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REPORT ON
INITIAL SAFETY FACTOR ASSESSMENT
CELL 002 WEST
THOMAS HILL ENERGY CENTER
CLIFTON HILL, MISSOURI

by Haley & Aldrich, Inc.
Cleveland, Ohio

for Associated Electric Cooperative, Inc.
Springfield, Missouri

File No. 128064-006
April 2018

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17 April 2018
File No. 128064-006

Associated Electric Cooperative, Inc.
2814 South Golden Avenue
P.O. Box 754
Springfield, Missouri 65801

Attention: Kim Dickerson
Senior Environmental Analyst

Subject: Report on Initial Safety Factor Assessment
Cell 002 West
Thomas Hill Energy Center
Clifton Hill, Missouri

Ms. Dickerson:

We are pleased to submit herewith our report entitled, "Report on Initial Safety Factor Assessment, Cell 002 West, Thomas Hill Energy Center, Clifton Hill, Missouri." This report includes background information regarding the project, the results of our field investigation program, and the results of our initial safety factor assessment.

This work was performed by Haley & Aldrich, Inc. (Haley & Aldrich) on behalf of Associated Electric Cooperative, Inc. (AECI) in accordance with the United States Environmental Protection Agency's Hazardous and Solid Waste Management System; Disposal of Coal Combustion Residuals from Electric Utilities, 40 CFR Part 257, specifically §257.73(e). Based on the USEPA's issued CCR Rule Partial Vacatur in 2016, the inactive Cell 002 West impoundment at the THEC is subject to applicable requirements of the CCR Rule. The safety factor assessment discussed herein has been referred to as an "initial" assessment to coincide with the terminology used in §257.73(e) and §257.73(f) to distinguish it from the "periodic" assessments that are required every five years following the "initial" assessment has been completed.

The scope of our work in our initial safety factor assessment consisted of the following: 1) reviewing readily available reports, investigations, plans and data pertaining to the surface impoundment; 2) performing engineering evaluations related to seismic response analysis, liquefaction and slope stability; and 3) preparing and submitting this report presenting the results of our assessment.

Thank you for inviting us to complete this assessment and please feel free to contact us if you wish to discuss the contents of the report.

Sincerely yours,
HALEY & ALDRICH, INC.

Derrick A. Shelton
Geotechnical Program Manager | Senior Associate

Steven F. Putrich, P.E.
Principal

Enclosures

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1. Introduction

1.1 GENERAL

Haley & Aldrich, Inc. (Haley & Aldrich) has been contracted by Associated Electric Cooperative, Inc. (AECI) to perform the Initial Safety Factor Assessment for Cell 002 West located at Thomas Hill Energy Center in Clifton Hill, Missouri. This work was completed in accordance with the United States Environmental Protection Agency's (EPA's) Hazardous and Solid Waste Management System; Disposal of Coal Combustion Residuals (CCR) from Electric Utilities, 40 CFR Part 257, specifically §257.73(e) (EPA, 2015).

1.2 PURPOSE OF SAFETY FACTOR ASSESSMENT

The purpose of this study was to evaluate the subsurface soil and water conditions at the site and to perform the initial safety factor assessment in accordance with Section §257.73(e)(1) of the CCR Rule. To achieve the objective discussed above, the scope of work undertaken for this assessment included the tasks listed below.

- Reviewing readily available reports, investigations, plans and data pertaining to the surface impoundments.
- Evaluating liquefaction susceptibility of material used to construct the impoundment embankments.
- Performing static and seismic stability analyses for rotational failure surfaces using limit equilibrium methods.

1.3 ELEVATION DATUM AND HORIZONTAL CONTROL

The elevations referenced in this report are in feet and are based on the National Geodetic Vertical Datum of 1929 (NGVD29) unless otherwise noted. The horizontal control is the Missouri State Plane North Coordinate System (NAD 83) datum unless otherwise noted.

2. Description of Pond

A summary of relevant information associated with the pond is provided below. Additional details can be found in the Initial Structural Stability Assessment Report prepared by AECI under separate cover. Refer to **Figure 1**, "Project Locus" for the general site location.

2.1 DESCRIPTION OF CELL 002 WEST

Cell 002 West is an unlined surface CCR impoundment located south of the power plant at the Thomas Hill Energy Center in Clifton Hill, Missouri and is one of two basins that form the Cell 002 impoundment. The impoundment is inactive and is currently being pumped to maintain a dry condition to facilitate the ongoing removal of CCR. Cell 002 was designed by Burns & McDonnell during the period 1978-1979 as a single impoundment and was constructed shortly afterwards. It is understood that Cell 002 was modified in the 1980's when Cell 001 was constructed north of Cell 002. Modifications to Cell 002 in 2015 included construction of a separator berm that split the impoundment into east and west basins. The separator berm was designed by Gredell Engineering Resources, Inc.

Cell 002 West covers an area of approximately 12.5 acres. The stormwater storage capacity of Cell 002 West is estimated to be 72 acre-feet at its discharge elevation. On the north side, Cell 002 is partially abutted by Cell 001 and is partially incised. Similarly, the impoundment is incised on the west side. The crest of the separator berm on the east side of the impoundment is at approximately El. 721 and is approximately 8-ft wide (at the crest). Historic records show that the 21-ft high separator berm was constructed from clay obtained from an on-site borrow source that was placed and compacted in a controlled manner. The east and west slopes of the separator berm were constructed with approximately 3 horizontal to 1 vertical (3H:1V) slopes. To the south, the crest of the embankment that separates Cell 002 from Cell 003 is approximately 10-ft wide and is at approximately El. 727. The upstream slope of the 27-ft high embankment is approximately 3H:1V, while the downstream slope varies between approximately 2H:1V and 3H:1V.

Discharge from the west basin is via a 15-in. diameter corrugated metal pipe (CMP) with an invert at El. 718. The CMP penetrates the south embankment and discharges into Cell 003. Discharge from the east basin is via a concrete drop inlet structure built during the original construction of Cell 002. When the water level in the basin reaches normal pool level (El. 717), water enters the structure and flows to Cell 003 through a discharge pipe that runs through the south embankment.

3. Field Investigation Program

3.1 PREVIOUS EXPLORATIONS AND LABORATORY TESTING PERFORMED BY OTHERS

A subsurface exploration and laboratory testing program was previously completed at the site by others. The approximate locations of the relevant historic subsurface explorations performed by others are shown on the attached **Figure 2**. A brief summary of the explorations is provided below, and details of relevant explorations are presented in **Table I**[1]. Note that the term "relevant" explorations refers to explorations from previous investigations by others that were directly used in our safety factor assessment.

- One (1) test boring was performed by Geotechnology on 13 January 2010 as part of a global stability evaluation of Cell 002. The test boring log and laboratory test results associated with this investigation are included in **Appendix A**.

- One (1) cone penetrometer sounding was performed by Stratigraphics, Inc. on 3 February 2010 as part of a global stability evaluation of Cell 002. The log associated with this investigation is included in **Appendix A**.

[1] Note: A table that does not appear near its citation can be found in a separate table at the end of the report.

4. Subsurface Conditions

4.1 GEOLOGY

Thomas Hill Energy Center is located within the Dissected Till Plains subprovince of the Central Lowlands physiographic province and is underlain by recent alluvium and glacial till deposits. These deposits are underlain regionally by a sequence of bedrock formations ranging in age from Cambrian to Pennsylvanian (Miller and Vandike, 1997).

Alluvium and glacial till deposits underlying the ponds typically consist of clay, silty clay, silty clay with trace sand and gravel, and clayey to sandy silt. Siltstone and shale bedrock is present at a depth ranging from 27 to 36 feet (Geotechnology, 2010, 2012a, 2012b).

4.2 SUBSURFACE CONDITIONS

Descriptions of the soil conditions encountered in the test boring and CPT sounding during the historic subsurface exploration program conducted at the site are provided below in order of increasing depth below ground surface. Actual soil conditions between exploration locations may differ from these typical descriptions. Refer to the logs in **Appendix A** for specific descriptions of soil samples obtained from the historic test boring.

The subsurface conditions identified by the historic CPT sounding does not represent material classifications based on grain-size distributions, index tests, or visual observation. Rather, the historic CPT sounding provides an indicator of relative behavior type based on the mechanical characteristics measured during the sounding. For this reason, the descriptions of subsurface conditions discussed below are only based on classifications of samples obtained from the historic test boring and the results of historic laboratory testing.

- *SOUTH EMBANKMENT FILL* – Below the ground surface in historic test boring C-1 performed at the south embankment, there is a stratum of man-placed FILL primarily described as brown and gray lean CLAY (CL), with trace silt and sand. This stratum was fully penetrated, and the thickness of this stratum was approximately 11 ft. The consistency of fine-grained soils encountered in this stratum ranged from medium stiff to stiff.

- *CLAY* - Below the SOUTH EMBANKMENT FILL at test boring C-1, there is a stratum of natural soil primarily described as fat CLAY (CH) with sand and gravel and silty lean CLAY (CL). This stratum was not fully penetrated at boring C-1. The consistency of fine-grained soils encountered in this stratum ranged from medium stiff to stiff.

4.3 GROUNDWATER CONDITIONS

Water levels at the site discussed herein are based on the water levels measured in the historic test borings and estimated by the recent CPT soundings. A brief summary of measured water levels is provided below and summarized in **Table I**.

- Test Boring(s) - Water was not encountered in historic test boring C-1.

- CPT Sounding(s) – An estimated water level was not reported for historic CPT sounding CC01. *It should be noted that measurements typically estimated during a CPT sounding do not involve physical observation of water levels, but rather an estimated water level based on pore pressure measurements. The estimates of water levels at CPT soundings should only be considered accurate to the degree implied by the determination method.*

Water level readings have been made in the subsurface explorations at times and under conditions discussed herein. However, it must be noted that fluctuations in the level of the water may occur due to variations in power plant sluicing activities, season, rainfall, temperature, dewatering activities, and other factors not evident at the time measurements were made and reported herein.

5. Safety Factor Assessment

As mentioned previously, the purpose of this study was to perform an initial safety factor assessment in accordance with Section §257.73(e)(1) of the CCR Rule. As required by the CCR Rule, the certified initial safety factor assessment is performed for a CCR unit to determine calculated factors of safety for each CCR unit relative to the minimum prescribed safety factors for the critical cross section of the embankment. The minimum required safety factors are defined as follows:

- For dikes constructed of soils that have susceptibility to liquefaction, the calculated liquefaction factor of safety must equal or exceed 1.20.
- The calculated static factor of safety under the long-term, maximum storage pool loading conditions must equal or exceed 1.50.
- The calculated static factor of safety under the maximum surcharge pool loading condition must equal or exceed 1.40.
- The calculated seismic factor of safety must equal or exceed 1.00.

Stability analyses have been performed in general conformance with the principles and methodologies described in the USACE Slope Stability Manual (U.S. Army Corps of Engineers, 2003). Conventional static and seismic stability analyses of the impoundment embankments were performed for rotational failures using limit equilibrium methods. Limit equilibrium methods compare forces, moments, and stresses which cause instability of the mass of the embankment to those which resist that instability. The principle of the limit equilibrium method is to assume that if the slope under consideration were about to fail, or at the structural limit of failure, then one must determine the resulting shear stresses along the expected failure surface. These determined shear stresses are then compared with the shear strength of the soils along the expected failure surface to determine the safety factor. The details of the analyses performed for the Lined Pond are presented in the following sections of this report.

5.1 DESIGN WATER LEVELS

In accordance with the CCR Rule, the water retained in an impoundment must be modeled at the maximum storage pool and maximum surcharge pool levels for the static, long-term condition. For the seismic evaluation, the maximum storage pool level must be used to model the retained water. A summary of the maximum storage pool and surcharge pool water levels for Cell 002 are provided below.

Location	Crest	Maximum Storage Pool Level	Maximum Surcharge Pool Level
South Embankment	El. 727	El. 718	El. 727
Separator Berm	El. 721	El. 718	El. 721

The elevation of the phreatic surface within the south embankment and at the toe of slope were estimated based on conditions encountered in nearby subsurface explorations. Additionally, there is no current evidence of seepage emanating from the exterior slopes of the embankments, suggesting that the phreatic surface is contained within and/or below the embankments.

Given the prescribed impoundment pool levels and the observed static groundwater levels discussed above, a seepage analysis was performed to determine the piezometric head between the upstream slope of the impoundment embankments and the downstream toe of the embankments. The computer software program, Slide 7.0, developed by RocScience, Inc., was used to perform the seepage analyses. The water in Cell 003 was modeled at El. 710, which corresponds the normal operating level. The water in the Cell 002 east basin was conservatively modeled at El. 707, which is 10 ft below the discharge inlet elevation. Permeability values for embankment material layers were estimated from typical published values based on material description and correlations to grain size. During the course of the seepage analyses, minor adjustments were made to the permeability values and isotropic permeability ratios to best model the conditions observed in the field. Results from the seepage analysis provided pore pressure values within the seepage model that were then imported into the slope stability models for the southern embankment and separator berm.

The seepage models suggest that much of the seepage emanating from the impoundments is moving laterally through the southern embankment and separator berm. The phreatic surfaces used in the slope stability models are shown on the slope stability graphical output included in **Appendix B**.

5.2 MATERIAL PROPERTIES

The material properties used in our analyses have been evaluated using the results of the historic analyses performed by Geotechnology, historic subsurface explorations, and historic laboratory testing. In cases where subsurface explorations, laboratory test data, and historic properties did not exist for certain materials, properties were estimated based on typical values developed from empirical correlations and Haley & Aldrich's experience with similar materials. Specifically, the material properties for the separator berm fill were estimated using empirical correlations based on the laboratory testing and field QC testing performed during construction. A summary of the material properties used in our slope stability analyses are provide below in **Table III.** Additional details regarding soil property characterization are provided in **Appendix B**.

TABLE III
MATERIAL PROPERTIES

Material	Material Strength	Unit Weight (pcf)	Cohesion (psf)	Friction Angle (degrees)	Su (psf)
South Embankment Fill	Drained	125	100	28	--
	Undrained	125	--	--	1300
Separator Berm Fill	Drained	125	100	28	--
	Undrained	125	--	--	1300
Clay Layer 1	Drained	120	50	27	--
	Undrained	120	--	--	1700
Clay Layer 2	Drained	120	50	27	--
	Undrained	120	--	--	1600
Clay Layer 3	Drained	120	50	27	--
	Undrained	120	--	--	1100
Clay Layer 4	Drained	120	50	27	--
	Undrained	120	--	--	950

5.3 DESIGN SEISMIC EVENT

The earthquake conditions used in our analyses correspond to the peak ground acceleration for a seismic event with a 2% probability of exceedance in 50 years (2,500-year return period). The gridded hazard map data associated with the latest USGS National Seismic Hazard maps developed in 2014 indicates that the bedrock peak ground acceleration (PGA) at the site for the 2,500-year earthquake event is 0.057g, with the greatest contribution to the hazard coming from an earthquake with a modal magnitude of 7.8 as indicated on the deaggregation chart included in **Appendix B**. The bedrock PGA value was adjusted by the USGS site coefficient, F_{PGA}, of 1.6 for Site Class D to determine the peak free field ground acceleration of 0.091g. Note that the value of peak free field acceleration corresponds to the peak ground acceleration at the base of the impoundment embankment.

5.4 LIQUEFACTION POTENTIAL EVALUATION

During strong earthquake shaking, loose, saturated cohesionless soil deposits may experience a sudden loss of strength and stiffness, sometimes resulting in loss of bearing capacity, large permanent lateral displacements, and/or seismic settlement of the ground. This phenomenon is called soil liquefaction. In accordance with the requirements of §257.73(e)(1), evaluations have been performed to assess the potential for liquefaction of the soils used to construct the impoundment embankment.

The results of the historic subsurface explorations performed at the site indicate that the soils used to construct the impoundment embankments primarily consist of lean CLAY and fat CLAY. These materials are not susceptible to liquefaction due to their high fines content and plasticity. In accordance with the requirements of §257.73(e)(1), a post-liquefaction stability analysis is not required since the soils used to construct the embankment are not susceptible to liquefaction in their current state.

5.5 STABILITY ANALYSIS

5.5.1 Methodology for Analyses

The computer software program Slide 7.0 was used to evaluate the static and seismic stability of the impoundment embankments. Analyses for static stability were performed to evaluate long-term maximum storage pool condition and maximum surcharge pool condition using Spencer's method of slices. Spencer's method of slices was selected because it fully satisfies the requirements of force and moment equilibrium (limit equilibrium method).

Both circular and translational (block) failures were evaluated. Translational failures were only evaluated where subsurface conditions included a relatively weak embankment or foundation layer located above or below a relatively strong foundation layer, such as soft clay overlying dense sand. The results of our evaluation of circular and translational failures indicated that circular failure surfaces represent the critical slope failure case. Accordingly, the results presented herein are limited to the critical case analyses performed for circular failure surfaces.

Seismic stability was evaluated using pseudo-static analyses. Pseudo-static analyses model the seismic shaking as a "permanent" body force that is added to the force-body diagram of a conventional static limit-equilibrium analysis; typically, only the horizontal component of earthquake shaking is modeled because the effects of vertical forces tend to average out to near zero (Jibson, 2011). This is a traditional approach for evaluating the stability of a slope during earthquake shaking and provides a simplified safety factor analysis for one earthquake pulse. A 20 percent reduction in material strength was conservatively incorporated in the pseudo-static analyses to represent the approximate threshold between large and small strains induced by cyclic loading (Duncan, 2014). In pseudo-static analyses, a safety factor greater than or equal to one (FS ≥ 1.0) generally indicates a slope is stable and a safety factor below one (FS < 1.0) generally indicates that a slope is unstable.

5.5.2 Pseudo-static Coefficient

The pseudo-static coefficient, k_s, used in our seismic analyses was calculated using the equation below, which uses the peak free field acceleration discussed above after adjusting it to represent the acceleration at the top of the impoundment embankment, k_{max}. In addition, a reduction factor of 0.50 as recommended by Hynes-Griffin and Franklin was applied to the value of k_{max}.

$$k_s = 0.50 \times \frac{k_{\max}}{\mathrm{g}} = 0.50 \times \frac{0.14\mathrm{g}}{\mathrm{g}} = 0.07$$

5.5.3 Results of Stability Evaluation

The critical cross section is defined as that which is anticipated to be most susceptible to failure amongst all cross sections. To identify the critical cross section at our project site, we examined the following conditions at several cross-section locations at each impoundment:

a. the geometry of the upstream and downstream embankment slopes;
b. phreatic surface levels within and below the cross sections;
c. subsurface soil conditions;
d. presence or lack of surcharge loads behind the crest of the embankments; and
e. presence or lack of reinforcing measures in front of the embankments.

Examination of the conditions noted above resulted in the identification of two cross sections that could potentially be considered the critical cross-section. The location of each cross section is shown on **Figure 2**. The results of our analyses are presented below in **Table IV** and are shown on the Slide output files included in **Appendix B**.

As shown below, the static safety factors are above the minimum required values for each cross-section that was evaluated. Similarly, the pseudo-static analyses for the analyzed sections indicate an acceptable seismic safety factor.

The analyses discussed herein have been performed for the impoundment configuration understood at this time for the existing condition of Cell 002 West. Should impoundment heights, slope angles, surcharges, or water levels (upstream or downstream) change, the conclusions contained in this report should not be considered valid unless the changes are reviewed by Haley & Aldrich and the conclusions of this report modified or verified in writing.

TABLE IV SUMMARY OF STABILITY EVALUATIONS						
Cross Section	Condition[1]	Earthquake Event	Soil Strength	Pool Level	Required Safety Factor	Calculated Safety Factor
2A-2A′ (South Embankment)	Static	-	Drained	Maximum Storage	1.50	1.56
			Drained	Maximum Surcharge	1.40	1.53
	Seismic	2,500-year	Undrained[2]	Maximum Storage	1.00	1.55
2B-2B′ (Separator Berm)	Static	-	Drained	Maximum Storage	1.50	1.50
			Drained	Maximum Surcharge	1.40	1.50
	Seismic	2,500-year	Undrained[2]	Maximum Storage	1.00	1.80

1. Refer to Table III for material properties.
2. Soil strengths have been reduced by 20 percent for seismic analyses to represent the approximate threshold between large and small strains induced by cyclic loading.

5.6 CONCLUSIONS

The analyses associated with the safety factor assessment have been performed in accordance with the requirement of Section §257.73(e) of the CCR Rule. A summary of our conclusions as they relate to the rule requirements are provided below.

- §257.73(e)(1)(i) - *The calculated static factor of safety under the long-term, maximum storage pool loading conditions must equal or exceed 1.50.*

  As shown in Table **IV**, the static safety factors for the long-term maximum storage pool condition are above the minimum required value for the critical section analyzed. Accordingly, this requirement has been met.

- §257.73(e)(1)(ii) - *The calculated static factor of safety under the maximum surcharge pool loading condition must equal or exceed 1.40.*

  As shown in **Table IV**, the static safety factors for the maximum surcharge pool loading condition are above the minimum required value for the critical section analyzed. Accordingly, this requirement has been met.

- §257.73(e)(1)(iii) - *The calculated seismic factor of safety must equal or exceed 1.00.*

  As shown in **Table IV**, the calculated seismic safety factor is above the minimum required value for the critical section analyzed. Accordingly, this requirement has been met.

- §257.73(e)(1)(iv) - *For dikes constructed of soils that have susceptibility to liquefaction, the calculated liquefaction factor of safety must equal or exceed 1.20.*

The results of historic subsurface investigations indicate that the materials used to construct the impoundment embankments are not susceptible to liquefaction. Accordingly, this requirement has been met.

6. Certification

Based on our review of the information provided to us by AECI and the results our analyses, it is our opinion that the calculated factors of safety for the critical cross section of the impoundment embankment meets the minimum factors of safety specified in §257.73(e)(1)(i) through (iv) of the EPA's CCR Rule.

Certification Statement – Cell 002 West

I certify that the Initial Safety Factor Assessment for Cell 002 West at the Thomas Hill Energy Center meets the requirements of §257.73(e) of the EPA's CCR Rule.

Signed:____________________________________

Certifying Engineer

Print Name:	Steven F. Putrich
Missouri License No.:	2014035813
Title:	Project Principal
Company:	Haley & Aldrich, Inc.

Professional Engineer's Seal:

References

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3. Environmental Protection Agency, (2015). Code of Federal Regulations, "Hazardous and Solid Waste Management System; Disposal of Coal Combustion Residuals from Electric Utilities; Final Rule, "Title 40, Chapter I, Parts 257 and 261, April 17.

4. Geotechnology, Inc. (February 2012a). "Slope Stability and Seepage Analysis, Slag Dewatering Basin, Thomas Hill Energy Center". [*Note this refers to the current Cell 001 impoundment*]

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TABLES

TABLE I

SUMMARY OF RELEVANT HISTORIC SUBSURFACE EXPLORATIONS
AECI THOMAS HILL CELL 002 WEST
INITIAL SAFETY FACTOR ASSESSMENT
CLIFTON HILL, MISSOURI

Exploration Designation[1]	Performed By	Year	Ground Surface El.[2] (ft)	Total Exploration Depth (ft)	Water[3] Depth Below Ground Surface
TEST BORING					
C-1	Geotechnology, Inc.	2010	735.0	50.0	Not Measured
CONE PENETROMETER SOUNDING					
CC01	Stratigraphics, Inc.	2010	728.0	49.8	Unknown

Notes:

1) Technical monitoring of historic subsurface explorations was performed by others.

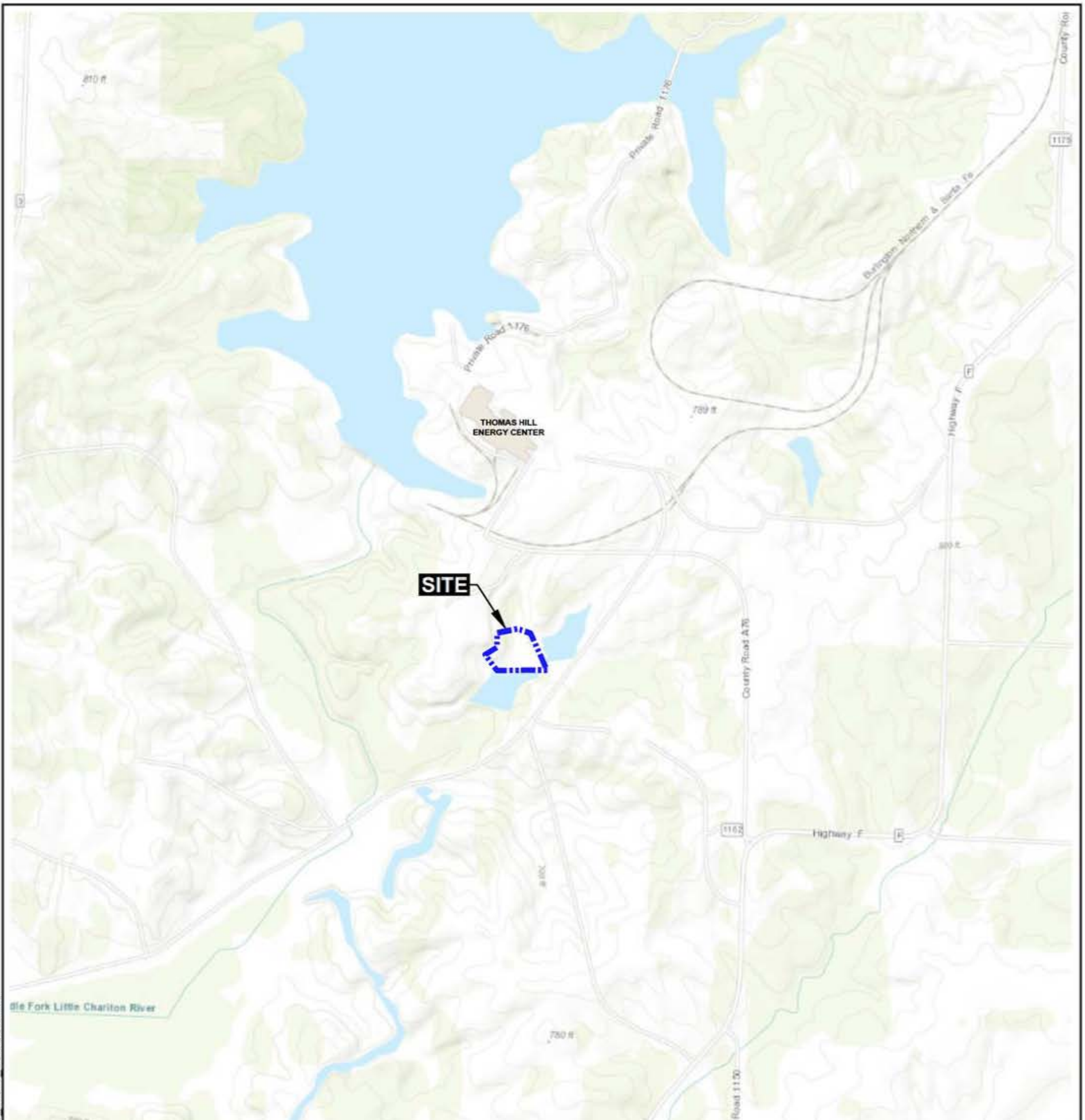
2) The elevation data are provided in feet and the vetical datum is unknown. Ground surface elevation of historic test borings were taken from boring logs prepared by Geotechnology, Inc. Ground surface elevation of historic cone penetrometer soundings were approximated by linear interpolation between ground surface elevation contour lines shown on Figure 2.

3) Water level readings have been made in the explorations by others at times and under conditions discussed herein. However it must be noted that fluctuations in the level of the water may occur due to variations in season, plant sluicing activities, rainfall, temperature, and other factors not evident at the time measurements were made and reported.

TABLE II
SUMMARY OF HISTORIC LABORATORY TEST RESULTS
AECI THOMAS HILL CELL 002 WEST
INITIAL SAFETY FACTOR ASSESSMENT
CLIFTON HILL, MISSOURI

Boring Designation	Sample Number	Sample Depth (ft)	USCS Symbol	Material Type/Stratum	Moisture Content (%)	LL	PL	PI	% Gravel	% Sand	% Fines	Tube Density		CU Triaxial		Hydraulic Conductivity				Standard Proctor	
												Average Moisture Content (%)	Average Total Density (pcf)	c' (psf)	ϕ' (degrees)	Moisture Content (%)	Dry Density (pcf)	k (cm/sec)	Confining Pressure (psf)	Max. Dry Density (pcf)	Optimum Mosisture (%)
HISTORIC TESTING BY GEOTECHNOLOGY, INC. IN APRIL 2010																					
C-1	SS2	3.5-5.0		SOUTH BERM FILL	24																
C-1	SS3	6.0-7.5	CH	SOUTH BERM FILL	24	52	28	24													
C-1	SS4	8.5-10.0	CH	SOUTH BERM FILL	23																
C-1	ST5	11.0-13.0	CH	NATIVE CLAY	14																
C-1	ST6	13.5-15.5	CH	NATIVE CLAY		51	25	26				30	126.1	0	26						
C-1	ST6	13.5-15.5	CH	NATIVE CLAY								22	120.8								
C-1	SS7	18.5-20.0	CH	NATIVE CLAY	16																
C-1	SS8	23.5-25.0	CH	NATIVE CLAY	27																
C-1	SS9	28.5-30.0	CH	NATIVE CLAY	24																
C-1	SS10	33.5-35.0	CL	NATIVE CLAY	24	44	18	26													
C-1	SS11	38.5-40.0	CL	NATIVE CLAY	24																
C-1	SS12	43.5-45.0	CH	NATIVE CLAY	27																
C-1	SS13	48.5-50.0	CH	NATIVE CLAY	29																
HISTORIC TESTING BY GREDELL ENGINEERING RESOURCES, INC. IN OCTOBER 2015																					
AECI-THEC 1	(remolded)		CL	SEPARATOR BERM FILL	19.6	41	17	24								18.0	109.0	7x10-9	90	112.4	15.6
AECI-THEC 2			CL	SEPARATOR BERM FILL	18	44	17	27													
AECI-THEC 3			CL	SEPARATOR BERM FILL	17.5	42	16	26													
AECI-THEC 4	(remolded)		CL	SEPARATOR BERM FILL	18.4	44	18	26												112.7	13.8
TP-1	BS1	1.0	CH	SEPARATOR BERM FILL	24.5	59	26	33													
TP-1	BS2	4.0	SC	SEPARATOR BERM FILL	16.7	38	18	20	0.0	61.0	39.0										
TP-1	BS3	10.0	CL	SEPARATOR BERM FILL	16.4	38	19	19													
TP-2	BS1	1.0	CH	SEPARATOR BERM FILL	25.8	53	23	31													
TP-2	BS2	2.5	CH	SEPARATOR BERM FILL	31.8	58	28	30													
TP-2	BS3	6.0	SC	SEPARATOR BERM FILL	19.6	34	18	16	0.0	75.0	25.0										
T-3	T-3 AECI-THEC			SEPARATOR BERM FILL	19.9	70	26	44													
On-site borrow area	(remolded)		CH	SEPARATOR BERM FILL	29	84	18	66												89.9	21.2

FIGURES



MAP SOURCE: ESRI

SITE COORDINATES: 39°32'42"N, 92°38'9"W

HALEY ALDRICH

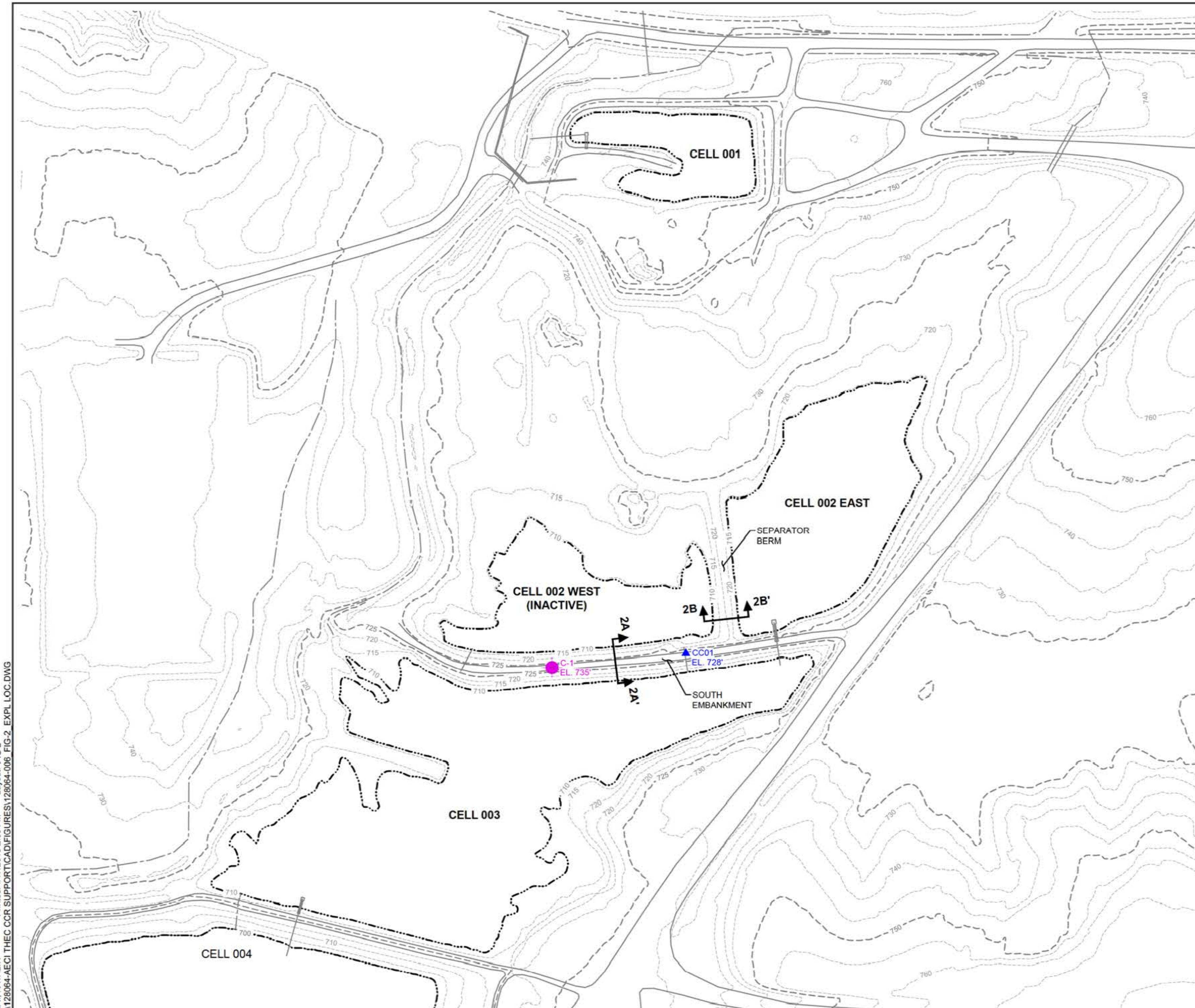
ASSOCIATED ELECTRIC COOPERATIVE, INC.
THOMAS HILL ENERGY CENTER
CLIFTON HILL, MISSOURI

PROJECT LOCUS

APPROXIMATE SCALE: 1IN = 2000 FT
APRIL 2018

FIGURE 1

VARI, KATALIN Printed: 4/16/2018 11:40 AM Layout: FIGURE 1
G:\128064-AECI THEC CCR SUPPORT\CAD\FIGURES\128064-006_FIG-1_LOCUS.DWG



LEGEND

CC01 EL. 728' — DESIGNATION AND APPROXIMATE LOCATION OF CONE PENETROMETER SOUNDING PERFORMED BY STRATIGRAPHIC, INC. OF PROPHETSTOWN, ILLINOIS ON FEBRUARY 3, 2010.

C-1 EL. 735' — DESIGNATION AND APPROXIMATE LOCATION OF TEST BORING PERFORMED BY GEOTECHNOLOGY, INC. OF ST. LOUIS, MISSOURI ON JANUARY 13, 2010.

2A 2A' — CROSS-SECTION LOCATION

NOTES

1. AERIAL SURVEY USED TO DEVELOP TOPOGRAPHY WAS PERFORMED BY PICTOMETRY INTERNATIONAL CORP. OF ROCHESTER, NEW YORK BETWEEN FEBRUARY 29,2016 AND APRIL 11, 2016.
   - HORIZONTAL CONTROL IS MISSOURI STATE PLANE NORTH COORDINATE SYSTEM (NAD 83).
   - ELEVATIONS IN THIS DRAWING ARE SHOWN IN FEET. THE VERTICAL DATUM FOR GROUND SURFACE ELEVATION CONTOUR LINES IS NGVD 29.
2. AS-DRILLED LOCATIONS OF TEST BORING PERFORMED BY GEOTECHNOLOGY, INC. AND CONE PENETROMETER SOUNDING PERFORMED BY STRATIGRAPHICS, INC. HAVE BEEN APPROXIMATED. GROUND SURFACE ELEVATION OF TEST BORING PERFORMED BY GEOTECHNOLOGY, INC. IS FROM BORING LOGS PREPARED BY GEOTECHNOLOGY, INC. GROUND SURFACE ELEVATION OF CONE PENETROMETER SOUNDING WAS APPROXIMATED BY LINEAR INTERPOLATION BETWEEN GROUND SURFACE ELEVATION CONTOUR LINES SHOWN ON THIS FIGURE.
3. TECHNICAL MONITORING OF SUBSURFACE EXPLORATIONS PERFORMED BY GEOTECHNOLOGY, INC. AND STRATIGRAPHICS, INC. WAS PERFORMED BY OTHERS.

0 250 500
SCALE IN FEET

HALEY ALDRICH

ASSOCIATED ELECTRIC COOPERATIVE, INC.
THOMAS HILL ENERGY CENTER
CLIFTON HILL, MO

SUBSURFACE EXPLORATION
LOCATION PLAN

SCALE: AS SHOWN
APRIL 2018

FIGURE 2

VARI, KATALIN Printed: 4/17/2018 2:56 PM Layout: FIG 2
G:\128064-AECI THEC CCR SUPPORT\CAD\FIGURES\128064-006 FIG-2_EXPL LOC.DWG

APPENDIX A

Historic Subsurface Exploration Logs and Laboratory Test Results

Surface Elevation: 735 | Completion Date: 1/13/10

Datum msl

DEPTH IN FEET	DESCRIPTION OF MATERIAL	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
	Crushed rock, slag and fly ash	4-4-6	SS1
	FILL: brown and gray clay, trace silt and sand	3-4-4	SS2
5		3-4-5	SS3
		4-5-6	SS4
10	Very stiff, yellow, brown and gray CLAY - (CH)		ST5
		97 99	ST6
15	Medium stiff to stiff, brown and gray CLAY with sand and gravel - CH	3-5-7	SS7
20			
		3-3-4	SS8
25			
		3-4-5	SS9
30	Stiff to medium stiff, gray, silty CLAY - (CL)	5-7-7	SS10
35		2-4-4	SS11

SHEAR STRENGTH, tsf

Δ - UU/2 ○ - QU/2 ▯ - SV

0.5 1.0 1.5 2.0 2.5

STANDARD PENETRATION RESISTANCE

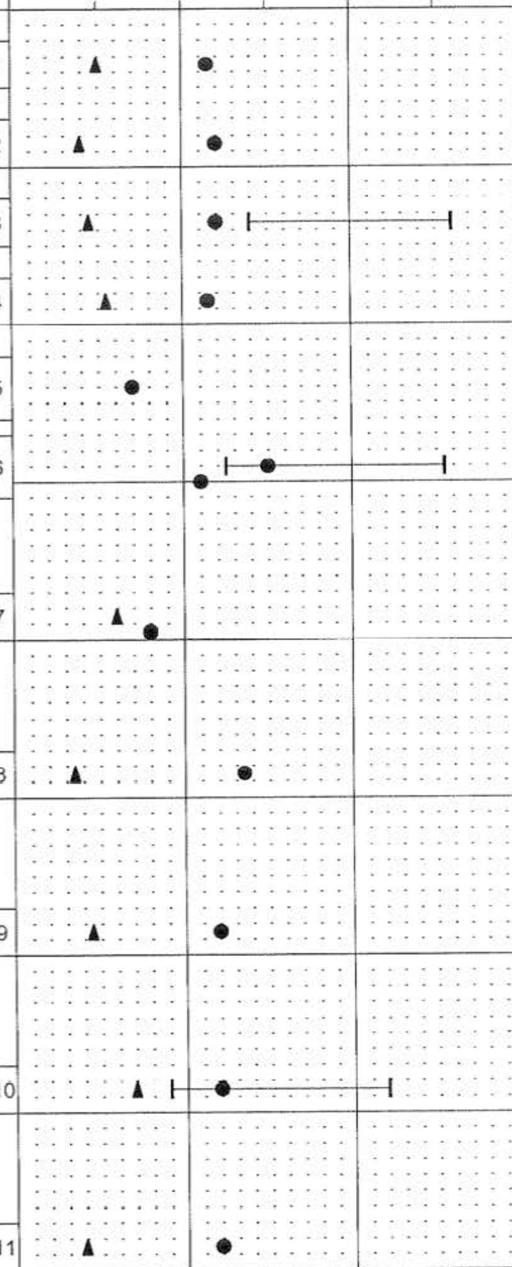
(ASTM D 1586)

▲ N-VALUE (BLOWS PER FOOT)

WATER CONTENT, %

PL ● LL

10 20 30 40 50



NOTE: STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARIES BETWEEN SOIL TYPES AND THE TRANSITION MAY BE GRADUAL. GRAPHIC LOG FOR ILLUSTRATION PURPOSES ONLY.

GROUNDWATER DATA

X FREE WATER NOT ENCOUNTERED DURING DRILLING

DRILLING DATA

__ AUGER 3 3/4" HOLLOW STEM

WASHBORING FROM 40 FEET

BS DRILLER RFW LOGGER

CME 550X DRILL RIG

HAMMER TYPE Auto

REMARKS:

Drawn by: KSA	Checked by: SK	App'vd. by: MHM
Date: 1/20/10	Date: 4/6/10	Date: 4/19/10

GEOTECHNOLOGY INC
FROM THE GROUND UP

Thomas Hill
Ash Pond Evaluation

LOG OF BORING: C-1

Project No. J011309.01

LOG OF BORING 2002 WL 1130901 - ASH POND.GPJ GTINC 0638301.GPJ 4/20/10

Surface Elevation: 735 | Completion Date: 1/13/10

Datum msl

DEPTH IN FEET	DESCRIPTION OF MATERIAL	GRAPHIC LOG	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES	SHEAR STRENGTH, tsf / STANDARD PENETRATION RESISTANCE / WATER CONTENT, %
	Stiff to medium stiff, gray, silty CLAY - (CL) *(continued)*				
	Medium stiff to stiff, brown and gray CLAY, trace sand - CH				
45			2-3-3	SS12	
			3-4-4	SS13	
50	Boring terminated at 50 feet.				
55					
60					
65					
70					
75					

SHEAR STRENGTH, tsf: Δ - UU/2 ○ - QU/2 □ - SV; 0.5 1.0 1.5 2.0 2.5

STANDARD PENETRATION RESISTANCE (ASTM D 1586)

▲ N-VALUE (BLOWS PER FOOT)

WATER CONTENT, %: PL ├──●──┤ LL; 10 20 30 40 50

NOTE: STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARIES BETWEEN SOIL TYPES AND THE TRANSITION MAY BE GRADUAL. GRAPHIC LOG FOR ILLUSTRATION PURPOSES ONLY

GROUNDWATER DATA

X FREE WATER NOT ENCOUNTERED DURING DRILLING

DRILLING DATA

__ AUGER 3 3/4" HOLLOW STEM

WASHBORING FROM 40 FEET

BS DRILLER RFW LOGGER

CME 550X DRILL RIG

HAMMER TYPE Auto

REMARKS:

Drawn by: KSA	Checked by: SK	App'vd. by: MHM
Date: 1/20/10	Date: 4/6/10	Date: 4/19/10

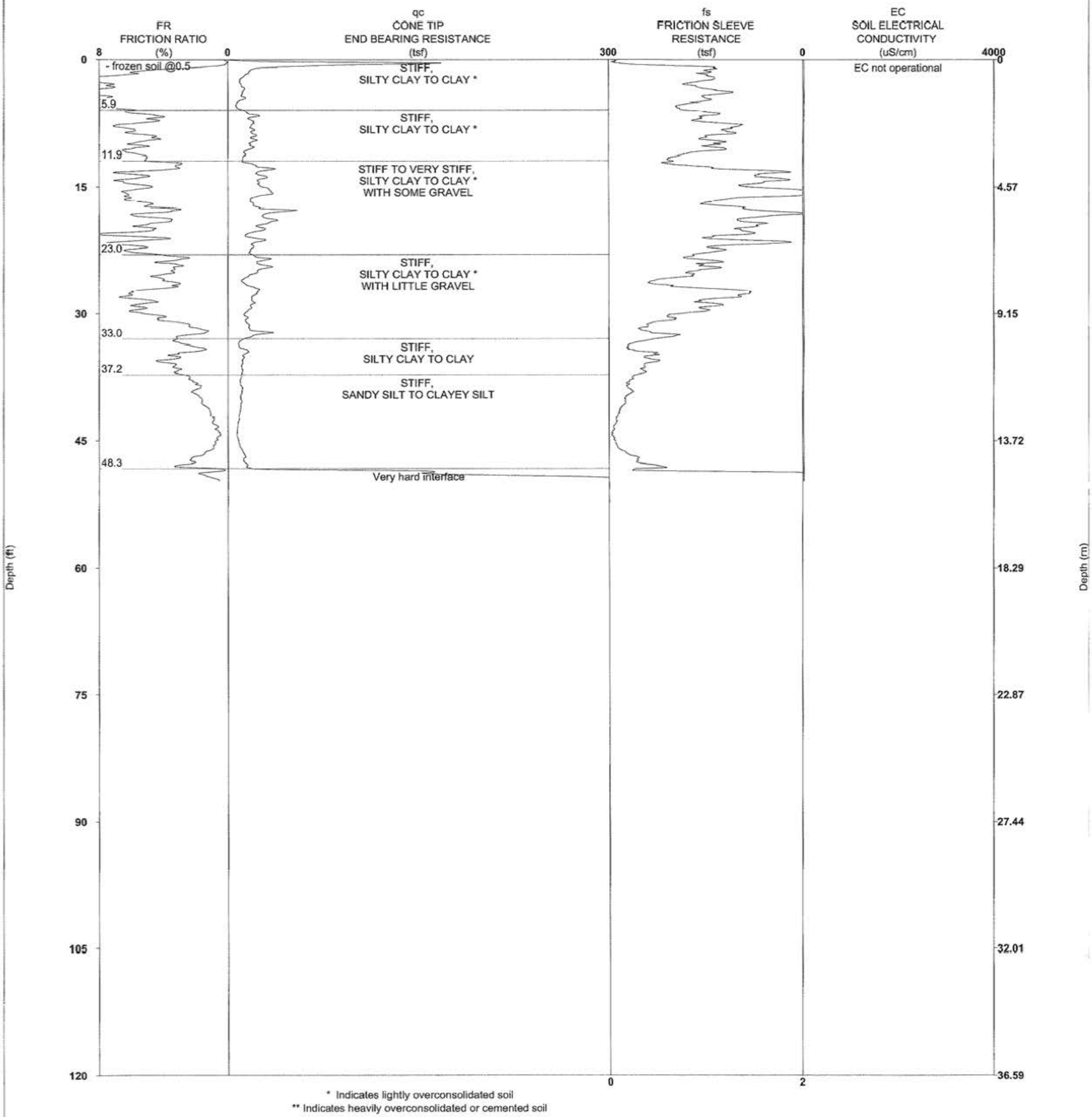
GEOTECHNOLOGY INC FROM THE GROUND UP

Thomas Hill
Ash Pond Evaluation

CONTINUATION OF
LOG OF BORING: C-1

Project No. J011309.01

CPTU-EC LOG WITH LITHOLOGIC EVALUATION CPCC01



* Indicates lightly overconsolidated soil

** Indicates heavily overconsolidated or cemented soil

Latitude: 39.54378 Longitude: -92.63682

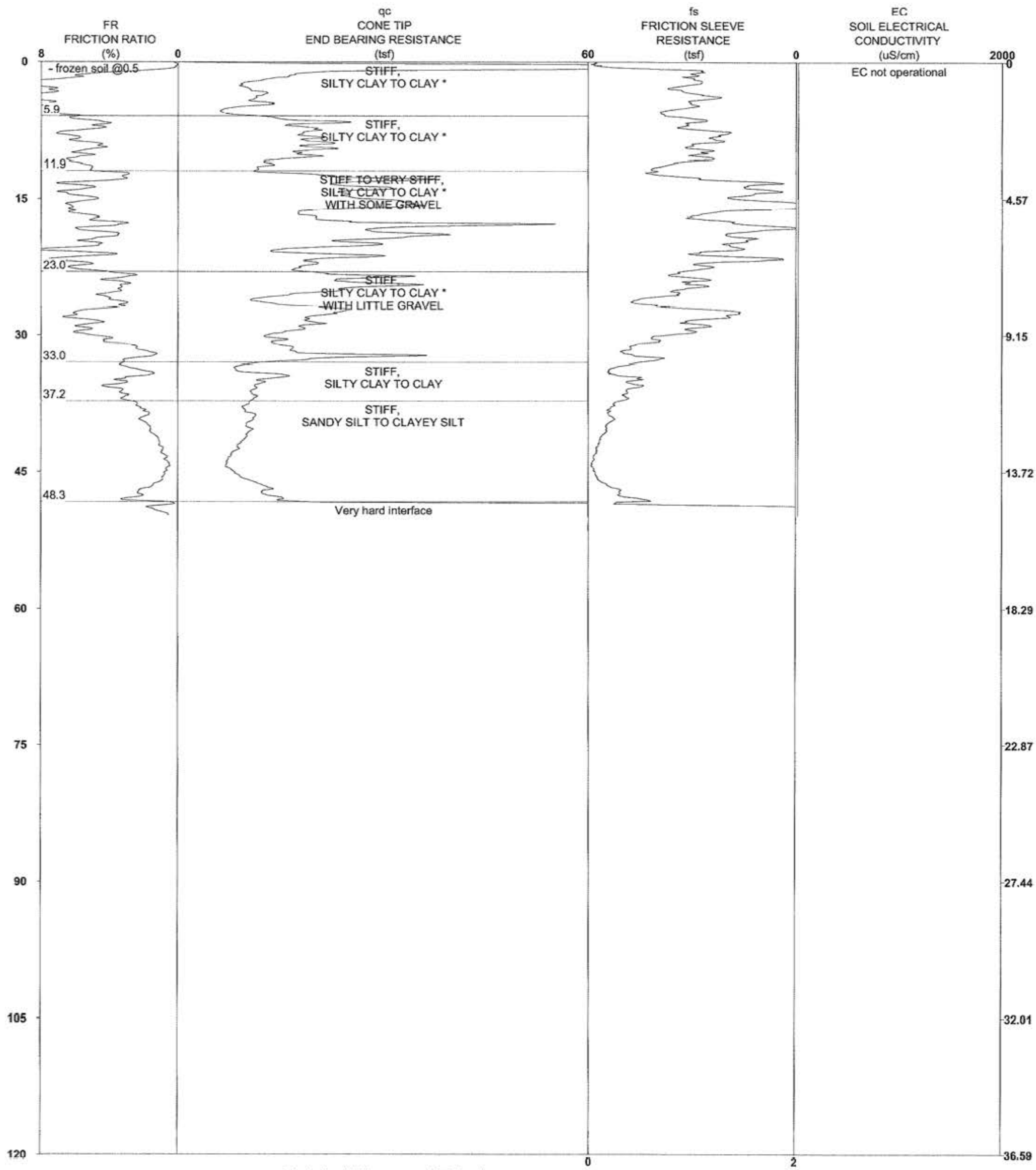
PROJECT NAME:Thomas Hill Site
PROJECT NUMBER:10-110-020

STRATIGRAPHICS

R1 DATE:2/3/2010 TIME:8:59 AM
SOUNDING NUMBER:CC-01

CPCC01

CPTU-EC LOG WITH LITHOLOGIC EVALUATION CPCC01



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** Indicates heavily overconsolidated or cemented soil

Latitude: 39.54378 Longitude: -92.63682

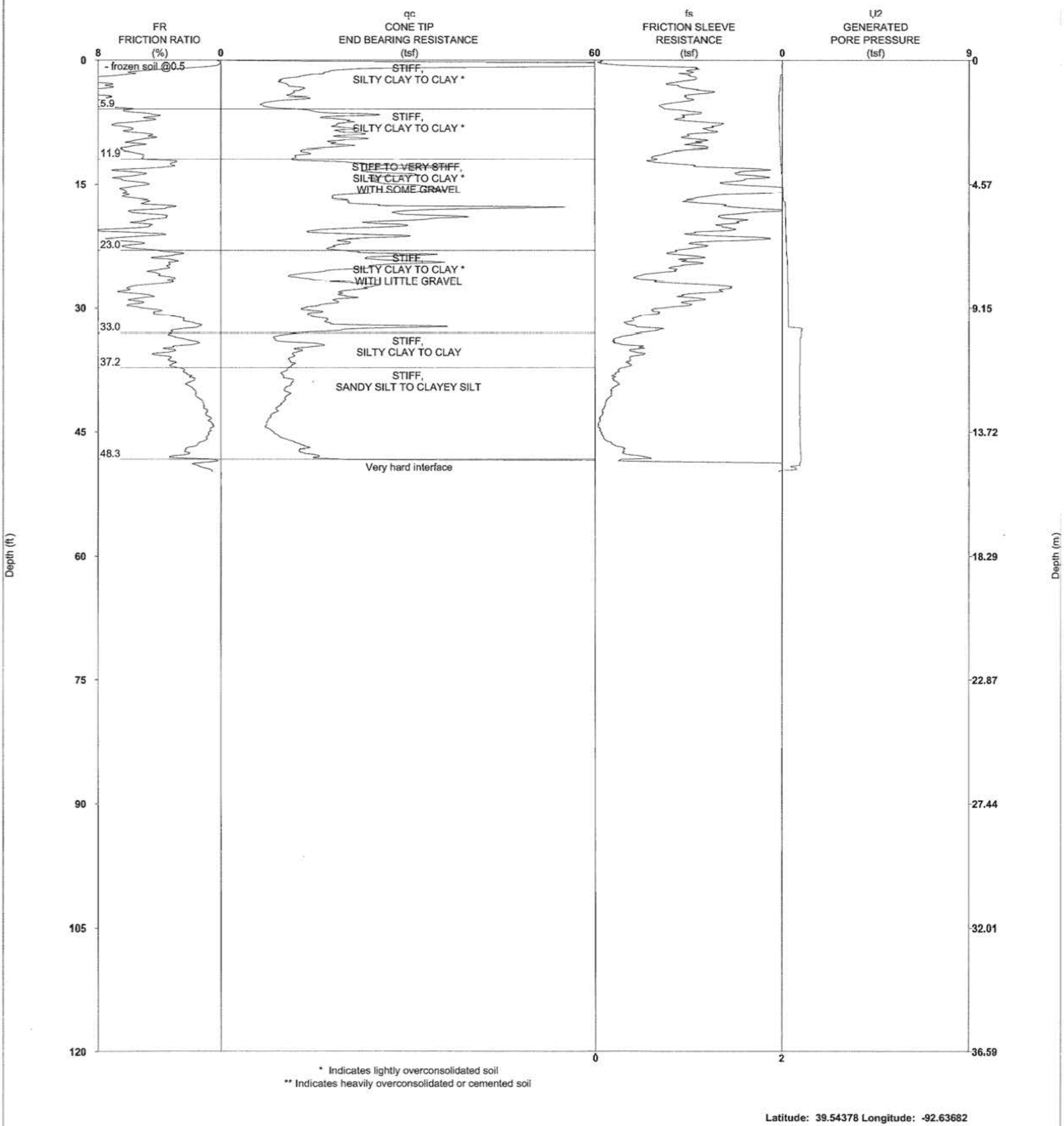
PROJECT NAME:Thomas Hill Site
PROJECT NUMBER:10-110-020

STRATIGRAPHICS

R1 DATE:2/3/2010 TIME:8:59 AM
SOUNDING NUMBER:CC-01

CPCC01

CPTU-EC LOG WITH LITHOLOGIC EVALUATION CPCC01



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

Latitude: 39.54378 Longitude: -92.63682

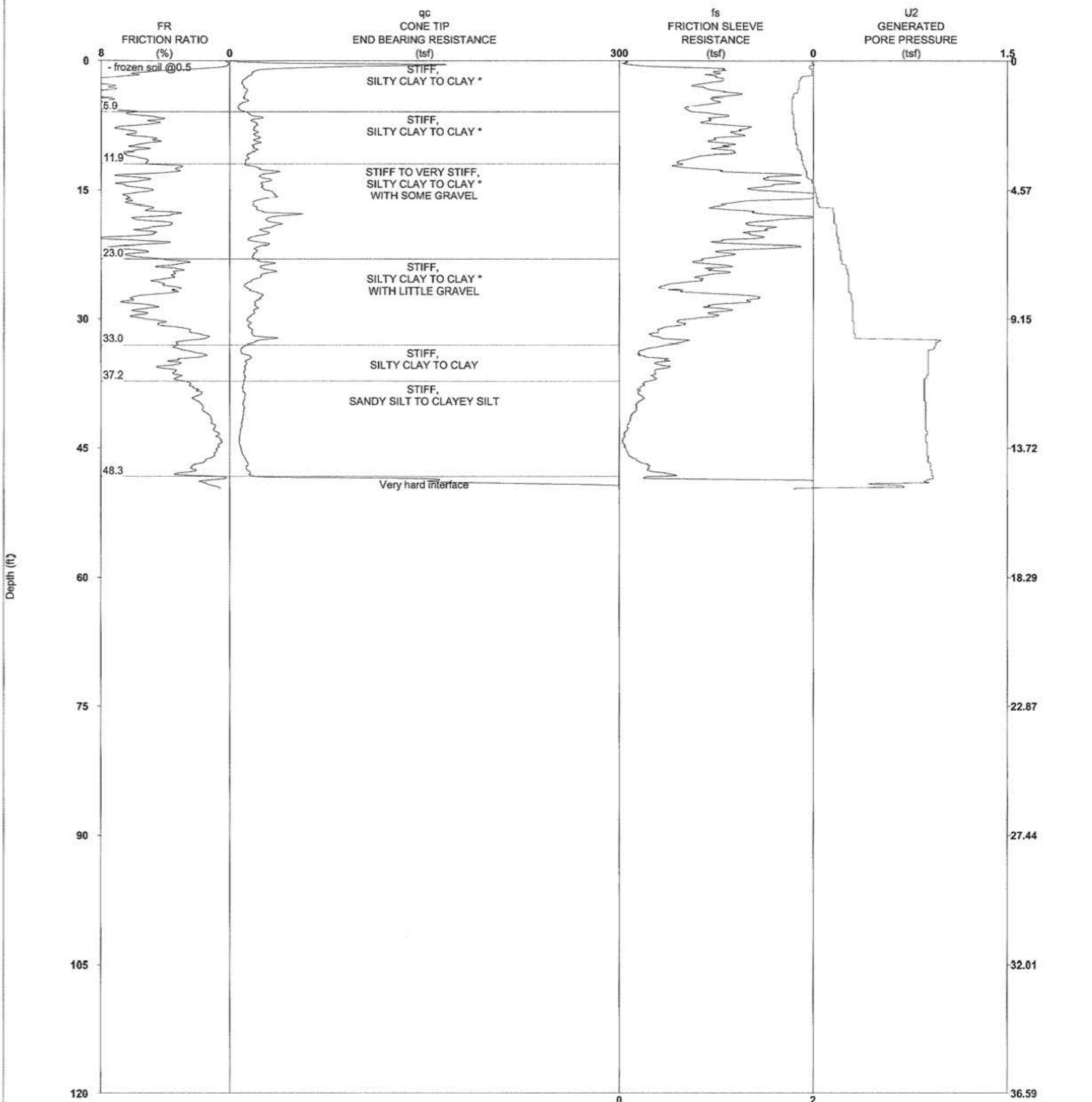
PROJECT NAME:Thomas Hill Site
PROJECT NUMBER:10-110-020

STRATIGRAPHICS

R1 DATE:2/3/2010 TIME:8:59 AM
SOUNDING NUMBER:CC-01

CPCC01

CPTU-EC LOG WITH LITHOLOGIC EVALUATION CPCC01



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Latitude: 39.54378 Longitude: -92.63682

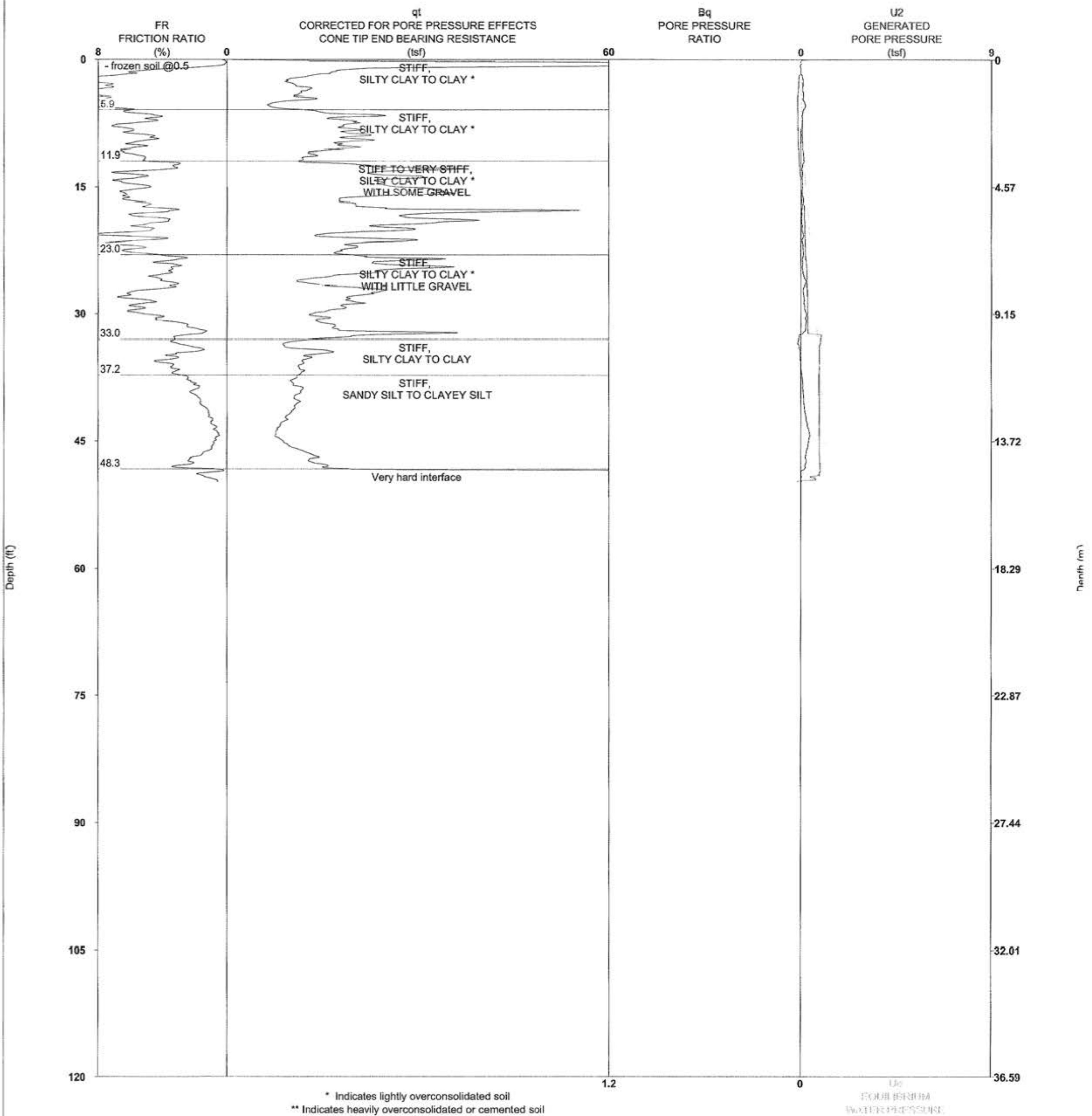
PROJECT NAME:Thomas Hill Site
PROJECT NUMBER:10-110-020

STRATIGRAPHICS

R1 DATE:2/3/2010 TIME:8:59 AM
SOUNDING NUMBER:CC-01

CPCC01

CPTU-EC LOG WITH LITHOLOGIC EVALUATION CPCC01



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Latitude: 39.54378 Longitude: -92.63682

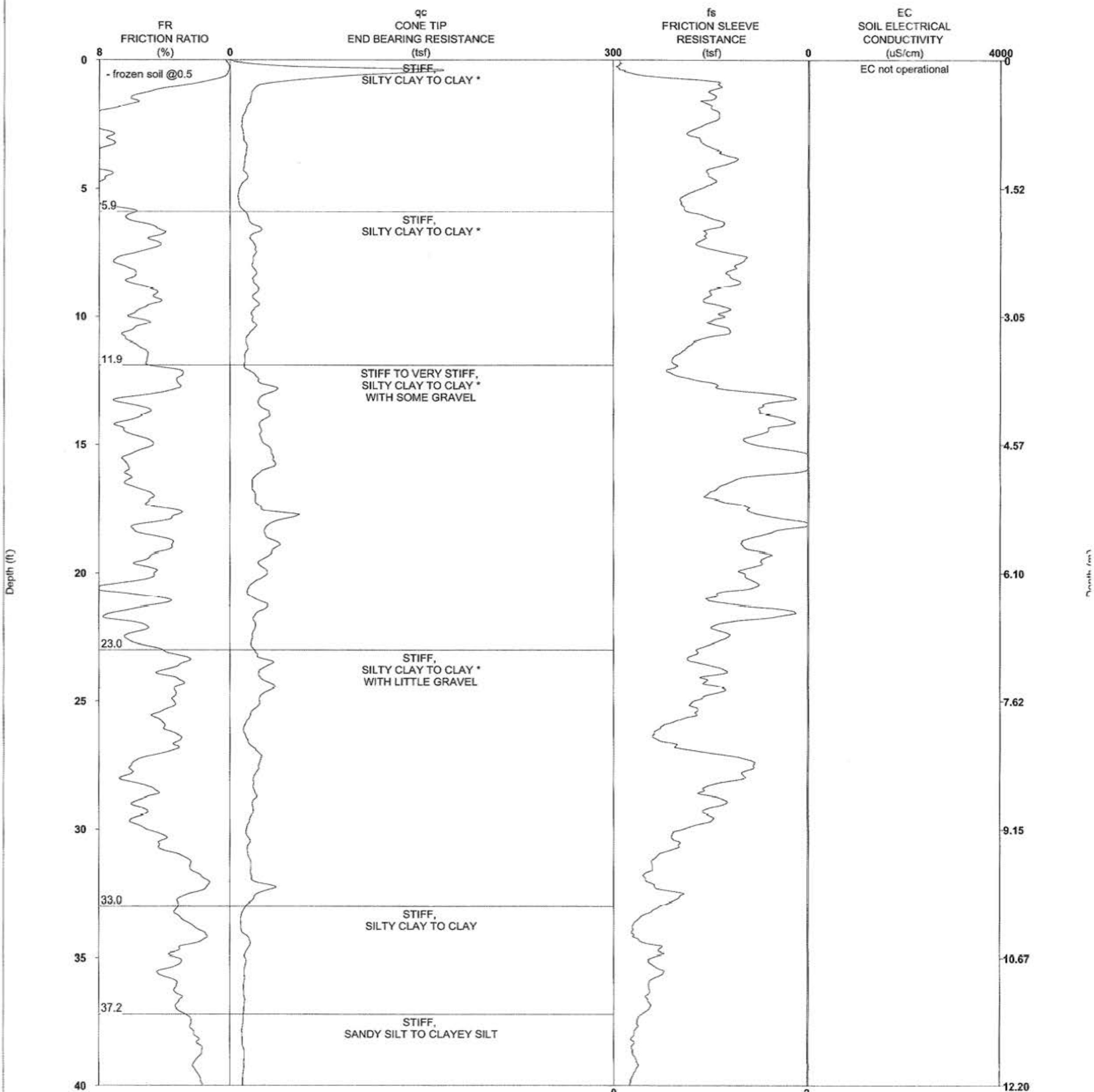
PROJECT NAME:Thomas Hill Site
PROJECT NUMBER:10-110-020

STRATIGRAPHICS

R1 DATE:2/3/2010 TIME:8:59 AM
SOUNDING NUMBER:CC-01

CPCC01

CPTU-EC LOG WITH LITHOLOGIC EVALUATION CPCC01



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Latitude: 39.54378 Longitude: -92.63682

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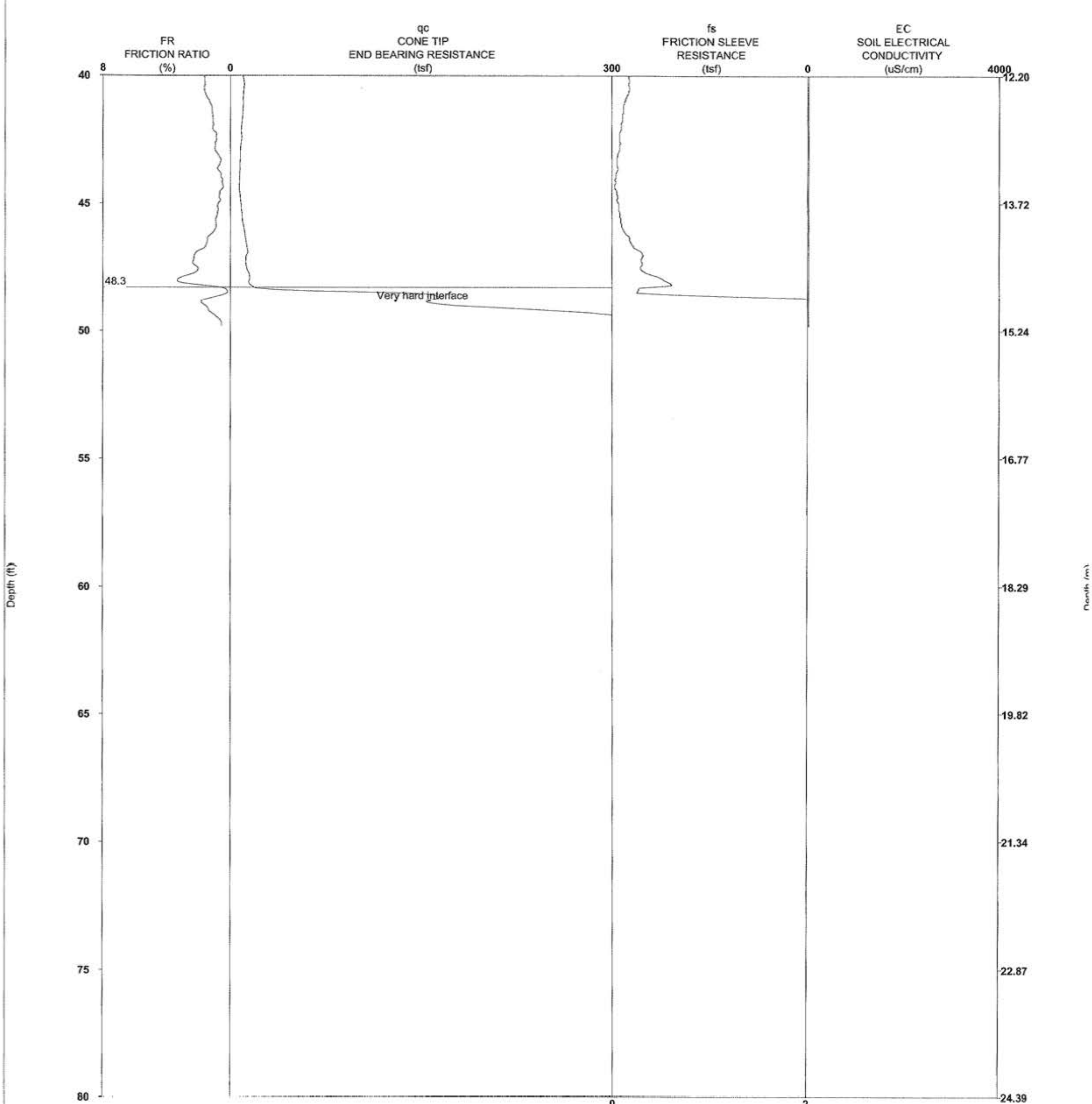
PROJECT NUMBER:10-110-020

STRATIGRAPHICS

R1 DATE:2/3/2010 TIME:8:59 AM
SOUNDING NUMBER:CC-01

CPCC01

CPTU-EC LOG WITH LITHOLOGIC EVALUATION CPCC01



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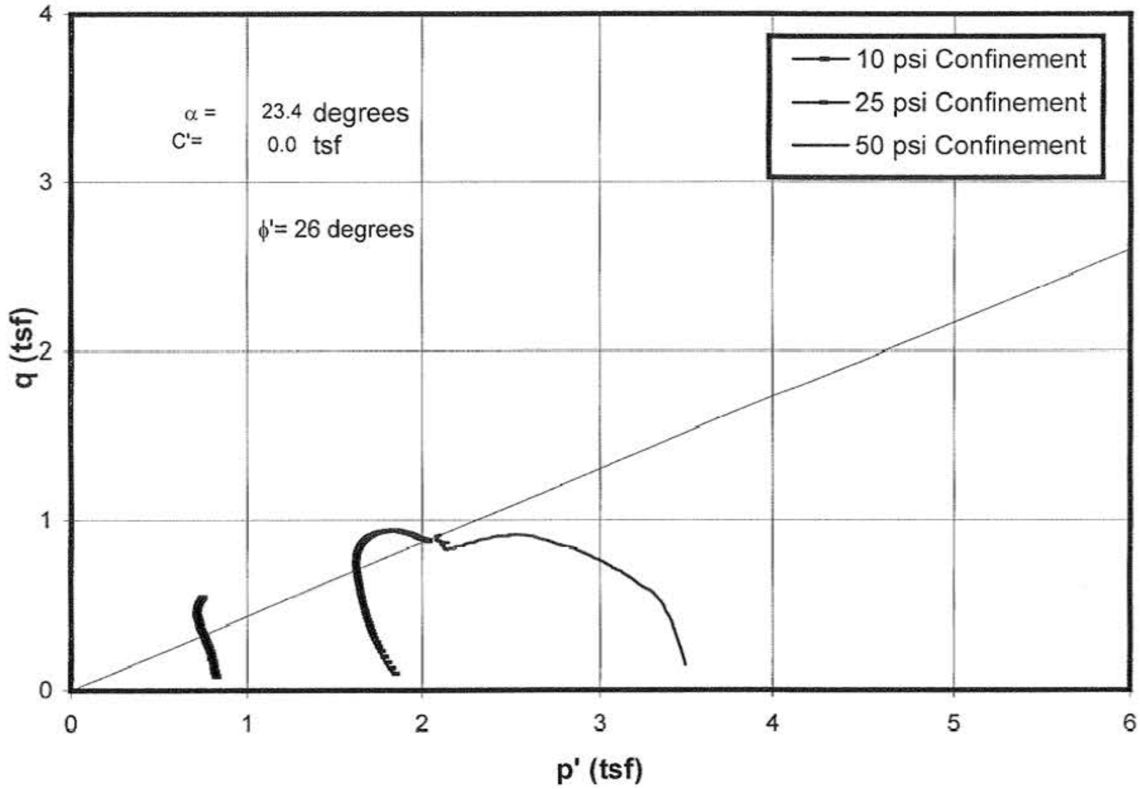
Latitude: 39.54378 Longitude: -92.63682

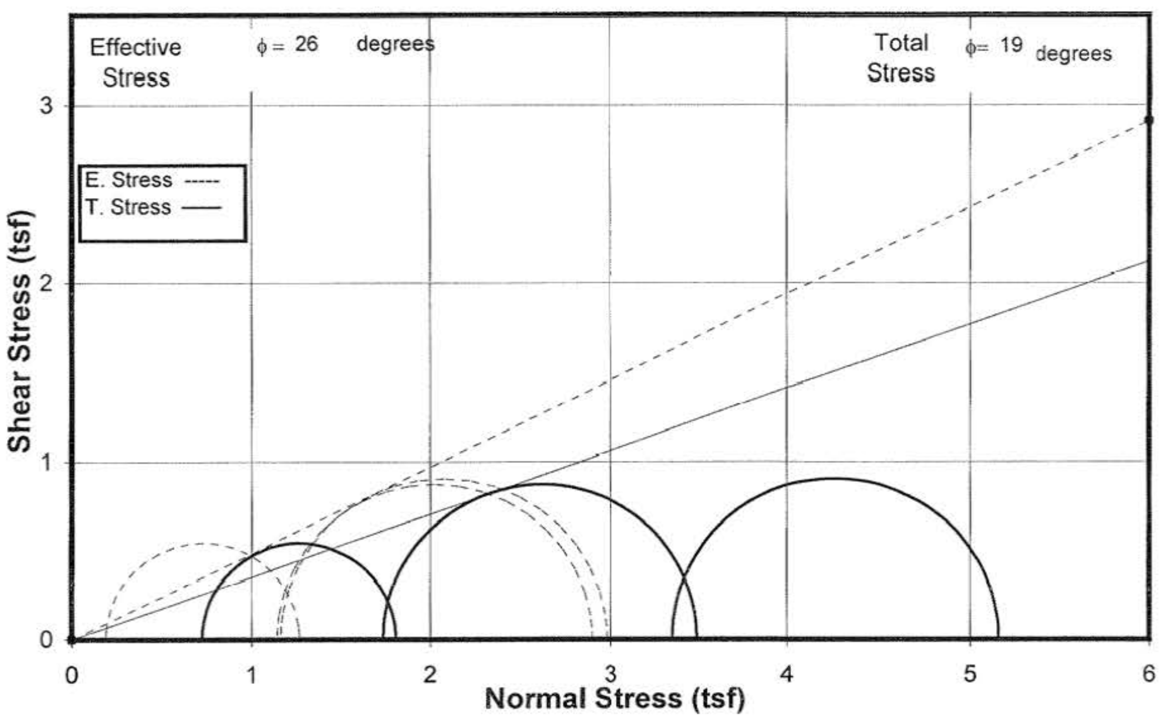
PROJECT NAME:Thomas Hill Site
PROJECT NUMBER:10-110-020

STRATIGRAPHICS

R1 DATE:2/3/2010 TIME:8:59 AM
SOUNDING NUMBER:CC-01

CPCC01





CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST

ASTM D 4767

Project No.: J011309.01

Boring: C-1

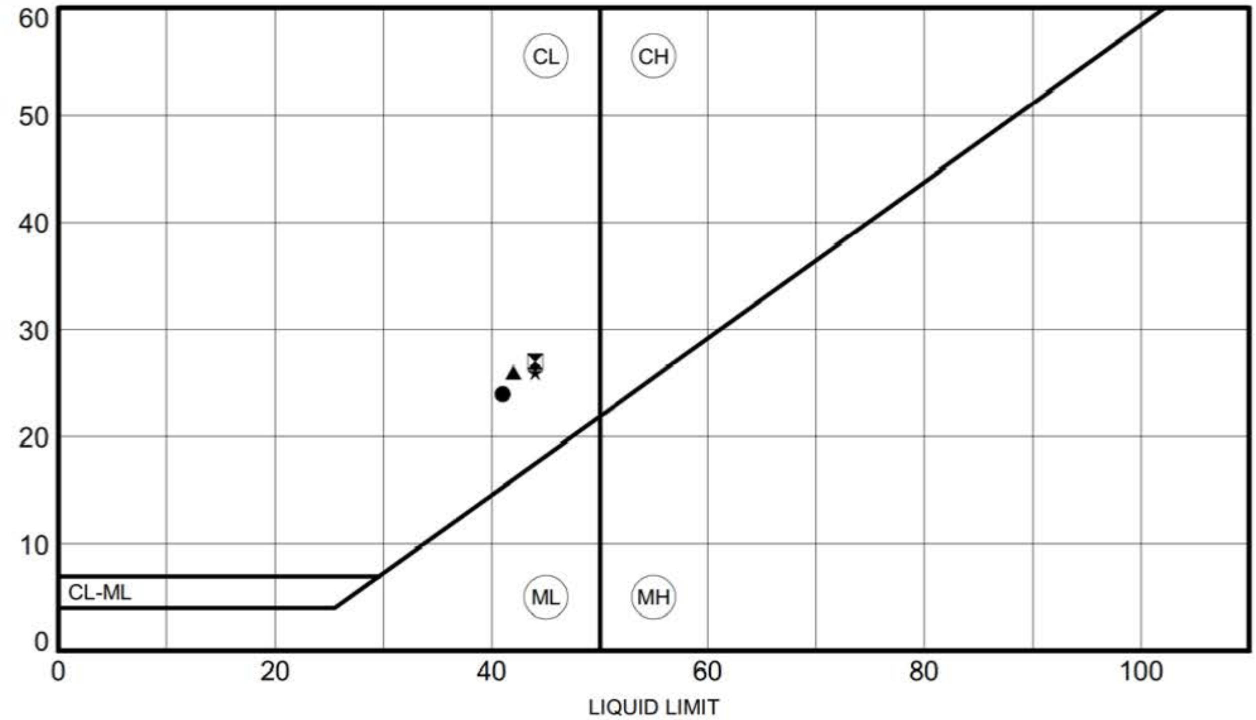
Sample: ST-6 - Depth: 13.5

4168 W. Kearney St.
Springfield, MO 65803
Telephone: (417) 864-6000
Fax: (417) 864-6004

ATTERBERG LIMITS' RESULTS

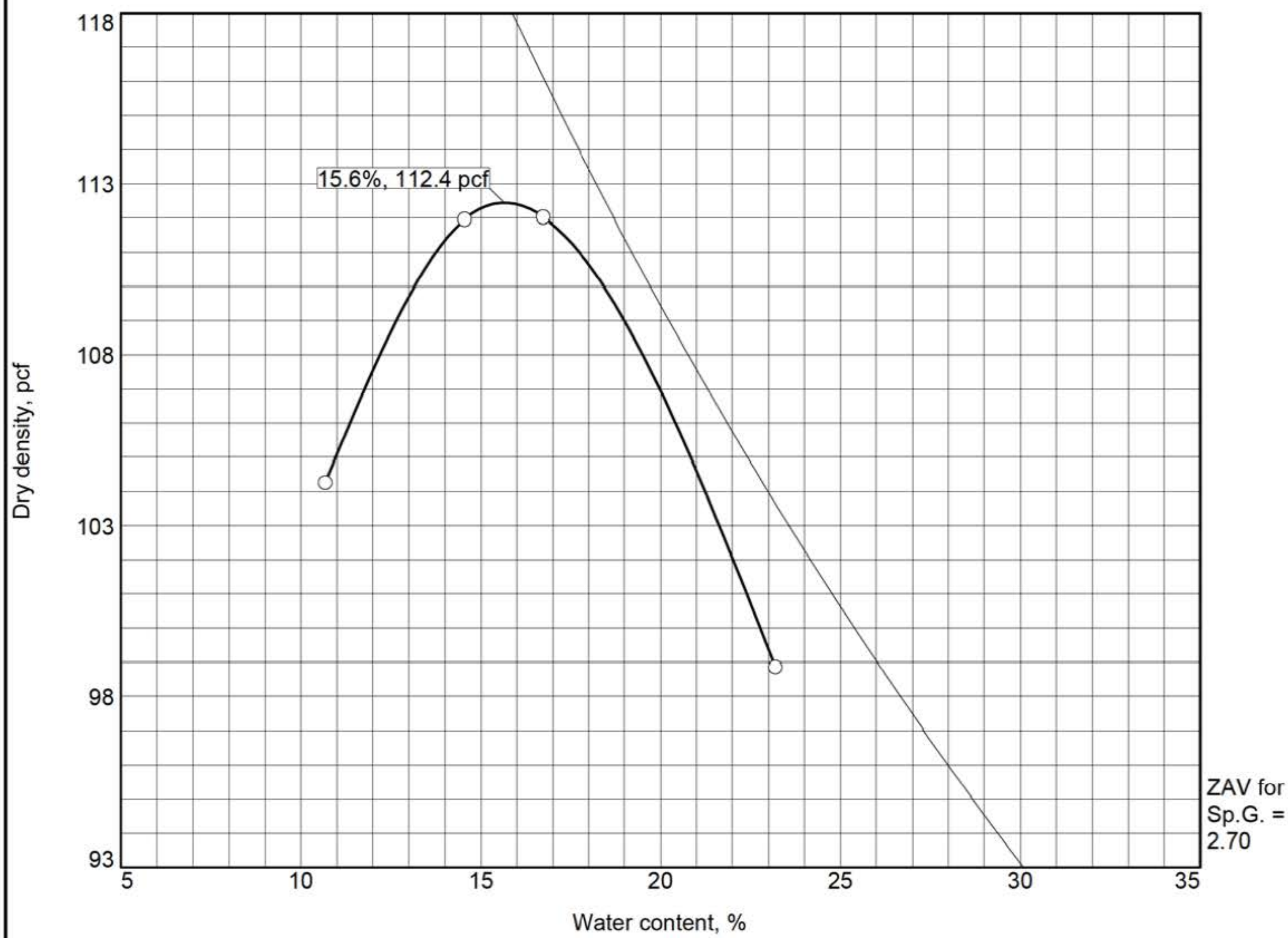
CLIENT Gredell Engineering Resources, Inc. **PROJECT NAME** Laboratory Testing AECI-THEC

PROJECT NO. 229780 **PROJECT LOCATION**



	SAMPLE	DEPTH	LL	PL	PI	w (%)	Classification
●	**AECI-THEC 1**	**NA**	**41**	**17**	**24**	19.6	Lean Clay
⧗	**AECI-THEC 2**	**NA**	**44**	**17**	**27**	18.0	Lean Clay
▲	**AECI-THEC 3**	**NA**	**42**	**16**	**26**	17.5	Lean Clay
★	**AECI-THEC 4**	**NA**	**44**	**18**	**26**	18.4	Lean Clay

COMPACTION TEST REPORT



Test specification: ASTM D 698-91 Procedure A Standard

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > #4	% < No.200
	USCS	AASHTO						
	CL		19.6		41	24		

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 112.4 pcf Optimum moisture = 15.6 %	Lean Clay

Project No. 229780 **Client:** Gredell Engineering Resources, Inc.
Project: Lab Testing AECI-THEC

○ **Location:** AECI-THEC 1

PALMERTON & PARRISH, INC.

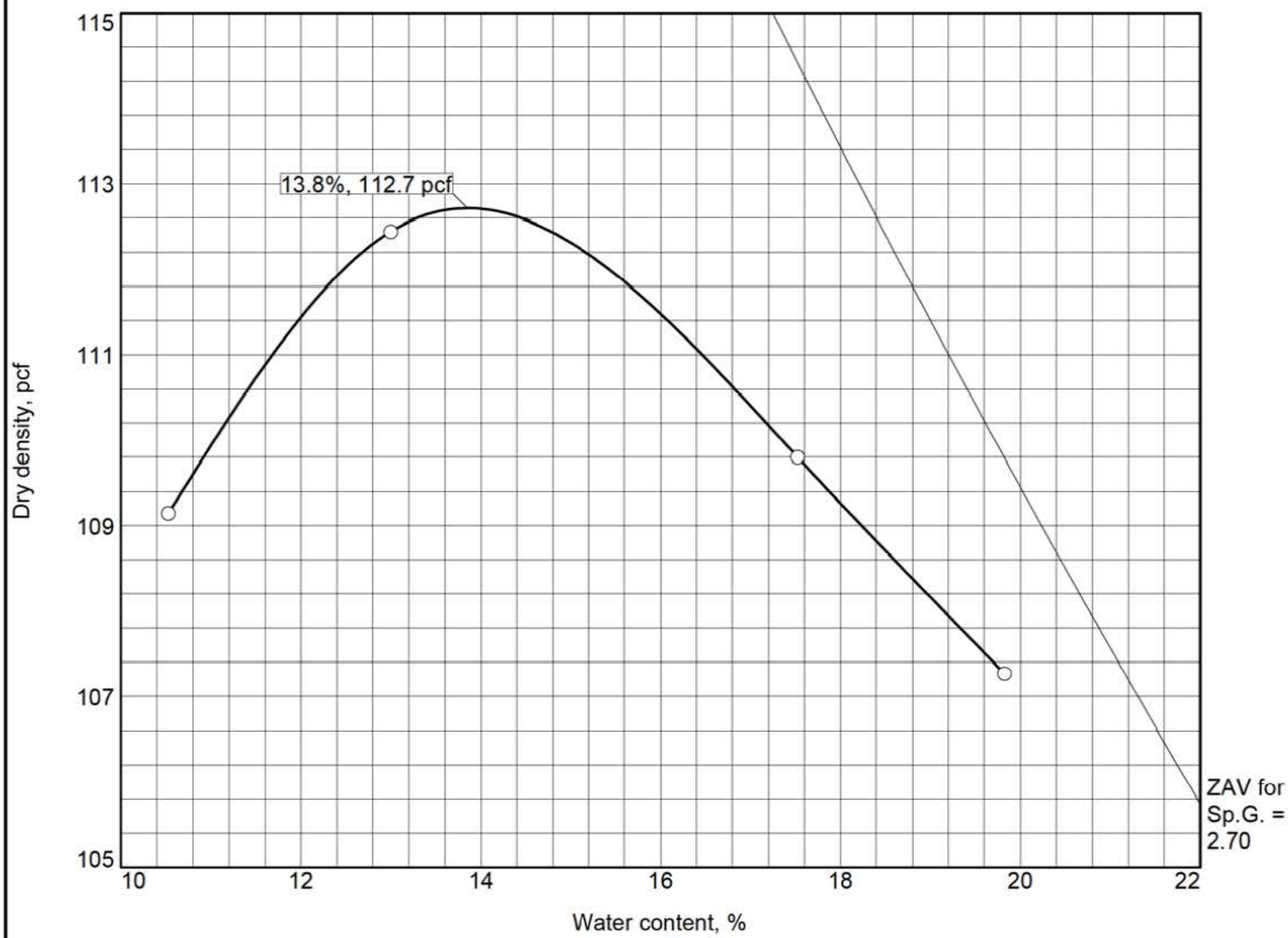
Springfield, MO

Remarks:

Figure

Tested By: KG **Checked By:** JM

COMPACTION TEST REPORT



Test specification: ASTM D 698-91 Procedure A Standard

Elev/ Depth	Classification USCS	Classification AASHTO	Nat. Moist.	Sp.G.	LL	PI	% > #4	% < No.200
	CL		18.4		44	26		

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 112.7 pcf Optimum moisture = 13.8 %	Lean Clay

Project No. 229780 **Client:** Gredell Engineering Resources, Inc.
Project: Lab Testing AECI-THEC

○ **Location:** AECI-THEC 4

PALMERTON & PARRISH, INC.

Springfield, MO

Remarks:

Figure

Tested By: KG **Checked By:** JM

Measurement of Hydraulic Conductivity of Saturated Porous Materials using a Flexible Wall Permeameter
ASTM D 5084 - 00 Method C Test with Increasing Tailwater Level

Client:	Gredell Engineering Resources, Inc.	Job #	229780
Project:	Lab Testing		
Description:	AECI-THEC 1	Date:	8/10/15
Notes:	Remolded to 96.9% of Max Standard Proctor Dry Density and +2.37% Wopt		

Moisture %

wt. tare	50.24	g
wet + tare	139.53	g
dry + tare	125.93	g
moisture	17.97	%

Density

Wet Wt.	627.81	g
Dry Wt.	532.19	g
Height	3.00	in
Diameter	2.81	in
Dry Density	109.0	lbs/ft³
Wet Density	128.6	lbs/ft³

Atterberg

Liquid Limit:	41
Plastic Limit:	17
Plasticity Index:	21

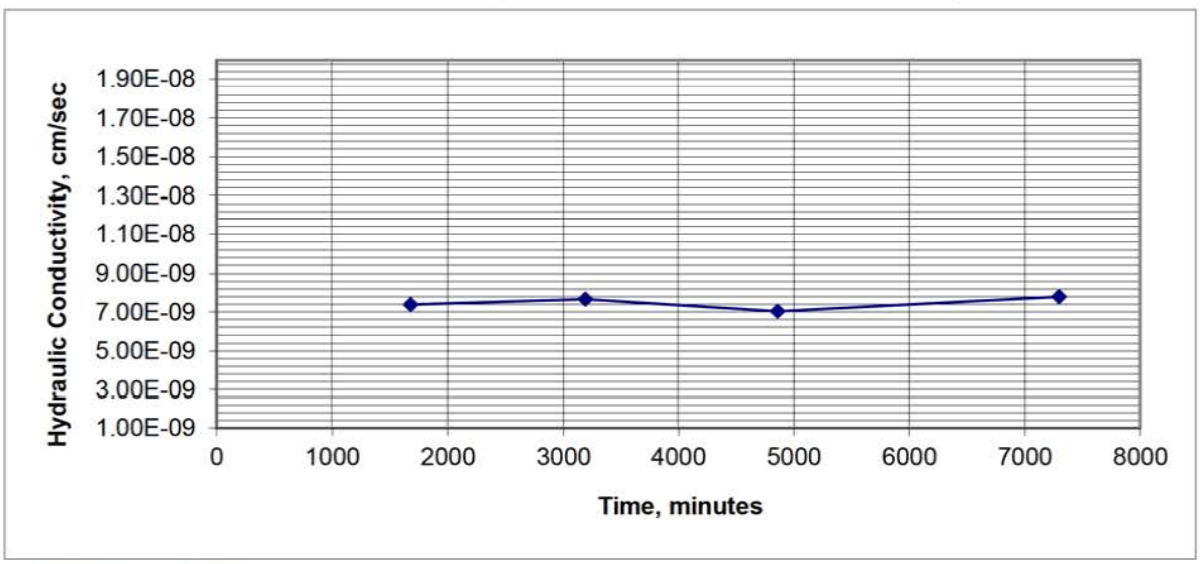
Standard Proctor

γ_{dmax}	112.4	pcf
w_{opt}	15.6	%
% Comp.	96.9	%

Cell Pressure: 90 ***Back Pressure:*** 85

H1, cm	H2, cm	Elapsed Time, min	Hyd Cond cm/sec	Outflow	Inflow	Out/In Ratio	Hyd Grad	% from Mean	Temp: C	Temp. Corr:
180.53	173.47	1677	7.40E-09	0.71	0.68	1.04	23.69	1.04%	20	1.005
173.47	167.12	1512	7.67E-09	0.57	0.68	0.84	22.76	2.59%	20	1.005
167.12	160.92	1670	7.04E-09	0.62	0.6	1.03	21.93	5.88%	20	1.005
160.92	151.37	2439	7.80E-09	0.84	1.04	0.81	21.12	4.34%	20	1.005

Hydraullic Conductivity (k) Average= 7.48E-09 cm/sec



- Geotechnical Services
- Environmental Services
- Material Testing Services

PALMERTON & PARRISH, INC.

JACOBI GEOTECHNICAL ENGINEERING, INC.
www.JacobiEngineer.com
798 Hoff Road O'Fallon, Missouri 63366-1920 (636) 978-7112 (636) 978-7113

September 28, 2015

Mr. Thomas R. Gredell, P.E.
GREDELL Engineering Resources, Inc.
1505 E High St
Jefferson City MO 65101-4826

RE: Laboratory Testing
Thomas Hill Energy Center
JGE No. 15118.3

Dear Mr. Gredell:

On September 15, 2015, Jacobi Geotechnical Engineering received six soil test pit samples, identified as seen below, for analysis. Tests assigned for the samples from your letter dated September 11, 2015 included Moisture Content (ASTM D 2216), Atterberg Limits (ASTM D 4318), and particle size analysis – hydrometer (ASTM D 422) on Test Pit 1, 4.0' and Test Pit 2, 6.0'. Test results are presented in the table below. Attached are the lab data sheets

Sample ID	Moisture Content %	Atterberg Limits	Hydrometer
Test Pit 1, 1.0'	24.5	Liquid Limit: 59 Plastic Limit: 26 Plastic Index: 33 Classification: CH	N/A
Test Pit 1, 4.0'	16.7	Liquid Limit: 38 Plastic Limit: 18 Plastic Index: 20 Classification: SC	Sand 61% Silt 24% Clay 15%
Test Pit 1, 10.0'	16.4	Liquid Limit: 38 Plastic Limit: 19 Plastic Index: 19 Classification: CL	N/A
Test Pit 2, 1.0'	25.8	Liquid Limit: 53 Plastic Limit: 23 Plastic Index: 31 Classification: CH	N/A
Test Pit 2, 2.5'	31.8	Liquid Limit: 58 Plastic Limit: 28 Plastic Index: 30 Classification: CH	N/A
Test Pit 2, 6.0'	19.6	Liquid Limit: 34 Plastic Limit: 18 Plastic Index: 16 Classification: SC	Sand 75% Silt 17% Clay 8%

We will discard the samples in 30 days unless other arrangements are made.

We appreciate the opportunity to provide services on this project. If you have any questions concerning this letter, please call.

Sincerely,

Jacobi Geotechnical Engineering, Inc.

Matt Schultz
Staff Engineer

Carl L. Jacobi, P.E.
Principal

STATE OF MISSOURI
CARL L.
JACOBI
NUMBER
PE-020950
PROFESSIONAL ENGINEER

Enclosure: Lab Data Sheets

JACOBI GEOTECHNICAL ENGINEERING, INC.

ATTERBERG LIMIT DATA SHEET
(ASTM D 4318)

GENERAL INFORMATION

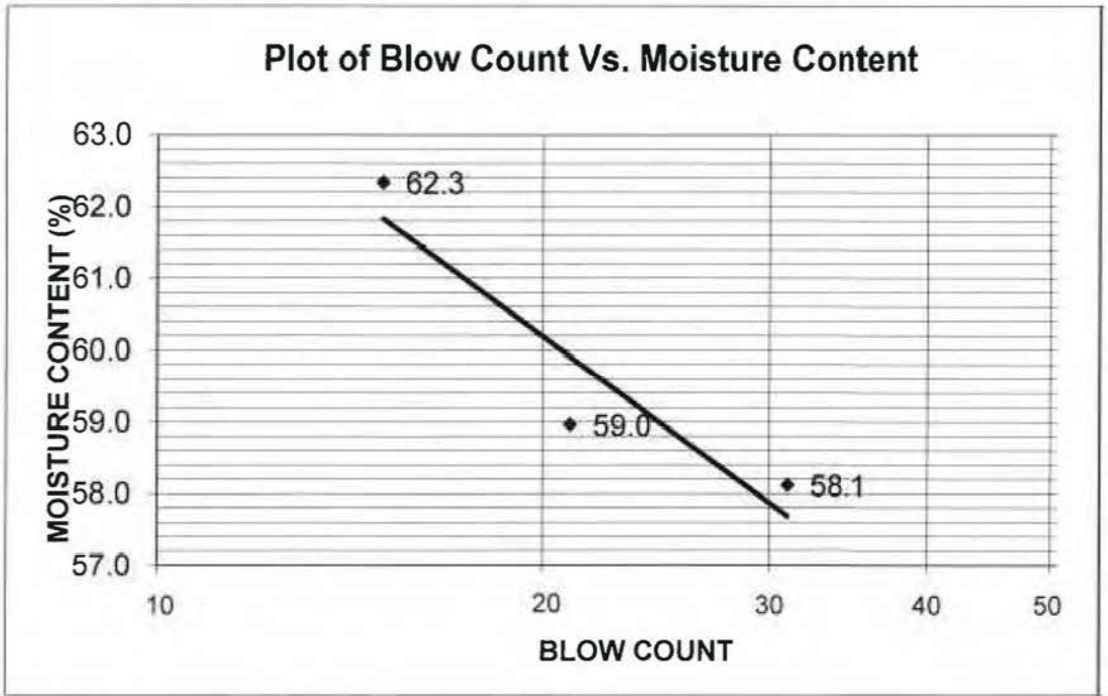
PROJECT NAME:	Thomas Hill Energy Center	TESTED BY:	JRP
JGE JOB No.:	15118	CALCULATED BY:	JRP
TEST DATE:	9/16/2015	CHECKED BY:	MJS
EQUIPMENT:	AL-2		

SOIL INFORMATION

TEST PIT NO.:	TP-1	SAMPLE :	BS-1	DEPTH:	1'
SOIL DESC:					

TESTING DATA

	LIQUID LIMIT			PLASTIC LIMIT		Natural
BLOW COUNT	31	21	15			
WET + TARE	13.86	12.39	11.67	8.22	10.93	
DRY + TARE	8.92	7.95	7.35	6.61	8.80	
WT. WATER	4.94	4.44	4.32	1.61	2.13	
WT. TARE	0.42	0.42	0.42	0.42	0.42	
WT. DRY SOIL	8.50	7.53	6.93	6.19	8.38	
% MOISTURE	58.1	59.0	62.3	26.0	25.4	24.5



LIQUID LIMIT	59
PLASTIC LIMIT	26
PLASTICITY INDEX	33
CLASSIFICATION	CH

Remarks:

ATTERBERG LIMIT DATA SHEET
(ASTM D 4318)

GENERAL INFORMATION

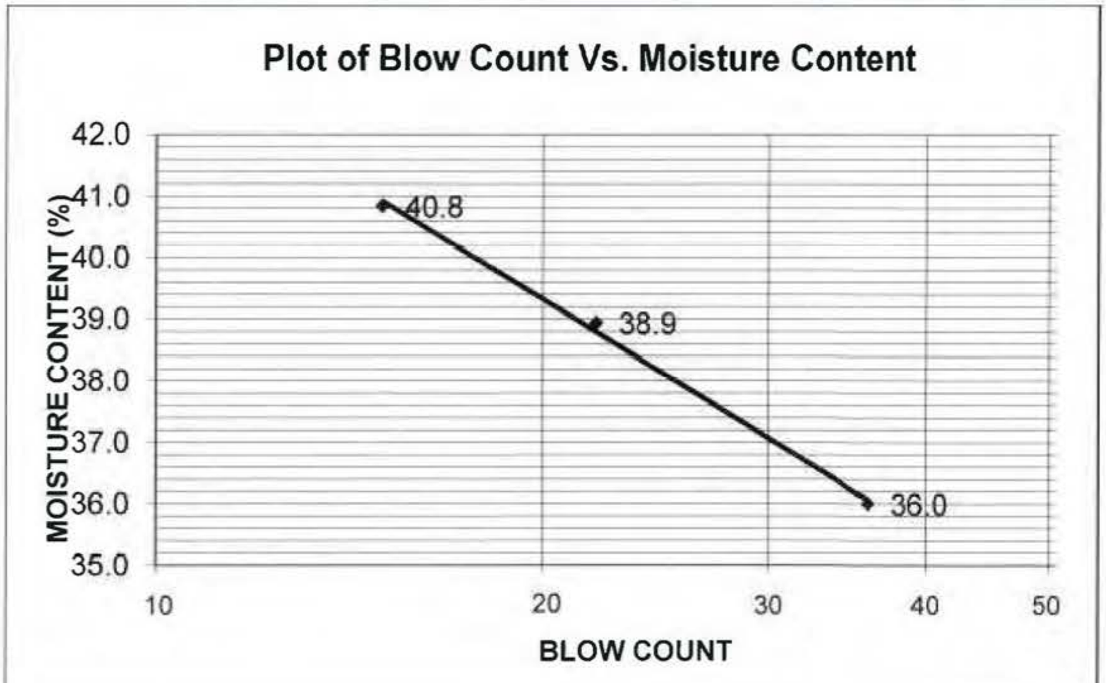
PROJECT NAME:	Thomas Hill Energy Center	**TESTED BY:**	JRP
JGE JOB No.:	15118	**CALCULATED BY:**	JRP
TEST DATE:	9/17/2015	**CHECKED BY:**	MJS
EQUIPMENT:	AL-2		

SOIL INFORMATION

TEST PIT NO.:	TP-1	**SAMPLE :**	BS-2	**DEPTH:**	4'
SOIL DESC:					

TESTING DATA

	LIQUID LIMIT			PLASTIC LIMIT		Natural
BLOW COUNT	36	22	15			
WET + TARE	17.76	16.62	15.18	9.12	8.93	
DRY + TARE	13.17	12.08	10.90	7.81	7.62	
WT. WATER	4.59	4.54	4.28	1.31	1.31	
WT. TARE	0.42	0.42	0.42	0.42	0.42	
WT. DRY SOIL	12.75	11.66	10.48	7.39	7.20	
% MOISTURE	36.0	38.9	40.8	17.7	18.2	16.7



LIQUID LIMIT	38
PLASTIC LIMIT	18
PLASTICITY INDEX	20
CLASSIFICATION	SC

Remarks:

JACOBI GEOTECHNICAL ENGINEERING, INC.

ATTERBERG LIMIT DATA SHEET
(ASTM D 4318)

GENERAL INFORMATION

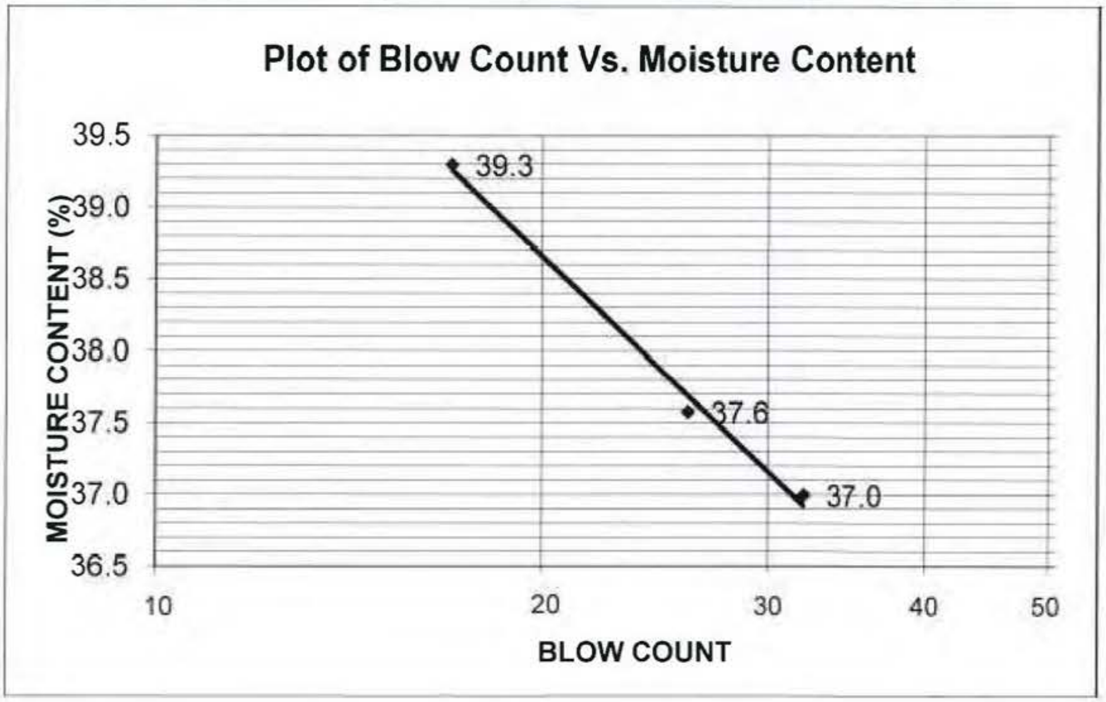
PROJECT NAME: Thomas Hill Energy Center		**TESTED BY:** JRP
JGE JOB No.: 15118		**CALCULATED BY:** JRP
TEST DATE: 9/16/2015		**CHECKED BY:** MJS
EQUIPMENT: AL-2		

SOIL INFORMATION

TEST PIT NO.: TP-1	**SAMPLE :** BS-3	**DEPTH:** 10'
SOIL DESC:		

TESTING DATA

	LIQUID LIMIT			PLASTIC LIMIT		Natural
BLOW COUNT	32	26	17			
WET + TARE	16.38	16.75	15.84	9.36	11.40	
DRY + TARE	12.07	12.29	11.49	7.95	9.64	
WT. WATER	4.31	4.46	4.35	1.41	1.76	
WT. TARE	0.42	0.42	0.42	0.42	0.42	
WT. DRY SOIL	11.65	11.87	11.07	7.53	9.22	
% MOISTURE	37.0	37.6	39.3	18.7	19.1	16.4



LIQUID LIMIT	38
PLASTIC LIMIT	19
PLASTICITY INDEX	19
CLASSIFICATION	CL

Remarks:

798 HOFF RD OFALLON, MO 63366-1920 PHONE: (636) 978-7112 FAX: (636) 978-7113 www.jacobiengineer.com

JACOBI GEOTECHNICAL ENGINEERING, INC.

ATTERBERG LIMIT DATA SHEET

(ASTM D 4318)

GENERAL INFORMATION

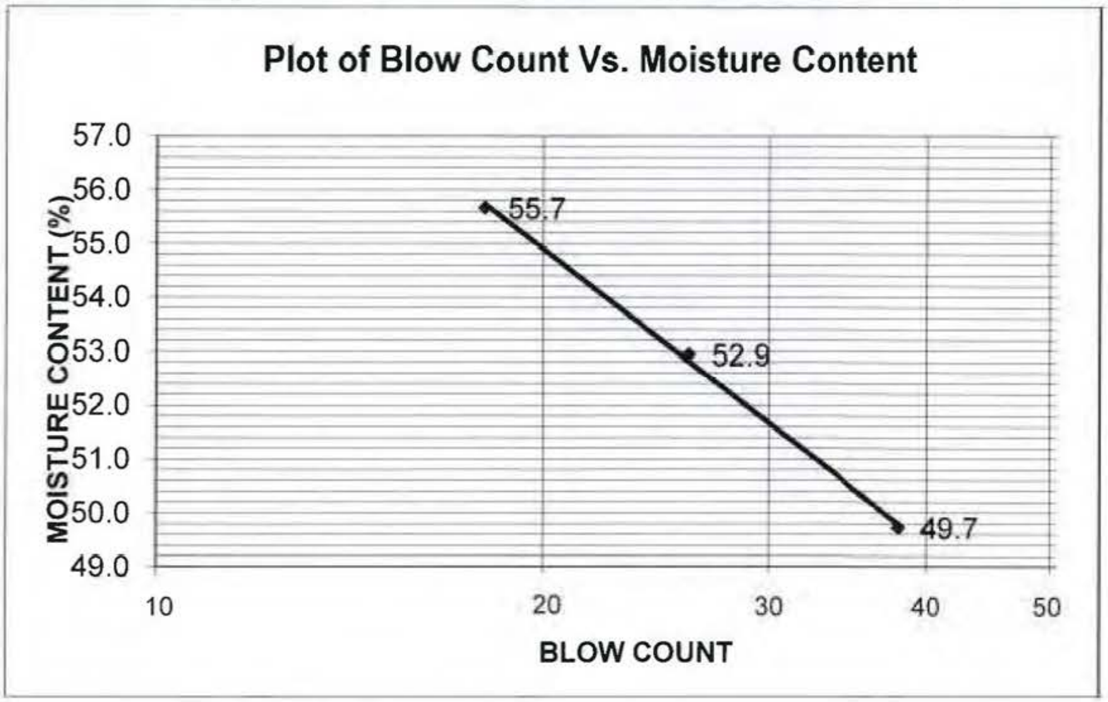
PROJECT NAME:	Thomas Hill Energy Center	**TESTED BY:**	JRP
JGE JOB No.:	15118	**CALCULATED BY:**	JRP
TEST DATE:	9/16/2015	**CHECKED BY:**	MJS
EQUIPMENT:	AL-2		

SOIL INFORMATION

TEST PIT NO.:	TP-2	**SAMPLE :**	BS-1	**DEPTH:**	1'
SOIL DESC:					

TESTING DATA

	LIQUID LIMIT			PLASTIC LIMIT		Natural
BLOW COUNT	38	26	18			
WET + TARE	14.21	14.46	13.06	8.30	8.63	
DRY + TARE	9.63	9.60	8.54	6.86	7.10	
WT. WATER	4.58	4.86	4.52	1.44	1.53	
WT. TARE	0.42	0.42	0.42	0.42	0.42	
WT. DRY SOIL	9.21	9.18	8.12	6.44	6.68	
% MOISTURE	49.7	52.9	55.7	22.4	22.9	25.8



LIQUID LIMIT	53
PLASTIC LIMIT	23
PLASTICITY INDEX	31
CLASSIFICATION	CH

Remarks:

JACOBI GEOTECHNICAL ENGINEERING, INC.

ATTERBERG LIMIT DATA SHEET
(ASTM D 4318)

GENERAL INFORMATION

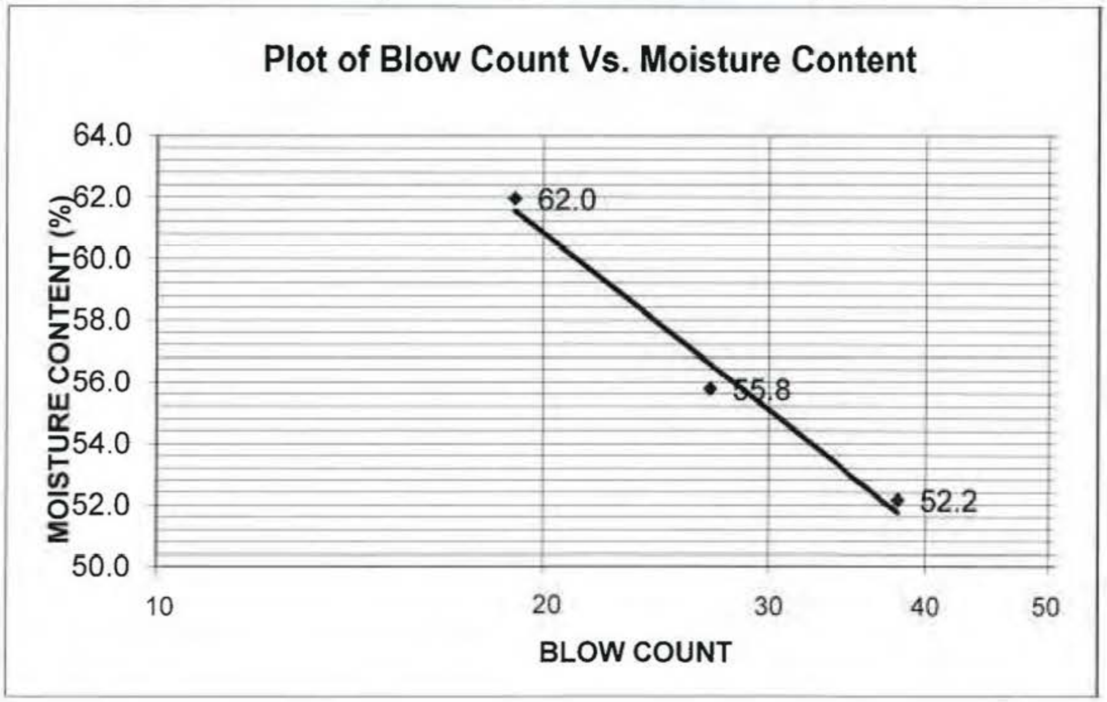
PROJECT NAME:	Thomas Hill Energy Center	**TESTED BY:**	JRP
JGE JOB No.:	15118	**CALCULATED BY:**	JRP
TEST DATE:	9/18/2015	**CHECKED BY:**	MJS
EQUIPMENT:	AL-2		

SOIL INFORMATION

TEST PIT NO.:	TP-2	**SAMPLE :**	BS-2	**DEPTH:**	2.5'
SOIL DESC:					

TESTING DATA

	LIQUID LIMIT			PLASTIC LIMIT		Natural
BLOW COUNT	38	27	19			
WET + TARE	13.78	13.52	11.66	9.02	9.62	
DRY + TARE	9.20	8.83	7.36	7.18	7.59	
WT. WATER	4.58	4.69	4.30	1.84	2.03	
WT. TARE	0.42	0.42	0.42	0.42	0.42	
WT. DRY SOIL	8.78	8.41	6.94	6.76	7.17	
% MOISTURE	52.2	55.8	62.0	27.2	28.3	31.8



LIQUID LIMIT	58
PLASTIC LIMIT	28
PLASTICITY INDEX	30
CLASSIFICATION	CH

Remarks:

798 HOFF RD OFALLON, MO 63366-1920 PHONE: (636) 978-7112 FAX: (636) 978-7113 www.jacobiengineer.com

JACOBI GEOTECHNICAL ENGINEERING, INC.

ATTERBERG LIMIT DATA SHEET

(ASTM D 4318)

GENERAL INFORMATION

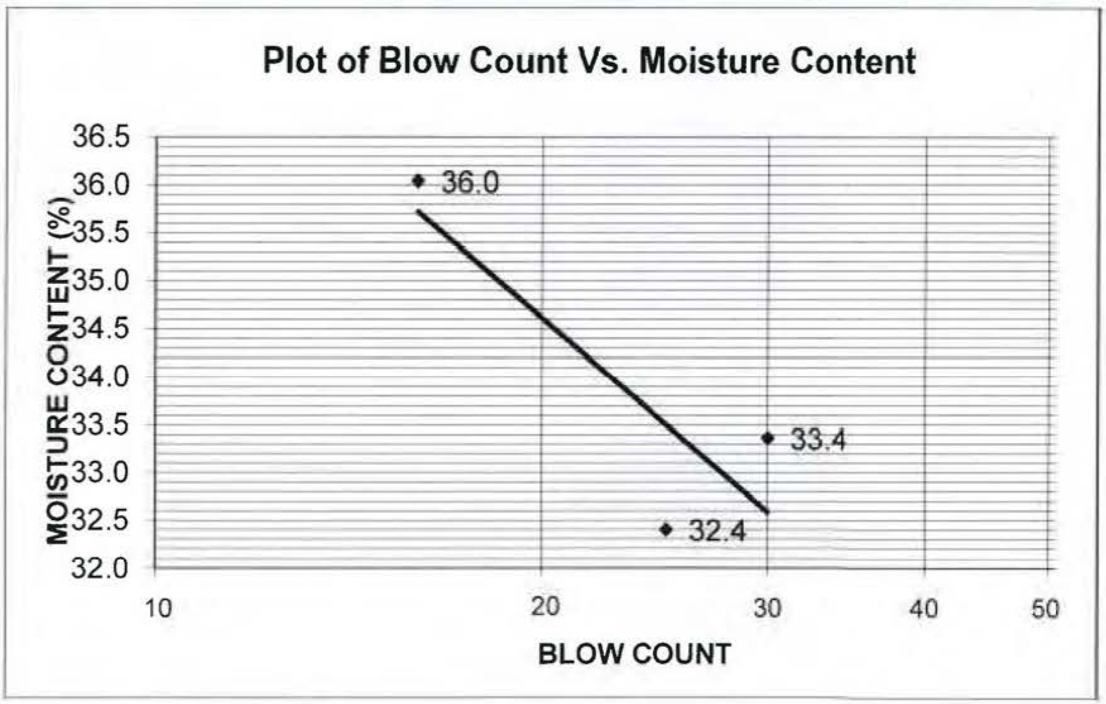
PROJECT NAME:	Thomas Hill Energy Center	**TESTED BY:**	JRP
JGE JOB No.:	15118	**CALCULATED BY:**	JRP
TEST DATE:	9/21/2015	**CHECKED BY:**	MJS
EQUIPMENT:	AL-2		

SOIL INFORMATION

TEST PIT NO.:	TP-2	**SAMPLE :**	BS-3	**DEPTH:**	6'
SOIL DESC:					

TESTING DATA

	LIQUID LIMIT			PLASTIC LIMIT		Natural
BLOW COUNT	30	25	16			
WET + TARE	17.61	17.42	16.01	8.25	10.06	
DRY + TARE	13.31	13.26	11.88	7.09	8.57	
WT. WATER	4.30	4.16	4.13	1.16	1.49	
WT. TARE	0.42	0.42	0.42	0.42	0.42	
WT. DRY SOIL	12.89	12.84	11.46	6.67	8.15	
% MOISTURE	33.4	32.4	36.0	17.4	18.3	19.6



LIQUID LIMIT	34
PLASTIC LIMIT	18
PLASTICITY INDEX	16
CLASSIFICATION	SC

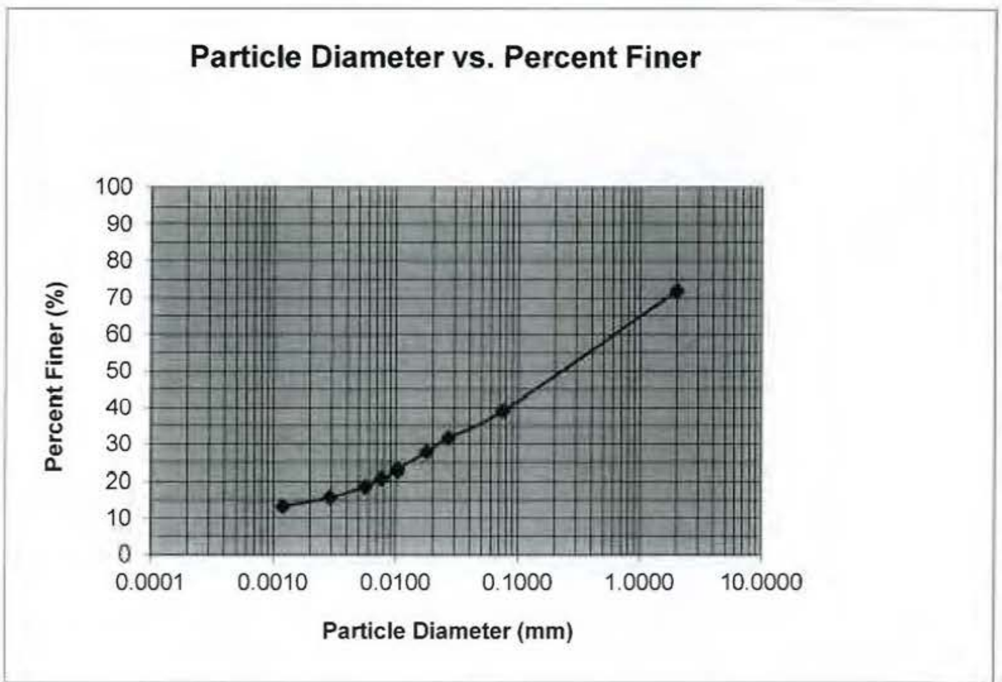
Remarks:

JACOBI GEOTECHNICAL ENGINEERING, INC.

HYDROMETER ANALYSIS DATA SHEET (MINUS # 10 FRACTION ONLY)

PROJECT:	Thomas Hill Energy Center	TEST DATE:	Sep-15
JOB NO.:	15118	TESTED BY:	JM
BORING NO.:	TP-1	CALCULATED BY:	JM
SAMPLE :		REMARKS:	
DEPTH:	4 feet		
SAMPLE DESCRIPTION:	Clayey Sand SC	MOISTURE CONTENT	
PERCENT RETAINED ON NO. 200 SIEVE	60.9%	TARE NO.:	
		WET + TARE:	
		DRY + TARE:	
		WT. TARE:	
		% MOISTURE:	

PARTICLE DIAMETER (mm)	PERCENT FINER PARTIAL (hydro only)	PERCENT FINER TOTAL w/+10 frac
2		71.86
0.07500		39.07
0.02680		31.65
0.01780		27.80
0.01030		23.04
0.00760		20.81
0.00560		18.65
0.00290		15.69
0.00120		13.29



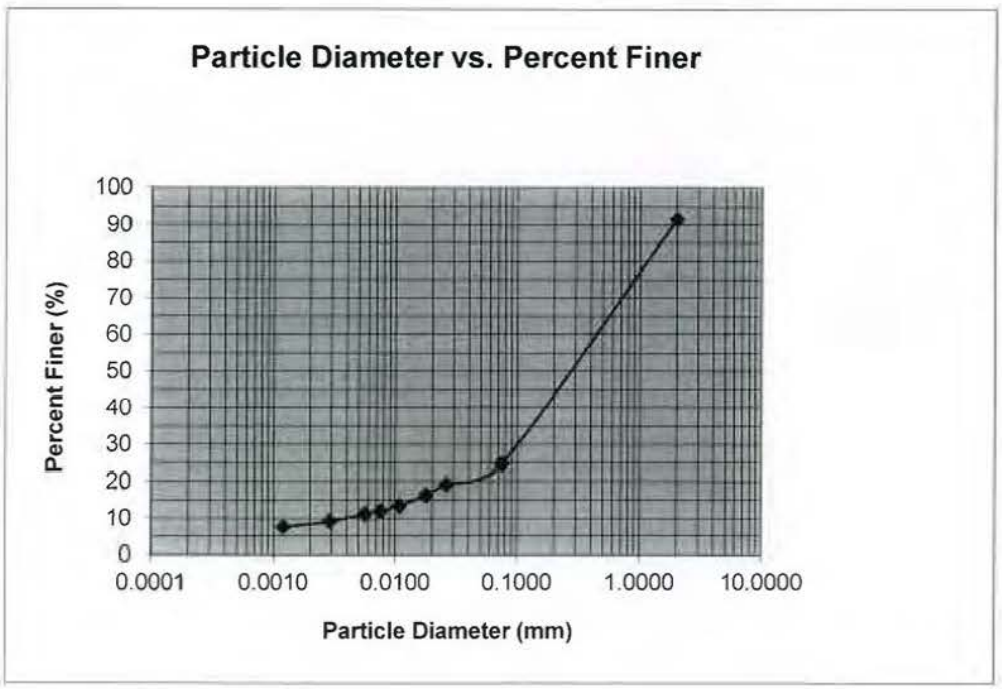
Soil Analysis	
Sand =	61%
Silt =	24%
Clay =	15%

JACOBI GEOTECHNICAL ENGINEERING, INC.

HYDROMETER ANALYSIS DATA SHEET (MINUS # 10 FRACTION ONLY)

PROJECT: Thomas Hill Energy Center		TEST DATE: Sep-15	
JOB NO.: 15118		TESTED BY: JM	
BORING NO.: TP-2		CALCULATED BY: JM	
SAMPLE :		REMARKS:	
DEPTH: 6 feet			
SAMPLE DESCRIPTION: Clayey Sand SC		MOISTURE CONTENT	
PERCENT RETAINED ON NO. 200 SIEVE	75.3%	TARE NO.:	
		WET + TARE:	
		DRY + TARE:	
		WT. TARE:	
		% MOISTURE:	

PARTICLE DIAMETER (mm)	PERCENT FINER PARTIAL (hydro only)	TOTAL w/+10 frac
2		91.43
0.07500		24.66
0.02630		19.29
0.01780		16.27
0.01080		13.46
0.00750		11.91
0.00560		11.11
0.00290		9.12
0.00120		7.57



Soil Analysis	
Sand =	75%
Silt =	17%
Clay =	8%

en i i e i n e i e
ee
e e n i
573-634-3455 573-634-8898 (fax)

October 5, 2015

Mr. Bruce Dawson
Gredell Engineering Resources, Inc.
1505 East High Street
Jefferson City, MO 65101

RE: Laboratory Soil Testing
Job No. 15-075M
Associated Electric Coop. Inc.
Thomas Hill, MO
Report # 21237

Dear Bruce,

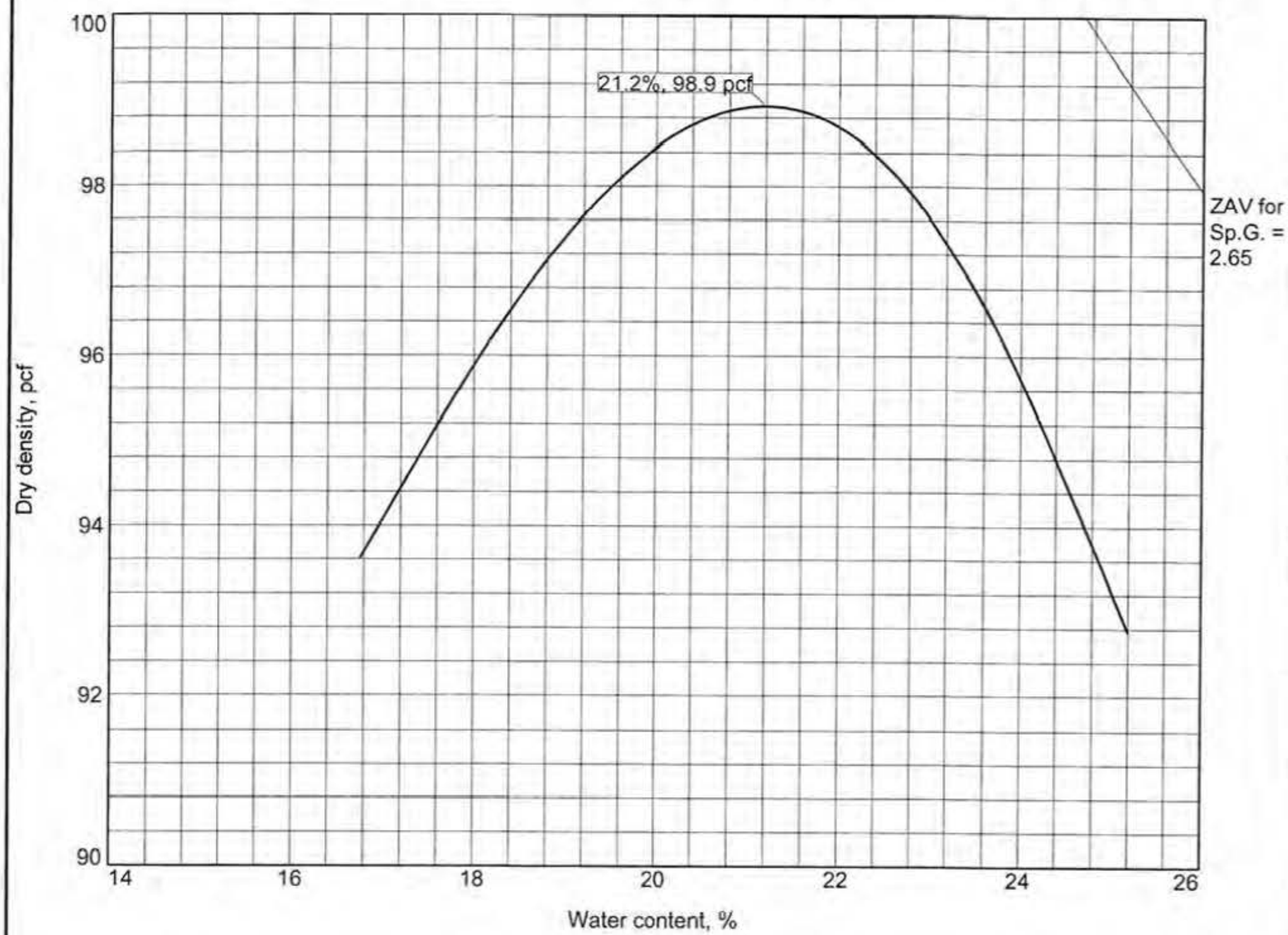
The following are the laboratory soil testing results conducted on soil delivered to our office on September 29, 2015.

Sample Number	Moisture Content %	Atterberg Limits		
		LL	PL	PI
T-3, AECI - THEC	19.9	70	26	44

If you have any questions, please contact me at 573-634-3455.
Sincerely,
CENTRAL MISSOURI PROFESSIONAL SERVICES, INC.

Robert M Bates, PE

LABORATORY COMPACTION CHARACTERISTICS OF SOIL



Test specification: ASTM D698, Method A

Elev/ Depth	Classification USCS	Classification AASHTO	Nat. Moist.	Sp.G.	LL	PI	% > #4	% < No.200
	CH		29.0		84	66		

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 98.9 pcf Optimum moisture = 21.2 %	FAT CLAY: Brown and tan, trace gray

Project No. C15076 **Client:** Gredell Engineering Resources, Inc
Project: Associated Electric Cooperative Earthwork Testing
5693 Highway F, Clifton Hill, MO 65244
○**Location:** On-Site Borrow Area **Sample Number:** 1

CROCKETT
GEOTECHNICAL · TESTING LAB

Remarks:

Date Sampled: 09/28/15
Date Tested: 09/30/15
Date Reported: 10/07/15

Tested By: Levi Strodtman **Checked By:** Shane Steinman, E.I.

Shane Stein

APPENDIX B

Analyses

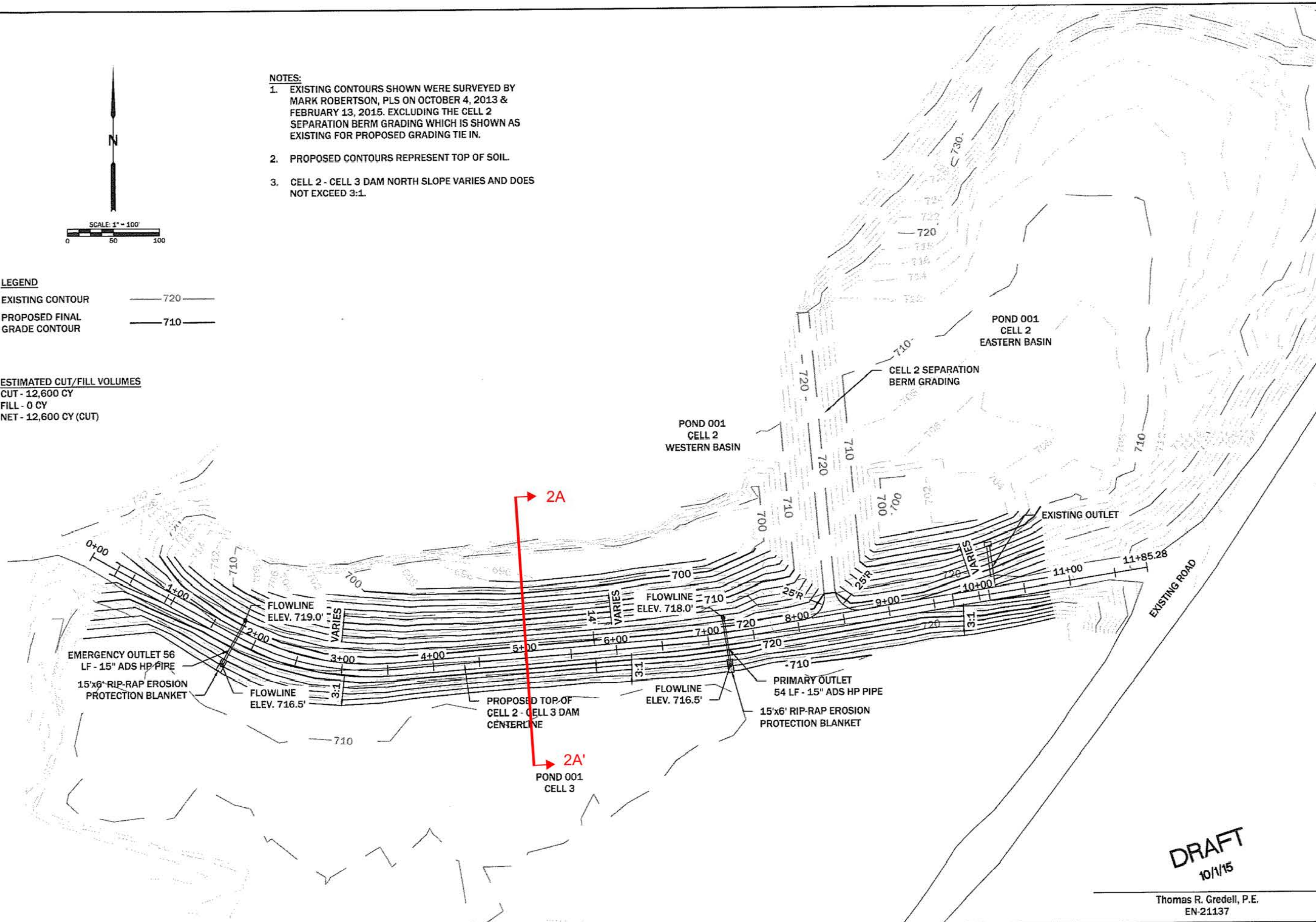
Historic Grading Plans and Cross Sections

NOTES:
1. EXISTING CONTOURS SHOWN WERE SURVEYED BY MARK ROBERTSON, PLS ON OCTOBER 4, 2013 & FEBRUARY 13, 2015. EXCLUDING THE CELL 2 SEPARATION BERM GRADING WHICH IS SHOWN AS EXISTING FOR PROPOSED GRADING TIE IN.
2. PROPOSED CONTOURS REPRESENT TOP OF SOIL.
3. CELL 2 - CELL 3 DAM NORTH SLOPE VARIES AND DOES NOT EXCEED 3:1.

LEGEND

EXISTING CONTOUR	——720——
PROPOSED FINAL GRADE CONTOUR	——710——

ESTIMATED CUT/FILL VOLUMES
CUT - 12,600 CY
FILL - 0 CY
NET - 12,600 CY (CUT)



Thomas R. Gredell, P.E.
EN-21137

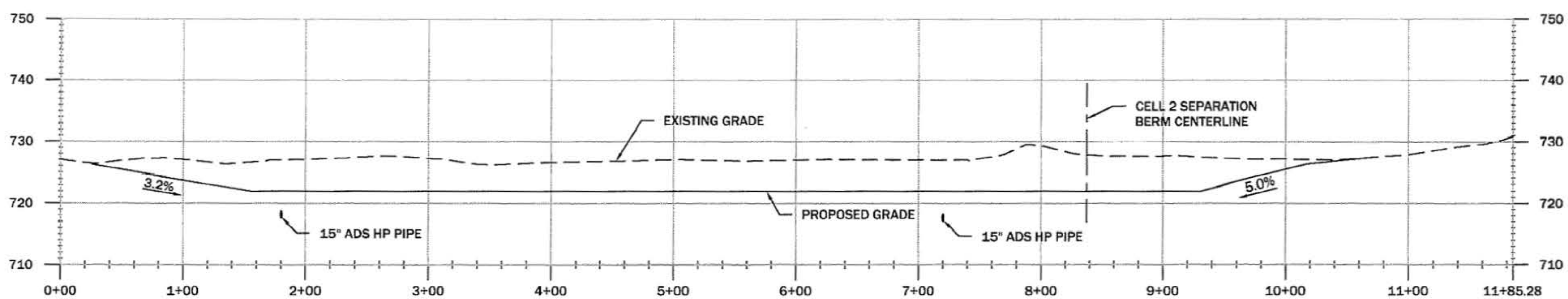
#	DATE	REVISION DESCRIPTION	BY

THOMAS HILL ENERGY CENTER

CELL 2 - CELL 3 DAM GRADING PLAN POND 001 - CELL 2 SEPARATION BERM

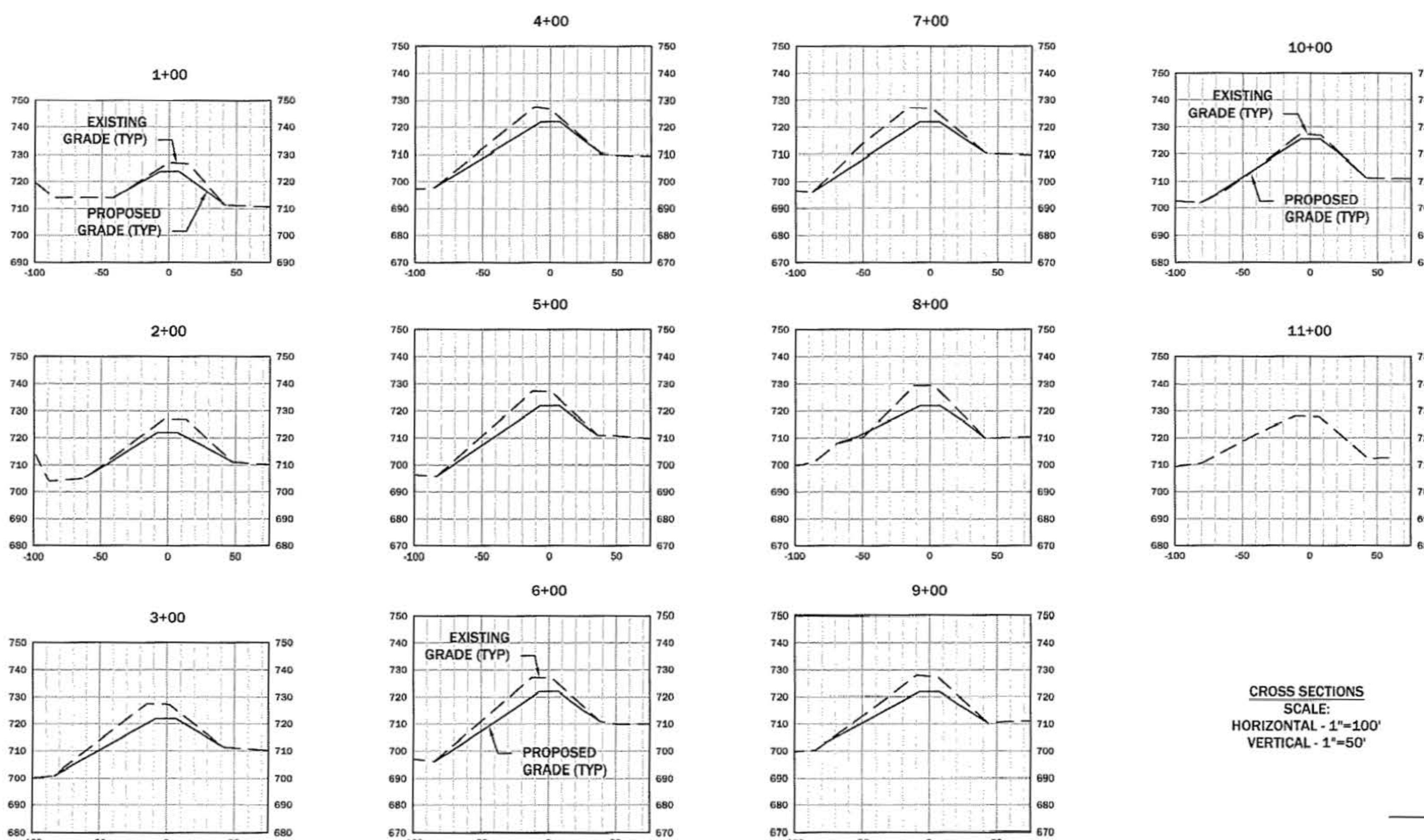
SURVEYED	DESIGNED	DRAWN	CHECKED	APPROVED	DATE	SCALE	PROJECT NAME	FILE NAME	SHEET #
NA	MW	AJK	AR	TRG	10/2015	AS NOTED	AECI/THEC	SEPARATION BERM	5 OF 7

GREDELL Engineering Resources, Inc.
ENVIRONMENTAL ENGINEERING LAND - AIR - WATER
1505 East High Street, Jefferson City, Missouri
Telephone: (573) 659-9078
Facsimile: (573) 659-9079
MO CORP. ENGINEERING LICENSE NO. E-2005001669-D



CELL 2-CELL 3 DAM
SCALE:
HORIZONTAL - 1"=100'
VERTICAL - 1"=20'

NOTE:
1. SEE SHEET 7 FOR CELL 2 - CELL 3 DAM DETAILS.



CROSS SECTIONS
SCALE:
HORIZONTAL - 1"=100'
VERTICAL - 1"=50'

DRAFT
10/1/15

Thomas R. Gredell, P.E.
EN-21137

#	DATE	REVISION DESCRIPTION	BY

CELL 2 - CELL 3 DAM PROFILE AND CROSS SECTIONS
POND 001 - CELL 2
SEPARATION BERM

THOMAS HILL ENERGY CENTER

aeci

SURVEYED	DESIGNED	DRAWN	CHECKED	APPROVED	DATE	SCALE	PROJECT NAME	FILE NAME	SHEET #
NA	MW	AJK	AR	TRG	10/2015	AS NOTED	AECI-THEC	SEPARATION BERM	6 OF 7

GREDELL Engineering Resources, Inc.
ENVIRONMENTAL ENGINEERING LAND - AIR - WATER
1505 East High Street
Jefferson City, Missouri
Telephone: (573) 659-9078
Facsimile: (573) 659-9079
MO CORP. ENGINEERING LICENSE NO. E-2001001669

M:\Share\CADD\Files\AECI\THEC\AECI-THEC ASH VOLUME\POND 001 CELL 2 - SEPARATION BERM\SEPARATION BERM.DWG, CELL 2 - CELL 3 DAM PROFILE AND CROSS SECTIONS, 10/1/2015 2:51:50 PM

NOTES:
1. EXISTING CONTOURS SHOWN WERE SURVEYED BY MARK ROBERTSON, PLS ON OCTOBER 4, 2013 & FEBRUARY 13, 2015.
2. PROPOSED CONTOURS REPRESENT TOP OF SOIL.
3. PROPOSED SEPARATION BERM FILL SHALL BE BENCHED INTO THE EXISTING SURFACE IN ACCORDANCE WITH SPECIFICATIONS.
4. QUANTITIES ON THIS SHEET WERE ESTIMATED BY COMPARING THE FEBRUARY 13, 2015 SURFACE TO THE PROPOSED CONTOURS AS SHOWN ON THIS SHEET. ACTUAL QUANTITIES MAY VARY.
5. THE SEPARATION BERM NORTH TIE IN LOCATION & WEST SLOPE TOE MAY DIFFER FROM WHAT IS SHOWN ON THIS PLAN DUE TO ONGOING CCR REMOVAL IN THIS AREA AS OF 10-1-15.

LEGEND
EXISTING CONTOUR — 720 —
PROPOSED FINAL GRADE CONTOUR — 710 —

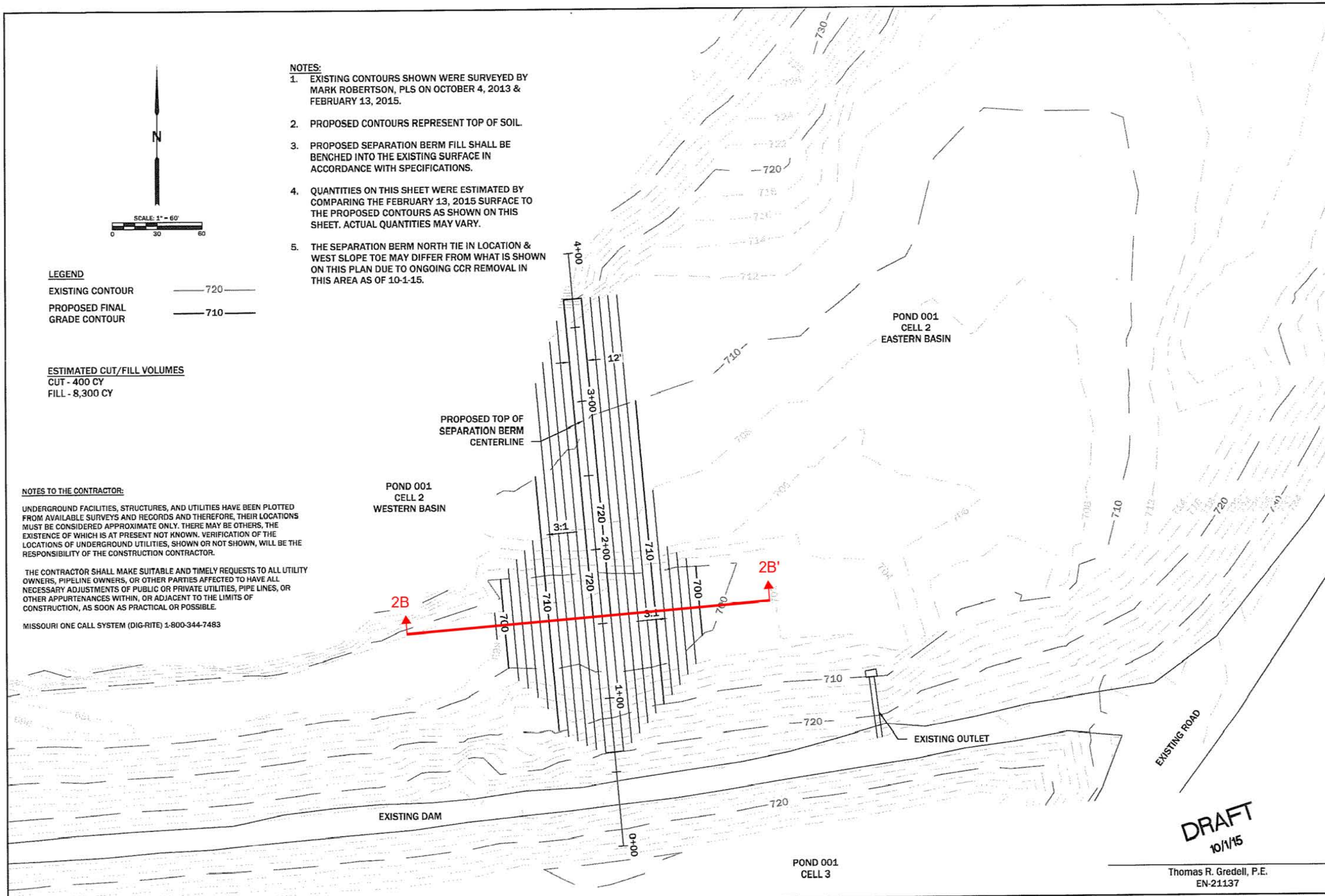
ESTIMATED CUT/FILL VOLUMES
CUT - 400 CY
FILL - 8,300 CY

NOTES TO THE CONTRACTOR:

UNDERGROUND FACILITIES, STRUCTURES, AND UTILITIES HAVE BEEN PLOTTED FROM AVAILABLE SURVEYS AND RECORDS AND THEREFORE, THEIR LOCATIONS MUST BE CONSIDERED APPROXIMATE ONLY. THERE MAY BE OTHERS, THE EXISTENCE OF WHICH IS AT PRESENT NOT KNOWN. VERIFICATION OF THE LOCATIONS OF UNDERGROUND UTILITIES, SHOWN OR NOT SHOWN, WILL BE THE RESPONSIBILITY OF THE CONSTRUCTION CONTRACTOR.

THE CONTRACTOR SHALL MAKE SUITABLE AND TIMELY REQUESTS TO ALL UTILITY OWNERS, PIPELINE OWNERS, OR OTHER PARTIES AFFECTED TO HAVE ALL NECESSARY ADJUSTMENTS OF PUBLIC OR PRIVATE UTILITIES, PIPE LINES, OR OTHER APPURTENANCES WITHIN, OR ADJACENT TO THE LIMITS OF CONSTRUCTION, AS SOON AS PRACTICAL OR POSSIBLE.

MISSOURI ONE CALL SYSTEM (DIG-RITE) 1-800-344-7483



Thomas R. Gredell, P.E.
EN-21137

#	DATE	REVISION DESCRIPTION	BY

CELL 2 SEPARATION BERM GRADING PLAN
POND 001 - CELL 2 SEPARATION BERM

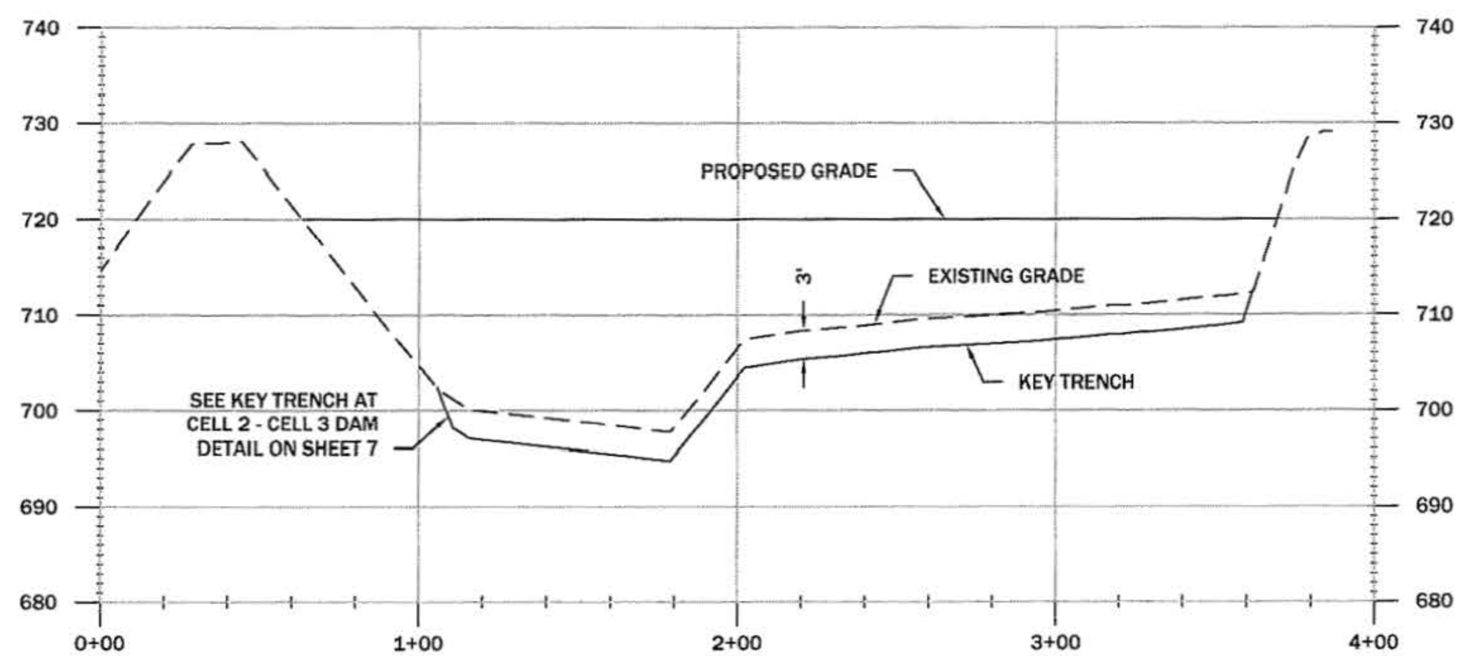
SHEET # 3 OF 7 | FILE NAME SEPARATION BERM | PROJECT NAME AECI/THEC | SCALE AS NOTED | DATE 10/2015

THOMAS HILL ENERGY CENTER

SURVEYED	DESIGNED	DRAWN	CHECKED	APPROVED
NA	MW	AJK	AR	TRG

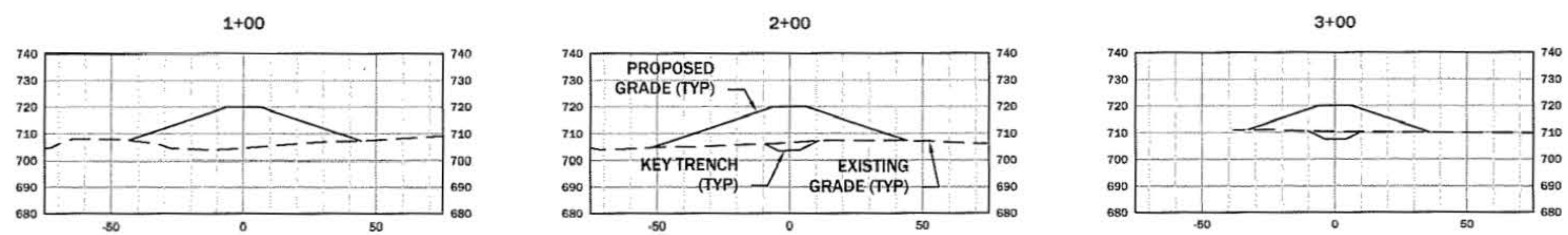
GREDELL Engineering Resources, Inc.
ENVIRONMENTAL ENGINEERING LAND - AIR - WATER
1505 East High Street
Jefferson City, Missouri
Telephone: (573) 659-9078
Facsimile: (573) 659-9079
MO CORP. ENGINEERING LICENSE NO. E-2001001669-0

M:\Share\CADD Files\AECI\THEC\AECI- THEC ASH VOLUME\POND 001 CELL 2 - SEPARATION BERM\SEPARATION BERM.DWG, CELL 2 SEPARATION BERM GRADING PLAN, 10/1/2015 3:35:21 PM



SEPARATION BERM
SCALE:
HORIZONTAL - 1"=60'
VERTICAL - 1"=20'

NOTE:
1. SEE SHEET 7 FOR SEPARATION BERM AND KEY TRENCH DETAILS.



CROSS SECTIONS
SCALE:
HORIZONTAL - 1"=60'
VERTICAL - 1"=60'

DRAFT
10/1/15

Thomas R. Gredell, P.E.
EN-21137

#	DATE	REVISION DESCRIPTION	BY

CELL 2 SEPARATION BERM
PROFILE AND CROSS SECTIONS
POND 001 - CELL 2
SEPARATION BERM

SURVEYED	DESIGNED	DRAWN	CHECKED	APPROVED	DATE	SCALE	PROJECT NAME	FILE NAME	SHEET #
NA	MW	AJK	AR	TRG	10/2015	AS NOTED	AECI-THEC	SEPARATION BERM	4 OF 7

THOMAS HILL
ENERGY CENTER

GREDELL Engineering Resources, Inc.
ENVIRONMENTAL ENGINEERING LAND - AIR - WATER
1505 East High Street Telephone: (573) 659-9078
Jefferson City, Missouri Facsimile: (573) 659-9079
MO CORP. ENGINEERING LICENSE NO. E2001001690

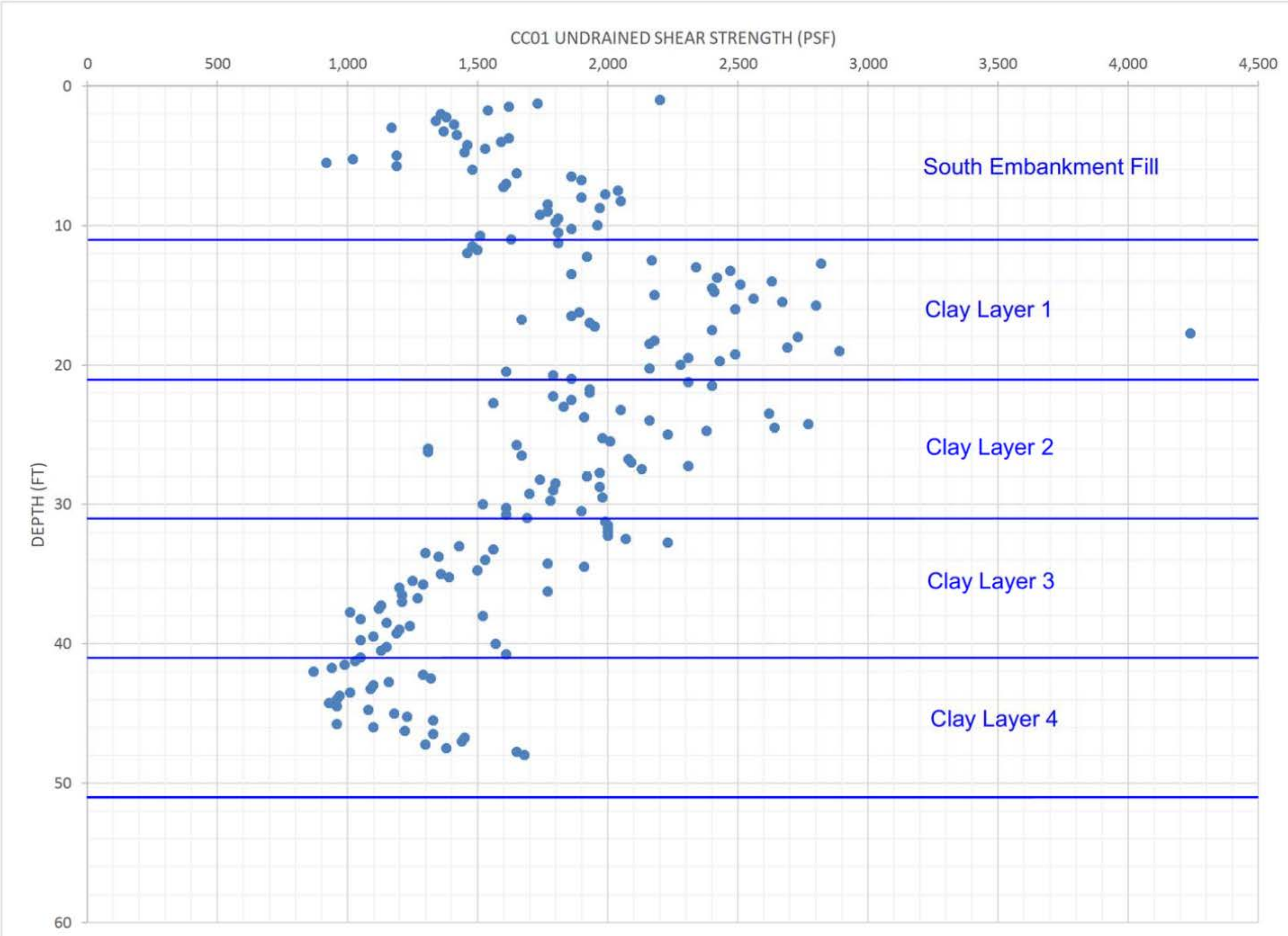
Design Soil Properties

SOIL PROPERTY CHARACTERIZATION - THOMAS HILL ENERGY CENTER CELL 002 WEST

<table>
<tr><th rowspan="4">Material[2]</th><th colspan="4">Total Unit Weight, γ_T</th><th colspan="10">Undrained Shear Strength, S_u</th><th colspan="10">Drained Shear Strength</th></tr>
<tr><th>CPT</th><th>Laboratory</th><th rowspan="2">Historic Design[1]</th><th rowspan="2">Current Design</th><th colspan="2">SPT</th><th colspan="2">CPT</th><th>CIU Trx</th><th colspan="3" rowspan="2">Historic Design[1]</th><th rowspan="2">Current Design</th><th colspan="2">SPT</th><th colspan="2">CPT</th><th colspan="2" rowspan="2">Laboratory CIU Trx</th><th colspan="2" rowspan="2">Historic Design[1]</th><th colspan="2" rowspan="2">Current Design</th></tr>
<tr><th>avg</th><th>Site-Wide Average</th><th>avg</th><th>avg - 1σ</th><th>avg</th><th>avg - 1σ</th><th>avg</th><th>avg</th><th>avg - 1σ</th><th>avg</th><th>avg - 1σ</th></tr>
<tr><th>γ_T</th><th>γ_T</th><th>γ_T</th><th>γ_T</th><th>S_u</th><th>S_u</th><th>S_u</th><th>S_u</th><th>S_u</th><th>c</th><th>ϕ</th><th>S_u</th><th>S_u</th><th>ϕ'</th><th></th><th>ϕ'</th><th>ϕ'</th><th>c'</th><th>ϕ'</th><th>c'</th><th>ϕ'</th><th>c'</th><th>ϕ'</th></tr>
<tr><td>South Embankment Fill</td><td>--</td><td>--</td><td>120 pcf</td><td>125 pcf</td><td>1,895 psf</td><td>1,638 psf</td><td>1,615 psf</td><td>1,320 psf</td><td>--</td><td>--</td><td>--</td><td>--</td><td>1,300 psf</td><td>--</td><td>--</td><td>--</td><td>--</td><td>--</td><td>--</td><td>100 psf</td><td>28°</td><td>100 psf</td><td>28°</td></tr>
<tr><td>Seperation Berm</td><td>--</td><td>--</td><td>--</td><td>125 pcf</td><td>--</td><td>--</td><td>--</td><td>--</td><td>--</td><td>--</td><td>--</td><td>--</td><td>1,300 psf</td><td>--</td><td>--</td><td>--</td><td>--</td><td>--</td><td>--</td><td>--</td><td>--</td><td>100 psf</td><td>28°</td></tr>
<tr><td>Native Clay 1</td><td>--</td><td>123 pcf</td><td rowspan="4">120 pcf</td><td>120 pcf</td><td>2,394 psf</td><td>2,394 psf</td><td>2,245 psf</td><td>1,734 psf</td><td>--</td><td>--</td><td>--</td><td>--</td><td>1,700 psf</td><td>--</td><td>--</td><td>--</td><td>--</td><td>0 psf</td><td>26°</td><td rowspan="4">50 psf</td><td rowspan="4">27°</td><td>50 psf</td><td>27°</td></tr>
<tr><td>Native Clay 2</td><td>--</td><td>--</td><td>120 pcf</td><td>1,596 psf</td><td>1,314 psf</td><td>1,954 psf</td><td>1,619 psf</td><td>--</td><td>--</td><td>--</td><td>--</td><td>1,600 psf</td><td>--</td><td>--</td><td>--</td><td>--</td><td>--</td><td>--</td><td>50 psf</td><td>27°</td></tr>
<tr><td>Native Clay 3</td><td>--</td><td>--</td><td>120 pcf</td><td>1,097 psf</td><td>674 psf</td><td>1,452 psf</td><td>1,113 psf</td><td>--</td><td>--</td><td>--</td><td>--</td><td>1,100 psf</td><td>--</td><td>--</td><td>--</td><td>--</td><td>--</td><td>--</td><td>50 psf</td><td>27°</td></tr>
<tr><td>Native Clay 4</td><td>--</td><td>--</td><td>120 pcf</td><td>1,397 psf</td><td>1,114 psf</td><td>1,177 psf</td><td>960 psf</td><td>--</td><td>--</td><td>--</td><td>--</td><td>950 psf</td><td>--</td><td>--</td><td>--</td><td>--</td><td>--</td><td>--</td><td>50 psf</td><td>27°</td></tr>
</table>

Notes:

1. Based on historic analyses performed by Geotechnology, Inc.
2. In cases where historic design properties, SPT/CPT correlations, and laboratory test data do not exist, the current design properties for these materials have been conservatively estimated using typical published values and Haley & Aldrich's experience with similar materials.



CC01 UNDRAINED SHEAR STRENGTH (PSF)
0
500
1,000
1,500
2,000
2,500
3,000
3,500
4,000
4,500
DEPTH (FT)
0
10
20
30
40
50
60
South Embankment Fill
Clay Layer 1
Clay Layer 2
Clay Layer 3
Clay Layer 4

Seismic Documents

HALEY ALDRICH	CALCULATIONS	File No.	128064-006
		Sheet	1 of 2
Client	Associated Electric Cooperative, Inc.	Date	5-Apr-18
Project	Thomas Hill Energy Center - Cell 002	Computed by	RJW
Subject	Pseudostatic Coefficient	Checked by	DAS

Objective:

-Determination of the pseudostatic coefficient for stability analyses of the Cell 002 embankment.

Step 1

Estimate peak horizontal bedrock acceleration, A_{max}, for 2% in 50 year using 2014 USGS text file by computing the average of the four corners grid using latitude and longitude of the site.

https://earthquake.usgs.gov/static/lfs/nshm/conterminous/2014/data/

Site Coordinates:

Latitude	39.545
Longitude	-92.637

PGA for 2% in 50 yr event = 0.057 g

Step 2

Classify site stiffness.

http://earthquake.usgs.gov/designmaps/us/application.php

Use USGS design application tool

2010 ASCE 7 (w/ march 2013)

Site Class = D

Step 3

Using the site latitude and longitude, determined site class, and the USGS design application tool 2010 ASCE 7 (with March 2013) estimate peak free field (ground surface) acceleration using the empirical charts. The peak free field acceleration corresponds to the bedrock acceleration at the base of the embankment, which is propogated upward through the existing soils at the site.

http://earthquake.usgs.gov/designmaps/us/application.php

Using Table 11.8-1 from the ASCE-10 Summary Report

USGS Site Coefficient, F_{PGA} = 1.6

Peak Free Field Acceleration* = PGA x F_{PGA} = 0.091 g

Step 4

Estimate peak acceleration at the top of the embankment using Figure 2 (Singh and Sun,1995).

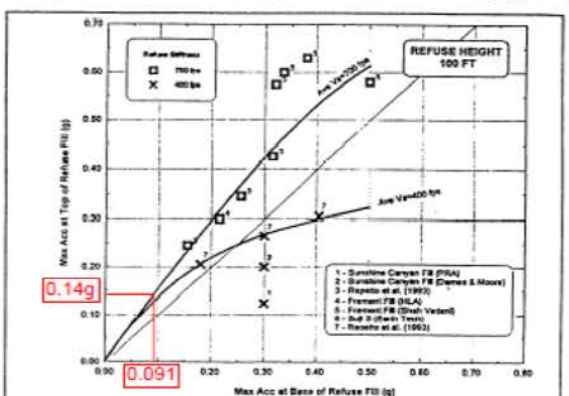


Figure 2 Approximate Relationship Between Max Accelerations at Base and Crest, 100-ft Refuse

(Singh and Sun, 1995)

Using Figure 2

peak acceleration at top of embankment = 0.14

Note: The peak acceleration at the top of the embankment has been conservatively estimated using Figure 2 from the Singh and Sun (1995) approach, which was developed for refuse.

HALEY ALDRICH	CALCULATIONS	File No.	128064-006
		Sheet	2 of 2
Client	Associated Electric Cooperative, Inc.	Date	5-Apr-18
Project	Thomas Hill Energy Center - Cell 002	Computed by	RJW
Subject	Pseudostatic Coefficient	Checked by	

Step 5

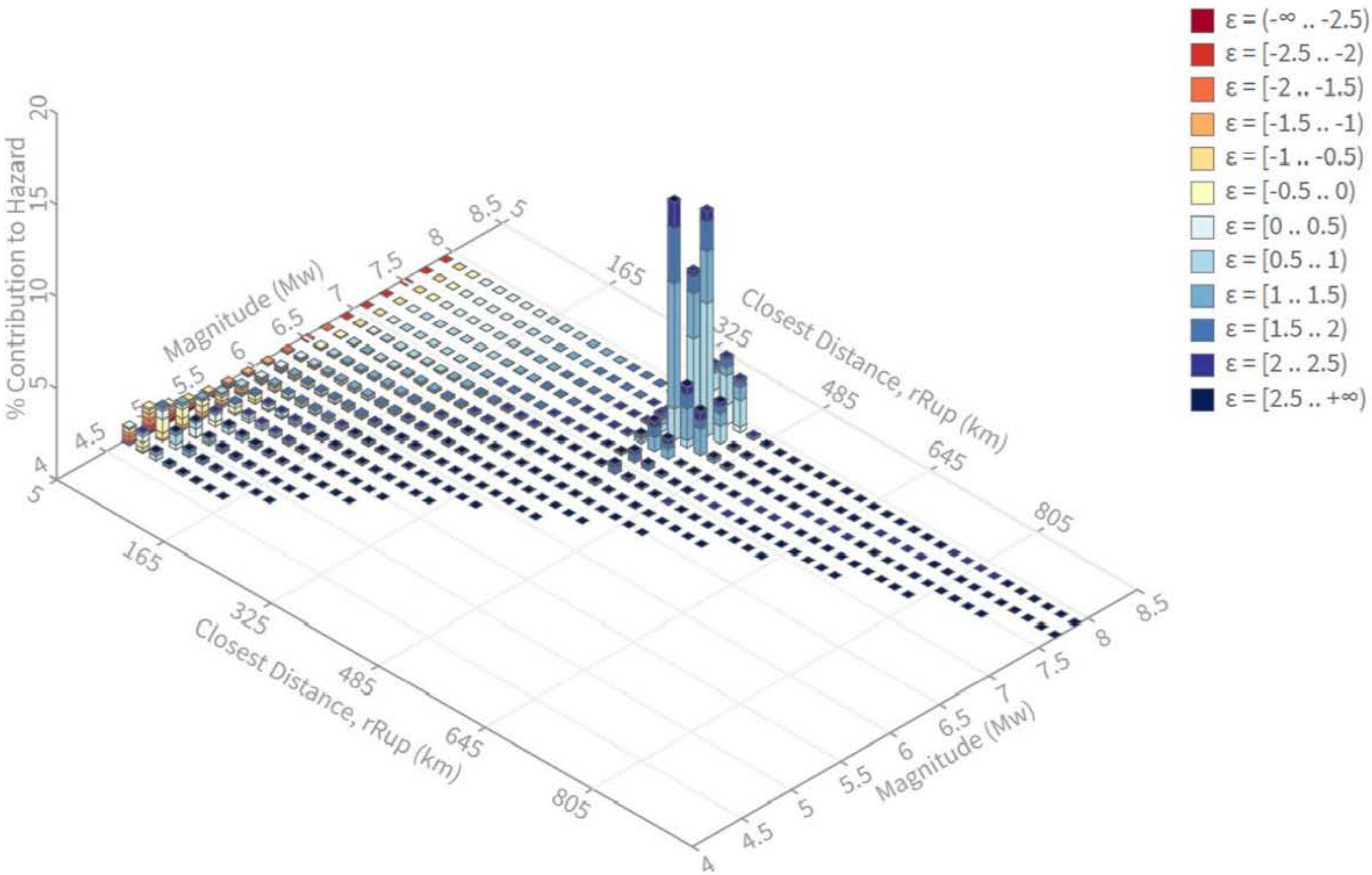
Calculate pseudo-static coefficeint using approach developed by Hynes-Griffin and Franklin (1984).

Pseudo-Static Coefficient = Peak Acceleration at Top of Embankment x 0.5 = 0.07 g

^ Deaggregation

Component

Total



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr^{-1}
PGA ground motion: 0.05556897 g

Recovered targets

Return period: 2484.1818 yrs
Exceedance rate: 0.00040254703 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 2.3 %

Mean (for all sources)

r: 291.98 km
m: 6.89
ε₀: 0.78 σ

Mode (largest r-m bin)

r: 407.22 km
m: 7.52
ε₀: 1.36 σ
Contribution: 12.95 %

Mode (largest ε₀ bin)

r: 424.18 km
m: 7.78
ε₀: 0.73 σ
Contribution: 7.23 %

Discretization

r: min = 0.0, max = 1000.0, Δ = 20.0 km
m: min = 4.4, max = 9.4, Δ = 0.2
ε: min = -3.0, max = 3.0, Δ = 0.5 σ

Epsilon keys

ε0: [-∞ .. -2.5)
ε1: [-2.5 .. -2.0)
ε2: [-2.0 .. -1.5)
ε3: [-1.5 .. -1.0)
ε4: [-1.0 .. -0.5)
ε5: [-0.5 .. 0.0)
ε6: [0.0 .. 0.5)
ε7: [0.5 .. 1.0)
ε8: [1.0 .. 1.5)
ε9: [1.5 .. 2.0)
ε10: [2.0 .. 2.5)
ε11: [2.5 .. +∞]

Deaggregation Contributors

Source Set ↳ Source	Type	r	m	ε_0	lon	lat	az	%
SSCn New Madrid	Cluster							35.72
NMFS RLME 1		419.97	7.68	1.18	89.288°W	36.995°N	133.07	10.65
NMFS RLME 5		427.90	7.67	1.24	89.288°W	36.995°N	133.07	6.34
NMFS RLME 2		413.30	7.68	1.14	89.288°W	36.995°N	133.07	4.95
NMFS RLME 4		419.04	7.68	1.18	89.020°W	37.270°N	127.61	4.61
NMFS RLME 6		419.70	7.67	1.18	89.288°W	36.995°N	133.07	2.98
NMFS RLME 7		426.74	7.67	1.23	89.020°W	37.270°N	127.61	2.75
NMFS RLME 3		412.51	7.68	1.14	89.020°W	37.270°N	127.61	2.14
NMFS RLME 8		418.74	7.67	1.18	89.020°W	37.270°N	127.61	1.29
USGS Fixed Smoothing Zone 1 (opt)	Grid							13.35
SSCn Fixed Smoothing Zone 1 (opt)	Grid							13.01
USGS New Madrid 500-year	Cluster							9.52
NMSZ: Center Model		421.66	7.66	1.21	89.070°W	37.165°N	129.26	6.67
NMSZ: Mid-West Model		411.78	7.65	1.17	89.193°W	37.218°N	129.67	1.09
USGS Adaptive Smoothing Zone 1 (opt)	Grid							6.62
SSCn Adaptive Smoothing Zone 1 (opt)	Grid							6.43
USGS New Madrid 750-year	Cluster							6.35
NMSZ: Center Model		421.66	7.66	1.21	89.070°W	37.165°N	129.26	4.45
USGS New Madrid 500-year	Fault							2.57
New Madrid central		408.44	7.70	1.06	89.070°W	37.165°N	129.26	1.79
USGS New Madrid 1500-year	Cluster							2.41
NMSZ: Center Model		426.53	7.71	1.16	89.583°W	36.687°N	138.98	1.69

USGS Design Maps Detailed Report

ASCE 7-10 Standard (39.544°N, 92.637°W)

Site Class D – "Stiff Soil", Risk Category IV (e.g. essential facilities)

Section 11.4.1 — Mapped Acceleration Parameters

Note: Ground motion values provided below are for the direction of maximum horizontal spectral response acceleration. They have been converted from corresponding geometric mean ground motions computed by the USGS by applying factors of 1.1 (to obtain S_S) and 1.3 (to obtain S_1). Maps in the 2010 ASCE-7 Standard are provided for Site Class B. Adjustments for other Site Classes are made, as needed, in Section 11.4.3.

From Figure 22-1 [1] $S_S = 0.124$ g

From Figure 22-2 [2] $S_1 = 0.077$ g

Section 11.4.2 — Site Class

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class D, based on the site soil properties in accordance with Chapter 20.

Table 20.3–1 Site Classification

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard Rock	>5,000 ft/s	N/A	N/A
B. Rock	2,500 to 5,000 ft/s	N/A	N/A
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff Soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the characteristics: • Plasticity index $PI > 20$, • Moisture content $w \geq 40\%$, and • Undrained shear strength $\bar{s}_u < 500$ psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²

Section 11.8.3 — Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F

From Figure 22-7 [4] PGA = $\boxed{0.057}$

Equation (11.8–1): $PGA_M = F_{PGA}PGA = 1.600 \times \boxed{0.057} = \boxed{0.091}$ g

Table 11.8–1: Site Coefficient F_{PGA}

Site Class	Mapped MCE Geometric Mean Peak Ground Acceleration, PGA				
	PGA ≤ 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA ≥ 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	$\boxed{1.6}$	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of PGA

For Site Class = D and PGA = 0.059 g, F_{PGA} = 1.600

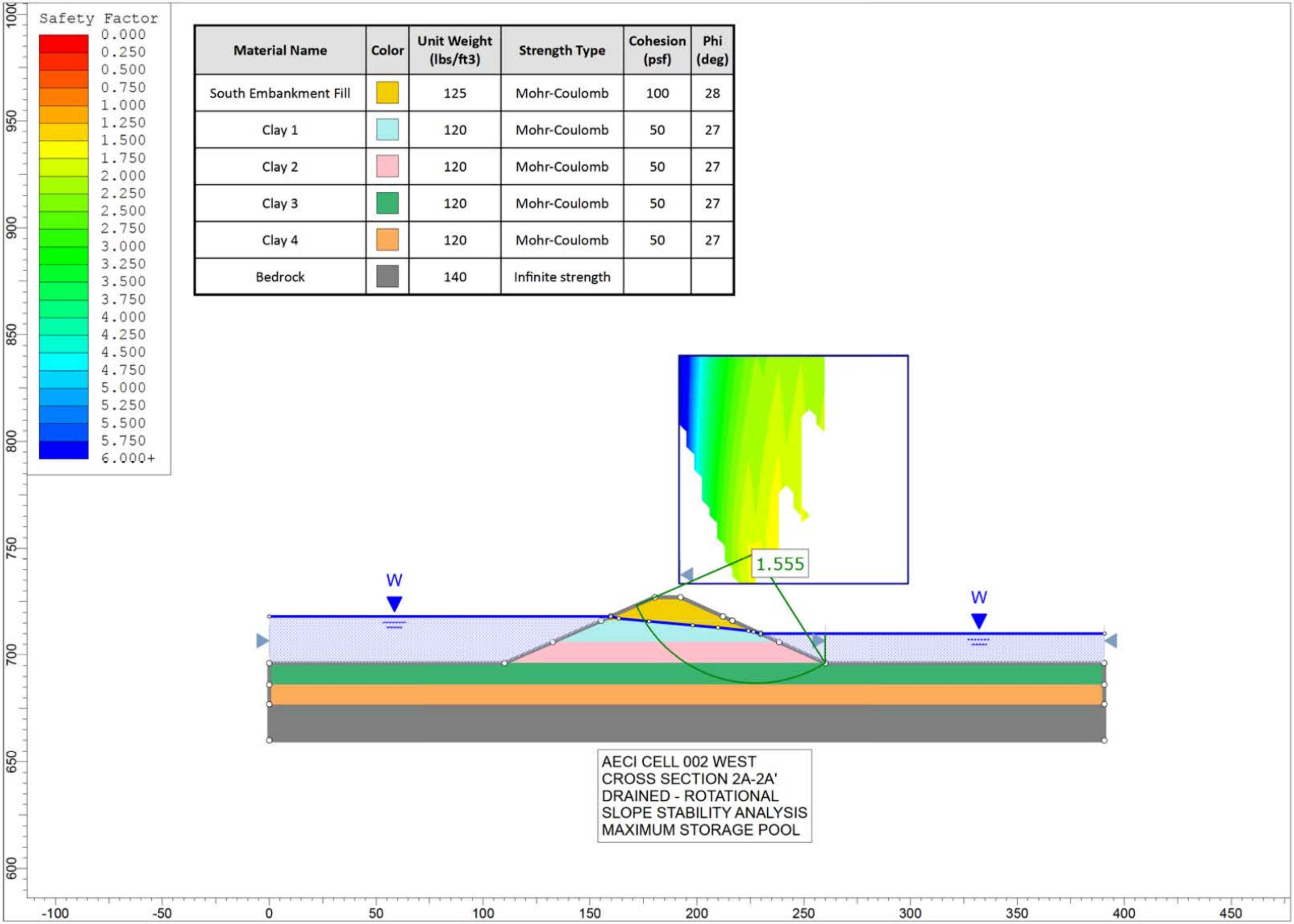
Section 21.2.1.1 — Method 1 (from Chapter 21 – Site-Specific Ground Motion Procedures for Seismic Design)

From Figure 22-17 [5] C_{RS} = 0.866

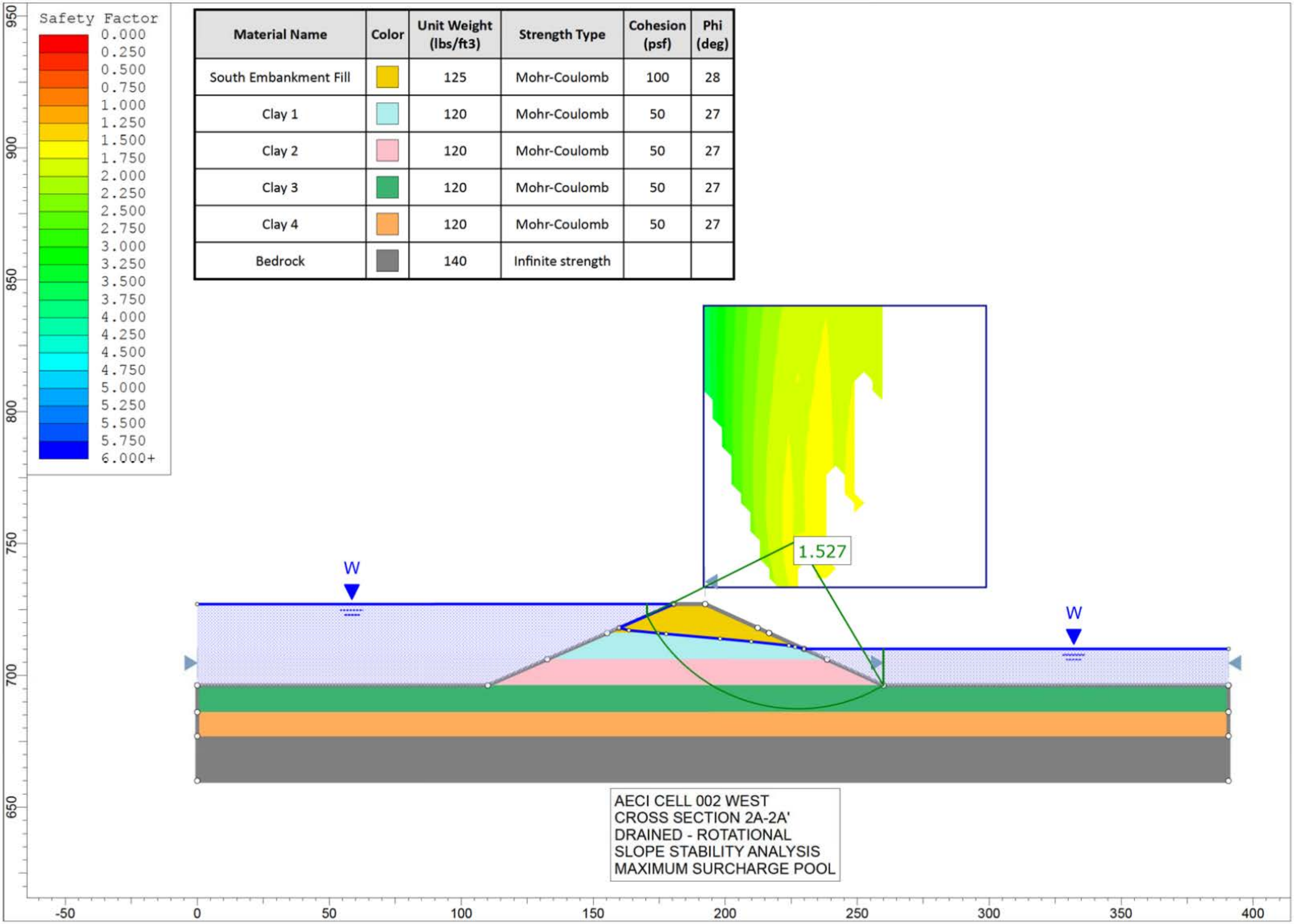
From Figure 22-18 [6] C_{R1} = 0.838

Slope Stability

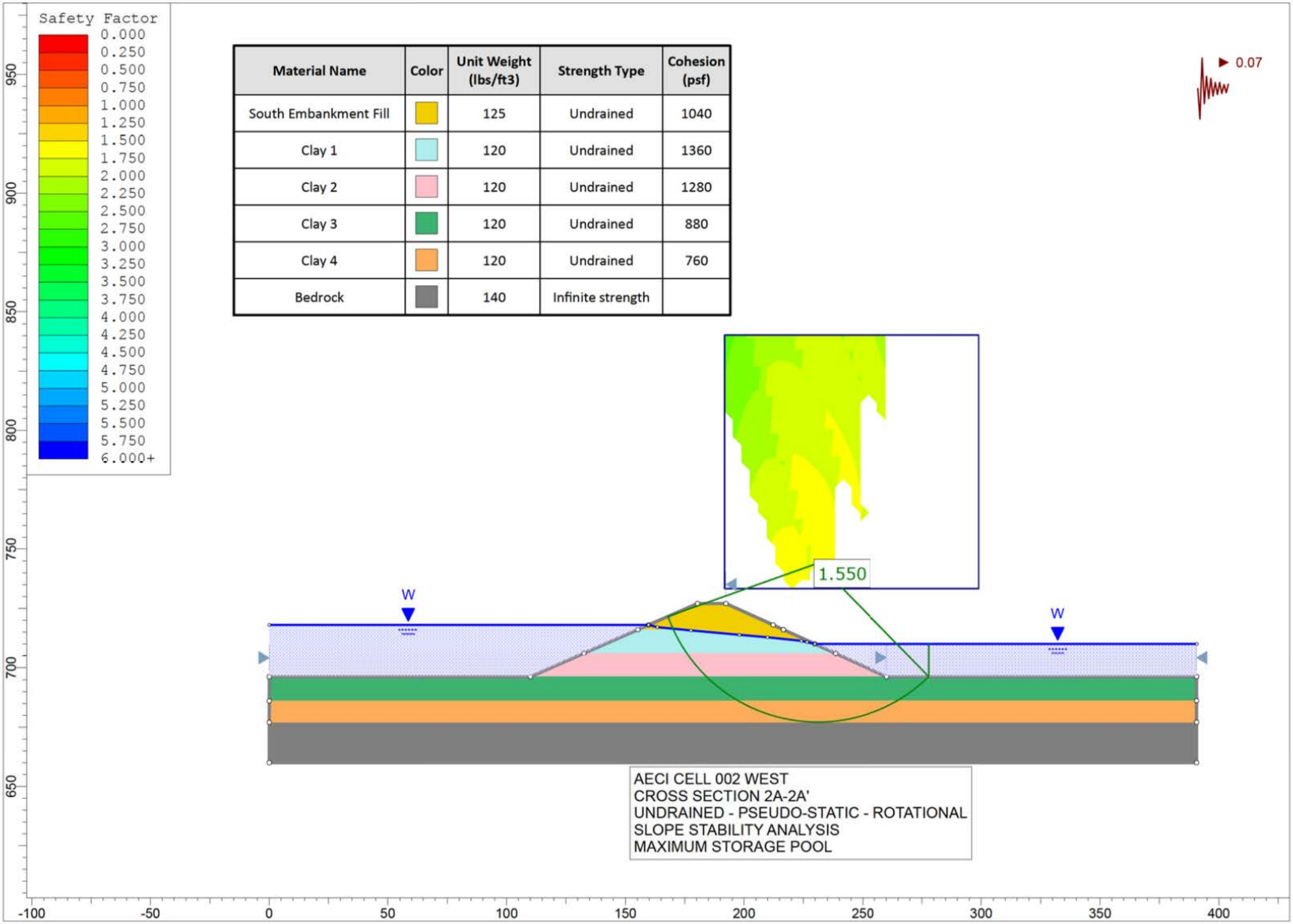
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
South Embankment Fill		125	Mohr-Coulomb	100	28
Clay 1		120	Mohr-Coulomb	50	27
Clay 2		120	Mohr-Coulomb	50	27
Clay 3		120	Mohr-Coulomb	50	27
Clay 4		120	Mohr-Coulomb	50	27
Bedrock		140	Infinite strength		



AECI CELL 002 WEST
CROSS SECTION 2A-2A'
DRAINED - ROTATIONAL
SLOPE STABILITY ANALYSIS
MAXIMUM STORAGE POOL

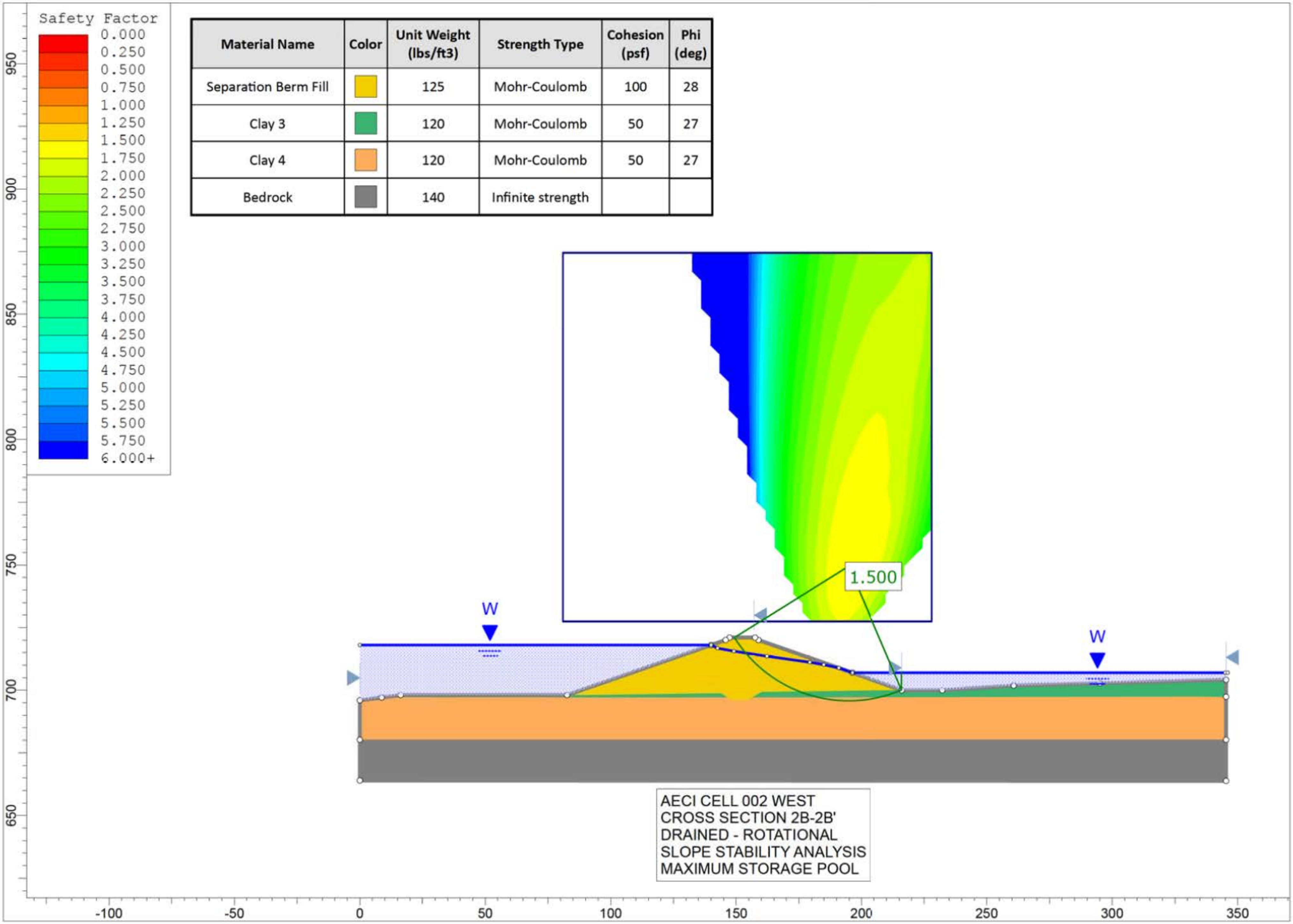


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
South Embankment Fill		125	Mohr-Coulomb	100	28
Clay 1		120	Mohr-Coulomb	50	27
Clay 2		120	Mohr-Coulomb	50	27
Clay 3		120	Mohr-Coulomb	50	27
Clay 4		120	Mohr-Coulomb	50	27
Bedrock		140	Infinite strength		

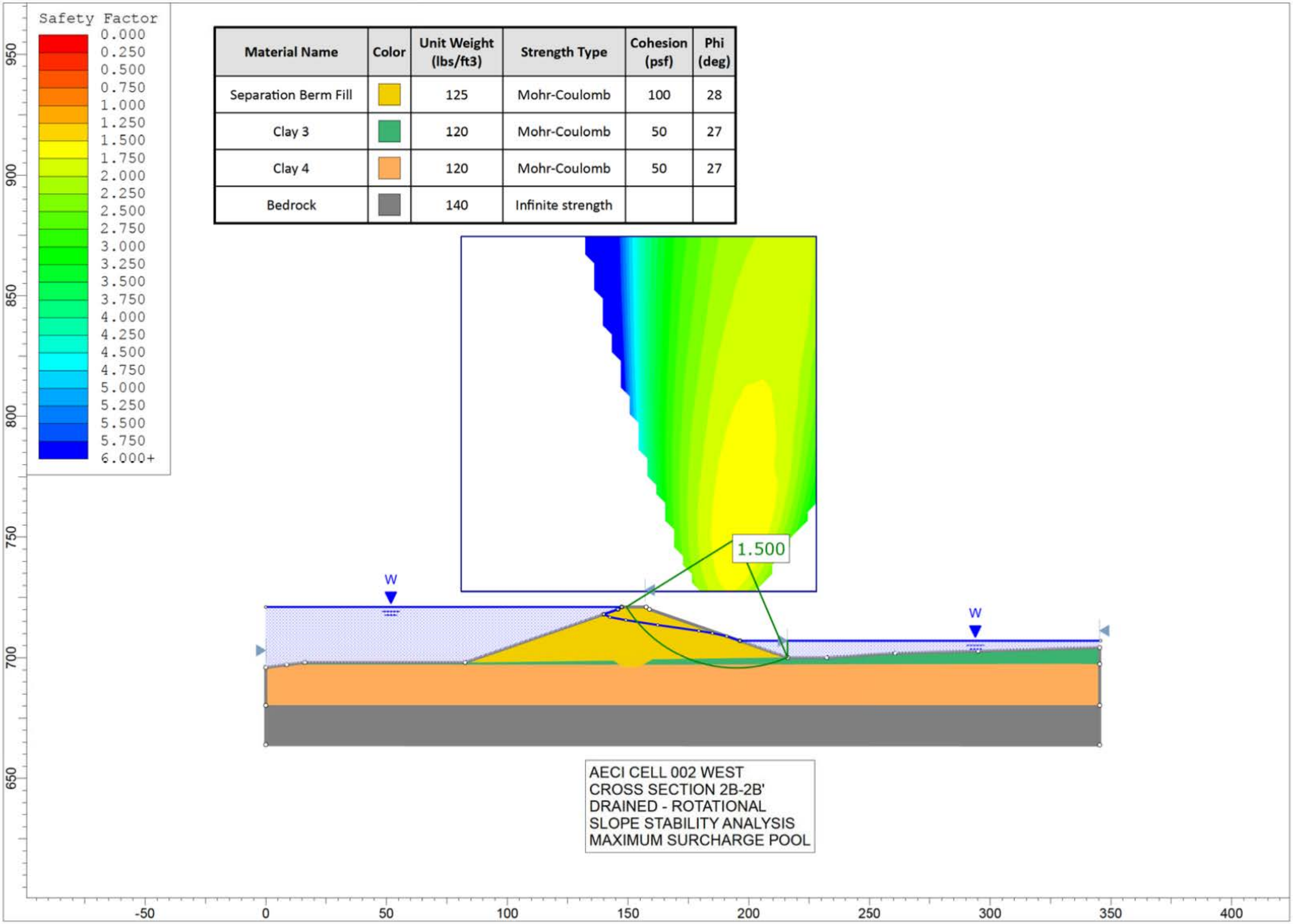


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)
South Embankment Fill		125	Undrained	1040
Clay 1		120	Undrained	1360
Clay 2		120	Undrained	1280
Clay 3		120	Undrained	880
Clay 4		120	Undrained	760
Bedrock		140	Infinite strength	

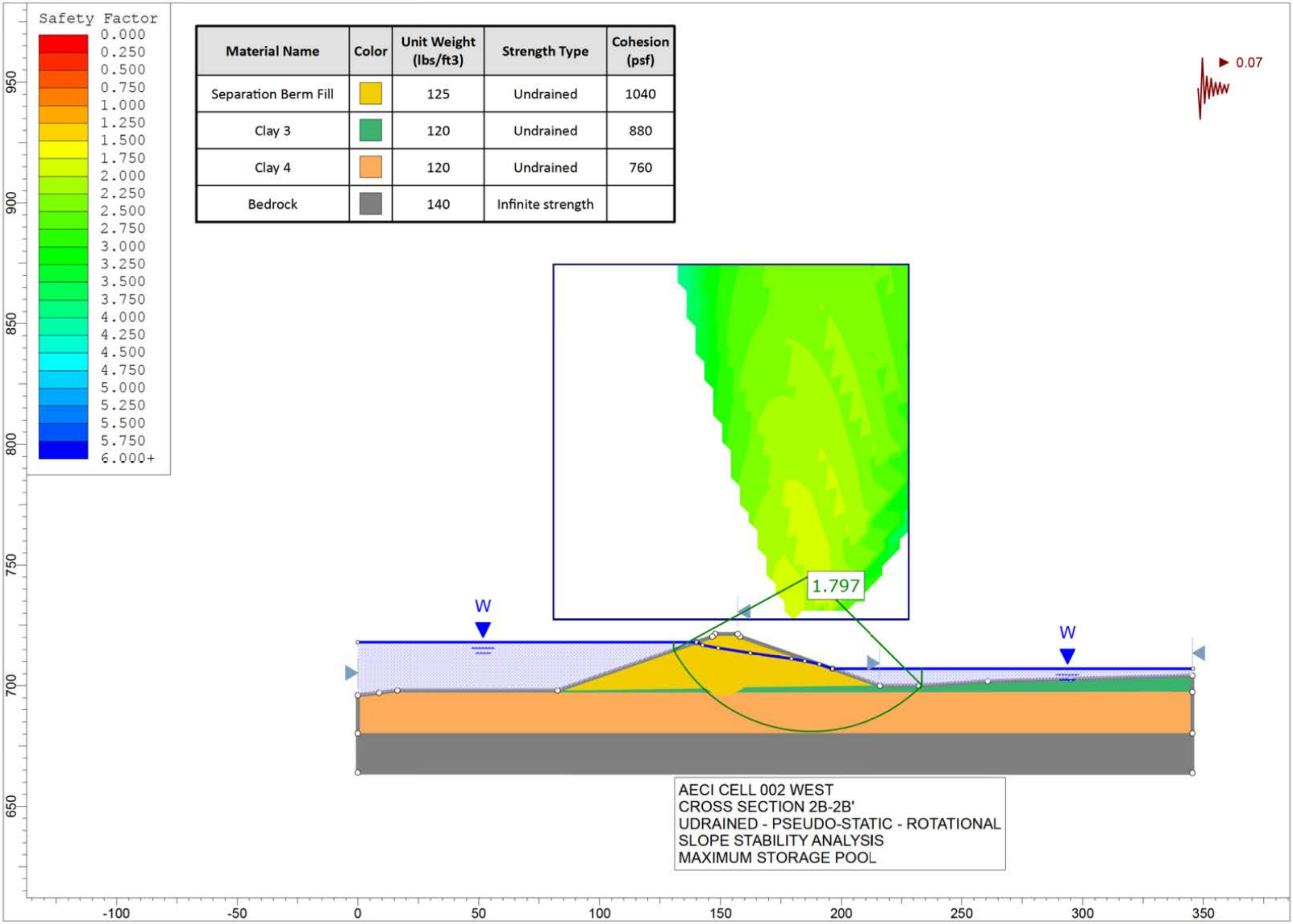
AECI CELL 002 WEST
CROSS SECTION 2A-2A'
UNDRAINED - PSEUDO-STATIC - ROTATIONAL
SLOPE STABILITY ANALYSIS
MAXIMUM STORAGE POOL



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Separation Berm Fill		125	Mohr-Coulomb	100	28
Clay 3		120	Mohr-Coulomb	50	27
Clay 4		120	Mohr-Coulomb	50	27
Bedrock		140	Infinite strength		



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Separation Berm Fill		125	Mohr-Coulomb	100	28
Clay 3		120	Mohr-Coulomb	50	27
Clay 4		120	Mohr-Coulomb	50	27
Bedrock		140	Infinite strength		



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)
Separation Berm Fill		125	Undrained	1040
Clay 3		120	Undrained	880
Clay 4		120	Undrained	760
Bedrock		140	Infinite strength	