silty sand	40-42	40-60				20 - 26	30 - 40
2.0	74.5	108.8	1.95	2.0	489	Dense, Silty sand to sandy silt	37-40	60-80				27 - 41	40 - 60
2.5	96.7	136.5	4.53	5.3	1272	Hard, Gravelly sandy clay to gravelly silty clay **			33	5.85	9.06	+ 71	+ 100
3.0	63.1	86.6	4.47	5.8	1075	Hard, Sandy clay to silty clay **			30	4.20	8.94	+ 73	+ 100
3.5	66.0	88.4	3.75	5.7	1078	Hard, Sandy clay to silty clay **			30	4.39	7.49	+ 75	+ 100
4.0	44.3	58.1	2.19	4.4	868	Very stiff, Gravelly sandy clay to gravelly silty clay *			30	2.94	4.38	31 - 46	40 - 60
4.5	44.4	57.0	2.22	5.0	995	Very stiff, Sandy clay to silty clay **			30	2.94	4.45	31 - 47	40 - 60
5.0	41.8	52.8	2.19	5.1	1001	Very stiff, Sandy clay to silty clay **			25	3.32	4.38	32 - 48	40 - 60
5.5	39.3	48.8	2.06	5.2	1031	Very stiff, Sandy clay to silty clay **			25	3.12	4.13	32 - 48	40 - 60
6.0	26.0	31.8	1.87	5.3	977	Very stiff, Silty clay to clay *			25	2.05	3.74	16 - 25	20 - 30
6.5	26.3	31.6	1.07	3.7	929	Very stiff, Sandy clay to silty clay *			25	2.07	2.13	12 - 17	15 - 20
7.0	30.5	36.2	1.35	4.0	914	Very stiff, Sandy clay to silty clay *			25	2.40	2.70	13 - 17	15 - 20
7.5	38.3	44.9	1.22	3.3	847	Very stiff, Sandy clay to silty clay *			25	3.03	2.43	17 - 26	20 - 30
8.0	30.6	35.5	0.67	1.8	994	Medium dense, Silty sand to sandy silt	27-31	40-60				05 - 09	06 - 10
8.5	33.0	37.8	1.65	4.4	751	Very stiff, Silty clay to clay *			25	2.60	3.29	17 - 26	20 - 30
9.0	38.7	43.9	2.26	6.0	877	Very stiff, Sandy clay to silty clay **			25	3.05	4.53	35 - 53	40 - 60
9.5	44.0	49.3	1.63	3.6	589	Very stiff, Sandy clay to silty clay *			25	3.47	3.26	18 - 27	20 - 30
10.0	41.0	45.6	2.02	4.3	653	Very stiff, Silty clay to clay *			25	3.23	4.04	18 - 27	20 - 30
10.5	29.3	32.2	1.19	3.4	648	Very stiff, Sandy clay to silty clay *			20	2.87	2.39	14 - 18	15 - 20
11.0	40.3	44.0	1.88	5.2	653	Very stiff, Sandy clay to silty clay **			25	3.17	3.76	28 - 37	30 - 40
11.5	37.0	40.0	0.97	2.9	320	Very stiff, Sandy silt to sandy clay			25	2.90	1.95	14 - 19	15 - 20
12.0	34.4	36.8	1.18	3.6	747	Very stiff, Sandy clay to silty clay *			25	2.69	2.36	14 - 19	15 - 20
12.5	21.6	22.9	0.82	2.6	419	Very stiff, Sandy silt to sandy clay			20	2.08	1.64	06 - 09	06 - 10
13.0	53.6	56.5	1.53	3.9	791	Hard, Sandy clay to silty clay *			25	4.23	3.05	28 - 38	30 - 40
13.5	23.1	24.1	0.96	2.5	536	Very stiff, Sandy silt to sandy clay			20	2.23	1.92	06 - 10	06 - 10
14.0	7.7	8.0	0.35	1.9	505	Stiff, Clayey silt to silty clay			10	1.37	0.70	00 - 02	00 - 02
14.5	29.3	30.2	0.76	2.0	676	Very stiff, Sandy silt to sandy clay			20	2.85	1.51	06 - 10	06 - 10
15.0	56.1	57.4	2.20	5.6	629	Very stiff, Sandy clay to silty clay **			30	3.68	4.41	39 - 59	40 - 60
15.5	13.4	13.6	1.16	3.8	885	Stiff, Silty clay to clay *			15	1.66	2.31	04 - 06	04 - 06
16.0	12.6	12.8	0.92	3.0	927	Stiff, Clayey silt to silty clay			15	1.56	1.84	04 - 06	04 - 06
16.5	43.6	43.7	2.14	4.3	862	Very stiff, Sandy clay to silty clay *			25	3.41	4.27	20 - 30	20 - 30
17.0	65.2	64.9	2.63	4.9	667	Hard, Sandy clay to silty clay **			30	4.28	5.26	40 - 60	40 - 60
17.5	35.2	34.9	1.80	4.1	854	Very stiff, Sandy clay to silty clay *			25	2.73	3.60	15 - 20	15 - 20
18.0	23.5	23.1	1.49	5.3	810	Very stiff, Silty clay to clay *			20	2.24	2.98	15 - 20	15 - 20
18.5	22.4	21.9	1.07	4.8	948	Very stiff, Silty clay to clay *			20	2.13	2.15	10 - 15	10 - 15
19.0	13.1	12.8	0.85	4.0	873	Stiff, Silty clay to clay *			15	1.60	1.70	04 - 06	04 - 06
19.5	6.1	5.9	0.39	4.3	795	Firm, Silty clay to clay			10	0.98	0.77	00 - 02	00 - 02
20.0	8.5	8.2	0.51	4.9	561	Stiff, Silty clay to clay *			10	1.47	1.02	04 - 06	04 - 06
20.5	16.6	15.9	0.46	2.2	987	Very stiff, Sandy silt to clayey silt			15	2.06	0.93	-04 - 06	04 - 06
21.0	22.7	21.6	1.61	7.2	1013	Very stiff, Silty clay to clay *			18	2.39	3.21	21 - 32	20 - 30
21.5	21.3	20.2	1.46	6.5	1044	Very stiff, Silty clay to clay *			18	2.22	2.91	16 - 21	15 - 20
22.0	21.4	20.2	1.61	6.8	1040	Very stiff, Silty clay to clay *			18	2.23	3.22	16 - 21	15 - 20
22.5	25.4	23.9	1.68	7.1	988	Very stiff, Silty clay to clay *			18	2.67	3.35	21 - 32	20 - 30
23.0	20.5	19.2	1.55	6.9	1040	Very stiff, Silty clay to clay *			18	2.12	3.09	16 - 21	15 - 20
23.5	12.9	12.1	0.96	6.2	1076	Stiff, Silty clay to clay *			14	1.64	1.93	06 - 11	06 - 10
24.0	13.4	12.6	0.89	7.1	963	Stiff, Silty clay to clay *			14	1.71	1.78	11 - 16	10 - 15
24.5	9.9	9.2	0.44	2.9	1026	Stiff, Silty clay to clay			15	1.13	0.88	00 - 02	00 - 02
25.0	13.9	13.0	0.86	5.5	1009	Stiff, Silty clay to clay *			15	1.66	1.71	06 - 11	06 - 10

* Indicates lightly overconsolidated soil
** Indicates heavily overconsolidated or cemented soil

Mixed soils containing both granular and fine grained particles (e.g. clayey sands) may undergo partial drained failure during CPT.
Both undrained and drained parameters can be estimated for these soils.

Structure rate of loading should be considered in choosing which strength parameters to use for design.
Drained and undrained parameters must not be combined as such combination will result in significant overprediction of in situ shear strength.

STRATIGRAPHICS Evaluated Properties Using Global Database

PROJECT NAME:Lansing Fly Ash Landfill Expansion
PROJECT NUMBER:00-120-160
R2DATE:11-21-2000 TIME:10:30:53.77
SOUNDING NUMBER:CP-04

Depth (ft)	Cone (tsf)	Norm Cone (tsf)	Friction (tsf)	Averaged Friction Ratio (%)	Soil Conductivity (uS/cm)	Evaluated Soil Type	Drained Friction Angle (deg)	Relative Density (%)	Nc	Undrained Shear Strength (ksf)	Undrained Large Strain Shear Strength (ksf)	SPT (N)	NORM SPT (N1')
25.5	9.8	9.1	0.57	4.8	992	Stiff, Silty clay to clay *			15	1.10	1.13	04 - 06	04 - 06
26.0	6.2	5.8	0.43	5.6	1079	Firm, Clay			12	0.78	0.86	02 - 04	02 - 04
26.5	7.2	6.7	0.45	6.9	649	Firm, Clay			12	0.94	0.91	04 - 06	04 - 06
27.0	3.4	3.1	0.10	1.8	798	Very soft, Sensitive fine grained soil			18	0.20	0.21	00 - 02	00 - 02
27.5	11.4	10.5	0.36	2.2	905	Stiff, Clayey silt to silty clay			15	1.30	0.72	00 - 02	00 - 02
28.0	22.9	21.0	1.17	4.7	1189	Very stiff, Silty clay to clay *			20	2.12	2.34	11 - 16	10 - 15
28.5	25.0	22.9	1.50	6.0	1410	Very stiff, Silty clay to clay *			20	2.33	3.00	16 - 22	15 - 20
29.0	20.3	18.5	1.26	5.7	1436	Stiff, Silty clay to clay *			20	1.86	2.53	11 - 16	10 - 15
29.5	23.3	21.2	1.37	4.8	1074	Very stiff, Silty clay to clay *			20	2.15	2.75	11 - 16	10 - 15
30.0	38.8	35.3	1.64	3.7	1230	Very stiff, Sandy clay to silty clay *			25	2.96	3.28	17 - 22	15 - 20
30.5	36.7	33.3	1.71	3.8	825	Very stiff, Sandy clay to silty clay *			25	2.79	3.43	17 - 22	15 - 20
31.0	52.6	47.6	1.41	1.6	592	Medium dense, Silty sand to sandy silt	36-37	40-60				11 - 17	10 - 15
31.5	118.8	107.3	3.79	2.7	889	Very dense, Silty sand to sandy silt	36-37	80-100				44 - 66	40 - 60
32.0	155.2	139.8	2.11	1.6	215	Dense, Silty sand to sandy silt	40-42	60-80				44 - 67	40 - 60
32.5	105.1	94.4	2.73	2.2	852	Dense, Silty sand to sandy silt	37-40	60-80				33 - 45	30 - 40
33.0	104.1	93.4	2.32	2.3	841	Dense, Silty sand to sandy silt	37-40	60-80				33 - 45	30 - 40
33.5	96.8	86.6	2.58	2.4	753	Dense, Silty sand to sandy silt	36-37	60-80				34 - 45	30 - 40
34.0	102.4	91.5	2.91	2.7	1085	Very dense, Silty sand to sandy silt	36-37	80-100				45 - 67	40 - 60
34.5	105.0	93.5	2.99	2.6	863	Very dense, Silty sand to sandy silt	36-37	80-100				45 - 67	40 - 60
35.0	117.6	104.5	1.86	1.7	692	Dense, Silty sand to sandy silt	37-40	60-80				34 - 45	30 - 40
35.5	84.3	74.8	2.69	2.6	1077	Dense, Silty sand to sandy silt	27-31	60-80				34 - 45	30 - 40
36.0	87.0	77.0	2.13	2.2	721	Dense, Silty sand to sandy silt	36-37	60-80				23 - 34	20 - 30
36.5	76.4	67.5	2.22	2.5	1416	Dense, Silty sand to sandy silt	27-31	60-80				23 - 34	20 - 30
37.0	88.8	78.3	2.06	2.0	1070	Dense, Silty sand to sandy silt	36-37	60-80				23 - 34	20 - 30
37.5	112.4	98.9	1.56	1.3	719	Medium dense, Sand to silty sand	37-40	40-60				23 - 34	20 - 30
38.0	134.7	118.3	3.47	2.6	1211	Very dense, Silty sand to sandy silt	36-37	80-100				46 - 68	40 - 60
38.5	126.4	110.7	3.82	3.0	1251	Hard, Gravelly clayey sand to gravelly sandy silt			30	8.27	7.65	46 - 68	40 - 60
39.0	60.4	52.8	2.64	2.8	1616	Hard, Sandy silt to sandy clay			25	4.65	5.29	23 - 34	20 - 30
39.5	51.4	44.9	1.24	1.3	1099	Medium dense, Silty sand to sandy silt	36-37	40-60				07 - 11	06 - 10
40.0	117.8	102.6	1.81	1.7	1021	Dense, Silty sand to sandy silt	37-40	60-80				34 - 46	30 - 40
40.5	97.7	85.0	2.28	2.0	1200	Dense, Silty sand to sandy silt	37-40	60-80				23 - 35	20 - 30
41.0	33.2	28.8	1.90	3.0	1505	Very stiff, Sandy clay to silty clay *			20	3.07	3.80	12 - 17	10 - 15
41.5	35.5	30.7	1.31	3.6	1294	Very stiff, Sandy clay to silty clay *			20	3.30	2.63	17 - 23	15 - 20
42.0	32.6	28.2	1.28	2.0	1390	Medium dense, Silty sand to sandy silt	27-31	40-60				07 - 12	06 - 10
42.5	88.4	76.3	1.87	2.3	1277	Dense, Silty sand to sandy silt	36-37	60-80				23 - 35	20 - 30
43.0	107.8	92.8	2.86	1.1	1380	Medium dense, Sand to silty sand	40-42	40-60				23 - 35	20 - 30

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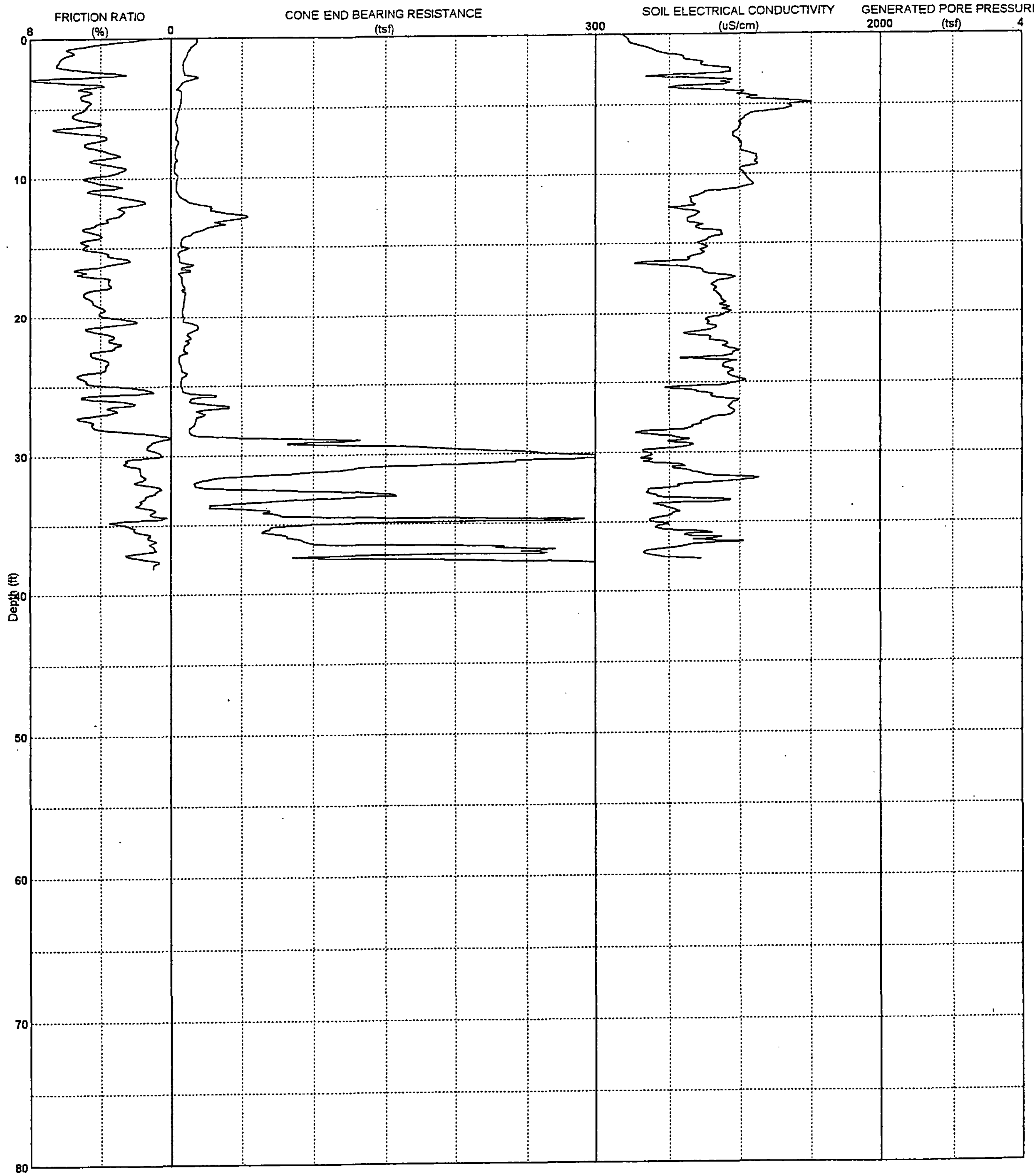
CPTU-EC LOG



PROJECT NAME:Lansing Fly Ash Landfill Expansion
PROJECT NUMBER:00-120-160

STRATIGRAPHICS

R2DATE:11-21-2000 TIME:14:30:30.51
SOUNDING NUMBER:CP-06

STRATIGRAPHICS Evaluated Properties Using Global Database

PROJECT NAME:Lansing Fly Ash Landfill Expansion
PROJECT NUMBER:00-120-160
R2DATE:11-21-2000 TIME:14:30:30.51
SOUNDING NUMBER:CP-06

Depth (ft)	Cone (tsf)	Norm Cone (tsf)	Friction (tsf)	Averaged Friction Ratio (%)	Soil Conductivity (uS/cm)	Evaluated Soil Type	Drained Friction Angle (deg)	Relative Density (%)	Nc	Undrained Shear Strength (ksf)	Undrained Large Strain Shear Strength (ksf)	SPT (N)	NORM SPT (N1')
1.0	11.7	18.9	0.73	5.4	375	Stiff, Silty clay to clay *			20	1.17	1.46	06 - 09	10 - 15
1.5	9.7	14.8	0.69	6.3	589	Stiff, Silty clay to clay *			14	1.37	1.39	07 - 10	10 - 15
2.0	8.6	12.6	0.59	6.5	725	Stiff, Silty clay to clay *			14	1.21	1.18	04 - 07	06 - 10
2.5	9.8	13.9	0.41	2.9	928	Stiff, Sandy clay to silty clay *			15	1.29	0.82	03 - 04	04 - 06
3.0	11.7	16.0	1.10	7.4	410	Stiff, Silty clay to clay *			14	1.64	2.20	07 - 11	10 - 15
3.5	5.1	6.8	0.38	4.9	903	Firm, Clay			10	0.97	0.75	01 - 03	02 - 04
4.0	7.2	9.4	0.35	4.9	940	Firm, Silty clay to clay *			15	0.92	0.70	03 - 05	04 - 06
4.5	7.0	8.9	0.32	4.6	1062	Firm, Silty clay to clay *			15	0.89	0.63	03 - 05	04 - 06
5.0	5.8	7.3	0.31	4.8	1365	Stiff, Clay			10	1.10	0.62	02 - 03	02 - 04
5.5	4.2	5.2	0.27	5.6	1202	Firm, Clay			12	0.64	0.55	00 - 02	00 - 02
6.0	3.3	4.1	0.16	4.0	1034	Soft, Clay			18	0.33	0.32	00 - 02	00 - 02
6.5	4.5	5.5	0.29	6.7	1008	Firm, Clay			12	0.69	0.57	02 - 03	02 - 04
7.0	3.4	4.0	0.15	3.7	953	Soft, Silty clay to clay			18	0.33	0.29	00 - 02	00 - 02
7.5	5.0	5.9	0.21	4.8	989	Firm, Clay			10	0.91	0.41	00 - 02	00 - 02
8.0	3.0	3.5	0.14	3.8	1013	Soft, Clay			18	0.28	0.28	00 - 02	00 - 02
8.5	2.8	3.2	0.11	3.3	1070	Soft, Silty clay to clay			18	0.25	0.21	00 - 02	00 - 02
9.0	2.5	2.9	0.12	3.6	1118	Very soft, Clay			18	0.22	0.24	00 - 02	00 - 02
9.5	2.4	2.7	0.11	2.8	1028	Very soft, Sensitive fine grained soil			18	0.20	0.21	00 - 02	00 - 02
10.0	4.7	5.2	0.21	4.9	1033	Firm, Clay			10	0.81	0.42	00 - 02	00 - 02
10.5	3.6	4.0	0.13	3.3	1073	Soft, Silty clay to clay			18	0.33	0.26	00 - 02	00 - 02
11.0	4.2	4.6	0.28	4.6	999	Soft, Clay			18	0.39	0.56	00 - 02	00 - 02
11.5	9.4	10.4	0.32	2.1	699	Stiff, Clayey silt to silty clay			15	1.17	0.64	00 - 02	00 - 02
12.0	25.2	27.5	0.72	2.4	650	Very stiff, Sandy silt to sandy clay			20	2.45	1.44	06 - 09	06 - 10
12.5	36.3	39.4	1.30	2.7	569	Very stiff, Sandy silt to sandy clay			25	2.84	2.59	14 - 18	15 - 20
13.0	45.7	49.4	1.78	3.6	669	Very stiff, Sandy clay to silty clay *			25	3.59	3.55	18 - 28	20 - 30
13.5	30.1	32.4	1.58	4.5	683	Very stiff, Silty clay to clay *			25	2.34	3.15	14 - 19	15 - 20
14.0	15.1	16.2	0.92	4.2	803	Stiff, Silty clay to clay *			15	1.90	1.84	06 - 09	06 - 10
14.5	7.3	7.8	0.53	4.9	803	Stiff, Clay			10	1.28	1.06	02 - 04	02 - 04
15.0	11.1	11.8	0.41	5.0	702	Stiff, Silty clay to clay *			15	1.36	0.82	04 - 06	04 - 06
15.5	4.7	5.0	0.26	3.6	720	Soft, Silty clay to clay			18	0.42	0.53	00 - 02	00 - 02
16.0	5.9	6.2	0.24	2.4	625	Firm, Clayey silt to silty clay			10	0.98	0.48	00 - 02	00 - 02
16.5	7.2	7.5	0.51	4.9	399	Stiff, Clay			10	1.24	1.01	02 - 04	02 - 04
17.0	6.3	6.6	0.48	5.2	735	Stiff, Clay			10	1.06	0.96	02 - 04	02 - 04
17.5	7.6	7.9	0.31	3.5	925	Stiff, Silty clay to clay			10	1.31	0.61	00 - 02	00 - 02
18.0	10.0	10.4	0.39	4.1	803	Stiff, Silty clay to clay *			15	1.19	0.78	04 - 06	04 - 06
18.5	9.6	10.0	0.47	4.9	829	Stiff, Silty clay to clay *			15	1.13	0.93	04 - 06	04 - 06
19.0	8.9	9.1	0.41	4.3	867	Stiff, Silty clay to clay *			15	1.03	0.82	04 - 06	04 - 06
19.5	8.0	8.2	0.32	3.8	909	Stiff, Silty clay to clay			10	1.36	0.64	02 - 04	02 - 04
20.0	9.3	9.5	0.36	3.5	891	Stiff, Silty clay to clay			15	1.08	0.72	02 - 04	02 - 04
20.5	15.7	16.1	0.44	2.4	771	Stiff, Clayey silt to silty clay			15	1.93	0.88	04 - 06	04 - 06
21.0	18.0	18.3	0.88	4.7	822	Very stiff, Silty clay to clay *			15	2.23	1.75	06 - 10	06 - 10
21.5	14.0	14.2	0.44	3.2	622	Stiff, Sandy clay to silty clay *			15	1.70	0.87	04 - 06	04 - 06
22.0	11.9	12.0	0.35	2.8	822	Stiff, Clayey silt to silty clay			15	1.41	0.70	02 - 04	02 - 04
22.5	9.8	9.9	0.45	4.2	904	Stiff, Silty clay to clay *			15	1.13	0.89	04 - 06	04 - 06
23.0	5.5	5.6	0.33	4.0	946	Firm, Silty clay to clay			10	0.83	0.66	00 - 02	00 - 02
23.5	6.9	6.9	0.28	3.6	939	Stiff, Silty clay to clay			10	1.10	0.57	00 - 02	00 - 02
24.0	9.8	9.8	0.41	4.3	927	Stiff, Silty clay to clay *			15	1.12	0.82	04 - 06	04 - 06
24.5	7.5	7.5	0.47	4.9	927	Stiff, Clay			10	1.21	0.94	02 - 04	02 - 04
25.0	7.0	6.9	0.27	3.1	987	Stiff, Silty clay to clay			10	1.09	0.53	00 - 02	00 - 02

* Indicates lightly overconsolidated soil
** Indicates heavily overconsolidated or cemented soil

Mixed soils containing both granular and fine grained particles (e.g. clayey sands) may undergo partial drained failure during CPT.
Both undrained and drained parameters can be estimated for these soils.

Structure rate of loading should be considered in choosing which strength parameters to use for design.
Drained and undrained parameters must not be combined as such combination will result in significant overprediction of in situ shear strength.

STRATIGRAPHICS Evaluated Properties Using Global Database

PROJECT NAME:Lansing Fly Ash Landfill Expansion
PROJECT NUMBER:00-120-160
R2DATE:11-21-2000 TIME:14:30:30.51
SOUNDING NUMBER:CP-06

Depth (ft)	Cone (tsf)	Norm Cone (tsf)	Friction (tsf)	Averaged Friction Ratio (%)	Soil Conductivity (uS/cm)	Evaluated Soil Type	Drained Friction Angle (deg)	Relative Density (%)	Nc	Undrained Shear Strength (ksf)	Undrained Large Strain Shear Strength (ksf)	SPT (N)	NORM SPT (N1')
25.5	13.3	13.1	0.22	1.6	621	Stiff, Sandy silt to clayey silt			15	1.57	0.44	00 - 02	00 - 02
26.0	14.6	14.4	0.84	3.6	798	Stiff, Silty clay to clay *			15	1.73	1.68	04 - 06	04 - 06
26.5	41.0	40.3	1.05	3.4	943	Very stiff, Sandy clay to silty clay *			25	3.15	2.10	15 - 20	15 - 20
27.0	23.8	23.3	1.02	3.8	961	Very stiff, Silty clay to clay *			20	2.22	2.05	10 - 15	10 - 15
27.5	17.8	17.4	0.93	4.6	790	Very stiff, Silty clay to clay *			15	2.15	1.86	06 - 10	06 - 10
28.0	13.5	13.2	0.71	4.3	690	Stiff, Silty clay to clay *			15	1.58	1.42	04 - 06	04 - 06
28.5	15.7	15.3	0.41	0.7	313	Very loose, Silty sand to sandy silt	31-36	0-20				00 - 02	00 - 02
29.0	126.3	122.4	1.26	1.0	607	Medium dense, Sand to silty sand	40-42	40-60				31 - 41	30 - 40
29.5	179.0	173.1	2.92	1.3	642	Dense, Sand to silty sand	40-42	60-80				41 - 62	40 - 60
30.0	263.4	253.9	1.46	0.5	361	Dense, Sandy gravel to gravelly sand	42-46	60-80				41 - 62	40 - 60
30.5	242.4	233.0	7.48	2.6	382	Very dense, Gravelly silty sand to clayey gravelly s	37-40	+100				+ 104	+ 100
31.0	129.3	124.0	3.31	1.7	567	Dense, Silty sand to sandy silt	37-40	60-80				42 - 63	40 - 60
31.5	53.8	51.4	1.46	1.5	733	Medium dense, Silty sand to sandy silt	36-37	40-60				10 - 16	10 - 15
32.0	16.6	15.9	0.64	1.9	1026	Stiff, Sandy silt to clayey silt			15	1.96	1.28	02 - 04	02 - 04
32.5	67.9	64.5	0.86	0.7	529	Loose, Sand to silty sand	37-40	20-40				11 - 16	10 - 15
33.0	148.7	141.1	1.81	1.4	399	Dense, Sand to silty sand	40-42	60-80				42 - 63	40 - 60
33.5	43.1	40.7	1.58	1.8	840	Medium dense, Silty sand to sandy silt	27-31	40-60				11 - 16	10 - 15
34.0	70.0	66.0	0.59	0.9	510	Medium dense, Sand to silty sand	37-40	40-60				11 - 16	10 - 15
34.5	116.4	109.6	0.10	0.3	513	Medium dense, Sand to silty sand	40-42	40-60				21 - 32	20 - 30
35.0	103.1	96.8	4.93	2.6	406	Very dense, Silty sand to sandy silt	36-37	80-100				43 - 64	40 - 60
35.5	65.1	61.0	1.38	1.8	452	Medium dense, Silty sand to sandy silt	36-37	40-60				16 - 21	15 - 20
36.0	86.4	80.7	1.18	1.2	655	Medium dense, Sand to silty sand	37-40	40-60				21 - 32	20 - 30
36.5	133.9	124.8	2.50	1.1	768	Medium dense, Sand to silty sand	40-42	40-60				32 - 43	30 - 40
37.0	252.5	234.7	2.68	1.3	379	Dense, Sand to silty sand	42-46	60-80				65 - 106	60 - 99
37.5	102.4	95.0	3.56	1.5	449	Dense, Silty sand to sandy silt	37-40	60-80				22 - 32	20 - 30
38.0	389.6	360.4	5.90	1.0	N/A	Very dense, Sand to silty sand	42-46	80-100				65 - 107	60 - 99

* Indicates lightly overconsolidated soil
** Indicates heavily overconsolidated or cemented soil

Mixed soils containing both granular and fine grained particles (e.g. clayey sands) may undergo partial drained failure during CPT.
Both undrained and drained parameters can be estimated for these soils.

Structure rate of loading should be considered in choosing which strength parameters to use for design.
Drained and undrained parameters must not be combined as such combination will result in significant overprediction of in situ shear strength.

APPENDIX A
TABLE OF CONTENTS

1.0 EVALUATION OF GEOTECHNICAL PARAMETERS

CPT data have been correlated with soil type, drained friction angle, undrained shear strength, relative density, and equivalent SPT blowcounts, among others. Correlations have been developed by comparing CPT results to laboratory tests on drilled samples and to other in situ tests, such as vane and pressuremeter. Laboratory CPT testing on large scale samples of known composition and classical bearing capacity and cavity expansion theory have also been used. Site specific information, where available, can be used to fine tune correlations.

A two parameter correlation scheme has proved useful for CPT data evaluation. Geotechnical properties often exhibit well defined trends when plotted against the logarithm of the CPT cone end bearing resistance and friction ratio. For instance, increased grain size increases cone end bearing resistance, while increased plasticity and compressibility increase friction ratio. A chart illustrating these and other trends is presented in Figure A2. A discussion of CPT data evaluation is presented in Douglas and Olsen, 1981.

<u>A1.1 CPT Soil Behavior Types</u> CPT soil behavior type correlations have been developed from geotechnical theory and comparisons of borehole data with CPT data (Douglas and Olsen, 1981). The CPT soil behavior type tabulations are indicative of the response of the soil to the large shear deformations imposed on the soil during penetrometer advance. Soil shear response is not entirely controlled by grain size distribution. However, it has been found that soil types correlated with CPT generally agree with classifications based on soil grain size distribution methods such as the Unified Soil Classification System (USCS). A soil classification chart developed for Midwestern United States soils is presented in Figure A3.

<u>A1.2 CPT Relative Density</u> Relative densities of granular soils are correlated with CPT data on the basis of laboratory CPT on large scale samples of known composition (Schmertmann, 1978, and Villet and Mitchell, 1981). The effect of soil fines content has been empirically accounted for by extrapolating trends in the two parameter correlation model (Douglas and Strutynsky, 1984). A relative density chart is presented in Figure A4.

<u>A1.3 CPT Drained Static Strength</u> Drained friction angles have been correlated with CPT results on the basis of CPT soundings and laboratory tests on drilled samples, and on theoretical analyses of the cone end bearing capacity problem (Schmertmann, 1978, Durgunoglu and Mitchell, 1974, and Villet and Mitchell, 1981). The effect of soil fines content on friction angles has been accounted for by extrapolating trends in the two parameter correlation model, as was done for the relative density correlation. A drained friction angle chart is also presented in Figure A4.

<u>A1.4 CPT Undrained Static Strength</u> The correlation between CPT data and undrained shear strength has been extensively studied (Douglas and others, 1984, Lunne and others, 1976, Sanglerat, 1972, and Schmertmann, 1978). The following bearing capacity equation can be used for computing undrained shear strength from CPT data: qu = (Su * Nc) + Sv (Eq. A1); where: qu = ultimate bearing capacity; Su = undrained shear strength; Nc = a dimensionless bearing capacity factor; and Sv = the estimated total vertical stress. By setting qu equal to the cone end bearing resistance, qc, and rearranging the equation, a value of the undrained shear strength can be computed as: Su = (qc - Sv)/Nc (Eq. A2).

The primary difficulty in using this equation has been the selection of Nc applicable to cone penetration in a particular soil. Bearing capacity and cavity expansion theory and other in situ and laboratory test results performed adjacent to CPT soundings have been used to calculate Nc values. These Nc values have ranged from 5 to over 25, but are most often between about 12 and 20. Higher Nc values are typically associated with overconsolidated clays and lower plasticity clays and clayey silts.

A compilation of Nc values as a function of cone end bearing resistance and friction ratio is presented in Figure A5. This figure was developed from comparisons of CPT to results of laboratory consolidated-undrained (CU) strength tests. This is important to note as undrained shear strength is not a unique property of a soil - it is test type and stress path dependent.

Many design methodologies are based on a particular strength test on a particular type of sample. These semi-empirical design methods are successfully used by experienced designers. Engineering judgment must be applied in using the results of any type of testing - whether in situ or laboratory - to assure both adequate safety and design economy.

High Strain, Remolded Strength Another measure of the in situ undrained shear strength is provided by the CPT friction sleeve resistance. The friction sleeve interacts with soil that has already undergone bearing capacity failure induced by the tip of the penetrometer. Thus, the friction sleeve resistance is a measure of soil large strain, remolded strength. The ratio between strengths calculated from the cone end bearing and from the friction sleeve is indicative of soil sensitivity.

In moderately to highly overconsolidated, non-sensitive clays, friction sleeve resistances can indicate higher strengths than those calculated using the cone end bearing resistance. This often reflects the dilative (strain hardening) nature of shear failure in overconsolidated soils. Engineering judgment must be applied in deciding which strain level, and thus which strength, is representative for the design problem to be solved.

A1.5 Equivalent SPT Blowcount N-Values An equivalent SPT blowcount can be correlated with CPT data by using an analytical model of the SPT procedure (Douglas and Olsen, 1981). This procedure has been checked by comparison to SPT results at various sites throughout the world (Douglas and others, 1981, Douglas and Strutynsky, 1984, and Olsen and Farr, 1986) with generally good results.

The particular SPT equipment used to develop the CPT-SPT correlation chart (Figure A6) consisted of a SPT trip hammer system. This SPT hammer is characterized by reasonably repeatable, measured hammer input energy efficiencies of about 60 to 70% (Douglas and Strutynsky, 1984). This hammer input energy level is similar to that recommended (Seed and others, 1984) as the "standard" Standard Penetration Test input energy.

SPT results are both equipment and operator dependent. SPT hammer efficiencies have been measured to range from 35 to over 90% of the theoretical 4200 in-lbs (30 inch height of fall, 140 lbs hammer) SPT input energy. Variable SPT input energy results in variable blowcounts (Douglas and Strutynsky, 1984, Seed and others, 1984). This problem of non-uniform input energy during SPT provides a limitation for quantitative design purposes.

The approach of using the extensive SPT data base, in addition to the CPT data base, by performing CPT and then deriving equivalent SPT blowcount N-values, typically results in higher quality site information. This is because CPT is continuous, has higher resolution, is less expensive, and is much more consistent and repeatable than SPT. The chart that was used for correlating CPT to SPT for this study is presented in Figure A6. After determining the overburden normalized equivalent SPT N'-value, the equivalent SPT blowcount N-value was calculated by dividing the overburden normalized value by the overburden normalization factor CN, as defined in Eq. A3.

The equivalent SPT N-values reflect the higher resolution of the CPT measurements as compared to actual SPT. Performance of actual SPT includes averaging of soil resistance over about a 24 inch interval (18 inch sampler embedment and 2 to 3 sampler diameters ahead of the sampler). Equivalent SPT values correlated with CPT data have a resolution of about six inches. Rather than coarsen the 6 inch resolution equivalent SPT N-value to fit a 24 inch resolution actual SPT N-value, interpreted values are based on point by point CPT data. These high resolution, equivalent SPT values should be more useful for design purposes, especially in interlayered deposits, where thin, weak soil seams cannot be adequately characterized by actual SPT blowcount methods. The high resolution equivalent SPT values and actual SPT measurements should be similar in thick homogeneous strata.

Discrepancies between CPT equivalent SPT N-values and actual, measured SPT N-values are often due to inconsistencies in the performance of actual SPT. Poor fit of CPT equivalent and actual SPT in weak soils with very low blowcounts (0 to 3) can be due to limited accuracy of high capacity CPT loadcells used at the extreme low end of their range, but are more likely caused by extensive borehole disturbance in easily disturbed soil, and set of the SPT sampler under the self-weight of the hammer and drillrods. Discrepancies between equivalent and actual SPT values in very dense or hard soils with high blowcounts, especially in gravelly soils, can be due to both erratic penetrometer or SPT sampler interaction with large soil particles, and basic differences in modes of penetration of the two techniques. Indications of very weak soils, using any method, should strongly encourage additional testing, including undisturbed sampling and sophisticated laboratory testing.

A2.0 OVERBURDEN PRESSURE NORMALIZATION

Overburden normalization of CPT data for correlation purposes is necessary in order to remove the effects of increasing confining pressure with depth on measured results. Cone end bearing resistances can be normalized to an effective vertical overburden pressure of 1 TSF by using the following equations: $qc1 = qc * CN$ (Eq. A3); and $CN = 1.0 - 0.5 * \log(Sv')$ (Eq. A4); where: qc1 is the overburden normalized cone end bearing resistance, in TSF; qc is the measured cone resistance, in TSF; CN is the overburden normalization factor; and Sv' is the effective vertical overburden stress in TSF.

Overburden normalization curves are variable (Douglas and Martin, 1980). Most were developed using laboratory CPT and SPT on large scale samples of clean sands, compacted at various relative densities and subjected to various overburden pressures. Application of laboratory results to natural soils may be limited. The CN presented in Equation A4 is similar to that proposed (Seed and others, 1977) for the effect of overburden on SPT blowcounts.

The friction ratio is not normalized based on the assumption that overburden pressure affects friction sleeve and cone end bearing resistance similarly. Since the quantities are divided by each other to compute friction ratio, overburden effects should cancel. Some experience (Olsen and Farr, 1986) indicates that this assumption may oversimplify actual conditions for deep soundings. The friction resistance may be less sensitive to overburden pressure than the cone end bearing resistance. Thus, in soundings deeper than about 100 ft, the friction ratio may gradually decrease with increased penetration, independent of any changes in soil conditions, other than overburden pressure. Due to the variability in overburden normalization curves, no specific correction for overburden pressure on friction ratio has been recommended or used for this study. For this study, effective stresses in Equation A4 were computed using assumed water tables and soil unit weights.

A3.0 TEST DRAINAGE CONDITION

The CPT loading rate is such that drained and undrained conditions exist during penetration of sands and clays, respectively. Partial drainage may occur in mixed soils. Lack of boundary drainage control during any in situ test complicates data analysis, especially in mixed soils, as both frictional and cohesive behavior can be exhibited during testing.

CPTU piezometric data indicate that minor differences in cone end bearing and friction ratio response can correspond with major changes in pore water pressure response during the test (Douglas and others, 1985). The complex volumetric strain field around the penetrometer (Davidson and Boghrat, 1983) precludes reliable geotechnical effective stress analysis of CPTU results in partially drained soil.

Empirical estimates of either drained or undrained parameters can be made in soils composed of mixtures of granular and fine grained particles. These parameters must not be combined - they are to be used alternatively. Combination of the drained and undrained parameters for geotechnical analysis will result in significant overestimation of in situ shear strength.

Structure rate of loading will help determine which geotechnical parameters, whether drained or undrained, should be appropriate for design use. Depending on project needs and extent of such soils at a site, geotechnical laboratory testing including CU tests with pore pressure measurements and consolidation tests will also be useful in assigning appropriate design parameters. Field instrumentation during construction using low volume change piezometers may be appropriate for some projects.

A4.0 RECOMMENDED PRACTICES

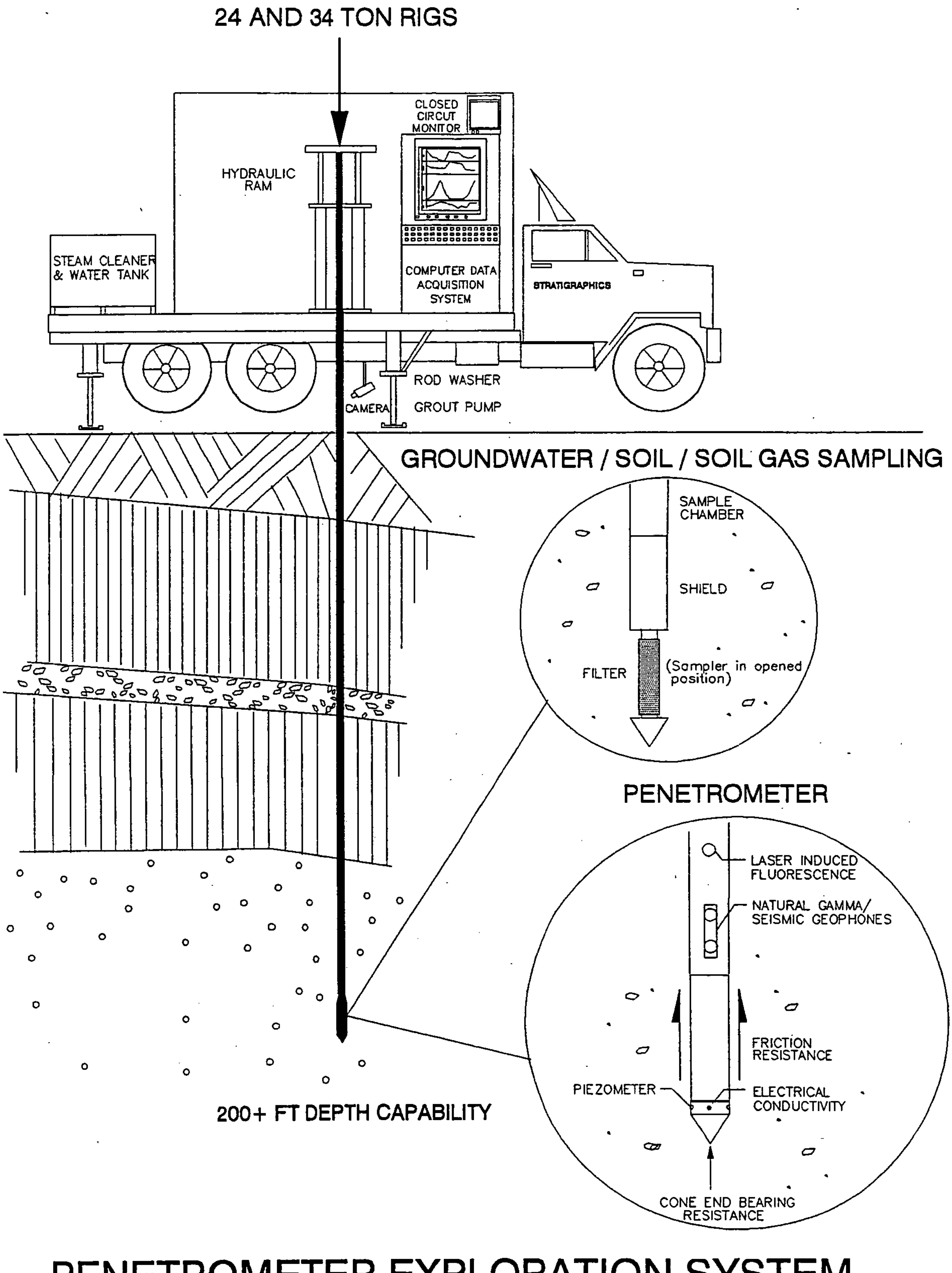
The STRATIGRAPHICS parameter evaluation program tracks the CPT data through a series of correlation charts, Figures A2 through A6. Parameters are computer evaluated and tabulated at discrete intervals. The following practices are recommended when reviewing tabulated data and correlated parameters:

Stratigraphic units should be defined on the basis of the continuous sounding logs and project requirements. The tabulations are then used in evaluation of layer properties. Use of the tabulations without the review of the continuous sounding logs can lead to the choice of non-representative parameters, especially in interlayered deposits. It should be noted that taking discontinuous borehole soil samples also often provides a poor representation of subsurface conditions.

CPT correlations have been developed using empiricism. This data base is world-wide, and includes decades of CPT experience. However, unique local conditions may differ from those in the global data base used for this study. Thus, the provided tabulations of evaluated parameters should be viewed as indicating trends rather than as the exact equivalent of specific laboratory tests performed under boundary and drainage controlled conditions.

While CPT suffers from none of the effects of sample disturbance as found during drilled investigations, boundary and drainage conditions are not well defined during any in situ test such as CPT. The derived parameters are not intended to replace appropriate drilling and undisturbed sampling, other in situ and laboratory testing, and use of engineering judgment.

Review of CPT results and project requirements is used to define the need for additional information. Zones delineated by CPT (or, in fact, any other test) providing low factors of safety should be further investigated. Select undisturbed sampling followed by geotechnical triaxial and consolidation testing may be indicated for low strength cohesive or partially drained mixed soil strata. Monitoring wells may be installed or groundwater samples taken in CPT(U) identified high permeability strata during geo-environmental investigations. Laboratory and other test results can then be extrapolated across the site based on CPT(U) defined stratigraphy.



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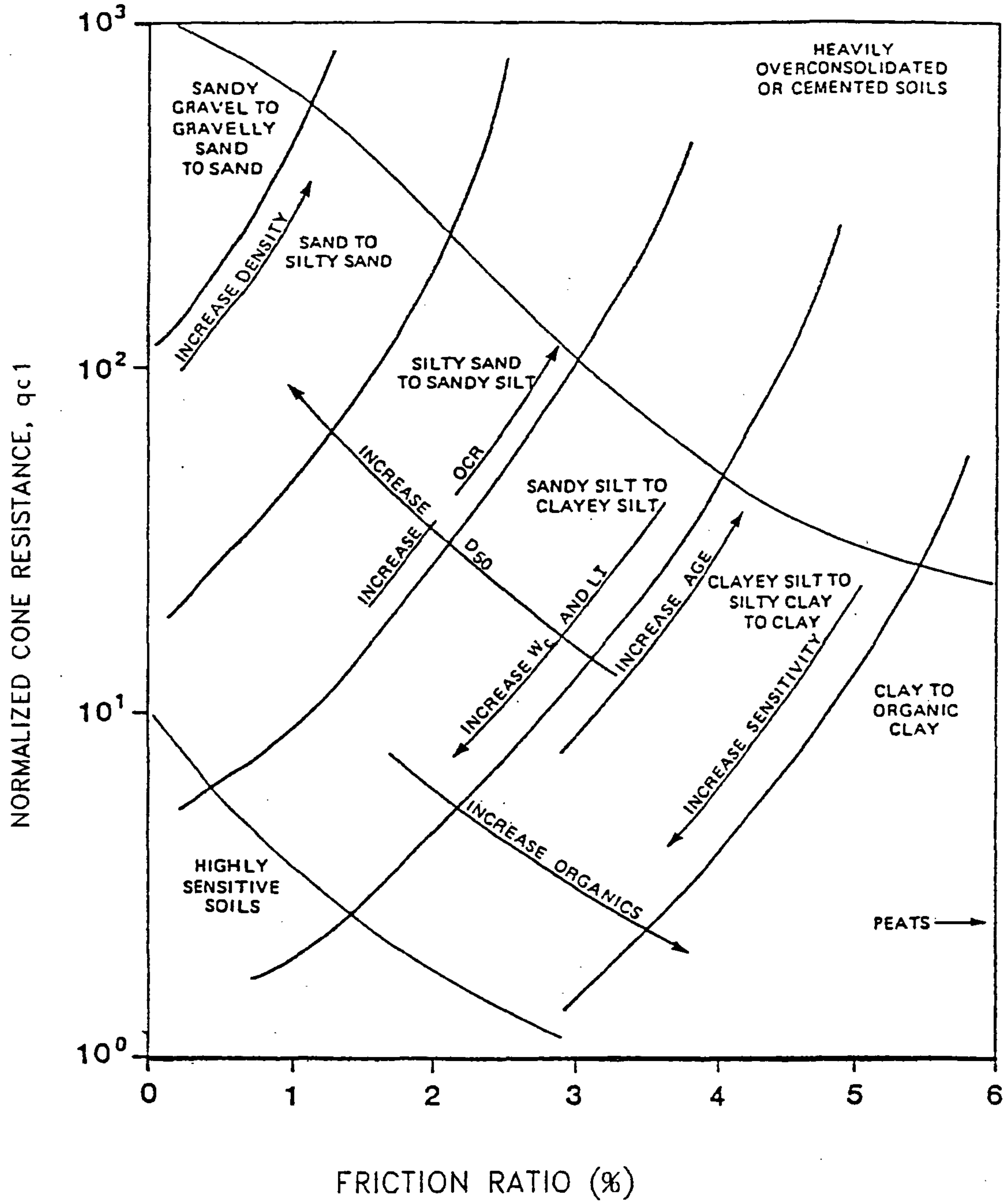


SOIL BEHAVIOR TYPE CLASSIFICATION CHART

After Douglas and Olsen, 1981

STRATIGRAPHICS

Figure 2

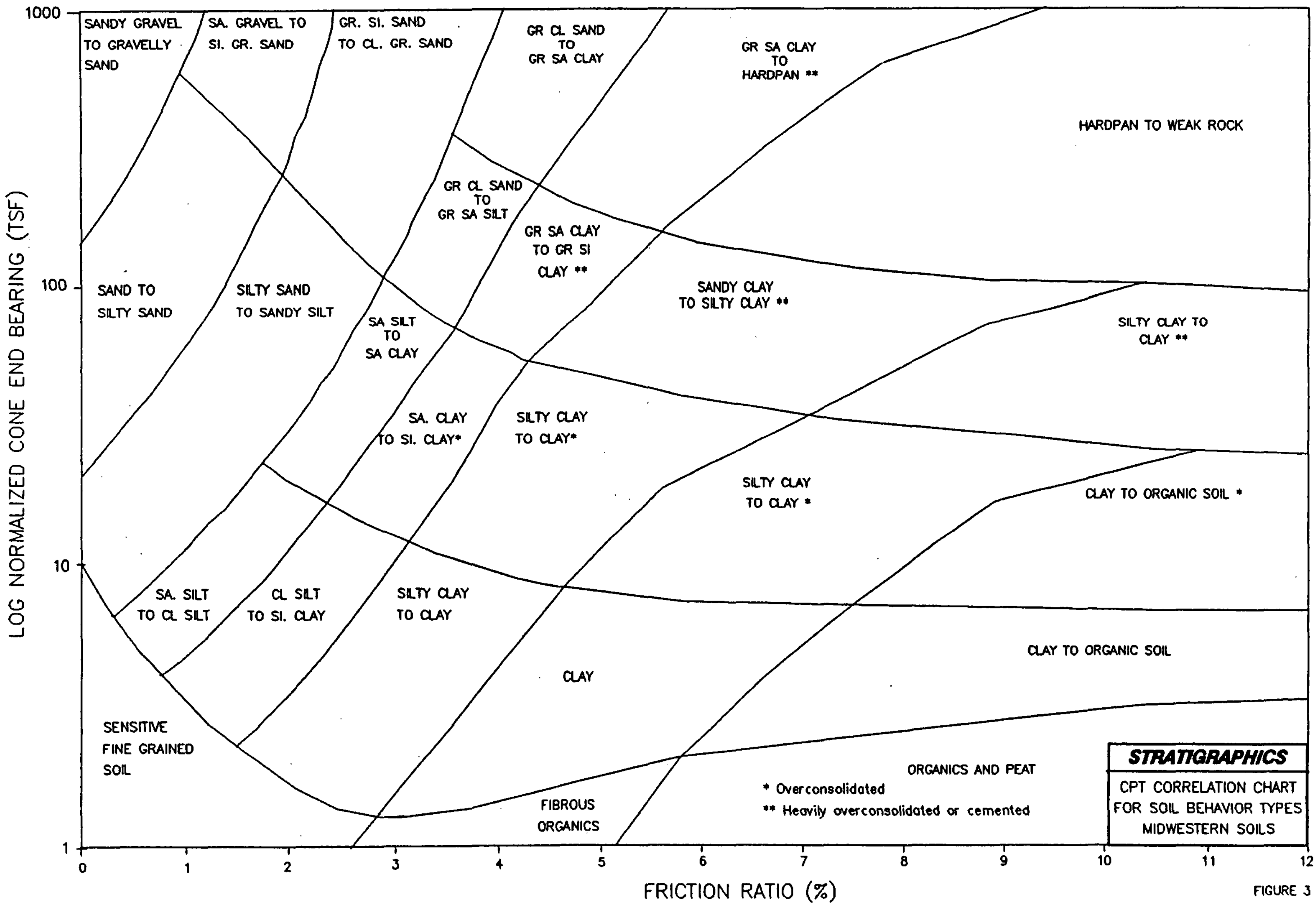
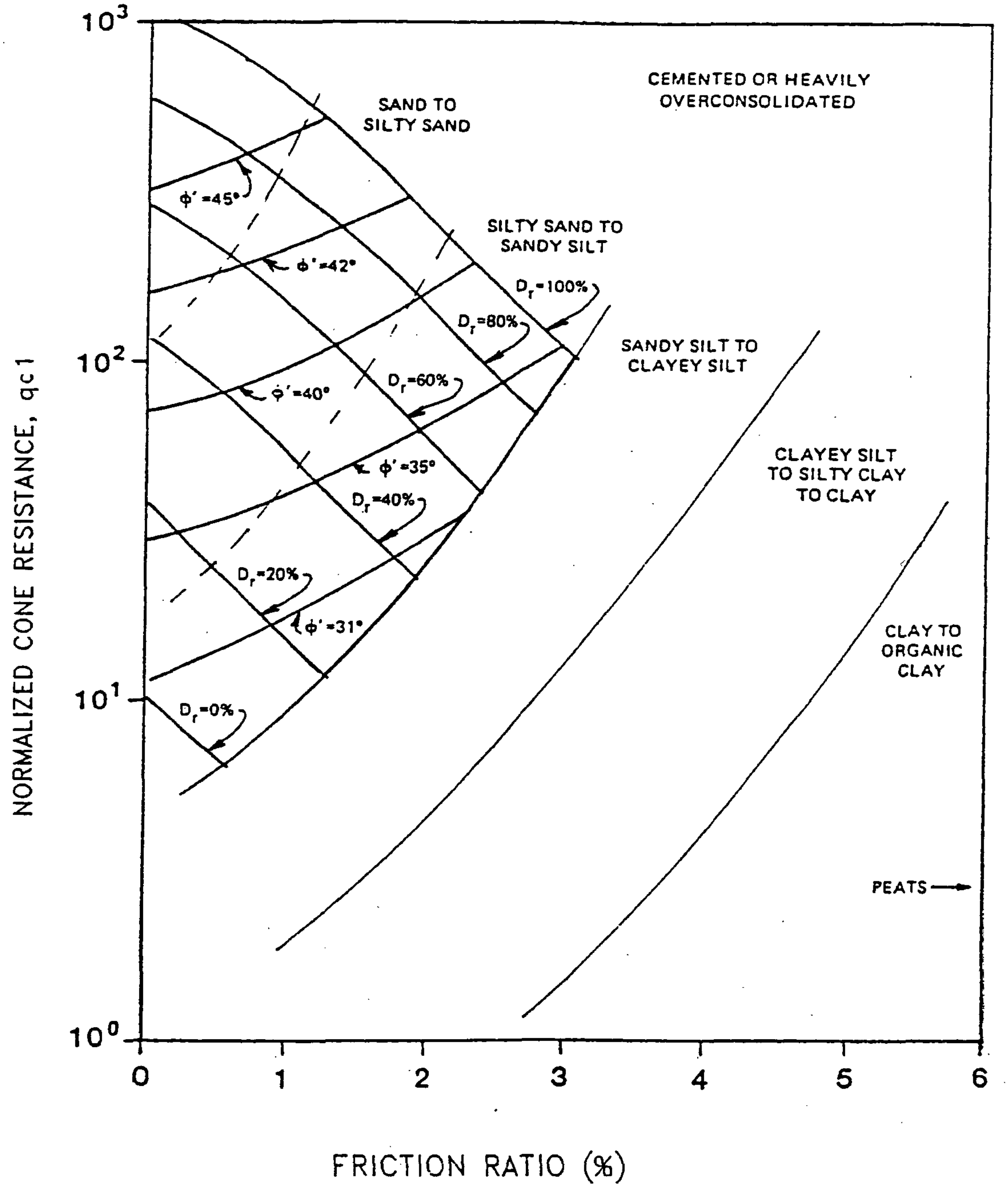

STRATIGRAPHICS
CPT CORRELATION CHART FOR SOIL BEHAVIOR TYPES MIDWESTERN SOILS
LOG NORMALIZED CONE END BEARING (TSF)
FRICTION RATIO (%)
1000
100
10
1
0
1
2
3
4
5
6
7
8
9
10
11
12
SANDY GRAVEL TO GRAVELLY SAND
SA. GRAVEL TO SI. GR. SAND
GR. SI. SAND TO CL. GR. SAND
GR CL SAND TO GR SA CLAY
GR SA CLAY TO HARDPAN **
HARDPAN TO WEAK ROCK
GR CL SAND TO GR SA SILT
GR SA CLAY TO GR SI CLAY **
SAND TO SILTY SAND
SILTY SAND TO SANDY SILT
SA SILT TO SA CLAY
SANDY CLAY TO SILTY CLAY **
SILTY CLAY TO CLAY **
SA. CLAY TO SI. CLAY*
SILTY CLAY TO CLAY*
SILTY CLAY TO CLAY *
CLAY TO ORGANIC SOIL *
SA. SILT TO CL SILT
CL SILT TO SI. CLAY
SILTY CLAY TO CLAY
CLAY
CLAY TO ORGANIC SOIL
SENSITIVE FINE GRAINED SOIL
FIBROUS ORGANICS
ORGANICS AND PEAT
* Overconsolidated
** Heavily overconsolidated or cemented

FIGURE 3

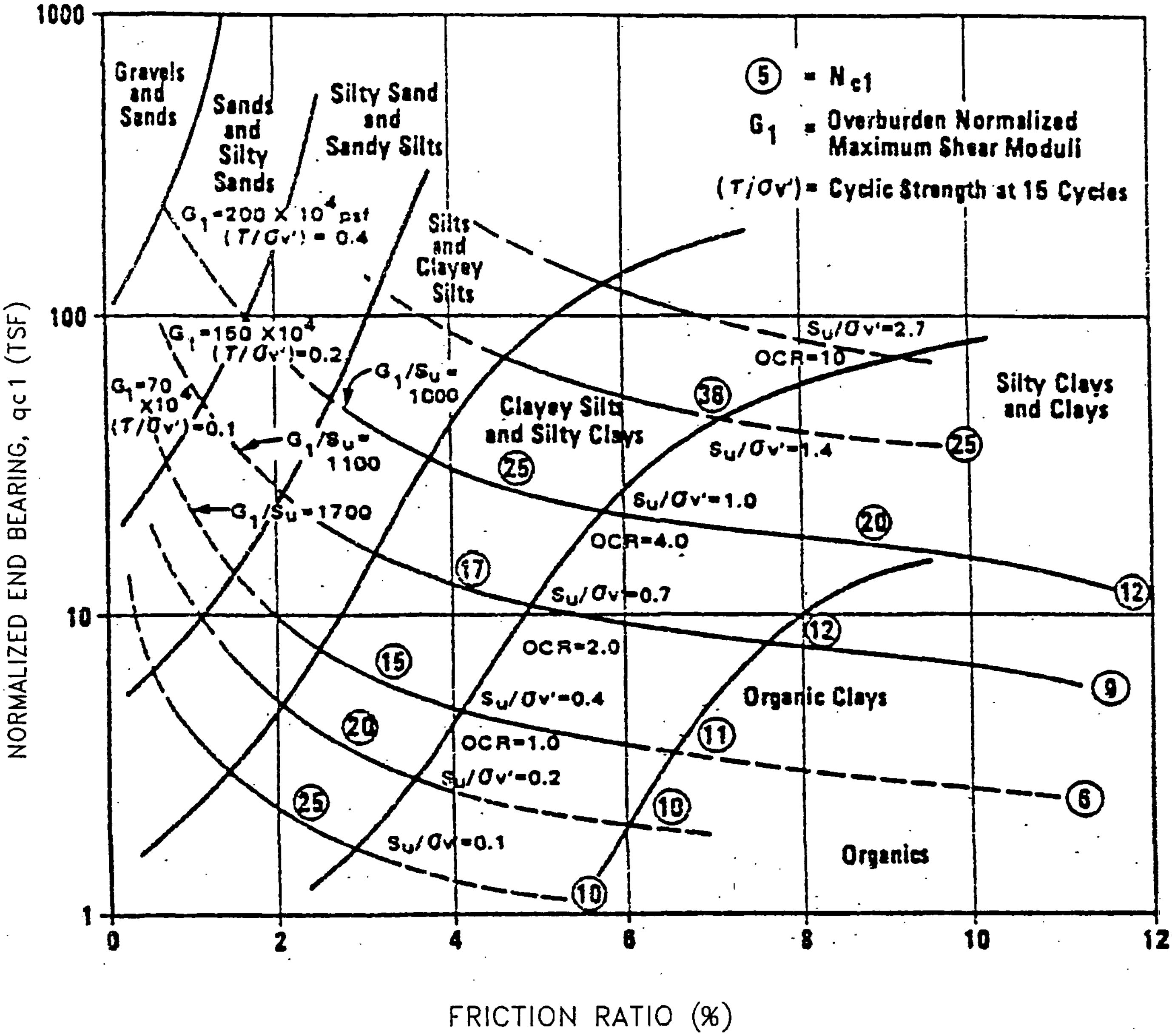


EXPANDED SOIL BEHAVIOR TYPE CLASSIFICATION CHART WITH EQUIVALENT OVERBURDEN NORMALIZED FRICTION ANGLE AND RELATIVE DENSITY TRENDS

After Douglas and Strutynsky, 1984

STRATIGRAPHICS

Figure 4



COMPOSITE TRENDS IN UNDRAINED SOIL PROPERTIES

After Douglas, Strutynsky, et. al., 1985

STRATIGRAPHICS

Figure 5

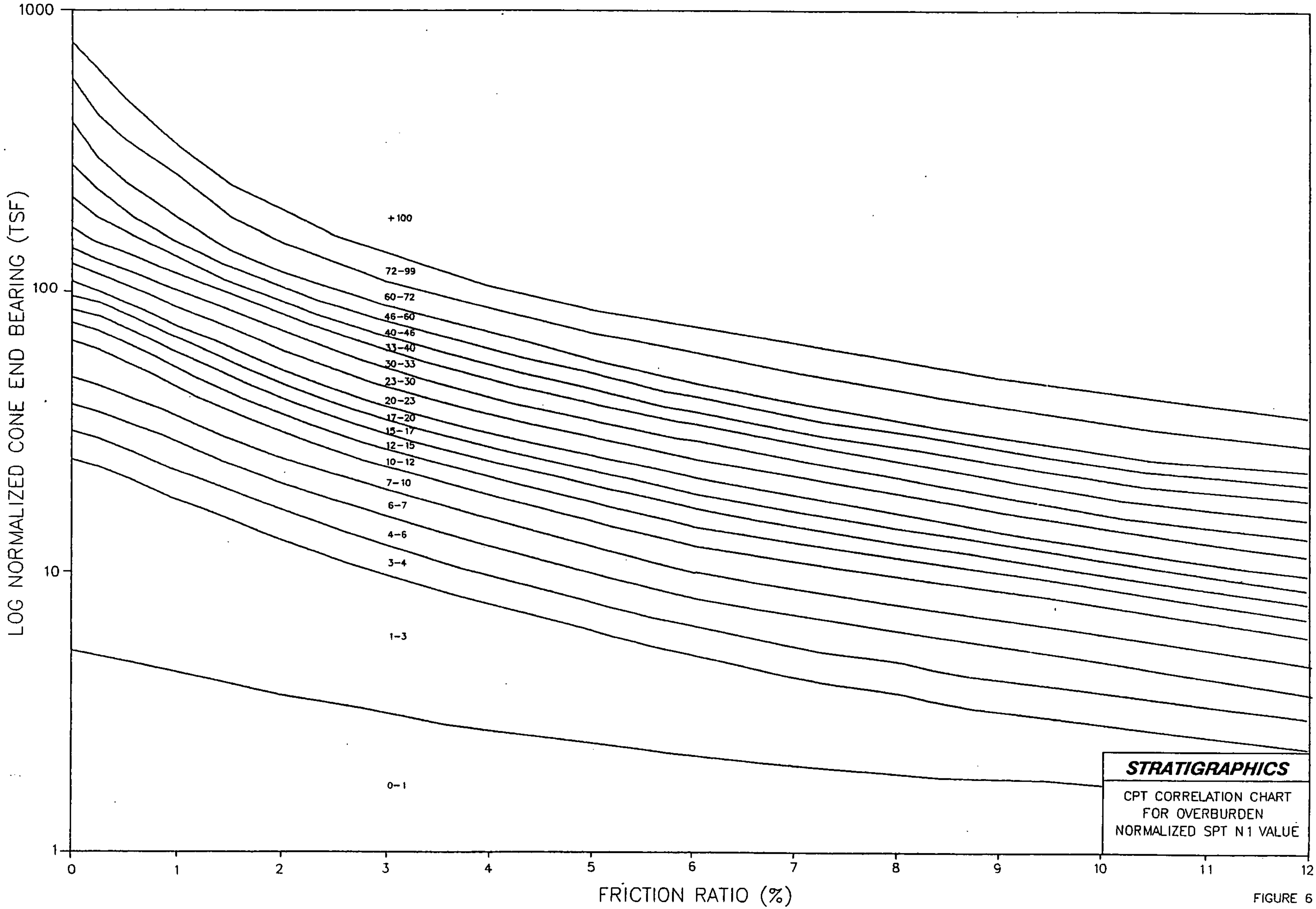
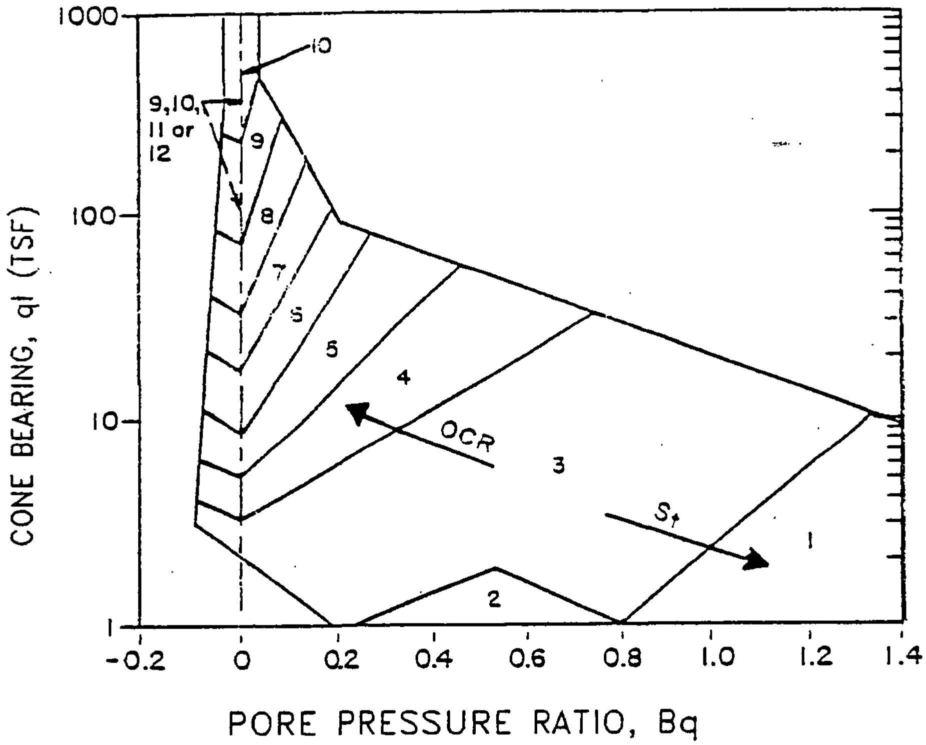

LOG NORMALIZED CONE END BEARING (TSF)
1000
100
10
1
+100
72–99
60–72
46–60
40–46
33–40
30–33
23–30
20–23
17–20
15–17
12–15
10–12
7–10
6–7
4–6
3–4
1–3
0–1
0
1
2
3
4
5
6
7
8
9
10
11
12
FRICTION RATIO (%)
STRATIGRAPHICS
CPT CORRELATION CHART
FOR OVERBURDEN
NORMALIZED SPT N1 VALUE

FIGURE 6



ZONE	SOIL BEHAVIOR TYPE
1	ORGANIC MATERIAL
2	CLAY
3	CLAY
4	SILTY CLAY TO CLAY
5	CLAYEY SILT TO SILTY CLAY
6	SANDY SILT TO CLAYEY SILT
7	SILTY SAND TO SANDY SILT
8	SAND TO SILTY SAND
9	SAND
10	GRAVELLY SAND TO SAND
11	VERY STIFF FINE GRAINED (*)
12	SAND TO CLAYEY SAND (*)
(*)	OVERCONSOLIDATED TO CEMENTED

After Robertson & Others, 1986

STRATIGRAPHICS

Figure 7

APPENDIX B

Excerpt from Baligh, M.M. and J. Levadoux, "Pore Pressure Dissipation After Cone Penetration," Department of Civil Engineering, Massachusetts Institute of Technology, Cambridge, Massachusetts, 1980.

6.2.4 Evaluation of c_h (probe)

At a given degree of consolidation, the predicted horizontal coefficient of consolidation c_h (probe) is obtained from the expression

$$c_h \text{ (probe)} = R^2T/t \qquad (6.2)$$

where R is the radius of the cone shaft, t is the measured time to reach this degree of consolidation; and T is the time factor. Table 5.1 provides values of T for different probe types at various degrees of consolidation.

An analytical method {equivalent to the graphical method described in Section 6.2.3} to check the validity of the prediction method consists of determining c_h at different dissipation stages, i.e., different u. Large differences between c_h at various degrees of consolidation indicate an inadequate initial distribution of excess pore pressure or significant coupling, or creep behavior.

The estimated values of c_h (probe) at 50% dissipation can be used in foundation problems involving horizontal water flow due to unloading or reloading of clays above the maximum past pressure. For problems involving vertical water flow in the overconsolidated range, the vertical coefficient of consolidation, c_v(probe), can be estimated from the expression:

$$c_v \text{ (probe)} = (k_v/k_h)\ c_h \text{ (probe)} \qquad (6.3)$$

where k_v and k_h are the vertical and horizontal coefficients of permeability, respectively. Reliable estimates of the in situ anisotropy of clays as expressed by the ratio k_h/k_v is difficult to determine in the laboratory because of the effects of sample size, sample disturbance, ... etc. and is the subject of controversy (Rowe, 1972; Casagrande and Poulos, 1969). In situ tests to determine k_h/k_v are almost nonexistent. Table 6.2 provides rough estimates of k_h/k_v for different clays.

6.2.5 Prediction of k_h (probe)

Approximate estimates of the horizontal coefficient of permeability, k_h (probe), can be obtained from the expression:

$$k_h \text{ (probe)} = (g_w/2.3s_{vo}) * RR\text{(probe)} * c_h \text{ (probe)} \qquad (6.4)$$

where s_{vo} is the initial vertical effective stress (kg/cm^2); g_w is the unit weight of water (=10^{-3} kg/cm^3); and, RR(probe) is the recompression ratio during early stages of consolidation around the probe (50% dissipation, say).

Results in both the upper and lower Boston Blue Clays indicate that:

$$\text{the average RR(probe)} = 10^{-2} \qquad (6.5)$$

$$\text{and generally } 0.5 * 10^{-2} < \text{RR(probe)} < 2 * 10^{-2} \qquad (6.6)$$

6.2.6 Prediction of c_v(NC)

For foundation clays consolidated in the normally consolidated range, estimates of the coefficients of consolidation can be obtained from c_h (probe) by means of the expressions:

$$c_h(NC) = (RR(probe)/CR) * c_h(probe) \qquad (6.7)$$

for horizontal water flow, and

$$c_v(NC) = (RR(probe)/CR) * (k_v/k_h) * c_h(probe) \qquad (6.8)$$

for vertical water flow.

The compression of ratio CR is the average slope of the strain vs. log effective stress plot in the appropriate effective stress range expected during consolidation of the foundation clay. Values of CR should be obtained from good quality samples carefully tested in the laboratory. Table 6.2 provides rough estimates of CR based on empirical correlation with index properties of various clays.

Table 6.2 Empirical Correlation and Typical Properties of Clays

1. Compression Ratio CR (from Ladd, 1973)

CR = $C_c/1+e_o$ = slope of the strain vs. log stress curve	
e_o = initial void ratio	
c_c = virgin compression index = slope of e vs. log stress	
w_L = liquid limit	
w_N = natural water content	
$c_c = 0.009 (w_L\% - 10\%)$	Terzaghi and Peck (1967)
$C_c = 0.54 (e_o - 0.35)$	Nishida (1958)
C_c = 0.01 to 0.15 (w_N%)	MPMR (1958)
$C_c = 0.6 (e_o - 1)$ for $e_o < 6$	
$C_c = 0.85 (e_o - 2)$ for $6 < e_o < 14$	Kapp, (1966)

2. Anisotropic Permeability of Clays (from Ladd, 1976)

Nature of Clay	k_h/k_v
1. No evidence of layering	1.2 +- 0.2
2. Slight layering, e.g., sedimentary clays with occasional silt dustings to random lenses	2 to 5
3. Varved clays in northeastern U.S.	10 +/-5

APPENDIX C

Cone Penetration Testing in Geotechnical Practice

Tom Lunne
Peter K. Robertson
John J.M. Powell

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may be liable for serious error, especially when based on general empirical correlations. Conceptually, total stress undrained measurements from a CPT are difficult to correlate to drained parameters without the addition of pore pressure measurements. The prediction of consolidation deformation parameters based on cone resistance may be in error by as much as ± 100%. However, with local experience individual site-specific or area-specific correlations (as indicated above) can be developed for certain soil types with greater reliability.

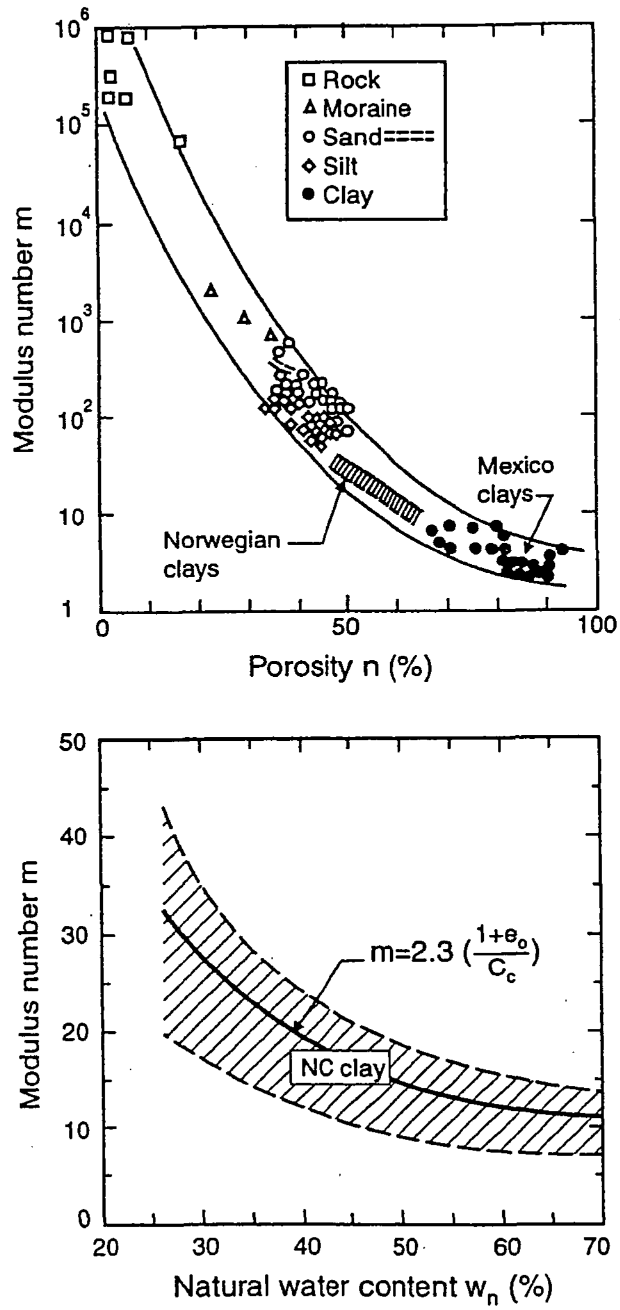


Figure 5.31 (a) Trend of modulus number m with porosity; (b) Trend of modulus number m with water content (Janbu, 1963).

5.4.3.2 Undrained Young's modulus

The estimation of undrained Young's modulus, E_u, is usually made using empirical correlations with the undrained shear strength, s_u, in the form:

$$E_u = n \cdot s_u \tag{5.29}$$

where n is a constant that depends on shear stress level, overconsolidation ratio, clay sensitivity and other factors (Ladd *et al.* 1977). As discussed earlier, because soil behaviour is non-linear, the choice of relevant shear stress level is very important. Figure 5.35a presents data for normally consolidated soils from Ladd *et al.* (1977) that shows the variation of the E_u/s_u with stress level for seven different cohesive soils, ($15 < I_P < 75$). Figure 5.35b shows the variation of E_u/s_u with overconsolidation ratio (OCR) at two shear stress levels for the same soil types shown in Figure 5.35a. Figure 5.36 shows the variation of stiffness ratio at 25% of the failure stress with OCR as proposed by Duncan and Buchignani (1976).

The recommended procedure for estimation of the undrained Young's modulus, E_u, is to first estimate the undrained shear strength, s_u, from CPT/CPTU profiles, as previously discussed in section 5.5.2.1, then estimate the

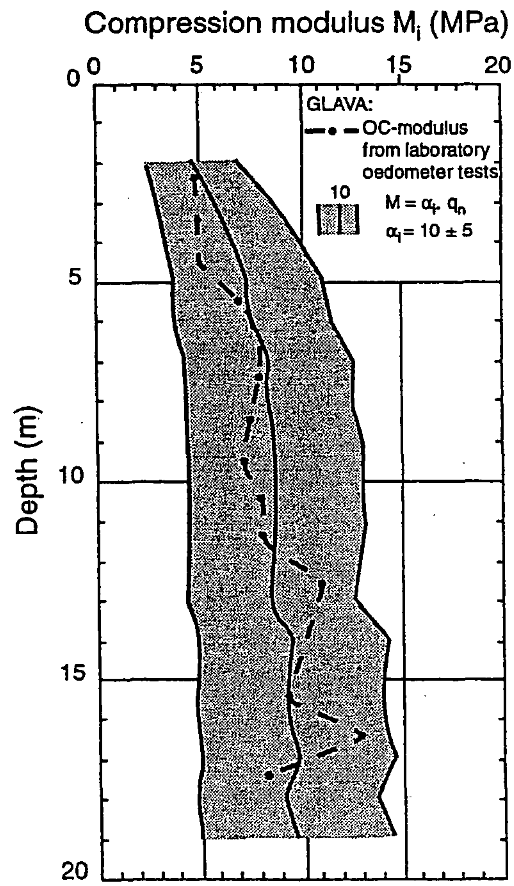


Figure 5.32 Compression modulus M_i for Glava clay (from Senneset *et al.*, 1989).

stress history (OCR). Then, using Figure 5.35, estimate E_u for the relevant shear stress level appropriate to the particular problem. A knowledge of the plasticity index (I_P) would significantly improve the estimate.

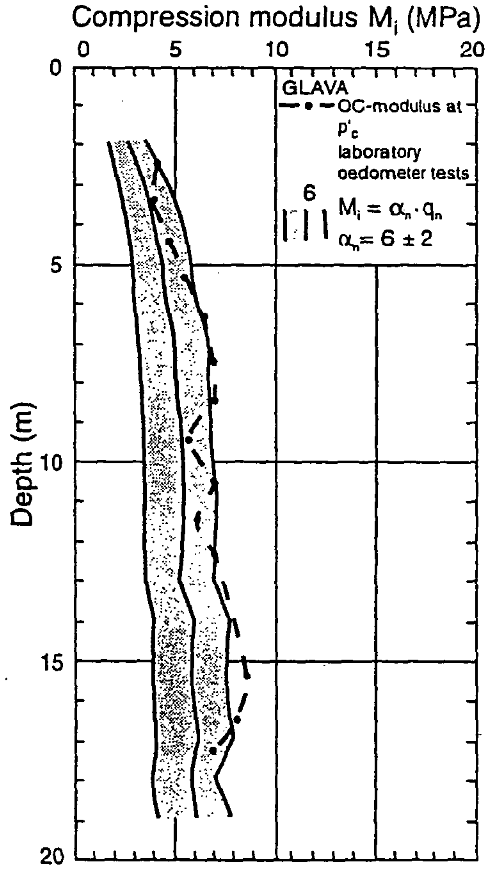


Figure 5.33 Compression modulus M_n for Glava clay (from Senneset *et al.*, 1989).

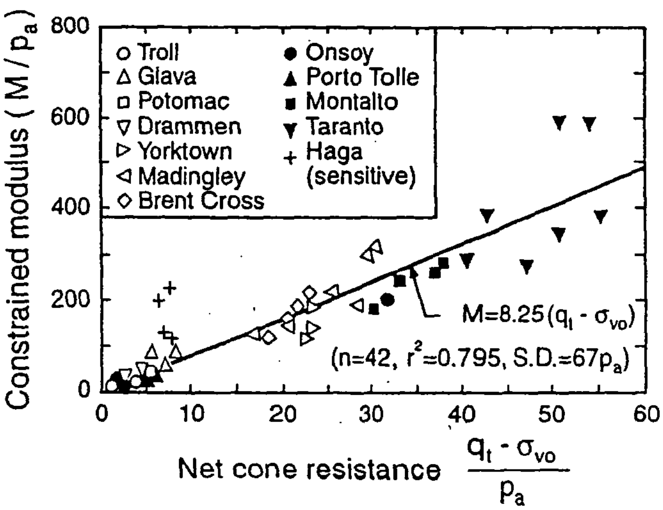


Figure 5.34 General relationship between constrained modulus and net cone resistance (from Kulhawy and Mayne, 1990).

5.4.3.3 Small strain shear modulus

The shear modulus is largest at very low strains and decreases with increasing shear strain. It has generally been found that the initial maximum shear modulus is constant for shear strains less than about 10^{-3}%. This initial, small strain modulus is often denoted G_o.

Mayne and Rix (1993) showed that the small strain shear modulus varied with void ratio (e) and cone penetration resistance (q_t) for a wide range of clays and can be expressed as:

$$G_o = 99.5\,(p_a)^{0.305}\,\frac{(q_t)^{0.695}}{(e_o)^{1.130}} \tag{5.30}$$

where:

p_a = atmospheric reference stress in the same units as G_o and q_t.

The strong dependence of G_o upon void ratio (e) requires that CPT q_c is only successful as a profiler of G_o if comparison profiles of e_o are known. This is not usually the case. However, elastic theory relates the maximum shear modulus, G_o, to soil density, ρ, and shear wave velocity, V_s, as follows:

$$G_o = \rho \cdot V_s^2 \tag{5.31}$$

where:

ρ = mass density of the soil $= \gamma/g$

and this supports the recommendation of making direct measurements of *in situ* shear wave velocity using the seismic CPT (see section 7.4).

Based on these observations, Robertson *et al.* (1995) suggested a chart to identify soil type using seismic CPT data, as shown in Figure 5.10. This chart can also be used to estimate G_o based on an estimate of soil type from the basic CPT soil classification charts.

However, care must always be taken when using any of these charts or correlations as it should be remembered that G_o is not independent of the direction of shear (Powell and Butcher, 1991). Butcher and Powell (1995a) showed that the shear wave velocity in clays, and therefore the G_o value deduced, was dependent on the stresses in the directions of propagation and polarization of the shear waves and can vary by up to 300% in heavily overconsolidated clays.

5.4.4 Flow and consolidation characteristics

Flow and consolidation characteristics of soil are normally expressed in terms of the coefficient of consolidation, c, and hydraulic conductivity or permeability, k. They are interlinked through the formula:

$$c = k \cdot \frac{M}{\gamma_w} \tag{5.32}$$

where M is the constrained modulus relevant to the problem modelled (that is, unloading, reloading, virgin loading).

The parameters c and k vary over many orders of magnitude and are some of the most difficult parameters to measure in geotechnical engineering. It is often considered that an accuracy within one order of magnitude is acceptable. Nevertheless, c and k are parameters that are often essential input in some geotechnical calculations.

Due to soil anisotropy both c and k have different values in the horizontal (c_h, k_h) and vertical (c_v, k_v) direction. The relevant design values depend on drainage and loading direction.

5.4.4.1 Coefficient of consolidation

Rate of consolidation parameters may be assessed from the piezocone test by measuring the dissipation or decay of pore pressure with time after a stop in penetration.

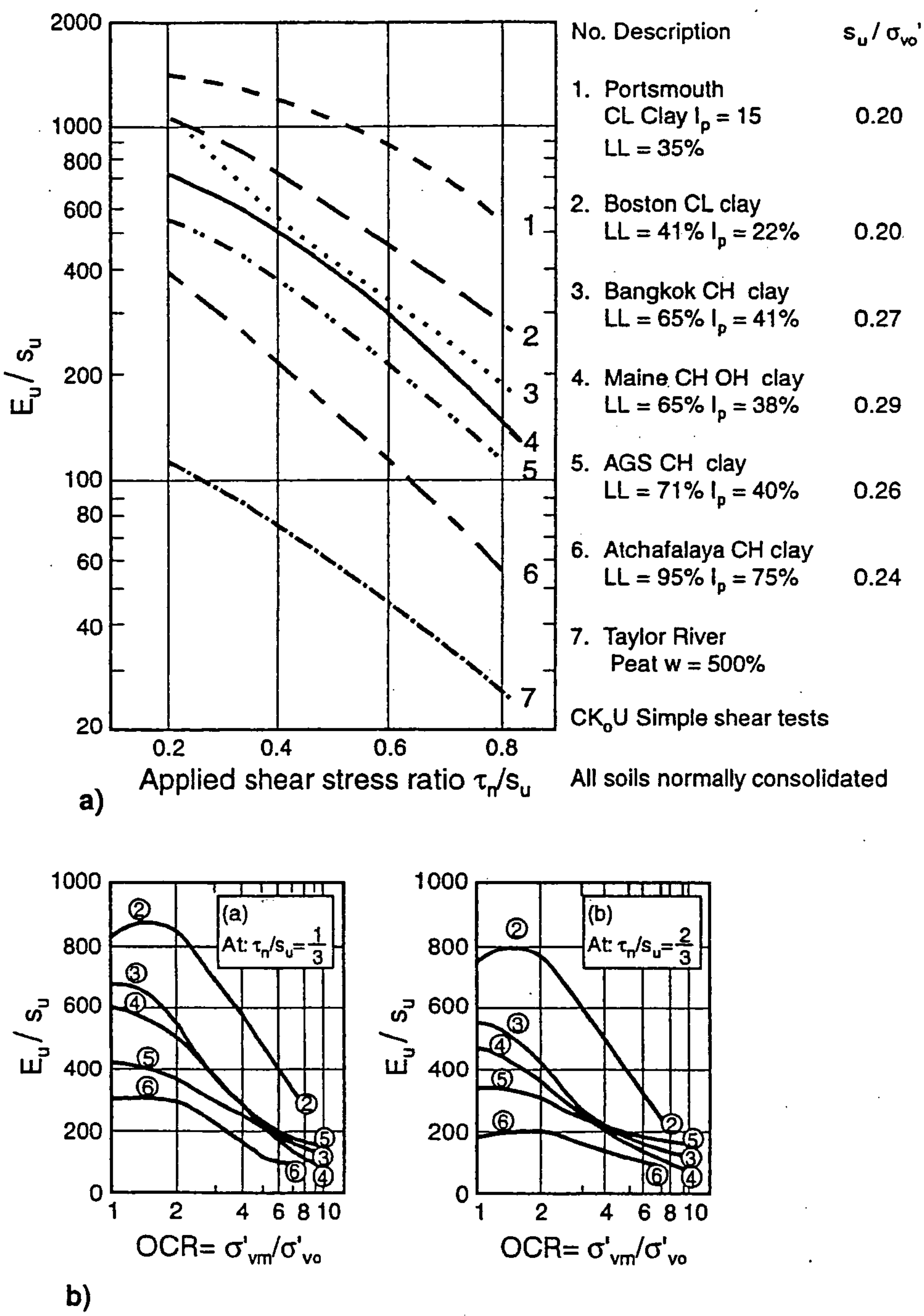


Figure 5.35 Stiffness ratio, E/s_u, as function of I_p (adapted from Ladd *et al.*, 1977).

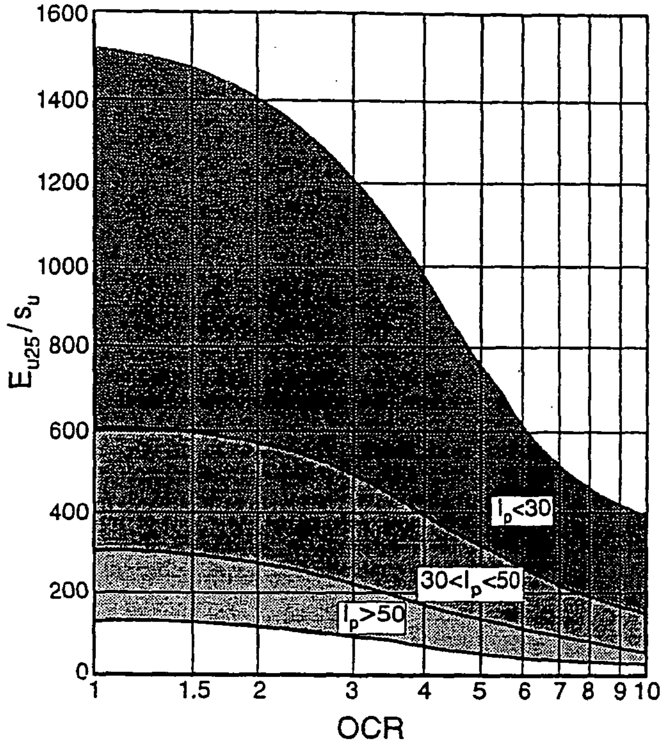


Figure 5.36 Stiffness ratio, E/s_u, as function of OCR (after Duncan and Buchignani, 1976).

Figure 5.37a shows typical dissipation curves for a soft clay (Bothkennar clay) plotted on a logarithmic time scale. The results vary with the filter position. For interpretation it is best to normalize the pore pressure relative to the initial pore pressure at the beginning of dissipation, u_i, and the equilibrium *in situ* pore pressure u_o. The normalized excess pore pressure, U, at time t, is thus expressed as:

$$U = \frac{u_t - u_o}{u_i - u_o} \tag{5.33}$$

where:

u_t = the pore pressure at time t

u_i = initial pore pressure at $t = 0$

u_o = *in situ* pore pressure before penetration.

The results of Figure 5.37a are replotted in normalized form in Figure 5.37b.

Over the last 10 to 15 years, theoretical and semi-empirical solutions have been developed for deriving the coefficient of consolidation from pore pressure dissipation data.

Table 5.9 presents an overview of the main solutions available to calculate the coefficient of consolidation from piezocone dissipation data.

Torstensson (1975, 1977) developed an interpretation model based on cavity expansion theories. Initial pore pressures were computed assuming an elasto-plastic soil model and spherical or cylindrical cavity expansion theory, as shown in Table 5.9. Torstensson then used linear uncoupled one-dimensional consolidation to compute the dissipation of pore pressures.

Torstensson suggested that the coefficient of consolidation should be interpreted at 50% dissipation from the following formula:

$$c = \frac{T_{50}}{t_{50}} \cdot r_o^2 \tag{5.34}$$

where the time factor T_{50} is found from the theoretical solutions, t_{50} is the measured time for 50% dissipation and r_o = penetrometer radius (cylindrical model) or equivalent penetrometer radius for spherical model.

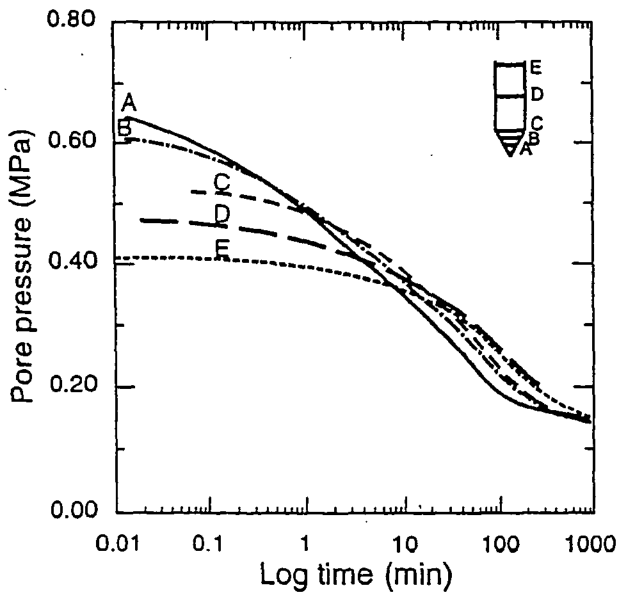


Figure 5.37a Dissipation test results from Bothkennar.

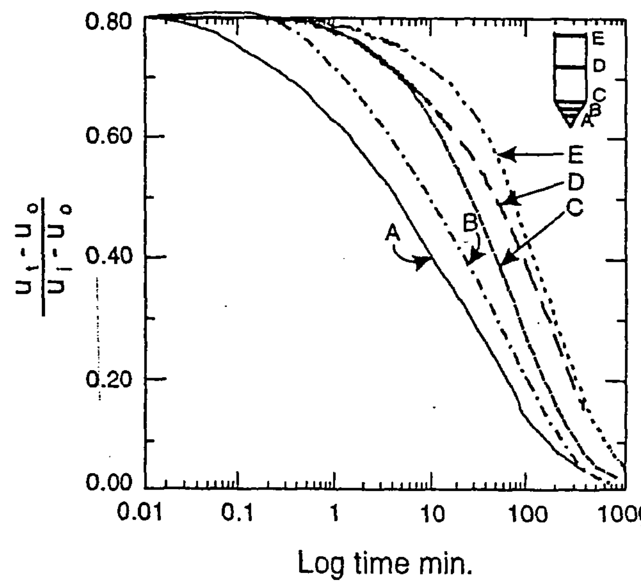


Figure 5.37b Normalized dissipation test data from Bothkennar.

5.5.3 Deformation characteristics

A reliable determination of sand stiffness *in situ* is of great practical interest in view of the difficulties in obtaining deformation modulus from tests on laboratory specimens. Undisturbed samples are extremely difficult to obtain and often more or less impossible using conventional techniques in cohesionless soils.

Obtaining reliable estimates of soil stiffness from reconstituted samples is considered far less likely than obtaining reliable large strain strength parameters.

It is difficult to obtain reliable estimates of sand stiffness from *in situ* penetration tests because (1) modulus depends on effective stress and on stress history, (2) the *in situ* test conditions, stress level, drainage and direction of loading cannot be controlled and (3) reference modulus values are rarely or seldom documented. Any modulus value should refer to the drainage conditions, stress level, and strain level over which the modulus is applicable.

5.5.3.1 Young's modulus

Research using calibration chamber data has shown that the drained Young's modulus in sand mainly depends on relative density, overconsolidation ratio and current mean stress level.

Figure 5.59 presents a chart to estimate the secant Young's modulus (E_s') for an average axial strain of 0.1% for

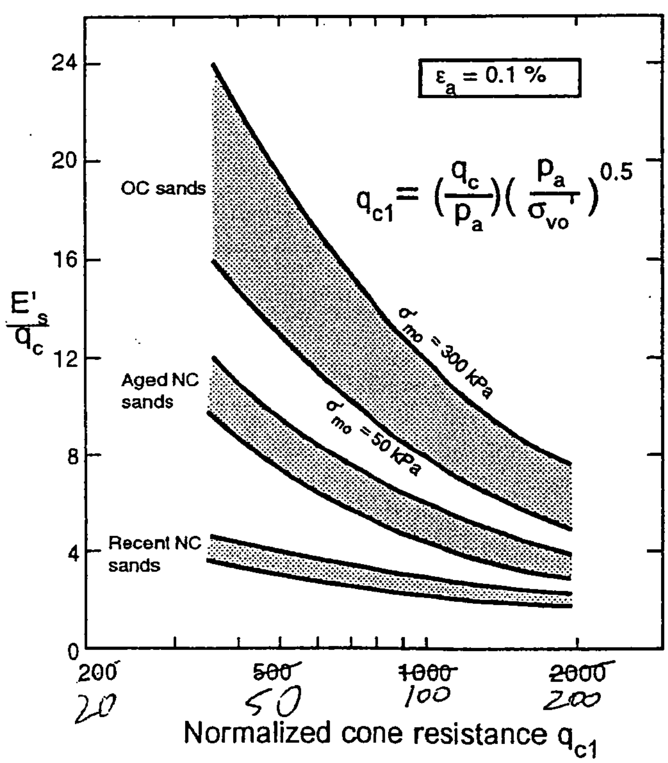


Figure 5.59 Evaluation of drained Young's modulus from CPT for silica sands (from Bellotti *et al.*, 1989).

a range of stress histories and ageing. This level of strain is reasonably representative for many well-designed foundations. The stiffness of normally consolidated aged sands (> 1000 years) appears to fall between that of very recent normally consolidated sands and overconsolidated sands.

Robertson (1989) suggested a modified correlation for shallow foundations based on the degree of loading (section 6.3.2).

5.5.3.2 Constrained modulus

Most correlations between CPT results and the drained constrained modulus, M, refer to the tangent modulus, as found from oedometer tests. The reference value of M is normally based on the effective vertical stress, σ'_{vo}, before the start of the *in situ* test; this value is denoted M_o.

Based on a review of available calibration chamber tests, Lunne and Christophersen (1983) recommended that an estimate of M_o for NC unaged and uncemented predominantly silica sands can be obtained from:

$M_o = 4q_c$ for $q_c < 10$ MPa

$M_o = 2q_c + 20$ (MPa) for 10 MPa $< q_c <$ 50 MPa

$M_o = 120$ MPa for $q_c > 50$ MPa

Lunne and Christophersen also included OC sands in their study and recommended as a rough guideline to use:

$M_o = 5q_c$ for $q_c < 50$ MPa

$M_o = 250$ MPa for $q_c > 50$ MPa

For an additional stress $\Delta\sigma'_v$, Lunne and Christophersen recommended Janbu's (1963) formulation to compute M for the stress range σ'_{vo} to $\sigma'_{vo} + \Delta\sigma'_v$:

$$M = M_o\sqrt{\frac{\sigma'_{vo} + \Delta\sigma_v/2}{\sigma'_{vo}}} \tag{5.57}$$

Recently Eslaamizaad and Robertson (1996) suggested an alternative method to estimate M_o from CPT results, based on assessment of extensive calibration tests on quartz sands (Baldi *et al.*, 1986 and Fioravante *et al.*, 1991). Curve fitting techniques were used to develop correlations for sands with different stress history.

The method presents a correlation incorporating normalized cone resistance and normalized vertical effective stress in the form of:

$$\mathrm{M_o} = k_M p_a \left(\frac{\sigma'_{vo}}{p_a}\right)^n \tag{5.58}$$

where

n = stress exponent equal to 0.200 for normally consolidated sands, and 0.128 for overconsolidated sands

p_a = atmospheric pressure, in same units as σ'_{vo}, M_o and q_c

k_M = dimensionless modulus number which can be determined using Fig. 5.60, based on normalized cone penetration resistance (q_c/p_a) and estimated overconsolidation ratio (OCR).

This method has the advantage that a prior knowledge of relative density is not required. On average, the predicted M_o is between 75 to 125 per cent of the corresponding value measured in the calibration chamber test.

5.5.3.3 Small strain shear modulus

The shear modulus is largest at very low strains and decreases with increasing shear strain. It has generally been found that the maximum shear modulus is constant for shear strains less than 10^{-3}%. This initial, small strain modulus is often denoted G_o. Elastic theory relates the small strain shear modulus, G_o, to soil density, ρ, and shear wave velocity, V_s, as follows:

$$G_o = \rho V_s^2 \tag{5.59}$$

where ρ = mass density of the soil = γ/g.

Therefore, G_o can be found by measuring the shear wave velocity using the seismic CPT (Robertson *et al.*, 1986). Alternatively, G_o can be estimated using empirical correlations.

Jamiolkowski *et al.* (1988) showed that the same variables of soil density and *in situ* effective stresses controls both q_c and G_o. Hence, a correlation between q_c and G_o can be found for uncemented and unaged cohesionless soils. Cementation and ageing have different influences over q_c and G_o (Figure 5.10). Compressibility can have a significant influence on the correlation between G_o and q_c. Based on calibration chamber results as well as field measurements, Rix and Stokoe (1992) suggest the correlation shown in Figure 5.61. The wide range of G_o/q_c at low values of normalized cone resistance is most likely due to variations in soil compressibility. More compressible sands appear to produce lower values of normalized cone resistance and hence higher values of G_o/q_c.

A similar relationship was shown in Figure 5.10.

5.6 AVAILABLE EXPERIENCE AND INTERPRETATION IN OTHER MATERIAL

Most references and interpretation methods are related to either sand (fully drained conditions) or clay (fully undrained conditions). However, very frequently other soil types (for example, those with partial drainage during penetration) are encountered and CPTs are also used in materials such as, silt, peat, mine tailings, permafrost and so on.

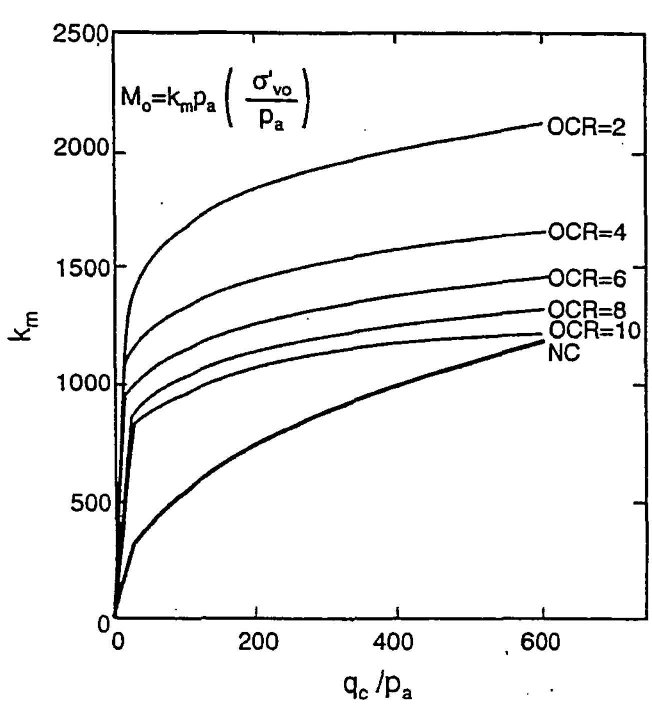


Figure 5.60 Constrained modulus number of sand as function of cone resistance and OCR (after Eslaamizaad and Robertson, 1996).

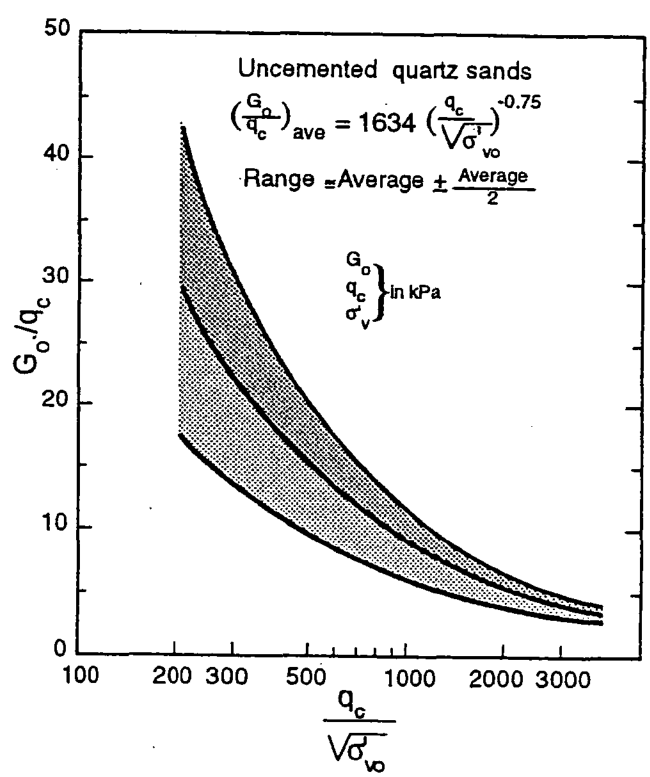


Figure 5.61 G_{max}/q_c (after Rix and Stokoe, 1992).

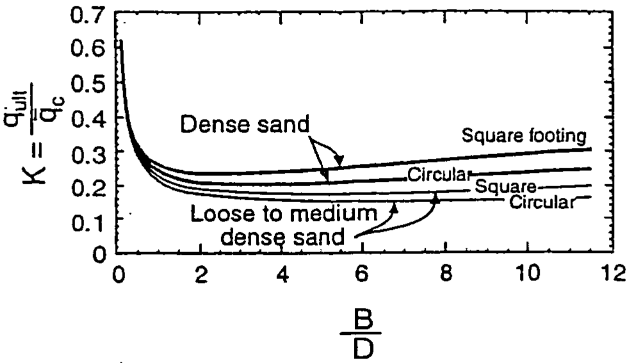


Figure 6.9 Correlation between bearing capacity of footing on cohesionless soils and average cone resistance.

Eslaamizaad and Robertston (1996) suggest the variation shown in Figure 6.9, where

$$q_{\mathrm{ult}} = \mathrm{K} \cdot \bar{q}_c \tag{6.15}$$

6.3.2 Settlement

Vertical settlements are usually the most common aspect of deformation. For shallow foundations on granular soils (sands, silty sands, gravels, sandy gravels), settlement is usually the controlling design factor, except for very narrow foundations (width $B < 1$ m). A quick estimate of settlements in sands can be made using the simple chart proposed by Burland *et al.* (1977) and shown in Figure 6.10. The upper limits are shown for dense sand and medium dense sand. Probable settlements can in each case be taken as half the upper limit, and maximum settlements will not normally exceed 1.5 times the probable value. The upper limit should be regarded as tentative, since much of the data in the upper zone related to very loose silty organic sands. Suggested CPT values are given to estimate sand density in Figure 6.10. More refined methods for estimating sand density taking stress level into account is given in section 5.5.1.

Meyerhof (1974) suggested a simple but conservative method to estimate the settlement of a footing on sand directly from the CPT penetration resistance:

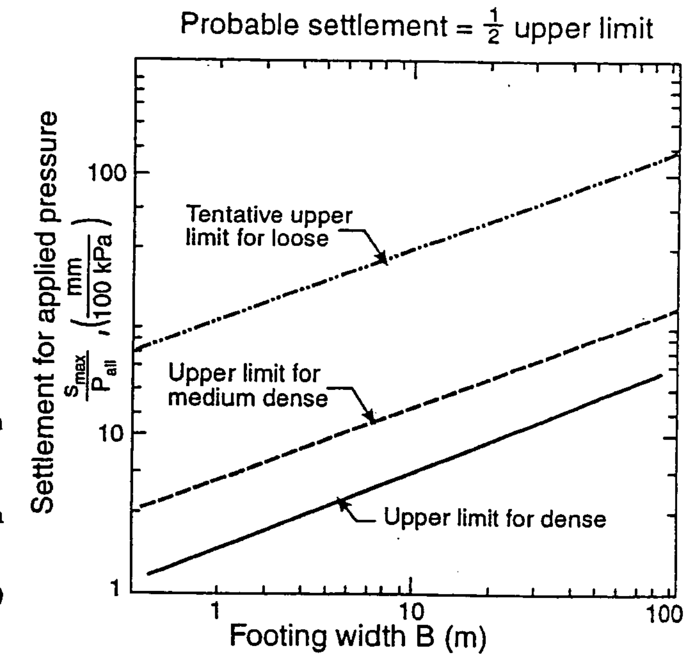
$$\text{settlement, } s = \frac{\Delta p B}{2\bar{q}_c} \tag{6.16}$$

where:

Δp = net foundation pressure.

The cone resistance ($\bar{q}_c$) is taken as the average over a depth equal to the width of the footing (B). This approach is roughly equivalent to using a Young's modulus, $E = 2q_c$.

Schmertmann (1970, 1978) developed a method to estimate the vertical settlement of footings on sand using the CPT. This method is based on the strain influence approach and elastic theory.



Sands	N_{SPT}	q_c (MPa)
Loose	< 10	< 5
Medium loose	10-30	5-15
Dense	> 30	> 15

Figure 6.10 Approximate range of settlement for footings on sand (Burland *et al.*, 1977).

The total vertical settlement is given by:

$$s = C_1 C_2 \Delta p \sum_{1}^{n} \frac{I_z}{C_3 E} \Delta z \tag{6.17}$$

where:

C_1 = correction for depth of embedment
C_2 = correction for creep
C_3 = correction for shape of footing
Δp = net foundation pressure (same units as q_c)
= foundation pressure (p) minus effective overburden pressure at the foundation level, σ'_1 (Figure 6.11)
I_z = strain influence factor
Δz = thickness of sublayer
E = Equivalent Young's modulus
$= \alpha q_c$

In this method (Figure 6.11), the sand is divided into a number of layers, n, of thickness, Δz, down to a depth below the base of the footings equal to $2B$ for a square footing and $4B$ for a strip footing (length of footing, $L > 10B$). A value of q_c is assigned to each layer, as illustrated on Figure 6.12. Strain within each layer is taken as $I_z \Delta p/E$, where $E = \alpha q_c$. Schmertmann (1978) suggested the following values for the shape factor correction, C_3:

For a square footing, $C_3 = 1.25$
For a strip footing ($L > 10B$), $C_3 = 1.75$

In the calculation, the strain distribution diagram is constructed such that the peak value of I_z is obtained from the following:

$$I_{zp} = 0.5 + 0.1\,(\Delta p/\sigma'_z)^{0.5} \tag{6.18}$$

Where σ'_z is the effective stress at the depth of the peak value of I_z. The values of I_z/E are divided by C_3 then summed, and multiplied by Δp, C_1 and C_2. For $1 < L/B < 10$, the results can be interpolated between the $L/B = 1$ and $L/B = 10$ cases.

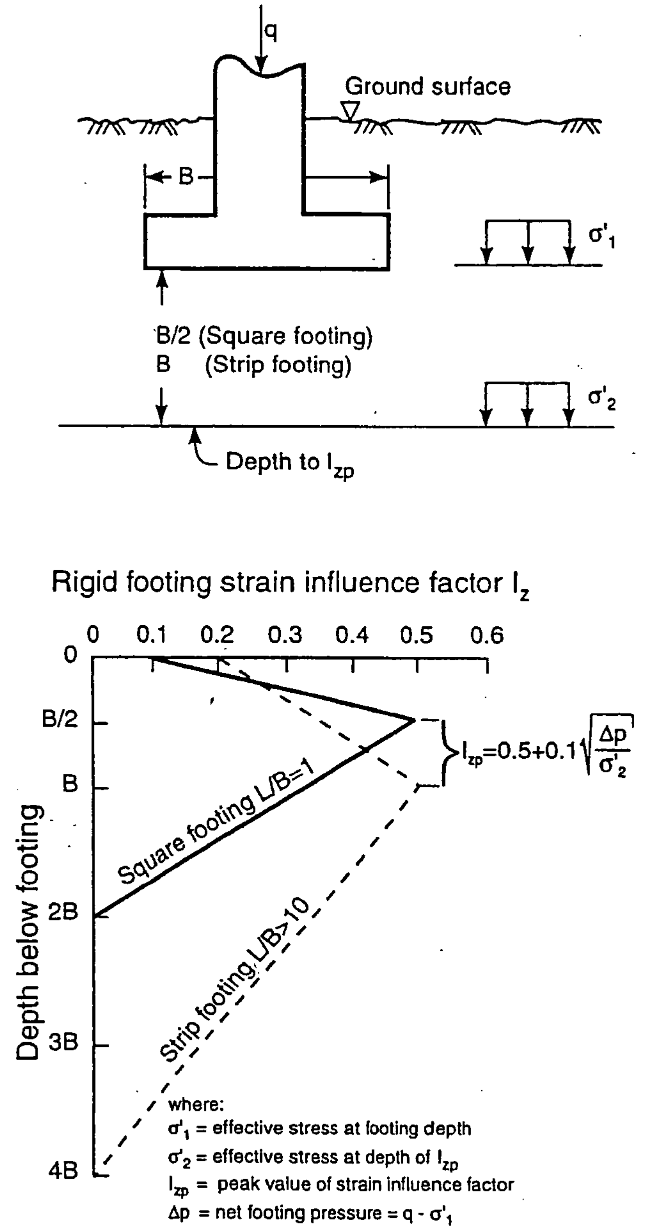


Figure 6.11 Strain Influence method for footings on sand (Schertmann, 1970).

The depth of embedment correction is:

$$C_1 = 1 - 0.5(\sigma'_{vo}/\Delta p)$$

with a value not less than 0.5.

The creep correction is:

$$C_2 = 1 - 0.2\log_{10}(10t)$$

where t = time in years from load application.

Schmertmann (1970) suggested that a value of $\alpha = 2$ should be applied for normally consolidated, unaged and uncemented predominantly quartz sands, and is based on a load increment from 100 to 300 kN/m^2. It is probable that somewhat higher α values may be appropriate for loose sands and somewhat lower values for very dense sands (section 5.5.3). Young's modulus, E, for either mechanically overconsolidated or aged sands can be significantly higher, but it is suggested that values not more than two or three times those for normally consolidated sands should be used. Caution should be exercised before increasing the E/q_c ratio for sands because of the uncertainty in estimating OCR for a sand.

Robertson (1991) suggested an alternate method to estimate the Equivalent Young's modulus from CPT results in sands based on the degree of loading, as shown on Figure 6.13. The degree of loading is the ratio of the applied foundation stress, q, to the calculated ultimate bearing capacity, q_{ult}. The ratio of E/q_c for overconsolidated sands is taken directly from Figure 6.13. Values for aged sands are reduced by a factor of 2 and for young normally consolidated sands by a factor of 3. The variation of estimated E as a function of the degree of loading is due to the variation in mobilized average strain beneath the footing. More details on how to estimate soil modulus in sands are given in Chapter 5.

Settlement for structures on fine-grained soils, such as clay, can be calculated from deformation moduli, which are not so readily estimated from CPT or piezocone data. However, section 5.4.3 includes some guidance in this respect. Piezocone dissipation tests may also be used to derive values of coefficient of consolidation, which can be used to calculate rate of settlement (section 5.4.4).

6.4 GROUND IMPROVEMENT – QUALITY CONTROL

Ground improvement can be in many forms depending on soil type and project requirements. For non-cohesive soil such as sands, silty sands and so on, deep compaction is a common ground improvement technique. Deep compaction can comprise: vibrocompaction, vibroreplacement, dynamic compaction, compaction piles, and deep blasting.

The CPT has been found to be one of the best methods to monitor and document the effect of deep compaction due to the continuous, reliable and repeatable nature of the data.

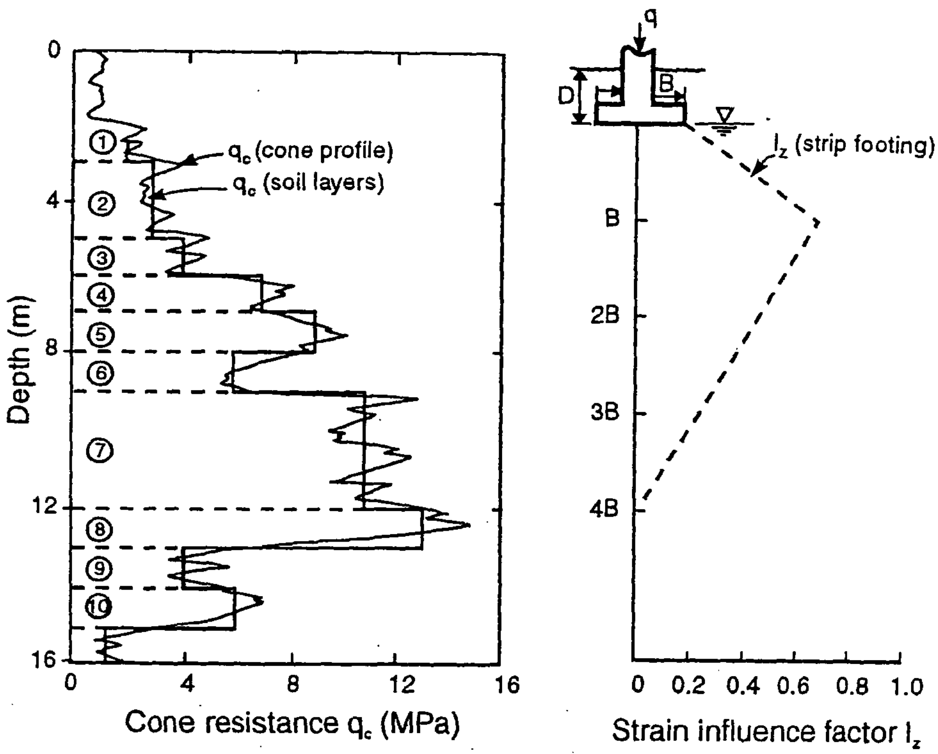


Figure 6.12 Application of Schertmann Method for settlement of footings on sand (Schertmann, 1978).

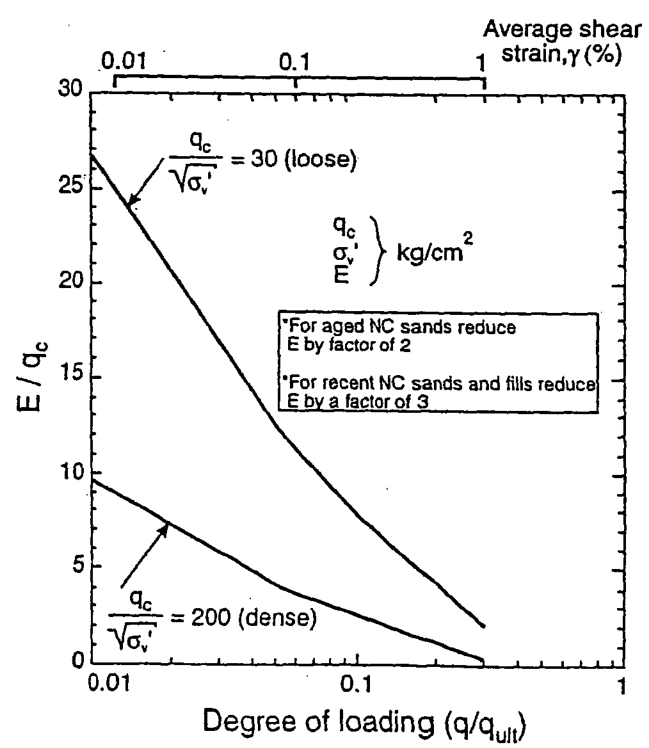


Figure 6.13 Estimation of equivalent Young's modulus for sand based on degree of loading (Robertson, 1991).

Most deep compaction techniques involve cyclic shear stresses in the form of vibration to induce an increase in soil density. Vibratory compaction is generally more effective in soil deposits with a friction ratio less than 1%. When the friction ratio exceeds about 1.5% vibratory compaction is usually not efficient (Massarsch, 1994). These recommendations apply to average values in a soil deposit. Local seams or thin layers with higher friction ratio values are often of little practical importance for the overall performance of a project and their effect should be carefully evaluated when compaction specifications are prepared. Soils with an initial cone resistance below about 3 MPa can be highly compressible (for example, if they have a high shell content) or contain organic matter, silt and clay. The zone of soil behaviour where vibratory compaction is most applicable is shown on the CPT soil behaviour charts in Figure 6.14. Soils with a high initial cone resistance will not show significant compaction and generally do not need compaction. It is also important to establish the level and variation of the groundwater table before compaction since some compaction methods are less effective in dry or partially saturated soils. The CPTU provides the required information on groundwater conditions.

Often the aim of deep compaction is for one or more of the following:

1. Increase bearing capacity (that is, increase shear strength).

ATTACHMENT D

LOG OF TEST BORINGS (BOART LONGYEAR)
WELL DETAIL
LOG OF TEST BORING-GENERAL NOTES
UNIFIED SOIL CLASSIFICATION SYSTEM
ABANDONMENT FORMS

LOG OF TEST BORING

CGC Inc.

Project: Alliant Ash Landfill Expansion

Location: Lansing, Iowa

Boring No. SPT-1

Surface Elevation (ft) 680.1

Job No. C20207

Sheet 1 of 1

3011 PERRY STREET, MADISON, WIS. 53713 (608) 288-4100, FAX (608) 288-7887

SAMPLE No.	TYPE	Rec (in.)	Moist	N	Depth (ft)	VISUAL CLASSIFICATION and Remarks	qu (qa) (tsf)	W	LL	PL	P200
1		16	M	4		FILL: Very Loose to Loose, Black to Gray Fly Ash					
2		20	M	7							
3		24	M	2	5						
4		24	M	2							
5		24	M	2	10						
6		34	M	1							
7		24	M	1	15						
8		24	M	1							
9		34	M	1	20						
10		24	M	2	▼						
11		24	M	1	25						
12		24	M	6							
					30	End of Boring at 29.5 ft Borehole backfilled with bentonite grout					
					35						

WATER LEVEL OBSERVATIONS

While Drilling ⊻ NW — Upon Completion of Drilling

Time After Drilling				3/4 hr
Depth to Water				22.5' ▼
Depth to Cave in				

The stratification lines represent the approximate boundary between soil types and the transition may be gradual.

GENERAL NOTES

Start 11/27/00 End 11/27/00

Driller Boart Chief MM Rig 811

Logger MM Editor WWW

Drill Method 4 1/4" HSA

CGC Inc.

LOG OF TEST BORING

Project: Alliant Ash Landfill Expansion

Location: Lansing, Iowa

Boring No. SPT-2/MW-10
Surface Elevation (ft) 684.6
Job No. C20207
Sheet 1 of 1

3011 PERRY STREET, MADISON, WIS. 53713 (608) 288-4100, FAX (608) 288-7887

SAMPLE						VISUAL CLASSIFICATION and Remarks	SOIL PROPERTIES				
No.	TYPE	Rec (in.)	Moist	N	Depth (ft)		qu (qa) (tsf)	W	LL	PL	P200
1		18	M	15		FILL: Medium Dense to Dense, Brown to Gray Fine to Medium SAND, Some Silt and Gravel, Scattered Silt and/or Clay Layers (Dike Material)					
2		22	M	15							
3		15	M	16	5						
4		15	M	20							
5		20	M	21	10						
6		15	M	38							
7		0	M	30	15						
8		7	M	18							
9		13	M	63	20	Medium Dense to Very Dense, Light Brown to Gray-Brown SAND and GRAVEL, Some Silt, Scattered Cobbles (SM/GM) (Weathered Dolomite)					
10		7	M	50							
11		15	M	25	25						
12		15	M	27							
					30	End of Boring at 30 ft Set Well at 29 ft					
					35						

WATER LEVEL OBSERVATIONS

While Drilling ∇ NW   Upon Completion of Drilling ______

Time After Drilling ______

Depth to Water ______

Depth to Cave in ______

The stratification lines represent the approximate boundary between soil types and the transition may be gradual.

GENERAL NOTES

Start 11/28/00 End 11/28/00

Driller Boart Chief MM Rig 811

Logger MM Editor WWW

Drill Method 4 1/4" HSA

FILE COPY

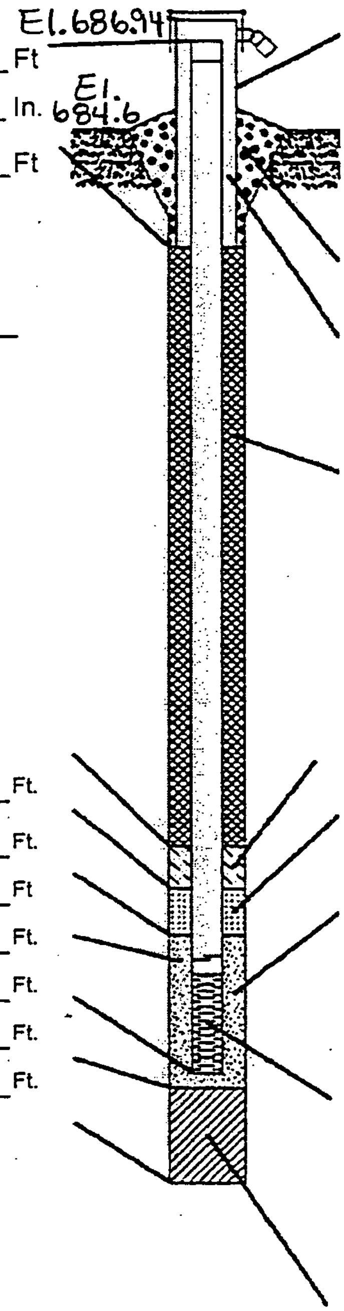
Job#: 10813
Location: Lansing IA
Well Name: MW-10
Date Installed: 11/28/2000
Chief: M. Mueller

Type of Well

- ☑ Water Table Observation
- ☐ Piezometer
- ☐ Other ______

A. Height of Well Casing above Ground 2.5 Ft
B. Diameter of Well Casing: 2.0 In.
C. Surface Seal Bottom: ____ Ft
D. Well Casing: Flush Threaded

- ☑ Schedule 40
- ☐ Schedule 80
- ☐ Other ______



E. Bentonite Seal Top 5.0 Ft.
F. Fine Sand Top NA Ft.
G. Filter Pack Top 17.5 Ft
H. Screen Joint Top 19.0 Ft.
I. Well Bottom 29.0 Ft.
J. Filter Pack Bottom 30.0 Ft.
K. Borehole Bottom 30.0 Ft.

1. **Locking Cap**
   ☑ Yes ☐ No
2. **Protective Cover**
   Inside Diameter: 4.0 In.
   Length: 7.0 Ft.
   Material: ☑ Steel ☐ Other ______
   Bumper Posts?
   ☐ Yes ☐ 3" ☐ 4"
   ☑ No
3. **Surface Seal:**
   ☐ Bentonite
   ☑ Concrete
4. **Material between Casing and Protop:**
   ☐ Bentonite
   ☑ Other Bentonite Chips
5. **Annual Space Seal:**
   ☐ Granular Bentonite
   ☑ Bentonite Slurry 30 Gallons
   ☐ Cement-Bentonite Grou
   How Installed:
   ☐ Gravity
   ☑ Tremie Pumped
6. **Bentonite Seal:**
   ☑ Granules
   ☐ Pellets
7. **Type of Fine Sand:** NA
8. **Type of Filter Pack:** #30 American Materials
   375 lbs/ Gravity Placement
9. **Screen Material:**
   ☑ PVC
   ☐ Stainless Steel
   ☐ Other ______
   Type ☑ Factory Cut
   ☐ Continuous Slot
   Slot Size: 0.010 In.
   Length: 10.0 Ft.
10. **Backfill Material:**
    ☑ None
    ☐ Other ______

Well Construction Report

BOART LONGYEAR
101 Alderson Street
PO Box 109
Schofield WI 54476

CGC Inc.

LOG OF TEST BORING

Project: Alliant Ash Landfill Expansion

Location: Lansing, Iowa

Boring No. SPT-3
Surface Elevation (ft) 689.1
Job No. C20207
Sheet 1 of 1

3011 PERRY STREET, MADISON, WIS. 53713 (608) 288-4100, FAX (608) 288-7887

SAMPLE No.	TYPE	Rec (in.)	Moist	N	Depth (ft)	VISUAL CLASSIFICATION and Remarks	qu (qa) (tsf)	W	LL	PL	P200
1		18	M	17		FILL: Medium Dense to Dense, Brown to Gray Fine to Medium SAND, Some Silt and Gravel, Scattered Silt and/or Clay Layers (Dike Material)					
2		22	M	30							
3			M	19	5						
4		18	M	32							
5		18	M	36	10						
6		8	M	37							
7		7	M	68	15						
8		10	M	44		Dense to Very Dense, Light Brown to Gray-Brown SAND and GRAVEL, Some Silt, Scattered Cobbles (SM/GM) (Weathered Dolomite)					
9		8	M	54	20						
10		11	M	55							
11		12	M	70	25						
12		12	M	62							
					30	End of Boring at 29.5 ft Borehole backfilled with bentonite chips					
					35						

WATER LEVEL OBSERVATIONS

While Drilling ∇ NW   Upon Completion of Drilling ____
Time After Drilling
Depth to Water
Depth to Cave in

The stratification lines represent the approximate boundary between soil types and the transition may be gradual.

GENERAL NOTES

Start 11/28/00 End 11/28/00
Driller Boart Chief MM Rig 811
Logger MM Editor WWW
Drill Method 4 1/4" HSA

CGC Inc.

LOG OF TEST BORING

Project: Alliant Ash Landfill Expansion

Location: Lansing, Iowa

Boring No. SPT-4

Surface Elevation (ft) 693.3

Job No. C20207

Sheet 1 of 1

3011 PERRY STREET, MADISON, WIS. 53713 (608) 288-4100, FAX (608) 288-7887

SAMPLE						VISUAL CLASSIFICATION and Remarks	SOIL PROPERTIES				
No.	TYPE	Rec (in.)	Moist	N	Depth (ft)		qu (qa) (tsf)	W	LL	PL	P200
1		18	M	16		FILL: Medium Dense to Dense, Brown to Gray Fine to Medium SAND, Some Silt and Gravel, Scattered Silt and/or Clay Layers (Dike Material)					
2		16	M	20							
3		17	M	18	5						
4		15	M	23							
5		18	M	33	10						
6		15	M	29							
7		21	M	21	15						
8		187	M	24							
9		20	M	18	20						
10		12	M	18		Medium Dense to Very Dense, Light Brown to Gray-Brown SAND and GRAVEL, Some Silt, Scattered Cobbles (SM/GM) (Weathered Dolomite)					
11		5	M	50/5"	25						
12		12	M	78		Stiff Clay Seam near 29 ft	(1.5)				
					30	End of Boring at 29.5 ft Borehole backfilled with bentonite chips					
					35						

WATER LEVEL OBSERVATIONS

While Drilling ⊻ NW   Upon Completion of Drilling ______

Time After Drilling ______

Depth to Water ______

Depth to Cave in ______

The stratification lines represent the approximate boundary between soil types and the transition may be gradual.

GENERAL NOTES

Start 11/28/00 End 11/28/00

Driller Boart Chief MM Rig 811

Logger MM Editor WWW

Drill Method 4 1/4" HSA

CGC Inc.

LOG OF TEST BORING

Project: Alliant Ash Landfill Expansion

Location: Lansing, Iowa

Boring No.: SPT-5
Surface Elevation (ft): 697.3
Job No.: C20207
Sheet 1 of 1

3011 PERRY STREET, MADISON, WIS. 53713 (608) 288-4100, FAX (608) 288-7887

SAMPLE / VISUAL CLASSIFICATION and Remarks / SOIL PROPERTIES

No.	Type	Rec (in.)	Moist	N	Depth (ft)	Visual Classification and Remarks	qu (qa) (tsf)	W	LL	PL	P200
1		18	M	11		FILL: Medium Dense to Dense, Brown to Gray Fine to Medium SAND, Some Silt and Gravel, Scattered Silt and/or Clay Layers (Dike Material)					
2		18	M	30							
3		18	M	44	5						
4		18	M	36							
5		12	M	12	10						
6		18	M	12							
7		15	M	23	15						
8		18	M	17							
9		18	M	15	20						
10		20	M	15		Medium Dense, Greenish Gray Sandy SILT, Trace Gravel (ML)					
11		16	M	26	25	Medium Dense, Light Brown to Gray-Brown SAND and GRAVEL, Some Silt, Scattered Cobbles (SM/GM) (Weathered Dolomite)					
12		21	M	23							
					30	End of Boring at 29.5 ft Borehole backfilled with bentonite chips					
					35						

WATER LEVEL OBSERVATIONS

While Drilling ⊻ NW Upon Completion of Drilling ______
Time After Drilling ______
Depth to Water ______
Depth to Cave in ______

The stratification lines represent the approximate boundary between soil types and the transition may be gradual.

GENERAL NOTES

Start 11/28/00 End 11/28/00
Driller Boart Chief MM Rig 811
Logger MM Editor WWW
Drill Method 4 1/4" HSA

CGC Inc.

LOG OF TEST BORING

Project: Alliant Ash Landfill Expansion

Location: Lansing, Iowa

Boring No. SPT-6
Surface Elevation (ft) 703.5
Job No. C20207
Sheet 1 of 1

3011 PERRY STREET, MADISON, WIS. 53713 (608) 288-4100, FAX (608) 288-7887

No.	TYPE	Rec (in.)	Moist	N	Depth (ft)	VISUAL CLASSIFICATION and Remarks	qu (qa) (tsf)	W	LL	PL	P200
1		21	M	3		FILL: Medium Stiff, Sandy Lean CLAY, Frequent Loose Sand & Silt Layers (CL) (Dike Material)					
2		24	M	14							
3		18	M	3	5		(1.0)				
4		17	M	11							
5		15	M	3	10						
6		13	M	1			(0.5)				
7		12	M	4	15		(0.75)				
8		0	M	2							
9		16	M	8	20		(1.0)				
10		18	M	8							
11		12	M	10	25						
12		12	M	16		Medium Dense, Black Sandy SILT, Trace Gravel (ML) (Possible Buried Topsoil)					
					30	End of Boring at 29.5 ft Borehole backfilled with bentonite chips					
					35						

WATER LEVEL OBSERVATIONS

While Drilling ∇ NW Upon Completion of Drilling ______
Time After Drilling
Depth to Water
Depth to Cave in

The stratification lines represent the approximate boundary between soil types and the transition may be gradual.

GENERAL NOTES

Start 11/28/00 End 11/28/00
Driller Boart Chief MM Rig 811
Logger MM Editor WWW
Drill Method 4 1/4" HSA

CGC, Inc.

LOG OF TEST BORING

General Notes

Descriptive Soil Classification

GRAIN SIZE TERMINOLOGY

Soil Fraction	Particle Size	U.S. Standard Sieve Size
Boulders	Larger than 12"	Larger than 12"
Cobbles	3" to 12"	3" to 12"
Gravel: Coarse	3/4" to 3"	3/4" to 3"
Fine	4.76 mm to 3/4"	#4 to 3/4"
Sand: Coarse	2.00 mm to 4.76 mm	#10 to #4
Medium	0.42 to mm to 2.00 mm	#40 to #10
Fine	0.074 mm to 0.42 mm	#200 to #40
Silt	0.005 mm to 0.074 mm	Smaller than #200
Clay	Smaller than 0.005 mm	Smaller than #200

Plasticity characteristics differentiate between silt and clay.

GENERAL TERMINOLOGY

Physical Characteristics
Color, moisture, grain shape, fineness, etc.
Major Constituents
Clay, silt, sand, gravel
Structure
Laminated, varved, fibrous, stratified, cemented, fissured, etc.
Geologic Origin
Glacial, alluvial, eolian, residual, etc.

RELATIVE DENSITY

Term	"N" Value
Very Loose	0-4
Loose	4-10
Medium Dense	10-30
Dense	30-50
Very Dense	Over 50

RELATIVE PROPORTIONS OF OF COHESIONLESS SOILS

Proportional Term	Defining Range by Percentage of Weight
Trace	0%-5%
Little	5%-12%
Some	12%-35%
And	35%-50%

CONSISTENCY

Term	q_u-tons/sq. ft.
Very Soft	0.0 to 0.25
Soft	0.25 to 0.50
Medium	0.50 to 1.0
Stiff	1.0 to 2.0
Very Stiff	2.0 to 4.0
Hard	Over 4.0

ORGANIC CONTENT BY COMBUSTION METHOD

Soil Description	Loss on Ignition
Non Organic	Less than 4%
Organic Silt/Clay	4-12%
Sedimentary Peat	12-50%
Fibrous and Woody Peat	More than 50%

PLASTICITY

Term	Plastic Index
None to Slight	0-4
Slight	5-7
Medium	8-22
High to Very High	Over 22

The penetration resistance, N, is the summation of the number of blows required to effect two successive 6" penetrations of the 2" split-barrel sampler. The sampler is driven with a 140 lb. weight falling 30" and is seated to a depth of 6" before commencing the standard penetration test.

SYMBOLS

DRILLING AND SAMPLING

CS--Continuous Sampling
RC--Rock Coring: Size AW, BW, NW, 2"W
RQD--Rock Quality Designator
RB--Rock Bit
FT--Fish Tail
DC--Drove Casing
C--Casing: Size 2 1/2", NW, 4", HW
CW--Clear Water
DM--Drilling Mud
HSA--Hollow Stem Auger
FA--Flight Auger
HA--Hand Auger
COA--Clean-Out Auger
SS--2" Diameter Split-Barrel Sample
2ST--2" Diameter Thin-Walled Tube Sample
3ST--3" Diameter Thin-Walled Tube Sample
PT--3" Diameter Piston Tube Sample
AS--Auger Sample
WS--Wash Sample
PTS--Peat Sample
PS--Pitcher Sample
NR--No Recovery
S--Sounding
PMT--Borehole Pressuremeter Test
VS--Vane Shear Test
WPT--Water Pressure Test

LABORATORY TESTS

q_a--Penetrometer Reading, tons/sq. ft.
q_u--Unconfined Strength, tons/sq. ft.
W--Moisture Content, %
LL--Liquid Limit, %
PL--Plastic Limit, %
SL--Shrinkage Limit, %
LI--Loss on Ignition, %
D--Dry Unit Weight, lbs/cu. ft.
pH--*Measure of Soil Alkalinity or Acidity*
FS--Free Swell, %

WATER LEVEL MEASUREMENT

∇ --Water Level at time shown
NW--No Water Encountered
WD--While Drilling
BCR--Before Casing Removal
ACR--After Casing Removal
CW--Caved and Wet
CM--Caved and Moist

Note: Water level measurements shown on the boring logs represent conditions at the time indicated and may not reflect static levels, especially in cohesive soils.

UNIFIED SOIL CLASSIFICATION SYSTEM

COARSE-GRAINED SOILS

(More than half of material is larger than No. 200 seive size.)

GRAVELS More than half of coarse fraction larger than No. 4 sieve size	**Clean Gravels** (Little or no fines)		
	GW	Well-graded gravels, gravel-sand mixtures, little or no fines	
	GP	Poorly graded gravels, gravel-sand mixtures, little or no fines	
	Gravels with Fines (Appreciable amount of fines)		
	GM $_u^d$	Silty gravels, gravel-sand-silt mixtures	
	GC	Clayey gravels, gravel-sand-clay mixtures.	
SANDS More than half of coarse fraction smaller than No. 4 sieve size	**Clean Sands** (Little or no fines)		
	SW	Well-graded sands, gravelly sands, little or no fines	
	SP	Poorly graded sands, gravelly sands, little or no fines	
	Sands with Fines (Appreciable amount of fines)		
	SM $_u^d$	Silty sands, sand-silt mixtures	
	SC	Clayey sands, sand-clay mixtures	

LABORATORY CLASSIFICATION CRITERIA

GW	$C_u = \frac{D_{60}}{D_{10}}$ greater than 4; $C_c = \frac{(D_{30})^2}{D_{10} X D_{60}}$ between 1 and 3	
GP	Not meeting all gradation requirements for GW	
GM	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	Atterberg limits above "A" line with P.I. greater than 7	
SW	$C_u = \frac{D_{60}}{D_{10}}$ greater than 6; $C_c = \frac{(D_{30})^2}{D_{10} X D_{60}}$ between 1 and 3	
SP	Not meeting all gradation requirements for SW	
SM	Atterberg limits below "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	Atterberg limits above "A" line with P.I. greater than 7	

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:

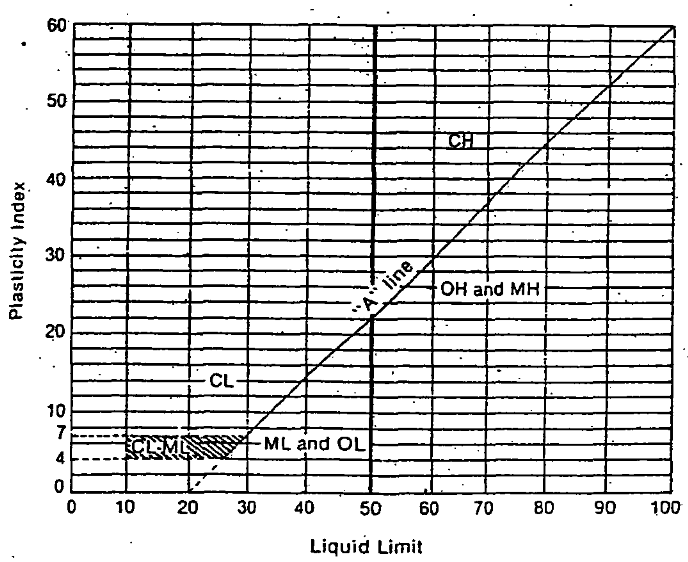
Less than 5 per cent	GW, GP, SW, SP
More than 12 per cent	GM, GC, SM, SC
5 to 12 per cent	Borderline cases requiring dual symbols

FINE-GRAINED SOILS

(More than half of 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



For classification of fine-grained soils and fine fraction of coarse-grained soils.

Atterberg Limits plotting in hatched area are borderline classifications requiring use of dual symbols.

Equation of A-line: PI = 0.73 (LL - 20)

BOART LONGYEAR

WELL/BORING ABANDONMENT FORM

CLIENT: BT2

JOB NAME: Alliance Power

LOCATION: Lansing IA

JOB NO: 10813

WELL/BORING NO: SB-1 (SPT-1)

REASON FOR ABANDONMENT: Test Boring

DATE OF ABANDONMENT: 11/27/00

ABANDONMENT DONE BY: M. Mueller

Construction Type: ☒ Drilled ☐ Driven ☐ Other: _

Formation Type: ☒ Unconsolidated ☐ Bedrock

Sealing Method: ☐ Gravity ☒ Pumped ☐ Other: ______

Sealing Materials:
☐ Bent. Chips ☒ Cement-Bent. Grout ☐ Other: ______

Sealing Material	From (Ft.)	To (Ft.)	# Bags or Vol.	
Bentonite Grout	0	29.5	60	Gallons

WELL INFORMATION ONLY

Total Well Depth	______	Screen Removed?	☐ yes	☐ no
Casing Diameter	______	Overdrilled?	☐ yes	☐ no
Casing Depth	______	Casing Pulled?	☐ yes	☐ no
Depth to Water	______	Cut Below Surface?	☐ yes	☐ no

BOART LONGYEAR

BOART LONGYEAR

WELL/BORING ABANDONMENT FORM

CLIENT: BT2

JOB NAME: Alliance Power

LOCATION: Lansing IA

JOB NO: 10813

WELL/BORING NO: SB-3 (SPT-3)

REASON FOR ABANDONMENT: Test Boring

DATE OF ABANDONMENT: 11/28/00

ABANDONMENT DONE BY: M. Mueller

Construction Type: ☒ Drilled ☐ Driven ☐ Other: _

Formation Type: ☒ Unconsolidated ☐ Bedrock

Sealing Method: ☒ Gravity ☐ Pumped ☐ Other: ____

Sealing Materials:
☒ Bent. Chips ☐ Cement-Bent. Grout ☐ Other: ____

Sealing Material	From (Ft.)	To (Ft.)	# Bags or Vol.	
Bentonite Chips	0	29.5	12	Bags

WELL INFORMATION ONLY

Total Well Depth		Screen Removed?	☐ yes	☐ no
Casing Diameter		Overdrilled?	☐ yes	☐ no
Casing Depth		Casing Pulled?	☐ yes	☐ no
Depth to Water		Cut Below Surface?	☐ yes	☐ no

BOART LONGYEAR

BOART LONGYEAR

WELL/BORING ABANDONMENT FORM

CLIENT: BT2

JOB NAME: Alliance Power

LOCATION: Lansing IA

JOB NO: 10813

WELL/BORING NO: SB-4 (SPT-4)

REASON FOR ABANDONMENT: Test Boring

DATE OF ABANDONMENT: 11/28/00

ABANDONMENT DONE BY: M. Mueller

Construction Type: ☒ Drilled ☐ Driven ☐ Other: _

Formation Type: ☒ Unconsolidated ☐ Bedrock

Sealing Method: ☒ Gravity ☐ Pumped ☐ Other: ______

Sealing Materials:
☒ Bent. Chips ☐ Cement-Bent. Grout ☐ Other: ______

Sealing Material	From (Ft.)	To (Ft.)	# Bags or Vol.	
Bentonite Chips	0	29.5	10	Bags

WELL INFORMATION ONLY

Total Well Depth ______
Casing Diameter ______
Casing Depth ______
Depth to Water ______

Screen Removed? ☐ yes ☐ no
Overdrilled? ☐ yes ☐ no
Casing Pulled? ☐ yes ☐ no
Cut Below Surface? ☐ yes ☐ no

BOART LONGYEAR

BOART LONGYEAR

WELL/BORING ABANDONMENT FORM

CLIENT: BT2

JOB NAME: Alliance Power

LOCATION: Lansing IA

JOB NO: 10813

WELL/BORING NO: SB-5 (SPT-5)

REASON FOR ABANDONMENT: Test Boring

DATE OF ABANDONMENT: 11/28/00

ABANDONMENT DONE BY: M. Mueller

Construction Type: ☒ Drilled ☐ Driven ☐ Other: _

Formation Type: ☒ Unconsolidated ☐ Bedrock

Sealing Method: ☒ Gravity ☐ Pumped ☐ Other: ______

Sealing Materials:
☒ Bent. Chips ☐ Cement-Bent. Grout ☐ Other: ______

Sealing Material	From (Ft.)	To (Ft.)	# Bags or Vol.	
Bentonite Chips	0	29.5	13	Bags

WELL INFORMATION ONLY

Total Well Depth ______
Casing Diameter ______
Casing Depth ______
Depth to Water ______

Screen Removed? ☐ yes ☐ no
Overdrilled? ☐ yes ☐ no
Casing Pulled? ☐ yes ☐ no
Cut Below Surface? ☐ yes ☐ no

BOART LONGYEAR

BOART LONGYEAR

WELL/BORING ABANDONMENT FORM

CLIENT: BT2

JOB NAME: Alliance Power

LOCATION: Lansing IA

JOB NO: 10813

WELL/BORING NO: SB-~~0~~6 (SPT-6)

REASON FOR ABANDONMENT: Test Boring

DATE OF ABANDONMENT: 11/28/00

ABANDONMENT DONE BY: M. Mueller

Construction Type: ☒ Drilled ☐ Driven ☐ Other: _

Formation Type: ☒ Unconsolidated ☐ Bedrock

Sealing Method: ☒ Gravity ☐ Pumped ☐ Other: ______

Sealing Materials:
☒ Bent. Chips ☐ Cement-Bent. Grout ☐ Other: ______

Sealing Material	From (Ft.)	To (Ft.)	# Bags or Vol.	
Bentonite Chips	0	29.5	12	Bags

WELL INFORMATION ONLY

Total Well Depth	______	Screen Removed?	☐ yes	☐ no
Casing Diameter	______	Overdrilled?	☐ yes	☐ no
Casing Depth	______	Casing Pulled?	☐ yes	☐ no
Depth to Water	______	Cut Below Surface?	☐ yes	☐ no

BOART LONGYEAR

ATTACHMENT E

PREVIOUS TERRACON REPORT

Existing Landfill

Terracon
CONSULTANTS, INC.
5855 Willow Creek Drive S.W.
P.O. Box H
Cedar Rapids, Iowa 52406-2987
(319) 366-8321 Fax: (319) 366-0032

Larry K. Davidson, P.E.
Dennis E. Whited, P.E.
André M. Gallet, P.E.
Timothy T. Wiles, P.E.
Jeffrey L. Magner, E.I.T.
Thomas A. Salm

September 16, 1996

Howard R. Green Company
4250 Glass Road NE
PO Box 9009
Cedar Rapids, Iowa 52409-9009

Attention: Mr. Gene Fritch

Re: Preliminary Subsurface Exploration
Proposed Fly Ash Embankment
Interstate Power Company
Lansing, Iowa
Job No. 06967025

Dear Mr. Fritch:

As requested, Terracon Consultants, Inc. performed a preliminary subsurface exploration for the referenced site. To date, eleven borings were performed to depths ranging from about 20 to 55.5 feet below the existing grade. The boring locations were selected and staked by Terracon representatives, and the approximate locations are shown on the attached Boring Location Diagram. We understand that Howard R. Green Company personnel may survey the boring locations at a later date.

The borings were drilled with a track-mounted rotary drilling rig using continuous flight augers to advance the boreholes. Representative samples were obtained using thin-walled tube and split-barrel sampling procedures in accordance with ASTM Specifications D-1587 and D-1586, respectively. In the thin-walled tube sampling procedure, a thin-walled, seamless steel tube with a sharp cutting edge is pushed hydraulically into the ground to obtain relatively undisturbed samples of cohesive or moderately cohesive soils. In the split-barrel sampling procedure, a standard 2-inch O.D. split-barrel sampling spoon is driven into the ground with a 140-pound hammer falling a distance of 30 inches. The number of blows required to advance the sampling spoon the last 12 inches of a normal 18-inch penetration is recorded as the standard penetration resistance value. These values are indicated on the boring logs at the depths of occurrence. Auger probes were also performed within the existing sluice pond in order to obtain bulk samples. The samples were sealed and returned to the laboratory. As requested, no laboratory testing has been performed.

Field logs of each boring were prepared by the field crew with the supervision of a field geotechnical engineer. These logs include visual classification of the materials encountered during the drilling operation as well as the driller's interpretation of the subsurface conditions between samples. The description and stratification of the subsurface soil conditions encountered by the drill crew are illustrated in the form of soil profiles on the attached Soil Boring Logs. Stratification boundaries on the boring logs represent the approximate location of changes in the soil and rock types; in situ, the transition between samples may be gradual.

It should be noted that the soil descriptions indicated on the boring logs are based solely on the driller's interpretation, and further visual and laboratory testing would be required for engineering classification. In addition, all the boreholes were backfilled with bentonite hole plug at the interface of the native soils and the fly ash fill material.

The borings were monitored for the presence and level of groundwater. Water levels observed in the borings are noted on the boring logs. It should be recognized that fluctuations of the groundwater may occur due to seasonal variations in the amount of rainfall, runoff, and other factors not evident at the time the borings were performed. In addition, perched groundwater conditions could occur. Longer term monitoring in cased holes or piezometers would be required for a more accurate evaluation of the groundwater conditions.

Based on our review of the field data, the previously dredged fly ash materials within the proposed embankment fill area were very soft and underconsolidated; thus, significant settlement and possible global slope stability and foundation bearing failures could result with the construction of the proposed 10 to 35 feet high embankments within this area. Further extensive testing and analyses would be required to evaluate these conditions. We understand that at this time, based on the limited data obtained, the plan for placing additional fill within this area is being abandoned.

This report has been prepared for the exclusive use of our client for the specific application to the project discussed and has been prepared in accordance with general accepted geotechnical engineering practices. No warranty, expressed or implied, is provided. Terracon has not been requested to provided detailed analyses of the enclosed data or provide design and/or construction recommendations based on the data, and thus, cannot assume responsibility or liability of interpretation of this data by others.

We appreciate the opportunity to be of service to you on this phase of your project and look forward in assisting you in the future. If you have any questions regarding this report, or if we may be of further service to you, please contact us.

Sincerely,
TERRACON CONSULTANTS, INC.

Prepared By:

André M. Gallet/AD

André M. Gallet, P.E.
Iowa No. 13430

Reviewed By:

Dennis E. Whited, P.E.
Iowa No. 8538

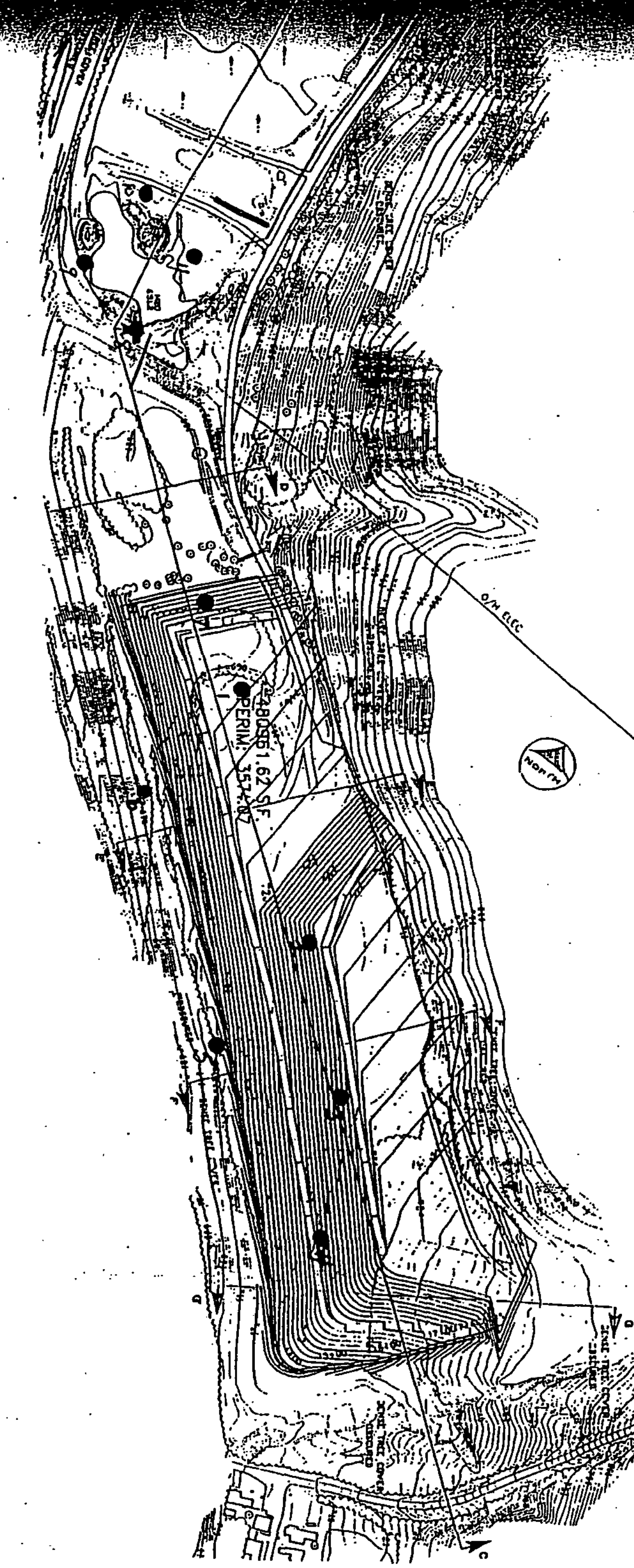
AMG/DEW:amd\reports06967025

Attachments

Copies to: Addressee (3)

APPENDIX

Terracon



Note: This diagram is not to scale and is intended for general location purposes only.

BORING LOCATION DIAGRAM
PROPOSED FLY ASH EMBANKMENT
INTERSTATE POWER COMPANY
LANSING, IOWA

Job #	Date	Drawn	Scale
06967025	9/13/96	by others	NTS

Terracon

Form 149-10-85

GENERAL NOTES

DRILLING & SAMPLING SYMBOLS:

SS	:	Split Spoon - 1⅜" I.D., 2" O.D., unless otherwise noted
ST	:	Thin-Walled Tube - 2" O.D., Unless otherwise noted
PA	:	Power Auger
HA	:	Hand Auger
DB	:	Diamond Bit - 4", N, B
AS	:	Auger Sample
HS	:	Hollow Stem Auger
PS	:	Piston Sample
WS	:	Wash Sample
FT	:	Fish Tail Bit
RB	:	Rock Bit
BS	:	Bulk Sample
PM	:	Pressuremeter
DC	:	Dutch Cone
WB	:	Wash Bore

Standard "N" Penetration: Blows per foot of a 140 pound hammer falling 30 inches on a 2 inch OD split spoon, except where noted.

WATER LEVEL MEASUREMENT SYMBOLS:

WL	:	Water Level
WCI	:	Wet Cave In
DCI	:	Dry Cave In
AB	:	After Boring
WS	:	While Sampling
WD	:	While Drilling
BCR	:	Before Casing Removal
ACR	:	After Casing Removal

Water levels indicated on the boring logs are the levels measured in the borings at the times indicated. In pervious soils, the indicated levels may reflect the location of groundwater. In low permeability soils, the accurate determination of ground water levels is not possible with only short term observations.

DESCRIPTIVE SOIL CLASSIFICATION:

Soil Classification is based on the Unified Soil Classification System and ASTM Designations D-2487 and D-2488. Coarse Grained Soils have more than 50% of their dry weight retained on a #200 sieve; they are described as: boulders, cobbles, gravel or sand. Fine Grained Soils have less than 50% of their dry weight retained on a #200 sieve; they are described as: clays, if they are plastic, and silts if they are slightly plastic or non-plastic. Major constituents may be added as modifiers and minor constituents may be added according to the relative proportions based on grain size. In addition to gradation, coarse grained soils are defined on the basis of their relative in-place density and fine grained soils on the basis of their consistency. Example: Lean clay with sand, trace gravel, stiff (CL); silty sand, trace gravel, medium dense (SM).

CONSISTENCY OF FINE-GRAINED SOILS:

Unconfined Compressive Strength, Qu, psf	Consistency
< 500	Very Soft
500 - 1,000	Soft
1,001 - 2,000	Medium
2,001 - 4,000	Stiff
4,001 - 8,000	Very Stiff
8,001 -16,000	Hard
> -16,000	Very Hard

RELATIVE PROPORTIONS OF SAND AND GRAVEL

Descriptive Term(s) (of Components Also Present in Sample)	Percent of Dry Weight
Trace	< 15
With	15 - 29
Modifier	> 30

RELATIVE PROPORTIONS OF FINES

Descriptive Term(s) (of Components Also Present in Sample)	Percent of Dry Weight
Trace	< 5
With	5 - 12
Modifier	> 12

RELATIVE DENSITY OF COARSE-GRAINED SOILS:

N-Blows/ft.	Relative Density
0-3	Very Loose
4-9	Loose
10-29	Medium Dense
30-49	Dense
50-80	Very Dense
80+	Extremely Dense

GRAIN SIZE TERMINOLOGY

Major Component Of Sample	Size Range
Boulders	Over 12 in. (300mm)
Cobbles	12 in. to 3 in. (300mm to 75mm)
Gravel	3 in. to #4 sieve (75mm to 4.75mm)
Sand	#4 to #200 sieve (4.75mm to 0.075mm)
Silt or Clay	Passing #200 sieve (0.075mm)

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GENERAL NOTES

Sedimentary Rock Classification

DESCRIPTIVE ROCK CLASSIFICATION:

Sedimentary rocks are composed of cemented clay, silt and sand sized particles. The most common minerals are clay, quartz and calcite. Rock composed primarily of calcite is called limestone; rock of sand size grains is called sandstone, and rock of clay and silt size grains is called mudstone or claystone, siltstone, or shale. Modifiers such as shaly, sandy, dolomitic, calcareous, carbonaceous, etc. are used to describe various constituents. Examples: sandy shale; calcareous sandstone.

LIMESTONE	Light to dark colored, crystalline to fine-grained texture, composed of $CaCo_3$, reacts readily with HCl.
DOLOMITE	Light to dark colored, crystalline to fine-grained texture, composed of $CaMg(CO_3)_2$, harder than limestone, reacts with HCl when powdered.
CHERT	Light to dark colored, very fine-grained texture, composed of micro-crystalline quartz ($Si0_2$), brittle, breaks into angular fragments, will scratch glass.
SHALE	Very fine-grained texture, composed of consolidated silt or clay, bedded in thin layers. The unlaminated equivalent is frequently referred to as siltstone, claystone or mudstone.
SANDSTONE	Usually light colored, coarse to fine texture, composed of cemented sand size grains of quartz, feldspar, etc. Cement usually is silica but may be such minerals as calcite, iron-oxide, or some other carbonate.
CONGLOMERATE	Rounded rock fragments of variable mineralogy varying in size from near sand to boulder size but usually pebble to cobble size (½ inch to 6 inches). Cemented together with various cementing agents. Breccia is similar but composed of angular, fractured rock particles cemented together.

DEGREE OF WEATHERING:

SLIGHT	Slight decomposition of parent material on joints. May be color change.
MODERATE	Some decomposition and color change throughout.
HIGH	Rock highly decomposed, may be extremely broken.

Classification of rock materials has been estimated from disturbed samples. Core samples and petrographic analysis may reveal other rock types.

Terracon

UNIFIED SOIL CLASSIFICATION SYSTEM

Criteria for Assigning Group Symbols and Group Names Using Laboratory Tests[A]				Soil Classification: Group Symbol	Soil Classification: Group Name[B]
Coarse-Grained Soils More than 50% retained on No. 200 sieve	Gravels More than 50% of coarse fraction retained on No. 4 sieve	Clean Gravels Less than 5% fines[C]	$Cu \geq 4$ and $1 \leq Cc \leq 3$[E]	GW	Well-graded gravel[F]
			$Cu < 4$ and/or $1 > Cc > 3$[E]	GP	Poorly graded gravel[F]
		Gravels with Fines More than 12% fines[C]	Fines classify as ML or MH	GM	Silty gravel[F, G, H]
			Fines classify as CL or CH	GC	Clayey gravel[F, G, H]
	Sands 50% or more of coarse fraction passes No. 4 sieve	Clean Sands Less than 5% fines[E]	$Cu \geq 6$ and $1 \leq Cc \leq 3$[E]	SW	Well-graded sand[I]
			$Cu < 6$ and/or $1 > Cc > 3$[E]	SP	Poorly graded sand[I]
		Sands with Fines More than 12% fines[D]	Fines classify as ML or MH	SM	Silty sand[G, H, I]
			Fines classify as CL or CH	SC	Clayey sand[G, H, I]
Fine-Grained Soils 50% or more passes the No. 200 sieve	Silts and Clays Liquid limit less than 50	inorganic	PI > 7 and plots on or above "A" line[J]	CL	Lean clay[K, L, M]
			PI < 4 or plots below "A" line[J]	ML	Silt[K, L, M]
		organic	$\frac{\text{Liquid limit — oven dried}}{\text{Liquid limit — not dried}} < 0.75$	OL	Organic clay[K, L, M, N] / Organic silt[K, L, M, O]
	Silts and Clays Liquid limit 50 or more	inorganic	PI plots on or above "A" line	CH	Fat clay[K, L, M]
			PI plots below "A" line	MH	Elastic silt[K, L, M]
		organic	$\frac{\text{Liquid limit — oven dried}}{\text{Liquid limit — not dried}} < 0.75$	OH	Organic clay[K, L, M, P] / Organic silt[K, L, M, Q]
Highly organic soils	Primarily organic matter, dark in color, and organic odor			PT	Peat

[A] Based on the material passing the 3-in. (75-mm) sieve.

[B] If field sample contained cobbles or boulders, or both, add "with cobbles or boulders, or both" to group name.

[C] Gravels with 5 to 12% fines require dual symbols:
GW-GM well-graded gravel with silt
GW-GC well-graded gravel with clay
GP-GM poorly graded gravel with silt
GP-GC poorly graded gravel with clay

[D] Sands with 5 to 12% fines require dual symbols:
SW-SM well-graded sand with silt
SW-SC well-graded sand with clay
SP-SM poorly graded sand with silt
SP-SC poorly graded sand with clay

[E] $Cu = D_{60}/D_{10}$ $\quad Cc = \frac{(D_{30})^2}{D_{10} \times D_{60}}$

[F] If soil contains ≥ 15% sand, add "with sand" to group name.

[G] If fines classify as CL-ML, use dual symbol GC-GM, or SC-SM.

[H] If fines are organic, add "with organic fines" to group name.

[I] If soil contains ≥ 15% gravel, add "with gravel" to group name.

[J] If Atterberg limits plot in shaded area, soil is a CL-ML, silty clay.

[K] If soil contains 15 to 29% plus No. 200, add "with sand" or "with gravel", whichever is predominant.

[L] If soil contains ≥ 30% plus No. 200 predominantly sand, add "sandy" to group name.

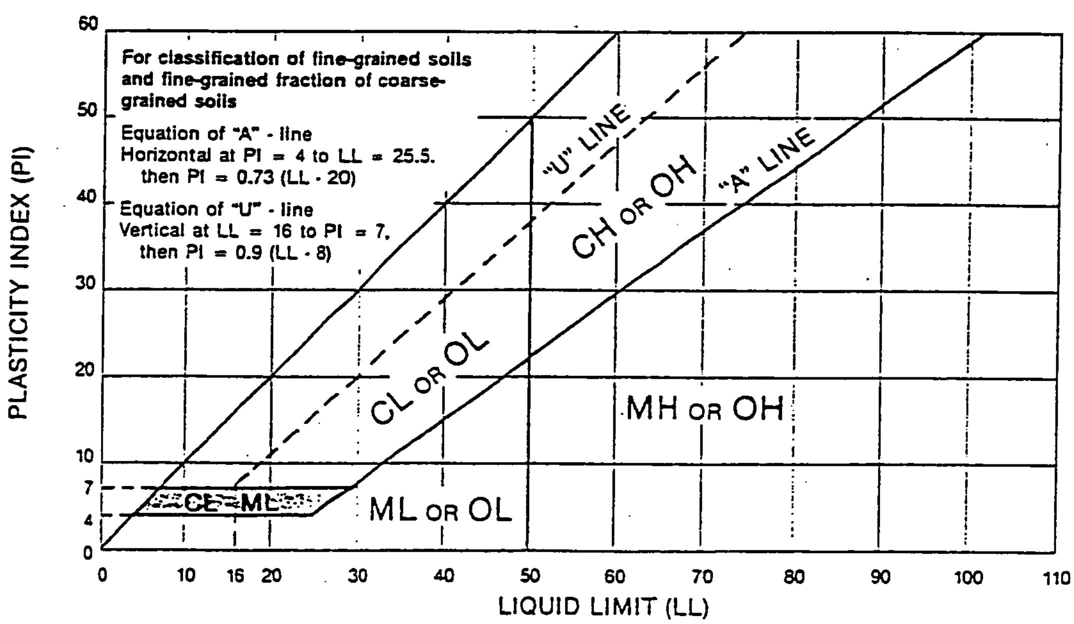
[M] If soil contains ≥ 30% plus No. 200, predominantly gravel, add "gravelly" to group name.

[N] PI ≥ 4 and plots on or above "A" line.

[O] PI < 4 or plots below "A" line.

[P] PI plots on or above "A" line.

[Q] PI plots below "A" line.



Terracon

LOG OF BORING NO. 1

OWNER: INTERSTATE POWER COMPANY	ENGINEER: HOWARD R. GREEN COMPANY
SITE: LANSING, IOWA	PROJECT: PROPOSED FLY ASH EMBANKMENT

GRAPHIC LOG	DESCRIPTION	DEPTH (FT.)	USCS SYMBOL	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: RECOVERY, IN.	SAMPLES: **SPT - N BLOWS / FT.	TESTS: MOISTURE, %	TESTS: DRY DENSITY PCF	TESTS: UNCONFINED STRENGTH, PSF
0.5	4" Root Zone				HS					
	FILL, LEAN CLAY, TRACE SAND & ORGANICS, Dark Brown									
	- Trace brick @ 4 feet.	5		1	SS	10	4			
	∇				HS					
	FILL, FLY ASH WITH FINE SAND, Gray and Dark Gray									
		10		2	SS	18	2			
					HS					
13										
	FILL, FINE SAND, TRACE FLY ASH & BRICK, Brown	15		3	SS	18	3			
					HS					
17										
	FILL, FINE SAND WITH SILT & LIMESTONE SEAMS, TRACE FLY ASH, Gray	20		4	SS	18	1			
21					HS					
	FILL, FLY ASH, TRACE SAND, Gray	25		5	SS	18	2			
					HS					
		30		6	SS	18	0 WOH			
					HS					

Continued Next Page

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

Calibrated Hand Penetrometer*
CME 140 Lb. Auto. SPT Hammer **

WATER LEVEL OBSERVATIONS (FT.)			
WL	∇ 5.5	WS ▼ 34	(8/29/96)
WL	∇	▼	
WL	WCI @ 36' (8/29/96)		

Terracon

BORING STARTED	8-28-96
BORING COMPLETED	8-28-96
RIG #37	FOREMAN REF
APPROVED AMG	JOB # 06967025

N3DLGE 67025 9/16/96

LOG OF BORING NO. 1

OWNER	ENGINEER
INTERSTATE POWER COMPANY	HOWARD R. GREEN COMPANY
SITE	**PROJECT**
LANSING, IOWA	PROPOSED FLY ASH EMBANKMENT

GRAPHIC LOG	DESCRIPTION	DEPTH (FT.)	USCS SYMBOL	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: RECOVERY, IN.	SAMPLES: **SPT - N BLOWS / FT.	TESTS: MOISTURE, %	TESTS: DRY DENSITY PCF	TESTS: UNCONFINED STRENGTH, PSF
	FILL, FLY ASH, TRACE SAND, Gray	35		7	SS	18	0 WOH			
					HS					
		40		8	SS	18	0 WOH			
					HS					
44										
44.5	LEAN CLAY, TRACE SAND, Brown and Dark Brown	45		9	SS	18	21			*2000
45.5	***HIGHLY WEATHERED SANDSTONE, Light Brown									
	BOTTOM OF BORING									

***Classification estimated from disturbed samples. Core samples and petrographic analysis may reveal other rock types.

NOTE: Material descriptions are based on driller's visual classification only.

- WOH refers to Weight of Hammer.

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

Calibrated Hand Penetrometer*
CME 140 Lb. Auto. SPT Hammer **

WATER LEVEL OBSERVATIONS (FT.)	
WL ∇ 5.5	WS ▼ 34 (8/29/96)
WL ∇	▼
WL	WCI @ 36' (8/29/96)

Terracon

BORING STARTED	8-28-96
BORING COMPLETED	8-28-96
RIG #37	FOREMAN REF
APPROVED AMG	JOB # 06967025

N3BLGE 67026 9/10/96

LOG OF BORING NO. 2

OWNER	ENGINEER
INTERSTATE POWER COMPANY	HOWARD R. GREEN COMPANY
SITE	**PROJECT**
LANSING, IOWA	PROPOSED FLY ASH EMBANKMENT

GRAPHIC LOG (depth)	DESCRIPTION	DEPTH (FT.)	USCS SYMBOL	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES RECOVERY, IN.	SAMPLES **SPT - N BLOWS / FT.	TESTS MOISTURE, %	TESTS DRY DENSITY PCF	TESTS UNCONFINED STRENGTH, PSF
0.5	4" Root Zone FILL, LEAN CLAY, TRACE SAND & ORGANICS, Dark Brown				HS					
	FILL, FLY ASH, TRACE SAND, Dark Gray ▽	5		1	SS	10	4			
					HS					
10.5		10		2	SS	18	2			
					HS					
	FILL, FINE TO COARSE SAND WITH LIMESTONE PIECES & FLY ASH, Brown	15		3	SS	16	11			
					HS					
		20		4	SS	2	8			
					HS					
		25		5	SS	16	13			
28					HS					
	***FILL, EXTREMELY WEATHERED LIMESTONE, Light Gray	30		6	SS	10	13			
					HS					

Continued Next Page

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

Calibrated Hand Penetrometer*
CME 140 Lb. Auto. SPT Hammer **

WATER LEVEL OBSERVATIONS (FT.)		
WL	▽ 6	WS ▼ NONE (8/29/96)
WL	▽	▼
WL	DCI @ 42' (8/29/96)	

Terracon

BORING STARTED	8-28-96
BORING COMPLETED	8-28-96
RIG #37	FOREMAN REF
APPROVED AMG	JOB # 06967025

H3BLGE 67025 9/16/96

LOG OF BORING NO. 2

OWNER	ENGINEER
INTERSTATE POWER COMPANY	HOWARD R. GREEN COMPANY
SITE	**PROJECT**
LANSING, IOWA	PROPOSED FLY ASH EMBANKMENT

GRAPHIC LOG	DESCRIPTION	DEPTH (FT.)	USCS SYMBOL	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: RECOVERY, IN.	SAMPLES: **SPT - N BLOWS / FT.	TESTS: MOISTURE, %	TESTS: DRY DENSITY PCF	TESTS: UNCONFINED STRENGTH, PSF
37.5	***FILL, EXTREMELY WEATHERED LIMESTONE, Light Gray	35		7	SS	16	0 WOH			
					HS					
45	FILL, SILTY FINE SAND, TRACE LIMESTONE GRAVEL, Dark Gray	40		8	SS	16	10			
					HS					
		45		9	SS	18	3			
46	FILL, FINE TO COARSE SAND, Brown				HS					
50	***FILL, HIGHLY WEATHERED LIMESTONE & SANDSTONE, Light Brown	50		10	SS	12	10			*3000
					HS					
55.5	CLAYEY SILT, TRACE SAND & LIMESTONE PIECES, Brown Gray	55		11	SS	14	6			*1000

BOTTOM OF BORING

***Classification estimated from disturbed samples. Core samples and petrographic analysis may reveal other rock types.

NOTE: Material descriptions are based on driller's visual classification only.

- WOH refers to Weight of Hammer.

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

Calibrated Hand Penetrometer*
CME 140 Lb. Auto. SPT Hammer **

WATER LEVEL OBSERVATIONS (FT.)		Terracon	BORING STARTED	8-28-96
WL ▽ 6	WS ▼ NONE (8/29/96)		BORING COMPLETED	8-28-96
WL ▽	▼		RIG #37	FOREMAN REF
WL DCI @ 42' (8/29/96)			APPROVED AMG	JOB # 06967025

LOG OF BORING NO. 3

OWNER	ENGINEER
INTERSTATE POWER COMPANY	HOWARD R. GREEN COMPANY
SITE	**PROJECT**
LANSING, IOWA	PROPOSED FLY ASH EMBANKMENT

Depth to base (ft.)	DESCRIPTION	DEPTH (FT.)	USCS SYMBOL	SAMPLES: NUMBER	TYPE	RECOVERY, IN.	**SPT - N BLOWS / FT.	TESTS: MOISTURE, %	DRY DENSITY PCF	UNCONFINED STRENGTH, PSF
0.5	3" Root Zone				HS					
	FILL, LEAN CLAY, TRACE SAND & ORGANICS, Dark Brown									
6	FILL, FLY ASH, TRACE SAND, Dark Gray ∇	5		1	SS	18	2			
					HS					
17	FILL, LEAN CLAY, TRACE GRAVEL WITH SAND SEAMS, Brown and Dark Brown	10		2	SS	14	6			
					HS					
		15		3	SS	4	4			
					HS					
22	CLAYEY SILT, TRACE SAND & WOOD, Brown Gray	20		4	SS	16	5			*1500
					HS					
30.5	LEAN CLAY, TRACE SAND, GRAVEL & ORGANICS, Brown and Dark Brown	25		5	SS	18	7			*5000
					HS					
		30		6	3" ST					
	CLAYEY SILT, TRACE SAND WITH LIMESTONE GRAVEL PIECES & SAND SEAMS, Brown				HS					

Continued Next Page

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

Calibrated Hand Penetrometer*
CME 140 Lb. Auto. SPT Hammer **

WATER LEVEL OBSERVATIONS (FT.)		
WL	∇ 5	WS ▼ NONE (8/29/96)
WL	∇	▼
WL	DCI @ 31' (8/29/96)	

Terracon

BORING STARTED	8-28-96		
BORING COMPLETED	8-28-96		
RIG	#37	FOREMAN	REF
APPROVED	AMG	JOB #	06967025

LOG OF BORING NO. 3

OWNER	ENGINEER
INTERSTATE POWER COMPANY	HOWARD R. GREEN COMPANY
SITE	PROJECT
LANSING, IOWA	PROPOSED FLY ASH EMBANKMENT

GRAPHIC LOG	DESCRIPTION	DEPTH (FT.)	USCS SYMBOL	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: RECOVERY, IN.	SAMPLES: **SPT - N BLOWS / FT.	TESTS: MOISTURE, %	TESTS: DRY DENSITY PCF	TESTS: UNCONFINED STRENGTH, PSF
	CLAYEY SILT, TRACE SAND WITH LIMESTONE GRAVEL PIECES & SAND SEAMS, Brown	35		7	SS	16	9			*1500
36					HS					
	FINE TO MEDIUM SAND, Brown									
40.5		40		8	SS	18	25			
	BOTTOM OF BORING									

NOTE: Material descriptions are based on driller's visual classification only.

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

Calibrated Hand Penetrometer*
CME 140 Lb. Auto. SPT Hammer **

WATER LEVEL OBSERVATIONS (FT.)		
WL	▽ 5	WS ▼ NONE (8/29/96)
WL	▽	▼
WL	DCI @ 31' (8/29/96)	

Terracon

BORING STARTED	8-28-96		
BORING COMPLETED	8-28-96		
RIG	#37	FOREMAN	REF
APPROVED	AMG	JOB #	06967025

N3BLGE 67025 9/16/96

LOG OF BORING NO. 4

OWNER	ENGINEER
INTERSTATE POWER COMPANY	HOWARD R. GREEN COMPANY
SITE	**PROJECT**
LANSING, IOWA	PROPOSED FLY ASH EMBANKMENT

Depth to bottom of stratum (ft.)	DESCRIPTION
4	6" Root Zone FILL, FINE TO COARSE SAND WITH GRAVEL, TRACE SILT, Brown and Dark Brown
7	SANDY LEAN CLAY, TRACE ORGANICS, Dark Gray
17.5	CLAYEY SILT WITH SAND SEAMS, Medium Light Gray
22	SILTY FINE TO MEDIUM SAND, TRACE GRAVEL WITH CLAY SEAMS, Brown
25.5	***WEATHERED LIMESTONE WITH SANDSTONE PIECES, Light Brown Gray

BOTTOM OF BORING
***Classification estimated from disturbed samples. Core samples and petrographic analysis may reveal other rock types.

NOTE: Material descriptions are based on driller's visual classification only.

SAMPLES: NUMBER	TYPE	RECOVERY, IN.	**SPT - N BLOWS / FT.	TESTS: MOISTURE, %	DRY DENSITY PCF	UNCONFINED STRENGTH, PSF
	HS					
1	SS	16	7			*2000
	HS					
2	3" ST					
	HS					
3	3" ST					
	HS					
4	SS	18	6			
	HS					
5	SS	12	35			

Depth scale (FT.): 5, 10, 15, 20, 25. USCS SYMBOL column blank.

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

Calibrated Hand Penetrometer*
CME 140 Lb. Auto. SPT Hammer **

WATER LEVEL OBSERVATIONS (FT.)		
WL	NONE	WS NONE (8/29/96)
WL		
WL	DCI @ 17' (8/29/96)	

Terracon

BORING STARTED	8-28-96
BORING COMPLETED	8-28-96
RIG #37	FOREMAN REF
APPROVED AMG	JOB # 06967025

LOG OF BORING NO. 5

OWNER	ENGINEER
INTERSTATE POWER COMPANY	HOWARD R. GREEN COMPANY
SITE	PROJECT
LANSING, IOWA	PROPOSED FLY ASH EMBANKMENT

GRAPHIC LOG	DESCRIPTION	DEPTH (FT.)	USCS SYMBOL	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES RECOVERY, IN.	SAMPLES ••SPT - N BLOWS / FT.	TESTS MOISTURE, %	TESTS DRY DENSITY PCF	TESTS UNCONFINED STRENGTH, PSF
	3" Root Zone				HS					
	FILL, CLAYEY SILT, TRACE SAND & LIMESTONE GRAVEL, Brown and Gray	5		1	SS	16	7			
					HS					
		10		2	3" ST					
					HS					
		15		3	3" ST					
16	FILL, SANDY LEAN CLAY & CLAYEY SILT, TRACE GRAVEL, Brown and Dark Brown				HS					
		20		4	SS	18	7			
21	SANDY LEAN CLAY, Medium Dark Brown				HS					
		25		5	3" ST					
					HS					
28	FINE TO COARSE SAND & LIMESTONE GRAVEL, Brown	30		6	SS	8	21			
					HS					
32.5										

Continued Next Page

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

Calibrated Hand Penetrometer*
CME 140 Lb. Auto. SPT Hammer **

WATER LEVEL OBSERVATIONS (FT.)		
WL	▽ NONE	WS ▼ NONE (8/29/96)
WL	▽	▼
WL	DCI @ 29' (8/29/96)	

Terracon

BORING STARTED	8-28-96		
BORING COMPLETED	8-28-96		
RIG	#37	FOREMAN	REF
APPROVED	AMG	JOB #	06967025

1130LGE 07025 9/16/96

LOG OF BORING NO. 6

OWNER	ENGINEER
INTERSTATE POWER COMPANY	HOWARD R. GREEN COMPANY
SITE	PROJECT
LANSING, IOWA	PROPOSED FLY ASH EMBANKMENT

GRAPHIC LOG	DESCRIPTION	DEPTH (FT.)	USCS SYMBOL	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES RECOVERY, IN.	SAMPLES **SPT - N BLOWS / FT.	TESTS MOISTURE, %	TESTS DRY DENSITY PCF	TESTS UNCONFINED STRENGTH, PSF
	4" Root Zone				HS					
		5		1	SS	16	27			
	FILL, FINE TO COARSE SAND WITH LIMESTONE PIECES, TRACE GRAVEL & SILT, Brown and Gray				HS					
		10		2	SS	16	11			
					HS					
		15		3	SS	0	8			
17					HS					
		20		4	SS	10	4			
	FILL, SANDY LEAN CLAY, TRACE GRAVEL WITH LIMESTONE COBBLES, Brown and Dark Brown				HS					
		25		5	SS	12	51			
					HS					
		30		6	SS	14	9			
32.5					HS					

Continued Next Page

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

Calibrated Hand Penetrometer*
CME 140 Lb. Auto. SPT Hammer **

WATER LEVEL OBSERVATIONS (FT.)			BORING STARTED	8-28-96
WL	▽ 43	WS ▽ 42 (8/29/96)	BORING COMPLETED	8-28-96
WL	▽	▽	RIG #37	FOREMAN REF
WL	WCI @ 43' (8/29/96)		APPROVED AMG	JOB # 06967025

Terracon

LOG OF BORING NO. 5

OWNER	ENGINEER
INTERSTATE POWER COMPANY	HOWARD R. GREEN COMPANY
SITE	PROJECT
LANSING, IOWA	PROPOSED FLY ASH EMBANKMENT

GRAPHIC LOG	DESCRIPTION	DEPTH (FT.)	USCS SYMBOL	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: RECOVERY, IN.	SAMPLES: **SPT - N BLOWS / FT.	TESTS: MOISTURE, %	TESTS: DRY DENSITY PCF	TESTS: UNCONFINED STRENGTH, PSF
	***WEATHERED TO HARD LIMESTONE, Light Brown Gray									
	34.5			7	SS	0	50/4"			
	BOTTOM OF BORING									

***Classification estimated from disturbed samples. Core samples and petrographic analysis may reveal other rock types.

NOTE: Material descriptions are based on driller's visual classification only.

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

Calibrated Hand Penetrometer*
CME 140 Lb. Auto. SPT Hammer **

WATER LEVEL OBSERVATIONS (FT.)			
WL	▽ NONE	WS	▼ NONE (8/29/96)
WL	▽		▽
WL	DCI @ 29' (8/29/96)		

Terracon

BORING STARTED	8-28-96
BORING COMPLETED	8-28-96
RIG #37	FOREMAN REF
APPROVED AMG	JOB # 06967025

LOG OF BORING NO. 7

OWNER	ENGINEER
INTERSTATE POWER COMPANY	HOWARD R. GREEN COMPANY
SITE	**PROJECT**
LANSING, IOWA	PROPOSED FLY ASH EMBANKMENT

DESCRIPTION	DEPTH (FT.)	USCS SYMBOL	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: RECOVERY, IN.	SAMPLES: **SPT - N BLOWS / FT.	TESTS: MOISTURE, %	TESTS: DRY DENSITY PCF	TESTS: UNCONFINED STRENGTH, PSF
3" Root Zone				HS					
FILL, FINE TO COARSE SAND WITH GRAVEL, TRACE SILT & SHELLS, Brown	5		1	SS	14	15			
				HS					
8									
FILL, LEAN CLAY, TRACE ORGANICS WITH SAND SEAMS, Dark Gray	10		2	SS	18	3			
12 ▽				HS					
	15		3	SS	18	1			
				HS					
FILL, FLY ASH WITH SAND SEAMS, Gray	20		4	SS	18	0 WOH			
				HS					
▼									
	25		5	SS	18	0 WOH			
				HS					
	30		6	SS	18	0 WOH			
				HS					

Continued Next Page

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

Calibrated Hand Penetrometer*
CME 140 Lb. Auto. SPT Hammer **

WATER LEVEL OBSERVATIONS (FT.)			
WL	▽ 12	WS	▼
WL	▼ 24	(2 HRS AB)	▼
WL			

Terracon

BORING STARTED	8-29-96		
BORING COMPLETED	8-29-96		
RIG	#37	FOREMAN	REF
APPROVED	AMG	JOB #	06967025

N3ULGE 6/025 9/16/96

LOG OF BORING NO. 6

OWNER	ENGINEER
INTERSTATE POWER COMPANY	HOWARD R. GREEN COMPANY
SITE	PROJECT
LANSING, IOWA	PROPOSED FLY ASH EMBANKMENT

GRAPHIC LOG	DESCRIPTION	DEPTH (FT.)	USCS SYMBOL	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: RECOVERY, IN.	SAMPLES: **SPT - N BLOWS / FT.	TESTS: MOISTURE, %	TESTS: DRY DENSITY PCF	TESTS: UNCONFINED STRENGTH, PSF
	***HIGHLY WEATHERED LIMESTONE WITH CLAY SEAMS, Light Brown	35		7	SS	10	23			
					HS					
		40		8	SS	8	14			
					HS					
43										
	FINE TO MEDIUM SAND, Brown	45		9	SS	17	11			
					HS					
50		50		10	SS	12	35			
	***WEATHERED LIMESTONE WITH SAND POCKETS, Light Gray Brown				HS					
55.5		55		11	SS	16	42			

BOTTOM OF BORING

***Classification estimated from disturbed samples. Core samples and petrographic analysis may reveal other rock types.

NOTE: Material descriptions are based on driller's visual classification only.

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

Calibrated Hand Penetrometer*
CME 140 Lb. Auto. SPT Hammer **

WATER LEVEL OBSERVATIONS (FT.)		
WL	∇ 43	WS ▼ 42 (8/29/96)
WL	∇	▼
WL	WCI @ 43' (8/29/96)	

Terracon

BORING STARTED	8-28-96		
BORING COMPLETED	8-28-96		
RIG	#37	FOREMAN	REF
APPROVED	AMG	JOB #	06967025

N3BLGE 67025 9/16/96

LOG OF BORING NO. 8

OWNER: INTERSTATE POWER COMPANY

ENGINEER: HOWARD R. GREEN COMPANY

SITE: LANSING, IOWA

PROJECT: PROPOSED FLY ASH EMBANKMENT

GRAPHIC LOG	DESCRIPTION	DEPTH (FT.)	USCS SYMBOL	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: RECOVERY, IN.	SAMPLES: **SPT - N BLOWS / FT.	TESTS: MOISTURE, %	TESTS: DRY DENSITY PCF	TESTS: UNCONFINED STRENGTH, PSF
	▽				PA					
9	FILL, SLUICE POND SETTLINGS (FINE FLY ASH), Black	5								
17	FILL, BOTTOM ASH MIXED WITH FINE SAND, Gray	10, 15								
20	SANDY LEAN CLAY, Brown	20								
	BOTTOM OF BORING									

NOTE: Material descriptions are based on driller's visual classification only.

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

Calibrated Hand Penetrometer*
CME 140 Lb. Auto. SPT Hammer **

WATER LEVEL OBSERVATIONS (FT.)			
WL	▽ 1	WD	▼
WL	▽		▼
WL			

Terracon

BORING STARTED	8-29-96		
BORING COMPLETED	8-29-96		
RIG	#37	FOREMAN	REF
APPROVED	AMG	JOB #	06967025

130LGE 67025 9/16/96

LOG OF BORING NO. 7

OWNER	ENGINEER
INTERSTATE POWER COMPANY	HOWARD R. GREEN COMPANY
SITE	PROJECT
LANSING, IOWA	PROPOSED FLY ASH EMBANKMENT

GRAPHIC LOG	DESCRIPTION	DEPTH (FT.)	USCS SYMBOL	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: RECOVERY, IN.	SAMPLES: **SPT - N BLOWS / FT.	TESTS: MOISTURE, %	TESTS: DRY DENSITY PCF	TESTS: UNCONFINED STRENGTH, PSF
35	FILL, FLY ASH WITH SAND SEAMS, Gray									
		35		7	SS	16	8			
38.5	FILL, LEAN CLAY, TRACE SAND & ORGANICS, Dark Brown and Dark Gray				HS					
41	FINE TO COARSE SAND WITH GRAVEL, TRACE LIMESTONE PIECES, Brown	40		8	SS	12	7			
	BOTTOM OF BORING									

NOTE: Material descriptions are based on driller's visual classification only.

- WOH refers to Weight of Hammer.

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

Calibrated Hand Penetrometer*
CME 140 Lb. Auto. SPT Hammer **

WATER LEVEL OBSERVATIONS (FT.)		
WL	∇ 12	WS ∇
WL	∇ 24	(2 HRS AB) ∇
WL		

Terracon

BORING STARTED	8-29-96
BORING COMPLETED	8-29-96
RIG #37	FOREMAN REF
APPROVED AMG	JOB # 06967025

N3BLGE 67025 9/16/96

LOG OF BORING NO. 10

OWNER	ENGINEER
INTERSTATE POWER COMPANY	HOWARD R. GREEN COMPANY
SITE	**PROJECT**
LANSING, IOWA	PROPOSED FLY ASH EMBANKMENT

GRAPHIC LOG (depth)	DESCRIPTION	DEPTH (FT.)	USCS SYMBOL	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: RECOVERY, IN.	SAMPLES: **SPT - N BLOWS / FT.	TESTS: MOISTURE, %	TESTS: DRY DENSITY PCF	TESTS: UNCONFINED STRENGTH, PSF
7	FILL, ASH, BTM, TRACE HYDRATED FLY ASH, COARSE (GRAVELLY), Black				PA					
8	FILL, SILTY FINE FLY ASH									
20	FILL, GOOPY FINE FLY ASH, Black									
	BOTTOM OF BORING	20								

NOTE: Material descriptions are based on driller's visual classification only.

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

Calibrated Hand Penetrometer*
CME 140 Lb. Auto. SPT Hammer **

WATER LEVEL OBSERVATIONS (FT.)		
WL	∇ 2	WD ▼
WL	▼	▼
WL		

Terracon

BORING STARTED	8-29-96		
BORING COMPLETED	8-29-96		
RIG	#37	FOREMAN	REF
APPROVED	AMG	JOB #	06967025

LOG OF BORING NO. 9

OWNER	ENGINEER
INTERSTATE POWER COMPANY	HOWARD R. GREEN COMPANY
SITE	PROJECT
LANSING, IOWA	PROPOSED FLY ASH EMBANKMENT

GRAPHIC LOG	DESCRIPTION	DEPTH (FT.)	USCS SYMBOL	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: RECOVERY, IN.	SAMPLES: **SPT - N BLOWS / FT.	TESTS: MOISTURE, %	TESTS: DRY DENSITY PCF	TESTS: UNCONFINED STRENGTH, PSF	
	∇ FILL, SLUICE POND SETTLINGS BOTTOM ASH, SAND & HYDRATED FLY ASH, Black, Brown and Gray	0–11			PA						
11	FILL, SILTY BOTTOM ASH (LESS COARSE), Black	11–20									
20		20									

NOTE: Material descriptions are based on driller's visual classification only.

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

Calibrated Hand Penetrometer*
CME 140 Lb. Auto. SPT Hammer **

WATER LEVEL OBSERVATIONS (FT.)		
WL	∇ 1	WD ▼
WL	∇	▼
WL		

Terracon

BORING STARTED	8-29-96		
BORING COMPLETED	8-29-96		
RIG	#37	FOREMAN	REF
APPROVED	AMG	JOB #	06967025

H30LGE 67025 9/16/96

LOG OF BORING NO. 11

OWNER	ENGINEER
INTERSTATE POWER COMPANY	HOWARD R. GREEN COMPANY
SITE	**PROJECT**
LANSING, IOWA	PROPOSED FLY ASH EMBANKMENT

GRAPHIC LOG	DESCRIPTION	DEPTH (FT.)	USCS SYMBOL	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: RECOVERY, IN.	SAMPLES: **SPT - N BLOWS / FT.	TESTS: MOISTURE, %	TESTS: DRY DENSITY PCF	TESTS: UNCONFINED STRENGTH, PSF
	▽	0			PA					
	FILL, HYDRATED FLY ASH (GRAVELLY TEXTURE), Black and Brown	5								
	- Finer goopy fly ash @ about 7 to 13 feet.	10								
		15								
17	SANDY LEAN CLAY, Brown									
20	BOTTOM OF BORING	20								

NOTE: Material descriptions are based on driller's visual classification only.

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

Calibrated Hand Penetrometer*
CME 140 Lb. Auto. SPT Hammer **

WATER LEVEL OBSERVATIONS (FT.)		
WL	▽ 0.5	WD ▼
WL	▽	▼
WL		

Terracon

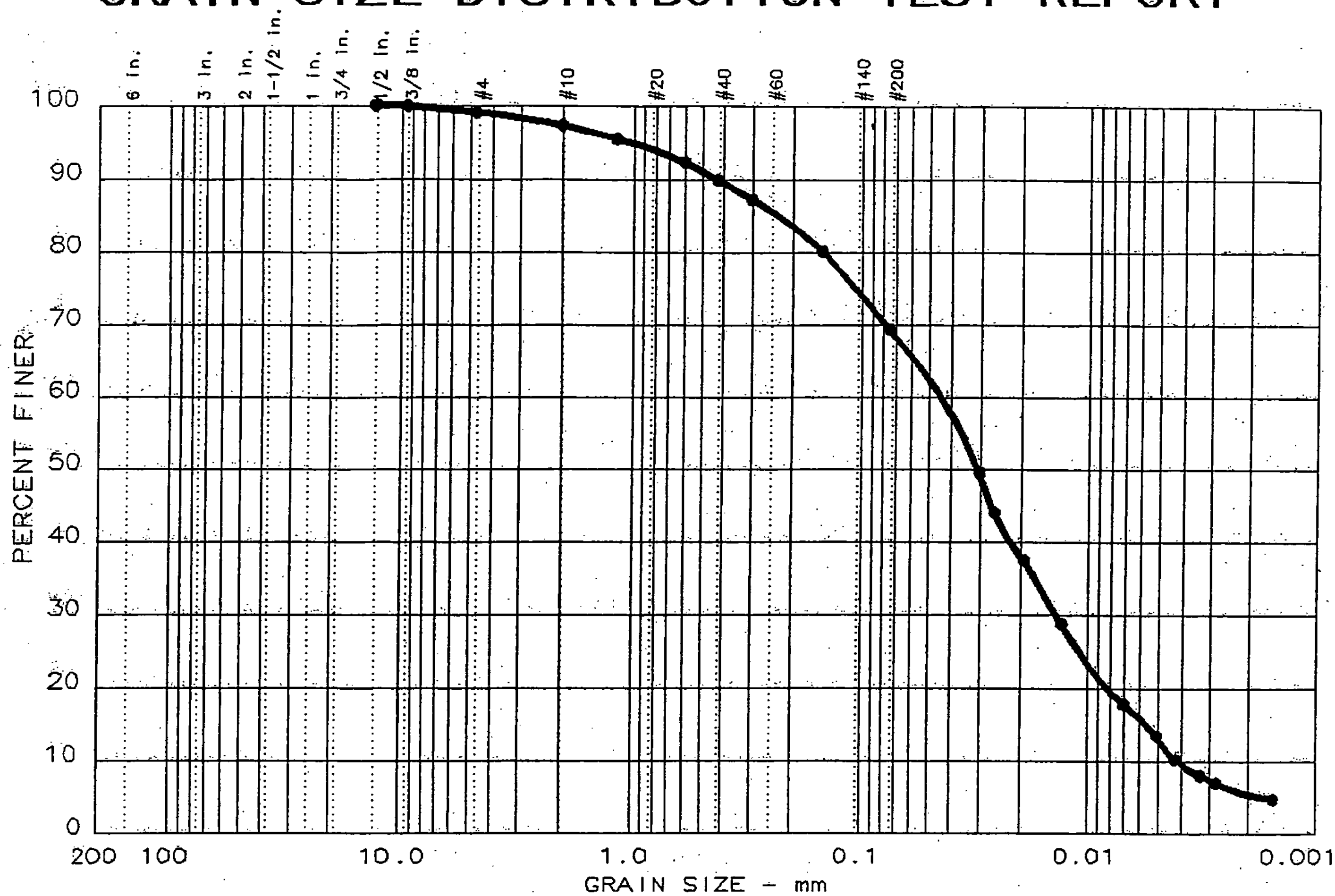
BORING STARTED			8-29-96
BORING COMPLETED			8-29-96
RIG	#37	FOREMAN	REF
APPROVED	AMG	JOB #	06967025

N30LGE 67025 9/16/96

ATTACHMENT F

LABORATORY TEST RESULTS

GRAIN SIZE DISTRIBUTION TEST REPORT



	Test	% +3"	% GRAVEL	% SAND	% SILT	% CLAY
●	20	0.0	0.9	29.8	56.2	13.1

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●	54	6	0.232		0.031	0.014	0.0055	0.0042	1.03	10.6

MATERIAL DESCRIPTION	USCS	AASHTO
● Black SILT, Some Sand&Clay, Trace Gravel (Fly Ash)	ML/MH	A-5

Project No.: CGC# 20207.00
Project: Lansing Fly Ash
● Location: Pail: Black

Date: November 30, 2000

Remarks:
Tested By: D Arenander
Input By : D Arenander
Checked By : M Schultz
Approved By :D Arenander

GRAIN SIZE DISTRIBUTION TEST REPORT
CGC, Inc.

Figure No. ______

GRAIN SIZE DISTRIBUTION TEST DATA

Test No.: 20

Date: November 30, 2000
Project No.: CGC# 20207.00
Project: Lansing Fly Ash

Sample Data

Location of Sample: Pail: Black
Sample Description: Black SILT,Some Sand&Clay,Trace Gravel(Fly Ash)
USCS Class: ML/MH
Liquid limit: 54
AASHTO Class: A-5
Plasticity index: 6

Notes

Remarks: Tested By: D Arenander Input By : D Arenander
Checked By : M Schultz Approved By :D Arenander
Fig. No.:

Mechanical Analysis Data

	Initial
Dry sample and tare=	762.06
Tare =	0.00
Dry sample weight =	762.06

Tare for cumulative weight retained= 0

Sieve	Cumul. Wt. retained	Percent finer
0.5 inches	0.00	100.0
0.375 inches	1.00	99.9
# 4	6.80	99.1
# 10	20.60	97.3
# 16	34.20	95.5
# 30	58.70	92.3
# 40	76.90	89.9
# 50	97.60	87.2
# 100	151.30	80.1
# 200	233.60	69.3

Hydrometer Analysis Data

Separation sieve is number 10
Percent -# 10 based on complete sample= 97.3
Weight of hydrometer sample: 100
Hygroscopic moisture correction:
Moist weight & tare = 852.30
Dry weight & tare = 851.90
Tare = 190.00
Hygroscopic moisture= 0.1 %
Calculated biased weight= 102.72

utomatic temperature correction
Composite correction at 20 deg C =-6

eniscus correction only= 0
pecific gravity of solids= 2.25
pecific gravity correction factor= 1.121
ydrometer type: 152H   Effective depth L= 16.294964 - 0.164 x Rm

Elapsed time, min	Temp, deg C	Actual reading	Corrected reading	K	Rm	Eff. depth	Diameter mm	Percent finer
2.0	22.0	51.0	45.4	0.0153	51.0	7.9	0.0305	49.5
3.0	22.0	46.0	40.4	0.0153	46.0	8.8	0.0261	44.1
6.0	22.0	40.0	34.4	0.0153	40.0	9.7	0.0195	37.5
15.0	22.0	32.0	26.4	0.0153	32.0	11.0	0.0131	28.8
60.0	22.0	22.0	16.4	0.0153	22.0	12.7	0.0070	17.9
120.0	22.0	18.0	12.4	0.0153	18.0	13.3	0.0051	13.5
180.0	22.0	15.0	9.4	0.0153	15.0	13.8	0.0042	10.3
300.0	22.0	13.0	7.4	0.0153	13.0	14.2	0.0033	8.1
420.0	22.0	12.0	6.4	0.0153	12.0	14.3	0.0028	7.0
1440.0	22.0	10.0	4.4	0.0153	10.0	14.7	0.0015	4.8

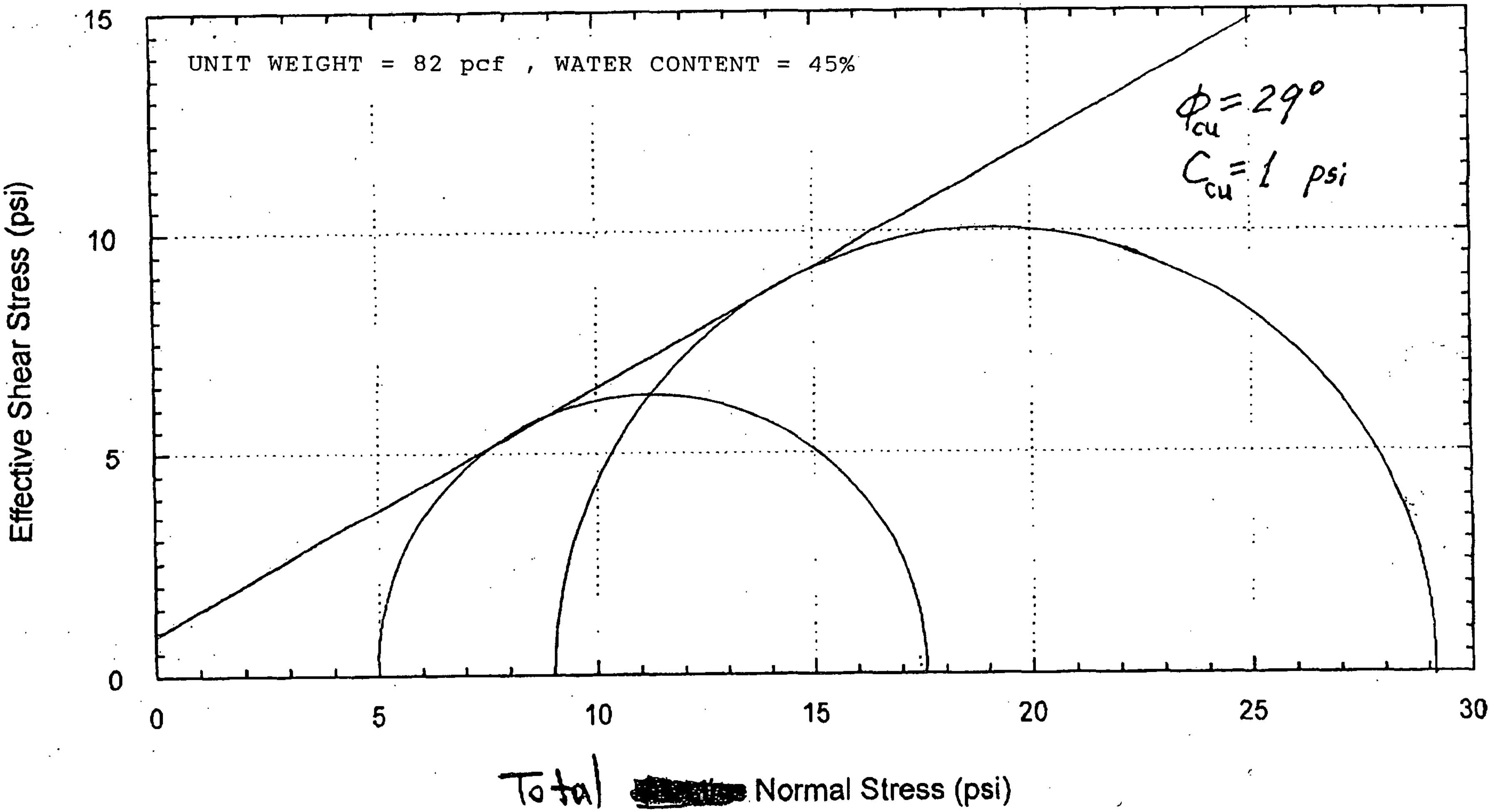
Fractional Components

ravel/Sand based on #4 sieve
and/Fines based on #200 sieve
 + 3 in. = 0.0   % GRAVEL = 0.9   % SAND = 29.8
 SILT = 56.2   % CLAY = 13.1

85= 0.23 D60= 0.044 D50= 0.031
30= 0.0138 D15= 0.00555 D10= 0.00416
c = 1.0340 Cu = 10.6047

ALLIANT FLY ASH

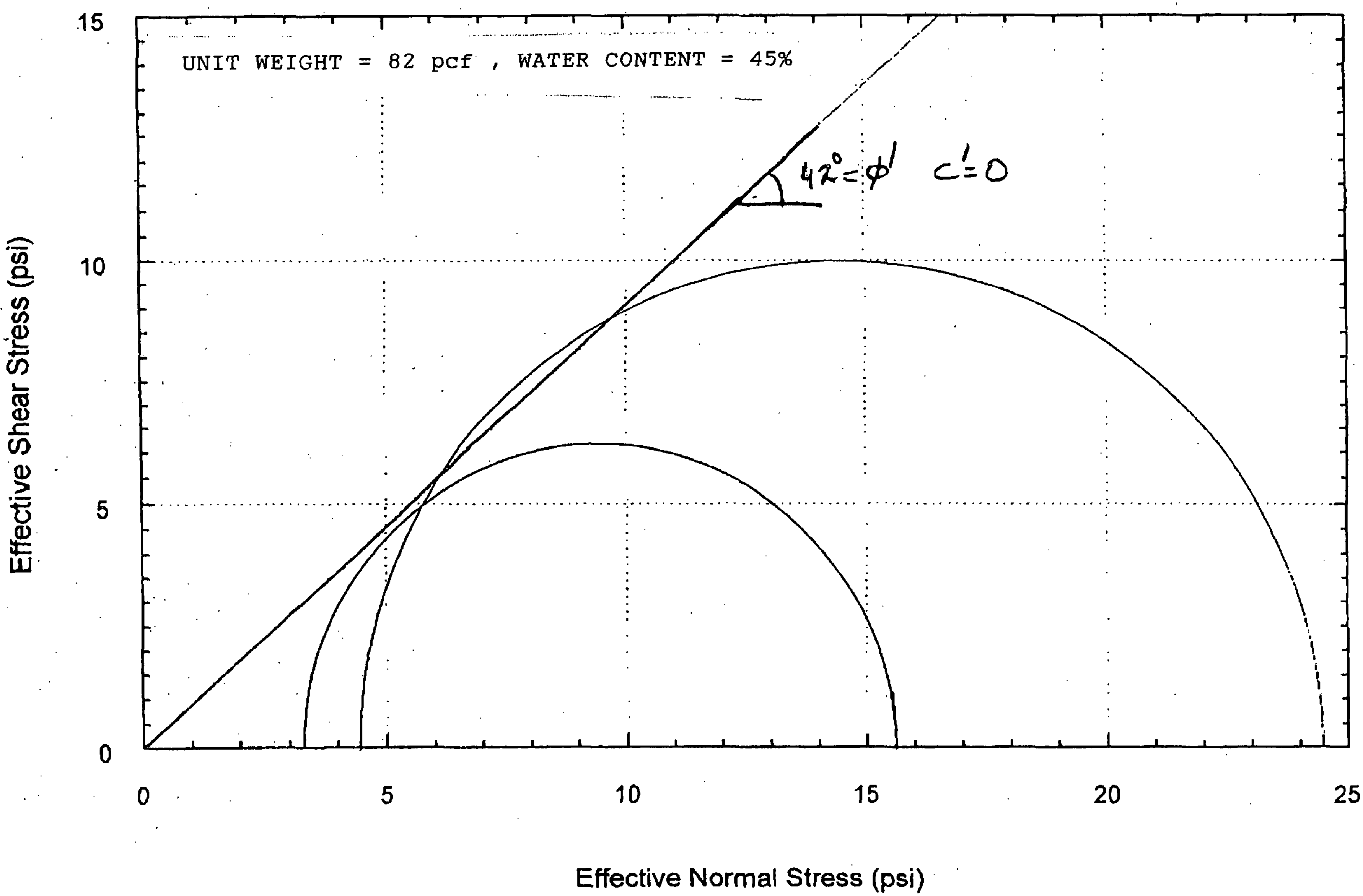
CONSOLIDATED-UNDRAINED TEST



PREPARED BY UW-MADISON
GEOTECHNICAL LABORATORY

ALLIANT FLY ASH

CONSOLIDATED-UNDRAINED TEST



PREPARED BY UW-MADISON

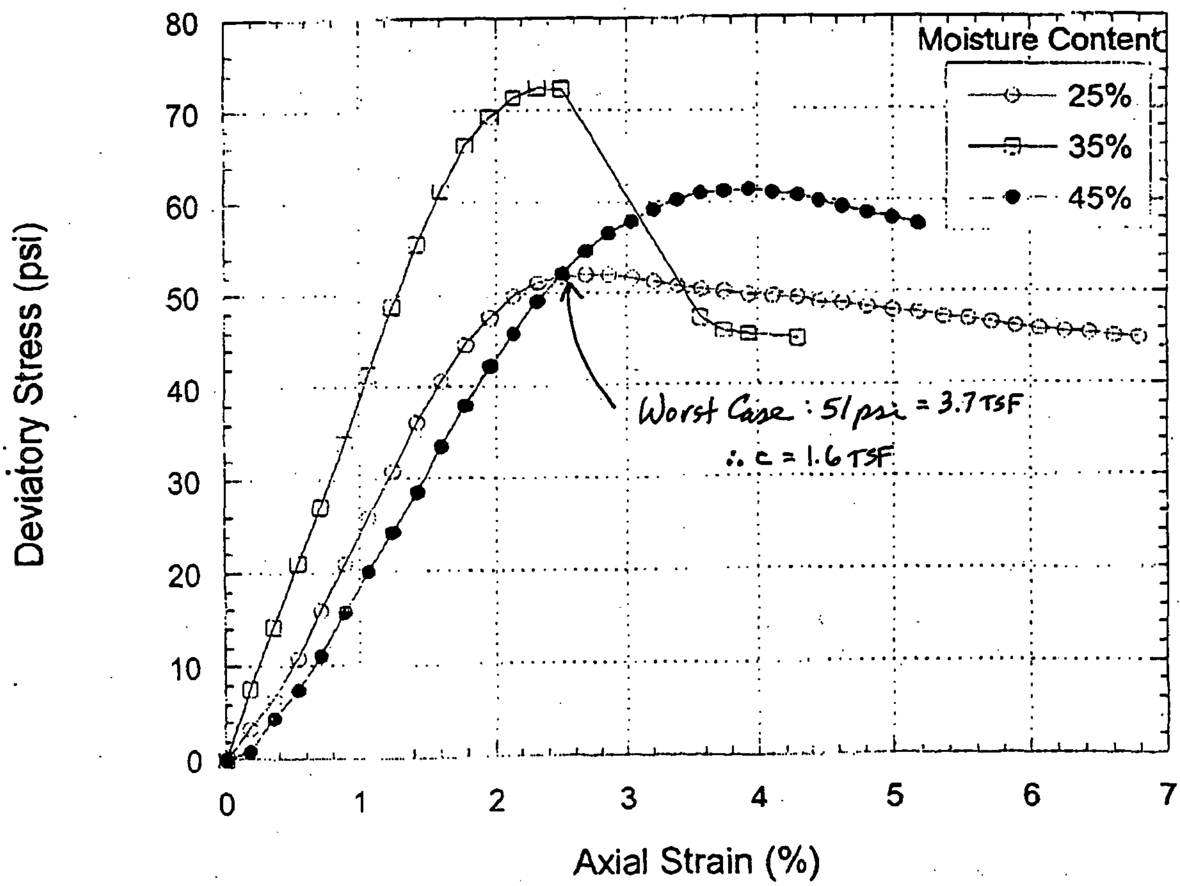
12-10-1995 7:34AM FROM

Post-it® Fax Note 7671	Date	# of pages ▶
To Mike Schultz	From T. Edil	
Co./Dept.	Co.	
Phone #	Phone #	
Fax # 298-7887	Fax #	

PREPARED BY UW-MADISON
GEOTECHNICAL LABORATORY

UNCONSOLIDATED-UNDRAINED TESTS

UU Test on FlyAsh
Minor Principal Stress = 7 psi

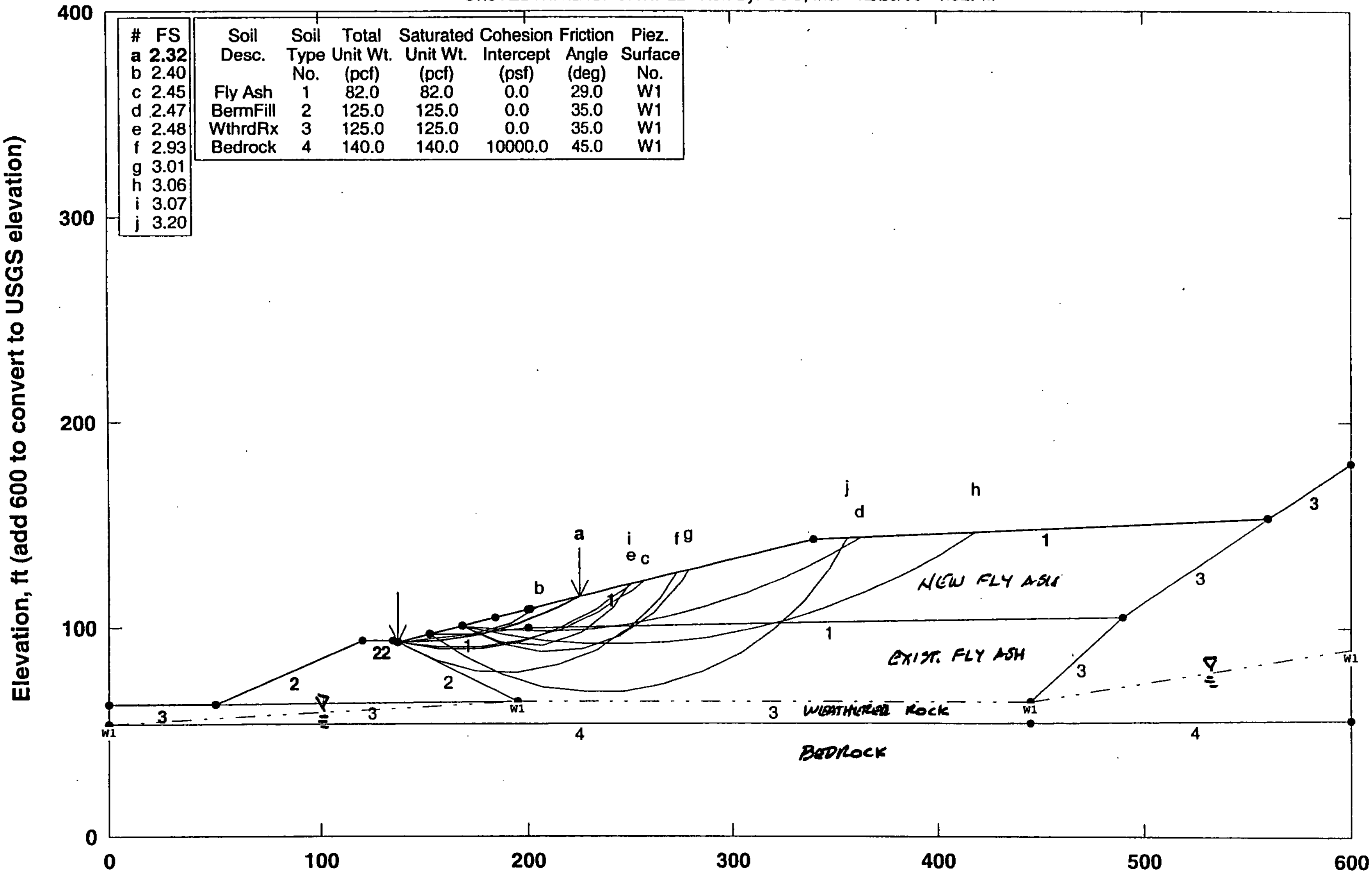


ATTACHMENT G

SLOPE STABILITY ANALYSES
• FIGURES G-1 THROUGH G-6

Alliant - Fly Ash Landfill Lansing, IA

C:\STEDWIN\LNSNGAT.PL2 Run By: CGC, Inc. 12/28/00 1:52PM



PCSTABL5M/si FSmin=2.32

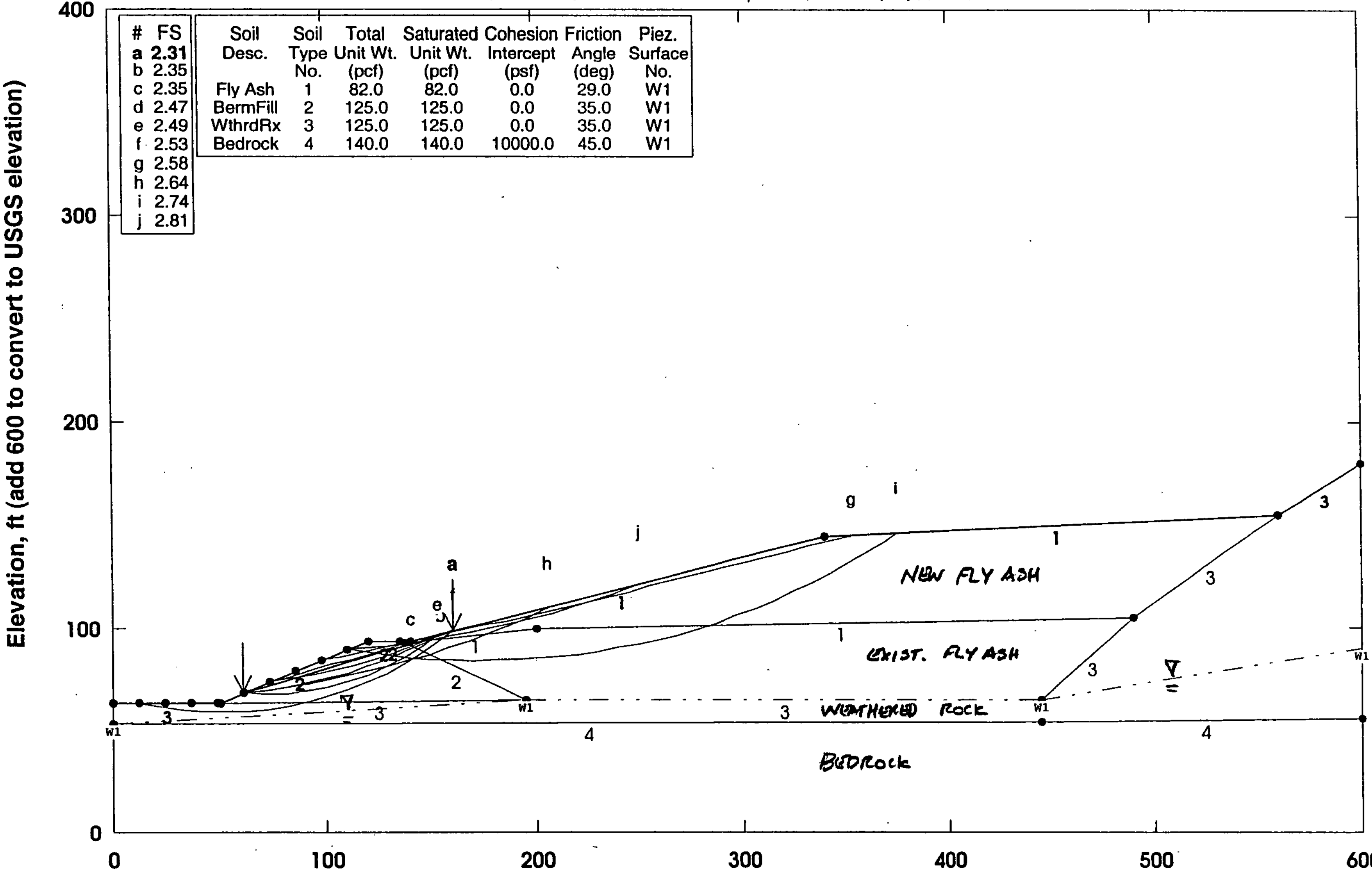
Safety Factors Are Calculated By The Modified Bishop Method

CGC, Inc.

Figure G-1
Failure through Ash Slope
Typical Earth Berm Strength Parameters

Alliant - Fly Ash Landfill Lansing, IA

C:\STEDWIN\LNSNGBT.PL2 Run By: CGC, Inc. 12/28/00 1:46PM

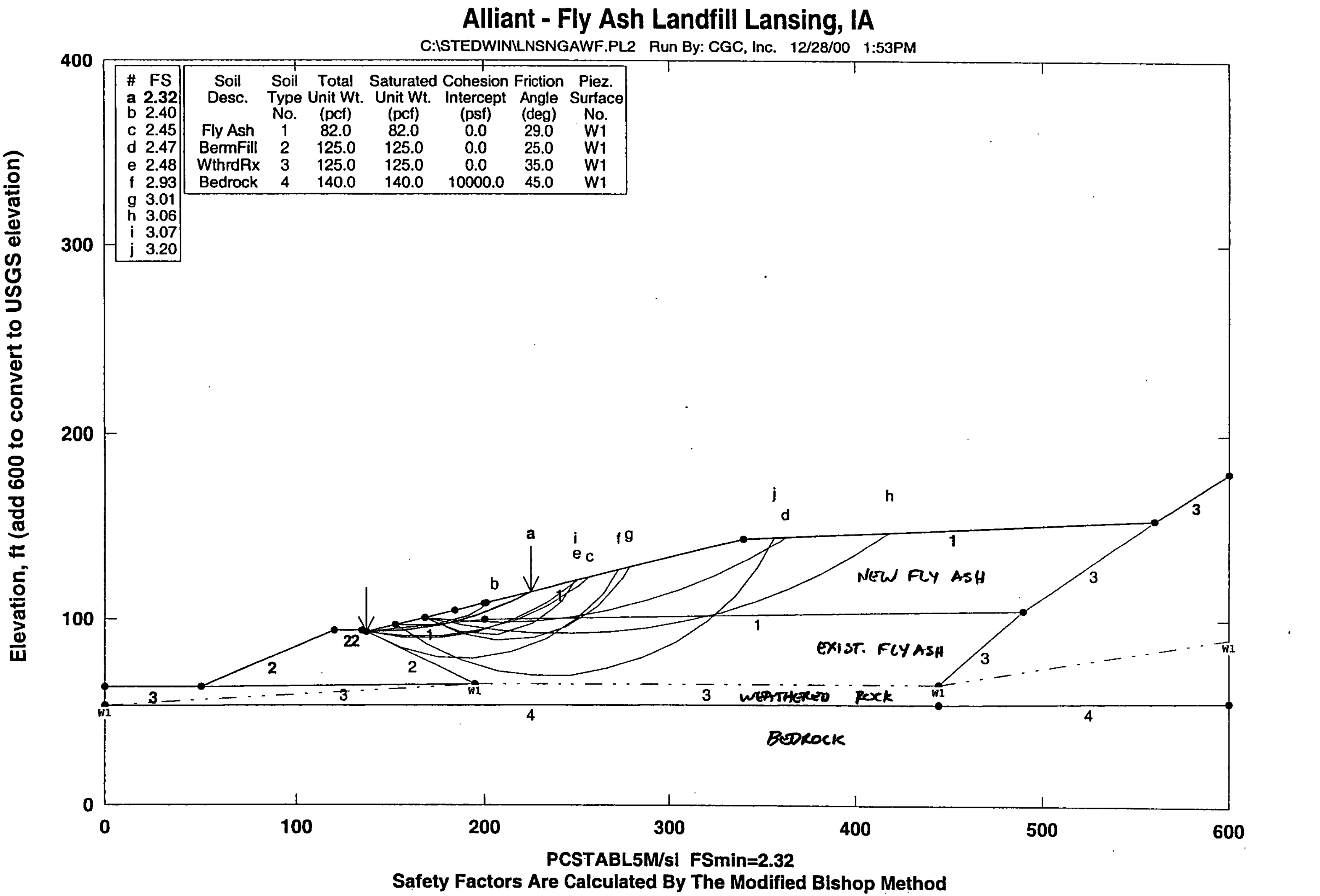


PCSTABL5M/si FSmin=2.31

Safety Factors Are Calculated By The Modified Bishop Method

CGC, Inc.

Figure G-2
Failure through Earth Berm
Typical Earth Berm Strength Parameters

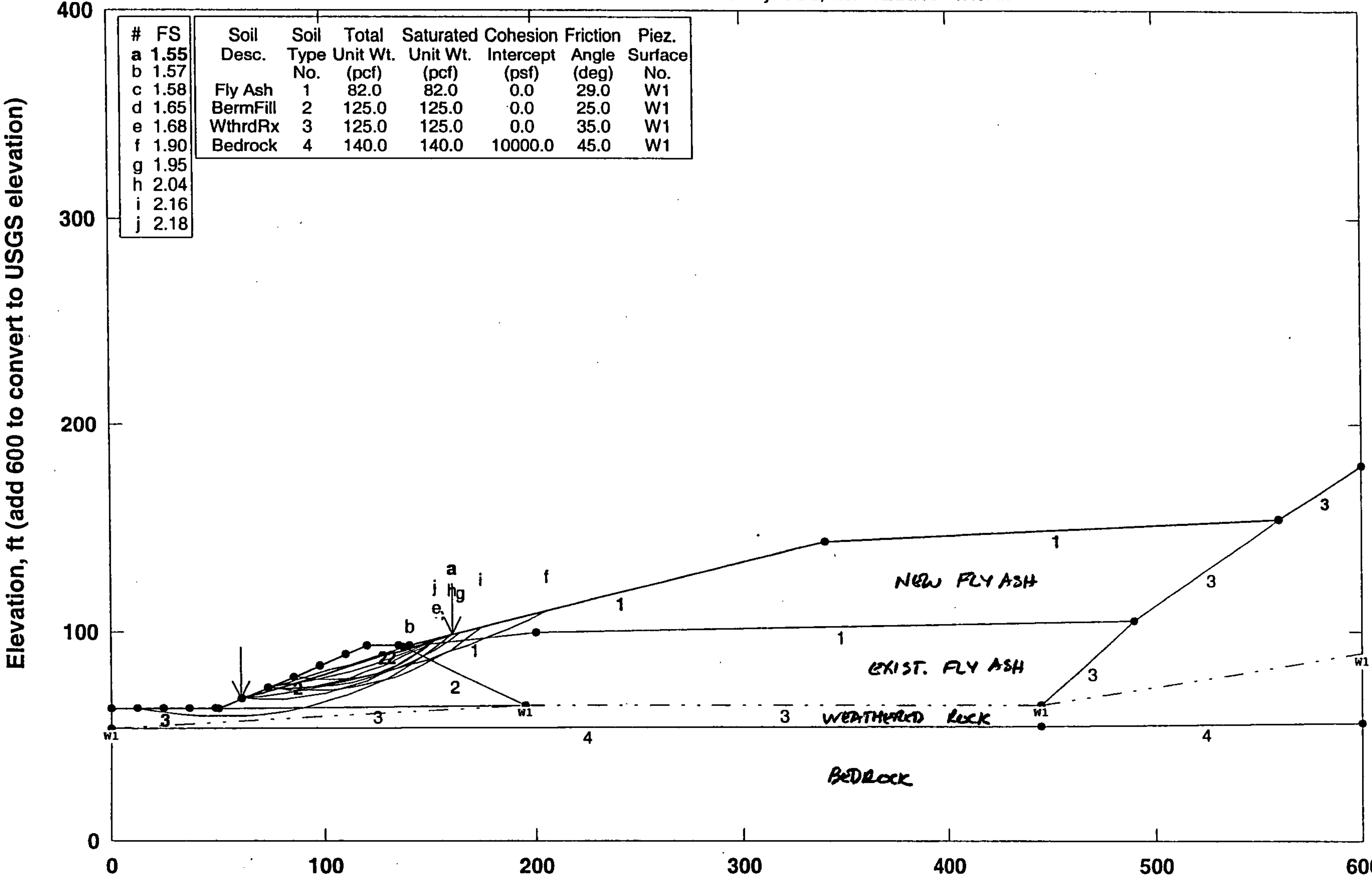


CGC, Inc.

Figure G-3
Failure through Ash Slope
"Weak" Earth Berm Strength Parameters
(Frictional Material)

Alliant - Fly Ash Landfill Lansing, IA

C:\STEDWIN\LNSNGBWF.PL2 Run By: CGC, Inc. 12/28/00 1:48PM



PCSTABL5M/sl FSmin=1.55

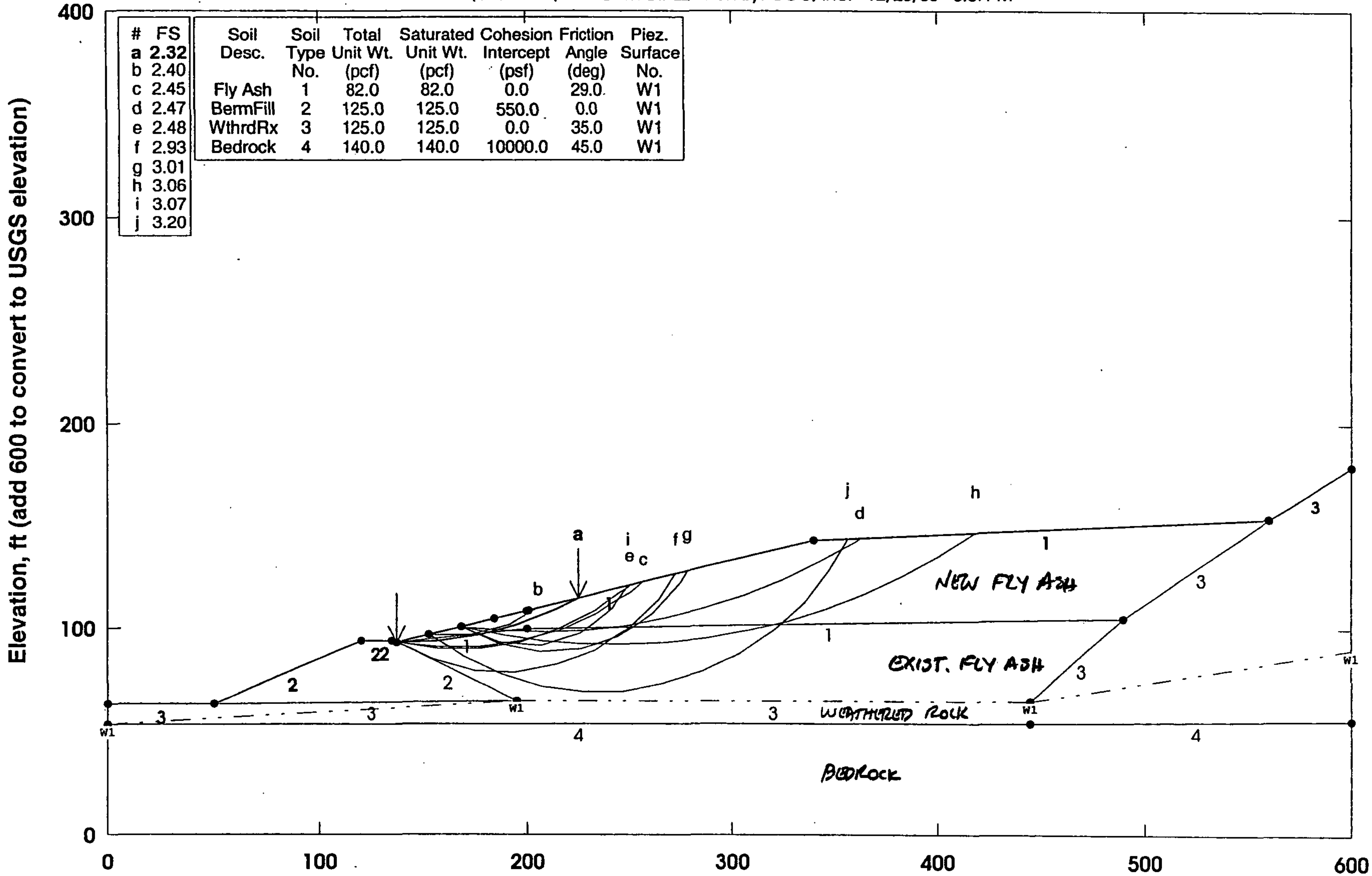
Safety Factors Are Calculated By The Modified Bishop Method

CGC, Inc.

Figure G-4
Failure through Earth Berm
"Weak" Earth Berm Strength Parameters
(Frictional Material)

Alliant - Fly Ash Landfill Lansing, IA

C:\STEDWIN\LNSNGAWC.PL2 Run By: CGC, Inc. 12/28/00 3:57PM



#	FS
a	2.32
b	2.40
c	2.45
d	2.47
e	2.48
f	2.93
g	3.01
h	3.06
i	3.07
j	3.20

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fly Ash	1	82.0	82.0	0.0	29.0	W1
BermFill	2	125.0	125.0	550.0	0.0	W1
WthrdRx	3	125.0	125.0	0.0	35.0	W1
Bedrock	4	140.0	140.0	10000.0	45.0	W1

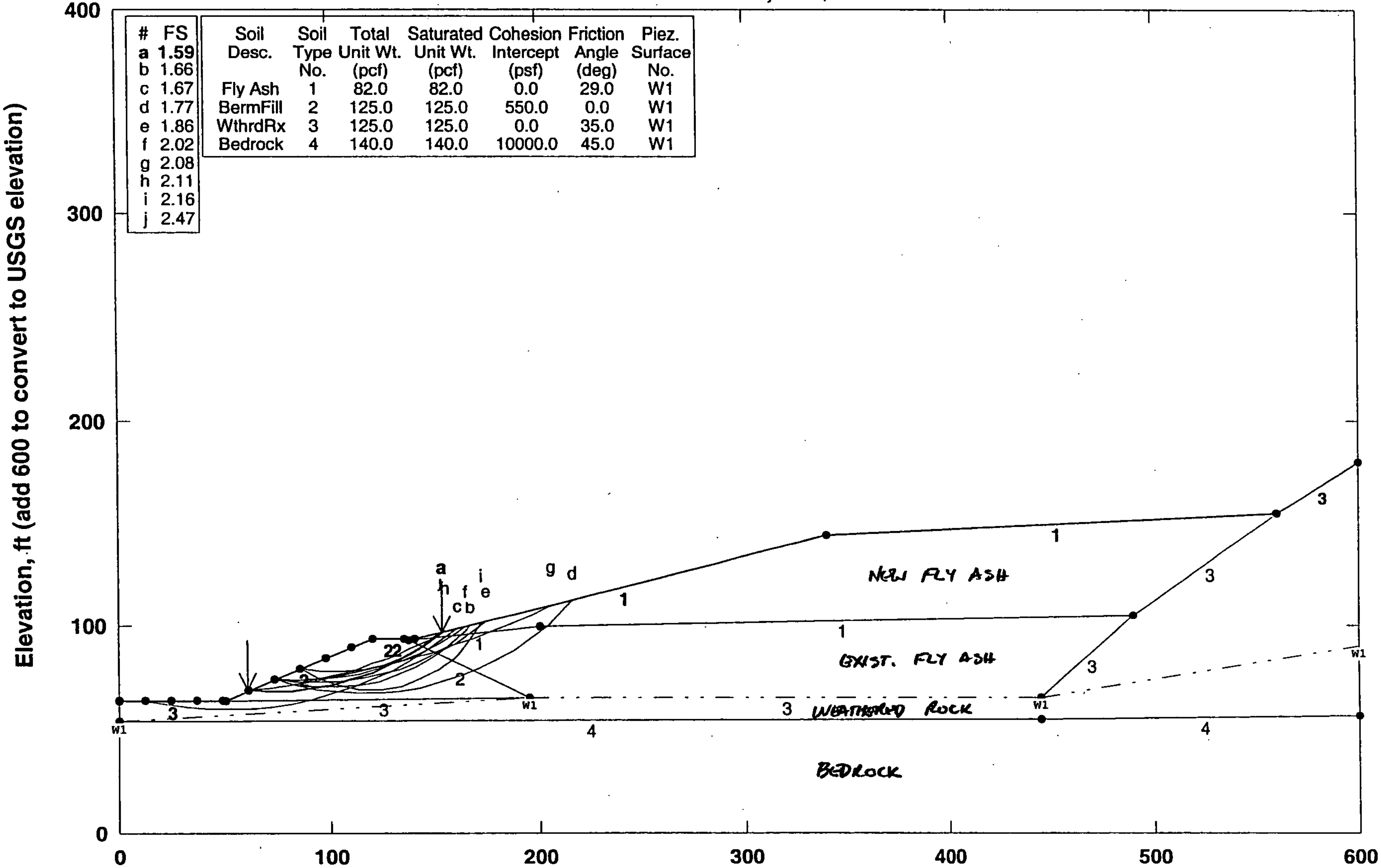
PCSTABL5M/si FSmin=2.32
Safety Factors Are Calculated By The Modified Bishop Method

CGC, Inc.

Figure G-5
Failure through Ash Slope
"Weak" Earth Berm Strength Paramete
(Cohesive Material)

Alliant - Fly Ash Landfill Lansing, IA

C:\STEDWIN\LNSNGBWC.PL2 Run By: CGC, Inc. 12/28/00 4:00PM



PCSTABL5M/si FSmin=1.59

Safety Factors Are Calculated By The Modified Bishop Method

CGC, Inc.

Figure G-6
Failure through Earth Berm
"Weak" Earth Berm Strength Parameter
(C[redacted]ive[redacted]eria[redacted]

ATTACHMENT H

DOCUMENT QUALIFICATIONS

ATTACHMENT H
DOCUMENT QUALIFICATIONS
I. GENERAL RECOMMENDATIONS/LIMITATIONS

CGC, Inc. should be provided the opportunity for a general review of the final design and specifications to confirm that earthwork and foundation requirements have been properly interpreted in the design and specifications. CGC should be retained to provide soil engineering services during excavation and subgrade preparation. This will allow us to observe that construction proceeds in compliance with the design concepts, specifications and recommendations, and also will allow design changes to be made in the event that subsurface conditions differ from those anticipated prior to the start of construction. CGC does not assume responsibility for compliance with the recommendations in this report unless we are retained to provide construction testing and observation services.

This report has been prepared in accordance with generally accepted soil and foundation engineering practices and no other warranties are expressed or implied. The opinions and recommendations submitted in this report are based on interpretation of the subsurface information revealed by the test borings indicated on the location plan. The report does not reflect potential variations in subsurface conditions between or beyond these borings. Therefore, variations in soil conditions can be expected between the boring locations and fluctuations of groundwater levels may occur with time. The nature and extent of the variations may not become evident until construction.

II. IMPORTANT INFORMATION
ABOUT YOUR
GEOTECHNICAL ENGINEERING REPORT

More construction problems are caused by site subsurface conditions than any other factor. As troublesome as subsurface problems can be, their frequency and extent have been lessened considerably in recent years, due in large measure to programs and publications of ASFE/The Association of Engineering Firms Practicing in the Geosciences.

The following suggestions and observations are offered to help you reduce the geotechnical-related delays, cost-overruns and other costly headaches that can occur during a construction project.

A GEOTECHNICAL ENGINEERING REPORT IS BASED ON A UNIQUE SET OF PROJECT-SPECIFIC FACTORS

A CGC geotechnical report is based on a subsurface exploration plan designed to incorporate a unique set of project-specific factors. These typically include: the general nature of the structure involved, its size and configuration; the location of the structure on the site and its orientation; physical concomitants such as access roads, parking lots, and underground utilities, and the level of additional risk which the client assumed by virtue of limitations imposed upon the exploratory program. To help avoid costly problems, consult CGC's geotechnical engineers to determine how any factors which change subsequent to the date of the report may affect its recommendations.

Unless CGC indicates otherwise, *your geotechnical engineering report should not be used:*

- When the nature of the proposed structure is changed, for example, if an office building will be erected instead of a parking garage, or if a refrigerated warehouse will be built instead of an unrefrigerated one;
- when the size or configuration of the proposed structure is altered;
- when the location or orientation of the proposed structure is modified;
- when there is a change of ownership, or
- for application to an adjacent site.

CGC geotechnical engineers cannot accept responsibility for problems which may develop if they are not consulted after factors considered in their report's development have changed.

MOST GEOTECHNICAL "FINDINGS" ARE PROFESSIONAL ESTIMATES

Site exploration identifies actual subsurface conditions only at those points where samples are taken, when they are taken. Data derived through sampling and subsequent laboratory testing are extrapolated by geotechnical engineers who then render an opinion about overall subsurface conditions, their likely reaction to proposed construction activity, and appropriate foundation design. Even under optimal circumstances, actual conditions may differ from those inferred to exist, because no geotechnical engineer, no matter how qualified, and no subsurface exploration program, no matter how comprehensive, can reveal what is hidden by earth, rock and time. The actual interface between materials may be far more gradual or abrupt than a report indicates. Actual conditions in areas not sampled may differ from predictions. *Nothing can be done to prevent the unanticipated, but steps can be taken to help minimize their impact.* For this reason, *most experienced owners retain their geotechnical consultants through the construction stage,* to identify variances, conduct additional tests which may be needed, and to recommend solutions to problems encountered on site.

SUBSURFACE CONDITIONS CAN CHANGE

Subsurface conditions may be modified by constantly-changing natural forces. Because a geotechnical engineering report is based on conditions which existed at the time of subsurface exploration, *construction decisions* *should not be based on a geotechnical engineering report whose adequacy may have been affected by time.* Speak with CGC's geotechnical consultant to learn if additional tests are advisable before construction starts.

Construction operations at or adjacent to the site and natural events such as floods, earthquakes or groundwater fluctuations may also affect subsurface conditions and, thus, the continuing adequacy of a geotechnical report. CGC's geotechnical engineer should be kept apprised of any such events, and should be consulted to determine if additional tests are necessary.

GEOTECHNICAL SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES AND PERSONS

CGC geotechnical reports are prepared to meet the specific needs of specific individuals. A report prepared for a consulting civil engineer may not be adequate for a construction contractor, or even some other consulting civil engineer. Unless indicated otherwise, this report was prepared expressly for the client involved and expressly for purposes indicated by the client. Use by any other persons for any purpose, or by the client for a different purpose, may result in problems. *No individual other than the client should apply this report for its intended purpose without first conferring with the geotechnical engineer. No person should apply this report for any purpose other than that originally contemplated without first conferring with the geotechnical engineer.*

A GEOTECHNICAL ENGINEERING REPORT IS SUBJECT TO MISINTERPRETATION

Costly problems can occur when other design professionals develop their plans based on misinterpretations of a geotechnical engineering report. To help avoid these problems, CGC's geotechnical engineer should be retained to work with other appropriate design professionals to explain relevant geotechnical findings and to review the adequacy of their plans and specifications relative to geotechnical issues.

BORING LOGS SHOULD NOT BE SEPARATED FROM THE ENGINEERING REPORT

Final boring logs are developed by CGC engineers based upon their interpretation of field logs (assembled by site personnel) and laboratory evaluation of field samples. Only final boring logs customarily are included in geotechnical engineering reports. *These logs should not under any circumstances be redrawn* for inclusion in architectural or other design drawings, because drafters may commit errors or omissions in the transfer process. Although photographic reproduction eliminates this problem, it does nothing to minimize the possibility of contractors misinterpreting the logs during bid preparation. When this occurs, delays, disputes and unanticipated costs are the all-to-frequent result.

To minimize the likelihood of boring log misinterpretation, *give contractors ready access to the complete geotechnical engineering report* prepared or authorized for their use. Those who do not provide such access may proceed under the *mistaken* impression that simply disclaiming responsibility for the accuracy of subsurface information always insulates them from attendant liability. Providing the best available information to contractors helps prevent costly construction problems and the adversarial attitudes which aggravate them to disproportionate scale.

READ RESPONSIBILITY CLAUSES CLOSELY

Because geotechnical engineering is based extensively on judgment and opinion, it is far less exact than other design disciplines. This situation has resulted in wholly unwarranted claims being lodged against geotechnical consultants. To help prevent this problem, CGC geotechnical engineers have developed model clauses for use in written transmittals. These are not exculpatory clauses designed to foist our geotechnical engineers' liabilities onto someone else. Rather, they are definitive clauses which identify where our geotechnical engineers' responsibilities begin and end. Their use helps all parties involved recognize their individual responsibilities and take appropriate action. Some of these definitive clauses are likely to appear in your geotechnical engineering report, and you are encouraged to read them closely. CGC's geotechnical engineers will be pleased to give full and frank answers to your questions.

OTHER STEPS YOU CAN TAKE TO REDUCE RISK

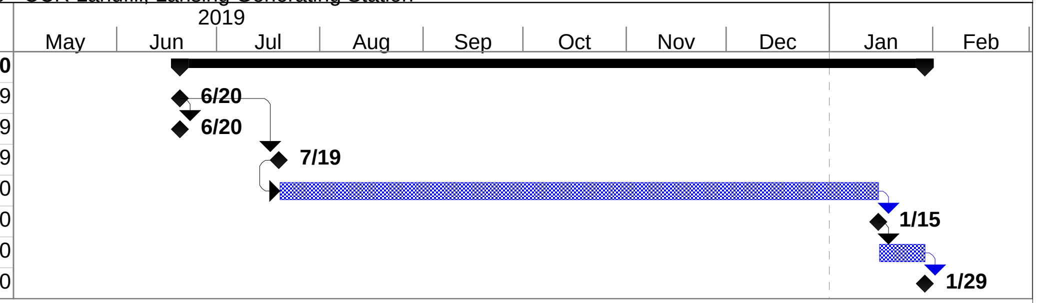
CGC's geotechnical engineers will be pleased to discuss other techniques which can be employed to mitigate risk. In addition, ASFE has developed a variety of materials which may be beneficial. Contact ASFE for a complimentary copy of its publications directory.

APPENDIX B

Closure Schedule

Initial Closure Plan Schedule - CCR Landfill, Lansing Generating Station

ID	Task Name	Duration	Start	Finish
1	**Entire CCR Unit Closure**	**224 days**	**Thu 6/20/19**	**Wed 1/29/20**
2	Ash filling ceases	0 days	Thu 6/20/19	Thu 6/20/19
3	Other regulatory permits - none	0 days	Thu 6/20/19	Thu 6/20/19
4	Notification of Intent to Close	0 days	Fri 7/19/19	Fri 7/19/19
5	Construction	180 days	Sat 7/20/19	Wed 1/15/20
6	Notification of Closure Completion	0 days	Wed 1/15/20	Wed 1/15/20
7	Documentation Report	14 days	Thu 1/16/20	Wed 1/29/20
8	State Submittal:Documentation Report	0 days	Wed 1/29/20	Wed 1/29/20



Project: Closure Plan
Date: Wed 9/21/16

Task		Inactive Milestone		Finish-only	
Split		Inactive Summary		External Tasks	
Milestone		Manual Task		External Milestone	
Summary		Duration-only		Progress	
Project Summary		Manual Summary Rollup		Deadline	
External Tasks		Manual Summary			
External Milestone		Start-only			