REPORT
2018120907

SAFETY FACTOR ASSESSMENT FOR SIKESTON POWER STATION FLY ASH POND SIKESTON, MISSOURI

Prepared for

GREDELL ENGINEERING RESOURCES, INC.
Jefferson City, Missouri

and

SIKESTON BOARD OF MUNICIPAL UTILITIES
Sikeston, Missouri

Prepared by

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CONSULTING ENGINEERS

STATE OF MISSOURI
JEFFREY LYNN FOUSE
NUMBER E-21043
REGISTERED PROFESSIONAL ENGINEER

3/12/2018

March 12, 2018

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March 12, 2018

GREDELL Engineering Resources, Inc.
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Attention: Mr. Mikel C. Carlson, R.G.
Senior Geologist

RE: Report of Safety Factor Assessment for
Sikeston BMU / Sikeston Power Station Fly Ash Pond
Sikeston, Missouri

Introduction

This report presents our findings from our safety factor assessment for the fly ash pond at the Sikeston Board of Municipal Utilities (BMU) Sikeston Power Station, which is located on West Wakefield Avenue in Sikeston, Missouri. Our analyses were done to satisfy the requirements of 40 CFR Part 257.73(e) "Periodic Safety Factor Assessments" for an existing Coal Combustion Residuals (CCR) surface impoundment that was published on April 17, 2015 (U.S. EPA "CCR Rule"). This is the initial safety factor assessment for the fly ash pond. This work was done in general accordance with our proposal to GREDELL Engineering Resources, Inc. (GREDELL), dated January 2, 2018.

The safety factor assessment requires the calculation of the factor of safety against slope failure of the dike of the CCR unit at the critical cross-section for four loading conditions:

1. Static load condition under the long-term, maximum storage pool;
2. Static load condition under the maximum surcharge;
3. Seismic load condition; and
4. For dikes constructed of soils that have susceptibility to liquefaction, the static load condition with reduced soil shear strengths to take into account liquefaction.

Previous Investigations

We reviewed reports of previous investigations and safety factor assessments by others that were furnished to us by GREDELL and Sikeston BMU. These were:

- Burns & McDonnell (1977). *Report of Preliminary Subsurface Investigation for Board of Municipal Utilities, Sikeston, Missouri,* 76-076-1. (This is the original investigation for construction of the power station.)

- O'Brien & Gere Engineers (2010). *Dam Safety Assessment of CCW Impoundments, Sikeston Power Station,* a report to the U.S. EPA.
- Geotechnology (2011). *Global Stability Evaluation, Fly Ash and Bottom Ash Ponds, Sikeston Power Station, Sikeston, Missouri*, a report for Sikeston BMU. (This evaluation was done in accordance with the Missouri Department of Natural Resources (MDNR) Dam Safety Program, which is not the same as the 2015 EPA CCR Rule.)
- Haley & Aldrich (2016). *Detailed Initial Safety Factor Assessment, Sikeston Power Station, Bottom Ash Pond, Sikeston, Missouri*, a report for Sikeston BMU.

An investigation of the shear-wave velocity profile at the Sikeston Power Station was done by the University of Memphis as part of the safety factor assessment by Haley & Aldrich (Appendix C in the above report). The findings from the measurement of the shear wave velocity profile by the University of Memphis were used in this safety factor assessment with permission from Sikeston BMU.

General Description of Fly Ash Pond

The plan of the fly ash pond is shown in Figure 1. The fly ash pond is a combination of incised and diked surface impoundment. The dike is about 4800 feet long and encloses about 30 acres. The southern dike separates the fly ash pond from the active bottom ash pond. The top of the dike is at el. 322 to el. 322.6. Based upon the 2016 topographic survey by Surdex Corp., the height of the dikes is about 11 to 12 feet. The report by O'Brien & Gere states that the pond is incised a depth of 4 feet below the outside toe of the dike. The top of the CCR in the fly ash pond is a maximum of el. 319.6 to 321.6. Two areas of free water on the west side of the fly ash pond were at el. 315.7 and el. 316.5 when photo surveyed by Surdex. The topographic survey showed that the exterior slope of the dike is about 2.1(H)-to-1(V). The interior slope of the dike is reportedly about 2(H)-to-1(V). The 2011 borings by Geotechnology confirmed that the dike is composed of compacted silty sands and fine sands with some layers of clayey sand. An outlet structure at the northwest corner of the fly ash pond controls the height of the water. The weir on the outlet structure is at el. 318.

Field Investigation

Borings had been made by Geotechnology in 2011 in the dike on the west and northwest corner sides of the fly ash pond. However, there were no data on the subsurface soil strata on the north and east sides. We planned four Cone Penetrometer Test (CPT) soundings on the north, northeast and east sides of the fly ash pond, approximately 25 to 30 feet beyond the railroad track that runs along the outside toe of the dike. The purpose of the CPT soundings was to obtain data on the natural soil strata beyond the influence of the fly ash pond and dike. The soil strata beneath the pond and dike have been consolidated or densified by the weight of the CCR and dike, and therefore would have greater shear strength properties. The shear strength properties of the unconsolidated soil strata beyond the dike would be critical to the stability assessment.

The CPT soundings were performed by Bulldog Drilling, Inc. using an AMS-probe rig, under a subcontract with Reitz & Jens. Reitz & Jens owns and operates the CPT equipment. Reitz & Jens' geologist, the drill crew and rig mobilized to the site on February 14, 2018. After obtaining the excavation permit from the Sikeston Power Station, we began CPT-2. The first sounding met refusal at 15 feet. So, we moved about 10 feet further from the railroad track and completed CPT-2A to 51.6 feet.

We moved to CPT-3, and began that sounding on February 15. We lost the signal from the CPT probe at about 8 feet. We extracted the probe and re-set a new CPT probe in the original hole. We lost the signal from the new CPT probe at about 40.2 feet. We found that a circuit board in the CPT equipment was bad. We demobilized on February 15. We processed the data from CPT-2A and CPT-3. The data were very similar, and were consistent with the findings from previous borings by others. Therefore, we judged that it was not necessary to re-mobilize to do soundings CPT-1 and CPT-4 to complete our safety factor assessment.

The cone penetrometer is a 1.5-inch diameter, 100-MPa capacity, electronic piezocone (CPTu), which continuously records tip pressure, sleeve friction and porewater pressure as it is hydraulically pushed into the ground. The CPT soundings were done in general accordance with ASTM D5778. The holes were backfilled with Bentonite crumbles. The CPT soundings were performed under the direction of a Reitz & Jens' geologist, who set up and operated the CPT equipment, monitored data collection, and determined the termination depths of the soundings. The CPT logs are shown in Figures 2-1 and 2-2, with symbol key and notes in Figure 2-0.

CPT Calculations

The field data were analyzed in our office using the program CPT-Pro, Ver. 5.49 by Geosoft. The program automatically applies corrections for depth, and post/pre-data collection baseline readings. These corrected field data are plotted in the CPT logs, which are: field tip resistance (q_c), sleeve friction (f_s) and pore water pressure (u2). Soil types were determined based upon the Robertson (1986) method (see Figure 2-0 for references). Undrained shear strengths (s_u) were calculated for cohesive materials based upon the Lunne (1997) method. Equivalent Standard Penetration Test (SPT) N_{60}–values were calculated by CPT-Pro, with empirical corrections developed by Reitz & Jens. The estimates of internal friction angle (ϕ) in coarse-grain soil were based upon the measured q_c values using Bowles (1996). The computed parameters N_{60}, s_u and ϕ are also plotted in the CPT logs.

Summary of Subsurface Conditions

The two CPT soundings revealed generally silty sands to about 4.5 to 5 feet, followed by sands and gravelly sands to the depths of the soundings (40.2 and 51.6 feet). The calculated internal friction angle (ϕ) varied from 30˚ near the surface, and increased to about 36˚ to 39˚ at a depth of 10 feet. The ϕ varied between about 34˚ and 40˚ below about 10 feet to the termination depths of the soundings. Geotechnology and Haley & Aldrich had assumed a ϕ of 35˚ in their analyses. So, our findings were consistent with their previous analyses.

The groundwater level when the CPT soundings were made are calculated from the piezometric data (u2) from the CPT probe. It appears that the groundwater level was at a depth of 10 feet at the time of our field investigation. The site is located in the flood plain of the Mississippi River. Therefore, the depth of the groundwater will vary significantly with the Mississippi River level and recent precipitation. Long-term groundwater data from piezometers and monitoring wells at the Sikeston Power Station indicate that the groundwater depth at the time of our investigation is typical.

Seismic Assessment

A site-specific seismic analysis was completed using the program SHAKE2000. Whereas the other procedures use generalized parameters for the soil properties and earthquake motions, this procedure is more site-specific because it uses field data for the soils, coupled with earthquake acceleration time histories. A site-specific seismic analysis has two components – to determine the probable seismic acceleration (or "time history") for the bedrock beneath the site, and to determine the impact or amplification of the seismic acceleration at the ground surface due to the soils.

Ten pseudo bedrock acceleration time-histories developed for each of St. Louis, Carbondale, Illinois and Memphis are included in SHAKE2000. The development of these pseudo earthquakes is documented in the Chiun-Lin Wu and Y.K. Wen (1999) report "Uniform Hazard Ground Motions and Response Spectra for Mid-American Cities." Their method of simulation is based on the latest seismicity information in the region, and the most recent ground motion and simulation models that are appropriate for engineering applications in this region. The seismological data are mainly from the USGS open-file Report 96-532. The sets of ground motions were selected from a large pool of simulated ground motions such that the median of the response spectra matched those of the 10% and 2% exceedance in 50 years. We selected pseudo bedrock acceleration time-histories for Memphis because it is more similar to Sikeston geologically than St. Louis or Carbondale. We ran multiple SHAKE2000 analyses using short-duration, medium-duration, and long-duration time-histories:

Properties of Memphis Pseudo Time-Histories Selected for Analyses

Designation	Duration t, seconds	Magnitude Mw	Distance Re, km	Peak Acceleration
Short	30	8	170.4	0.26g
Medium	50	8	117.6	0.40g
Long	75	8	97.6	0.28g

We compared the response spectra accelerations for a short-period (Ss) and a 1-second period (S_1) from the USGS Seismic Hazards Design Maps website for a Site Class "B" site at Memphis and at the Sikeston Power Station, and for an earthquake with a 2% chance of exceedance in 50 years per IBC 2012/2015. We found that the accelerations at the Sikeston Power Station are about 2.5 times that at Memphis. Therefore, we scaled or multiplied the bedrock accelerations in the time-histories in the SHAKE2000 program by 2.5 to produce our bedrock accelerations at the site.

The second step in the site specific seismic analyses – determination of the impact or amplification of the seismic acceleration at the ground surface due to the soils – was completed using the SHAKE2000 computer program. We developed the input soils properties for the analyses based upon the results from the University of Memphis study (Appendix C, Table 1) to a depth of 325 feet, and with consideration of the results from our two CPT soundings. The University of Memphis reported a shear wave velocity of 2000 to 2200 m/sec (6500 to 7200 ft/sec) for the Cretaceous strata from 325 feet to 770 feet. However, the SHAKE2000 program returned an error message that a shear wave velocity greater than 4000 ft/sec is not reasonable for rock. Therefore, we made the bottom of our soil column model at 325 feet.

The results from our SHAKE2000 analyses are shown in Figures 3-1 through 3-9 for the long-, medium- and short-duration earthquakes. The response spectral accelerations are shown in Figures 3-1, 3-4 and

3-7. We included in these plots the design response spectral accelerations calculated from the USGS 2012 Seismic Hazard Design Maps for the Sikeston Power Station, for a Site Class "E", and the design earthquake (2% chance of exceedance in 50 years) for comparison. The results of the Peak Horizontal Ground Accelerations (PHGA) versus depth are plotted in Figures 3-2, 3-5 and 3-8.

Liquefaction Analyses

The CCR rule states [40 CFR Part 257.73(e)(iv)] that "For dikes constructed of soils that have susceptibility to liquefaction, the calculated liquefaction factor of safety must equal or exceed 1.20." The dikes of the fly ash pond are composed of fine sands and silty sands which might be susceptible to liquefaction except that (1) the fill is generally in a compacted state, and (2) the dike is generally above the groundwater level. If the phreatic surface (line of seepage) through the dike were elevated, then a portion of the dike would be saturated and could be subject to liquefaction. The long-term depth readings from the piezometers installed by Geotechnology in the dike in 2011 indicate that the phreatic surface is below the dike. Therefore, it is unlikely that the 4th load condition is applicable if only the potential liquefaction of the dike itself is considered.

However, it is our judgement that liquefaction potential of the subsurface soil strata must be considered, not only the liquefaction potential of the dike itself. If such liquefaction could occur immediately following the design seismic event, then the dike may be susceptible to failure. Therefore, we evaluated the liquefaction potential of the subsurface soil strata for the 4th load condition.

Liquefaction occurs when ground shaking is sufficient to produce cyclic particle movements that cause excess pore water pressures to build to the point that nearly all the strength of the soil is lost. After ground shaking has stopped, the soil will potentially reconsolidate to denser configuration, which results in settlement. Liquefaction is most problematic in loose sandy soils with less than about 35 percent fines (soils which are finer than standard sieve size #200) but can occur in very loose soils with up to 50 percent fines and soils up to the size of fine gravel.

Factors of Safety (FS) against liquefaction were calculated for the 2 CPT soundings using the cyclic stress approach outlined in Idriss and Boulanger (2008). The CPT soundings were analyzed using the cone tip pressure, which was corrected for overburden pressure and fines content, termed $(q_{1N})_{CS}$. The fines content in the CPT soundings were those associated with the descriptions on the CPT Logs:

CPT Log Descriptive Phrase	Fines Content (%)
Sand	1
Sand to Silty Sand	10
Silty Sand to Sandy Silt	36

The liquefaction analyses were based upon the calculated horizontal ground acceleration at the depths of each soil stratum that were derived from each of the 3 pseudo time-histories (Figures 3-2, 3-5 and 3-8). Our analyses indicate that is potential for liquefaction in several sand strata, as illustrated in Figure 4-3 for the static load condition with liquefaction.

Results of Slope Stability Analyses

We analyzed the stability of the critical embankment cross-section using the program SLIDE 7.0. This program uses the Spencer method, which resolves the static forces on each vertical slice of soil profile along a given circular or irregular assumed failure surface. The program searches for the minimum factor of safety (FS) against slope failure for each center point in the grid by incrementally varying the radius of the failure surface. The plotted results from the program show the minimum FS, the center and radius of the failure surface with the minimum FS. The output of the program also plots contours of equal FS within the grid of possible center points. The properties of the fly ash in the pond and of the dike were taken from the previous study by Geotechnology. We considered the cross-section of the dike at 4 locations, but the geometries from the typographic survey by Surdex were very similar. Also, the soil properties from the 2 CPT soundings were very similar. Therefore, we selected the cross-section at the location shown in Figure 1 as the "critical cross-section" for our slope stability analyses.

Static Load Condition

The CCR Rule stipulates 2 static load conditions: (i) long-term with maximum storage pool, and (ii) under the maximum surcharge which is presumably a short-term, temporary load condition. However, since the fly ash pond is nearly full, we judged that only the first load condition is applicable. The cross-section with the properties of the CCR, dike and soil strata are shown in Figure 4-1. The minimum FS is 1.64, which is for a surficial slide on the downstream slope, which would have little impact on the stability of the dike. Two of the other trial slope failures are also shown in Figure 4-1 which are more substantial and could jeopardize the containment of the CCR, but each of these had a FS greater than 3.6. The minimum required FS for this load condition is 1.50. Therefore, the fly ash pond is satisfactory for the long-term, maximum storage load condition.

Seismic Load Condition

The minimum required FS for the seismic load condition is 1.00. However, the CCR does not stipulate how to calculate a pseudo-static horizontal acceleration for this slope stability analyses. The peak ground acceleration (PGA) is extremely short in duration. Therefore, the full PGA is not applicable for this slope stability analyses. For the bottom ash pond, Haley & Aldrich used the recommendation from the Mine Safety and Health Administration (MSHA) 2009 *Engineering and Design Manual for Coal Refuse Disposal Facilities* which states that the maximum acceptable deformation of the dike for a surface impoundment is 25% of the freeboard. The preamble to the CCR [Unit VI.E(3)(b)(ii)(d)] states that "all CCR surface impoundments must also be capable of withstanding a design earthquake without damage to the foundation or embankment that would cause a discharge of its contents." Therefore, we understand that the common practice is to use the MSHA design criterion to determine a pseudo-static acceleration (Ks) that would produce the maximum acceptable deformation using the Newmark (1965) method of analyses. The Newmark method is part of the SHAKE2000 program. A trial Ks is input to the program. The Ks is compared to the ground accelerations in a time-history. When the ground acceleration exceeds the Ks the associated lateral displacement is calculated using the empirical relationship developed by Makdisi and Seed (1978). The lateral displacements are cumulated over the time-history assuming that all of the displacements occur in the same direction. We ran trial analyses until a Ks was found for each of the 3 pseudo time-histories that resulted in a calculated lateral deformation at the ground surface equal to about the maximum acceptable 25% of the freeboard. The current freeboard of the fly ash pond is 4.5 feet (el. 322.5 minus el. 318). Therefore, the maximum acceptable deformation is 25% of 4.5 feet or 13.5 inches. Results of the Newmark analyses are shown in Figures 3-3, 3-6 and 3-9. The Ks is 0.13g for the short-duration event, 0.17g for the medium-duration event, and 0.19g for the long-duration event.

We calculated the FS for the seismic load condition by two methods. First, the critical cross-section was analyzed using the pseudo-static acceleration Ks of 0.19g. The results are presented in Figure 4-2. The minimum FS is 1.09, which is for a surficial slide on the downstream slope, which would have little risk of allowing discharge of the CCR and water from the pond. Two of the other trial slope failures are also shown in Figure 4-2 which are more substantial and would jeopardize the containment of the CCR, but each of these had a FS greater than 1.6 and 2.7.

For the second method, we analyzed the critical cross-section to determine the yield pseudo-static acceleration (Ky) that resulted in a minimum FS of 1.0. A Ky = 0.21g resulted in a FS of 1.0 for a surficial slide in the downstream slope that would have little impact on the stability of the dike. A Ky = 0.25g resulted in a more substantial slide that involved more of the dike and the foundation strata and also had a FS of about 1.0. For this method, the FS of the seismic load case was defined as the ratio of the yield pseudo-static acceleration (Ky) to the Ks that produced the maximum acceptable deformation at the ground surface. The FS by this method = (0.25g/0.19g) = 1.32.

The minimum required FS for this load condition is 1.00. Therefore, the fly ash pond is satisfactory for the seismic load condition.

Static Load Condition with Liquefaction

For the static load condition with liquefaction, we applied a residual cohesive shear strength to those sand strata that have a potential to liquefy under the design earthquake, specifically the medium- and long-duration events. The residual shear strength is based upon published correlations. The results are presented in Figure 4-3. The minimum FS is 1.68, which is for a surficial slide on the downstream slope, which would have little risk of allowing discharge of the CCR and water from the pond. Two of the other trial slope failures are also shown in Figure 4-3 which are more substantial and would jeopardize the containment of the CCR, but each of these had a FS greater than 3.1 and 4.1.

The minimum required FS for this load condition is 1.20. Therefore, the fly ash pond is satisfactory for the static load condition with liquefaction of the foundation soil strata.

Closure

Based upon our field investigation and analyses, we judge that the existing condition of the Sikeston Power Station's fly ash pond meets or exceeds the minimum factor of safety criteria of 40 CFR Part 257.73(e).

We welcome any questions or comments that GREDELL or Sikeston BMU may have regarding this report. We appreciate the opportunity to continue our working relationship with GREDELL and Sikeston BMU.

Sincerely,
REITZ & JENS, Inc.

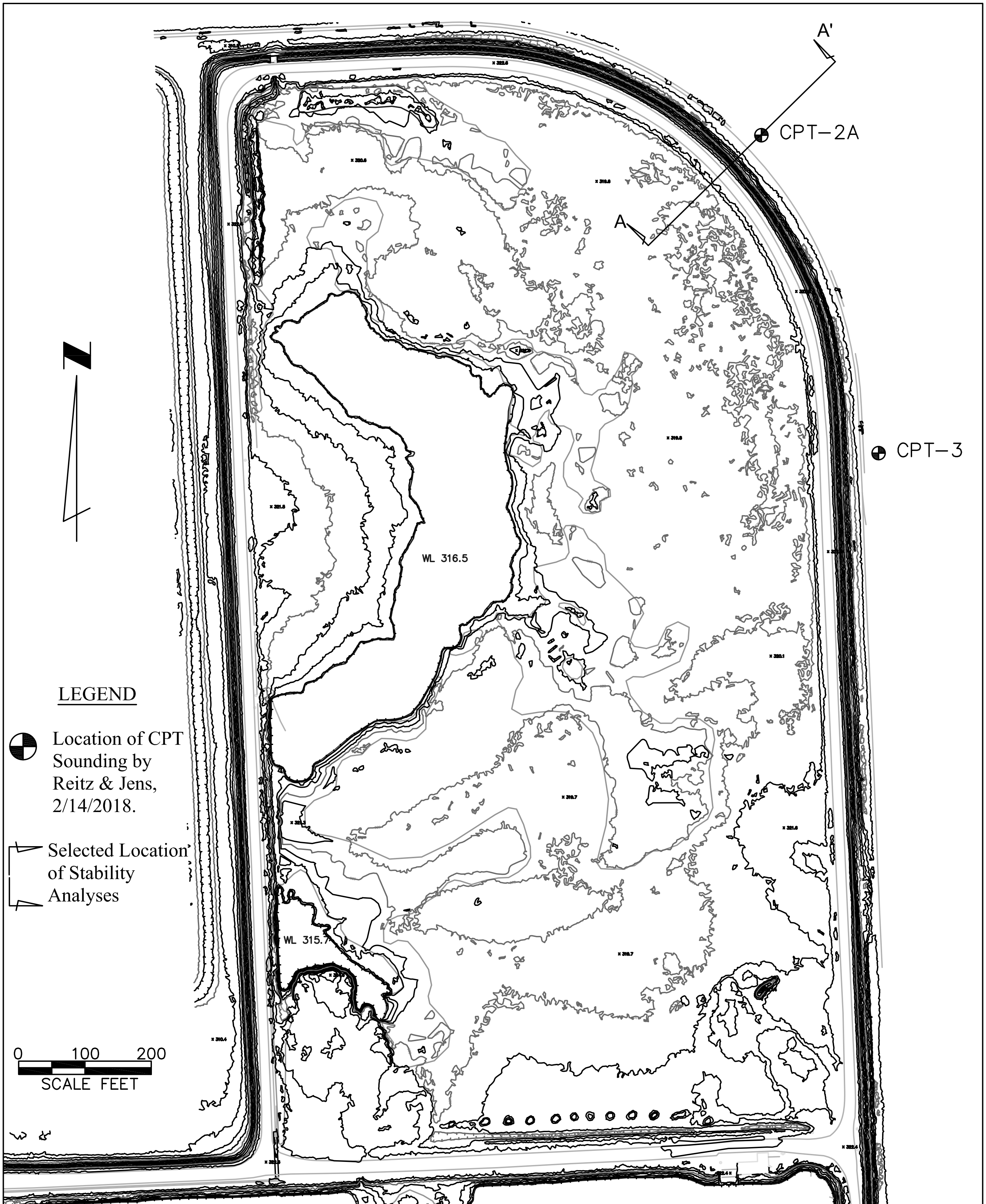
Jeffrey L. Fouse, P.E.
Principal
Email: jfouse@reitzjens.com
Cell phone: 314-852-1110

Cc: Mr. Mark E. McGill, Sikeston BMU / Sikeston Power Station

The following attachments complete this report:

Figure 1	Plan of Fly Ash Pond and Locations of CPT Soundings
Figure 2-0	Key to CPT Sounding Log
Figures 2-1 and 2-2	Individual CPT Sounding Logs
Figures 3-1 to 3-9	Results of Analyses from SHAKE2000
Figures 4-1 to 4-3	Results of Slope Stability Analyses

Copies submitted: 1 bound, emailed PDF



Note: figure based upon orthophotographic map by Surdex Corp. for GREDELL Engineering Resources. Photo dated May 6, 2016.

Sikeston BMU / Sikeston Power Station
Safety Factor Assessment of Fly Ash Pond
PLAN OF FLY ASH POND AND
LOCATIONS OF CPT SOUNDINGS

REITZ & JENS, INC.

Figure 1

LEGEND

Symbol Description

KEY TO SOIL SYMBOLS

Symbol	Description
	Organic Material
	Clay
	Silty Clay to Clay
	Clayey Silt to Silty Clay
	Sandy Silt to Clayey Silt
	Silty Sand to Sandy Silt
	Sand to Silty Sand
	Sand
	Gravelly Sand to Sand
	Sand to Clayey Sand

qc = Cone Tip Pressure, tons/sq. ft.

fs = Skin Friction, tons/sq. ft.

Rf = Friction ratio (fs/qc) in %

u2 = Porewater Pressure, psi

N60 = Calculated Equivalent N-value, blows/foot, (Standard Penetration Test)

Su = Calculated Undrained Shear Strength, ksf

Phi = Friction Angle, degrees

TA = Tilt Angle, degrees

Notes:

1. Soundings were made on February 14 and 15, 2018, by Bulldog Drilling, Inc. using 1.5" diameter cone penetrometer with pore pressure measurements (CPTu) owned and operated by Reitz & Jens. Soundings were backfilled the same day with Bentonite crumbles.
2. Soundings were located by Reitz & Jens, and were staked after drilling. Elevations at the CPT locations were not provided; thus, elevations are not shown on the logs.
3. Soundings were logged in the field by Reitz & Jens' geologist who monitored and conducted all CPT related work.
4. Soil classification and equivalent N_{60} were based upon Robertson 1986[1].
5. Undrained shear strength (Su) is based on Lunne, Robertson, Powell (1997)[2]. Internal friction Angle (Phi or Ø) is based on Bowles (1996)[3].
6. Stratification lines shown on the log represent approximate soil boundaries; actual changes in strata may be gradual.

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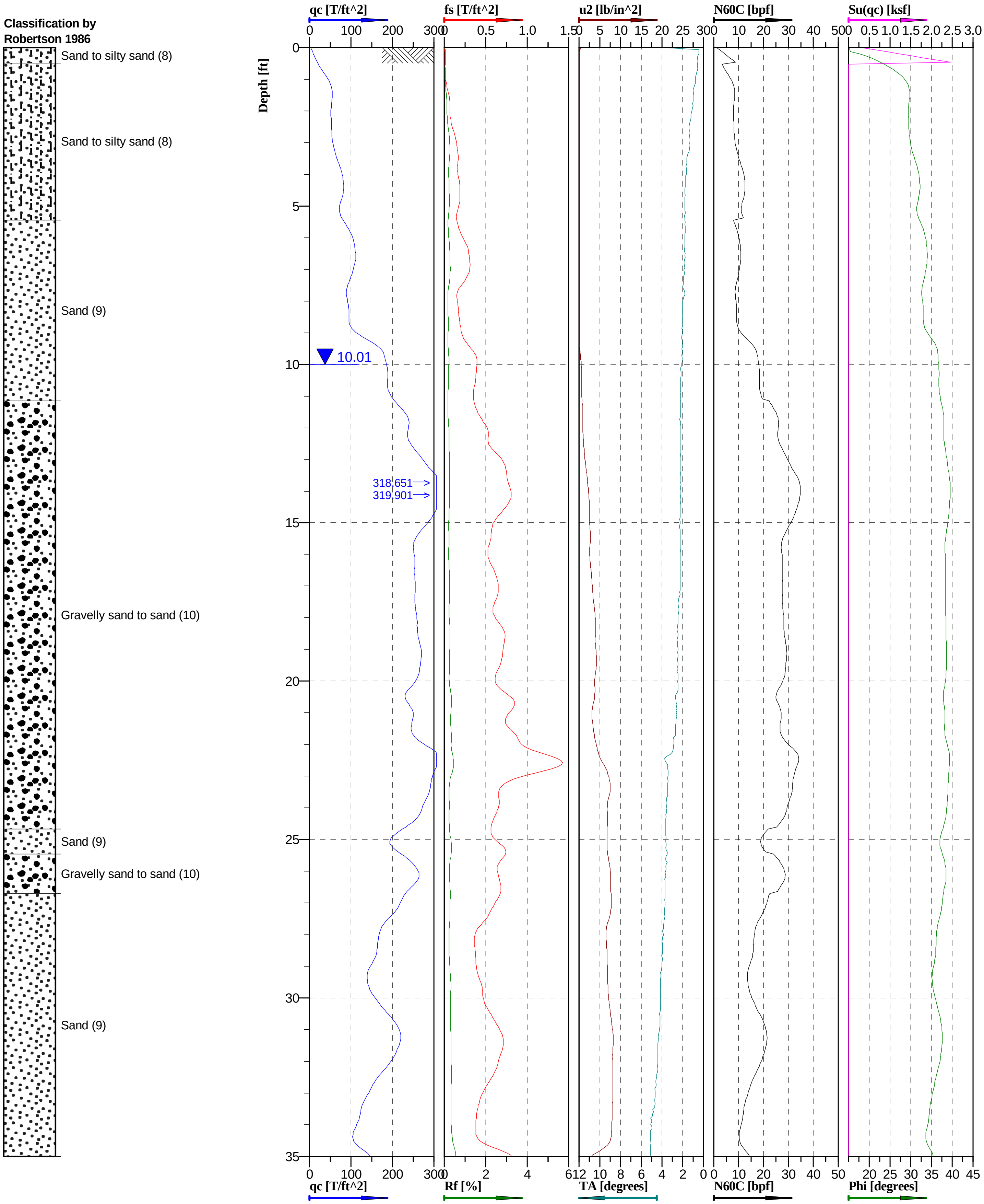
[1] Robertson et al. (1986) *Use of piezometer cone data.* Proceedings of the ASCE Specialty Conference: In Situ 86: Use of In Situ Tests in Geotechnical Engineering. ASCE 1986

[2] Lunne, T. Robertson, P.K. and Powell, J.J.M. (1997) Cone Penetration Testing in Geotechnical Practice, Published by Blackie Academic & Professional.

[3] Bowles, Joseph E. (1996) Foundation Analysis and Design. McGraw-Hill. 5th ed. Page 180.

Figure 2-0



Cone No: 4274
Tip area [cm2]: 10
Sleeve area [cm2]: 150

Location: Sikeston, MO	Position: X: 0.00 ft, Y: 0.00 ft	Ground level: 0.00	Test no: CPT-2A
Project ID: 2018120907	Client: Gredell Engineering Resources, Inc.	Date: 2/14/2018	Scale: 1 : 50
Project: BMU Sikeston Power Station Fly Ash Pond Stabilit		Page: 1/2	Fig: 2-1
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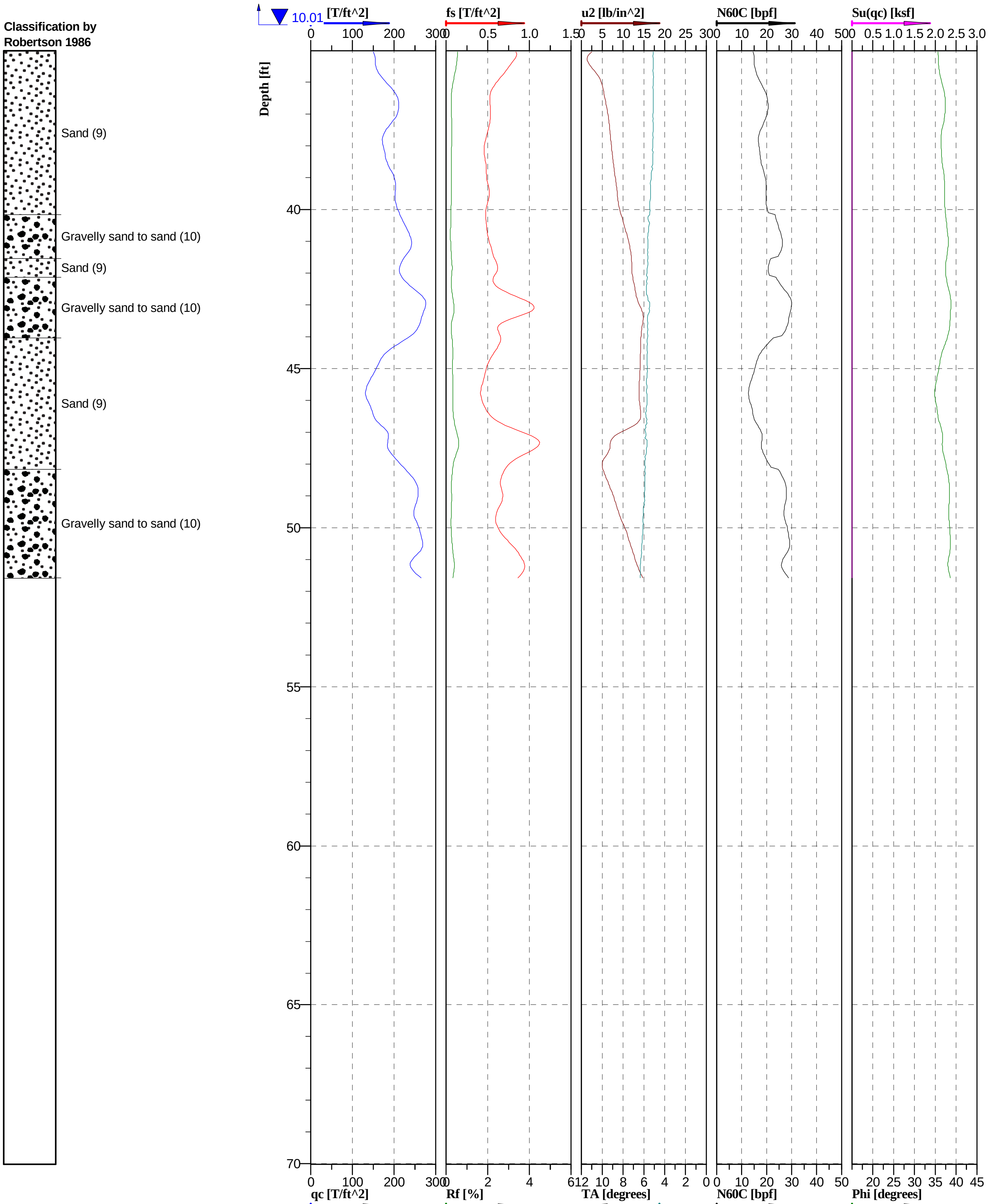

Cone No: 4274
Tip area [cm2]: 10
Sleeve area [cm2]: 150

Location: Sikeston, MO	Position: X: 0.00 ft, Y: 0.00 ft	Ground level: 0.00	Test no: CPT-2A
Project ID: 2018120907	Client: Gredell Engineering Resources, Inc.	Date: 2/14/2018	Scale: 1 : 50
Project: BMU Sikeston Power Station Fly Ash Pond Stabilit		Page: 2/2	Fig: 2-1
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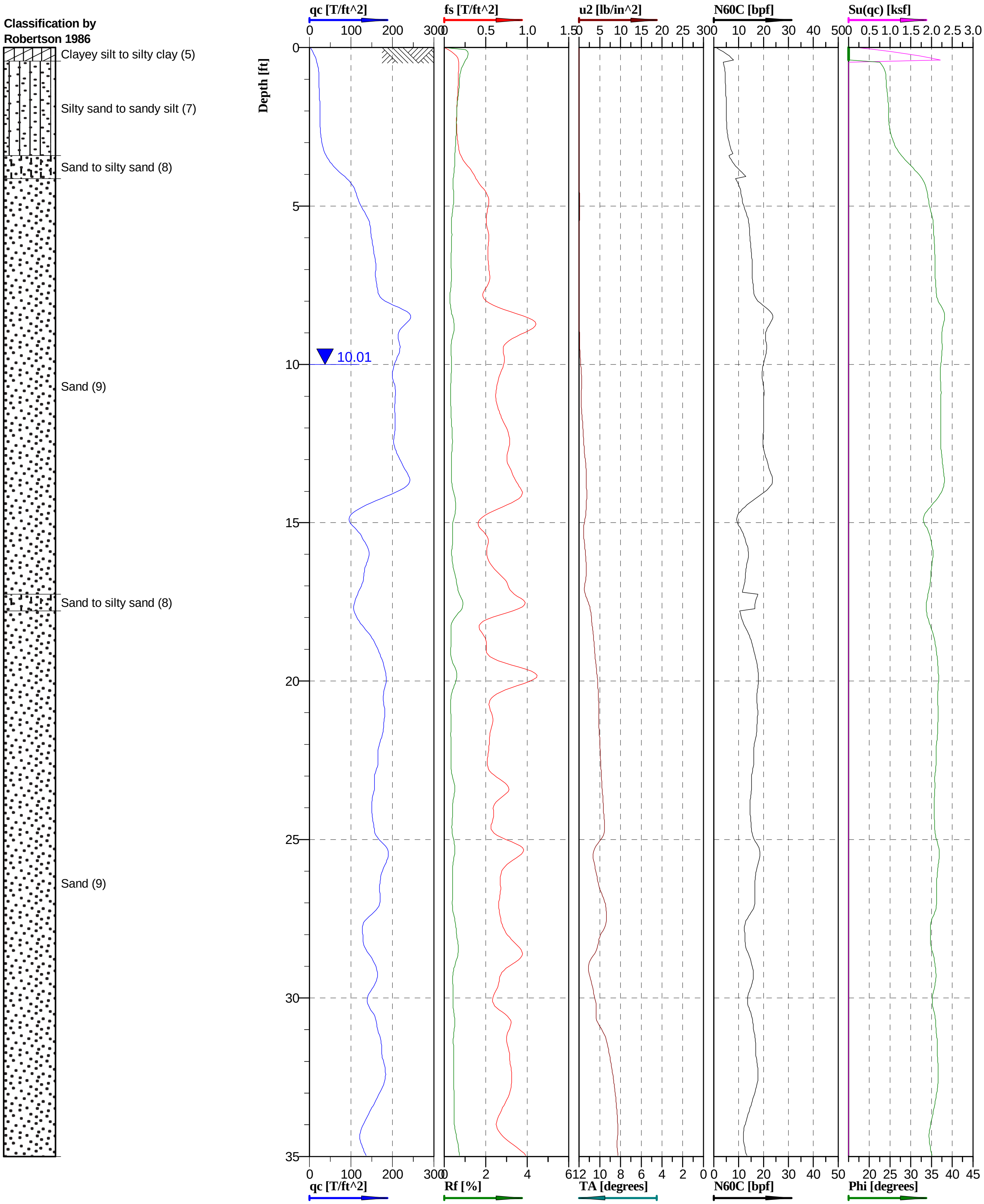
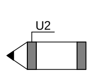
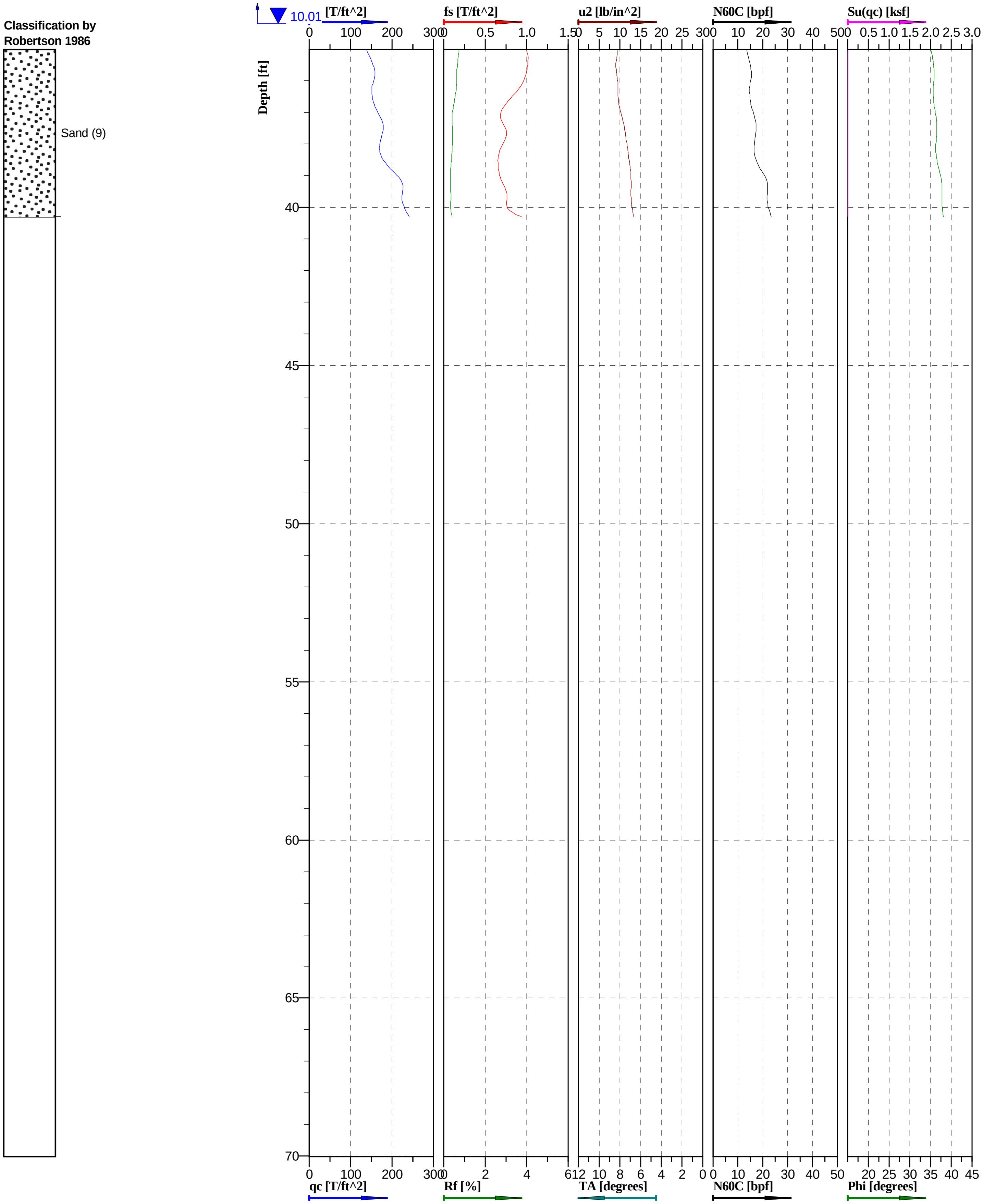




Cone No: 4274
Tip area [cm2]: 10
Sleeve area [cm2]: 150

Location: Sikeston, MO	Position: X: 0.00 ft, Y: 0.00 ft	Ground level: 0.00	Test no: CPT-3
Project ID: 2018120907	Client: Gredell Engineering Resources, Inc.	Date: 2/15/2018	Scale: 1 : 50
Project: BMU Sikeston Power Station Fly Ash Stability		Page: 1/2	Fig: 2-2
		File: cpt-3_corr.csv	



U2

Cone No: 4274
Tip area [cm2]: 10
Sleeve area [cm2]: 150

Location: Sikeston, MO	Position: X: 0.00 ft, Y: 0.00 ft	Ground level: 0.00	Test no: CPT-3
Project ID: 2018120907	Client: Gredell Engineering Resources, Inc.	Date: 2/15/2018	Scale: 1 : 50
Project: BMU Sikeston Power Station Fly Ash Stability		Page: 2/2	Fig: 2-2
		File: cpt-3_corr.csv	

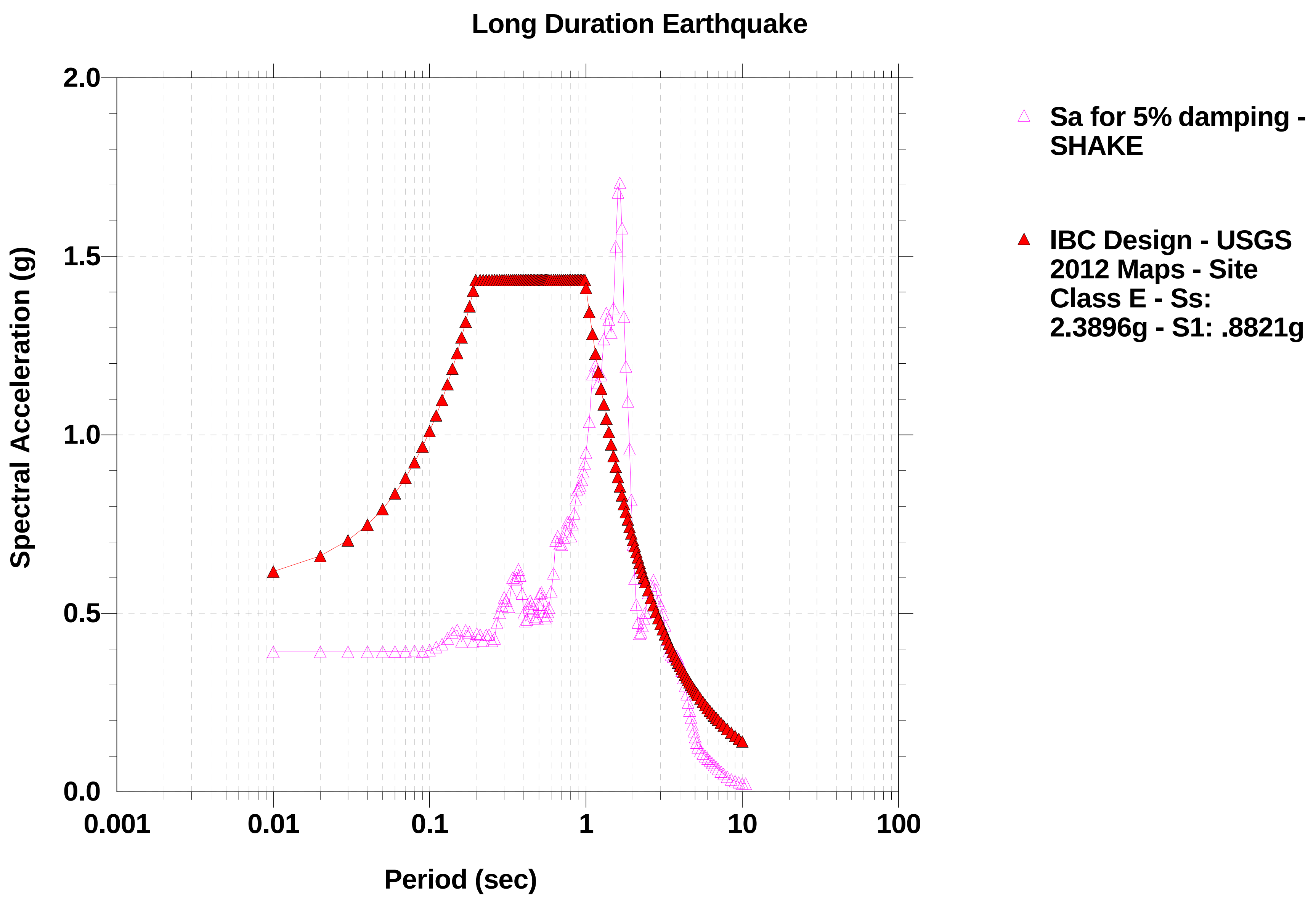


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Figure 3-1

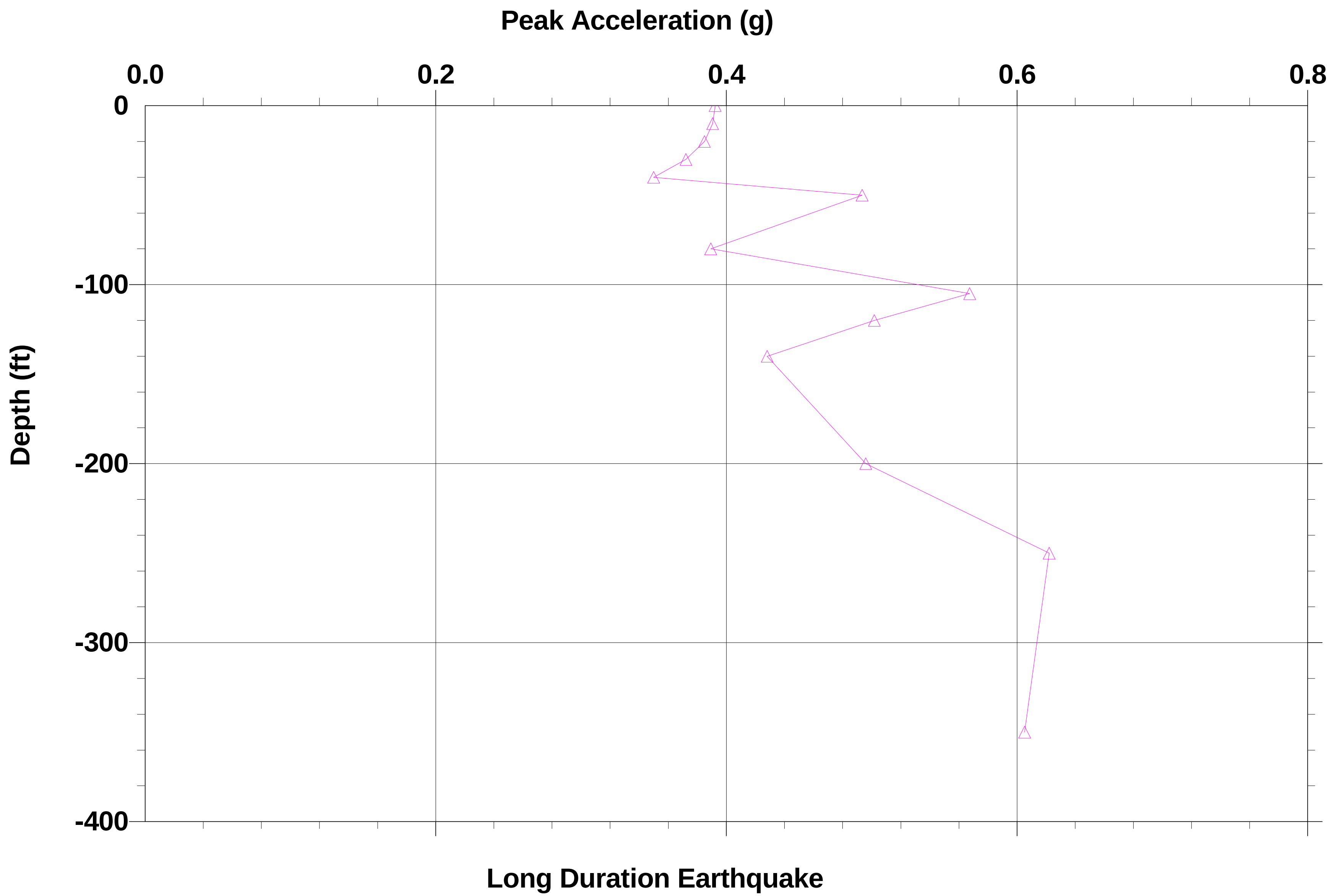

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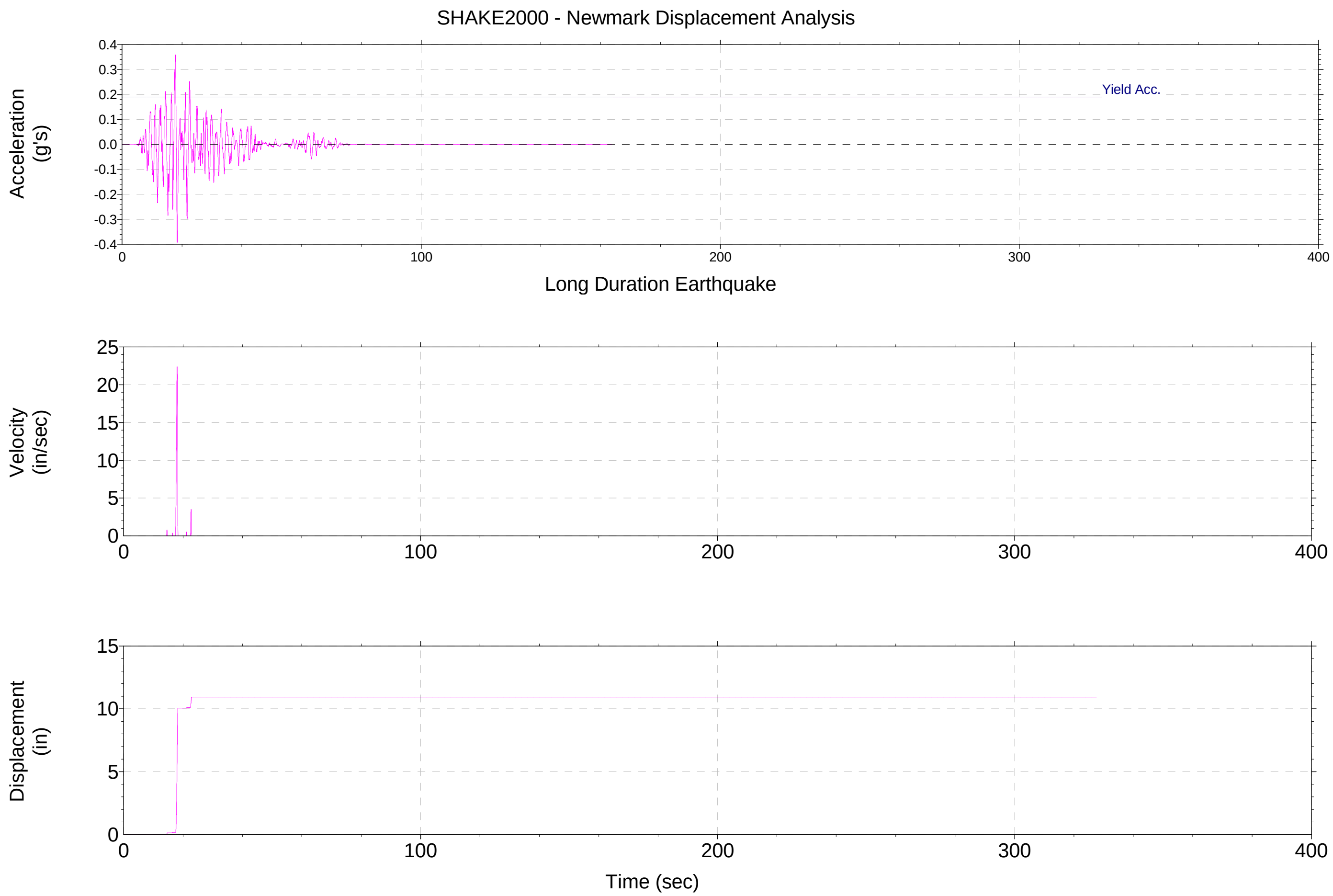
Figure 3-2

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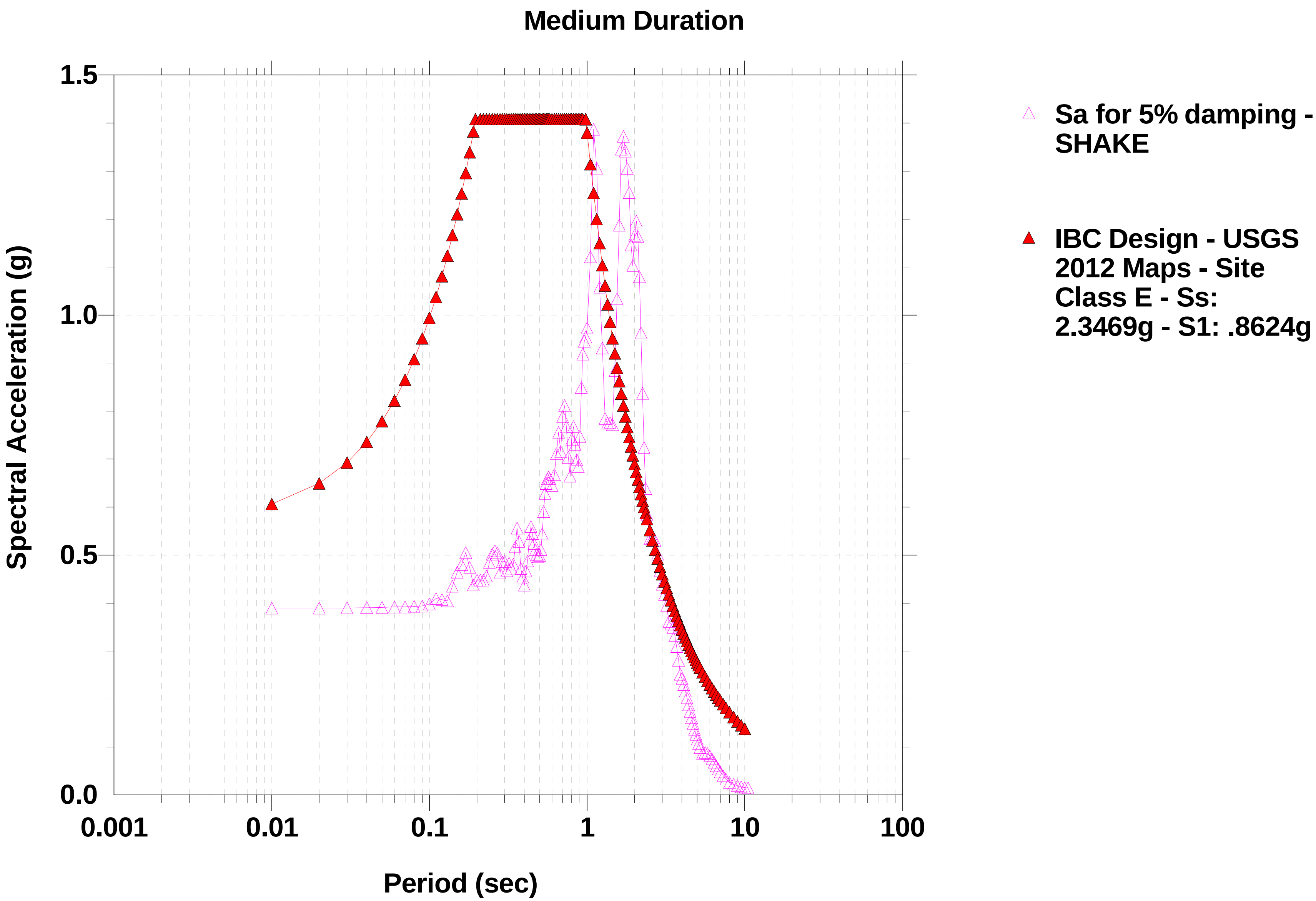
Notes:
      Displacement Analysis - Newmark Method
      Project: SHAKE2000 - Newmark Displacement Analysis

      Newmark Method by Kavazanjian & Matasovic
      Constant Yield Acceleration:      .19 (g)
      Acceleration Time History: Outcrop - Soil Profile No.  1 - AHL - Layer:
                           1 - Analysis: 1 - Soil Deposit: 1
      Acceleration Time History File: P:\GREDELL\2018120907\calc\Shake\Long Duration-L1A1D1-1-Soil Pro-G01000.ahl
      Peak Acceleration Value:   .39225 (g)
      Upslope Movement not Included in Analysis
      Acceleration due to gravity:   386.4 (in/sec^2)

      Displacement computed:    10.92798 in
```


Medium Duration
1.5
1.0
0.5
0.0
Spectral Acceleration (g)
0.001
0.01
0.1
1
10
100
Period (sec)
Sa for 5% damping - SHAKE
IBC Design - USGS 2012 Maps - Site Class E - Ss: 2.3469g - S1: .8624g

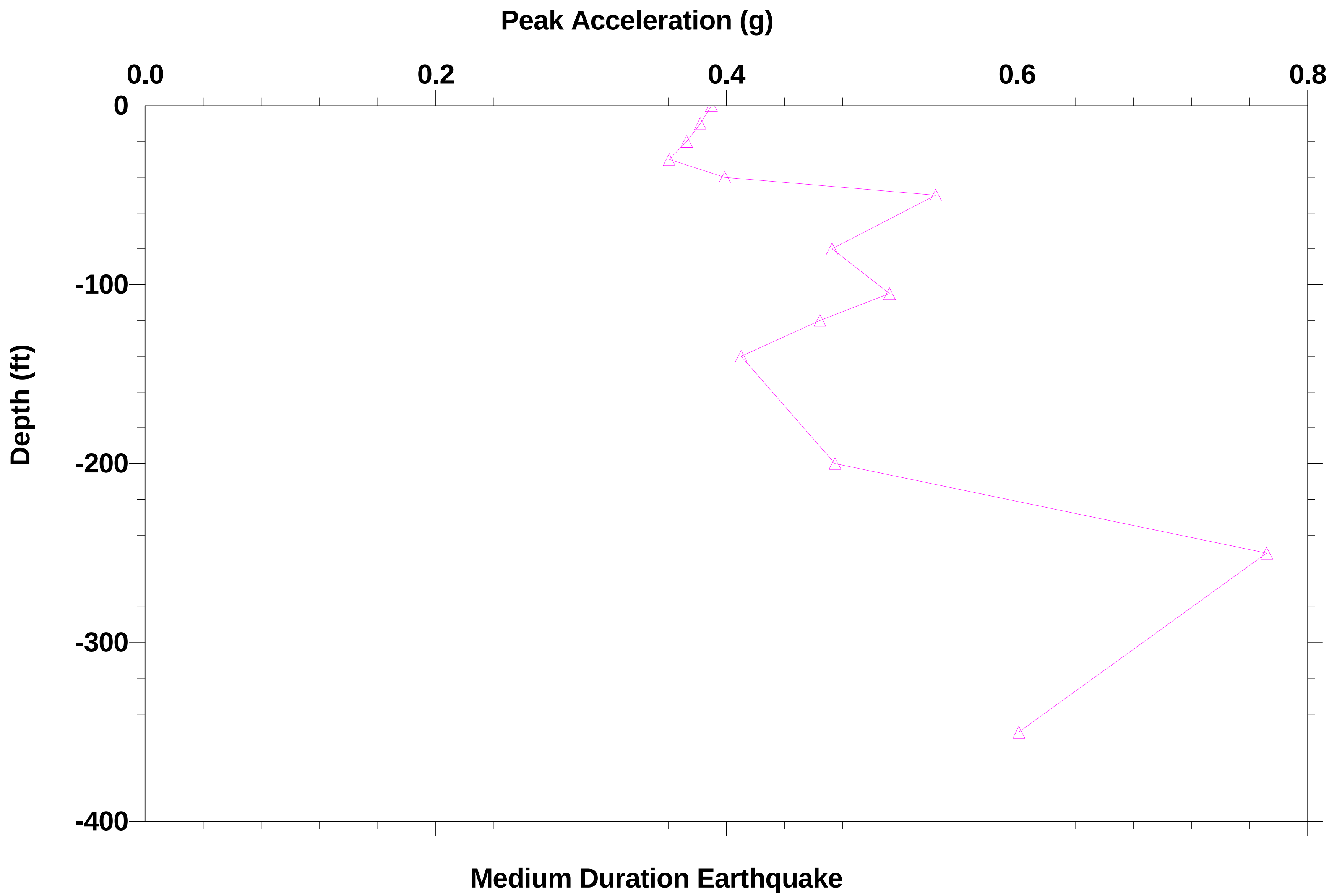


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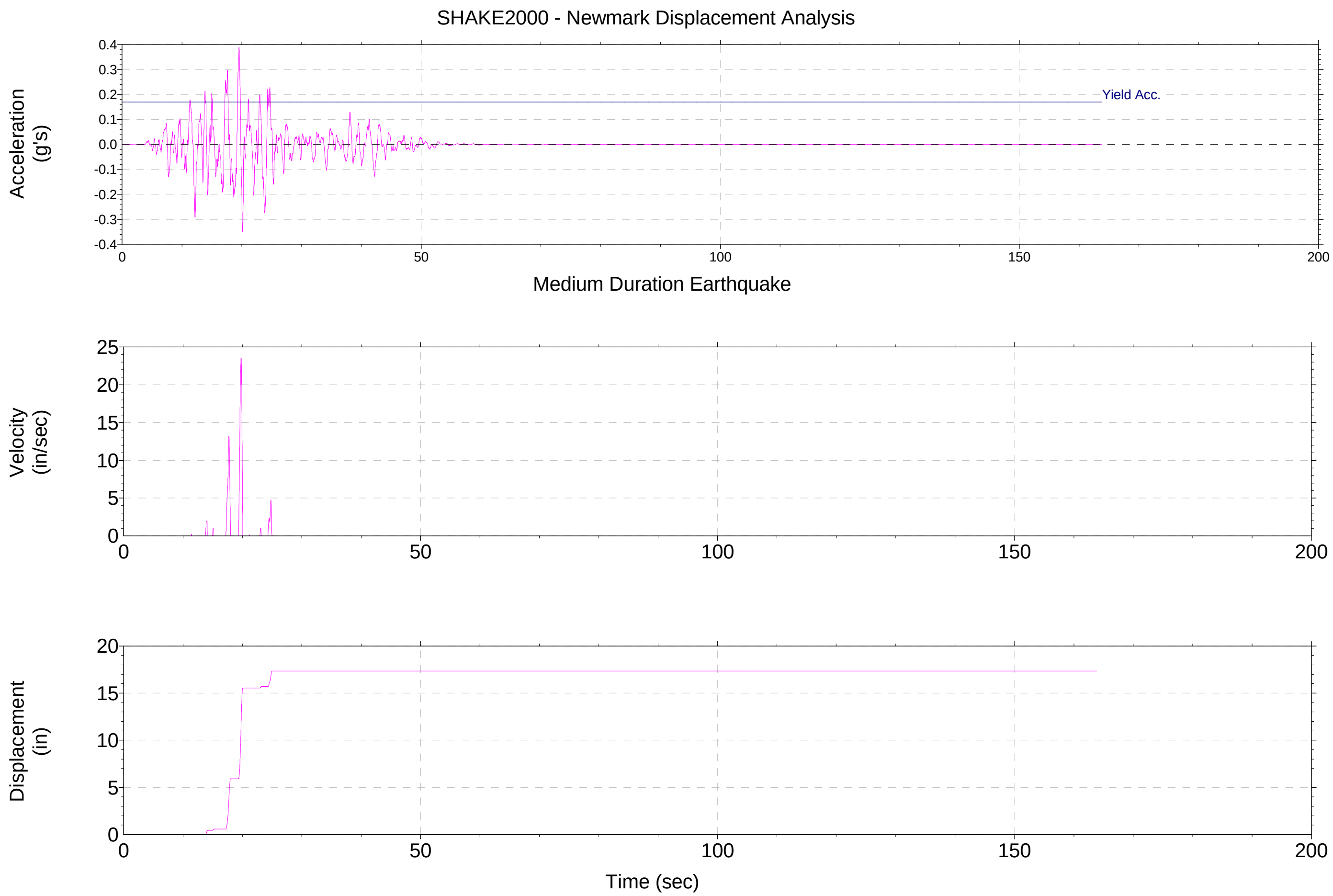
Figure 3-5

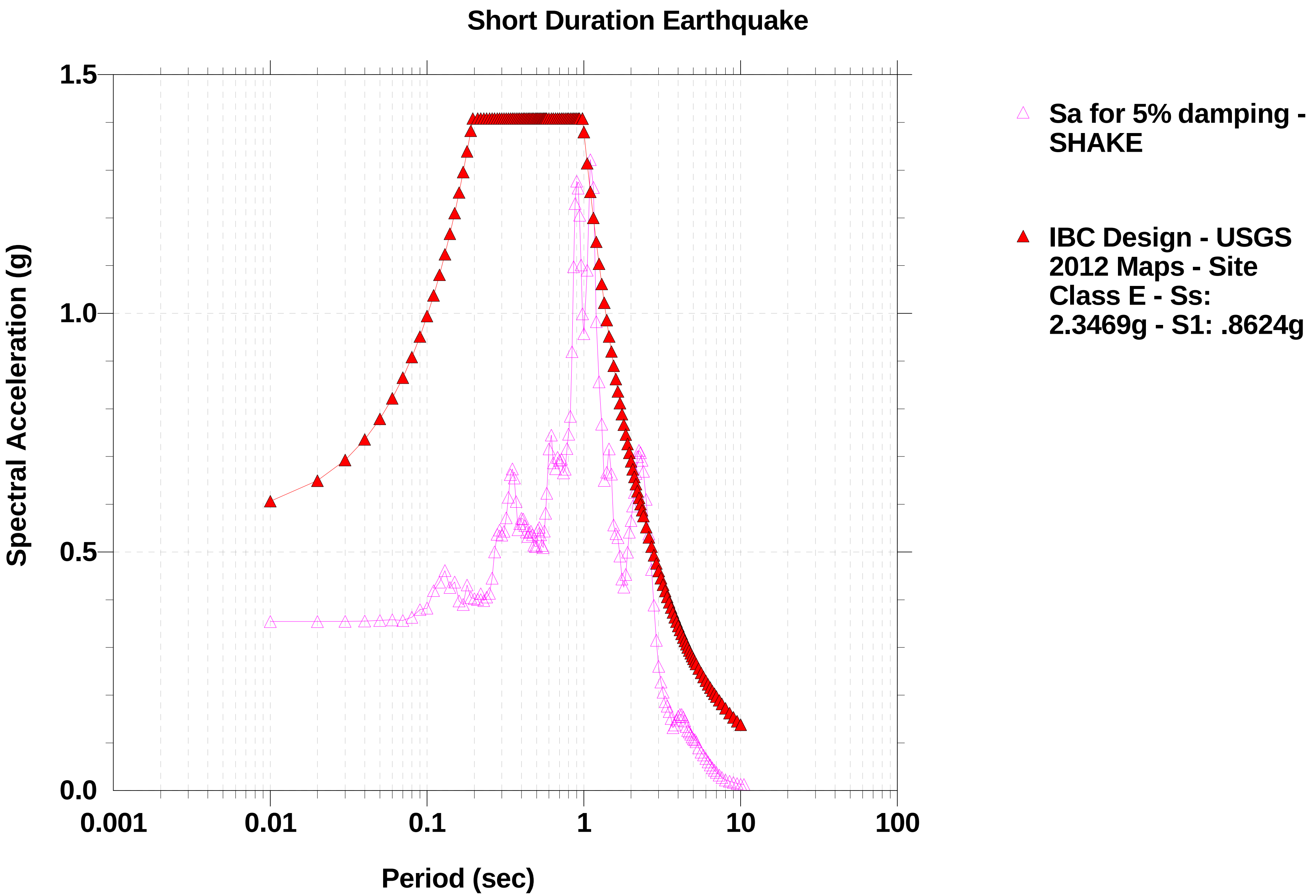


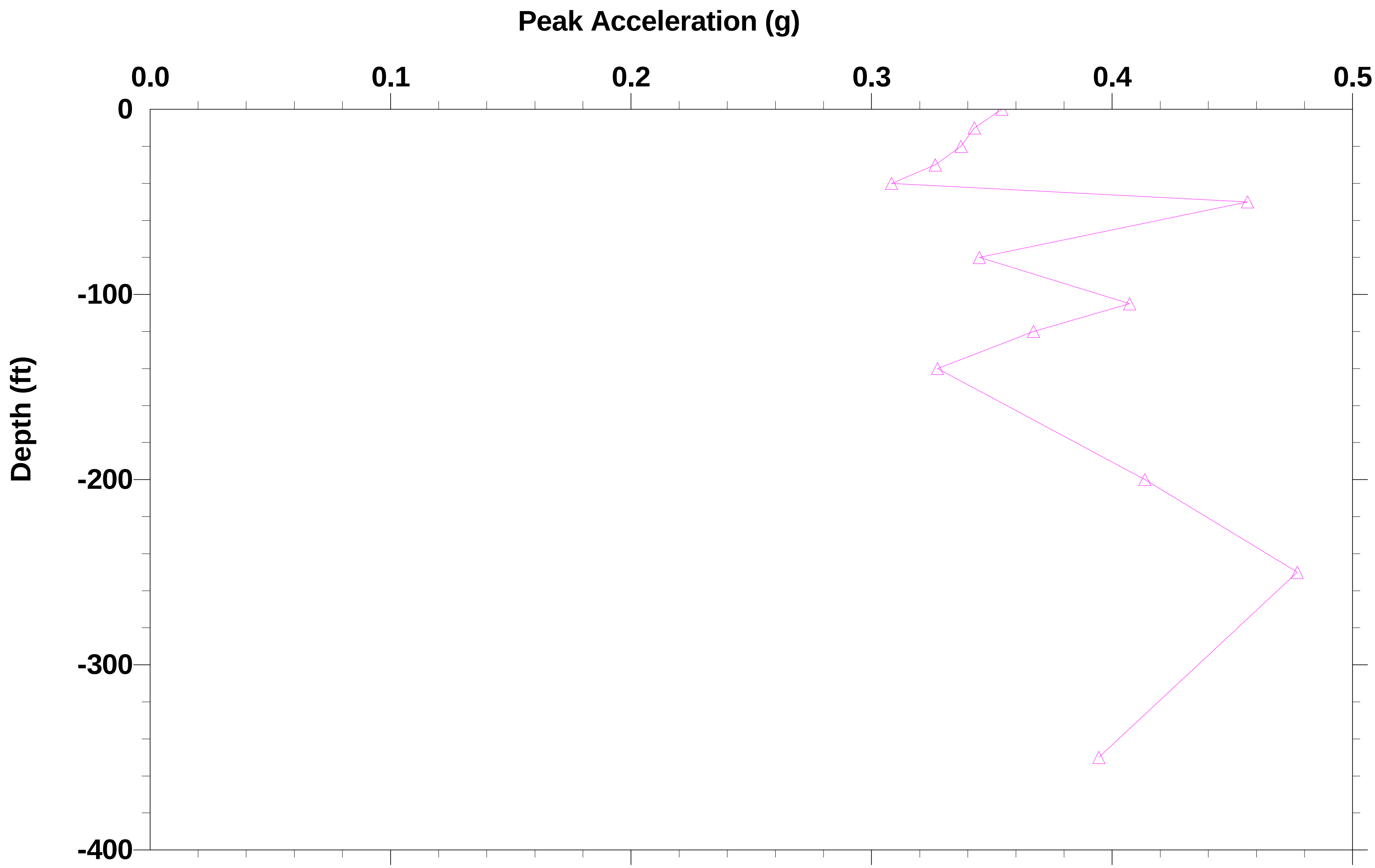
Notes:

Displacement Analysis - Newmark Method
Project: SHAKE2000 - Newmark Displacement Analysis

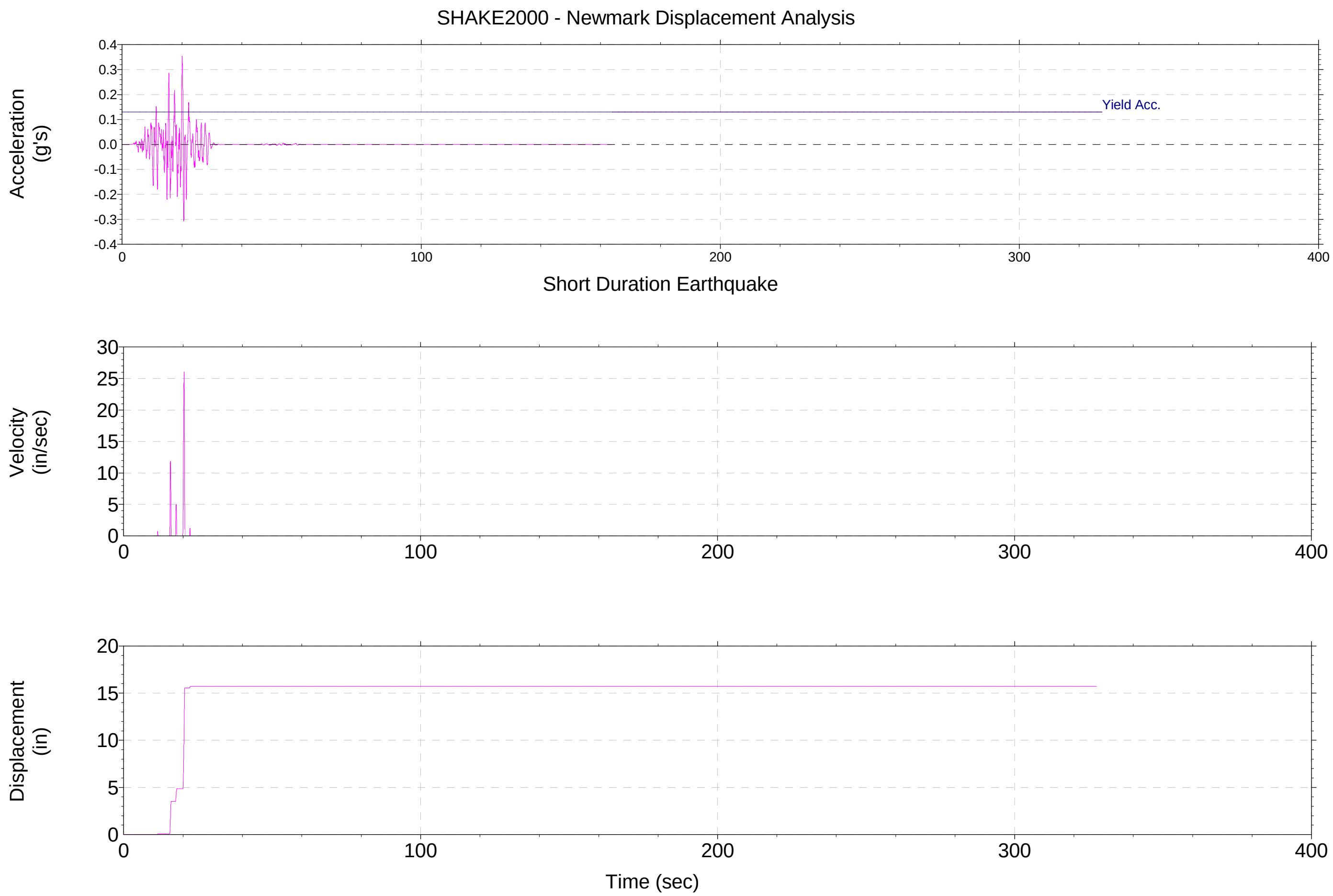
Newmark Method by Kavazanjian & Matasovic
Constant Yield Acceleration: .17 (g)
Acceleration Time History: Outcrop - Soil Profile No. 1 - AHL - Layer: 1 - Analysis: 1 - Soil Deposit: 1
Acceleration Time History File: P:\GREDELL\2018120907\calc\Shake\Med Duration-L1A1D1-1-Soil Pro-TUJ352.ahl
Peak Acceleration Value: .389637 (g)
Upslope Movement not Included in Analysis
Acceleration due to gravity: 386.4 (in/sec^2)

Displacement computed: 17.34549 in


Short Duration Earthquake
Spectral Acceleration (g)
Period (sec)
1.5
1.0
0.5
0.0
0.001
0.01
0.1
1
10
100
Sa for 5% damping - SHAKE
IBC Design - USGS 2012 Maps - Site Class E - Ss: 2.3469g - S1: .8624g



Soil Profile No. 1 - Analysis No. 1 - Profile No. 1 - Soil Pro-TUJ352




```
Notes:
     Displacement Analysis - Newmark Method
     Project: SHAKE2000 - Newmark Displacement Analysis

     Newmark Method by Kavazanjian & Matasovic
     Constant Yield Acceleration:      .13 (g)
     Acceleration Time History: Outcrop - Soil Profile No.  1 - AHL - Layer:
                           1 - Analysis: 1 - Soil Deposit: 1
     Acceleration Time History File: P:\GREDELL\2018120907\calc\Shake\short Duration-L1A1D1-1-Soil Pro-TUJ352.ahl
     Peak Acceleration Value:  .354113 (g)
     Upslope Movement not Included in Analysis
     Acceleration due to gravity:   386.4 (in/sec^2)

     Displacement computed:    15.70344 in
```

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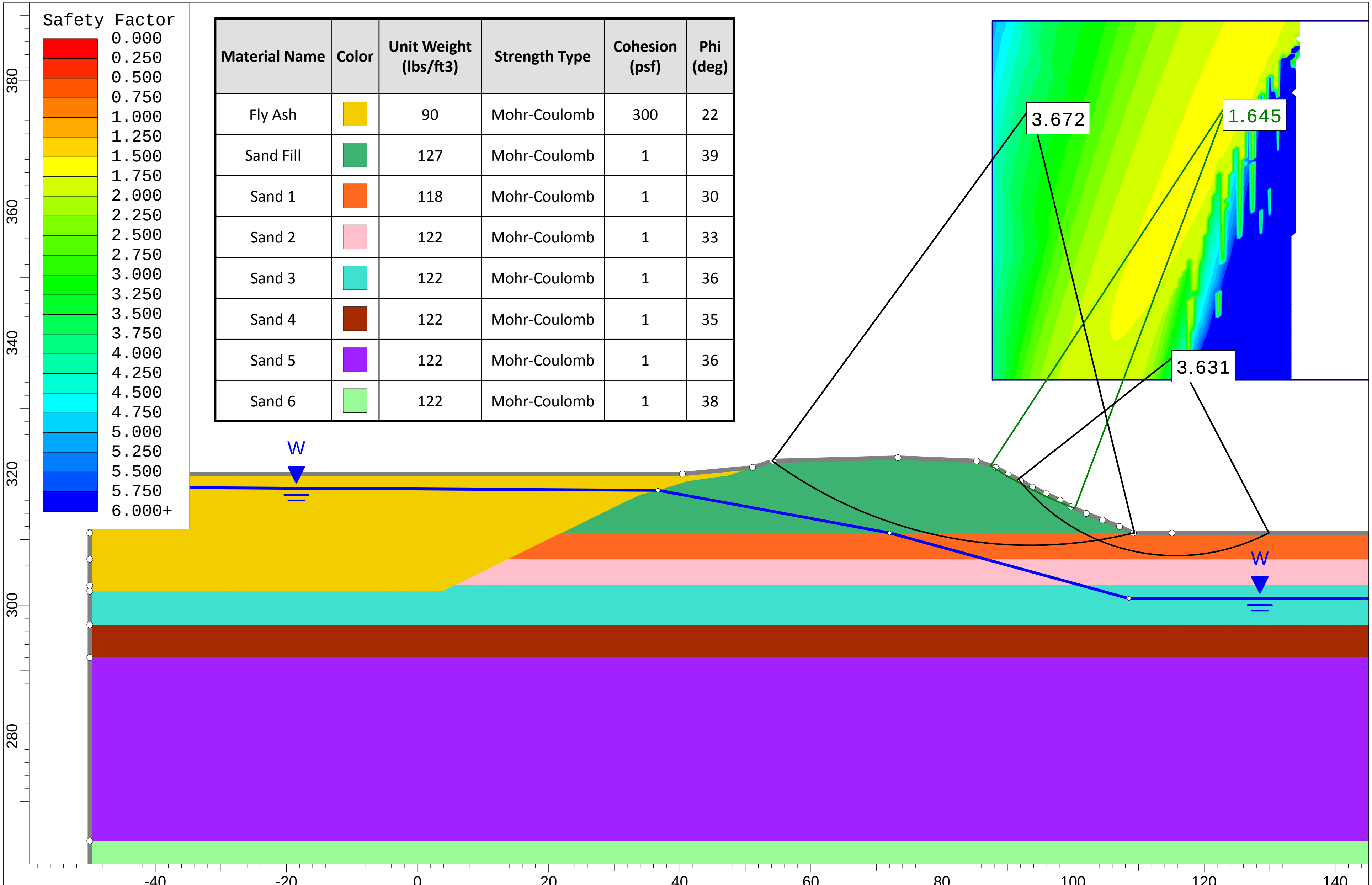
Figure 3-9



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Fly Ash		90	Mohr-Coulomb	300	22
Sand Fill		127	Mohr-Coulomb	1	39
Sand 1		118	Mohr-Coulomb	1	30
Sand 2		122	Mohr-Coulomb	1	33
Sand 3		122	Mohr-Coulomb	1	36
Sand 4		122	Mohr-Coulomb	1	35
Sand 5		122	Mohr-Coulomb	1	36
Sand 6		122	Mohr-Coulomb	1	38

Sikeston BMU / Sikeston Power Station Fly Ash Pond
Section A-A', Static, Long-Term (Drained) Properties, Maximum Pool

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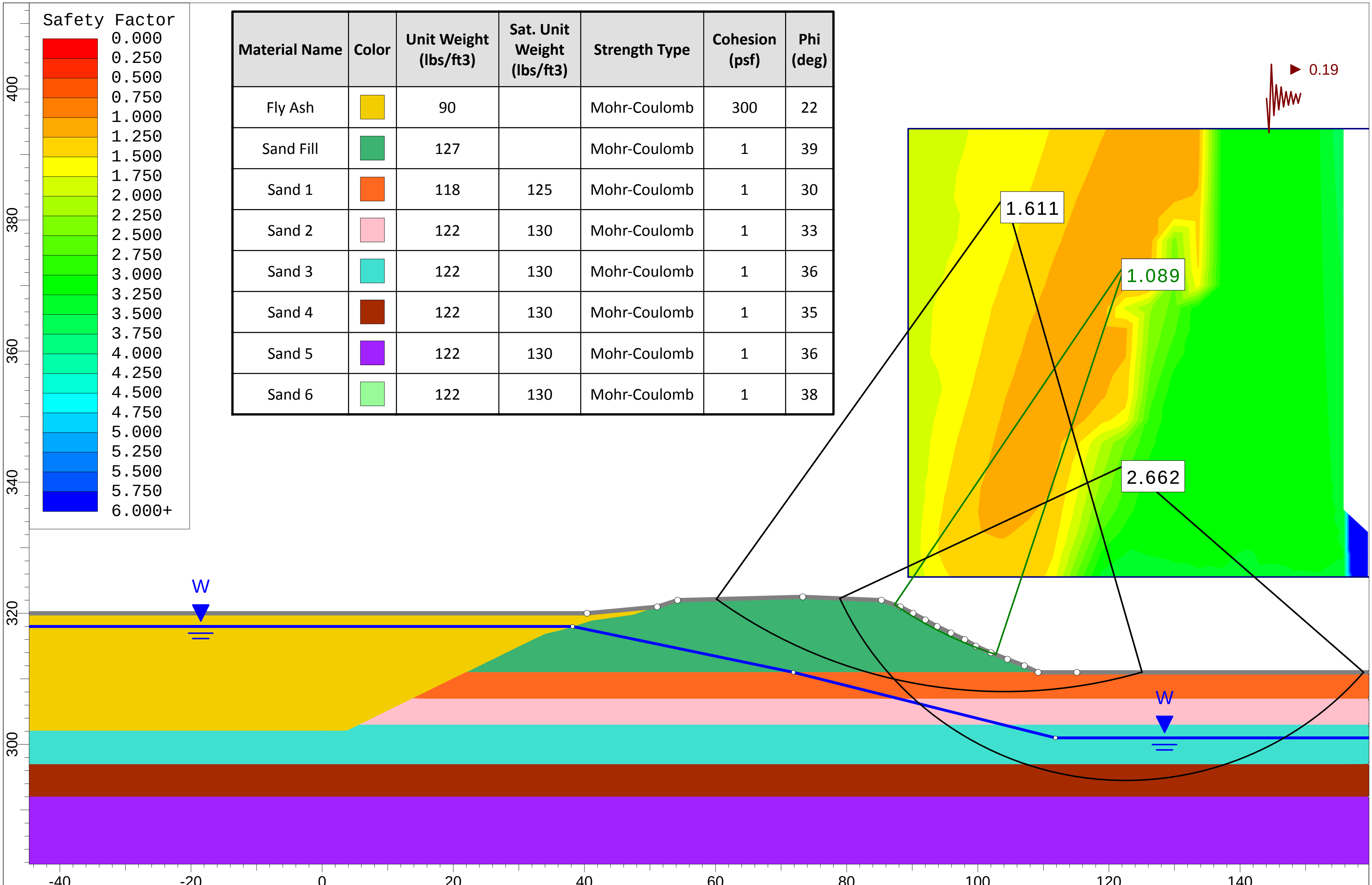
Figure 4-1

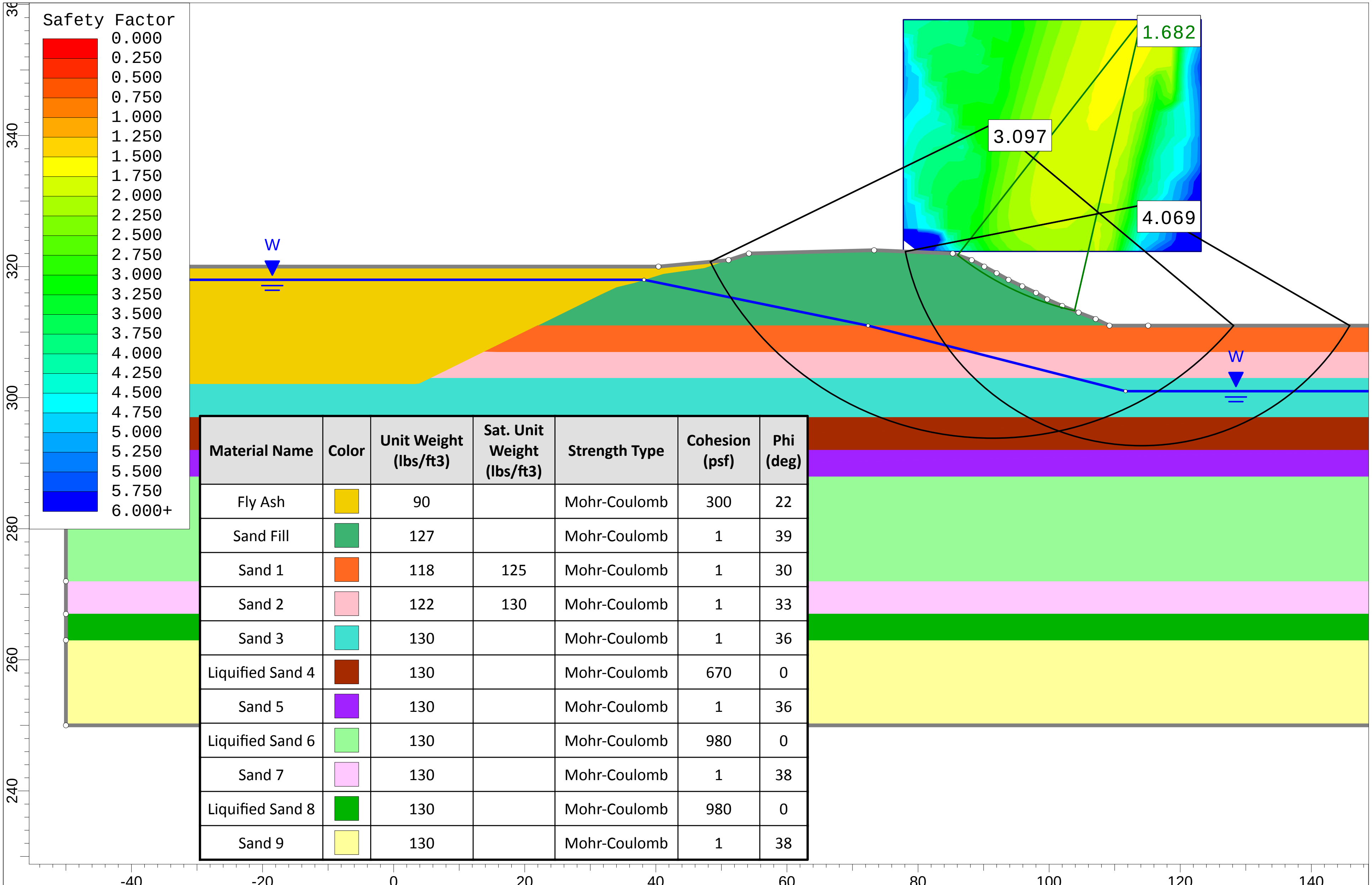


Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Fly Ash		90		Mohr-Coulomb	300	22
Sand Fill		127		Mohr-Coulomb	1	39
Sand 1		118	125	Mohr-Coulomb	1	30
Sand 2		122	130	Mohr-Coulomb	1	33
Sand 3		122	130	Mohr-Coulomb	1	36
Sand 4		122	130	Mohr-Coulomb	1	35
Sand 5		122	130	Mohr-Coulomb	1	36
Sand 6		122	130	Mohr-Coulomb	1	38

Sikeston BMU / Sikeston Power Station Fly Ash Pond
Section A-A', Pseudostatic, Long-Term Properties, Maximum Pool

REITZ & JENS, INC.

Figure 4-2



Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Fly Ash		90		Mohr-Coulomb	300	22
Sand Fill		127		Mohr-Coulomb	1	39
Sand 1		118	125	Mohr-Coulomb	1	30
Sand 2		122	130	Mohr-Coulomb	1	33
Sand 3		130		Mohr-Coulomb	1	36
Liquified Sand 4		130		Mohr-Coulomb	670	0
Sand 5		130		Mohr-Coulomb	1	36
Liquified Sand 6		130		Mohr-Coulomb	980	0
Sand 7		130		Mohr-Coulomb	1	38
Liquified Sand 8		130		Mohr-Coulomb	980	0
Sand 9		130		Mohr-Coulomb	1	38

Sikeston BMU / Sikeston Power Station Fly Ash Pond
Section A-A', With Liquefaction, Maximum Pool

REITZ & JENS, INC.

Figure 4-3