Attorney Client Privileged

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CCR Certification:
Safety Factor Assessment
§257.73 (e)

for the

Ash Pond

at the

A.B. Brown Generating Station

Revision 0

Table of Contents

List of Tables

List of Appendices

Executive Summary

This Coal Combustion Residuals (CCR) Safety Factor Assessment for the Ash Pond at the Southern Indiana Gas & Electric Company, dba Vectren Power Supply, Inc., A.B. Brown Generating Station has been prepared in accordance with the requirements specified in the USEPA CCR Rule under 40 Code of Federal Regulations §257.73 (e)(1). These regulations require that the specified documentation, assessments and plans for an existing CCR surface impoundment be prepared by October 17, 2016.

The Ash Pond meets the regulatory requirements for the safety factor assessment analysis, as summarized in **Table ES-1**.

Table ES-1 – Certification Summary

Report Section	CCR Rule Reference	Requirement Summary	Requirement Met?	Comments
Safety Factor Assessment				
6.1	§257.73 (e)(1)(i)	*Maximum storage pool safety factor must be at least 1.50*	Yes	Safety factors were calculated to be 3.21 and higher.
6.2	§257.73 (e)(1)(ii)	*Maximum surcharge pool safety factor must be at least 1.40*	Yes	Safety factors were calculated to be 3.06 and higher.
6.3	§257.73 (e)(1)(iii)	*Seismic safety factor must be at least 1.00*	Yes	Safety factors were calculated to be 1.32 and higher.
6.4	§257.73 (e)(1)(iv)	*Liquefaction safety factor must be at least 1.20*	Yes	Safety factors were calculated to be 1.23 and higher.

1 Introduction

1.1 Purpose of this Report

The purpose of the Safety Factor Assessment is to document that the requirements specified in 40 Code of Federal Regulations (CFR) §257.73 (e) have been met to support the certification required under each of the applicable regulatory provisions for the A.B. Brown Generating Station (Brown) Ash Pond. The Ash Pond is an existing CCR surface impoundment as defined by 40 CFR §257.53. The CCR Rule requires that the Safety Factor Assessment for an existing CCR surface impoundment be prepared by October 17, 2016.

The Brown station has an interconnected, existing CCR surface impoundment, the Ash Pond, which consists of a lower pool and an upper pool. The following table summarizes the documentation required within the CCR Rule and the sections that specifically respond to those requirements of this assessment.

Table 1-1 – CCR Rule Cross Reference Table

Report Section	Title	CCR Rule Reference
6.1	Factor of Safety: Maximum Storage Pool Loading	§257.73 (e)(1)(i)
6.1	Factor of Safety: Maximum Surcharge Pool Loading	§257.73 (e)(1)(ii)
6.2	Factor of Safety: Seismic	§257.73 (e)(1)(iii)
6.2	Factor of Safety: Post-Liquefaction	§257.73 (e)(1)(iv)

The purpose of the geotechnical investigation and analyses is to evaluate the design, performance, and condition of the Brown Ash Pond using available design drawings, construction records, inspection reports, previous engineering investigations, reports and analyses, station operating records, and other pertinent documents provided by Southern Indiana Gas & Electric Company, dba Vectren Power Supply, Inc. (SIGECO). This information was used in combination with subsurface investigations, laboratory testing, and engineering analyses to evaluate the design and operation of the surface impoundment using current regulatory and engineering practice, and to identify potential geotechnical deficiencies that may require additional investigation, repair or remediation. The regulatory criteria and current engineering practice related to the design of CCR ash impoundments was used as guidance during development of geotechnical analysis and stability evaluations.

Geotechnical field investigations supporting the evaluation were conducted starting in the Spring of 2015 and continued into early Winter 2016, under various mobilizations. These investigations were performed by AECOM and Cardno ATC. The combined field program consisted of 25 conventional hollow stem auger (HSA) borings, and 5 Cone Penetration testing (CPT) soundings. Laboratory testing was conducted on the materials obtained through various sampling techniques to assist in characterization of the subsurface conditions.

In addition to the 2015 / 2016 investigations, historical data available from SIGECO was also reviewed and utilized. Historical data included borings drilled on or in the vicinity of the dam from two previous investigations: one performed by ATC Associates in 2002 (which included seven borings); and the second was performed by Harding Lawson and Associates in 1982, and included seven borings.

Using the collective data set, stability analyses were performed by AECOM to evaluate the potential for slope instabilities, in accordance with the EPA regulation 40 CFR 257.73(d) and (e). The potential for slope instability is dependent on factors such as slope geometry, piezometer/phreatic surface conditions, seismic activity, and soil shear strengths of the embankment and foundation soils. A summary of the geotechnical field program, laboratory testing program and stability evaluations are presented in the following sections.

1.2 Brief Description of Impoundment

The Brown station is a coal-fired power plant located approximately 10 miles east of Mount Vernon in Posey County, Indiana and is owned and operated by SIGECO. The station is situated just west of the Vanderburgh-Posey County line and north of the Ohio River with the Ash Pond positioned on the east side of the generating station.

The Brown Ash Pond was commissioned in 1978. An earthen dam was constructed across an existing valley to create the impoundment. In 2003, a second dam was constructed east of the original dam and further up the valley to increase the storage capacity. This temporarily created an upper pond and a lower pond. The upper and lower ponds were operated separately until 2016 when the upper dam was decommissioned. A 10’ wide breach was installed in the upper embankment and the normal pool elevation was lowered. Currently, the upper pool and the lower pool act as one CCR unit referred to as the Ash Pond, which has a surface area of approximately 159 acres.

The Ash Pond dam embankment is approximately 1,540 feet long, 30 feet high, and has 3 to 1 (horizontal to vertical) side slopes covered with grassy vegetation. The embankment crest elevation is 450.9 feet[1] and has a crest width of 20 feet. An earthen buttress was constructed against the outboard slope of the dam. The buttress crest extends the length of the dam, is up to 200 feet wide and varies in elevation from 442.0 feet to 432.0 feet. The operating elevation of the pool fluctuates from 439.0 feet to 444.0 feet. However, the pool normally operates at an elevation of 441.5 feet. The surface area of the lower pool impoundment is approximately 57 acres. The surface area of the upper pool impoundment is approximately 102 acres and has a normal operating level of 450 feet. A Site Location Map showing the area surrounding the station is included as **Figure 1** of **Appendix A**. **Figure 2** in **Appendix A** presents the Brown Site Map.

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[1] Unless otherwise noted, all elevations in this report are in the NAVD88 datum.

2 Summary of Field Investigations

Subsurface explorations were performed at the Brown Ash Pond dam in 2015 and 2016, and included 25 soil borings, and a program of 5, cone-penetration test (CPT) soundings, with seismic wave velocity measurements and pore pressure dissipation testing. Boring depths ranged from 26 to 94 ft, and CPT depths ranged from 54 to 94 ft below existing grades. Boring and CPT locations are depicted in **Figure 3 (Appendix A).** Boring and CPT exploration location data (ID, easting, northing, and ground surface elevation) are summarized in **Table 2-1**. Boring logs are provided in **Appendix B** and CPT data plots are provided in **Appendix C**.

All borings were drilled by Cardno ATC of Indianapolis, Indiana, who was subcontracted directly to SIGECO. Borings B-201 through B-219 were drilled between April 15 and July 16, 2015. Borings AECOM-B-1 through AECOM B-5, and CPT-1 through CPT-5 were performed between October 8 and October 12, 2015. Boring AECOM B-8 was advanced on January 27, 2016. A Cardno ATC representative logged borings B-201 through B-219. An AECOM geotechnical engineer logged borings AECOM-B1 through AECOM-B5 and AECOM-B8. Cardno ATC used an All-Terrain Vehicle-mounted drill rig (GeoProbe 8040DT) and hollow stem augers (3.25-inch inner diameter) to drill the borings.

CPT soundings were performed by Cardno ATC, with full-time oversight by an AECOM geotechnical engineer. The soundings were performed by Cardno using a GeoProbe 8040DT rig equipped to advance CPT tooling and instrumentation with real-time data collection. The SCPTu soundings were completed in accordance with ASTM D5778 and provided nearly continuous digital logging of tip and sleeve resistance and generated pore pressure with depth. Shear wave measurements were taken during soundings at two-meter intervals in order to provide a shear wave velocity profile for the subsurface materials to support seismic site response analyses. Pore pressure dissipation tests were conducted at selected locations in each sounding.

Historical geotechnical investigations performed by Harding Lawson Associates in 1982 (Boring 1 through Boring 7, located at the northern area of the dam) were also considered in the interpretation and analysis of the site's geologic conditions.

Additional borings performed in the area of the former upper dam were also reviewed and considered herein. These borings were utilized only to establish a general characterization of the impounded sluiced ash within the Ash Pond and do not directly influence the stability evaluation performed herein. Location maps, logs, and lab testing data associated with these borings are provided in **Appendix D**.

Representative soil samples were collected from each of the borings for classification and/or testing. The soil samples were obtained using split spoon samplers and in accordance with the Standard Penetration Test (SPT) methodology (ASTM D 1586). Undisturbed samples of fine-grained soils (silts and clays) were obtained using 3-inch outside diameter steel (Shelby) tubes, either conventionally pushed in accordance with ASTM D1587 or by utilizing a piston sampler in accordance with ASTM D6519 (in very soft soils). Selected SPT and Shelby tube soil samples were tested at the GeoTesting Express Laboratory in Acton, Massachusetts or by Cardno ATC. Laboratory testing associated with seismic strength characterization was performed at the GeoTesting Express Laboratory.

Table 2-1 – Boring and CPT Exploration Location Data

Exploration ID	Firm and Date	Easting (ft, NAD83)	Northing (ft, NAD83)	Elevation (ft, NAVD88)
Borings				
AECOM-B1	AECOM (2015)	Adjacent to B-201		451.3
AECOM-B2	AECOM (2015)	Adjacent to B-210		451.2
AECOM-B3	AECOM (2015)	Adjacent to B-219		417.9
AECOM-B4	AECOM (2015)	Adjacent to B-205		416.1
AECOM-B5	AECOM (2015)	Adjacent to B-215		416.4
AECOM-B8	AECOM (2016)	2770903.02	968016.65	427.7
B-201	Cardno ATC (2015)	2771353.5	967075.1	450.9
B-202	Cardno ATC (2015)	2771274.5	967334.1	450.7
B-203	Cardno ATC (2015)	2771191.0	967637.5	450.8
B-204	Cardno ATC (2015)	2771106.2	967924.7	450.8
B-205	Cardno ATC (2015)	2771053.2	967603.2	415.6
B-206	Cardno ATC (2015)	2771114.7	967362.0	414.8
B-207	Cardno ATC (2015)	2770917.0	967453.0	395.0
B-208	Cardno ATC (2015)	2770911.3	967590.7	396.7
B-209	Cardno ATC (2015)	2771087.7	967991.4	450.9
B-210	Cardno ATC (2015)	2771131.0	967838.5	450.9
B-211	Cardno ATC (2015)	2771162.2	967727.2	451.1
B-212	Cardno ATC (2015)	2771214.9	967535.1	450.2
B-213	Cardno ATC (2015)	2771306.0	967234.0	451.0
B-214	Cardno ATC (2015)	2771330.8	967147.5	451.0
B-215	Cardno ATC (2015)	2771017.3	967805.7	416.1
B-216	Cardno ATC (2015)	2771057.7	967701.0	416.5
B-217	Cardno ATC (2015)	2771095.3	967516.0	416.3
B-218	Cardno ATC (2015)	2771166.6	967245.4	416.1
B-219	Cardno ATC (2015)	2771199.9	967126.1	417.6
HLA-1	Harding Lawson and Associates (1982)	**	**	**

Table 2-1 – Boring and CPT Exploration Location Data

Exploration ID	Firm and Date	Easting (ft, NAD83)	Northing (ft, NAD83)	Elevation (ft, NAVD88)
HLA-2	Harding Lawson and Associates (1982)	**	**	**
HLA-3	Harding Lawson and Associates (1982)	**	**	**
HLA-4	Harding Lawson and Associates (1982)	**	**	**
HLA-5	Harding Lawson and Associates (1982)	**	**	**
HLA-6	Harding Lawson and Associates (1982)	**	**	**
HLA-6A	Harding Lawson and Associates (1982)	**	**	**
HLA-7	Harding Lawson and Associates (1982)	**	**	**
CPT Soundings				
CPT-1	AECOM (2015)	2771277.3	967331.9	451.4
CPT-2	AECOM (2015)	2771188.1	967638.0	450.9
CPT-3	AECOM (2015)	2771196.7	967136.2	417.4
CPT-4	AECOM (2015)	2771107.0	967358.4	414.8
CPT-5	AECOM (2015)	2771056.0	967606.9	415.8

** Survey coordinates for the historical borings were not available. Locations shown on Figure 3 have been estimated based on location maps provided in the historical data.

3 Summary of Site-Specific Subsurface Conditions

3.1 Site Stratigraphy

3.1.1 Regional Geologic Setting

The Brown station is situated on the western edge of the Boonville Hills Physiographic subdivision of the Southern Hills and Lowlands Region of Indiana. This region is underlain by Pennsylvanian bedrock of the Mcleansboro group (lower part), which is predominantly shale, sandstone and limestone with interbedded thin coal layers.

The Heusler Fault is located roughly 2½ miles northwest of the site The New Madrid Seismic Zone, located in southeastern Missouri, and the Wabash Valley Fault System in southwestern Indiana, are both capable of significant seismic accelerations in the region that could impact the site.

3.1.2 Site-Specific Stratigraphy

Six strata were encountered during the geotechnical investigations at the Ash Pond dam:

1) Impounded Ash Materials: No ash materials were present in the Ash Pond dam. Ash materials are impounded behind the dam, within the pond. Based on historical information, these materials are primarily bottom ash and fly ash, and are generally in a very loose to loose condition.

2) Embankment Fill Materials: Embankment Fill materials were encountered from the ground surface and extending to depths ranging from approximately 37 to 58 ft below ground surface (bgs) from the crest boring and 5.5 to 26.5 ft bgs from the bench borings. Embankment Fill materials were typically a mixture of lean clays (CL) and silty clays (CL-ML) with varying amounts of sand. Visual classifications were most often described as slightly moist to moist, reddish brown to brown, silty clay to sandy lean clay. Uncorrected field Standard Penetration Test (SPT) N-values in the embankment ranged widely between 3 and 50 blows per foot (bpf) with an average of 16 bpf, indicating a stiff to very stiff overall consistency. Plasticity indices from Atterberg limit testing ranged from 3 to 26 percent, with an average of 13 percent. Liquid limits ranged from 24 to 38 percent with an average value of 30 percent. CPT results indicated a Cone Tip Resistance ranging from 56.6 to 111.7 tons per square foot (tsf) with an average of 71.3 tsf. Cone Sleeve Resistance ranged from 1.8 tsf to 3.0 tsf with an average value of 2.3 tsf. Shear wave velocity results ranged from 670 to 878 ft per second (ft/sec) with an average of 815 ft/sec.

3) Foundation Silts: Natural, alluvial silt deposits were encountered in most borings drilled in the lower bench area and beyond the toe of the dam. Silts were not encountered at any of the borings drilled at the crest of the dam, indicating that the deposit grades out moving from west to east across the width of the dam and buttress structures. The deposits consisted of a moist to wet, brown to gray, very soft to very stiff silt (ML) with occasion traces of fine sand. Silts varied in thickness from approximately 2.0 ft to 27.5 ft. Uncorrected field SPT N-values ranged between 0 and 23 blows bpf with an average of 7 bpf, indicating a medium stiff consistency overall. The fines content of the silt layers (as indicated by material that passes through a No. 200 sieve) was often above 95%, Atterberg limits testing indicated about half of the samples to be non-plastic, with others exhibiting very low plasticity indices, often below 7 percent. CPT results within the Foundation Silts indicated Cone Tip Resistance values ranging from 23.9 to 50.3 tsf with an average of 34.0 and Cone Sleeve Resistance values ranged from 0.64 to 1.32 tsf with an average of 0.90 tsf. Shear wave velocity results ranged from 533 to 737 ft/sec with an average of 692 tsf.

4) Foundation Silty Clay: The silt horizons discussed above were interbedded within native lean clays that made up much of the foundation materials of the Ash Pond dam, especially at the eastern regions of the dam footprint and below the crest. These clays consisted primarily of moist to wet, light brown to gray, very soft to very stiff lean clays (CL) to silty clays (CL-ML) with varying amounts of sand. The thickness of the clays varied widely, becoming more interbedded with silt layers to the west towards the bench and downstream toe of the embankment. Uncorrected field SPT N-values ranged between 0 and 33 bpf with an average of 10 bpf, indicating a typically stiff consistency. CPT results exhibited Cone Tip Resistances ranging from 17.5 to 38.4 tsf with an average of 26.6 and Cone Sleeve Resistances ranged from 0.46 to 1.43 tsf with an average of 0.91 tsf. Shear wave velocity results ranged from 804 to 984 ft/sec with an average of 882 ft/sec.

5) Buttress Fill: The buttress fill was obtained from near-site borrow sources, and consists of fine-grained soils most typically classified as lean clay (CL). Plasticity indices of the fill material generally ranging from 6 to 14 percent, with an average of about 12 percent. To a much lesser extent, the buttress fill includes materials classified as silt (ML). The fill was placed and compacted in lifts, and density testing of each lift using nuclear methods was performed. The compaction specification was to achieve 95% of the Standard Proctor Maximum Dry Density.

6) Bedrock: Bedrock was encountered in most of the borings advanced at the site. Borings were terminated at the top of bedrock or after collecting a single split spoon sample in rock in all cases (rock was not cored). As revealed in these limited samples, bedrock primarily consisted of gray to brown weathered to severely weathered siltstone with instances of gray weathered shale and gray to brown weathered to severely weathered sandstone. **Table 3-1** summarizes the depth/elevation of the top of rock as encountered in the borings. In general, the bedrock was found at a shallower depth (and elevation) on the north end of the dam and was found at a depth greater from ground surface at the south end of the dam.

Table 3-1 – Summary of Bedrock Depth and Elevation

Boring No.	Depth at Top of Rock (ft bgs)	Elevation at Top of Rock (ft NAVD88)	Rock Type
AECOM-B2	77.5	373.7	Siltstone
B-202	94	356.7	Siltstone
B-203	91.5	359.0	Siltstone
B-204	74.5	376.0	Siltstone
B-205	61.5	354.0	Siltstone
B-206	79	335.8	Siltstone
B-207	45	350.0	Siltstone
B-208	44	352.7	Siltstone
B-209	69.5	381.5	Sandstone
B-214	69	382.0	Shale
B-215	52	363.0	Shale

Table 3-1 – Summary of Bedrock Depth and Elevation

Boring No.	Depth at Top of Rock (ft bgs)	Elevation at Top of Rock (ft NAVD88)	Rock Type
B-216	53.9	361.1	Siltstone
B-218	57.4	357.6	Siltstone
B-219	46.8	368.2	Sandstone/Shale
HLA-1	71	379.9	Siltstone
HLA-3	52.5	399.4	Siltstone
HLA-4	55	394.6	Siltstone
HLA-5	34	382.1	Siltstone
HLA-6	24	392.2	Siltstone
HLA-7	28	373.6	Siltstone

Logs of the borings and CPT soundings are included in **Appendices B and C**, respectively, and laboratory test results are included in **Appendix D**.

3.2 Groundwater Conditions

The presence of groundwater was noted on the boring logs at the time of drilling on the drilling tools. Standpipe piezometers were installed during the additional field exploration in boring location B-212 on the crest and B-217 on the mid-slope bench. Ongoing readings of these piezometers appear to indicate steady-state water levels had equilibrated near a depth of 25.8 ft (approximate elevation of 424 ft) at crest boring B-212 and a depth of 8.6 ft (approximately 406 ft) at the mid-slope bench boring B-217.

The 1982 work by Harding Lawson indicated groundwater elevations similar to those above in the northern area of the Ash Pond dam. Steady state water levels below the crest of the dam were near El. 420 ft and near the toe of the dam were near El. 410 ft at the time of that investigation. One piezometer, located approximately 200 ft beyond the dam toe, had a water level near El. 395 ft.

An existing sand blanket and perforated drainage pipe system alleviates pore water pressure along the upstream face of the dam as well as along the flat bench area below the existing gravity buttress. The elevation of the drainage blanket in the flat area is approximately 412 ft. The drainage blanket has substantially greater hydraulic conductivity than the surrounding soils, and is intended to intercept seepage through the dam embankment, convey it downstream of the toe, and lower the phreatic surface through the dam.

4 Summary of Laboratory Testing

4.1 Summary of Laboratory Testing Scope

The laboratory testing program performed for the Ash Pond dam was intended to obtain information on index properties and shear strength properties of the subsurface materials at the site. The laboratory testing program for characterization of the materials at the Ash Pond dam are summarized in **Table 4-1**.

Table 4-1 – Summary of Laboratory Testing Program for Ash Pond Dam

ASTM Designation	Test Type	Number of Tests			
		Total	Embankment	Foundation Clay	Foundation Silt
D2216	Moisture Content	417	198	128	94
D2937	Dry Unit Weight	42	20	14	12
D4318	Atterberg Limits	105	32	39	34
D422	Sieve/Hydrometer	54	17	22	20
D5084	Hydraulic Conductivity	6	1	1	4
D4767	Consolidated Undrained Triaxial (CIU)	27	5	12	10
D6528	Cyclic Direct Simple Shear	6	0	0	6

4.2 Summary of Laboratory Testing Results

A summary of laboratory test results for the embankment fill, foundation clay, and foundation silt at the Ash Pond dam are presented in **Tables 4-2, 4-3,** and **4-4**, respectively. Seismic laboratory test results of the foundation silts are summarized in **Table 4-5.** See **Appendix D** and boring logs in **Appendix B** for a complete list of laboratory test data and results.

4.2.1 Embankment Fill

Table 4-2 summarizes the results of static laboratory testing performed within the Embankment fill.

Table 4-2 – Summary of Lab Test Results: Embankment Fill		
LAB TEST	**Range**	**Average**
Index/General Properties:		
Moisture Content (%)	11.7 – 25.8	17.5
Atterberg Limits (%)		
Liquid Limit	24 – 38	31
Plastic Limit	12 – 27	18
Plasticity Index	1– 30	14
Particle Size Analysis (%)		
Percent Fines (passing No. 200 Sieve)	58.7 – 99.5	85.9
Moist Unit Weight (pcf)	120.4 – 137.4	129.9
Dry Unit Weight (pcf)	101.0 – 119.0	110.4
Strength Properties:	**Friction Angle ϕ (degrees)**	**Cohesion c (psf)**
Drained (Effective) Strength	30	50
Peak Undrained (Total) Strength	22	600

4.2.2 Foundation Silty Clay Soils

Table 4-3 summarizes the results of static laboratory testing performed within the foundation clays.

Table 4-3 – Summary of Lab Test Results: Foundation Silty Clay Soils		
LAB TEST	**Range**	**Average**
Index/General Properties:		
Moisture Content (%)	8.0 – 48.1	24.0
Atterberg Limits (%)		
Liquid Limit	21 – 75	33
Plastic Limit	13 – 27	19
Plasticity Index	4 – 48	14
Particle Size Analysis (%)		
Percent Fines (passing No. 200 Sieve)	43.6 – 99.6	83.8

Table 4-3 – Summary of Lab Test Results: Foundation Silty Clay Soils

LAB TEST	Range	Average
Index/General Properties:		
Moist Unit Weight (pcf)	112.0 – 132.1	123.5
Dry Unit Weight (pcf)	77.0 – 111.0	98.2
Strength Properties:	Friction Angle ϕ (degrees)	Cohesion c (psf)
Drained (Effective) Strength	31	80
Peak Undrained (Total) Strength	23	400

4.2.3 Foundation Silt Soils

Table 4-4 summarizes the results of static laboratory testing performed within the foundation silts.

Table 4-4 – Summary of Lab Test Results: Foundation Silt Soils

LAB TEST	Range	Average
Index/General Properties:		
Moisture Content (%)	18.1 – 54.3	30.0
*Atterberg Limits (%)**		
Liquid Limit	23 – 38	29
Plastic Limit	20 – 35	26
Plasticity Index	1 – 6	3
Particle Size Analysis (%)		
Percent Fines (passing No. 200 Sieve)	71.2 – 99.9	95.2
Moist Unit Weight (pcf)	106.4 – 128.6	120.8
Dry Unit Weight (pcf)	71 – 106.2	93.2
Strength Properties:	Friction Angle ϕ (degrees)	Cohesion c (psf)
Drained (Effective) Strength	33	0
Peak Undrained (Total) Strength	22	650

**Note: Of 32 samples subject to Atterberg limits testing, 17 were classified as “Non-Plastic.” Ranges and averages listed are from the16 samples that exhibited plasticity.*

Stress-controlled, Cyclic Direct Simple Shear (CDSS) testing (per ASTM D6528) was performed on undisturbed silt samples obtained from multiple locations within silt zones beneath the Ash Pond dam. A total of six samples were tested. Samples were loaded to normal stresses at or slightly above the existing overburden pressure estimate for that sample.

Laboratory data from the CDSS tested are presented in **Appendix D.** The test results (including excess pore pressure generated and axial strain) are presented as a function of the number of cycles that have been applied at any point in the test. Herein, failure (i.e., liquefaction) was interpreted at the cycle where the single-phase axial strain exceeded 5% (or 10% peal-to-peak) or the excess pore pressure ratio reached 85% of the applied normal stress, whichever was less.

The results of CDSS testing are summarized in **Table 4-5** below.

Table 4-5 – Summary of Lab Test Results: CDSS Testing of Foundation Silts

Boring No.	Depth (ft)	CSR	Vertical Consolidation Stress (psf)	Number of Load Cycles To Failure	Failure Mechanism
AECOM-B1	39-41	0.25[1]	4,275	4	Strain Criteria
AECOM-B2	56-58	0.15	4,950	17	Excess Pressure Criteria
	62-64	0.20	6,040	3	Strain Criteria
AECOM-B4	33-35	0.08	2,965	>50	Sample did not liquefy
	46-48	0.20	3,380	6	Excess Pressure Criteria
AECOM-B5	30-32	0.15	2,660	20	Excess Pressure Criteria

5 Slope Stability Analyses

Slope stability analyses were performed for varying loading conditions at selected cross-sections, as described in the following sub-sections. Analysis section development, soil material properties, and seismic analyses related to the slope stability analysis are also discussed in the following sub-sections.

5.1 Cross-Sections for Analysis

Five cross-sections were identified for the stability evaluation of the Ash Pond dam. The analysis sections were selected based on factors including the height and steepness of the downstream embankment slope and subsurface conditions in the foundation of the embankment as revealed by the borings. Taken together, the five analysis sections are considered to comprehensively represent the Ash Pond dam. Descriptions of each analysis cross-section are given below and the locations of the sections are shown on **Figure 3 (Appendix A).**

- **Cross-Section A:** This section was analyzed based on stratigraphy from borings B-210 with offset boring AECOM-B2) at the crest and B-215 (with offset boring AECOM-B5) on the bench.
- **Cross-Section B:** This section was analyzed based on stratigraphy from borings B-203 (with offset CPT sounding AECOM-C2) at the crest, B-205 (along with offsets AECOM-B4 and -C5) on the bench, and B-208 at the toe. The Foundation Silt layer featured most prominently within this cross-section. Additionally, this cross-section models the tallest height (vertical difference between crest of the embankment and the toe of the embankment fill) of the dam embankment.
- **Cross-Section C:** This section was analyzed based on stratigraphy from borings B-202 (with offset CPT sounding AECOM-C1) at the crest, B-206 (with offset CPT sounding AECOM-C4) on the bench, and B-207 at the toe. Additional borings in the vicinity of this cross-section (including B-217 and B-218), were also reviewed to assess continuity of various interbedded silt layers. The embankment is relatively tall at this section, similar to Section B.
- **Cross-Section D:** This section is representative of the southern end of the dam. The section southernmost was analyzed based on stratigraphy from borings B-201 (with offset boring AECOM-B1) at the crest and B-219 (along with offsets AECOM-B3 and -C3) on the bench.
- **Cross-Section E:** This section is representative of the northern end of the dam, where bedrock rises sharply in elevation and the groundwater level at and beyond the toe of the dam is higher than at other areas. The cross-section was analyzed based on stratigraphy from borings B-208 and B-209 at the crest and AECOM-B8 at the toe.

The topography for each analysis cross-section was determined based on specific ground surveys performed to support this project (for Cross-Section A thru D) or from the aerial basemapping shown on **Figure 3** of **Appendix A** (for Section E)**.** Stratigraphy was established from the subsurface information indicated by the borings and CPT soundings. The relevant CPT soundings and test borings that were used to develop subsurface stratigraphy at the five analysis sections are shown on the geologic sections shown in **Figure 3 (Appendix A)**.

5.2 Stability Analysis Conditions Considered

Consistent with the criteria provided in §257.73(e), the stability of the Ash Pond dam was evaluated for the following four load cases.

5.2.1 Static, Steady-State, Normal Pool Condition

This case models the embankment and connected buttress under static, long-term conditions, at normal water level within the impoundment. The CCR Rule requires a maximum storage pool factor of safety greater than or equal to 1.50.

5.2.2 Static, Maximum Surcharge Pool Condition

This case models the conditions under short-term surcharge pool conditions, with the water level in the pond corresponding to the anticipated level during the design flood condition (which is a 1,000 year recurrence interval flood event for this site). This condition requires a minimum Factor of Safety greater than or equal to 1.40.

5.2.3 Seismic Slope Stability Analysis

These analyses incorporate a horizontal seismic coefficient k_h selected to be representative of expected loading during the design earthquake event (i.e., a “pseudostatic” analysis). The design earthquake event is one with a 2% probability of exceedance in 50 years (approximately 2,500 year recurrence interval), as required by the CCR Rule. The seismic coefficient was selected on the basis of the results of the site-specific, Probabilistic Seismic Hazard Analysis (PSHA) and dynamic response analysis. The analyses utilized peak undrained strength parameters for soils that are not considered to be rapidly draining materials (including the dam embankment and buttress soils, silty clay foundation stratum, and silt foundation stratum). The phreatic surface and pore water pressures corresponding to the steady state pool from the static analyses were utilized. This condition requires a minimum Factor of Safety greater than or equal to 1.00.

5.2.4 Post-Liquefaction Condition

These analyses were performed at each stability cross-section where liquefaction triggering analysis indicates potential liquefaction of non-plastic materials or cyclic softening of fine-grained soils. The purpose of the post-liquefaction stability analysis is to assess stability conditions immediately following the design seismic event. No horizontal seismic coefficient is included in these analyses, but selection of strength parameters for the analyses takes into account the potential for the softening/weakening of the soils as a result of pore pressures generated in sand-like materials, or cyclic softening in clay-like materials due to the earthquake shaking. Liquefaction potential analysis was performed on the foundation silt deposits, using cyclic stress ratios (CSRs) determined from finite element dynamic response analysis, and cyclic resistance ratios (CRRs) determined from the results of cyclic direct simple shear testing. The liquefaction potential analysis is presented in **Appendix I**.

The CCR Rule requires a minimum Factor of Safety greater than or equal to 1.20 for the post-liquefaction slope stability analysis.

5.2.5 Sudden Drawdown of Adjacent Water Bodies

The Ash Pond dam is not adjacent to any external water bodies. Therefore, analysis of a sudden drawdown condition is not applicable.

5.3 Material Properties

Material properties for slope stability analyses were developed using both laboratory testing data (index and strength testing) and strength correlations from CPT and SPT data. Material strength parameter characterization used in the slope stability analyses for each of the pertinent strata are provided in **Table 5-1**. A detailed presentation of the calculations and interpretations related to the strength characterization is provided in **Appendix E**. Application of the material properties in the table to the specific stability analysis loading conditions is discussed in **Section 5.4**.

Table 5-1 – Material Properties For Slope Stability Analyses

Material	Unit Weight (pcf)	Effective (drained) Shear Strength Parameters		Total (undrained) Shear Strength Parameters		Post-Earthquake Shear Strength Parameters		
		c' (psf)	Φ' (°)	c (psf)	Φ (°)	c (psf)	Φ (°)	S_{ur} / σ'_{vc}
Embankment Fill	128	50	30	600	22	475	18	-
Foundation Silt	119	0	33	650	22	-	-	0.10
Foundation Clay	126	80	31	400	23	320	19	
Buttress Fill	123	45	27	540	20	425	16	-
Sluiced Ash	100	0	32	100	12	-	-	0.12
Bedrock	Assumed to be impenetrable in the slope stability models							

Peak effective and undrained strengths were selected based on interpretation of triaxial test data in accordance with the Modified Mohr-Coulomb plot (a p-q and p'-q plot) procedures, as described in Appendix D of the United States Corps of Engineers Manual EM-1110-2-1902 "Slope Stability." In analyzing the test results, a number of definitions of failure were considered, including the point of peak deviator stress during the test, the deviator stress corresponding to an axial strain of 12% and 15%, and the point of the test with the maximum effective principle stress ratio (obliquity) from the tabulated CU test data. For both effective and total strength conditions, defining the failure point to coincide with the deviator stress corresponding to 15% strain was selected to establish the shear strength parameters. P-Q plots are provided in **Appendix E**.

Liquefaction of the foundation silt deposit is predicted under the design earthquake. Steady-state strength was therefore estimated for use in the post-liquefaction stability analysis. The steady state strength was determined based on the empirical, SPT and CPT-based procedures given in "Soil Liquefaction During Earthquakes" by Idriss and Boulanger (2008), as presented in detail in **Appendix E.**

The embankment fill, buttress fill, and silty clay foundation soils are generally stiff to very stiff fine-grained materials. Static laboratory strength test results do not indicate significant post-peak softening in these materials, which indicates low susceptibility to cyclic softening. However as a conservative interpretation, the strength of these soils was reduced for the post-liquefaction stability analyses. Specifically, the strength used for this condition corresponded to 80% of the peak undrained shear strength of the materials.

For impounded Coal Ash materials, strength properties were selected based on past experience and conservative engineering judgment. Furthermore, liquefaction was conservatively assumed by inspection, and steady-state strengths were also assigned based on conservative engineering judgment. It is noted that the impounded ash has little to no influence in the stability analyses.

Unit weight of the buttress fill was established based on review of the field compaction test data generated during its construction. The unit weight assigned in the models was the average of all tests performed. Strength testing of the buttress materials was not performed. The buttress fills are similar to the embankment fill materials in consistency and index properties and were placed and compacted using modern construction techniques. Strength of the buttress fill is therefore anticipated to be similar to the embankment. As a conservative assumption, strength parameters assigned to the buttress are approximately 90% of the strength of the embankment materials.

5.4 Methodology of Analyses

Limit equilibrium stability analysis was completed using the two-dimensional Slope/W computer program by Geo-Slope International. Factors of safety were calculated using Spencer’s method and using iterative analyses of both circular and block failure surfaces to determine the critical failure surface for each analysis section and load case. Shallow finite slope failure surfaces or failure surfaces occurring at a depth less than 10 ft were not analyzed as they correspond to sloughing failure which can be addressed as part of regular maintenance. Critical surfaces with respect to dam safety were considered to be those which intersected the dam crest at or upstream of the centerline, which are considered to have the potential to create an immediate threat to dam safety. Pore pressures were assigned as hydrostatic pressure under the phreatic surface.

The earthen buttress that is present against the downstream slope is intended to stabilize the dam against earthquake-induced accelerations and liquefaction. The buttress works by gravity, adding stabilizing forces to the dam, which offset the effects of earthquake loading. A similar stabilizing effect is imparted under static conditions as well. The buttress and its effects on the dam are included in all the slope stability models.

A summary of the analyses is presented in the following sections. A more detailed discussion is provided in **Appendix F**.

5.4.1 Static Analysis Conditions

5.4.1.1 Pool Elevations

The static analysis conditions include the steady-state normal pool and maximum surcharge pool loading conditions. Static stability was evaluated for steady-state conditions using a maximum normal pool elevation of 444.0 ft, and a maximum pool surcharge elevation of 446.8 ft. The latter elevation corresponds to the anticipated water level in the pond during the IDF event, as identified in AECOM’s *CCR Certification: Initial Inflow Design Flood Control System Plan* (October 2016).

5.4.1.2 Phreatic Surface

The phreatic surface used in the steady-state normal pool condition was established using the water levels in the piezometers installed near the centerline of the dam. Depths and elevations of free water as indicated in the borings and observations of water flow in the streams and ditches that lie to the west of the dam were also used to compare against the piezometer data for sections located away from the centerline (especially to estimate groundwater elevations in the far field beyond the toe of the dam). The water elevations were drawn into the stability models with straight line interpolation between the pool elevation and piezometer locations. AECOM reviewed the water elevations and cross-checked the interpolated phreatic surface with finite element seepage analysis using GeoStudio's SEEP/W software. Phreatic surfaces calculated in SEEP/W were in reasonable agreement with the straight-line interpolations from the available field groundwater measurements, but generally resulted in a lower phreatic level than the field measurements. Therefore, the straight-line interpolation was conservatively selected for the slope stability models.

For the maximum surcharge pool condition, the pool level in the pond was raised to the design flood level. The straight-line interpolation described above was adjusted accordingly to the raised water level. Therefore, the phreatic surface used for this loading condition corresponds to steady-state seepage to the raised pool level. This is a conservative representation, as the maximum storage pool water level is likely to be a short-term event and steady state seepage conditions through the dam are unlikely to develop.

5.4.1.3 Shear Strength Parameters

For the steady-state normal pool condition, drained (effective stress) shear strength parameters were used for all materials.

The change in water level from the normal pool case to the maximum surcharge pool condition is relatively small (less than 3 vertical ft). The small forcing effect created by this change is not expected to generate an undrained stress condition in the dam or its foundation. Therefore, drained (effective stress) shear strength parameters were used for all materials in the maximum surcharge pool condition as well.

5.4.2 Earthquake Analysis Conditions

A site specific seismic hazard assessment (PSHA) was performed to identify the earthquake loads at the site, and dynamic response analysis was performed to determine the appropriate seismic loads and material properties for the earthquake stability analysis load cases. Liquefaction triggering analyses were completed to assess the potential for liquefaction or cyclic softening of the materials and determine the appropriate material properties for use in the seismic and post-liquefaction slope stability loading conditions.

5.4.2.1 Probabilistic Seismic Hazard Analysis

The PSHA was completed for the Brown station to develop 2,500-year earthquake ground motions for use in liquefaction and dynamic response analyses of the facility. The PSHA results were used to compute a 2,500-yr return period Uniform Hazard Spectrum (UHS) for both hard rock (Class A rock, with shear wave velocity greater than 9,200 ft/s) and firm rock (Class B rock, with shear wave velocity between 2,500 and 9,200 ft/s). Parameters were developed including magnitude, distance, style of faulting, response spectra, and Arias Intensity. All seismically capable fault systems in the project region were considered, including the Illinois Basin Extended Basin Zone, New Madrid Seismic Zone which lies to the west and the Wabash Valley Seismic Zone.

Table 5-2 summarizes the UHS computed from the PSHA for the top of firm rock at the site, and **Table 5-3** summarizes modal magnitude and source distance which represent the highest contributor to the hazard for the design return period.

Table 5-2 – Uniform Hazard Response Spectrum For Firm Rock

Period	Spectral Acceleration (g)
0.01	0.53
0.02	0.96
0.03	1.16
0.04	1.21
0.10	1.02
0.20	0.68
0.40	0.40
1.0	0.14
2.0	0.07
3.0	0.041
4.0	0.028

Table 5-3 – Modal Earthquake Magnitude and Source Distance

Period	Modal Magnitude (M*)	Modal Source Distance (D*)
PGA	5.1	12.5 km
0.4 (bimodal)	7.1 7.6	12.5 km 238 km
1.0	7.6	238 km

Four sets of time histories were developed for each design spectrum. The time histories represent the site-specific ground motions associated with the controlling near-field or far-field earthquake event, and consider the magnitude, distance, and Arias Intensity. The site-specific acceleration time histories were then used in two-dimensional dynamic response analysis (see section below) to estimate site-specific seismic loads for liquefaction triggering and seismic (pseudo-static) stability analysis.

Details of the PSHA are included in **Appendix G**.

5.4.2.2 Dynamic Response Analysis

The dynamic response of the Ash Pond embankment was evaluated by analyzing Cross-Section B using the most recent version of the finite element program QUAD4M (Hudson et al. 1994). This is a modified version of the program QUAD4, originally developed by Idriss, et al. (1973). The dynamic response analysis was useful for more precisely estimating the amplification / attenuation characteristics of the dam structure and local foundation soils to the design ground motions at the top of firm rock and to estimate site-specific PGA values at the embankment crest for use in liquefaction triggering and seismic (pseudo-static) slope stability analysis. In addition, the dynamic response analysis was used to estimate the cyclic stress ratios (CSR) induced by the earthquake loading. Input to the dynamic response analyses includes the acceleration time histories developed as part of the PSHA for the station.

The QUAD4M program uses a two-dimensional, dynamic finite-element formulation that utilizes equivalent-linear, strain-dependent modulus and damping properties. The program performs a time-domain analysis that allows variable damping throughout the model, and uses an iterative process to approximate the nonlinear behavior of soil. Shear moduli and damping ratios are estimated initially for each element in the model, and the system is analyzed using those properties. After each iteration, values of the effective shear strain are computed and the modulus and damping values are updated to correspond to the computed strain level for each element. The analysis iterations are repeated until compatibility between moduli, damping, and strain levels is achieved in all elements.

Based on the dynamic response analyses at Section B, the calculated site-specific PGA values for a 2,500-year event were approximately 0.53g at the embankment crest, and CSRs in the foundation silt deposit ranged from 0.11 to 0.27. These values were used to define the earthquake loading for the liquefaction triggering analysis and pseudostatic stability analysis for all five analysis cross-sections.

Details of the dynamic response analysis are included in **Appendix H**.

5.4.2.3 Seismic Coefficient

The seismic coefficient, k_h, was calculated for use in the seismic loading condition slope stability analysis based on the simplified procedure developed by Makdisi and Seed (1978) and using the site-specific acceleration at the crest of the dam from the dynamic response analysis. For the site-specific value of PGA at the embankment crest of 0.53g and the full-height critical slip surfaces that were identified in the stability analysis (presented in **Appendix F**), a seismic coefficient of 0.18g was used in the pseudo-static analysis.

5.4.2.4 Liquefaction Triggering Analysis

Liquefaction triggering analysis was used to evaluate the potential for liquefaction of the foundation silt deposit under the 2,500-year event. Liquefaction triggering evaluations were performed using two methods:

1. An empirical SPT-based Procedure

2. A laboratory-based procedure, in which the cyclic resistance is established on the basis of laboratory cyclic direct simple shear testing.

The SPT- based liquefaction triggering analyses were performed using the procedure proposed by Idriss and Boulanger (2008, 2014). The procedure considers a stress-based approach to evaluate the potential for liquefaction triggering, and compares calculated earthquake-induced cyclic stress ratios (CSRs) with the estimated cyclic resistance ratios (CRRs) of the soil to establish the factor of safety against liquefaction triggering. CSRs used as input to this analysis were based on the results of the site-specific dynamic response analyses. Within the method, CRRs are a function of the soil's fines content (FC), relative density and effective stress, and penetration resistance (SPT). The CRR is also dependent on the duration of shaking, and is adjusted to the site-specific design earthquake using a Magnitude Scaling Factor (MSF). Fines content, density, and other material parameters used as input to the analysis were based on the laboratory test data obtained as part of this project. The magnitude of the design earthquake was input as M 7.1, based on the modal results from the site-specific PSHA.

In the laboratory-based procedure, the calculated cyclic stress ratios (CSRs) from the dynamic response analysis were compared to cyclic resistance ratios (CRRs), established from interpretation of the cyclic direct simple shear testing performed on representative silt samples.

In both procedures, the ratio of CRR to CSR is the triggering factor of safety. For calculated triggering factors of safety less than 1.20, the material was considered to be potentially liquefiable.

Details of the liquefaction triggering analysis are provided in **Appendix I.**

5.4.2.5 Pool Elevations and Phreatic Surface

Pool elevation in the pond and the phreatic surface for both the seismic and post-liquefaction loading conditions were the same as utilized in the steady-state normal pool loading condition.

5.4.2.6 Shear Strength Parameters

All soil strata at the site are considered to be fine-grained materials which are not expected to rapidly drain as a result of seismic shaking. Therefore, peak undrained strength parameters (as summarized in **Table 5-1**) were utilized in the slope stability analyses of the seismic loading condition. As this condition incorporates a horizontal seismic coefficient, liquefied strengths are not pertinent to the analysis and were not utilized.

The post-liquefaction loading case represents conditions following the design earthquake, and no horizontal seismic coefficient is incorporated. As described in **Section 6.2.1** below and further presented in **Appendix I,** liquefaction of the foundation silt deposit is predicted as a result of the design earthquake. Therefore, steady-state (liquefied) strength was assigned to this stratum in the slope stability analysis of the post-liquefaction loading condition. The steady-state strength was estimated based on correlations with SPT and CPT-resistance and methodologies presented in Idriss and Boulanger (2008, 2014), as described in **Appendix E**. The resulting strength is presented in **Table 5-1**.

Liquefaction of the sluiced ash impounded by the dam has been assumed by inspection herein. Steady-state strength of this deposit (as given in **Table 5-1**) was therefore also assumed in the post-liquefaction loading condition analysis.

The embankment fill, buttress fill, and silty clay foundation soils are generally stiff to very stiff fine-grained materials. Static laboratory strength test results do not indicate significant post-peak softening in these materials, which indicates low susceptibility to cyclic softening. However as a conservative interpretation, the strength of these soils was reduced for the post-liquefaction stability analyses. Specifically, the strength used for this condition corresponded to 80% of the peak undrained shear strength of the materials, as established through laboratory testing.

6 Results

Regulatory Citation: 40 CFR §257.73 (e); Periodic safety factor assessments. (1) The owner or operator must conduct an initial and periodic safety factor assessments for each CCR unit and document whether the calculated factors of safety for each CCR unit achieve the minim safety factors specified in paragraphs (e)(1)(i) through (iv) of this section for the critical cross-section of the embankment..

6.1 Results of Static Stability Analyses

Regulatory Citation: 40 CFR §257.73 (e)(1);

- *(i) The calculated static factor of safety under the long-term, maximum storage pool loading condition must equal or exceed 1.50.*
- *(ii) The calculated static factor of safety under the maximum surcharge pool loading condition must equal or exceed 1.40.*

The results of the limit equilibrium slope stability analyses for the static load cases are summarized in **Table 6-1**. The Slope/W output figures showing the critical slip surfaces and details of the analyses are included in **Appendix F**.

Table 6-1 – Summary of Minimum Slope Stability Factors of Safety for Static Load Cases

Load Case	Criteria	Cross-Section A	Cross-Section B	Cross-Section C	Cross-Section D	Cross-Section E
Steady State (Normal Pool)	FS ≥ 1.50	3.43	3.42	3.21	3.32	3.36
Max Surcharge Pool (Flood Pool)	FS ≥ 1.40	3.33	3.32	3.06	3.22	3.36

The calculated factors of safety at all analysis sections are greater than the minimum values required in §257.73 (e)(i) and (ii), thereby satisfying the regulatory requirement.

6.2 Results of Earthquake Stability Analyses

Regulatory Citation: 40 CFR §257.73 (e)(1);

- *(iii) The calculated seismic factor of safety must equal or exceed 1.00.*
- *(iv) For dikes constructed of soils that have susceptibility to liquefaction, the calculated liquefaction factor of safety must equal or exceed 1.20.*

6.2.1 Liquefaction Triggering Analysis

The liquefaction triggering analyses using the SPT-based procedure results in factors of safety against liquefaction in the silt deposit that are consistently below 1.20 (with a majority of the results being less than 1.0). Furthermore, the laboratory-based analysis procedure predicts that liquefaction of the silt deposit will occur in seven to nine cycles of the equivalent reference loading corresponding to the design earthquake. For the M 7.1 design earthquake being considered herein, the estimated cycles of equivalent loading is approximately 12. These results are presented in detail in **Appendix I**.

The results of both triggering analysis procedures are consistent and indicate that liquefaction of the silt deposit is likely as a result of the design earthquake event. As a result of this conclusion, steady-state (liquefied) strength was assigned to this stratum in the slope stability analysis of the post-liquefaction loading condition.

6.2.2 Slope Stability Analysis

The results of the slope stability analyses for the seismic load cases are summarized in **Table 6-2**. The Slope/W output figures showing the critical slip surfaces and details of the analyses are included in **Appendix F**.

Table 6-2 – Summary of Minimum Slope Stability Factors of Safety for Earthquake Load Cases

Load Case	Program Criteria	Cross-Section A	Cross-Section B	Cross-Section C	Cross-Section D	Cross-Section E
Seismic (Pseudostatic)	FS ≥ 1.00	1.51	1.56	1.32	1.49	1.56
Post-Liquefaction	FS ≥ 1.20	1.23	1.25	1.32	1.25	1.32

The calculated factors of safety at all analysis sections are greater than the minimum values required in §257.73 (e)(iii) and (iv), satisfying the regulatory requirement.

6.3 Critical Cross-Sections

CCR Rule §257.73 (e) requires identification of a critical cross-section to represent the impoundment. As presented herein, five cross-sections of the dam have been evaluated, to provide a thorough evaluation of the stratigraphic and topographic conditions across the structure. As such, the resulting factors of safety for each loading condition considered vary between cross-sections and certain sections are critical. Herein, the critical cross-section for any given load case has been interpreted as that section which has the lowest factor of safety for that particular load case. **Table 6-3** below summarizes the critical cross-section and corresponding factor of safety for each load case. The factors of safety presented in this table correspond to the values being certified in this document.

Table 6-3 – Summary of Critical Cross-Section and Factors of Safety For Stability Analysis Loading Conditions

Load Case	Critical Cross-Section	Minimum Factor of Safety
Steady State (Normal Pool)	Section C	3.21
Max Surcharge Pool (Flood Pool)	Section C	3.06
Seismic (Pseudostatic)	Section C	1.32
Post-Liquefaction	Section A	1.23

7 Conclusions

The calculated factors of safety from the limit equilibrium slope stability analysis satisfy the CCR Rule §257.73 (e) requirements for all the load cases analyzed at the critical analysis sections for the Brown Ash Pond dam embankment. Load cases analyzed for this study included static (steady-state) normal pool, maximum flood surcharge pool, seismic (pseudo-static), and static post-liquefaction.

8 Certification

This Certification Statement documents that the Ash Pond at the A.B. Brown Generating Station meets the Safety Factor Assessment requirements specified in 40 CFR §257.73 (e). The Ash Pond is an existing CCR surface impoundment as defined by 40 CFR §257.53. The CCR Rule requires that the Safety Factor Assessment for an existing CCR surface impoundment be prepared by October 17, 2016.

CCR Unit: Southern Indiana Gas & Electric Company; A.B. Brown Generating Station; Ash Pond

I, Vikram K. Gautam, being a Registered Professional Engineer in good standing in the State of Indiana, do hereby certify, to the best of my knowledge, information, and belief that the information contained in this certification has been prepared in accordance with the accepted practice of engineering. I certify, for the above referenced CCR Unit, that the Safety Factor Assessment dated October 13, 2016 meets the requirements of 40 CFR § 257.73 (e).

Vikram K. Gautam
Printed Name

10/13/16
Date

VIKRAM K. GAUTAM
No. PE11600291
10/13/16

9 Limitations

Background information, design basis, and other data have been furnished to AECOM by SIGECO. AECOM has used this data in preparing this report. AECOM has relied on this information as furnished, and is not responsible for the accuracy of this information. Our recommendations are based on available information from previous and current investigations. These recommendations may be updated as future investigations are performed.

Borings have been spaced as closely as economically feasible, but variations in soil properties between borings, that may become evident at a later date, are possible. The conclusions developed in this report are based on the assumption that the subsurface soil, rock, and groundwater conditions do not deviate appreciably from those encountered in the site-specific exploratory borings. If any variations or undesirable conditions are encountered in any future exploration, we should be notified so that additional analyses can be made, if necessary.

The conclusions presented in this report are intended only for the purpose, site location, and project indicated. The recommendations presented in this report should not be used for other projects or purposes. Conclusions or recommendations made from these data by others are their responsibility. The conclusions and recommendations are based on AECOM’s understanding of current plant operations, maintenance, stormwater handling, and ash handling procedures at the station, as provided by SIGECO. Changes in any of these operations or procedures may invalidate the findings in this report until AECOM has had the opportunity to review the findings, and revise the report if necessary.

This geotechnical investigation was performed in accordance with the standard of care commonly used as state-of-practice in our profession. Specifically, our services have been performed in accordance with accepted principles and practices of the geological and geotechnical engineering profession. The conclusions presented in this report are professional opinions based on the indicated project criteria and data available at the time this report was prepared. Our services were provided in a manner consistent with the level of care and skill ordinarily exercised by other professional consultants under similar circumstances. No other representation is intended.

10 References

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Appendix A
Figures

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AECOM DRAWING PATH: P:\Projects\Environmental\60442676_VectrenCCR\7.0_CAD_GIS\7.04_Plan_Sheets\BROWN--LOCATION MAP.dwg

AECOM

9400 Amberglen Boulevard
Austin, TX 78729-1100
512-454-4797 (phone)
512-454-8807 (fax)

SOUTHERN INDIANA GAS AND ELECTRIC COMPANY
dba VECTREN POWER SUPPLY, INC.

One Vectren Square
Evansville, IN 47708
1-800-227-1376 (phone)

A.B. BROWN
GENERATING STATION
MT. VERNON, IN

CCR CERTIFICATION
ASH POND

ISSUED FOR CERTIFICATION

ISSUED FOR BIDDING DATE BY

ISSUED FOR CONSTRUCTION DATE BY

REVISIONS

NO.	DESCRIPTION	DATE
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△		
△		
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AECOM PROJECT NO:	60442676
DRAWN BY:	MJC
DESIGNED BY:	MJC
CHECKED BY:	TLE
DATE CREATED:	
PLOT DATE:	4/22/2016
SCALE:	AS SHOWN
ACAD VER:	2014

SHEET TITLE

LOCATION MAP

FIGURE 1

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AECOM DRAWING PATH: P:\Projects\Environmental\60442676_VectrenCCR\7.0_CAD_GIS\7.04_Plan_Sheets\BROWN-SITE MAP.dwg

AECOM
9400 Amberglen Boulevard
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SOUTHERN INDIANA GAS AND ELECTRIC COMPANY dba VECTREN POWER SUPPLY, INC.
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A.B. BROWN
GENERATING STATION
MT. VERNON, IN

CCR CERTIFICATION
ASH POND

ISSUED FOR CERTIFICATION

ISSUED FOR BIDDING ____ DATE BY

ISSUED FOR CONSTRUCTION ____ DATE BY

REVISIONS

NO.	DESCRIPTION	DATE
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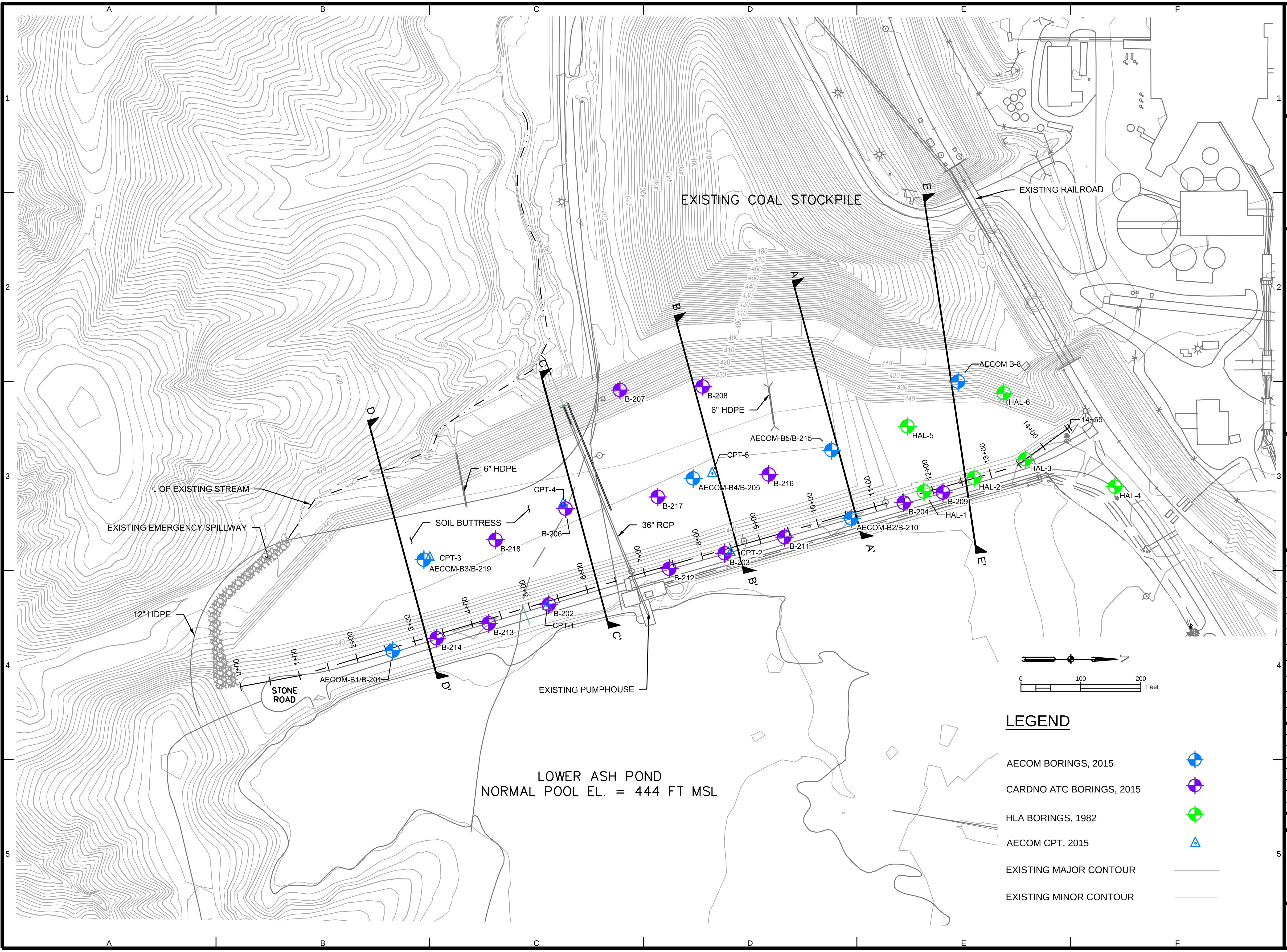
AECOM PROJECT NO:	60442676
DRAWN BY:	MJC
DESIGNED BY:	MJC
CHECKED BY:	TLE
DATE CREATED:	
PLOT DATE:	4/22/2016
SCALE:	AS SHOWN
ACAD VER:	2014

SHEET TITLE

SITE MAP

FIGURE 2



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AECOM DRAWING PATH: K:\Projects\V\Vectren Corporation\60442676_ABBrown\DWGs\C\GEOTECH\Figures\Boring Location Map-rev0.dwg

AECOM

1300 E. 9TH STREET
SUITE 500
CLEVELAND, OH
216-622-2300 (PHONE)

VECTREN
Live Smart

P.O.BOX 209
EVANSVILLE, IN 47702
1-800-227-1376

A.B. BROWN
GENERATING STATION
POSEY COUNTY, IN

ISSUED FOR BIDDING DATE BY

ISSUED FOR CONSTRUCTION DATE BY

REVISIONS

NO.	DESCRIPTION	DATE

AECOM PROJECT NO:	60442676
DRAWN BY:	ACI
DESIGNED BY:	ACI
CHECKED BY:	VKG
DATE CREATED:	9/9/2016
PLOT DATE:	9/9/2016
SCALE:	AS SHOWN
ACAD VER:	AUTOCAD CIVIL 3D 2016

SHEET TITLE

FIGURE 3 -
GEOTECHNICAL
CROSS-SECTION PLAN

Appendix B
Boring Logs

Project: A.B. Brown Ash Pond Lower Dam Evaluation

Project Location: Posey County, Indiana

Project Number: 60442676

Log of Boring AECOM-B1

Sheet 1 of 2

Date(s) Drilled	**10/09/2015 12:00 AM to 10/12/2015 12:00 AM**	Logged By	**M. Jones**	Checked By	**V. Gautam**
Drilling Method	**Hollow Stem Auger**	Drill Bit Size/Type	**3.25" I.D. HSA**	Borehole Depth	**51.0 ft**
Drill Rig Type	**GeoProbe 8040DT**	Drilling Contractor	**Cardno ATC**	Surface Elevation	**451.3 ft NAVD88**
Borehole Backfill	**Grout**	Sampling Method(s)	**18" Split Spoon 2" ID, 30" Shelby Tube 3" ID**	Hammer Data	**Auto-Hammer, 81% efficiency**
Boring Location	**Adjacent to B-201 (ft NAD83)**	Groundwater Level(s)	**37 ft on 4/17/2015**		

Elevation (feet)	Depth (feet)	SAMPLES Type Number	Sampling Resist. OR Core RQD (%)	Recovery (%)	MATERIAL DESCRIPTION	Natural Moisture Content (%)	Total Unit Weight (pcf)	Liquid Limit	Plasticity Index	Pocket Pen. Su (ksf)	Torvane Su (ksf)	TXUU (ksf)	REMARKS
451.3	0.0				Refer to Cardno ATC boring log B-201 for complete description and approximate depths of materials, stratigraphy, and groundwater levels. Blank drilling conducted between sample intervals.								
435.8	15.5	SS-1	5 4 6	100	Stiff, moist, orange to reddish brown with gray, silty, low plasticity, lean CLAY (CL) [FILL]								
		ST-1		42		18.0	126.0						
		ST-1A		92		17.0	131.0	33	16				*Passing No. 200 Sieve = 95.4%*
425.8	25.5	SS-2	2 4 6	100	Stiff, moist, reddish to orange brown with some gray, silty, low to moderate plasticity, lean CLAY (CL) [FILL]								
		ST-2		100		16.3	129.0	29	4				*Passing No. 200 Sieve = 97.9%*

Project: A.B. Brown Ash Pond Lower Dam Evaluation

Project Location: Posey County, Indiana

Project Number: 60442676

Log of Boring AECOM-B1

Sheet 2 of 2

Elevation (feet)	Depth (feet)	SAMPLES Type Number	Sampling Resist. OR Core RQD (%)	Recovery (%)	MATERIAL DESCRIPTION	Natural Moisture Content (%)	Total Unit Weight (pcf)	Liquid Limit	Plasticity Index	Pocket Pen. Su (ksf)	Torvane Su (ksf)	TXUU (ksf)	REMARKS
413.8	37.5				Very soft, wet, brown, SILT (ML) [ALLUVIUM]								
		SS-3	WH WH WH	100									
	40	ST-3		100		27.5		NP	NP				*Passing No. 200 Sieve = 99.6%*
408.8	42.5				Soft, wet, brown with occasional gray lenses, SILT (ML) [ALLUVIUM]								
		SS-4	1 2 1	100									
	45	ST-4		100		26.5	124.0						*Passing No. 200 Sieve = 99.6%*
403.8	47.5				Very soft, wet, gray, SILT (ML) [ALLUVIUM]								
		SS-5	WH WH WH	100									
	50	ST-5		100	-Material recovered in ST-5 appeared more clayey than SPT-5	26.8	123.0						
400.3	51.0				End of Boring at 51 ft								

Project: A.B. Brown Ash Pond Lower Dam Evaluation

Project Location: Posey County, Indiana

Project Number: 60442676

Log of Boring AECOM-B2

Sheet 1 of 3

Date(s) Drilled	10/12/2015 12:00 AM to 10/12/2015 12:00 AM	Logged By	M. Jones	Checked By	V. Gautam
Drilling Method	Hollow Stem Auger	Drill Bit Size/Type	3.25" I.D. HSA	Borehole Depth	77.7 ft
Drill Rig Type	GeoProbe 8040DT	Drilling Contractor	Cardno ATC	Surface Elevation	451.2 ft NAVD88
Borehole Backfill	Grout	Sampling Method(s)	18" Split Spoon 2" ID, 30" Shelby Tube 3" ID	Hammer Data	Auto-Hammer, 81% efficiency
Boring Location	Adjacent to B-210 (ft NAD83)	Groundwater Level(s)	54.5 ft on 7/1/2015		

Elevation (feet)	Depth (feet)	SAMPLES Type	Number	Sampling Resist. OR Core RQD (%)	Recovery (%)	Graphic Symbol	MATERIAL DESCRIPTION	Natural Moisture Content (%)	Total Unit Weight (pcf)	Liquid Limit	Plasticity Index	Pocket Pen. Su (ksf)	Torvane Su (ksf)	TXUU (ksf)	REMARKS
451.2	0.0						Refer to Cardno ATC boring log B-210 for complete description and depths of materials, stratigraphy, and groundwater levels. Blank drilling conducted between sample intervals.								
422.7	28.5	▨	1	5 9 12	100	^ ^ ^	Very Stiff, moist, brown and orange brown, silty, low plasticity, lean CLAY (CL) [FILL]								

Project: A.B. Brown Ash Pond Lower Dam Evaluation

Project Location: Posey County, Indiana

Project Number: 60442676

Log of Boring AECOM-B2

Sheet 2 of 3

Elevation (feet)	Depth (feet)	SAMPLES Type	Number	Sampling Resist. OR Core RQD (%)	Recovery (%)	MATERIAL DESCRIPTION	Natural Moisture Content (%)	Total Unit Weight (pcf)	Liquid Limit	Plasticity Index	Pocket Pen. Su (ksf)	Torvane Su (ksf)	TXUU (ksf)	REMARKS
	30	▤	1		92		17.9	127.0						*Passing No. 200 Sieve = 79.6%*
404.7	46.5	▨	2	6 7 10	100	Very Stiff, moist, brown, silty, low plasticity, lean CLAY (CL) [FILL]								
		▤	2		88		15.2	128.0						*Passing No. 200 Sieve = 63.7%*
396.7	54.5	▨	3	3 6 7	100	Stiff, moist, brown and orange brown with gray, clayey SILT (ML) [ALLUVIUM]								
		▤	3		92		25.0		NP	NP				*Passing No. 200 Sieve = 99.1%*
392.7	58.5	▨	4	4 4 5	100	Stiff, wet, gray and orange brown, SILT (ML) [ALLUVIUM]								
		▤	4		54		25.9	124.0						
		▤	4A		100		24.7							*Passing No. 200 Sieve = 99.7%*
	65													

Project: A.B. Brown Ash Pond Lower Dam Evaluation

Project Location: Posey County, Indiana

Project Number: 60442676

Log of Boring AECOM-B2

Sheet 3 of 3

Elevation (feet)	Depth (feet)	SAMPLES Type	Number	Sampling Resist. OR Core RQD (%)	Recovery (%)	Graphic Symbol	MATERIAL DESCRIPTION	Natural Moisture Content (%)	Total Unit Weight (pcf)	Liquid Limit	Plasticity Index	Pocket Pen. Su (ksf)	Torvane Su (ksf)	TXUU (ksf)	REMARKS
384.7	66.5		5	2 3 4	100		Medium stiff, moist to wet, brown, clayey SILT (ML) [ALLUVIUM]								
			5		92										
379.7	71.5		6	5 6 9	100		Very Stiff, moist with occasional wet silty zones, orange brown and gray, silty CLAY (CL) [ALLUVIUM]								
377.7	73.5		7	7 9 11	100		-trace fine sand								
373.7	77.5														
373.5	77.7		8	50/0.2'	100		Highly weathered SILTSTONE, grayish brown								
							End of Boring at 77.7 ft								

Elevation (feet): 385, 380, 375, 370, 365, 360, 355

Depth (feet): 70, 75, 80, 85, 90, 95, 100

Report: GEO_SOIL; File C:\USERS\MESIS\DESKTOP\VECTREN\AB BROWN BORING LOGS.GPJ; 9/12/2016 11:56:44 AM

Project: A.B. Brown Ash Pond Lower Dam Evaluation

Project Location: Posey County, Indiana

Project Number: 60442676

Log of Boring AECOM-B3

Sheet 1 of 2

Date(s) Drilled	**10/08/2015 12:00 AM to 10/08/2015 12:00 AM**	Logged By	**M. Jones**	Checked By	**V. Gautam**
Drilling Method	**Hollow Stem Auger**	Drill Bit Size/Type	**3.25" I.D. HSA**	Borehole Depth	**40.0 ft**
Drill Rig Type	**GeoProbe 8040DT**	Drilling Contractor	**Cardno ATC**	Surface Elevation	**417.9 ft NAVD88**
Borehole Backfill	**Grout**	Sampling Method(s)	**18" Split Spoon 2" ID, 30"**	Hammer Data	**Auto-Hammer, 81% efficiency**
Boring Location	**Adjacent to B-219 (ft NAD83)**	Groundwater Level(s)	**6.9 ft on 7/13/2015**		

Elevation (feet)	Depth (feet)	SAMPLES Type	Number	Sampling Resist. OR Core RQD (%)	Recovery (%)	Graphic Symbol	MATERIAL DESCRIPTION (Elevation (feet) / Depth (feet))	Natural Moisture Content (%)	Total Unit Weight (pcf)	Liquid Limit	Plasticity Index	Pocket Pen. Su (ksf)	Torvane Su (ksf)	TXUU (ksf)	REMARKS
417.9	0.0						Refer to Cardno ATC boring log B-219 for complete description and approximate depths of materials, stratigraphy, and groundwater levels. Blank drilling conducted between sample intervals.								
411.4	6.5	▨	1	7 5 6	100	▥	Stiff, wet, grayish brown and orange brown, SILT (ML) [ALLUVIUM]								
		▤	1		100	▥		30.6	115.0						
	10					▥									*3" SPT sampler used from 10' to 12'*
406.4	11.5	▨	2	1 WH 1	100	▥	Very soft, wet, brown, SILT (ML) [ALLUVIUM]								
		▤	2		0	▥									*3" SPT sampler used from 14' to 16'*
		▤	2A		88	▥									
391.4	26.5	▨	3	1 2 4	100	▨	Medium stiff, moist to wet, gray, silty, low plasticity, lean CLAY (CL) [ALLUVIUM]								
		▤	3		100	▨		21.2	127.0						
	30														

Project: A.B. Brown Ash Pond Lower Dam Evaluation

Project Location: Posey County, Indiana

Project Number: 60442676

Log of Boring AECOM-B3

Sheet 2 of 2

Elevation (feet)	Depth (feet)	SAMPLES Type	Number	Sampling Resist. OR Core RQD (%)	Recovery (%)	Graphic Symbol	MATERIAL DESCRIPTION	Elevation (feet)	Depth (feet)	Natural Moisture Content (%)	Total Unit Weight (pcf)	Liquid Limit	Plasticity Index	Pocket Pen. Su (ksf)	Torvane Su (ksf)	TXUU (ksf)	REMARKS
	30					▨											
385						▨											
	35					▨											
		▨	4	3 4 4	100	▨	Stiff, moist, orange brown with some grayish brown, silty, low plasticity, lean CLAY (CL) [ALLUVIUM]	381.4	36.5								
380		▤	4		92	▨											
	40						End of Boring at 40 ft	377.9	40.0								
375																	
	45																
370																	
	50																
365																	
	55																
360																	
	60																
355																	
	65																

Report: GEO_SOIL; File C:\USERS\MESIS\DESKTOP\VECTREN\AB BROWN BORING LOGS.GPJ; 9/12/2016 11:56:51 AM

Project: A.B. Brown Ash Pond Lower Dam Evaluation

Project Location: Posey County, Indiana

Project Number: 60442676

Log of Boring AECOM-B4

Sheet 1 of 2

Date(s) Drilled	**10/08/2015 12:00 AM to 10/08/2015 12:00 AM**	Logged By	**M. Jones**	Checked By	**V. Gautam**
Drilling Method	**Hollow Stem Auger**	Drill Bit Size/Type	**3.25" I.D. HSA**	Borehole Depth	**57.0 ft**
Drill Rig Type	**GeoProbe 8040DT**	Drilling Contractor	**Cardno ATC**	Surface Elevation	**416.1 ft NAVD88**
Borehole Backfill	**Grout**	Sampling Method(s)	**18" Split Spoon 2" ID, 30" Shelby Tube 3" ID**	Hammer Data	**Auto-Hammer, 81% efficiency**
Boring Location	**Adjacent to B-205 (ft NAD83)**	Groundwater Level(s)	**45 ft on 4/16/2015**		

Elevation (feet)	Depth (feet)	SAMPLES Type	Number	Sampling Resist. OR Core RQD (%)	Recovery (%)	Graphic Symbol	MATERIAL DESCRIPTION	Natural Moisture Content (%)	Total Unit Weight (pcf)	Liquid Limit	Plasticity Index	Pocket Pen. Su (ksf)	Torvane Su (ksf)	TXUU (ksf)	REMARKS
416.1	0.0						Refer to Cardno ATC boring log B-205 for complete description and approximate depths of materials, stratigraphy, and groundwater levels. Blank drilling conducted between sample intervals. Fine gravel and medium grained sand observed in cuttings from approximately 5 to 10 feet (apparent blanket drain material)								
405.6	10.5	▨	1	4 4 5	100	^ ^ ^	Stiff, moist, reddish brown, silty, low plasticity, lean CLAY (CL) [FILL]								
		▤	1		75	^ ^ ^		16.4	128.0						*Passing No. 200 Sieve = 82.9%*

Elevation scale: 415, 410, 405, 400, 395, 390

Depth scale: 0, 5, 10, 15, 20, 25, 30

AECOM

Project: A.B. Brown Ash Pond Lower Dam Evaluation

Project Location: Posey County, Indiana

Project Number: 60442676

Log of Boring AECOM-B4

Sheet 2 of 2

Elevation (feet)	Depth (feet)	SAMPLES Type	Number	Sampling Resist. OR Core RQD (%)	Recovery (%)	Graphic Symbol	MATERIAL DESCRIPTION	Natural Moisture Content (%)	Total Unit Weight (pcf)	Liquid Limit	Plasticity Index	Pocket Pen. Su (ksf)	Torvane Su (ksf)	TXUU (ksf)	REMARKS
384.6	31.5						Soft, moist, gray and orange brown, SILT (ML), trace organics [ALLUVIUM]								
		▨	2	2 2 2	100										
		▤	2		100			38.4	115.0	31	2				*Passing No. 200 Sieve = 99.7%*
371.6	44.5						Medium Stiff, moist to wet, gray, SILT (ML), trace organics [ALLUVIUM]								
		▨	3	2 2 3	100										
		▤	3		100			26.8	124.0						*Passing No. 200 Sieve = 99.9%*
362.6	53.5						Soft, moist, gray and bluish gray, low plasticity, silty CLAY (CL) [ALLUVIUM]								
		▨	4	WH WH 4	100										
		▤	4		100										
359.1	57.0						End of Boring at 57 ft								

Project: A.B. Brown Ash Pond Lower Dam Evaluation

Project Location: Posey County, Indiana

Project Number: 60442676

Log of Boring AECOM-B5

Sheet 1 of 2

Date(s) Drilled	10/08/2015 12:00 AM to 10/09/2015 12:00 AM	Logged By	M. Jones	Checked By	V. Gautam
Drilling Method	Hollow Stem Auger	Drill Bit Size/Type	3.25" I.D. HSA	Borehole Depth	39.0 ft
Drill Rig Type	GeoProbe 8040DT	Drilling Contractor	Cardno ATC	Surface Elevation	416.4 ft NAVD88
Borehole Backfill	Grout	Sampling Method(s)	18" Split Spoon 2" ID, 30" Shelby Tube 3" ID	Hammer Data	Auto-Hammer, 81% efficiency
Boring Location	Adjacent to B-215 (ft NAD83)	Groundwater Level(s)	6.5 ft on 7/16/2015		

Elevation (feet)	Depth (feet)	SAMPLES Type	Number	Sampling Resist. OR Core RQD (%)	Recovery (%)	Graphic Symbol	MATERIAL DESCRIPTION	Natural Moisture Content (%)	Total Unit Weight (pcf)	Liquid Limit	Plasticity Index	Pocket Pen. Su (ksf)	Torvane Su (ksf)	TXUU (ksf)	REMARKS
416.4	0.0						Refer to Cardno ATC boring log B-215 for complete description and approximate depths of materials and stratigraphy. Blank drilling conducted between sample intervals.								
413.4	3.0						Fine gravel and medium grained sand observed in cuttings from approximately 3 to 6.5 feet (apparent blanket drain material; jar sample retained)								
407.9	8.5	▨	1	4 7 12	100	▨	Very stiff, moist, yellowish brown with gray, silty, low plasticity, lean CLAY (CL) [FILL]								
	10	▤	1		100	▨									
387.9	28.5	▨	2	2 3 2	100	▥	Medium stiff, moist to wet, gray with some yellowish brown, SILT (ML) [ALLUVIUM]								

Elevation (feet)	Depth (feet)
415	
	5
410	
	10
405	
	15
400	
	20
395	
	25
390	
	30

AECOM

Project: A.B. Brown Ash Pond Lower Dam Evaluation

Project Location: Posey County, Indiana

Project Number: 60442676

Log of Boring AECOM-B5

Sheet 2 of 2

Elevation (feet)	Depth (feet)	SAMPLES Type	Number	Sampling Resist. OR Core RQD (%)	Recovery (%)	Graphic Symbol	MATERIAL DESCRIPTION Elevation (feet)		Depth (feet)	Natural Moisture Content (%)	Total Unit Weight (pcf)	Liquid Limit	Plasticity Index	Pocket Pen. Su (ksf)	Torvane Su (ksf)	TXUU (ksf)	REMARKS
385	30	▤	2		100					33.8							*Passing No. 200 Sieve = 71.2%*
380	35						380.9	Very soft, wet, gray, SILT (ML), trace fine sand and organics [ALLUVIUM]	35.5								
		▨	3	1 WH WH	100					49.8	106.0						
		▤	3		88												
							377.4	End of Boring at 39 ft	39.0								
375	40																
370	45																
365	50																
360	55																
355	60																
	65																

Project: A.B. Brown Ash Pond Lower Dam Evaluation

Project Location: Posey County, Indiana

Project Number: 60442676

Log of Boring AECOM-B8

Sheet 1 of 1

Date(s) Drilled	01/27/2016 12:00 AM to 01/27/2016 12:00 AM	Logged By	C. Siegel	Checked By	V. Gautam
Drilling Method	Hollow Stem Auger	Drill Bit Size/Type	3.25" I.D. HSA	Borehole Depth	26.3 ft
Drill Rig Type	Mobile B53 ATV	Drilling Contractor	Cardno ATC	Surface Elevation	427.7 ft NAVD88
Borehole Backfill	Grout	Sampling Method(s)	24" Split Spoon 2" ID	Hammer Data	
Boring Location	N 968016.65 E 2770903.02 (ft NAD83)	Groundwater Level(s)	16 ft on 1/27/2016		

SAMPLES Type	Number	Sampling Resist. OR Core RQD (%)	Recovery (%)
SS	SS-1	9 12 15 15	75
SS	SS-2	8 10 8 6	83
SS	SS-3	2 1 1 2	83
SS	SS-4	2 1 2 2	86
SS	SS-5	2 1 2 3	100
SS	SS-6	4 3 3 5	100
SS	SS-7	2 2 2 3	83
SS	SS-8	3 3 4 4	83
SS	SS-9	4 5 5 6	100
SS	SS-10	4 5 6 8	100
SS	SS-11	50/0.2'	17

Elevation (feet)	MATERIAL DESCRIPTION	Depth (feet)
427.7	Medium dense, moist, brown, SILT (ML), with clay, trace coal fragments and vegetation [FILL]	0.0
426.7	Medium dense, moist, brown SILT (ML), with clay [ALLUVIUM]	1.0
	becomes very loose and wet	
416.7	Very loose, wet, brown CLAY-SILT (CL-ML) [ALLUVIUM]	11.0
414.7	Medium stiff, moist, brown CLAY (CL) with silt and occasional weathered silt partings [ALLUVIUM]	13.0
	encountered water at 16-ft	
	becomes stiff	
401.7	SHALE highly weathered	26.0
401.4	End of Boring at 26.3 ft	26.3

Elevation (feet): 425, 420, 415, 410, 405, 400

Depth (feet): 0, 5, 10, 15, 20, 25, 30

Natural Moisture Content (%) | Total Unit Weight (pcf) | Liquid Limit | Plasticity Index | Pocket Pen. Su (ksf) | Torvane Su (ksf) | TXUU (ksf) | REMARKS

Cardno ATC

7988 Centerpoint Drive, Suite 100
Indianapolis, IN 46256
(317) 849-4990
Fax (317) 849-4278

TEST BORING LOG

CLIENT **Vectren Corporation**

PROJECT NAME **Ash Pond Safety Factor Assessment**

PROJECT LOCATION **A.B. Brown Generating Facility**
Posey County, Indiana

BORING # **B-201**

JOB # **170GC00108**

DRILLING and SAMPLING INFORMATION

Date Started **4/16/15**
Date Completed **4/17/15**
Drill Foreman **W. Bates**
Inspector **S. Marcum**
Boring Method **HSA**

Hammer Wt. **140** lbs.
Hammer Drop **30** in.
Spoon Sampler OD **2.0** in.
Rock Core Dia. **--** in.
Shelby Tube OD **--** in.

TEST DATA

SOIL CLASSIFICATION SURFACE ELEVATION 450.3	Stratum Elevation	Stratum Depth, ft	Depth Scale, ft	Sample No.	Sample Type	Sampler Graphics Recovery Graphics	Groundwater	Standard Penetration Test, Blows per 6 in. Increments	Moisture Content, %	Pocket Penetrometer PP-tsf	Remarks
Reddish brown, slightly moist, silty clay (EMBANKMENT FILL)				1	SS	☒ ■		13-6-6			Boring coordinates and ground surface elevation surveyed by Three I Design.
			5	2	SS	☒ ■		7-10-10	15.2	2.5	**Sample No. SS-2:** Atterberg limits: LL=29, PL=22, PI=7 Passing No. 200 sieve = 99.4%
				3	SS	☒ ■		13-15-15	18.1		
Reddish brown, slightly moist, fine sand (EMBANKMENT FILL)	441.3	9.0		4	SS	☒ ■		8-9-8			
Reddish brown and gray, slightly moist, silty clay (EMBANKMENT FILL)	439.8	10.5	10	5	SS	☒ ■		10-12-14	15.6	2.5	
Reddish brown, slightly moist, fine sand (EMBANKMENT FILL)	437.3	13.0		6	SS	☒ □		4-5-5			Borehole backfilled with cement/bentonite grout.
Reddish brown, moist, silty clay (EMBANKMENT FILL)	434.8	15.5	15	7	SS	☒ ■		9-9-11	14.5		**Sample No. SS-7:** Atterberg limits: LL=30, PL=19, PI=11 Passing No. 200 sieve = 96.8%
				8	SS	☒ ■		3-3-4	22.1		
Brown and gray, slightly moist, clayey silt (EMBANKMENT FILL)	429.8	20.5	20	9	SS	☒ ■		9-12-12	17.0		
				10	SS	☒ ■		5-7-8	16.1	2.5	
			25	11	SS	☒ ■		11-16-16	15.5		**Sample No. SS-11:** Atterberg limits: LL=29, PL=21, PI=8 Passing No. 200 sieve = 97.4%
				12	SS	☒ ■		6-9-11	16.7		
			30	13	SS	☒ ■		8-15-14	16.1		
				14	SS	☒ ■		9-8-8	23.4	3.0	
Brown, wet, soft to medium stiff, SILT (ML)	413.3	37.0	35	15	SS	☒ ■	●	11-11-10	24.3		
				16	SS	☒ ■		2-2-2	31.3		

Sample Type
SS - Driven Split Spoon
ST - Pressed Shelby Tube
CA - Continuous Flight Auger
RC - Rock Core
CU - Cuttings
CT - Continuous Tube

Depth to Groundwater
● Noted on Drilling Tools **37.0** ft.
▽ At Completion **--** ft.
▼ After **--** hours **--** ft.
▨ Cave Depth **--** ft.

Boring Method
HSA - Hollow Stem Augers
CFA - Continuous Flight Augers
CA - Casing Advancer
MD - Mud Drilling
HA - Hand Auger

Cardno ATC
7988 Centerpoint Drive, Suite 100
Indianapolis, IN 46256
(317) 849-4990
Fax (317) 849-4278

TEST BORING LOG

CLIENT **Vectren Corporation**
PROJECT NAME **Ash Pond Safety Factor Assessment**
PROJECT LOCATION **A.B. Brown Generating Facility**
Posey County, Indiana

BORING # **B-201**
JOB # **170GC00108**

DRILLING and SAMPLING INFORMATION

Date Started **4/16/15**
Date Completed **4/17/15**
Drill Foreman **W. Bates**
Inspector **S. Marcum**
Boring Method **HSA**

Hammer Wt. **140** lbs.
Hammer Drop **30** in.
Spoon Sampler OD **2.0** in.
Rock Core Dia. **--** in.
Shelby Tube OD **--** in.

TEST DATA

SOIL CLASSIFICATION (continued)	Stratum Elevation	Stratum Depth, ft	Depth Scale, ft	Sample No.	Sample Type	Sampler Graphics Recovery Graphics	Groundwater	Standard Penetration Test, Blows per 6 in. Increments	Moisture Content, %	Pocket Penetrometer PP-tsf	Remarks
Brown, wet, soft to medium stiff, SILT (ML)				17	SS			3-3-3	26.7		
			45	18	SS			2-2-3	29.0		**Sample No. SS-18:** Atterberg limits: non-plastic Passing No. 200 sieve = 99.7%
				19	SS			3-3-3	25.7		
	402.3	48.0									
Gray, moist, very soft to medium stiff, SILTY CLAY (CL)			50	20	SS			2-1-1	23.9	0.5	**Sample No. SS-20:** Atterberg limits: LL=32, PL=17, PI=15 Passing No. 200 sieve = 98.2%
				21	SS			0-0-0	23.0	0.5	
			55	22	SS			3-4-5	22.4	1.25	
	394.3	56.0									
Reddish brown and gray, moist, very stiff to stiff, SANDY CLAY (CL)				23	SS			6-8-8	18.6	3.0	
				24	SS			5-6-6	22.2	2.0	
	390.3	60.0	60								
Bottom of Test Boring at 60.0 ft											

Sample Type
SS - Driven Split Spoon
ST - Pressed Shelby Tube
CA - Continuous Flight Auger
RC - Rock Core
CU - Cuttings
CT - Continuous Tube

Depth to Groundwater
● Noted on Drilling Tools **37.0** ft.
▽ At Completion **--** ft.
▼ After **--** hours **--** ft.
Cave Depth **--** ft.

Boring Method
HSA - Hollow Stem Augers
CFA - Continuous Flight Augers
CA - Casing Advancer
MD - Mud Drilling
HA - Hand Auger

Cardno ATC
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TEST BORING LOG

CLIENT **Vectren Corporation**
PROJECT NAME **Ash Pond Safety Factor Assessment**
PROJECT LOCATION **A.B. Brown Generating Facility**
Posey County, Indiana

BORING # **B-202**
JOB # **170GC00108**

DRILLING and SAMPLING INFORMATION

Date Started	**4/20/15**	Hammer Wt.	**140** lbs.
Date Completed	**4/20/15**	Hammer Drop	**30** in.
Drill Foreman	**W. Bates**	Spoon Sampler OD	**2.0** in.
Inspector	**S. Marcum**	Rock Core Dia.	**--** in.
Boring Method	**HSA**	Shelby Tube OD	**--** in.

TEST DATA

SOIL CLASSIFICATION SURFACE ELEVATION 450.7	Stratum Elevation	Stratum Depth, ft	Depth Scale, ft	Sample No.	Sample Type	Sampler Graphics Recovery Graphics	Groundwater	Standard Penetration Test, Blows per 6 in. Increments	Moisture Content, %	Pocket Penetrometer PP-tsf	Remarks
Reddish brown, slightly moist, silty clay with crushed stone (EMBANKMENT FILL)				1	HA	☐ ■					Boring coordinates and ground surface elevation surveyed by Three I Design.
			5	2	HA	☐ ■					
	444.7	6.0									
Brown, moist to slightly moist, silty clay (EMBANKMENT FILL)				3	SS	☒ ■		6-6-2	15.1	2.5	Borehole backfilled with cement/bentonite grout.
			10	4	SS	☒ ■		7-7-6	16.3	3.0	
				5	SS	☒ ■		11-10-10	18.1		
			15	6	SS	☒ ■		11-6-9	14.0	3.0	**Sample No. SS-6:** Atterberg limits: LL=28, PL=18, PI=10 Passing No. 200 sieve = 95.2%
				7	SS	☒ ■		9-9-10	16.2	2.0	
			20	8	SS	☒ ■		8-13-10	17.3	2.25	
				9	SS	☒ ■		7-8-10	13.1		
	428.2	22.5									
Reddish brown, slightly moist, sandy clay (EMBANKMENT FILL)			25	10	SS	☒ ■		6-6-6	16.2	2.0	
				11	SS	☒ ■		6-6-8	15.6	2.5	**Sample No. SS-11:** Atterberg limits: LL=33, PL=15, PI=18 Passing No. 200 sieve = 66.1%
			30	12	SS	☒ ■		5-5-7	15.7	2.5	
				13	SS	☒ ■		8-8-8	16.0	3.0	
	417.7	33.0									
Brown and gray, moist, silty clay with little sand and gravel (EMBANKMENT FILL)			35	14	SS	☒ ■		5-6-7	18.0	3.5	
				15	SS	☒ ■		7-11-14	13.9		**Sample No. SS-16:** Atterberg limits: LL=32, PL=17, PI=15 Passing No. 200 sieve = 81.7%
				16	SS	☒ ■		10-7-9	14.7	3.5	

Sample Type
SS - Driven Split Spoon
ST - Pressed Shelby Tube
CA - Continuous Flight Auger
RC - Rock Core
CU - Cuttings
CT - Continuous Tube

Depth to Groundwater
● Noted on Drilling Tools **80.0** ft.
▽ At Completion **--** ft.
▼ After **--** hours **--** ft.
▨ Cave Depth **--** ft.

Boring Method
HSA - Hollow Stem Augers
CFA - Continuous Flight Augers
CA - Casing Advancer
MD - Mud Drilling
HA - Hand Auger

CLIENT **Vectren Corporation**

BORING # **B-202**

PROJECT NAME **Ash Pond Safety Factor Assessment**

JOB # **170GC00108**

PROJECT LOCATION **A.B. Brown Generating Facility**
Posey County, Indiana

DRILLING and SAMPLING INFORMATION

Date Started **4/20/15**
Date Completed **4/20/15**
Drill Foreman **W. Bates**
Inspector **S. Marcum**
Boring Method **HSA**

Hammer Wt. **140** lbs.
Hammer Drop **30** in.
Spoon Sampler OD **2.0** in.
Rock Core Dia. **--** in.
Shelby Tube OD **--** in.

TEST DATA

SOIL CLASSIFICATION (continued)	Stratum Elevation	Stratum Depth, ft	Depth Scale, ft	Sample No.	Sample Type	Sampler Graphics Recovery Graphics	Groundwater	Standard Penetration Test, Blows per 6 in. Increments	Moisture Content, %	Pocket Penetrometer PP-tsf	Remarks
Brown and gray, moist, silty clay with little sand and gravel (EMBANKMENT FILL)				17	SS			16-18-32	15.8		
			45	18	SS			7-9-8	17.4	4.0	
				19	SS			10-13-14	17.8		
Reddish brown, moist, very stiff to medium stiff SANDY CLAY (CL)	402.7	48.0	50	20	SS			4-6-7	17.1	2.5	
				21	SS			11-13-13	16.3		
			55	22	SS			10-10-11	14.0	1.5	**Sample No. SS-22:** Atterberg limits: LL=42, PL=16, PI=26 Passing No. 200 sieve = 66.1%
				23	SS			13-13-16	19.1	1.5	
			60	24	SS			4-5-5	22.8		
Gray, moist, medium stiff, SILTY CLAY (CL) with little sand	390.2	60.5		25	SS			5-7-7	8.0	3.0	**Sample No. SS-25:** Atterberg limits: LL=29, PL=19, PI=10 Passing No. 200 sieve = 88.6%
			65	26	ST				23.2		
Reddish brown, moist, medium stiff to very stiff, SANDY CLAY (CL)	385.2	65.5		27	SS			3-4-4	17.7	1.25	**Sample No. ST-26:** Atterberg limits: LL=26, PL=20, PI=6 Passing No. 200 sieve = 67.8%
Reddish brown, moist, very stiff to stiff, SILTY CLAY (CL-ML)	382.7	68.0	70	28	SS			7-7-9			
				29	SS			10-10-10	24.4		
			75	30	SS			4-5-9	24.4	1.5	**Sample No. SS-30:** Atterberg limits: LL=28, PL=21, PI=7 Passing No. 200 sieve = 99.3%
				31	SS			7-6-5	25.6		
Reddish brown, moist, very soft to stiff, SANDY CLAY (CL)	372.7	78.0		32	SS		●	0-1-2	16.4	0.75	**Sample No. SS-32:** Atterberg limits: LL=21, PL=13, PI=8 Passing No. 200 sieve =

Sample Type
- SS - Driven Split Spoon
- ST - Pressed Shelby Tube
- CA - Continuous Flight Auger
- RC - Rock Core
- CU - Cuttings
- CT - Continuous Tube

Depth to Groundwater
- ● Noted on Drilling Tools **80.0** ft.
- ▽ At Completion **--** ft.
- ▼ After **--** hours **--** ft.
- ▨ Cave Depth **--** ft.

Boring Method
- HSA - Hollow Stem Augers
- CFA - Continuous Flight Augers
- CA - Casing Advancer
- MD - Mud Drilling
- HA - Hand Auger

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TEST BORING LOG

CLIENT **Vectren Corporation**
PROJECT NAME **Ash Pond Safety Factor Assessment**
PROJECT LOCATION **A.B. Brown Generating Facility**
Posey County, Indiana

BORING # **B-202**
JOB # **170GC00108**

DRILLING and SAMPLING INFORMATION

Date Started	**4/20/15**	Hammer Wt.	**140** lbs.
Date Completed	**4/20/15**	Hammer Drop	**30** in.
Drill Foreman	**W. Bates**	Spoon Sampler OD	**2.0** in.
Inspector	**S. Marcum**	Rock Core Dia.	**--** in.
Boring Method	**HSA**	Shelby Tube OD	**--** in.

TEST DATA

SOIL CLASSIFICATION (continued)	Stratum Elevation	Stratum Depth, ft	Depth Scale, ft	Sample No.	Sample Type	Sampler Graphics Recovery Graphics	Groundwater	Standard Penetration Test, Blows per 6 in. Increments	Moisture Content, %	Pocket Penetrometer PP-tsf	Remarks
Reddish brown, moist, very soft to stiff, SANDY CLAY (CL)	367.7	83.0		33	SS			6-7-8	16.0	1.5	71.7%
Brown, very moist, stiff to very stiff SANDY CLAY (CL) with trace sandstone fragments			85	34	SS			5-6-9	15.3	2.5	**Sample No. SS-34:** Atterberg limits: LL=31, PL=15, PI=16 Passing No. 200 sieve = 43.6%
				35	SS			9-9-9	18.0		
	360.2	90.5	90	36	SS			6-6-7	20.4	1.0	
Bluish gray, slightly moist, very stiff, SANDY CLAY (CL)	356.7	94.0		37	SS			7-11-15	16.7		
Grayish brown, severely weathered, SILTSTONE	356.4	94.3		38	SS			20-50/0.3			
Bottom of Test Boring at 94.3 ft											

Sample Type
SS - Driven Split Spoon
ST - Pressed Shelby Tube
CA - Continuous Flight Auger
RC - Rock Core
CU - Cuttings
CT - Continuous Tube

Depth to Groundwater
● Noted on Drilling Tools **80.0** ft.
▽ At Completion **--** ft.
▼ After **--** hours **--** ft.
Cave Depth **--** ft.

Boring Method
HSA - Hollow Stem Augers
CFA - Continuous Flight Augers
CA - Casing Advancer
MD - Mud Drilling
HA - Hand Auger

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TEST BORING LOG

CLIENT **Vectren Corporation**
PROJECT NAME **Ash Pond Safety Factor Assessment**
PROJECT LOCATION **A.B. Brown Generating Facility**
Posey County, Indiana

BORING # **B-203**
JOB # **170GC00108**

DRILLING and SAMPLING INFORMATION

Date Started **4/21/15**
Date Completed **4/21/15**
Drill Foreman **W. Bates**
Inspector **S. Marcum**
Boring Method **HSA**

Hammer Wt. **140** lbs.
Hammer Drop **30** in.
Spoon Sampler OD **2.0** in.
Rock Core Dia. **--** in.
Shelby Tube OD **--** in.

TEST DATA

SOIL CLASSIFICATION SURFACE ELEVATION 450.5	Stratum Elevation	Stratum Depth, ft	Depth Scale, ft	Sample No.	Sample Type	Sampler Graphics Recovery Graphics	Groundwater	Standard Penetration Test, Blows per 6 in. Increments	Moisture Content, %	Pocket Penetrometer PP-tsf	Remarks
Reddish brown, slightly moist, sandy clay (EMBANKMENT FILL)				1	HA						Boring coordinates and ground surface elevation surveyed by Three I Design.
				2	HA						
	445.0	5.5	5								
Brown, slightly moist, clayey silt (EMBANKMENT FILL)				3	SS			7-7-9	16.9		Borehole backfilled with cement/bentonite grout.
				4	SS			3-4-5		1.5	
	440.0	10.5	10								
Reddish brown, moist, silt (EMBANKMENT FILL)				5	SS			9-10-10			
				6	SS			4-6-8			
	435.0	15.5	15								
Light brown and brown, slightly moist, silty clay (EMBANKMENT FILL)				7	SS			17-14-17	16.5	4.0	**Sample No. SS-7:** Atterberg limits: LL=31, PL=14, PI=17 Passing No. 200 sieve = 71.0%
			20	8	SS			5-7-8	15.0		
				9	SS			11-10-9	14.3	4.0	
	427.5	23.0									
Tan, slightly moist, fine sand (EMBANKMENT FILL)			25	10	SS			5-6-6			
	424.0	26.5									
Brown, slightly moist, silt (EMBANKMENT FILL)				11	SS			6-11-14			
	422.5	28.0									
Brown, moist, silty clay with little sand (EMBANKMENT FILL)				12	SS			6-7-11		2.5	
			30								
				13	SS			12-12-11	15.7		**Sample No. SS-13:** Atterberg limits: LL=25, PL=18, PI=7 Passing No. 200 sieve = 87.2%
				14	SS			8-6-7			
	415.0	35.5	35								
Reddish brown, moist, silty clay (EMBANKMENT FILL)				15	SS			6-9-9	19.2	2.0	
	412.5	38.0									
Light brown, moist, sandy clay (EMBANKMENT FILL)	410.5	40.0		16	SS			5-7-8	13.9		

Sample Type
SS - Driven Split Spoon
ST - Pressed Shelby Tube
CA - Continuous Flight Auger
RC - Rock Core
CU - Cuttings
CT - Continuous Tube

Depth to Groundwater
● Noted on Drilling Tools **74.5** ft.
▽ At Completion **--** ft.
▼ After **--** hours **--** ft.
▨ Cave Depth **--** ft.

Boring Method
HSA - Hollow Stem Augers
CFA - Continuous Flight Augers
CA - Casing Advancer
MD - Mud Drilling
HA - Hand Auger

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TEST BORING LOG

CLIENT **Vectren Corporation**
PROJECT NAME **Ash Pond Safety Factor Assessment**
PROJECT LOCATION **A.B. Brown Generating Facility**
Posey County, Indiana

BORING # **B-203**
JOB # **170GC00108**

DRILLING and SAMPLING INFORMATION

Date Started **4/21/15**
Date Completed **4/21/15**
Drill Foreman **W. Bates**
Inspector **S. Marcum**
Boring Method **HSA**

Hammer Wt. **140** lbs.
Hammer Drop **30** in.
Spoon Sampler OD **2.0** in.
Rock Core Dia. **--** in.
Shelby Tube OD **--** in.

TEST DATA

SOIL CLASSIFICATION (continued)	Stratum Elevation	Stratum Depth, ft	Depth Scale, ft	Sample No.	Sample Type	Sampler Graphics Recovery Graphics	Groundwater	Standard Penetration Test, Blows per 6 in. Increments	Moisture Content, %	Pocket Penetrometer PP-tsf	Remarks
Brown and gray, moist, sandy clay (EMBANKMENT FILL)				17	SS			9-13-14	12.9		
Brown, moist, silty clay (EMBANKMENT FILL)	407.5	43.0		18	SS			4-4-7	14.8		
Light brown, moist, sandy clay (EMBANKMENT FILL)	405.0	45.5	45	19	SS			9-9-6	17.3	1.5	
			50	20	SS			7-8-9	15.0		
				21	SS			10-12-13	12.7		
			55	22	SS			6-9-12	11.7		**Sample No. SS-22:** Atterberg limits: LL=26, PL=16, PI=10 Passing No. 200 sieve = 58.7%
				23	SS			10-14-16	17.1		
Brown, moist, medium stiff, SILTY CLAY (CL)	392.5	58.0	60	24	SS			3-4-4	24.4		
				25	SS			3-3-6	23.5		
				26	ST						**Sample No. ST-26:** Atterberg limits: LL=30, PL=19, PI=11 Passing No. 200 sieve = 96.6%
Gray, moist, stiff to medium stiff, SILTY CLAY (CL)	385.0	65.5	65	27	SS			3-5-5	34.0		
			70	28	SS			4-5-5	21.9	1.5	**Sample No. SS-28:** Atterberg limits: LL=36, PL=19, PI=17 Passing No. 200 sieve = 99.6%
				29	SS			4-7-8	28.1	1.0	
			75	30	SS		●	3-3-5	41.9		
				31	SS			4-5-5	35.9		
				32	SS			3-4-5	32.5		

Sample Type
- SS - Driven Split Spoon
- ST - Pressed Shelby Tube
- CA - Continuous Flight Auger
- RC - Rock Core
- CU - Cuttings
- CT - Continuous Tube

Depth to Groundwater
- ● Noted on Drilling Tools **74.5** ft.
- ▽ At Completion **--** ft.
- ▼ After **--** hours **--** ft.
- ⊠ Cave Depth **--** ft.

Boring Method
- HSA - Hollow Stem Augers
- CFA - Continuous Flight Augers
- CA - Casing Advancer
- MD - Mud Drilling
- HA - Hand Auger

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TEST BORING LOG

CLIENT **Vectren Corporation**
PROJECT NAME **Ash Pond Safety Factor Assessment**
PROJECT LOCATION **A.B. Brown Generating Facility**
Posey County, Indiana
BORING # **B-203**
JOB # **170GC00108**

DRILLING and SAMPLING INFORMATION

Date Started **4/21/15**
Date Completed **4/21/15**
Drill Foreman **W. Bates**
Inspector **S. Marcum**
Boring Method **HSA**
Hammer Wt. **140** lbs.
Hammer Drop **30** in.
Spoon Sampler OD **2.0** in.
Rock Core Dia. **--** in.
Shelby Tube OD **--** in.

TEST DATA

SOIL CLASSIFICATION (continued)	Stratum Elevation	Stratum Depth, ft	Depth Scale, ft	Sample No.	Sample Type	Sampler Graphics Recovery Graphics	Groundwater	Standard Penetration Test, Blows per 6 in. Increments	Moisture Content, %	Pocket Penetrometer PP-tsf	Remarks
Gray, moist, medium stiff to stiff, SILTY CLAY (CL)	367.5	83.0		33	SS			4-6-7	32.1		
Gray, moist, medium stiff, SILTY CLAY (CL) with fine sand seams			85	34	SS			2-3-3	31.4	0.75	
	361.5	89.0		35	ST						
Bluish gray, very stiff, SANDY CLAY (CL)	359.0	91.5	90	36	SS			22-16-12			
Bluish gray, severely weathered SILTSTONE	358.1	92.4		37	SS			4-14-50/0.4			
Bottom of Test Boring at 92.4 ft											

Sample Type
SS - Driven Split Spoon
ST - Pressed Shelby Tube
CA - Continuous Flight Auger
RC - Rock Core
CU - Cuttings
CT - Continuous Tube

Depth to Groundwater
● Noted on Drilling Tools **74.5** ft.
▽ At Completion **--** ft.
▼ After **--** hours **--** ft.
☒ Cave Depth **--** ft.

Boring Method
HSA - Hollow Stem Augers
CFA - Continuous Flight Augers
CA - Casing Advancer
MD - Mud Drilling
HA - Hand Auger

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TEST BORING LOG

CLIENT **Vectren Corporation**

BORING # **B-204**

PROJECT NAME **Ash Pond Safety Factor Assessment**

JOB # **170GC00108**

PROJECT LOCATION **A.B. Brown Generating Facility**
Posey County, Indiana

DRILLING and SAMPLING INFORMATION

Date Started	**4/21/15**	Hammer Wt.	**140** lbs.
Date Completed	**4/21/15**	Hammer Drop	**30** in.
Drill Foreman	**W. Bates**	Spoon Sampler OD	**2.0** in.
Inspector	**S. Marcum**	Rock Core Dia.	**--** in.
Boring Method	**HSA**	Shelby Tube OD	**--** in.

TEST DATA

SOIL CLASSIFICATION SURFACE ELEVATION 450.5	Stratum Elevation	Stratum Depth, ft	Depth Scale, ft	Sample No.	Sample Type	Sampler Graphics Recovery Graphics	Groundwater	Standard Penetration Test, Blows per 6 in. Increments	Moisture Content, %	Pocket Penetrometer PP-tsf	Remarks
Brown, moist, silty clay (EMBANKMENT FILL)											Boring coordinates and ground surface elevation surveyed by Three I Design.
				1	HA						
				2	HA						
			5								
	444.5	6.0									
Reddish brown, moist, silty clay (EMBANKMENT FILL)				3	SS			4-7-7	15.0		
				4	SS			4-3-5	18.1	3.0	**Sample No. SS-4:** Atterberg limits: LL=32, PL=19, PI=13 Passing No. 200 sieve = 98.8%
			10								
				5	SS			5-11-12	21.6		
	437.5	13.0									
Light brown, slightly moist, silt (EMBANKMENT FILL)				6	SS			5-8-6	15.0		
	435.0	15.5	15								
Brown, slightly moist, silty clay (EMBANKMENT FILL)				7	SS			14-15-17	13.2	4.0	
				8	SS			7-7-7	19.9	2.0	
	430.0	20.5	20								
Brown, slightly moist, silt with interbedded silty clay (EMBANKMENT FILL)				9	SS			9-14-13	15.7	4.5+	Borehole backfilled with cement/bentonite grout.
	427.5	23.0									
Brown, slightly moist, silty clay (EMBANKMENT FILL)				10	SS			4-7-8	19.9		
			25								
				11	SS			9-9-15	13.9	4.0	
	422.5	28.0									
Reddish brown, moist, sandy clay (EMBANKMENT FILL)				12	SS			3-4-7	20.9	3.0	
	420.0	30.5	30								
Brown and light brown, moist, silty clay (EMBANKMENT FILL)				13	SS			10-15-20	15.0		
	417.5	33.0									
Brown, moist, silty clay with interbedded sandy clay (EMBANKMENT FILL)				14	SS			8-12-10	15.6		
			35								
				15	SS			12-13-13	16.3	3.0	**Sample No. SS-15:** Atterberg limits: LL=29, PL=22, PI=7 Passing No. 200 sieve = 99.5%
				16	SS			11-9-10	17.6		

Sample Type
SS - Driven Split Spoon
ST - Pressed Shelby Tube
CA - Continuous Flight Auger
RC - Rock Core
CU - Cuttings
CT - Continuous Tube

Depth to Groundwater
● Noted on Drilling Tools **66.0** ft.
▽ At Completion **--** ft.
▼ After **--** hours **--** ft.
▨ Cave Depth **--** ft.

Boring Method
HSA - Hollow Stem Augers
CFA - Continuous Flight Augers
CA - Casing Advancer
MD - Mud Drilling
HA - Hand Auger

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TEST BORING LOG

CLIENT **Vectren Corporation**

PROJECT NAME **Ash Pond Safety Factor Assessment**

PROJECT LOCATION **A.B. Brown Generating Facility**
Posey County, Indiana

BORING # **B-204**

JOB # **170GC00108**

DRILLING and SAMPLING INFORMATION

Date Started **4/21/15**

Date Completed **4/21/15**

Drill Foreman **W. Bates**

Inspector **S. Marcum**

Boring Method **HSA**

Hammer Wt. **140** lbs.

Hammer Drop **30** in.

Spoon Sampler OD **2.0** in.

Rock Core Dia. **--** in.

Shelby Tube OD **--** in.

TEST DATA

SOIL CLASSIFICATION (continued)	Stratum Elevation	Stratum Depth, ft	Depth Scale, ft	Sample No.	Sample Type	Sampler Graphics Recovery Graphics	Groundwater	Standard Penetration Test, Blows per 6 in. Increments	Moisture Content, %	Pocket Penetrometer PP-tsf	Remarks
Brown, moist, silty clay with interbedded sandy clay (EMBANKMENT FILL)				17	SS			10-17-18	14.9	4.5+	
				18	SS			5-7-9	17.0		
			45								
Reddish brown, moist, stiff to medium, stiff CLAYEY SILT (ML)	404.5	46.0									
				19	SS			5-7-8	21.9		
				20	SS			4-7-7	23.5	1.5	**Sample No. SS-20:** Atterberg limits: LL=27, PL=22, PI=5 Passing No. 200 sieve = 97.2%
			50								
				21	SS			5-5-5	27.8		
Gray, moist, medium stiff to very soft, SILTY CLAY (CL-ML)	397.5	53.0									
				22	SS			4-3-4			
			55								
				23	SS			5-4-4	28.1		**Sample No. SS-23:** Atterberg limits: LL=28, PL=21, PI=7 Passing No. 200 sieve = 99.3%
				24	SS			3-3-3	27.3		
			60								
				25	SS			3-4-5			
				26	SS			0-0-0			
			65								
							●				
				27	SS			1-2-3			
Reddish brown, moist, stiff, SANDY CLAY (CL)	382.5	68.0									
				28	ST						
			70								
				29	SS			4-5-7	19.6		
Orange, moist, hard, SANDY SILT (ML)	377.5	73.0									
Orange and gray, severely weathered, SILTSTONE	376.0	74.5		30	SS			16-38-50/0.1			
Bottom of Test Boring at 74.6 ft	375.9	74.6									

Sample Type

- SS - Driven Split Spoon
- ST - Pressed Shelby Tube
- CA - Continuous Flight Auger
- RC - Rock Core
- CU - Cuttings
- CT - Continuous Tube

Depth to Groundwater

- ● Noted on Drilling Tools **66.0** ft.
- ▽ At Completion **--** ft.
- ▼ After **--** hours **--** ft.
- ☒ Cave Depth **--** ft.

Boring Method

- HSA - Hollow Stem Augers
- CFA - Continuous Flight Augers
- CA - Casing Advancer
- MD - Mud Drilling
- HA - Hand Auger

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TEST BORING LOG

CLIENT **Vectren Corporation**
PROJECT NAME **Ash Pond Safety Factor Assessment**
PROJECT LOCATION **A.B. Brown Generating Facility**
Posey County, Indiana
BORING # **B-205**
JOB # **170GC00108**

DRILLING and SAMPLING INFORMATION

Date Started **4/16/15**
Date Completed **4/16/15**
Drill Foreman **W. Bates**
Inspector **S. Marcum**
Boring Method **HSA**
Hammer Wt. **140** lbs.
Hammer Drop **30** in.
Spoon Sampler OD **2.0** in.
Rock Core Dia. **--** in.
Shelby Tube OD **--** in.

TEST DATA

SOIL CLASSIFICATION SURFACE ELEVATION 415.5	Stratum Elevation	Stratum Depth, ft	Depth Scale, ft	Sample No.	Sample Type	Sampler Graphics Recovery Graphics	Groundwater	Standard Penetration Test, Blows per 6 in. Increments	Moisture Content, %	Pocket Penetrometer PP-tsf	Remarks
Reddish brown, moist, silty clay (EMBANKMENT FILL)	413.5	2.0		1	SS			6-5-8	17.1	3.0	Boring coordinates and ground surface elevation surveyed by Three I Design.
Reddish brown, moist sandy clay with trace gravel (EMBANKMENT FILL)			5	2	SS			6-11-14	14.4		
	407.5	8.0		3	SS			8-6-8	20.2		Borehole backfilled with cement/bentonite grout.
Light brown and gray, moist, silty clay with little sand (EMBANKMENT FILL)	405.0	10.5	10	4	SS			7-7-7	14.0		
Brown, moist, silty clay (EMBANKMENT FILL)				5	SS			8-9-9	16.3	2.5	**Sample No. SS-5:** Atterberg limits: LL=33, PL=15, PI=18 Passing No. 200 sieve = 88.5%
	399.0	16.5	15	6	SS			3-3-3	20.7	1.5	
Brown and light brown, slightly moist, silty clay with trace sand (EMBANKMENT FILL)	397.5	18.0		7	SS			5-6-8	17.3		
Brown, moist, silty clay (EMBANKMENT FILL)			20	8	SS			6-6-8	19.3	2.0	
				9	SS			7-7-14	16.2	2.5	
	389.0	26.5	25	10	SS			8-10-7	20.1	2.0	
Gray, slightly moist, very stiff, CLAYEY SILT (ML)				11	SS			8-9-10	20.9		
			30	12	SS			7-8-9	21.9		
	382.5	33.0		13	SS			9-9-9	22.2	3.0	
Dark gray, very moist, soft to medium stiff, SILT (ML) with trace organics and trace fine sand			35	14	SS			3-2-3	36.6	2.0	**Sample No. SS-14:** Atterberg limits: non-plastic Passing No. 200 sieve = 95.0%
				15	SS			4-4-6	40.9		
				16	SS			2-3-3	33.1		

Sample Type
SS - Driven Split Spoon
ST - Pressed Shelby Tube
CA - Continuous Flight Auger
RC - Rock Core
CU - Cuttings
CT - Continuous Tube

Depth to Groundwater
● Noted on Drilling Tools **45.0** ft.
▽ At Completion **--** ft.
▼ After **--** hours **--** ft.
▨ Cave Depth **--** ft.

Boring Method
HSA - Hollow Stem Augers
CFA - Continuous Flight Augers
CA - Casing Advancer
MD - Mud Drilling
HA - Hand Auger

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TEST BORING LOG

CLIENT **Vectren Corporation**

PROJECT NAME **Ash Pond Safety Factor Assessment**

PROJECT LOCATION **A.B. Brown Generating Facility**
Posey County, Indiana

BORING # **B-205**

JOB # **170GC00108**

DRILLING and SAMPLING INFORMATION

Date Started	**4/16/15**	Hammer Wt.	**140** lbs.
Date Completed	**4/16/15**	Hammer Drop	**30** in.
Drill Foreman	**W. Bates**	Spoon Sampler OD	**2.0** in.
Inspector	**S. Marcum**	Rock Core Dia.	**--** in.
Boring Method	**HSA**	Shelby Tube OD	**--** in.

TEST DATA

SOIL CLASSIFICATION (continued)	Stratum Elevation	Stratum Depth, ft	Depth Scale, ft	Sample No.	Sample Type	Groundwater	Standard Penetration Test, Blows per 6 in. Increments	Moisture Content, %	Pocket Penetrometer PP-tsf	Remarks
Dark gray, moist, soft to medium stiff, SILT (ML) with trace organics and trace fine sand				17	SS		3-4-4	38.9		
			45	18	SS	●	2-2-4	43.3		
				19	SS		4-4-4	43.5		**Sample No. SS-19:** Atterberg limits: non-plastic Passing No. 200 sieve = 92.9%
			50	20	SS		3-4-4	34.2		
				21	SS		4-4-5	27.0		
Gray and bluish gray, very moist, very soft to medium stiff, SILTY CLAY (CL)	361.5	54.0	55	22	SS		0-1-2	19.4	1.0	
				23	SS		3-4-4	19.0	2.0	
			60	24	SS		3-4-5	19.4	4.0	
Gray, severely weathered, SILTSTONE	354.0 353.3	61.5 62.2		25	SS		19-38-50/0.2			
Bottom of Test Boring at 62.2 ft										

Sample Type
- SS - Driven Split Spoon
- ST - Pressed Shelby Tube
- CA - Continuous Flight Auger
- RC - Rock Core
- CU - Cuttings
- CT - Continuous Tube

Depth to Groundwater
- ● Noted on Drilling Tools **45.0** ft.
- ▽ At Completion **--** ft.
- ▼ After **--** hours **--** ft.
- ⊠ Cave Depth **--** ft.

Boring Method
- HSA - Hollow Stem Augers
- CFA - Continuous Flight Augers
- CA - Casing Advancer
- MD - Mud Drilling
- HA - Hand Auger

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TEST BORING LOG

CLIENT **Vectren Corporation**

PROJECT NAME **Ash Pond Safety Factor Assessment**

PROJECT LOCATION **A.B. Brown Generating Facility**
Posey County, Indiana

BORING # **B-206**

JOB # **170GC00108**

DRILLING and SAMPLING INFORMATION

Date Started **4/16/15**
Date Completed **4/16/15**
Drill Foreman **W. Bates**
Inspector **S. Marcum**
Boring Method **HSA**

Hammer Wt. **140** lbs.
Hammer Drop **30** in.
Spoon Sampler OD **2.0** in.
Rock Core Dia. **--** in.
Shelby Tube OD **--** in.

TEST DATA

SOIL CLASSIFICATION SURFACE ELEVATION 414.8	Stratum Elevation	Stratum Depth, ft	Depth Scale, ft	Sample No.	Sample Type	Sampler Graphics Recovery Graphics	Groundwater	Standard Penetration Test, Blows per 6 in. Increments	Moisture Content, %	Pocket Penetrometer PP-tsf	Remarks
Reddish brown, slightly moist, sandy clay (EMBANKMENT FILL)				1	SS			9-10-10			Boring coordinates and ground surface elevation surveyed by Three I Design.
	410.8	4.0									
Brown, moist to very moist, sand with trace gravel (EMBANKMENT FILL)			5	2	SS			3-5-8			
				3	SS			8-8-4			Borehole backfilled with cement/bentonite grout.
	406.8	8.0									
Light brown, moist, silty clay (EMBANKMENT FILL)			10	4	SS			3-5-6	18.5	2.0	
				5	SS			7-6-8	17.9		
	401.8	13.0									
Gray, slightly moist, clayey silt (EMBANKMENT FILL)			15	6	SS			4-6-6	19.7		
				7	SS			9-10-8	20.8	4.5+	
	396.8	18.0									
Brown and gray, slightly moist to moist, very stiff, SILT (ML)			20	8	SS			7-11-12	21.7	1.75	
				9	SS			10-10-11	23.7		**Sample No. SS-9:** Atterberg limits: non-plastic Passing No. 200 sieve = 98.3%
	391.8	23.0									
Brown and gray, slightly moist to moist, stiff, CLAYEY SILT (ML)			25	10	SS			5-6-7	20.5		
				11	SS			6-7-7	20.7	2.5	
				12	ST		●		21.1		**Sample No. ST-12:** Atterberg limits: LL=23, PL=20, PI=3 Passing No. 200 sieve = 96.6%
	384.3	30.5	30								
Light brown and gray, moist, medium stiff to very stiff, SILTY CLAY (CL) with little sand				13	SS			3-3-3	20.9	0.75	**Sample No. SS-13:** Atterberg limits: LL=32, PL=15, PI=17 Passing No. 200 sieve = 80.3%
			35	14	SS			3-6-9	18.4	1.5	
				15	SS			7-8-8	23.2	1.5	
				16	ST				24.2		**Sample No. ST-16:** Atterberg limits: LL=29, PL=16, PI=13
	374.8	40.0									

Sample Type
SS - Driven Split Spoon
ST - Pressed Shelby Tube
CA - Continuous Flight Auger
RC - Rock Core
CU - Cuttings
CT - Continuous Tube

Depth to Groundwater
● Noted on Drilling Tools **29.5** ft.
▽ At Completion **--** ft.
▼ After **--** hours **--** ft.
▨ Cave Depth **--** ft.

Boring Method
HSA - Hollow Stem Augers
CFA - Continuous Flight Augers
CA - Casing Advancer
MD - Mud Drilling
HA - Hand Auger

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TEST BORING LOG

CLIENT **Vectren Corporation**
PROJECT NAME **Ash Pond Safety Factor Assessment**
PROJECT LOCATION **A.B. Brown Generating Facility**
Posey County, Indiana

BORING # **B-206**
JOB # **170GC00108**

DRILLING and SAMPLING INFORMATION

Date Started	**4/16/15**	Hammer Wt.	**140** lbs.
Date Completed	**4/16/15**	Hammer Drop	**30** in.
Drill Foreman	**W. Bates**	Spoon Sampler OD	**2.0** in.
Inspector	**S. Marcum**	Rock Core Dia.	**--** in.
Boring Method	**HSA**	Shelby Tube OD	**--** in.

TEST DATA

SOIL CLASSIFICATION (continued)	Stratum Elevation	Stratum Depth, ft	Depth Scale, ft	Sample No.	Sample Type	Sampler Graphics Recovery Graphics	Groundwater	Standard Penetration Test, Blows per 6 in. Increments	Moisture Content, %	Pocket Penetrometer PP-tsf	Remarks
Gray, wet medium stiff to soft, CLAYEY SILT (ML)				17	SS			3-3-3	24.6		Passing No. 200 sieve = 82.3%
Gray, moist, medium stiff to stiff, SILTY CLAY (CL)	371.8	43.0	45	18	SS			3-2-2	27.3	0.5	**Sample No. SS-17:** Atterberg limits: LL=26, PL=23, PI=3 Passing No. 200 sieve = 94.2%
				19	SS			7-10-5	40.3	0.5	**Sample No. SS-19:** Atterberg limits: LL=48, PL=23, PI=25 Passing No. 200 sieve = 99.0%
			50	20	SS			3-3-3	24.8	1.5	
				21	SS			4-5-5	32.6		
Gray, moist, very soft to soft, SILTY CLAY (CL) with little sand	360.8	54.0	55	22	SS			0-1-1	39.1		
				23	SS			3-2-3	17.7		
Gray, wet, soft to medium stiff, CLAYEY SILT (ML)	356.8	58.0	60	24	SS			2-3-3	36.9		**Sample No. SS-24:** Atterberg limits: LL=33, PL=31, PI=2 Passing No. 200 sieve = 96.4%
				25	SS			3-3-4	39.8		**Sample No. SS-25:** Atterberg limits: LL=38, PL=34, PI=4 Passing No. 200 sieve = 96.3%
			65	26	SS			2-2-3	42.5		
				27	SS			2-2-4	54.1		
			70	28	SS			2-2-2	37.8		
				29	SS			3-2-2	54.3		
Bluish gray, moist medium stiff to very stiff, SILTY CLAY (CL) with trace sand	341.8	73.0	75	30	SS			3-4-5	20.3	2.0	
				31	SS			19-13-14	18.3	4.0	
Gray, severely weathered, SILTSTONE	335.8 334.8	79.0 80.0		32	SS			18-35-50/0.4			

Sample Type
SS - Driven Split Spoon
ST - Pressed Shelby Tube
CA - Continuous Flight Auger
RC - Rock Core
CU - Cuttings
CT - Continuous Tube

Depth to Groundwater
● Noted on Drilling Tools **29.5** ft.
▽ At Completion **--** ft.
▼ After **--** hours **--** ft.
☒ Cave Depth **--** ft.

Boring Method
HSA - Hollow Stem Augers
CFA - Continuous Flight Augers
CA - Casing Advancer
MD - Mud Drilling
HA - Hand Auger

TEST BORING LOG

CLIENT **Vectren Corporation**
PROJECT NAME **Ash Pond Safety Factor Assessment**
PROJECT LOCATION **A.B. Brown Generating Facility**
Posey County, Indiana

BORING # **B-207**
JOB # **170GC00108**

DRILLING and SAMPLING INFORMATION

Date Started **4/15/15**
Date Completed **4/15/15**
Drill Foreman **W. Bates**
Inspector **S. Marcum**
Boring Method **HSA**

Hammer Wt. **140** lbs.
Hammer Drop **30** in.
Spoon Sampler OD **2.0** in.
Rock Core Dia. **--** in.
Shelby Tube OD **--** in.

TEST DATA

SOIL CLASSIFICATION SURFACE ELEVATION 395.0	Stratum Elevation	Stratum Depth, ft	Depth Scale, ft	Sample No.	Sample Type	Sampler Graphics Recovery Graphics	Groundwater	Standard Penetration Test, Blows per 6 in. Increments	Moisture Content, %	Pocket Penetrometer PP-tsf	Remarks
Brown, moist, silty clay with coal ash (EMBANKMENT FILL)				1	SS	☒		3-3-4			Boring coordinates and ground surface elevation surveyed by Three I Design.
	392.0	3.0									
Brown, moist, silty clay (EMBANKMENT FILL)				2	SS	☒		5-5-4	18.0		
			5								Borehole backfilled with cement/bentonite grout.
	388.0	7.0		3	SS	☒		7-8-11	16.1	3.0	
Reddish brown, moist, silty clay (EMBANKMENT FILL)	387.0	8.0									
Brown and gray, moist, silty clay (EMBANKMENT FILL)				4	SS	☒		5-5-6	18.9		
			10								
				5	SS	☒		10-9-9	18.4	3.0	
	382.0	13.0									
Brown and gray, moist, very soft to medium stiff, SILTY CLAY (CL-ML)				6	SS	☒		2-1-2	23.9	0.75	
			15								
				7	SS	☒		6-6-4	20.4	0.25	**Sample No. SS-7:** Atterberg limits: LL=24, PL=19, PI=5 Passing No. 200 sieve = 94.8%
	376.5	18.5									
Bluish gray, moist, medium stiff to soft, SILTY CLAY (CL)				8	ST	■			23.4		**Sample No. ST-8:** Atterberg limits: LL=31, PL=16, PI=15 Passing No. 200 sieve = 92.9%
			20								
				9	SS	☒		3-3-3	27.4	1.0	
				10	SS	☒		2-2-3	28.6	1.0	
			25								
				11	SS	☒		3-2-3	25.7	1.25	
	366.0	29.0									
Gray, wet, medium stiff, CLAYEY SILT (ML)				12	SS	☒	●	3-4-3	27.1		
			30								
				13	SS	☒		2-3-2	26.7		**Sample No. SS-13:** Atterberg limits: non-plastic Passing No. 200 sieve = 95.2%
				14	SS	☒		1-4-4	24.3		
			35	15	ST	■			32.0		**Sample No. ST-15:** Atterberg limits: LL=31, PL=25, PI=6 Passing No. 200 sieve = 73.5%
	357.0	38.0									
Gray, very moist, medium stiff, SANDY CLAY (CL)				16	SS	☒		3-3-4	17.6	0.5	

Sample Type
SS - Driven Split Spoon
ST - Pressed Shelby Tube
CA - Continuous Flight Auger
RC - Rock Core
CU - Cuttings
CT - Continuous Tube

Depth to Groundwater
● Noted on Drilling Tools **29.0** ft.
▽ At Completion -- ft.
▼ After -- hours -- ft.
▨ Cave Depth -- ft.

Boring Method
HSA - Hollow Stem Augers
CFA - Continuous Flight Augers
CA - Casing Advancer
MD - Mud Drilling
HA - Hand Auger

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TEST BORING LOG

CLIENT **Vectren Corporation**

PROJECT NAME **Ash Pond Safety Factor Assessment**

PROJECT LOCATION **A.B. Brown Generating Facility**
Posey County, Indiana

BORING # **B-207**

JOB # **170GC00108**

DRILLING and SAMPLING INFORMATION

Date Started **4/15/15**
Date Completed **4/15/15**
Drill Foreman **W. Bates**
Inspector **S. Marcum**
Boring Method **HSA**

Hammer Wt. **140** lbs.
Hammer Drop **30** in.
Spoon Sampler OD **2.0** in.
Rock Core Dia. **--** in.
Shelby Tube OD **--** in.

TEST DATA

SOIL CLASSIFICATION (continued)	Stratum Elevation	Stratum Depth, ft	Depth Scale, ft	Sample No.	Sample Type	Sampler Graphics Recovery Graphics	Groundwater	Standard Penetration Test, Blows per 6 in. Increments	Moisture Content, %	Pocket Penetrometer PP-tsf	Remarks
Gray, very moist, medium stiff, SANDY CLAY (CL)	352.0	43.0		17	SS			2-3-4	19.5		**Sample No. SS-16:** Atterberg limits: LL=30, PL=15, PI=15 Passing No. 200 sieve = 61.7%
Gray, wet, dense, SILTY SAND (SM)	350.0	45.0	45	18	SS			11-13-23			
Gray, severely weathered, SILTSTONE	347.9	47.1		19	SS			31-41-50/0.1			
Bottom of Test Boring at 47.1 ft											

Sample Type
SS - Driven Split Spoon
ST - Pressed Shelby Tube
CA - Continuous Flight Auger
RC - Rock Core
CU - Cuttings
CT - Continuous Tube

Depth to Groundwater
● Noted on Drilling Tools **29.0** ft.
▽ At Completion **--** ft.
▼ After **--** hours **--** ft.
☒ Cave Depth **--** ft.

Boring Method
HSA - Hollow Stem Augers
CFA - Continuous Flight Augers
CA - Casing Advancer
MD - Mud Drilling
HA - Hand Auger

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TEST BORING LOG

CLIENT **Vectren Corporation**
PROJECT NAME **Ash Pond Safety Factor Assessment**
PROJECT LOCATION **A.B. Brown Generating Facility**
Posey County, Indiana

BORING # **B-208**
JOB # **170GC00108**

DRILLING and SAMPLING INFORMATION

Date Started **4/15/15**
Date Completed **4/15/15**
Drill Foreman **W. Bates**
Inspector **S. Marcum**
Boring Method **HSA**

Hammer Wt. **140** lbs.
Hammer Drop **30** in.
Spoon Sampler OD **2.0** in.
Rock Core Dia. **--** in.
Shelby Tube OD **--** in.

TEST DATA

SOIL CLASSIFICATION SURFACE ELEVATION 396.7	Stratum Elevation	Stratum Depth, ft	Depth Scale, ft	Sample No.	Sample Type	Sampler Graphics Recovery Graphics	Groundwater	Standard Penetration Test, Blows per 6 in. Increments	Moisture Content, %	Pocket Penetrometer PP-tsf	Remarks
Black, coal ash (EMBANKMENT FILL)	395.2	1.5									Boring coordinates and ground surface elevation surveyed by Three I Design.
Brown, moist, silty clay (EMBANKMENT FILL)				1	SS			5-8-8			
				2	SS			6-7-8	16.8		
			5								
				3	SS			12-14-15	19.6	2.5	Borehole backfilled with cement/bentonite grout.
				4	SS			8-9-10	18.3		
			10								
				5	SS			10-9-9	20.4	1.5	
	383.7	13.0									
Brown, slightly moist to moist, very stiff to medium stiff, CLAYEY SILT (ML)				6	SS			3-4-4	20.5		
			15								
				7	SS			4-4-3	26.3		**Sample No. SS-7:** Atterberg limits: LL=26, PL=22, PI=4 Passing No. 200 sieve = 99.7%
	378.7	18.0									
Dark gray, moist to very moist, medium stiff, CLAYEY SILT (ML) with trace fine sand and trace organics				8	SS			3-4-5	35.6		
			20								
				9	SS			4-3-5	36.8		
				10	SS			2-3-3	37.4		
			25								
				11	SS			3-3-4	36.9		
				12	SS			2-3-4	29.8		
			30								
				13	SS			5-5-5	27.6		**Sample No. SS-13:** Atterberg limits: LL=28, PL=24, PI=4 Passing No. 200 sieve = 99.6%
				14	SS			3-3-4	18.1		
	361.7	35.0	35								
Bluish gray, moist, medium stiff, SILTY CLAY (CL) with little sand				15	SS			4-5-5	18.8	0.75	**Sample No. SS-15:** Atterberg limits: LL=33, PL=16, PI=17 Passing No. 200 sieve = 84.1%
				16	SS			3-5-4	22.2	1.5	

Sample Type
SS - Driven Split Spoon
ST - Pressed Shelby Tube
CA - Continuous Flight Auger
RC - Rock Core
CU - Cuttings
CT - Continuous Tube

Depth to Groundwater
● Noted on Drilling Tools **44.0** ft.
▽ At Completion **--** ft.
▼ After **--** hours **--** ft.
▨ Cave Depth **--** ft.

Boring Method
HSA - Hollow Stem Augers
CFA - Continuous Flight Augers
CA - Casing Advancer
MD - Mud Drilling
HA - Hand Auger

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TEST BORING LOG

CLIENT **Vectren Corporation**
PROJECT NAME **Ash Pond Safety Factor Assessment**
PROJECT LOCATION **A.B. Brown Generating Facility**
Posey County, Indiana
BORING # **B-208**
JOB # **170GC00108**

DRILLING and SAMPLING INFORMATION

Date Started **4/15/15**
Date Completed **4/15/15**
Drill Foreman **W. Bates**
Inspector **S. Marcum**
Boring Method **HSA**
Hammer Wt. **140** lbs.
Hammer Drop **30** in.
Spoon Sampler OD **2.0** in.
Rock Core Dia. **--** in.
Shelby Tube OD **--** in.

TEST DATA

SOIL CLASSIFICATION (continued)	Stratum Elevation	Stratum Depth, ft	Depth Scale, ft	Sample No.	Sample Type	Sampler Graphics Recovery Graphics	Groundwater	Standard Penetration Test, Blows per 6 in. Increments	Moisture Content, %	Pocket Penetrometer PP-tsf	Remarks
Bluish gray, moist, medium stiff, SILTY CLAY (CL) with little sand	354.5	42.2		17	SS	☒ ■		7-7-7			
Gray, wet, medium dense, SILTY SAND (SM)	352.7	44.0		18	SS	☒ ■	●	15-50/0.3			
Gray, severely weathered, SILTSTONE	351.7	45.0	45								
Bottom of Test Boring at 45.0 ft											

Sample Type
SS - Driven Split Spoon
ST - Pressed Shelby Tube
CA - Continuous Flight Auger
RC - Rock Core
CU - Cuttings
CT - Continuous Tube

Depth to Groundwater
● Noted on Drilling Tools **44.0** ft.
▽ At Completion **--** ft.
▼ After **--** hours **--** ft.
☒ Cave Depth **--** ft.

Boring Method
HSA - Hollow Stem Augers
CFA - Continuous Flight Augers
CA - Casing Advancer
MD - Mud Drilling
HA - Hand Auger

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TEST BORING LOG

CLIENT **Vectren Corporation**
PROJECT NAME **Ash Pond Safety Factor Assessment**
PROJECT LOCATION **A.B. Brown Generating Facility**
Posey County, Indiana

BORING # **B-209**
JOB # **170GC00108**

DRILLING and SAMPLING INFORMATION

Date Started **6/30/15**
Date Completed **6/30/15**
Drill Foreman **J. Cook**
Inspector **M. Foye**
Boring Method **HSA**

Hammer Wt. **140** lbs.
Hammer Drop **30** in.
Spoon Sampler OD **2.0** in.
Rock Core Dia. **--** in.
Shelby Tube OD **--** in.

TEST DATA

SOIL CLASSIFICATION SURFACE ELEVATION 451	Stratum Elevation	Stratum Depth, ft	Depth Scale, ft	Sample No.	Sample Type	Sampler Graphics Recovery Graphics	Groundwater	Standard Penetration Test, Blows per 6 in. Increments	Moisture Content, %	Pocket Penetrometer PP-tsf	Remarks
Topsoil and Crushed Stone	450.5	0.5									
Reddish brown, slightly moist, sandy clay (EMBANKMENT FILL)				1	SS			8-9-9			Ground surface elevation estimated from available topographic data.
				2	SS			5-8-9			
	445.5	5.5	5								
Reddish brown and gray, moist, sandy clay (EMBANKMENT FILL)				3	SS			8-9-7	20.8	2.5	Borehole backfilled with cement/bentonite grout.
	443.0	8.0									
Brown and gray, moist, silty clay (EMBANKMENT FILL)				4	SS			7-7-9		1.5	
			10								
				5	SS			8-11-9	19.7	3.0	
				6	SS			6-5-9		3.0	
			15								
				7	SS			11-13-12			
	433.0	18.0									
Brown, moist, silt (EMBANKMENT FILL)				8	SS			7-6-9			
			20								
				9	SS			4-8-6			
	428.0	23.0									
Light brown, moist, clayey silt (EMBANKMENT FILL)				10	SS			7-9-9		4.0	
			25								
	425.0	26.0									
Red and brown, moist, sandy clay (EMBANKMENT FILL)				11	SS			6-6-7	18.0		
	422.0	29.0									
Reddish brown and gray, moist, clayey sand (EMBANKMENT FILL)				12	SS			5-8-13	15.9	4.5+	
	420.5	30.5	30								
Brown, moist, silt (EMBANKMENT FILL)				13	SS			7-8-9			
	418.0	33.0									
Brown, moist, sandy clay (EMBANKMENT FILL)				14	SS			7-12-10	15.6	3.0	
			35								
				15	SS			5-8-11	18.7		**Sample No. SS-16:** Atterberg limits: LL=25, PL=14, PI=11
				16	SS			9-11-10	14.7		

Sample Type
- SS - Driven Split Spoon
- ST - Pressed Shelby Tube
- CA - Continuous Flight Auger
- RC - Rock Core
- CU - Cuttings
- CT - Continuous Tube

Depth to Groundwater
- ● Noted on Drilling Tools **47.3** ft.
- ▽ At Completion **--** ft.
- ▼ After **--** hours **--** ft.
- ▨ Cave Depth **--** ft.

Boring Method
- HSA - Hollow Stem Augers
- CFA - Continuous Flight Augers
- CA - Casing Advancer
- MD - Mud Drilling
- HA - Hand Auger

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TEST BORING LOG

CLIENT **Vectren Corporation**
PROJECT NAME **Ash Pond Safety Factor Assessment**
PROJECT LOCATION **A.B. Brown Generating Facility**
Posey County, Indiana
BORING # **B-209**
JOB # **170GC00108**

DRILLING and SAMPLING INFORMATION

Date Started **6/30/15**
Date Completed **6/30/15**
Drill Foreman **J. Cook**
Inspector **M. Foye**
Boring Method **HSA**
Hammer Wt. **140** lbs.
Hammer Drop **30** in.
Spoon Sampler OD **2.0** in.
Rock Core Dia. **--** in.
Shelby Tube OD **--** in.

TEST DATA

SOIL CLASSIFICATION (continued)	Stratum Elevation	Stratum Depth, ft	Depth Scale, ft	Sample No.	Sample Type	Sampler Graphics Recovery Graphics	Groundwater	Standard Penetration Test, Blows per 6 in. Increments	Moisture Content, %	Pocket Penetrometer PP-tsf	Remarks
Brown, moist, sandy clay (EMBANKMENT FILL)				17	SS			2-3-4	20.1	1.0	
				18	SS			2-2-4	20.1		
	405.5	45.5	45								
Brown, wet, very soft to soft SILT (ML)				19	SS		●	2-2-2	29.3		**Sample No. SS-19:** Atterberg limits: Non-plastic
				20	SS			1-2-1			
			50								
				21	SS			1-1-3			
	398.0	53.0									
Gray, moist, soft, SILTY CLAY (CL)				22	SS			1-2-2	24.0	1.0	
			55								
	395.0	56.0									
Brown, very moist, soft to stiff, SILTY CLAY (CL)				23	SS			2-2-3	29.2	1.5	**Sample No. SS-23:** Atterberg limits: LL=38, PL=18, PI=20
				24	SS			4-4-5	28.0		
			60								
				25	SS			6-6-7	26.4		
				26	SS			4-6-6	21.0	1.5	
	385.5	65.5	65								
Brown, moist, medium stiff, SANDY CLAY (CL)				27	SS			4-4-6		1.75	
	381.5	69.5		28	SS			5-20-50/0.2			
Reddish brown and gray, weathered, SANDSTONE	381.3	69.7	70								
Bottom of Test Boring at 69.7 ft											

Sample Type
SS - Driven Split Spoon
ST - Pressed Shelby Tube
CA - Continuous Flight Auger
RC - Rock Core
CU - Cuttings
CT - Continuous Tube

Depth to Groundwater
● Noted on Drilling Tools **47.3** ft.
▽ At Completion **--** ft.
▼ After **--** hours **--** ft.
▨ Cave Depth **--** ft.

Boring Method
HSA - Hollow Stem Augers
CFA - Continuous Flight Augers
CA - Casing Advancer
MD - Mud Drilling
HA - Hand Auger

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TEST BORING LOG

CLIENT **Vectren Corporation**
PROJECT NAME **Ash Pond Safety Factor Assessment**
PROJECT LOCATION **A.B. Brown Generating Facility**
Posey County, Indiana

BORING # **B-210**
JOB # **170GC00108**

DRILLING and SAMPLING INFORMATION

Date Started **7/1/15**
Date Completed **7/1/15**
Drill Foreman **J. Cook**
Inspector **M. Foye**
Boring Method **HSA**

Hammer Wt. **140** lbs.
Hammer Drop **30** in.
Spoon Sampler OD **2.0** in.
Rock Core Dia. **--** in.
Shelby Tube OD **--** in.

TEST DATA

SOIL CLASSIFICATION SURFACE ELEVATION 451	Stratum Elevation	Stratum Depth, ft	Depth Scale, ft	Sample No.	Sample Type	Sampler Graphics Recovery Graphics	Groundwater	Standard Penetration Test, Blows per 6 in. Increments	Moisture Content, %	Pocket Penetrometer PP-tsf	Remarks
Topsoil and Crushed Stone	450.5	0.5									Ground surface elevation estimated from available topographic data.
Reddish brown, slightly moist, sandy clay (EMBANKMENT FILL)				1	SS			5-5-9			
	445.5	5.5	5	2	SS			6-6-7		3.0	
Brown and gray, slightly moist to moist, silty clay (EMBANKMENT FILL)				3	SS			5-6-11	18.4	2.5	Borehole backfilled with cement/bentonite grout.
			10	4	SS			6-6-7		2.0	
	438.0	13.0		5	SS			10-11-9	21.1		
Tan, slightly moist, clayey silt (EMBANKMENT FILL)			15	6	SS			6-6-9			
	432.0	19.0		7	SS			5-9-11			
Tan, slightly moist, sandy silt (EMBANKMENT FILL)	430.0	21.0	20	8	SS			7-9-11			
Tan and gray, moist, sandy clay (EMBANKMENT FILL)	427.5	23.5		9	SS			6-5-8		3.0	
Brown, moist, silt (EMBANKMENT FILL)	425.0	26.0	25	10	SS			3-4-10			
Brown, moist, silty clay (EMBANKMENT FILL)				11	SS			4-5-6	21.7	2.0	
			30	12	SS			4-6-7	18.9	1.75	
	417.0	34.0		13	SS			3-4-6	16.0	2.0	**Sample No. SS-13:** Atterberg limits: LL=37, PL=17, PI=26
Gray, moist, clayey silt (EMBANKMENT FILL)	415.0	36.0	35	14	SS			8-11-13	17.3		
Tan and gray, moist, sandy clay (EMBANKMENT FILL)				15	SS			5-5-7	25.8		
				16	SS			4-4-6	13.6	3.5	

Sample Type
SS - Driven Split Spoon
ST - Pressed Shelby Tube
CA - Continuous Flight Auger
RC - Rock Core
CU - Cuttings
CT - Continuous Tube

Depth to Groundwater
● Noted on Drilling Tools **54.5** ft.
▽ At Completion **--** ft.
▼ After **--** hours **--** ft.
▨ Cave Depth **--** ft.

Boring Method
HSA - Hollow Stem Augers
CFA - Continuous Flight Augers
CA - Casing Advancer
MD - Mud Drilling
HA - Hand Auger

TEST BORING LOG

CLIENT **Vectren Corporation**

PROJECT NAME **Ash Pond Safety Factor Assessment**

PROJECT LOCATION **A.B. Brown Generating Facility**
Posey County, Indiana

BORING # **B-210**

JOB # **170GC00108**

DRILLING and SAMPLING INFORMATION

Date Started **7/1/15**
Date Completed **7/1/15**
Drill Foreman **J. Cook**
Inspector **M. Foye**
Boring Method **HSA**

Hammer Wt. **140** lbs.
Hammer Drop **30** in.
Spoon Sampler OD **2.0** in.
Rock Core Dia. **--** in.
Shelby Tube OD **--** in.

TEST DATA

SOIL CLASSIFICATION (continued)	Stratum Elevation	Stratum Depth, ft	Depth Scale, ft	Sample No.	Sample Type	Groundwater	Standard Penetration Test, Blows per 6 in. Increments	Moisture Content, %	Pocket Penetrometer PP-tsf	Remarks
Tan and gray, moist sandy clay (EMBANKMENT FILL)	407.5	43.5		17	SS		6-6-8	17.5	2.0	**Sample No. SS-17:** Atterberg limits: LL=35, PL=13, PI=22
Reddish brown, moist, silty clay (EMBANKMENT FILL)			45	18	SS		2-4-5	20.5	1.5	
				19	SS		5-5-5	14.8	2.5	
			50	20	SS		5-7-10	17.8	1.5	**Sample No. SS-20:** Atterberg limits: LL=27, PL=16, PI=11
	398.0	53.0		21	SS		4-5-6	18.7	1.0	
Brown, wet, stiff to soft, CLAYEY SILT (ML)			55	22	SS	●	5-7-7			
				23	SS		5-6-7	24.5		**Sample No. SS-23:** Atterberg limits: LL=26, PL=23, PI=3
			60	24	SS		4-4-4			
				25	SS		1-2-2			
	385.5	65.5	65	26	SS		2-3-3			
Brown, moist, medium stiff to stiff, SILTY CLAY (CL)				27	SS		12-3-5	20.2	1.5	**Sample No. SS-27:** Atterberg limits: LL=26, PL=17, PI=9
	381.0	70.0	70	28	SS		2-4-7		2.0	
Bottom of Test Boring at 70.0 ft										

Sample Type
SS - Driven Split Spoon
ST - Pressed Shelby Tube
CA - Continuous Flight Auger
RC - Rock Core
CU - Cuttings
CT - Continuous Tube

Depth to Groundwater
● Noted on Drilling Tools **54.5** ft.
▽ At Completion **--** ft.
▼ After **--** hours **--** ft.
▨ Cave Depth **--** ft.

Boring Method
HSA - Hollow Stem Augers
CFA - Continuous Flight Augers
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TEST BORING LOG

CLIENT **Vectren Corporation**
PROJECT NAME **Ash Pond Safety Factor Assessment**
PROJECT LOCATION **A.B. Brown Generating Facility**
Posey County, Indiana
BORING # **B-211**
JOB # **170GC00108**

DRILLING and SAMPLING INFORMATION

Date Started	**7/2/15**	Hammer Wt.	**140** lbs.
Date Completed	**7/2/15**	Hammer Drop	**30** in.
Drill Foreman	**J. Cook**	Spoon Sampler OD	**2.0** in.
Inspector	**M. Foye**	Rock Core Dia.	**--** in.
Boring Method	**HSA**	Shelby Tube OD	**--** in.

TEST DATA

SOIL CLASSIFICATION SURFACE ELEVATION 451	Stratum Elevation	Stratum Depth, ft	Depth Scale, ft	Sample No.	Sample Type	Sampler Graphics Recovery Graphics	Groundwater	Standard Penetration Test, Blows per 6 in. Increments	Moisture Content, %	Pocket Penetrometer PP-tsf	Remarks
▨ Topsoil and Crushed Stone	450.5	0.5									
▨ Brown, slightly moist, silty clay (EMBANKMENT FILL)				1	SS	☒ ■		2-4-3			Ground surface elevation estimated from available topographic data.
				2	SS	☒ ■		6-7-9			
			5								
				3	SS	☒ ■		8-7-10		4.5	Borehole backfilled with cement/bentonite grout.
				4	SS	☒ ■		5-6-8	21.0	1.5	
			10								
				5	SS	☒ ■		7-8-8		4.0	
▨ Reddish brown, brown and gray, moist, silty clay (EMBANKMENT FILL)	438.0	13.0									
				6	SS	☒ ■		4-6-7	17.1	3.0	
			15								
				7	SS	☒ ■		5-8-8		4.0	
				8	SS	☒ ■		5-6-9	15.3		
			20								
				9	SS	☒ ■		7-9-11			
▨ Brown and gray, moist clayey silt (EMBANKMENT FILL)	428.0	23.0									
				10	SS	☒ ■		4-8-9			
▨ Brown, moist, silty clay (EMBANKMENT FILL)	425.5	25.5	25								
				11	SS	☒ ■		3-4-4	19.0		**Sample No. SS-11:** Atterberg limits: LL=31, PL=17, PI=14
				12	SS	☒ ■		4-7-7	16.8		
			30								
				13	SS	☒ ■		3-4-5	16.2		
				14	SS	☒ ■		3-5-6	21.2		
			35								
				15	SS	☒ ■		3-4-7	17.7		**Sample No. SS-15:** Atterberg limits: LL=30, PL=17, PI=13
▨ Brown, moist silty clay with interbedded red, sandy clay (EMBANKMENT FILL)	413.0	38.0									
				16	SS	☒ ■		8-8-16	18.8		

Sample Type
- SS - Driven Split Spoon
- ST - Pressed Shelby Tube
- CA - Continuous Flight Auger
- RC - Rock Core
- CU - Cuttings
- CT - Continuous Tube

Depth to Groundwater
- ● Noted on Drilling Tools **61.0** ft.
- ▽ At Completion **--** ft.
- ▼ After **--** hours **--** ft.
- ⊠ Cave Depth **--** ft.

Boring Method
- HSA - Hollow Stem Augers
- CFA - Continuous Flight Augers
- CA - Casing Advancer
- MD - Mud Drilling
- HA - Hand Auger

TEST BORING LOG

CLIENT **Vectren Corporation**

PROJECT NAME **Ash Pond Safety Factor Assessment**

PROJECT LOCATION **A.B. Brown Generating Facility**
Posey County, Indiana

BORING # **B-211**

JOB # **170GC00108**

DRILLING and SAMPLING INFORMATION

Date Started	**7/2/15**	Hammer Wt.	**140** lbs.
Date Completed	**7/2/15**	Hammer Drop	**30** in.
Drill Foreman	**J. Cook**	Spoon Sampler OD	**2.0** in.
Inspector	**M. Foye**	Rock Core Dia.	**--** in.
Boring Method	**HSA**	Shelby Tube OD	**--** in.

TEST DATA

SOIL CLASSIFICATION (continued)	Stratum Elevation	Stratum Depth, ft	Depth Scale, ft	Sample No.	Sample Type	Sampler Graphics Recovery Graphics	Groundwater	Standard Penetration Test, Blows per 6 in. Increments	Moisture Content, %	Pocket Penetrometer PP-tsf	Remarks
Brown, moist silty clay with interbedded red, sandy clay (EMBANKMENT FILL)				17	SS	☒ ■		4-3-6		1.5	
			45	18	SS	☒ ■		3-5-7	16.7	2.0	
				19	SS	☒ ■		4-6-8	19.8	1.5	
	400.5	50.5	50	20	SS	☒ ■		3-6-10	17.3	1.5	
Brown, moist to very moist, very stiff to stiff, SILTY CLAY (CL)				21	SS	☒ ■		5-8-12			**Sample No. SS-21:** Atterberg limits: LL=29, PL=19, PI=10
			55	22	SS	☒ ■		6-7-7			
				23	SS	☒ ■		6-8-9			
			60	24	SS	☒ ■		4-7-8	20.7		**Sample No. SS-24:** Atterberg limits: LL=30, PL=20, PI=10
	387.0	64.0		25	SS	☒ ■	●	9-8-7			
Gray, wet, medium stiff, SILT (ML)			65	26	SS	☒ ■		3-4-4			
				27	SS	☒ ■		2-2-4			
	381.0	70.0	70	28	SS	☒ ■		1-3-3	29.9		**Sample No. SS-28:** Atterberg limits: Non-plastic
Bottom of Test Boring at 70.0 ft											

Sample Type
SS - Driven Split Spoon
ST - Pressed Shelby Tube
CA - Continuous Flight Auger
RC - Rock Core
CU - Cuttings
CT - Continuous Tube

Depth to Groundwater
● Noted on Drilling Tools **61.0** ft.
▽ At Completion **--** ft.
▼ After **--** hours **--** ft.
▨ Cave Depth **--** ft.

Boring Method
HSA - Hollow Stem Augers
CFA - Continuous Flight Augers
CA - Casing Advancer
MD - Mud Drilling
HA - Hand Auger

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TEST BORING LOG

CLIENT **Vectren Corporation**

PROJECT NAME **Ash Pond Safety Factor Assessment**

PROJECT LOCATION **A.B. Brown Generating Facility**
Posey County, Indiana

BORING # **B-212**

JOB # **170GC00108**

DRILLING and SAMPLING INFORMATION

Date Started	**7/9/15**	Hammer Wt.	**140** lbs.
Date Completed	**7/10/15**	Hammer Drop	**30** in.
Drill Foreman	**J. Cook**	Spoon Sampler OD	**2.0** in.
Inspector	**M. Foye**	Rock Core Dia.	**--** in.
Boring Method	**HSA**	Shelby Tube OD	**--** in.

TEST DATA

SOIL CLASSIFICATION SURFACE ELEVATION 451	Stratum Elevation	Stratum Depth, ft	Depth Scale, ft	Sample No.	Sample Type	Sampler Graphics Recovery Graphics	Groundwater	Standard Penetration Test, Blows per 6 in. Increments	Moisture Content, %	Pocket Penetrometer PP-tsf	Remarks
Topsoil and Crushed Stone	450.5	0.5									
Brown, slightly moist, silty clay (EMBANKMENT FILL)				1	SS			7-6-6			Ground surface elevation estimated from available topographic data.
				2	SS			6-6-7			
			5	3	SS			8-8-10	19.3	4.5	Borehole backfilled with cement/bentonite grout.
	443.0	8.0									
Brown, moist, clayey silt (EMBANKMENT FILL)				4	SS			7-14-21			
			10	5	SS			8-10-12			Installed piezometer.
	438.0	13.0									
Brown, moist, silty clay (EMBANKMENT FILL)				6	SS			8-7-8	15.1	3.5	
			15	7	SS			6-6-8	17.6	1.5	
	432.5	18.5									
Tan, slightly moist, sandy silt (EMBANKMENT FILL)				8	SS			11-7-10			
	430.5	20.5	20								
Brown and gray, moist, silty clay (EMBANKMENT FILL)				9	SS			6-9-10	18.1		
				10	SS			7-9-9	15.6		
	425.0	26.0	25								
Gray, moist, silty clay (EMBANKMENT FILL)				11	SS			4-4-6	16.2		**Sample No. SS-11:** Atterberg limits: LL=38, PL=19, PI=19
	422.0	29.0									
Reddish brown, moist, silty clay (EMBANKMENT FILL)				12	SS			6-7-12	19.5		
			30	13	SS			6-7-12	17.1		
			35	14	SS			7-7-8	15.4	2.5	**Sample No. SS-14:** Atterberg limits: LL=34, PL=14, PI=20
				15	SS		▼	6-10-16	16.8		
	411.0	40.0		16	SS			7-5-7	16.6	1.5	

Sample Type
- SS - Driven Split Spoon
- ST - Pressed Shelby Tube
- CA - Continuous Flight Auger
- RC - Rock Core
- CU - Cuttings
- CT - Continuous Tube

Depth to Groundwater
- ● Noted on Drilling Tools **68.0** ft.
- ▽ At Completion **--** ft.
- ▼ After **1152** hours **35.7** ft.
- ▨ Cave Depth **--** ft.

Boring Method
- HSA - Hollow Stem Augers
- CFA - Continuous Flight Augers
- CA - Casing Advancer
- MD - Mud Drilling
- HA - Hand Auger

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TEST BORING LOG

CLIENT **Vectren Corporation**

PROJECT NAME **Ash Pond Safety Factor Assessment**

PROJECT LOCATION **A.B. Brown Generating Facility**
Posey County, Indiana

BORING # **B-212**

JOB # **170GC00108**

DRILLING and SAMPLING INFORMATION

Date Started **7/9/15**
Date Completed **7/10/15**
Drill Foreman **J. Cook**
Inspector **M. Foye**
Boring Method **HSA**

Hammer Wt. **140** lbs.
Hammer Drop **30** in.
Spoon Sampler OD **2.0** in.
Rock Core Dia. **--** in.
Shelby Tube OD **--** in.

SOIL CLASSIFICATION (continued)	Stratum Elevation	Stratum Depth, ft
Light brown, brown and gray, moist, silty clay (EMBANKMENT FILL)	405.0	46.0
Tan, reddish brown and gray, moist, stiff, SANDY CLAY (CL)	401.5	49.5
Gray, moist, very stiff, SILTY CLAY (CL) with trace organic matter	398.0	53.0
Brown and gray, moist, very stiff, SILTY CLAY (CL)	395.0	56.0
Reddish brown, moist, stiff to very stiff, SANDY CLAY (CL)	388.0	63.0
Gray, moist, very stiff, CLAYEY SILT (ML)	383.0	68.0
Reddish brown, moist, stiff to very stiff, SANDY CLAY (CL)	381.3	69.7
Red, wet, medium dense, SAND (SP)	381.0	70.0
Bottom of Test Boring at 70.0 ft		

TEST DATA

Depth Scale, ft	Sample No.	Sample Type	Groundwater	Standard Penetration Test, Blows per 6 in. Increments	Moisture Content, %	Pocket Penetrometer PP-tsf	Remarks
	17	SS		3-4-6	17.7	3.5	
45	18	SS		5-6-6	15.4		
	19	SS		4-5-8	18.8		
50	20	SS		6-9-13			
	21	SS		6-8-12			
55	22	SS		7-21-12	20.6		**Sample No. SS-22:** Atterberg limits: LL=27, PL=17, PI=10
	23	SS		3-5-8	15.0		
60	24	SS		8-10-8	16.6		
	25	SS		4-7-7			
65	26	SS		7-8-10	20.5		
	27	SS	●	5-6-6	22.2		**Sample No. SS-27:** Atterberg limits: LL=25, PL=22, PI=3
70	28	SS		4-6-7			

Sample Type
- SS - Driven Split Spoon
- ST - Pressed Shelby Tube
- CA - Continuous Flight Auger
- RC - Rock Core
- CU - Cuttings
- CT - Continuous Tube

Depth to Groundwater
- ● Noted on Drilling Tools **68.0** ft.
- ▽ At Completion **--** ft.
- ▼ After **1152** hours **35.7** ft.
- ▧ Cave Depth **--** ft.

Boring Method
- HSA - Hollow Stem Augers
- CFA - Continuous Flight Augers
- CA - Casing Advancer
- MD - Mud Drilling
- HA - Hand Auger

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TEST BORING LOG

CLIENT **Vectren Corporation**

PROJECT NAME **Ash Pond Safety Factor Assessment**

PROJECT LOCATION **A.B. Brown Generating Facility**
Posey County, Indiana

BORING # **B-213**

JOB # **170GC00108**

DRILLING and SAMPLING INFORMATION

Date Started	**7/8/15**	Hammer Wt.	**140** lbs.
Date Completed	**7/9/15**	Hammer Drop	**30** in.
Drill Foreman	**J. Cook**	Spoon Sampler OD	**2.0** in.
Inspector	**M. Foye**	Rock Core Dia.	**--** in.
Boring Method	**HSA**	Shelby Tube OD	**--** in.

TEST DATA

SOIL CLASSIFICATION SURFACE ELEVATION 451	Stratum Elevation	Stratum Depth, ft	Depth Scale, ft	Sample No.	Sample Type	Sampler Graphics Recovery Graphics	Groundwater	Standard Penetration Test, Blows per 6 in. Increments	Moisture Content, %	Pocket Penetrometer PP-tsf	Remarks
Topsoil and Crushed Stone	450.5	0.5									
Tan, slightly moist, silty clay (EMBANKMENT FILL)	448.0	3.0		1	SS	☒■		4-6-7		3.0	Ground surface elevation estimated from available topographic data.
Reddish brown and gray, moist, silty clay (EMBANKMENT FILL)	445.5	5.5	5	2	SS	☒■		4-6-8	18.8		
Brown, moist, silt (EMBANKMENT FILL)				3	SS	☒■		6-9-13			Borehole backfilled with cement/bentonite grout.
	440.5	10.5	10	4	SS	☒■		10-9-5	19.7		
Brown, moist, silty clay (EMBANKMENT FILL)				5	SS	☒■		5-10-10		3.0	
	435.5	15.5	15	6	SS	☒■		5-4-6		2.5	
Brown, moist clayey silt (EMBANKMENT FILL)	433.0	18.0		7	SS	☒■		7-8-9			
Brown and light brown, moist, silty clay (EMBANKMENT FILL)			20	8	SS	☒■		4-5-5		4.5	
				9	SS	☒■		5-6-8	14.6	4.0	**Sample No. SS-9:** Atterberg limits: LL=29, PL=16, PI=13
			25	10	SS	☒■		4-5-4			
	423.0	28.0		11	SS	☒■		4-3-5		2.0	
Brown, moist, clayey silt (EMBANKMENT FILL)			30	12	SS	☒■		4-3-4	18.4		**Sample No. SS-12:** Atterberg limits: LL=24, PL=21, PI=3
	418.0	33.0		13	SS	☒■		6-6-10	16.2		
Brown, moist, silty clay (EMBANKMENT FILL)			35	14	SS	☒■		5-5-6	20.4	3.5	
				15	SS	☒■		6-7-10	16.0	2.0	
	411.0	40.0		16	SS	☒■		3-7-7	15.9	3.0	

Sample Type

SS - Driven Split Spoon
ST - Pressed Shelby Tube
CA - Continuous Flight Auger
RC - Rock Core
CU - Cuttings
CT - Continuous Tube

Depth to Groundwater

● Noted on Drilling Tools **67.1** ft.
▽ At Completion **--** ft.
▼ After **--** hours **--** ft.
☒ Cave Depth **--** ft.

Boring Method

HSA - Hollow Stem Augers
CFA - Continuous Flight Augers
CA - Casing Advancer
MD - Mud Drilling
HA - Hand Auger

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TEST BORING LOG

CLIENT **Vectren Corporation**

BORING # **B-213**

PROJECT NAME **Ash Pond Safety Factor Assessment**

JOB # **170GC00108**

PROJECT LOCATION **A.B. Brown Generating Facility**
Posey County, Indiana

DRILLING and SAMPLING INFORMATION

Date Started **7/8/15**
Date Completed **7/9/15**
Drill Foreman **J. Cook**
Inspector **M. Foye**
Boring Method **HSA**

Hammer Wt. **140** lbs.
Hammer Drop **30** in.
Spoon Sampler OD **2.0** in.
Rock Core Dia. **--** in.
Shelby Tube OD **--** in.

TEST DATA

SOIL CLASSIFICATION (continued)	Stratum Elevation	Stratum Depth, ft	Depth Scale, ft	Sample No.	Sample Type	Groundwater	Standard Penetration Test, Blows per 6 in. Increments	Moisture Content, %	Pocket Penetrometer PP-tsf	Remarks
Light brown and gray, moist, silty clay (EMBANKMENT FILL)				17	SS		6-8-10	15.5		**Sample No. SS-17:** Atterberg limits: LL=37, PL=16, PI=21
	405.0	46.0	45	18	SS		9-11-13	15.0		
Brown, moist, clayey silt (EMBANKMENT FILL)				19	SS		7-8-11	17.7		
	400.0	51.0	50	20	SS		6-4-6	18.1		**Sample No. SS-20:** Atterberg limits: LL=26, PL=22, PI=4
Brown and light brown, moist silty clay with little sand (EMBANKMENT FILL)	398.0	53.0		21	SS		5-4-4	21.1		
Brown, very moist, medium stiff, SILTY CLAY (CL)	395.0	56.0	55	22	SS		14-3-4	24.0		
Brown, wet, medium stiff, SILT (ML)	393.0	58.0		23	SS		3-4-3	26.0		
Brown and gray, moist to very moist, soft to medium stiff, SILTY CLAY (CL)			60	24	SS		2-2-3	22.2		
				25	SS		2-3-4	23.3		**Sample No. SS-25:** Atterberg limits: LL=35, PL=16, PI=19
			65	26	SS		4-4-4	20.9		
				27	SS	●	2-3-2	21.4		
	381.0	70.0	70	28	SS		3-3-3			
Bottom of Test Boring at 70.0 ft										

Sample Type
- SS - Driven Split Spoon
- ST - Pressed Shelby Tube
- CA - Continuous Flight Auger
- RC - Rock Core
- CU - Cuttings
- CT - Continuous Tube

Depth to Groundwater
- ● Noted on Drilling Tools **67.1** ft.
- ▽ At Completion **--** ft.
- ▼ After **--** hours **--** ft.
- ▨ Cave Depth **--** ft.

Boring Method
- HSA - Hollow Stem Augers
- CFA - Continuous Flight Augers
- CA - Casing Advancer
- MD - Mud Drilling
- HA - Hand Auger

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TEST BORING LOG

CLIENT **Vectren Corporation**

BORING # **B-214**

PROJECT NAME **Ash Pond Safety Factor Assessment**

JOB # **170GC00108**

PROJECT LOCATION **A.B. Brown Generating Facility**
Posey County, Indiana

DRILLING and SAMPLING INFORMATION

Date Started	**7/7/15**	Hammer Wt.	**140** lbs.
Date Completed	**7/8/15**	Hammer Drop	**30** in.
Drill Foreman	**J. Cook**	Spoon Sampler OD	**2.0** in.
Inspector	**M. Foye**	Rock Core Dia.	**--** in.
Boring Method	**HSA**	Shelby Tube OD	**--** in.

TEST DATA

SOIL CLASSIFICATION SURFACE ELEVATION 451	Stratum Elevation	Stratum Depth, ft	Depth Scale, ft	Sample No.	Sample Type	Sampler Graphics Recovery Graphics	Groundwater	Standard Penetration Test, Blows per 6 in. Increments	Moisture Content, %	Pocket Penetrometer PP-tsf	Remarks
Topsoil and Crushed Stone	450.5	0.5									
Brown, moist, silty clay (EMBANKMENT FILL)				1	SS			6-6-8			Ground surface elevation estimated from available topographic data.
				2	SS			4-4-5	21.3	2.0	
	445.5	5.5	5								
Brown, moist, clayey silt (EMBANKMENT FILL)				3	SS			11-11-13			Borehole backfilled with cement/bentonite grout.
				4	SS			6-7-9	18.9		
	440.5	10.5	10								
Light brown to brown, moist, silty clay (EMBANKMENT FILL)				5	SS			4-6-8			
				6	SS			8-6-7	16.8		
			15								
	435.0	16.0									
Reddish brown, moist, silty clay (EMBANKMENT FILL)				7	SS			8-11-15		4.0	
	433.0	18.0									
Brown, moist, silty clay (EMBANKMENT FILL)				8	SS			11-15-11	17.5	3.0	
			20								
				9	SS			9-12-11		4.5+	
				10	SS			6-6-7	19.5		
			25								
				11	SS			7-9-12	16.6	2.5	**Sample No. SS-11:** Atterberg limits: LL=31, PL=17, PI=14
				12	SS			5-7-11	19.9	2.5	
			30								
				13	SS			7-6-9	16.4		
				14	SS			8-7-9	19.8		
			35								
				15	SS			6-7-10	19.7		
				16	SS			6-8-9	16.1		

Sample Type
SS - Driven Split Spoon
ST - Pressed Shelby Tube
CA - Continuous Flight Auger
RC - Rock Core
CU - Cuttings
CT - Continuous Tube

Depth to Groundwater
● Noted on Drilling Tools **54.1** ft.
▽ At Completion **--** ft.
▼ After **--** hours **--** ft.
▨ Cave Depth **--** ft.

Boring Method
HSA - Hollow Stem Augers
CFA - Continuous Flight Augers
CA - Casing Advancer
MD - Mud Drilling
HA - Hand Auger

TEST BORING LOG

CLIENT **Vectren Corporation**
PROJECT NAME **Ash Pond Safety Factor Assessment**
PROJECT LOCATION **A.B. Brown Generating Facility**
Posey County, Indiana

BORING # **B-214**
JOB # **170GC00108**

DRILLING and SAMPLING INFORMATION

Date Started	**7/7/15**	Hammer Wt.	**140** lbs.
Date Completed	**7/8/15**	Hammer Drop	**30** in.
Drill Foreman	**J. Cook**	Spoon Sampler OD	**2.0** in.
Inspector	**M. Foye**	Rock Core Dia.	**--** in.
Boring Method	**HSA**	Shelby Tube OD	**--** in.

TEST DATA

SOIL CLASSIFICATION (continued)	Stratum Elevation	Stratum Depth, ft	Depth Scale, ft	Sample No.	Sample Type	Sampler Graphics Recovery Graphics	Groundwater	Standard Penetration Test, Blows per 6 in. Increments	Moisture Content, %	Pocket Penetrometer PP-tsf	Remarks
Brown, moist, silty clay (EMBANKMENT FILL)				17	SS			7-5-10	15.6		**Sample No. SS-17:** Atterberg limits: LL=29, PL=18, PI=11
			45	18	SS			7-9-10	18.2	2.0	
				19	SS			8-7-8	22.9		
	403.0	48.0									
Brown to gray, wet, soft, SILTY CLAY (CL-ML)			50	20	SS			1-2-2	27.4		**Sample No. SS-20:** Atterberg limits: LL=28, PL=24, PI=4
				21	SS			2-2-2			
	398.0	53.0									
Brown, moist to very moist, very soft to stiff, SILTY CLAY (CL)			55	22	SS		●	0-3-4	23.3		
				23	SS			2-1-1	22.5	1.0	**Sample No. SS-23:** Atterberg limits: LL=29, PL=16, PI=13
			60	24	SS			1-1-3		1.0	
				25	SS			4-5-7	22.2		
			65	26	SS			5-4-4			
	385.0	66.0									
Gray, moist, stiff, SANDY CLAY (CL)				27	SS			8-8-8			
	382.0	69.0									
Gray and brown, weathered, SHALE	381.0	70.0	70	28	SS			6-9-11			
Bottom of Test Boring at 70.0											

Sample Type
SS - Driven Split Spoon
ST - Pressed Shelby Tube
CA - Continuous Flight Auger
RC - Rock Core
CU - Cuttings
CT - Continuous Tube

Depth to Groundwater
● Noted on Drilling Tools **54.1** ft.
▽ At Completion **--** ft.
▼ After **--** hours **--** ft.
▨ Cave Depth **--** ft.

Boring Method
HSA - Hollow Stem Augers
CFA - Continuous Flight Augers
CA - Casing Advancer
MD - Mud Drilling
HA - Hand Auger

TEST BORING LOG

CLIENT **Vectren Corporation**
PROJECT NAME **Ash Pond Safety Factor Assessment**
PROJECT LOCATION **A.B. Brown Generating Facility**
Posey County, Indiana
BORING # **B-215**
JOB # **170GC00108**

DRILLING and SAMPLING INFORMATION

Date Started **7/16/15**
Date Completed **7/16/15**
Drill Foreman **J. Cook**
Inspector **K. Sweet**
Boring Method **HSA**

Hammer Wt. **140** lbs.
Hammer Drop **30** in.
Spoon Sampler OD **2.0** in.
Rock Core Dia. **--** in.
Shelby Tube OD **--** in.

TEST DATA

SOIL CLASSIFICATION SURFACE ELEVATION 415	Stratum Elevation	Stratum Depth, ft	Depth Scale, ft	Sample No.	Sample Type	Sampler Graphics Recovery Graphics	Groundwater	Standard Penetration Test, Blows per 6 in. Increments	Moisture Content, %	Pocket Penetrometer PP-tsf	Remarks
Topsoil	414.7	0.3									Ground surface elevation estimated from available topographic data.
Brown, moist, silty clay with some sand (EMBANKMENT FILL)	411.5	3.5		1	SS			7-8-11			
Brown, moist, sand (EMBANKMENT FILL)	408.5	6.5	5	2	SS			13-14-15			
Gray and brown, moist, silty clay with little sand (EMBANKMENT FILL)	406.5	8.5		3	SS		●	2-3-5	18.8	1.75	Borehole backfilled with cement/bentonite grout.
Brown, wet, clayey sand (EMBANKMENT FILL)	405.5	9.5	10	4	SS			4-3-7	16.1		**Sample No. SS-4:** Atterberg limits: LL=28, PL=12, PI=16
Reddish brown to brown, moist, silty clay (EMBANKMENT FILL)	401.5	13.5		5	SS			6-8-11	15.5		
Brown and gray, moist, clayey silt (EMBANKMENT FILL)	399.0	16.0	15	6	SS			4-7-12			
Reddish brown, moist, sandy clay (EMBANKMENT FILL)	395.5	19.5		7	SS			4-3-5	17.1		
			20	8	SS			0-1-2	25.1		
Brown, moist, very soft, SILTY CLAY (CL)	394.0	21.0									
Brown and gray, very moist, medium stiff, SILTY CLAY (CL)	389.0	26.0		9	SS			3-4-3	27.3		**Sample No. SS-9:** Atterberg limits: LL=29, PL=20, PI=9
			25	10	SS			3-3-4			
Gray, very moist, very soft, SILTY CLAY (CL)	387.0	28.0		11	SS			1-0-1	41.5	0.5	
Gray, wet, soft to very soft, SILT (ML)	381.0	34.0		12	SS			2-2-2	36.3	0.5	**Sample No. SS-12:** Atterberg limits: Non-plastic
			30	13	SS			1-2-1			
Gray, wet, very soft, SILT (ML) with trace organic matter			35	14	SS			0-0-2	36.0		
				15	SS			0-0-2	35.9	0.75	**Sample No. SS-15:** Atterberg limits: Non-plastic
				16	SS			0-0-1			

Sample Type
SS - Driven Split Spoon
ST - Pressed Shelby Tube
CA - Continuous Flight Auger
RC - Rock Core
CU - Cuttings
CT - Continuous Tube

Depth to Groundwater
● Noted on Drilling Tools **6.5** ft.
▽ At Completion **--** ft.
▼ After **--** hours **--** ft.
▨ Cave Depth **--** ft.

Boring Method
HSA - Hollow Stem Augers
CFA - Continuous Flight Augers
CA - Casing Advancer
MD - Mud Drilling
HA - Hand Auger

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TEST BORING LOG

CLIENT **Vectren Corporation**
PROJECT NAME **Ash Pond Safety Factor Assessment**
PROJECT LOCATION **A.B. Brown Generating Facility**
Posey County, Indiana

BORING # **B-215**
JOB # **170GC00108**

DRILLING and SAMPLING INFORMATION

Date Started **7/16/15**
Date Completed **7/16/15**
Drill Foreman **J. Cook**
Inspector **K. Sweet**
Boring Method **HSA**

Hammer Wt. **140** lbs.
Hammer Drop **30** in.
Spoon Sampler OD **2.0** in.
Rock Core Dia. **--** in.
Shelby Tube OD **--** in.

TEST DATA

SOIL CLASSIFICATION (continued)	Stratum Elevation	Stratum Depth, ft	Depth Scale, ft	Sample No.	Sample Type	Sampler Graphics Recovery Graphics	Groundwater	Standard Penetration Test, Blows per 6 in. Increments	Moisture Content, %	Pocket Penetrometer PP-tsf	Remarks
Gray, wet, very soft, SILT (ML) with trace organic matter				17	SS			1-1-2			
	372.0	43.0									
Gray, wet, very soft, CLAYEY SILT (ML)			45	18	SS			1-1-1	26.5	0.75	**Sample No. SS-18:** Atterberg limits: LL=26, PL=23, PI=3
	369.0	46.0									
Dark gray, moist, medium stiff, SILTY CLAY (CL)				19	SS			0-4-5		1.5	
			50	20	SS			3-3-4		0.75	
	364.0	51.0									
Bluish gray, miost, hard, SILTY CLAY (CL)	363.0	52.0		21	SS			50/0.4			
Gray, weathered, SHALE			55	22	SS			50/0.2			
				23	SS			50/0.1			
	355.0	60.0	60	24	SS			50/0.1			
Bottom of Test Boring at 60.0 ft											

Sample Type
SS - Driven Split Spoon
ST - Pressed Shelby Tube
CA - Continuous Flight Auger
RC - Rock Core
CU - Cuttings
CT - Continuous Tube

Depth to Groundwater
● Noted on Drilling Tools **6.5** ft.
▽ At Completion **--** ft.
▼ After **--** hours **--** ft.
▨ Cave Depth **--** ft.

Boring Method
HSA - Hollow Stem Augers
CFA - Continuous Flight Augers
CA - Casing Advancer
MD - Mud Drilling
HA - Hand Auger

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TEST BORING LOG

CLIENT **Vectren Corporation**
PROJECT NAME **Ash Pond Safety Factor Assessment**
PROJECT LOCATION **A.B. Brown Generating Facility**
Posey County, Indiana

BORING # **B-216**
JOB # **170GC00108**

DRILLING and SAMPLING INFORMATION

Date Started **7/15/15**
Date Completed **7/15/15**
Drill Foreman **J. Cook**
Inspector **B. Kleeman**
Boring Method **HSA**

Hammer Wt. **140** lbs.
Hammer Drop **30** in.
Spoon Sampler OD **2.0** in.
Rock Core Dia. **--** in.
Shelby Tube OD **--** in.

SOIL CLASSIFICATION

SURFACE ELEVATION 415

Soil Classification	Stratum Elevation	Stratum Depth, ft
Topsoil	414.7	0.3
Reddish brown, moist, sandy clay (EMBANKMENT FILL)	411.0	4.0
Brown, moist, sandy gravel (EMBANKMENT FILL)	410.5	4.5
Brown, very moist to wet, sand (EMBANKMENT FILL)	408.2	6.8
Reddish brown, moist, clay (EMBANKMENT FILL)	404.0	11.0
Reddish brown, moist, silty clay with trace sand (EMBANKMENT FILL)	401.5	13.5
Reddish brown, moist, sandy clay (EMBANKMENT FILL)	396.5	18.5
Brown, moist, silty clay (EMBANKMENT FILL)	391.5	23.5
Gray and brown, very moist to wet, stiff to soft, SILT (ML) with little sand	382.0	33.0
Gray, wet, very soft to soft, SILT (ML)		

TEST DATA

Sample No.	Sample Type	Groundwater	Standard Penetration Test, Blows per 6 in. Increments	Moisture Content, %	Pocket Penetrometer PP-tsf	Remarks
1	SS		6-9-8			Ground surface elevation estimated from available topographic data.
2	SS		8-10-13			
3	SS	●	3-4-5	15.0	3.5	Borehole backfilled with cement/bentonite grout.
4	SS		4-4-6			
5	SS		3-4-5	16.8	2.5	
6	SS		6-4-6	21.6	2.0	**Sample No. SS-6:** Atterberg limits: LL=36, PL=15, PI=21
7	SS		6-6-8	17.9	2.25	
8	SS		5-10-11			
9	SS		1-2-4	23.9		**Sample No. SS-9:** Atterberg limits: LL=30, PL=17, PI=13
10	SS		3-4-7	21.9		
11	SS		3-3-3	24.3		**Sample No. SS-11:** Atterberg limits: Non-plastic
12	SS		2-1-1			
13	SS		1-1-2	35.4		
14	SS		1-2-1			
15	SS		1-2-2			**Sample No. SS-16:** Atterberg limits: Non-plastic
16	SS		2-2-3	33.9		

Sample Type
SS - Driven Split Spoon
ST - Pressed Shelby Tube
CA - Continuous Flight Auger
RC - Rock Core
CU - Cuttings
CT - Continuous Tube

Depth to Groundwater
● Noted on Drilling Tools **6.0** ft.
▽ At Completion **--** ft.
▼ After **--** hours **--** ft.
▨ Cave Depth **--** ft.

Boring Method
HSA - Hollow Stem Augers
CFA - Continuous Flight Augers
CA - Casing Advancer
MD - Mud Drilling
HA - Hand Auger

TEST BORING LOG

CLIENT **Vectren Corporation**

PROJECT NAME **Ash Pond Safety Factor Assessment**

PROJECT LOCATION **A.B. Brown Generating Facility**
Posey County, Indiana

BORING # **B-216**

JOB # **170GC00108**

DRILLING and SAMPLING INFORMATION

Date Started	**7/15/15**	Hammer Wt.	**140** lbs.
Date Completed	**7/15/15**	Hammer Drop	**30** in.
Drill Foreman	**J. Cook**	Spoon Sampler OD	**2.0** in.
Inspector	**B. Kleeman**	Rock Core Dia.	**--** in.
Boring Method	**HSA**	Shelby Tube OD	**--** in.

TEST DATA

SOIL CLASSIFICATION (continued)	Stratum Elevation	Stratum Depth, ft	Depth Scale, ft	Sample No.	Sample Type	Sampler Graphics Recovery Graphics	Groundwater	Standard Penetration Test, Blows per 6 in. Increments	Moisture Content, %	Pocket Penetrometer PP-tsf	Remarks
Gray, wet, soft to medium stiff, SILT (ML)				17	SS			2-1-3			
				18	SS			2-2-4	51.8		
	369.0	46.0	45								
Gray, wet, soft, CLAYEY SILT (ML)				19	SS			1-3-2	25.6		
	367.0	48.0									
Gray and brown, moist, soft to stiff, SILTY CLAY (CL)				20	SS			1-2-3	20.2		
			50								
				21	SS			4-4-7	24.7		
	361.1	53.9									
Gray and brown, weathered, SILTSTONE				22	SS			7-11-19			
			55								
				23	SS			49-50/0.1			
				24	SS			47-50/0.3			
	355.0	60.0	60								
Bottom of Test Boring at 60.0 ft											

Sample Type
SS - Driven Split Spoon
ST - Pressed Shelby Tube
CA - Continuous Flight Auger
RC - Rock Core
CU - Cuttings
CT - Continuous Tube

Depth to Groundwater
● Noted on Drilling Tools **6.0** ft.
▽ At Completion **--** ft.
▼ After **--** hours **--** ft.
▨ Cave Depth **--** ft.

Boring Method
HSA - Hollow Stem Augers
CFA - Continuous Flight Augers
CA - Casing Advancer
MD - Mud Drilling
HA - Hand Auger

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TEST BORING LOG

CLIENT **Vectren Corporation**
PROJECT NAME **Ash Pond Safety Factor Assessment**
PROJECT LOCATION **A.B. Brown Generating Facility**
Posey County, Indiana
BORING # **B-217**
JOB # **170GC00108**

DRILLING and SAMPLING INFORMATION

Date Started **7/14/15**
Date Completed **7/14/15**
Drill Foreman **J. Cook**
Inspector **B. Kleeman**
Boring Method **HSA**
Hammer Wt. **140** lbs.
Hammer Drop **30** in.
Spoon Sampler OD **2.0** in.
Rock Core Dia. **--** in.
Shelby Tube OD **--** in.

TEST DATA

SOIL CLASSIFICATION SURFACE ELEVATION 415	Stratum Elevation	Stratum Depth, ft	Depth Scale, ft	Sample No.	Sample Type	Sampler Graphics Recovery Graphics	Groundwater	Standard Penetration Test, Blows per 6 in. Increments	Moisture Content, %	Pocket Penetrometer PP-tsf	Remarks
Topsoil	414.7	0.3									
Brown, slightly moist, silty clay (EMBANKMENT FILL)				1	SS			8-7-5			Ground surface elevation estimated from available topographic data.
				2	SS			7-7-8			
			5								
	408.0	7.0		3	SS			6-4-3			Borehole backfilled with cement/bentonite grout.
Brown, wet, sandy gravel (EMBANKMENT FILL)	406.5	8.5					▼				
Grayish brown, moist, sandy clay with trace shale and sandstone fragments				4	SS			5-5-5	18.1	1.5	
			10								
				5	SS			5-5-6	13.7		Installed piezometer.
				6	SS			4-6-7	17.3		
			15								
				7	SS			2-3-4	21.8		
				8	SS			3-4-5	20.5		**Sample No. SS-8:** Atterberg limits: LL=28, PL=17, PI=11
			20								
	394.0	21.0									
Brown and gray, moist, medium stiff to stiff, SILTY CLAY (CL)				9	SS			5-5-5			
				10	SS			7-8-12	17.8		**Sample No. SS-10:** Atterberg limits: LL=29, PL=20, PI=9
			25								
				11	SS			3-4-5			
				12	SS			0-3-7	22.1		
			30								
	384.0	31.0									
Gray, moist, medium stiff, SILTY CLAY (CL)				13	SS		●	2-3-6	24.4		
				14	SS			3-2-4			
			35								
	379.0	36.0									
Brown to gray, wet, medium stiff to soft, SILTY CLAY (CL-ML)				15	SS			3-3-3	30.6		**Sample No. SS-15:** Atterberg limits: LL=29, PL=21, PI=8
				16	SS			1-2-2			

Sample Type
SS - Driven Split Spoon
ST - Pressed Shelby Tube
CA - Continuous Flight Auger
RC - Rock Core
CU - Cuttings
CT - Continuous Tube

Depth to Groundwater
● Noted on Drilling Tools **32.0** ft.
▽ At Completion **--** ft.
▼ After **1152** hours **8.4** ft.
⊠ Cave Depth **--** ft.

Boring Method
HSA - Hollow Stem Augers
CFA - Continuous Flight Augers
CA - Casing Advancer
MD - Mud Drilling
HA - Hand Auger

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TEST BORING LOG

CLIENT **Vectren Corporation**
PROJECT NAME **Ash Pond Safety Factor Assessment**
PROJECT LOCATION **A.B. Brown Generating Facility**
Posey County, Indiana

BORING # **B-217**
JOB # **170GC00108**

DRILLING and SAMPLING INFORMATION

Date Started **7/14/15**
Date Completed **7/14/15**
Drill Foreman **J. Cook**
Inspector **B. Kleeman**
Boring Method **HSA**

Hammer Wt. **140** lbs.
Hammer Drop **30** in.
Spoon Sampler OD **2.0** in.
Rock Core Dia. **--** in.
Shelby Tube OD **--** in.

TEST DATA

SOIL CLASSIFICATION (continued)	Stratum Elevation	Stratum Depth, ft	Depth Scale, ft	Sample No.	Sample Type	Sampler Graphics Recovery Graphics	Groundwater	Standard Penetration Test, Blows per 6 in. Increments	Moisture Content, %	Pocket Penetrometer PP-tsf	Remarks
Brown to gray, wet, medium stiff to soft, SILTY CLAY (CL-ML)				17	SS	☒ ■		1-1-2			
	372.0	43.0									
Dark gray, wet, very soft to medium stiff, SILT (ML)				18	SS	☒ ■		0-1-2	39.5		**Sample No. SS-18:** Atterberg limits: LL=37, PL=35, PI=2
			45								
				19	SS	☒ ■		0-1-2			
				20	SS	☒ ■		0-1-2			
			50								
				21	SS	☒ ■		3-2-5	26.4		**Sample No. SS-21:** Atterberg limits: Non-plastic
				22	SS	☒ ■		1-3-3			
	359.5	55.5	55								
Dark gray, moist, soft, SILTY CLAY (CL)				23	SS	☒ ■		1-2-3	23.5		**Sample No. SS-23:** Atterberg limits: LL=38, PL=16, PI=22
				24	SS	☒ ■		1-2-3			
	355.0	60.0	60								
Bottom of Test Boring at 60.0 ft											

Sample Type
SS - Driven Split Spoon
ST - Pressed Shelby Tube
CA - Continuous Flight Auger
RC - Rock Core
CU - Cuttings
CT - Continuous Tube

Depth to Groundwater
● Noted on Drilling Tools **32.0** ft.
▽ At Completion **--** ft.
▼ After **1152** hours **8.4** ft.
▨ Cave Depth **--** ft.

Boring Method
HSA - Hollow Stem Augers
CFA - Continuous Flight Augers
CA - Casing Advancer
MD - Mud Drilling
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TEST BORING LOG

CLIENT **Vectren Corporation**

BORING # **B-218**

PROJECT NAME **Ash Pond Safety Factor Assessment**

JOB # **170GC00108**

PROJECT LOCATION **A.B. Brown Generating Facility**
Posey County, Indiana

DRILLING and SAMPLING INFORMATION

Date Started	**7/6/15**	Hammer Wt.	**140** lbs.
Date Completed	**7/6/15**	Hammer Drop	**30** in.
Drill Foreman	**J. Cook**	Spoon Sampler OD	**2.0** in.
Inspector	**M. Foye**	Rock Core Dia.	**--** in.
Boring Method	**HSA**	Shelby Tube OD	**--** in.

TEST DATA

SOIL CLASSIFICATION SURFACE ELEVATION 415	Stratum Elevation	Stratum Depth, ft	Depth Scale, ft	Sample No.	Sample Type	Sampler Graphics Recovery Graphics	Groundwater	Standard Penetration Test, Blows per 6 in. Increments	Moisture Content, %	Pocket Penetrometer PP-tsf	Remarks
Topsoil	414.7	0.3									Ground surface elevation estimated from available topographic data.
Tan, slightly moist, silty clay (EMBANKMENT FILL)				1	SS			4-6-7			
	410.2	4.8		2	SS			4-9-11			
Gray, moist, sand with trace gravel (EMBANKMENT FILL)	409.0	6.0	5								
				3	SS			8-6-7			Borehole backfilled with cement/bentonite grout.
Reddish brown, slightly moist to moist, silt (EMBANKMENT FILL)	406.5	8.5									
Brown, moist, very stiff to very soft, SILT (ML)				4	SS			8-11-7	22.3		**Sample No. SS-4:** Atterberg limits: LL=26, PL=25, PI=1
			10				●				
-wet below 11.0 ft				5	SS			2-1-1	30.6		**Sample No. SS-5:** Atterberg limits: LL=27, PL=26, PI=1
				6	SS			2-1-1			
			15								
				7	SS			2-1-2			
				8	SS			2-1-2			
			20								
	393.5	21.5									
Gray, wet, soft to very soft, SILT (ML)				9	SS			2-2-3			
	391.5	23.5									
Gray, very moist, soft to stiff, SILTY CLAY (CL-ML)				10	SS			0-0-1	23.4	0.5	**Sample No. SS-10:** Atterberg limits: LL=24, PL=18, PI=6
			25								
				11	SS			0-1-3	21.8	1.0	
				12	SS			2-3-6		2.0	
			30								
				13	SS			4-4-4	20.9		
				14	SS			4-4-7	18.9		
			35								
	379.0	36.0									
Reddish brown, moist, medium stiff, SANDY CLAY (CL)				15	SS			4-4-5			
	377.0	38.0									
Reddish brown, moist, medium stiff to very soft, SILTY CLAY (CL-ML)				16	SS			3-2-4	25.6		

Sample Type
SS - Driven Split Spoon
ST - Pressed Shelby Tube
CA - Continuous Flight Auger
RC - Rock Core
CU - Cuttings
CT - Continuous Tube

Depth to Groundwater
● Noted on Drilling Tools **11.0** ft.
▽ At Completion **--** ft.
▼ After **--** hours **--** ft.
▨ Cave Depth **--** ft.

Boring Method
HSA - Hollow Stem Augers
CFA - Continuous Flight Augers
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TEST BORING LOG

CLIENT **Vectren Corporation**
BORING # **B-218**
PROJECT NAME **Ash Pond Safety Factor Assessment**
JOB # **170GC00108**
PROJECT LOCATION **A.B. Brown Generating Facility**
Posey County, Indiana

DRILLING and SAMPLING INFORMATION

Date Started **7/6/15**
Date Completed **7/6/15**
Drill Foreman **J. Cook**
Inspector **M. Foye**
Boring Method **HSA**

Hammer Wt. **140** lbs.
Hammer Drop **30** in.
Spoon Sampler OD **2.0** in.
Rock Core Dia. **--** in.
Shelby Tube OD **--** in.

TEST DATA

SOIL CLASSIFICATION (continued)	Stratum Elevation	Stratum Depth, ft	Depth Scale, ft	Sample No.	Sample Type	Sampler Graphics Recovery Graphics	Groundwater	Standard Penetration Test, Blows per 6 in. Increments	Moisture Content, %	Pocket Penetrometer PP-tsf	Remarks
Reddish brown, moist, medium stiff to very soft, SILTY CLAY (CL-ML)				17	SS			2-2-2	28.6	1.25	**Sample No. SS-17:** Atterberg limits: LL=32, PL=25, PI=7
				18	SS			1-1-1			
			45								
Gray, wet, medium stiff to very soft, CLAYEY SILT (ML)	368.5	46.5		19	SS			2-4-3			
				20	SS			2-2-2	23.0		**Sample No. SS-20:** Atterberg limits: Non-plastic
			50								
				21	SS			0-0-0			
Gray and brown, moist, medium stiff to hard, SILTY CLAY (CL)	362.0	53.0		22	SS			3-4-6	20.9		**Sample No. SS-22:** Atterberg limits: LL=45, PL=16, PI=29
			55								
				23	SS			23-23-50/0.3			
Gray, weathered, SILTSTONE	357.6	57.4									
Bottom of Test Boring at 58.9 ft	356.1	58.9		24	SS			50/0.3			
			60								

Sample Type
SS - Driven Split Spoon
ST - Pressed Shelby Tube
CA - Continuous Flight Auger
RC - Rock Core
CU - Cuttings
CT - Continuous Tube

Depth to Groundwater
● Noted on Drilling Tools **11.0** ft.
▽ At Completion **--** ft.
▼ After **--** hours **--** ft.
☒ Cave Depth **--** ft.

Boring Method
HSA - Hollow Stem Augers
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TEST BORING LOG

CLIENT **Vectren Corporation**
PROJECT NAME **Ash Pond Safety Factor Assessment**
PROJECT LOCATION **A.B. Brown Generating Facility**
Posey County, Indiana

BORING # **B-219**
JOB # **170GC00108**

DRILLING and SAMPLING INFORMATION

Date Started **7/13/15**
Date Completed **7/13/15**
Drill Foreman **J. Cook**
Inspector **B. Kleeman**
Boring Method **HSA**

Hammer Wt. **140** lbs.
Hammer Drop **30** in.
Spoon Sampler OD **2.0** in.
Rock Core Dia. **--** in.
Shelby Tube OD **--** in.

TEST DATA

SOIL CLASSIFICATION SURFACE ELEVATION 415	Stratum Elevation	Stratum Depth, ft	Depth Scale, ft	Sample No.	Sample Type	Groundwater	Standard Penetration Test, Blows per 6 in. Increments	Moisture Content, %	Pocket Penetrometer PP-tsf	Remarks
Topsoil	414.5	0.5								
Brown, slightly moist, silty clay (EMBANKMENT FILL)	409.5	5.5		1	SS		5-7-8			Ground surface elevation estimated from available topographic data.
				2	SS		10-13-21			
Brown, very moist, soft to very soft, SILT (ML) -wet below 6.9 ft	398.8	16.2	5	3	SS	●	3-2-2	29.8		Borehole backfilled with cement/bentonite grout.
			10	4	SS		1-2-2	28.9		**Sample No. SS-4:** Atterberg limits: Non-plastic
				5	SS		2-2-1			
			15	6	SS		2-2-2			
Gray, wet, very soft, SILT (ML)	397.0	18.0		7	SS		1-1-1	29.9		**Sample No. SS-7:** Atterberg limits: Non-plastic
Gray, moist, very soft, SILTY CLAY (CL)	392.0	23.0	20	8	SS		0-1-1	23.9		**Sample No. SS-8:** Atterberg limits: LL=28, PL=18, PI=10
				9	SS		1-1-2	24.1		
Brown and gray, moist, medium stiff to stiff, SILTY CLAY (CL)	382.0	33.0	25	10	SS		4-4-5			
				11	SS		5-6-7	21.5		**Sample No. SS-11:** Atterberg limits: LL=30, PL=13, PI=17
			30	12	SS		4-5-6			
				13	SS		3-3-5			
Brown, very moist, medium stiff, SANDY CLAY (CL)	379.0	36.0	35	14	SS		3-3-5	22.6		
Brown, wet, medium stiff to very soft, SILTY CLAY (CL)				15	SS		5-5-5			**Sample No. SS-16:** Atterberg limits: LL=30, PL=20, PI=10
				16	SS		2-3-3	26.0		

Sample Type
SS - Driven Split Spoon
ST - Pressed Shelby Tube
CA - Continuous Flight Auger
RC - Rock Core
CU - Cuttings
CT - Continuous Tube

Depth to Groundwater
● Noted on Drilling Tools **6.9** ft.
▽ At Completion **--** ft.
▼ After **--** hours **--** ft.
▨ Cave Depth **--** ft.

Boring Method
HSA - Hollow Stem Augers
CFA - Continuous Flight Augers
CA - Casing Advancer
MD - Mud Drilling
HA - Hand Auger

TEST BORING LOG

CLIENT **Vectren Corporation**

PROJECT NAME **Ash Pond Safety Factor Assessment**

PROJECT LOCATION **A.B. Brown Generating Facility**
Posey County, Indiana

BORING # **B-219**

JOB # **170GC00108**

DRILLING and SAMPLING INFORMATION

Date Started **7/13/15**
Date Completed **7/13/15**
Drill Foreman **J. Cook**
Inspector **B. Kleeman**
Boring Method **HSA**

Hammer Wt. **140** lbs.
Hammer Drop **30** in.
Spoon Sampler OD **2.0** in.
Rock Core Dia. **--** in.
Shelby Tube OD **--** in.

TEST DATA

SOIL CLASSIFICATION (continued)	Stratum Elevation	Stratum Depth, ft	Depth Scale, ft	Sample No.	Sample Type	Sampler Graphics Recovery Graphics	Groundwater	Standard Penetration Test, Blows per 6 in. Increments	Moisture Content, %	Pocket Penetrometer PP-tsf	Remarks
Brown, wet, medium stiff to very soft, SILTY CLAY (CL)				17	SS			1-1-2			
			45	18	SS			1-1-2			
Light brown, weathered, SANDSTONE	368.2	46.8		19	SS			3-11-28			
			50	20	SS			22-50			
				21	SS			50/0.3			
			55	22	SS			50			
Gray, weathered, SHALE	357.6	57.4		23	SS			50/0.3			
	355.0	60.0	60	24	SS			50/0.2			
Bottom of Test Boring at 60.0 ft											

Sample Type
SS - Driven Split Spoon
ST - Pressed Shelby Tube
CA - Continuous Flight Auger
RC - Rock Core
CU - Cuttings
CT - Continuous Tube

Depth to Groundwater
● Noted on Drilling Tools **6.9** ft.
▽ At Completion **--** ft.
▼ After **--** hours **--** ft.
▨ Cave Depth **--** ft.

Boring Method
HSA - Hollow Stem Augers
CFA - Continuous Flight Augers
CA - Casing Advancer
MD - Mud Drilling
HA - Hand Auger

Equipment CME75/Solid Flight Auger

Elevation 450.9 ft. Date 5/6/82

Laboratory Tests	Blows/foot*	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD**	Depth (ft)	
						0	STILTY CLAY (CL) and CLAYEY SILT (ML) layers; Brown and light brown, medium stiff and medium dense, moist; trace sand and gravel; sandy layers; FILL
	16	20.0				5	
	17					10	Clayey sample; slightly moister below 9.0'
							Switched to rotary wash methods at 10.0'
	24	15.4	119			15	Clayey sample; becoming hard
	22					20	Silty sample
	21					25	Clayey and silty layers
Hyd LL = 29 PI = 18	24	15.5	119			30	Sandy silty clay; brown and reddish brown, hard
	28					35	Silty sample; ferrous (reddish brown) pockets; black specks (decayed shells?)
	23	18.5	112			40	Clayey sample; ferrous and clay nodules

*Generally standard penetration test (N) values; obtained by driving a 2.0" O.D. split spoon sampler with a 140-pound rammer free-falling 30 inches. Values followed by an asterisk obtained by driving a 3" O.D. sampler.

**Rock Quality Designation - percentage of core greater than 4 inches in length per run.

DRAFT

(Continuation of Log)

Laboratory Tests	Blows/foot	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD	Depth (ft)	
						40	SILTY CLAY (CL) and CLAYEY SILT (ML) layers; FILL
	8	20.9				45	CLAYEY SILT (ML) Brown, loose, moist; ferrous clay pockets; sandy layers
	8	29.6					SILT (ML) Brown and gray (mottled and layered), loose, wet; clayey and sandy layers; ferrous clay pockets; black specks
	8					50	
	6	31.0					
	5					55	
	4						
	4	26.2				60	
	4						CLAYEY SILT (ML) Brownish gray to grayish brown, loose, moist; trace fine sand
	9	25.4				65	
							SILTY CLAY (CL) Light brown to brown very stiff, moist; trace sand; black specks
	36					70	
				91	79	75	SILTSTONE, light brown, shaly; moderately hard with occ. soft clay seams, occ. undulating bedding; iron-stained bedding and fissures (to 78.0')
						80	Stratum extends to 80.5'

Harding Lawson Associates
Engineers, Geologists
& Geophysicists

LOG OF BORING 1
A. B. Brown Station
Southern Indiana Gas & Electric Company
West Franklin, Indiana

PLATE 4

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
MVMachalinski	9922,001.14	AJGallet	6/14/82		

DRAFT

(Continuation of Log)

Laboratory Tests	Blows/foot	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD	Depth (ft)	Sample	
						80		SILTSTONE, gray, shaly; moderately hard; frequent shale seams
								End of boring
						85		No free water encountered during auger drilling
								Piezometer set from 69.0' to 74.0' with sand packing from 65.0' to 75.0'
						90		
						95		
						100		
						105		
						110		
						115		
						120		

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LOG OF BORING 1 (Cont.)
A. B. Brown Station
Southern Indiana Gas & Electric Company
West Franklin, Indiana

PLATE 5

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
MVMachalinski	9922,001,14	AJGallet	6/14/82		

Equipment CME75/Solid Flight Auger

Elevation 450.7 ft. Date 5/13/82

Laboratory Tests	Blows/foot	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD	Depth (ft)	
						0	SILTY CLAY (CL) Brown, very stiff, moist; trace fine sand, sandy layers; FILL
	16	15.0	114				
						5	
							CLAYEY SILT (ML) Brown, medium dense, moist; trace fine sand; FILL
Disp D2*		19.2	101				
Hyd	41*	23.9	102			10	
LL = 26 PI = 1							
							SILTY CLAY (CL) Brown, hard, moist; trace fine sand, occ. ferrous nodules; silt layers; FILL
	26						
						15	
Disp ND1		17.3	114				
Disp ND2		17.6	111			20	
							Switched to rotary wash drilling at 21.0'
	14	20.5					
						25	SANDY SILTY CLAY (CL) Reddish brown, stiff, moist; trace fine gravel; brown silty clay pockets; FILL
							CLAYEY SILT (ML) Brown, medium dense moist; trace fine sand, occ. ferrous nodules, sandy layers; FILL
	25	19.9					
						30	
							SILTY CLAY (CL) Reddish brown, stiff, moist; trace fine sand, occ. grey clay pockets
	8	20.8	106				
						35	
	10						
						40	CLAYEY SILT (ML)

*Dispersion test classifications:

D1 and D2	Dispersive soils
ND4 and ND 3	Intermediate soils
ND2 and ND 1	Non-dispersive soils

DRAFT

(Continuation of Log)

Laboratory Tests	Blows/foot	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD	Depth (ft)	
						40	CLAYEY SILT (ML) Light brown, medium dense, wet
		27.2	98				SILT (ML) Light brown, loose, saturated; trace fine sand
						45	
							End of Boring
						50	No water encountered during auger drilling.
						55	Piezometer set from 43.0' to 48.0' with sand packing from 41.5' to 49.0'.
						60	
						65	
						70	
						75	
						80	

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LOG OF BORING 2
A. B. Brown Station
Southern Indiana Gas & Electric Company
West Franklin, Indiana

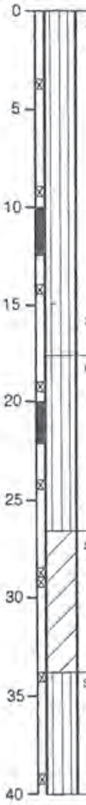
PLATE 6

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
MVMachalinski	9922,001.14	AJGallet	6/14/82		

Equipment CME75/Solid Flight Auger

Elevation 451.9 ft. Date 5/12/82

Laboratory Tests	Blows/foot	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD
	14	18.8			
Hyd					
LL = 28					
PI = 1	30				
Disp D2		20.7	108		
Perm (Plate 35)		18.9	107		
TxCU(3) (Plate 32)		18.5	114		
Hyd		17.5	110		
	26	14.6			
	19				
		15.9	117		
		12.9	124		
	54	16.6			
	18				
	11	26.8			
	7				



CLAYEY SILT (ML) Brown, medium dense, moist; trace sand and fine gravel; clay layers; FILL

Switched to rotary wash drilling at 15.0'

CLAYEY SILT (ML) Light brown, medium dense, to very dense, moist; trace sand occ. fine gravel; reddish brown (ferrous clay pockets; clay layers; cemented seams; FILL

SILTY CLAY (CL) Reddish brown, very stiff, moist; trace sand, occ. fine gravel; clay seams

SILT (ML) Brown, medium dense to loose, wet; trace find sand; ferrous seams

DRAFT

(Continuation of Log)

Laboratory Tests	Blows/foot	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD
LL = 29	9*				
PI = 9		25.6	99		
TxCU (Plate 33)					
	8	24.4			
	6	28.4			
	9				
	20				
	100/6"	15.2			
				100	70

SILTY CLAY (CL) Grayish brown, soft, very moist; ferrous seams

CLAY (CH) Reddish brown, medium stiff, moist; ferrous gravel layer at 44.5'

SILTY CLAY (CL) Reddish brown and brown, medium stiff to very stiff, moist; trace sand and gravel

SILTSTONE, light brown, very shaly; weakly cemented to 59.0'; moderately hard with soft pockets; iron-stained bedding and fissures; undulating bedding

SILTSTONE, gray, shaly; moderately hard; massive

End of Boring

No free water encountered during auger drilling

Set piezometer from 56.0' to 61.0' with sand packing from 56.0' to 64.0

HLA **Harding Lawson Associates**
Engineers, Geologists
& Geophysicists

LOG OF BORING 3
A. B. Brown Station
Southern Indiana Gas & Electric Company
West Franklin, Indiana

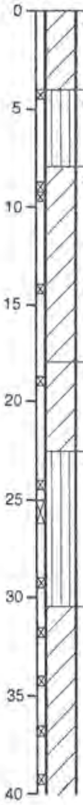
PLATE 7

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
MVMachalinski	9922,001.14	AJGallet	6/14/82		

Equipment CME75/Solid Flight Auger

Elevation 449.6 ft. Date 5/13/82

Laboratory Tests	Blows/foot	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD
	29	20.2			
	16	13.9			
	31				
	10	18.2			
	3	32.8			
Hyd LL = NP		32.0			
	4	25.1			
	5	22.9			
	3				
	4				
	5	25.4			



SILTY CLAY (CL) Brown, stiff, moist; gravelly for the upper 1.0'; FILL

CLAYEY SILT (ML) Light brown, medium dense, moist; trace fine sand; reddish brown (ferrous) clay seams; FILL

SILTY CLAY (CL) Light brown, very stiff to stiff, moist; trace sand and gravel; micaceous; cemented seams; FILL

Fine sandy silt pockets; ferrous seams

Switched to rotary wash drilling at 15.0'

SANDY SILTY CLAY (CL) Reddish brown, very stiff, moist; trace sand and fine gravel; slightly micaceous

CLAYEY SILT (ML) Brown, loose, wet; trace find sand; slightly micaceous; silt layers

SILTY CLAY (CL) Reddish brown, medium stiff to very stiff, moist; trace fine sand; occ. black specks (decayed shells?)

DRAFT

(Continuation of Log)

Laboratory Tests	Blows/foot	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD
	8				
	10				
	21	17.9			
	15				
LL = 93 PI = 36 TxCU (Plate 34)	36*	45.5 32.7	77 88		
	100/5"	18.4			
				96	37

SILTY CLAY (CL) becoming stiff

Becoming very stiff; trace sand and fine gravel, micaceous; ferrous nodules

CLAY (CL-CH) Light brown, medium stiff, very moist; trace fine sand, numerous black specks

SILTY CLAY (CL) Light brown and brown (mottled), hard, moist; silty sand pockets; cemented layers

SILTSTONE, light brown, very shaly, low hardness and soft pockets, weathered with iron-stained fissures

SILTSTONE, gray, shaly with shale seams and a 1" thick shale layer at 56.7'. Low hardness, weathered to slightly weathered, with iron-stained fissures above 57.2'. Moderately hard below 57.2' with undulating beds below 58.7'.

End of boring

No free water encountered during auger drilling

Piezometer set from 53.0' to 58.0' with sand packing from 52.0' to 60.3'

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Engineers, Geologists
& Geophysicists

LOG OF BORING 4
A. B. Brown Station
Southern Indiana Gas & Electric Company
West Franklin, Indiana

PLATE 8

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
MVMachalinski	9922,001.14	AJGallet	6/14/82		

Equipment CME75/Solid Flight Auger

Elevation 416.1 ft. Date 5/11/82

Laboratory Tests	Blows/foot	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD	Depth (ft)	
						0	CLAYEY SILT (ML) Brown, moist; trace fine sand; FILL
	23	20.2	101				CLAYEY SILT (ML) Brown, medium dense, moist; trace fine sand; slightly micaceous; clay layers
						5	
	6	30.8					Becoming reddish brown and brown, loose and wet below 9.0'
						10	
	7						SILT (ML) Gray (and reddish brown to ▽ 13.0'), loose, wet; trace fine sand; slightly micaceous
Hyd LL = NP	11*	27.5	98				Free water encountered during drilling; water level rising.
Disp D1 Perm (Plate 35)		29.6	95			15	
	6					20	Switched to rotary wash drilling at 15.0'
							Ferrous (reddish) pockets
	5	29.6					
LL = 29 PI = 9	10	23.6					SILTY CLAY (CL) Gray to grayish brown, medium stiff, moist
						25	
	17	21.0					SILTY CLAY (CL) Reddish brown, very stiff, moist; trace sand and fine gravel; sandy layers
						30	
	100/3"	19.1					SILTSTONE, light brown, shaly; moderately hard; undulating bedding; iron-stained bedding and fissures (to 37.3')
				100	87	35	
						40	

DRAFT

(Continuation of Log)

Laboratory Tests	Blows/foot	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD	Depth (ft)	
						40	SILTSTONE
							End of boring
						45	Piezometer located 10' north of Boring 5. Slotted portion set from 11.0' to 16.0' with sand packing from 10.0' to 18.0'
						50	
						55	
						60	
						65	
						70	
						75	
						80	

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LOG OF BORING 5
A. B. Brown Station
Southern Indiana Gas & Electric Company
West Franklin, Indiana

PLATE 9

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
MVMachalinski	9922,001.14	AJGallet	6/14/82		

DRAFT

Equipment [illegible] Solid Flight Auger

Elevation 416.2 ft. Date 5/11/82

∇ See note at end of boring

Laboratory Tests	Blows/foot	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	Cored Interval
	8	23.1			
	12	15.8	118		
	6				
	12	28.9			
	7				
LL = 75 PI = 48	7	43.3			
	4	48.1			
	15				
		19.0	111		
	100/6"	17.1			
				96	87

CLAYEY SILT (ML) Brown, loose to medium dense, moist; trace sand; mixed with coal to 2.2'; FILL (reworked?)

SILTY CLAY (CL) Reddish brown, medium stiff to stiff, very moist; trace fine sand; clayey silt layers

Switched to rotary wash drilling at 10.0'

Becoming stiff with sandy layers

CLAY (CH) Reddish brown, stiff, very moist; trace silt; soft silty pockets; black specks (decayed shells?)

SILTY CLAY (CL) Light brown, very stiff, moist; trace find sand, sandy silt pockets (slightly cemented);ferrous nodules and black specks

SILTSTONE, light brown from 24.0' to 25.4' and 26.8' to 27.8', gray from 25.4' to 26.8' and 27.8' to 33.0'; shaly; low hardness 26.8' to 27.8' otherwise moderately hard; slightly weathered to 27.8'; iron-stained bedding and fissures from 24.0' to 25.4' and 26.8' to 27.8'; massive below 27.8'

End of boring

Piezometer set from 7.0' to 12.0' with sand packing from 5.0' to 12.0'

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& Geophysicists

LOG OF BORING 6
A. B. Brown Station
Southern Indiana Gas & Electric Company
West Franklin, Indiana

PLATE 10

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
MVMachalinski	9922,001.14	AJGallet	6/14/82		

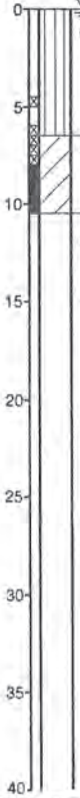
DRAFT

Equipment CME 55 Solid Flight Auger

Elevation 416.2 ft. Date 5/11/82

▽ See note at end of boring

Laboratory Tests	Blows/foot	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	Cored Interval
Hyd		22.5			
LL = 37		25.4			
PI = 15		28.2	94		
Disp ND1		28.1			
TxCU(2) (Plate 32		28.3	97		
Perm and 33)		28.5	94		
(Plate 35)		28.6	93		



CLAYEY SILT (ML) Brown, loose, moist; trace sand; FILL (reworked?)

SILTY CLAY (CL) Reddish brown, medium stiff to stiff, moist; trace fine sand; layer of ferrous gravel at top of strata; clayey silt layers

End of boring.

Free water encountered after driving sampler to 8.0'. Water level subsequently rose to ground surface.

Borehole was cement grouted to ground surface.

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LOG OF BORING 6A
A. B. Brown Station
Southern Indiana Gas & Electric Company
West Franklin, Indiana

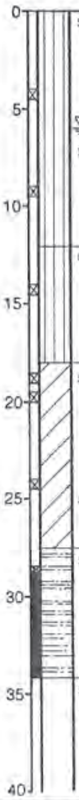
PLATE 11

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
MVMachalinski	9922,001.14	AJGallet	6/14/82		

Equipment [illegible]/S[illegible]d Flight Auger

Elevation 401.6 ft. Date 5/13/82

Laboratory Tests	Blows/foot	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	Cored Interval	Depth (ft)	Sample
	11	21.2					☒
	3	32.0					☒
	2	28.6					☒
		23.1					☒
	5						☒
	17	19.7					☒
	50/3"						
				100	91		



SILT (ML) Gray and brown (mottled), medium dense, dry to moist; trace fine sand; sandy layers

∇ Free water encountered at 6.0'

Becoming gray, loose and wet below 6.0'; with organic seams

CLAYEY SILT (ML) Gray, loose, very moist; trace fine sand

SILTY CLAY (CL) Light brown and reddish brown, stiff to very stiff; trace fine sand; occ. ferrous nodules

Switch to rotary wash drilling at 20.0'

SILTSTONE, light brown, shaly; moderately hard with occ. soft clay seams; iron-stained bedding and fissures (to 31.8'); undulated bedding; frequent shale seams below 33.3'

End of boring

Piezometer set from 26.0' to 31.0' with sand packing from 24.7' to 34.2'

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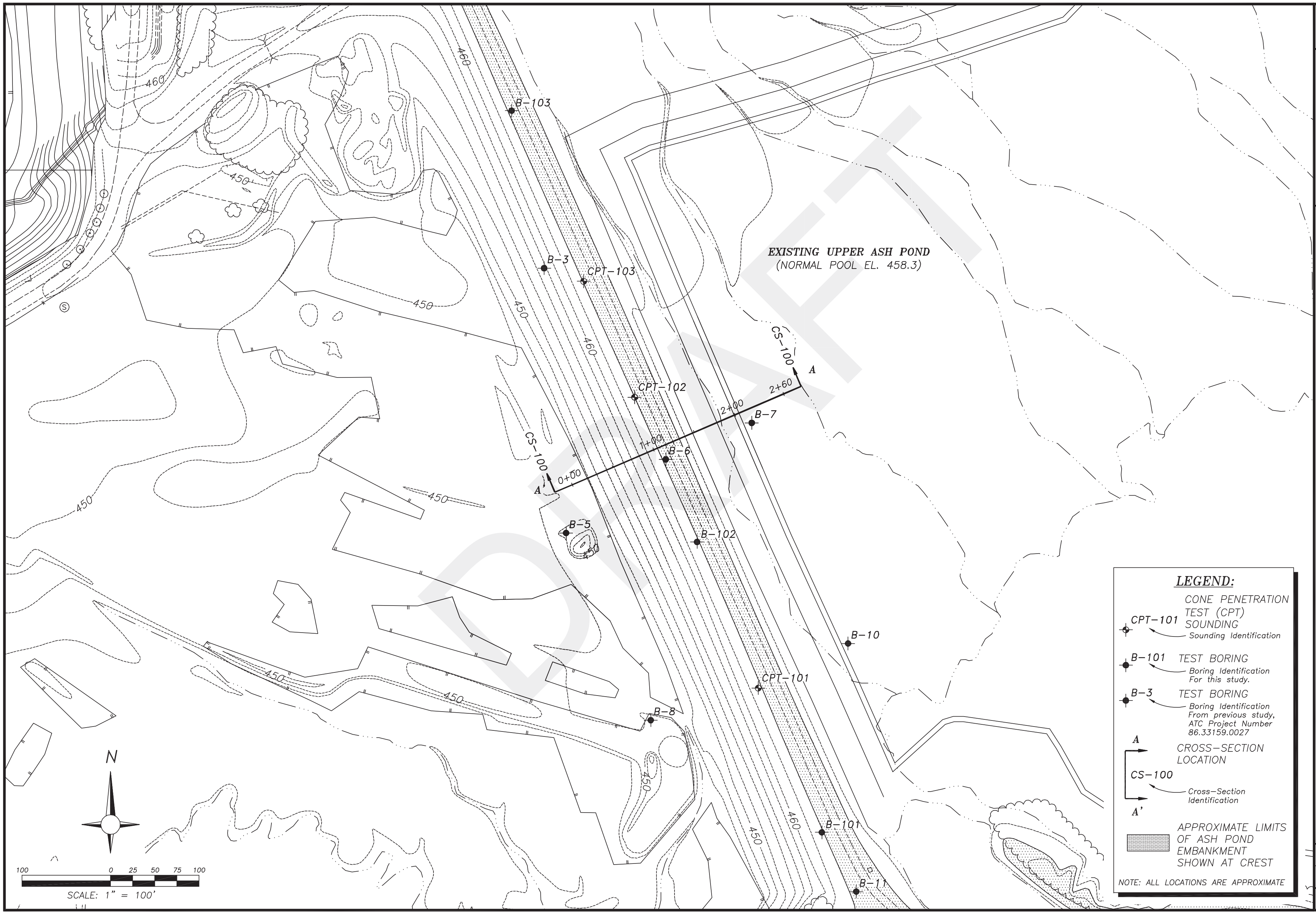
LOG OF BORING 7
A. B. Brown Station
Southern Indiana Gas & Electric Company
West Franklin, Indiana

PLATE
12

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
MVMachalinski	9922,001.14	AJGallet	6/14/82		

Test Boring Logs Drilled through Ash Materials



H:\2015\VECTREN CORPORATION\AB BROWN – SSA\GEO (170GC00108)\ABB-SSA-GEOTECH.DWG, BPLAN-UPPER

Drn. By: SP
Ckd. By: SM
App'd By:
Ckd. Date:

Project Number: 170GC00108
Drawing File: SEE LOWER LEFT

Cardno ATC

BORING PLAN – UPPER ASH POND
SAFETY FACTOR ASSESSMENT
UPPER AND LOWER ASH PONDS
VECTREN AB BROWN GENERATING STATION
POSEY COUNTY, INDIANA

Date: 6/15
Scale: AS SHOWN
Figure: 2

Cardno ATC
7988 Centerpoint Drive, Suite 100
Indianapolis, IN 46256
(317) 849-4990
Fax (317) 849-4278

TEST BORING LOG

CLIENT **Vectren Corporation**
PROJECT NAME **Ash Pond Safety Factor Assessment**
PROJECT LOCATION **A.B. Brown Generating Facility**
Posey County, Indiana

BORING # **B-101**
JOB # **170GC00108**

DRILLING and SAMPLING INFORMATION

Date Started **4/14/15**
Date Completed **4/14/15**
Drill Foreman **W. Bates**
Inspector **S. Marcum**
Boring Method **HSA**

Hammer Wt. **140** lbs.
Hammer Drop **30** in.
Spoon Sampler OD **2.0** in.
Rock Core Dia. **--** in.
Shelby Tube OD **--** in.

TEST DATA

SOIL CLASSIFICATION SURFACE ELEVATION 463.7	Stratum Elevation	Stratum Depth, ft	Depth Scale, ft	Sample No.	Sample Type	Sampler Graphics Recovery Graphics	Groundwater	Standard Penetration Test, Blows per 6 in. Increments	Moisture Content, %	Pocket Penetrometer PP-tsf	Remarks
Brown, very moist, sandy clay (EMBANKMENT FILL)	460.7	3.0		1	SS			2-3-3	22.0	0.75	Boring coordinates and ground surface elevation surveyed by Three I Design.
Dark brown, moist, sandy clay with coal ash (EMBANKMENT FILL)			5	2	SS			4-7-15	15.0	1.5	
	455.7	8.0		3	SS			19-17-16			Borehole backfilled with cement/bentonite grout.
Black, moist, fine to coarse, coal ash (EMBANKMENT FILL)			10	4	SS			17-21-16			
				5	SS			13-13-18	10.4		**Sample No. SS-5:** Atterberg limits: non-plastic Passing No. 200 sieve = 30.5%
			15	6	SS			8-12-11			
				7	SS			9-8-6			
	443.7	20.0	20	8	SS		●	12-6-6	19.5		**Sample No. SS-8:** Atterberg limits: non-plastic Passing No. 200 sieve = 30.5%
Black, wet, fine coal ash (FILL)				9	SS			4-3-3			
	437.2	26.5	25	10	SS			2-2-1			
Brown and black, very moist, silty clay with coal ash (FILL)				11	SS			2-1-2	24.1	0.75	**Sample No. SS-11:** Atterberg limits: LL=33, PL=20, PI=13 Passing No. 200 sieve = 97.2%
	432.7	31.0	30	12	SS			2-3-3	25.6		
Reddish brown, moist, medium stiff, SILTY CLAY (CL) with sandy clay seams				13	SS			3-5-5	20.7	1.5	**Sample No. SS-13:** Atterberg limits: LL=27, PL=10, PI=17 Passing No. 200 sieve = 97.5%
			35	14	SS			3-3-4	25.6	1.0	
				15	SS			3-3-3	32.7		**Sample No. SS-16:** Atterberg limits: LL=35, PL=18, PI=17 Passing No. 200 sieve = 99.4%
				16	SS			3-4-4	29.6	1.25	

Sample Type
- SS - Driven Split Spoon
- ST - Pressed Shelby Tube
- CA - Continuous Flight Auger
- RC - Rock Core
- CU - Cuttings
- CT - Continuous Tube

Depth to Groundwater
- ● Noted on Drilling Tools **20.0** ft.
- ∇ At Completion **--** ft.
- ▼ After **--** hours **--** ft.
- ☒ Cave Depth **--** ft.

Boring Method
- HSA - Hollow Stem Augers
- CFA - Continuous Flight Augers
- CA - Casing Advancer
- MD - Mud Drilling
- HA - Hand Auger

TEST BORING LOG

CLIENT **Vectren Corporation**
PROJECT NAME **Ash Pond Safety Factor Assessment**
PROJECT LOCATION **A.B. Brown Generating Facility**
Posey County, Indiana

BORING # **B-101**
JOB # **170GC00108**

DRILLING and SAMPLING INFORMATION

Date Started **4/14/15**
Date Completed **4/14/15**
Drill Foreman **W. Bates**
Inspector **S. Marcum**
Boring Method **HSA**

Hammer Wt. **140** lbs.
Hammer Drop **30** in.
Spoon Sampler OD **2.0** in.
Rock Core Dia. **--** in.
Shelby Tube OD **--** in.

TEST DATA

SOIL CLASSIFICATION (continued)	Stratum Elevation	Stratum Depth, ft	Depth Scale, ft	Sample No.	Sample Type	Sampler Graphics Recovery Graphics	Groundwater	Standard Penetration Test, Blows per 6 in. Increments	Moisture Content, %	Pocket Penetrometer PP-tsf	Remarks
Reddish brown, moist, medium stiff, SILTY CLAY (CL) with sandy clay seams	420.7	43.0		17	SS			6-3-4	27.7		
Reddish brown, slightly moist, very stiff to hard, SANDY CLAY (CL)	417.7	46.0	45	18	SS			14-9-13	19.0		
Brown and reddish brown, severely weathered SANDSTONE	415.2	48.5		19	SS			41-53-50/0.1	19.0		
Bottom of Test Boring at 48.5 ft											

Sample Type
SS - Driven Split Spoon
ST - Pressed Shelby Tube
CA - Continuous Flight Auger
RC - Rock Core
CU - Cuttings
CT - Continuous Tube

Depth to Groundwater
● Noted on Drilling Tools **20.0** ft.
▽ At Completion **--** ft.
▼ After **--** hours **--** ft.
☒ Cave Depth **--** ft.

Boring Method
HSA - Hollow Stem Augers
CFA - Continuous Flight Augers
CA - Casing Advancer
MD - Mud Drilling
HA - Hand Auger

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TEST BORING LOG

CLIENT **Vectren Corporation**
PROJECT NAME **Ash Pond Safety Factor Assessment**
PROJECT LOCATION **A.B. Brown Generating Facility**
Posey County, Indiana

BORING # **B-102**
JOB # **170GC00108**

DRILLING and SAMPLING INFORMATION

Date Started	**4/14/15**	Hammer Wt.	**140** lbs.
Date Completed	**4/14/15**	Hammer Drop	**30** in.
Drill Foreman	**W. Bates**	Spoon Sampler OD	**2.0** in.
Inspector	**S. Marcum**	Rock Core Dia.	**--** in.
Boring Method	**HSA**	Shelby Tube OD	**--** in.

TEST DATA

SOIL CLASSIFICATION SURFACE ELEVATION 463.4	Stratum Elevation	Stratum Depth, ft	Depth Scale, ft	Sample No.	Sample Type	Sampler Graphics Recovery Graphics	Groundwater	Standard Penetration Test, Blows per 6 in. Increments	Moisture Content, %	Pocket Penetrometer PP-tsf	Remarks
Brown, moist, sandy clay (EMBANKMENT FILL)				1	SS			3-4-5	19.1		Boring coordinates and ground surface elevation surveyed by Three I Design.
	459.4	4.0									
Dark brown, very moist, sandy silt with coal ash (EMBANKMENT FILL)				2	SS			5-6-7			Borehole backfilled with cement/bentonite grout.
			5								
				3	SS			17-17-21			
				4	SS			5-5-8	25.0		**Sample No. SS-4:** Atterberg limits: non-plastic Passing No. 200 sieve = 81.5%
			10								
				5	SS			12-12-14			
	450.4	13.0									
Dark brown, moist, silty sand with coal ash (EMBANKMENT FILL)				6	SS			6-11-9	17.5		**Sample No. SS-6:** Atterberg limits: non-plastic Passing No. 200 sieve = 33.9%
	447.9	15.5	15								
Reddish brown, moist, silty clay (EMBANKMENT FILL)				7	SS			7-5-8	17.0		**Sample No. SS-7:** Atterberg limits: LL=43, PL=17, PI=26 Passing No. 200 sieve = 81.0%
				8	SS			3-3-4	21.9		
			20				●				
	442.4	21.0									
Black, wet, fine coal ash (FILL)				9	SS			4-2-2			
				10	SS			3-1-1	56.5		**Sample No. SS-10:** Atterberg limits: non-plastic Passing No. 200 sieve = 74.5%
			25								
				11	SS			0-0-0			
				12	SS			0-0-0			
			30								
				13	SS			0-0-1	71.2		**Sample No. SS-13:** Atterberg limits: non-plastic Passing No. 200 sieve = 74.4%
				14	SS			5-1-1			
			35								
				15	SS			0-0-0			**Sample No. SS-16:** Atterberg limits: non-plastic Passing No. 200 sieve = 78.9%
				16	SS			1-0-0	57.7		

Sample Type
SS - Driven Split Spoon
ST - Pressed Shelby Tube
CA - Continuous Flight Auger
RC - Rock Core
CU - Cuttings
CT - Continuous Tube

Depth to Groundwater
● Noted on Drilling Tools **20.5** ft.
▽ At Completion **--** ft.
▼ After **--** hours **--** ft.
☒ Cave Depth **--** ft.

Boring Method
HSA - Hollow Stem Augers
CFA - Continuous Flight Augers
CA - Casing Advancer
MD - Mud Drilling
HA - Hand Auger

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(317) 849-4990
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TEST BORING LOG

CLIENT **Vectren Corporation**
PROJECT NAME **Ash Pond Safety Factor Assessment**
PROJECT LOCATION **A.B. Brown Generating Facility**
Posey County, Indiana
BORING # **B-102**
JOB # **170GC00108**

DRILLING and SAMPLING INFORMATION

Date Started	**4/14/15**	Hammer Wt.	**140** lbs.
Date Completed	**4/14/15**	Hammer Drop	**30** in.
Drill Foreman	**W. Bates**	Spoon Sampler OD	**2.0** in.
Inspector	**S. Marcum**	Rock Core Dia.	**--** in.
Boring Method	**HSA**	Shelby Tube OD	**--** in.

TEST DATA

SOIL CLASSIFICATION (continued)	Stratum Elevation	Stratum Depth, ft	Depth Scale, ft	Sample No.	Sample Type	Sampler Graphics Recovery Graphics	Groundwater	Standard Penetration Test, Blows per 6 in. Increments	Moisture Content, %	Pocket Penetrometer PP-tsf	Remarks
Black, wet, fine coal ash (FILL)				17	SS	☒		0-0-0			
			45	18	SS	☒		0-0-0			
				19	SS	☒ ■		0-0-0			
			50	20	SS	☒ ■		0-0-0	54.8		**Sample No. SS-20:** Atterberg limits: non-plastic Passing No. 200 sieve = 94.9%
				21	SS	☒		0-0-0			
			55	22	SS	☒		1-0-0			
				23	SS	☒		0-0-0			
			60	24	SS	☒		0-0-0			
				25	SS	☒		0-0-0			
			65	26	SS	☒		0-0-0			
	396.4	67.0		27	SS	☒ ■		1-2-1			
Reddish brown, very moist, medium stiff to stiff, SILTY CLAY (CL)			70	28	SS	☒ ■		3-3-4	24.3	1.0	**Sample No. SS-28:** Atterberg limits: LL=31, PL=20, PI=11 Passing No. 200 sieve = 98.0%
	390.4	73.0		29	SS	☒ ■		3-3-3	25.9		
Reddish brown, very moist, medium stiff to stiff, SILTY CLAY (CL-ML)			75	30	SS	☒ ■		3-3-3	24.6	1.0	**Sample No. SS-30:** Atterberg limits: LL=29, PL=22, PI=7 Passing No. 200 sieve = 91.0%
				31	SS	☒ ■		3-5-5	32.2	1.5	
Bottom of Test Boring at 80.0 ft	383.4	80.0		32	SS	☒ ■		5-6-7			

Sample Type
SS - Driven Split Spoon
ST - Pressed Shelby Tube
CA - Continuous Flight Auger
RC - Rock Core
CU - Cuttings
CT - Continuous Tube

Depth to Groundwater
● Noted on Drilling Tools **20.5** ft.
▽ At Completion **--** ft.
▼ After **--** hours **--** ft.
☒ Cave Depth **--** ft.

Boring Method
HSA - Hollow Stem Augers
CFA - Continuous Flight Augers
CA - Casing Advancer
MD - Mud Drilling
HA - Hand Auger

Cardno ATC
7988 Centerpoint Drive, Suite 100
Indianapolis, IN 46256
(317) 849-4990
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TEST BORING LOG

CLIENT **Vectren Corporation**
PROJECT NAME **Ash Pond Safety Factor Assessment**
PROJECT LOCATION **A.B. Brown Generating Facility**
Posey County, Indiana
BORING # **B-103**
JOB # **170GC00108**

DRILLING and SAMPLING INFORMATION

Date Started **4/15/15**
Date Completed **4/15/15**
Drill Foreman **W. Bates**
Inspector **S. Marcum**
Boring Method **HSA**
Hammer Wt. **140** lbs.
Hammer Drop **30** in.
Spoon Sampler OD **2.0** in.
Rock Core Dia. **--** in.
Shelby Tube OD **--** in.

TEST DATA

SOIL CLASSIFICATION SURFACE ELEVATION 463.7	Stratum Elevation	Stratum Depth, ft	Depth Scale, ft	Sample No.	Sample Type	Sampler Graphics Recovery Graphics	Groundwater	Standard Penetration Test, Blows per 6 in. Increments	Moisture Content, %	Pocket Penetrometer PP-tsf	Remarks
Brown, moist, sandy clay with coal ash (EMBANKMENT FILL)				1	SS			3-3-3			Boring coordinates and ground surface elevation surveyed by Three I Design.
			5	2	SS			5-6-6	20.5	0.5	**Sample No. SS-2:** Passing No. 200 sieve = 71.0%
	455.7	8.0		3	SS			7-10-11			Borehole backfilled with cement/bentonite grout.
Black, very moist, fine to coarse, coal ash (EMBANKMENT FILL) -wet below 10.5 ft			10	4	SS		●	6-5-4			
	450.7	13.0		5	SS			5-3-4	24.5		**Sample No. SS-5:** Atterberg limits: non-plastic Passing No. 200 sieve = 62.7%
Reddish brown, moist, silty clay (EMBANKMENT FILL)			15	6	SS			4-5-6	18.1	2.5	
				7	SS			7-7-6	17.1	3.5	**Sample No. SS-7:** Atterberg limits: LL=42, PL=18, PI=24 Passing No. 200 sieve = 90.2%
	443.7	20.0	20	8	ST				23.2	1.8	
Black, wet, fine, coal ash (FILL)				9	SS			2-2-2			**Sample No. ST-8:** Atterberg limits: LL=41, PL=18, PI=23 Passing No. 200 sieve = 95.4%
			25	10	SS			1-0-0			**Sample No. SS-10:** Atterberg limits: non-plastic Passing No. 200 sieve = 97.3%
				11	SS			0-0-0	62.9		
			30	12	SS			0-0-0			
				13	SS			0-1-1			
			35	14	SS			0-0-0			
	425.7	38.0		15	SS			0-0-0			**Sample No. SS-15:** Atterberg limits: non-plastic Passing No. 200 sieve = 96.0%
Brown, very moist to moist, soft to medium stiff, SILTY CLAY (CL)				16	SS			0-2-3	72.4	0.5	

Sample Type
SS - Driven Split Spoon
ST - Pressed Shelby Tube
CA - Continuous Flight Auger
RC - Rock Core
CU - Cuttings
CT - Continuous Tube

Depth to Groundwater
● Noted on Drilling Tools **10.5** ft.
▽ At Completion **--** ft.
▼ After **--** hours **--** ft.
☒ Cave Depth **--** ft.

Boring Method
HSA - Hollow Stem Augers
CFA - Continuous Flight Augers
CA - Casing Advancer
MD - Mud Drilling
HA - Hand Auger

TEST BORING LOG

CLIENT **Vectren Corporation**
PROJECT NAME **Ash Pond Safety Factor Assessment**
PROJECT LOCATION **A.B. Brown Generating Facility**
Posey County, Indiana

BORING # **B-103**
JOB # **170GC00108**

DRILLING and SAMPLING INFORMATION

Date Started **4/15/15**
Date Completed **4/15/15**
Drill Foreman **W. Bates**
Inspector **S. Marcum**
Boring Method **HSA**

Hammer Wt. **140** lbs.
Hammer Drop **30** in.
Spoon Sampler OD **2.0** in.
Rock Core Dia. **--** in.
Shelby Tube OD **--** in.

TEST DATA

SOIL CLASSIFICATION (continued)	Stratum Elevation	Stratum Depth, ft	Depth Scale, ft	Sample No.	Sample Type	Sampler Graphics Recovery Graphics	Groundwater	Standard Penetration Test, Blows per 6 in. Increments	Moisture Content, %	Pocket Penetrometer PP-tsf	Remarks
Brown, very moist to moist, soft to medium stiff, SILTY CLAY (CL)				17	SS			6-5-4	29.5		
			45	18	SS			0-2-3	25.2	0.75	**Sample No. SS-18:** Atterberg limits: LL=27, PL=18, PI=9 Passing No. 200 sieve = 97.2%
				19	SS			5-4-5	23.0		
	415.7	48.0									
Reddish Brown, moist, medium stiff to very stiff, SANDY CLAY (CL)				20	SS			3-3-3	20.3		
			50	21	SS			4-4-4	25.8	0.75	
			55	22	SS			5-6-6	20.1	1.0	**Sample No. SS-22:** Atterberg limits: LL=29, PL=11, PI=18 Passing No. 200 sieve = 53.0%
				23	SS			7-11-15	15.6	2.0	
	405.7	58.0									
Light brown, slightly moist, hard, SILT (ML)				24	SS			9-12-21	18.2	3.5	
	402.7	61.0	60								
Light brown, weathered, SILTSTONE	402.4	61.3		25	SS			50/0.3			
Bottom of Test Boring at 61.3 ft											

Sample Type
SS - Driven Split Spoon
ST - Pressed Shelby Tube
CA - Continuous Flight Auger
RC - Rock Core
CU - Cuttings
CT - Continuous Tube

Depth to Groundwater
● Noted on Drilling Tools **10.5** ft.
▽ At Completion **--** ft.
▼ After **--** hours **--** ft.
▨ Cave Depth **--** ft.

Boring Method
HSA - Hollow Stem Augers
CFA - Continuous Flight Augers
CA - Casing Advancer
MD - Mud Drilling
HA - Hand Auger

ATC Associates Inc.

RECORD OF SUBSURFACE EXPLORATION

Client SIGECO

Project Name Ash Pond Embankment

Project Location A.B. Brown Generating Station; West Franklin, Indiana

Boring # B-3

Job # 86.33159.0022

DRILLING and SAMPLING INFORMATION

Date Started	4/9/02	Hammer Wt.	140 lbs.
Date Completed	4/9/02	Hammer Drop	30 in.
Drill Foreman	W. Bates	Spoon Sampler OD	2.0 in.
Boring Inspector	J. Kleeman	Rock Core Dia.	-- in.
Drill Method	HSA	Shelby Tube OD	3.0 in.

SOIL CLASSIFICATION SURFACE ELEVATION	STRATUM DEPTH	DEPTH ft	SAMPLE NO.	SAMPLE TYPE	GROUND WATER	BLOWS/6-INCH INCREMENTS THREE 6-INCH INCREMENTS	RECOVERY, Percent (%)	BORING AND SAMPLING NOTES
Gray to black moist medium dense Coal Ash								
			1	SS		5/5/6	100	
					▽			Pushed tube from 6.0 ft to 8.0 ft.
			2	SS		3/3/4	95	
		5						
			ST 1	ST			100	
			4	SS		3/18/25	100	
		10						Bulk Sample
			5	SS		12/7/41	100	
			6	SS		7/11/10	100	
		15						
-wet and loose below 16 ft								
			7	SS		4/4/3	100	
-very loose below 18 ft								
			8	SS		1/0/1	100	
		20						

SAMPLER TYPE

SS — DRIVEN SPLIT SPOON
ST — PRESSED SHELBY TUBE
CA — CASING ADVANCER
RC — ROCK CORING
CU — CUTTING
CT — CONTINUOUS TUBE

▽ AT COMPLETION 3.0 FT.
▼ AFTER HRS. FT.
● WATER ON RODS FT.

HSA — HOLLOW STEM AUGERS
CFA — CONTINUOUS FLIGHT AUGERS
MD — MUD DRILLING
HA — HAND AUGER

ATC Associates Inc.

RECORD OF SUBSURFACE EXPLORATION

Client SIGECO
Boring # B-3
Project Name Ash Pond Embankment
Job # 86.33159.0022
Project Location A.B. Brown Generating Station; West Franklin, Indiana

DRILLING and SAMPLING INFORMATION

Date Started 4/9/02
Date Completed 4/9/02
Drill Foreman W. Bates
Boring Inspector J. Kleeman
Drill Method HSA

Hammer Wt. 140 lbs.
Hammer Drop 30 in.
Spoon Sampler OD 2.0 in.
Rock Core Dia. -- in.
Shelby Tube OD 3.0 in.

SOIL CLASSIFICATION SURFACE ELEVATION	STRATUM DEPTH	DEPTH ft	SAMPLE NO.	SAMPLE TYPE	GROUND WATER	BLOWS/6-INCH INCREMENTS THREE 6-INCH INCREMENTS	RECOVERY, Percent (%)	BORING AND SAMPLING NOTES
Gray to black wet very loose Coal Ash			9	SS		0/0/0	30	
		25	10	SS		0/0/0	100	
			11	SS		1/0/0	100	
		30	12	SS		0/0/0	100	
			13	SS		3/4/4	50	
Brown moist soft to medium stiff SILTY CLAY (CL-ML)	33.5	35	14	SS		2/1/2	100	
			15	SS		4/4/4	100	
Bottom of Test Boring at 37.5 ft	37.5							

SAMPLER TYPE

SS – DRIVEN SPLIT SPOON
ST – PRESSED SHELBY TUBE
CA – CASING ADVANCER
RC – ROCK CORING
CU – CUTTING
CT – CONTINUOUS TUBE

▽ AT COMPLETION 3.0 FT.
▼ AFTER HRS. FT.
● WATER ON RODS FT.

HSA – HOLLOW STEM AUGERS
CFA – CONTINUOUS FLIGHT AUGERS
MD – MUD DRILLING
HA – HAND AUGER

ATC Associates Inc.

RECORD OF SUBSURFACE EXPLORATION

Client SIGECO

Boring # B-5

Project Name Ash Pond Embankment

Job # 86.33159.0022

Project Location A.B. Brown Generating Station; West Franklin, Indiana

DRILLING and SAMPLING INFORMATION

Date Started	4/9/02	Hammer Wt.	140 lbs.
Date Completed	4/9/02	Hammer Drop	30 in.
Drill Foreman	W. Bates	Spoon Sampler OD	2.0 in.
Boring Inspector	J. Kleeman	Rock Core Dia.	— in.
Drill Method	HSA	Shelby Tube OD	3.0 in.

SOIL CLASSIFICATION SURFACE ELEVATION	STRATUM DEPTH	DEPTH ft	SAMPLE NO.	SAMPLE TYPE	GROUND WATER	BLOWS/6-INCH INCREMENTS THREE 6-INCH INCREMENTS	RECOVERY, Percent (%)	BORING AND SAMPLING NOTES
Gray to black moist very loose to medium dense Coal Ash			1	SS		2/2/3	100	
			T 1	ST	▽		7	
-wet below 5 ft		5						
			3	SS		0/0/1	100	
			4	SS		0/0/0	100	
-moist below 10 ft		10						Bulk Sample
			5	SS		8/6/11	100	
			6	SS		5/9/10	100	
		15						
			7	SS		2/0/6	100	
			8	SS		3/8/11	100	
		20						

SAMPLER TYPE

SS – DRIVEN SPLIT SPOON
ST – PRESSED SHELBY TUBE
CA – CASING ADVANCER
RC – ROCK CORING
CU – CUTTING
CT – CONTINUOUS TUBE

▽ AT COMPLETION 3.0 FT.*
▼ AFTER HRS. FT.
● WATER ON RODS FT.

HSA – HOLLOW STEM AUGERS
CFA – CONTINUOUS FLIGHT AUGERS
MD – MUD DRILLING
HA – HAND AUGER

ATC Associates Inc.

RECORD OF SUBSURFACE EXPLORATION

Client SIGECO

Boring # B-5

Project Name Ash Pond Embankment

Job # 86.33159.0022

Project Location A.B. Brown Generating Station; West Franklin, Indiana

DRILLING and SAMPLING INFORMATION

Date Started	4/9/02	Hammer Wt.	140 lbs.
Date Completed	4/9/02	Hammer Drop	30 in.
Drill Foreman	W. Bates	Spoon Sampler OD	2.0 in.
Boring Inspector	J. Kleeman	Rock Core Dia.	-- in.
Drill Method	HSA	Shelby Tube OD	3.0 in.

SOIL CLASSIFICATION SURFACE ELEVATION	STRATUM DEPTH	DEPTH ft	SAMPLE NO.	SAMPLE TYPE	GROUND WATER	BLOWS/6-INCH INCREMENTS THREE 6-INCH INCREMENTS	RECOVERY, Percent (%)	BORING AND SAMPLING NOTES
Gray to black wet very loose to loose Coal Ash								
			9	SS		4/2/1	100	
			10	SS		8/3/4	100	
		25						
			11	SS		2/1/0	30	
			12	SS		0/0/0	100	
		30						
			13	SS		2/1/0	100	
			14	SS		0/0/0	30	
		35						
			15	SS		0/0/0	80	
			16	SS		0/0/0	75	
		40						

SAMPLER TYPE

SS – DRIVEN SPLIT SPOON
ST – PRESSED SHELBY TUBE
CA – CASING ADVANCER
RC – ROCK CORING
CU – CUTTING
CT – CONTINUOUS TUBE

∇ AT COMPLETION 3.0 FT.*
▼ AFTER HRS. FT.
● WATER ON RODS FT.

HSA – HOLLOW STEM AUGERS
CFA – CONTINUOUS FLIGHT AUGERS
MD – MUD DRILLING
HA – HAND AUGER

ATC Associates Inc.

RECORD OF SUBSURFACE EXPLORATION

Client SIGECO

Boring # B-5

Project Name Ash Pond Embankment

Job # 86.33159.0022

Project Location A.B. Brown Generating Station; West Franklin, Indiana

DRILLING and SAMPLING INFORMATION

Date Started	4/9/02	Hammer Wt.	140 lbs.
Date Completed	4/9/02	Hammer Drop	30 in.
Drill Foreman	W. Bates	Spoon Sampler OD	2.0 in.
Boring Inspector	J. Kleeman	Rock Core Dia.	-- in.
Drill Method	HSA	Shelby Tube OD	3.0 in.

SOIL CLASSIFICATION SURFACE ELEVATION	STRATUM DEPTH	DEPTH ft	SAMPLE NO.	SAMPLE TYPE	GROUND WATER	BLOWS/6-INCH INCREMENTS THREE 6-INCH INCREMENTS	RECOVERY, Percent (%)	BORING AND SAMPLING NOTES
Gray to black wet very loose to loose Coal Ash	41.0							
Gray to brown soft to medium stiff SILTY CLAY (CL-ML)			17	SS		3/3/7	50	
			18	SS		3/3/4	80	
		45						
			19	SS		2/1/2	75	
	47.5							
Bottom of Test Boring at 47.5 ft								

SAMPLER TYPE

SS – DRIVEN SPLIT SPOON
ST – PRESSED SHELBY TUBE
CA – CASING ADVANCER
RC – ROCK CORING
CU – CUTTING
CT – CONTINUOUS TUBE

▽ AT COMPLETION 3.0 FT.*
▼ AFTER HRS. FT.
● WATER ON RODS FT.

HSA – HOLLOW STEM AUGERS
CFA – CONTINUOUS FLIGHT AUGERS
MD – MUD DRILLING
HA – HAND AUGER

ATC Associates Inc.

RECORD OF SUBSURFACE EXPLORATION

Client SIGECO

Project Name Ash Pond Embankment

Project Location A.B. Brown Generating Station; West Franklin, Indiana

Boring # B-6

Job # 86.33159.0022

DRILLING and SAMPLING INFORMATION

Date Started	4/9/02	Hammer Wt.	140 lbs.
Date Completed	4/9/02	Hammer Drop	30 in.
Drill Foreman	W. Bates	Spoon Sampler OD	2.0 in.
Boring Inspector	J. Kleeman	Rock Core Dia.	-- in.
Drill Method	HSA	Shelby Tube OD	3.0 in.

SOIL CLASSIFICATION SURFACE ELEVATION	STRATUM DEPTH	DEPTH ft	SAMPLE NO.	SAMPLE TYPE	GROUND WATER	BLOWS/6-INCH INCREMENTS THREE 6-INCH INCREMENTS	RECOVERY, Percent (%)	BORING AND SAMPLING NOTES
Gray to black moist very loose Coal Ash								
			1	SS		2/1/0	30	
-wet below 4.5 ft			2	SS	⊻	4/1/1	30	
		5						
			3	SS		0/0/0	0	
			4	SS		0/0/0	0	
		10						
			5	SS		0/0/0	100	
			6	SS		0/0/0	100	
		15						
			7	SS		1/1/1	100	
			8	SS		0/0/0	50	
		20						

SAMPLER TYPE

SS – DRIVEN SPLIT SPOON
ST – PRESSED SHELBY TUBE
CA – CASING ADVANCER
RC – ROCK CORING
CU – CUTTING
CT – CONTINUOUS TUBE

⊻ AT COMPLETION 4.5 FT.
⊻ AFTER HRS. FT.
● WATER ON RODS FT.

HSA – HOLLOW STEM AUGERS
CFA – CONTINUOUS FLIGHT AUGERS
MD – MUD DRILLING
HA – HAND AUGER

ATC Associates Inc.

RECORD OF SUBSURFACE EXPLORATION

Client SIGECO

Boring # B-6

Project Name Ash Pond Embankment

Job # 86.33159.0022

Project Location A.B. Brown Generating Station; West Franklin, Indiana

DRILLING and SAMPLING INFORMATION

Date Started	4/9/02	Hammer Wt.	140 lbs.
Date Completed	4/9/02	Hammer Drop	30 in.
Drill Foreman	W. Bates	Spoon Sampler OD	2.0 in.
Boring Inspector	J. Kleeman	Rock Core Dia.	-- in.
Drill Method	HSA	Shelby Tube OD	3.0 in.

SOIL CLASSIFICATION SURFACE ELEVATION	STRATUM DEPTH	DEPTH ft	SAMPLE NO.	SAMPLE TYPE	GROUND WATER	BLOWS/6-INCH INCREMENTS THREE 6-INCH INCREMENTS	RECOVERY, Percent (%)	BORING AND SAMPLING NOTES
Gray to black wet very loose Coal Ash								
			9	SS		0/0/1	100	
			10	SS		0/5/4	100	
		25						
			11	SS		1/0/1	100	
			12	SS		0/1/0	100	
		30						
			13	SS		0/0/0	100	
			14	SS		0/0/0	100	
		35						
			15	SS		0/0/0	100	
			16	SS		0/0/0	100	
		40						

SAMPLER TYPE

SS – DRIVEN SPLIT SPOON
ST – PRESSED SHELBY TUBE
CA – CASING ADVANCER
RC – ROCK CORING
CU – CUTTING
CT – CONTINUOUS TUBE

∇ AT COMPLETION 4.5 FT.
▼ AFTER HRS. FT.
● WATER ON RODS FT.

HSA – HOLLOW STEM AUGERS
CFA – CONTINUOUS FLIGHT AUGERS
MD – MUD DRILLING
HA – HAND AUGER

ATC Associates Inc.

RECORD OF SUBSURFACE EXPLORATION

Client SIGECO

Boring # **B-6**

Project Name Ash Pond Embankment

Job # 86.33159.0022

Project Location A.B. Brown Generating Station; West Franklin, Indiana

DRILLING and SAMPLING INFORMATION

Date Started	4/9/02	Hammer Wt.	140 lbs.
Date Completed	4/9/02	Hammer Drop	30 in.
Drill Foreman	W. Bates	Spoon Sampler OD	2.0 in.
Boring Inspector	J. Kleeman	Rock Core Dia.	-- in.
Drill Method	HSA	Shelby Tube OD	3.0 in.

SOIL CLASSIFICATION SURFACE ELEVATION	STRATUM DEPTH	DEPTH ft	SAMPLE NO.	SAMPLE TYPE	GROUND WATER	BLOWS/6-INCH INCREMENTS THREE 6-INCH INCREMENTS	RECOVERY, Percent (%)	BORING AND SAMPLING NOTES
Gray to black wet very loose Coal Ash			17	SS		0/0/0	100	
Brown to gray moist soft SILTY CLAY (CL-ML)	43.5		18	SS		1/2/3	100	
		45						
	47.5		19	SS		2/1/3	100	
Bottom of Test Boring at 47.5 ft								

SAMPLER TYPE

SS – DRIVEN SPLIT SPOON
ST – PRESSED SHELBY TUBE
CA – CASING ADVANCER
RC – ROCK CORING
CU – CUTTING
CT – CONTINUOUS TUBE

∇ AT COMPLETION **4.5** FT.
▼ AFTER HRS. FT.
● WATER ON RODS FT.

HSA – HOLLOW STEM AUGERS
CFA – CONTINUOUS FLIGHT AUGERS
MD – MUD DRILLING
HA – HAND AUGER

ATC Associates Inc.

RECORD OF SUBSURFACE EXPLORATION

Client SIGECO

Boring # B-7

Project Name Ash Pond Embankment

Job # 86.33159.0022

Project Location A.B. Brown Generating Station; West Franklin, Indiana

DRILLING and SAMPLING INFORMATION

Date Started	4/9/02	Hammer Wt.	140 lbs.
Date Completed	4/9/02	Hammer Drop	30 in.
Drill Foreman	W. Bates	Spoon Sampler OD	2.0 in.
Boring Inspector	J. Kleeman	Rock Core Dia.	-- in.
Drill Method	HSA	Shelby Tube OD	3.0 in.

SOIL CLASSIFICATION SURFACE ELEVATION	STRATUM DEPTH	DEPTH ft	SAMPLE NO.	SAMPLE TYPE	GROUND WATER	BLOWS/6-INCH INCREMENTS THREE 6-INCH INCREMENTS	RECOVERY, Percent (%)	BORING AND SAMPLING NOTES
Gray to black moist very loose to loose Coal Ash								
			1	SS		1/1/1	30	Pushed tube from 6.0 ft to 8.0 ft.
			2	SS		2/1/5	30	
		5						
-wet below 6 ft					▽			
			ST 1	ST			100	
			3	SS		6/7/5	100	
		10						Bulk Sample
			4	SS		1/0/0	60	
			5	SS		0/0/0	100	
		15						
			6	SS		0/0/0	50	
			7	SS		0/0/0	100	
		20						

SAMPLER TYPE

SS – DRIVEN SPLIT SPOON
ST – PRESSED SHELBY TUBE
CA – CASING ADVANCER
RC – ROCK CORING
CU – CUTTING
CT – CONTINUOUS TUBE

▽ AT COMPLETION 6.0 FT.*
▼ AFTER HRS. FT.
● WATER ON RODS FT.

HSA – HOLLOW STEM AUGERS
CFA – CONTINUOUS FLIGHT AUGERS
MD – MUD DRILLING
HA – HAND AUGER

ATC Associates Inc.

RECORD OF SUBSURFACE EXPLORATION

Client SIGECO

Project Name Ash Pond Embankment

Project Location A.B. Brown Generating Station; West Franklin, Indiana

Boring # B-7

Job # 86.33159.0022

DRILLING and SAMPLING INFORMATION

Date Started	4/9/02	Hammer Wt.	140 lbs.
Date Completed	4/9/02	Hammer Drop	30 in.
Drill Foreman	W. Bates	Spoon Sampler OD	2.0 in.
Boring Inspector	J. Kleeman	Rock Core Dia.	-- in.
Drill Method	HSA	Shelby Tube OD	3.0 in.

SOIL CLASSIFICATION SURFACE ELEVATION	STRATUM DEPTH	DEPTH ft	SAMPLE NO.	SAMPLE TYPE	GROUND WATER	BLOWS/6-INCH INCREMENTS THREE 6-INCH INCREMENTS	RECOVERY, Percent (%)	BORING AND SAMPLING NOTES
Gray to black moist very loose to loose Coal Ash			8	SS		0/0/0	100	
		25	9	SS		0/0/0	100	
			10	SS		0/0/0	50	
		30	11	SS		0/0/0	10	
			12	SS		0/0/0	100	
		35	13	SS		0/0/0	100	
			14	SS		0/0/0	100	
		40	15	SS		0/0/0	10	

SAMPLER TYPE

SS – DRIVEN SPLIT SPOON
ST – PRESSED SHELBY TUBE
CA – CASING ADVANCER
RC – ROCK CORING
CU – CUTTING
CT – CONTINUOUS TUBE

∇ AT COMPLETION 6.0 FT.*
▼ AFTER HRS. FT.
● WATER ON RODS FT.

HSA – HOLLOW STEM AUGERS
CFA – CONTINUOUS FLIGHT AUGERS
MD – MUD DRILLING
HA – HAND AUGER

ATC Associates Inc.

RECORD OF SUBSURFACE EXPLORATION

Client SIGECO

Boring # B-7

Project Name Ash Pond Embankment

Job # 86.33159.0022

Project Location A.B. Brown Generating Station; West Franklin, Indiana

DRILLING and SAMPLING INFORMATION

Date Started 4/9/02

Date Completed 4/9/02

Drill Foreman W. Bates

Boring Inspector J. Kleeman

Drill Method HSA

Hammer Wt. 140 lbs.

Hammer Drop 30 in.

Spoon Sampler OD 2.0 in.

Rock Core Dia. -- in.

Shelby Tube OD 3.0 in.

SOIL CLASSIFICATION SURFACE ELEVATION	STRATUM DEPTH	DEPTH ft	SAMPLE NO.	SAMPLE TYPE	GROUND WATER	BLOWS/6-INCH INCREMENTS THREE 6-INCH INCREMENTS	RECOVERY, Percent (%)	BORING AND SAMPLING NOTES
Gray to black moist very loose to loose Coal Ash			16	SS		0/0/0	80	
			17	SS		0/0/0	30	
		45						
	46.0							
Brown and gray moist soft to medium stiff SILTY CLAY (CL)			18	SS		3/3/2	40	
			19	SS		3/3/4	70	
	50.0	50						
Bottom of Test Boring at 50.0 ft								

SAMPLER TYPE

SS – DRIVEN SPLIT SPOON
ST – PRESSED SHELBY TUBE
CA – CASING ADVANCER
RC – ROCK CORING
CU – CUTTING
CT – CONTINUOUS TUBE

AT COMPLETION 6.0 FT.*
AFTER HRS. FT.
WATER ON RODS FT.

HSA – HOLLOW STEM AUGERS
CFA – CONTINUOUS FLIGHT AUGERS
MD – MUD DRILLING
HA – HAND AUGER

ATC Associates Inc.

RECORD OF SUBSURFACE EXPLORATION

Client SIGECO

Project Name Ash Pond Embankment

Project Location A.B. Brown Generating Station; West Franklin, Indiana

Boring # B-8

Job # 86.33159.0022

DRILLING and SAMPLING INFORMATION

Date Started 4/8/02

Date Completed 4/8/02

Drill Foreman W. Bates

Boring Inspector J. Kleeman

Drill Method HSA

Hammer Wt. 140 lbs.

Hammer Drop 30 in.

Spoon Sampler OD 2.0 in.

Rock Core Dia. -- in.

Shelby Tube OD 3.0 in.

SOIL CLASSIFICATION SURFACE ELEVATION	STRATUM DEPTH	DEPTH ft	SAMPLE NO.	SAMPLE TYPE	GROUND WATER	BLOWS/6-INCH INCREMENTS THREE 6-INCH INCREMENTS	RECOVERY, Percent (%)	BORING AND SAMPLING NOTES
Gray to black moist very loose to loose Coal Ash								
			1	SS		6/6/4	100	
			2	SS		3/2/1	100	
		5						
-wet below 6 ft					▽			
			3	SS		1/2/0	45	
			4	SS		0/0/1	100	
		10			●			Bulk Sample
			5	SS		0/0/1	100	
			6	SS		0/0/0	100	
		15						
			7	SS		0/0/0	75	
			8	SS		0/0/0	75	
		20						

SAMPLER TYPE

SS – DRIVEN SPLIT SPOON
ST – PRESSED SHELBY TUBE
CA – CASING ADVANCER
RC – ROCK CORING
CU – CUTTING
CT – CONTINUOUS TUBE

▽ AT COMPLETION 6.0 FT.
▽ AFTER HRS. FT.
● WATER ON RODS 10.0 FT.

HSA – HOLLOW STEM AUGERS
CFA – CONTINUOUS FLIGHT AUGERS
MD – MUD DRILLING
HA – HAND AUGER

ATC Associates Inc.

RECORD OF SUBSURFACE EXPLORATION

Client SIGECO
Boring # B-8
Project Name Ash Pond Embankment
Job # 86.33159.0022
Project Location A.B. Brown Generating Station; West Franklin, Indiana

DRILLING and SAMPLING INFORMATION

Date Started	4/8/02	Hammer Wt.	140 lbs.
Date Completed	4/8/02	Hammer Drop	30 in.
Drill Foreman	W. Bates	Spoon Sampler OD	2.0 in.
Boring Inspector	J. Kleeman	Rock Core Dia.	-- in.
Drill Method	HSA	Shelby Tube OD	3.0 in.

SOIL CLASSIFICATION SURFACE ELEVATION	STRATUM DEPTH	DEPTH ft	SAMPLE NO.	SAMPLE TYPE	GROUND WATER	BLOWS/6-INCH INCREMENTS THREE 6-INCH INCREMENTS	RECOVERY, Percent (%)	BORING AND SAMPLING NOTES
Gray to black moist very loose to loose Coal Ash								
			9	SS		0/0/0	100	
			10	SS		0/0/0	50	
		25						
			11	SS		0/0/0	100	
			12	SS		0/0/0	100	
		30						
			13	SS		0/0/0	100	
			14	SS		0/0/0	100	
		35						
	37.0		15	SS		2/1/2	100	
Brown wet soft SILTY CLAY (CL-ML)								
Bottom of Test Boring at 40.0 ft			16	SS		0/2/2	30	
	40.0	40						

SAMPLER TYPE

SS – DRIVEN SPLIT SPOON
ST – PRESSED SHELBY TUBE
CA – CASING ADVANCER
RC – ROCK CORING
CU – CUTTING
CT – CONTINUOUS TUBE

∇ AT COMPLETION 6.0 FT.
∇ AFTER HRS. FT.
● WATER ON RODS 10.0 FT.

HSA – HOLLOW STEM AUGERS
CFA – CONTINUOUS FLIGHT AUGERS
MD – MUD DRILLING
HA – HAND AUGER

ATC Associates Inc.

RECORD OF SUBSURFACE EXPLORATION

Client SIGECO

Project Name Ash Pond Embankment

Project Location A.B. Brown Generating Station; West Franklin, Indiana

Boring # B-10

Job # 86.33159.0022

DRILLING and SAMPLING INFORMATION

Date Started	4/8/02	Hammer Wt.	140 lbs.
Date Completed	4/8/02	Hammer Drop	30 in.
Drill Foreman	W. Bates	Spoon Sampler OD	2.0 in.
Boring Inspector	J. Kleeman	Rock Core Dia.	-- in.
Drill Method	HSA	Shelby Tube OD	3.0 in.

SOIL CLASSIFICATION SURFACE ELEVATION	STRATUM DEPTH	DEPTH ft	SAMPLE NO.	SAMPLE TYPE	GROUND WATER	BLOWS/6-INCH INCREMENTS THREE 6-INCH INCREMENTS	RECOVERY, Percent (%)	BORING AND SAMPLING NOTES
Gray to black moist very loose Coal Ash								
			1	SS		1/1/1	50	
			2	SS		4/2/1	50	
		5						
-wet below 7 ft			3	SS	▽	1/0/0	100	
			4	SS		0/1/0	100	
		10						Bulk Sample
			5	SS		0/0/0	100	
			6	SS		0/0/0	50	
		15						
			7	SS		0/0/0	100	
			8	SS		0/0/0	100	
		20						

SAMPLER TYPE

SS – DRIVEN SPLIT SPOON
ST – PRESSED SHELBY TUBE
CA – CASING ADVANCER
RC – ROCK CORING
CU – CUTTING
CT – CONTINUOUS TUBE

▽ AT COMPLETION 7.0 FT.
▼ AFTER HRS. FT.
● WATER ON RODS FT.

HSA – HOLLOW STEM AUGERS
CFA – CONTINUOUS FLIGHT AUGERS
MD – MUD DRILLING
HA – HAND AUGER

ATC Associates Inc.

RECORD OF SUBSURFACE EXPLORATION

Client SIGECO

Project Name Ash Pond Embankment

Project Location A.B. Brown Generating Station; West Franklin, Indiana

Boring # B-10

Job # 86.33159.0022

DRILLING and SAMPLING INFORMATION

Date Started	4/8/02	Hammer Wt.	140 lbs.
Date Completed	4/8/02	Hammer Drop	30 in.
Drill Foreman	W. Bates	Spoon Sampler OD	2.0 in.
Boring Inspector	J. Kleeman	Rock Core Dia.	-- in.
Drill Method	HSA	Shelby Tube OD	3.0 in.

SOIL CLASSIFICATION SURFACE ELEVATION	STRATUM DEPTH	DEPTH ft	SAMPLE NO.	SAMPLE TYPE	GROUND WATER	BLOWS/6-INCH INCREMENTS THREE 6-INCH INCREMENTS	RECOVERY, Percent (%)	BORING AND SAMPLING NOTES
Gray to black moist very loose Coal Ash			9	SS		0/0/0	100	
		25	10	SS		0/0/0	100	
Brown wet soft SILTY CLAY (CL-ML)	27.0		11	SS		0/0/1	100	
	30.0	30	12	SS		2/2/2	100	
Bottom of Test Boring at 30.0 ft								

SAMPLER TYPE

SS – DRIVEN SPLIT SPOON
ST – PRESSED SHELBY TUBE
CA – CASING ADVANCER
RC – ROCK CORING
CU – CUTTING
CT – CONTINUOUS TUBE

▽ AT COMPLETION 7.0 FT.
▼ AFTER HRS. FT.
● WATER ON RODS FT.

HSA – HOLLOW STEM AUGERS
CFA – CONTINUOUS FLIGHT AUGERS
MD – MUD DRILLING
HA – HAND AUGER

ATC Associates Inc.

RECORD OF SUBSURFACE EXPLORATION

Client SIGECO
Project Name Ash Pond Embankment
Project Location A.B. Brown Generating Station; West Franklin, Indiana

Boring # B-11
Job # 86.33159.0022

DRILLING and SAMPLING INFORMATION

Date Started 4/8/02
Date Completed 4/8/02
Drill Foreman W. Bates
Boring Inspector J. Kleeman
Drill Method HSA

Hammer Wt. 140 lbs.
Hammer Drop 30 in.
Spoon Sampler OD 2.0 in.
Rock Core Dia. -- in.
Shelby Tube OD 3.0 in.

SOIL CLASSIFICATION SURFACE ELEVATION	STRATUM DEPTH	DEPTH ft	SAMPLE NO.	SAMPLE TYPE	GROUND WATER	BLOWS/6-INCH INCREMENTS THREE 6-INCH INCREMENTS	RECOVERY, Percent (%)	BORING AND SAMPLING NOTES
Brown moist very soft Silty Clay (POSSIBLE FILL)			1	SS		2/1/1	100	
Brown wet very soft Clayey Silt (POSSIBLE FILL)	3.0		2	SS		1/1/1	75	
		5	3	SS		1/1/1	100	
Brown moist soft SILTY CLAY (CL-ML)	8.0		4	SS		1/2/3	100	
		10	5	SS		1/2/3	100	
			6	SS		2/2/3	100	
Reddish brown moist medium stiff SILTY CLAY (CL)	16.0	15	7	SS		2/3/3	100	
			8	SS		3/3/4	100	
		20						

SAMPLER TYPE

SS - DRIVEN SPLIT SPOON
ST - PRESSED SHELBY TUBE
CA - CASING ADVANCER
RC - ROCK CORING
CU - CUTTING
CT - CONTINUOUS TUBE

∇ AT COMPLETION FT.
▼ AFTER HRS. FT.
● WATER ON RODS 20.5 FT.

HSA - HOLLOW STEM AUGERS
CFA - CONTINUOUS FLIGHT AUGERS
MD - MUD DRILLING
HA - HAND AUGER

ATC Associates Inc.

RECORD OF SUBSURFACE EXPLORATION

Client: SIGECO
Project Name: Ash Pond Embankment
Project Location: A.B. Brown Generating Station; West Franklin, Indiana
Boring # B-11
Job # 86.33159.0022

DRILLING and SAMPLING INFORMATION

Date Started	4/8/02	Hammer Wt.	140 lbs.
Date Completed	4/8/02	Hammer Drop	30 in.
Drill Foreman	W. Bates	Spoon Sampler OD	2.0 in.
Boring Inspector	J. Kleeman	Rock Core Dia.	-- in.
Drill Method	HSA	Shelby Tube OD	3.0 in.

SOIL CLASSIFICATION SURFACE ELEVATION	STRATUM DEPTH	DEPTH ft	SAMPLE NO.	SAMPLE TYPE	GROUND WATER	BLOWS/6-INCH INCREMENTS THREE 6-INCH INCREMENTS	RECOVERY, Percent (%)	BORING AND SAMPLING NOTES
Reddish brown moist stiff to very stiff SILTY CLAY (CL)					●			
			9	SS		3/6/7	100	
-mottled yellow brown below 23 ft								
			10	SS		4/6/9	100	
	25.0	25						
Bottom of Test Boring at 25.0 ft								

SAMPLER TYPE
SS – DRIVEN SPLIT SPOON
ST – PRESSED SHELBY TUBE
CA – CASING ADVANCER
RC – ROCK CORING
CU – CUTTING
CT – CONTINUOUS TUBE

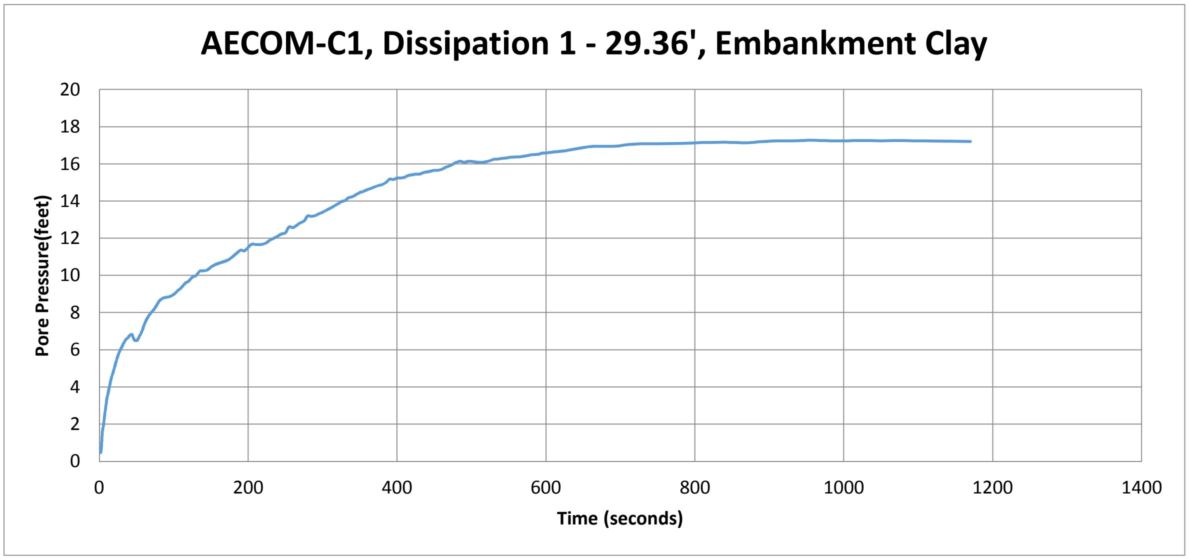
⊻ AT COMPLETION FT.
⊻ AFTER HRS. FT.
● WATER ON RODS 20.5 FT.

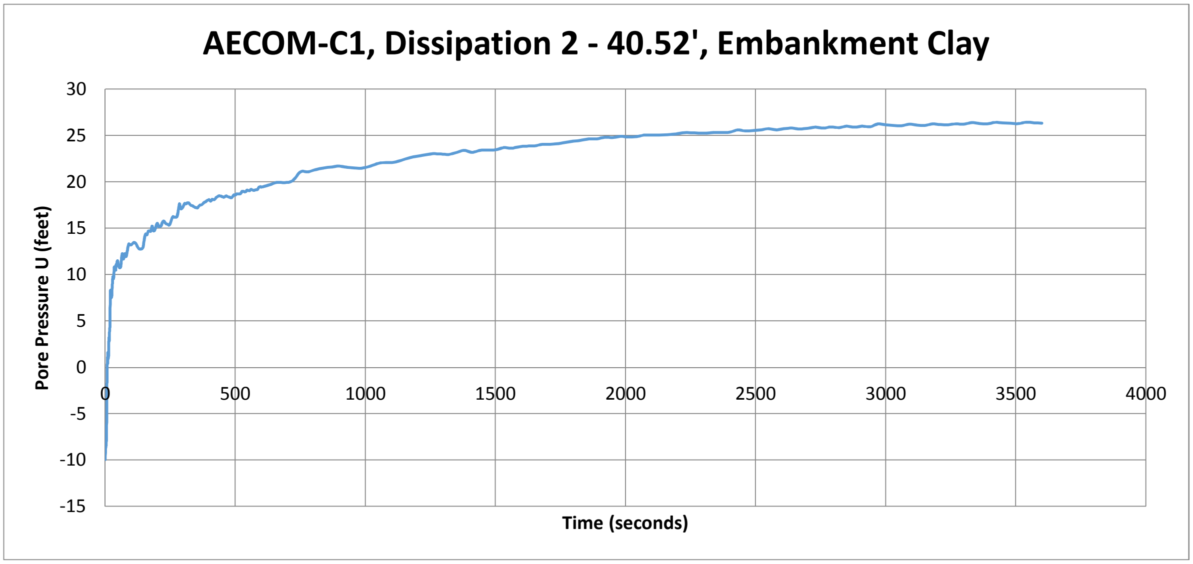
HSA – HOLLOW STEM AUGERS
CFA – CONTINUOUS FLIGHT AUGERS
MD – MUD DRILLING
HA – HAND AUGER

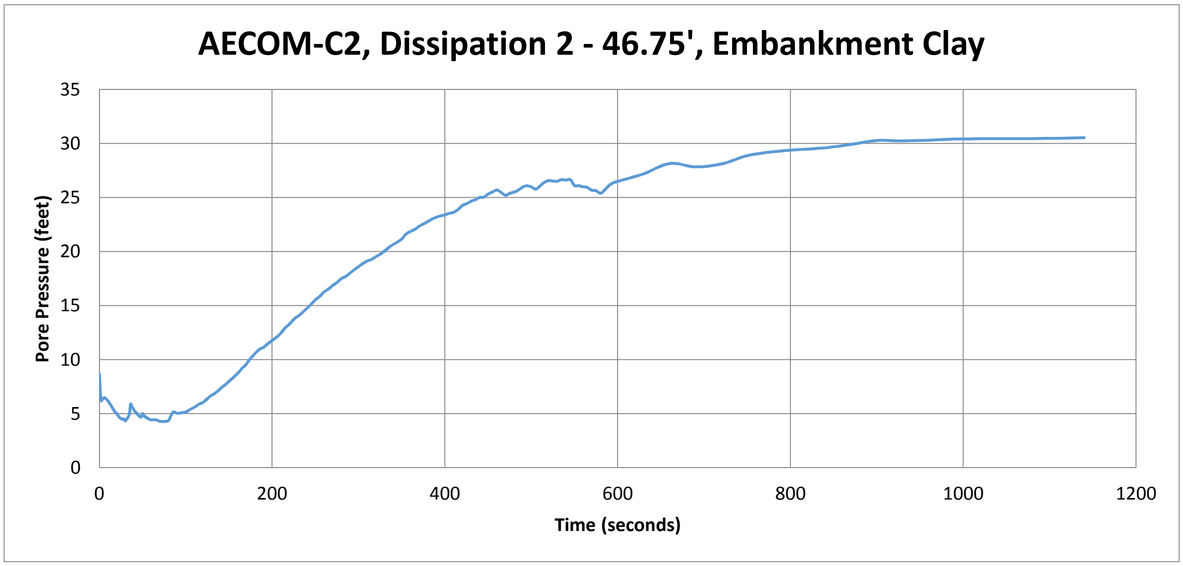
Appendix C
CPT Data Report

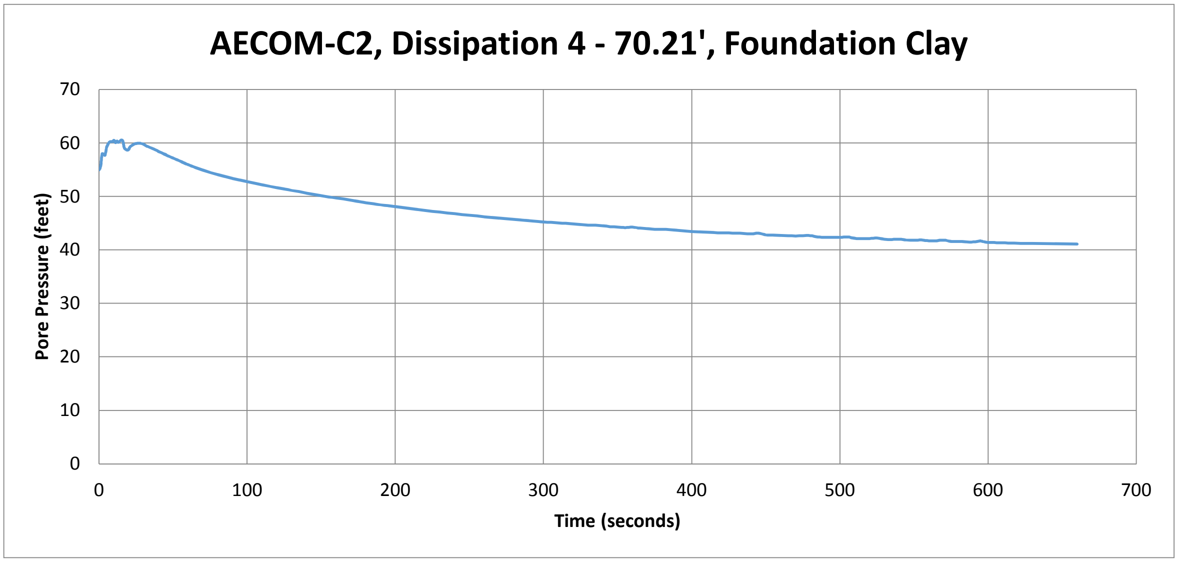
Shear Wave Velocity Summary			
CPT Sounding/Cardno Boring ID	Depth (ft)	Vs (ft/sec)	Material
AECOM-C1/B-202	13.2	704	Embankment Fill
	19.8	733.27	
	26.4	836.22	
	33	846.88	
	39.6	721.85	
	46.2	1185.27	
	52.8	878.12	Foundation Silty Clays
	59.4	696.13	
	66	982.25	
	72.6	747.15	
	79.2	1255.91	
	85.8	1046.85	
	92.4	1283.5	
AECOM-C2/B-203	13.2	823.46	Embankment Fill
	20.1	755.87	
	26.7	988.65	
	33.3	756.69	
	40	922.77	
	46.6	948.49	
	53.2	947.8	
	57	815.09	Foundation Silty Clays
	63.6	830.74	
	74	958.15	
	80.6	780.28	
	87.2	1163.52	
AECOM-C3/B-219	7	562.16	Foundation Silts
	13.6	504.69	
	20.2	631.86	Foundation Silty Clays
	26.8	988.65	
	33.4	928.57	
	40	721.36	
	46.6	991.4	
AECOM-C4/B-206	7.3	607.74	Embankment Fill
	13.9	733.7	
	20.5	765.12	Foundation Silts
	27.1	712.86	
	33.7	984.38	Foundation Silty Clays
	40.1	734.94	Foundation Silts
	46.7	694	Foundation Silty Clays
	53.3	613.52	
	60	792.91	Foundation Silts
	66.6	679.89	
	73.2	883.37	Foundation Silty Clays
AECOM-C5/B-205	6.6	649.54	Embankment Fill
	13.2	773.52	
	20.1	728.25	
	26.7	718.77	Foundation Silts
	33.3	661.41	
	40	723.92	
	45	725.59	
	50.1	725.72	
	55	845.8	Foundation Silty Clays

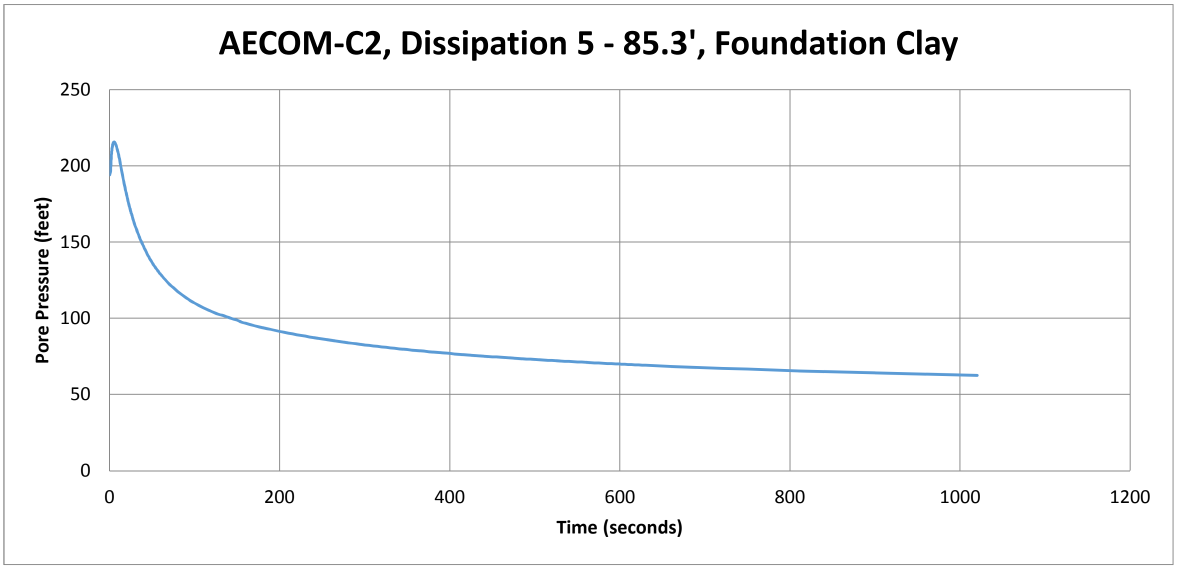
CPT Sounding	Dissipation Test	Depth (ft)	Estimated t50 (sec)	Estimated Hydraulic Conductivity kh (cm/sec)	Material	Run to apparent Equilibrium?
AECOM-C1	1	29.4	100	3.17E-06	Embankment Fill	Yes
	2	40.5	26	1.70E-05		Yes
	3	55.0	Poor Data	Poor Data	Foundation Clay	--
AECOM-C2	1	30.2	645	3.08E-07	Embankment Fill	No
	2	46.8	48	7.92E-06		Yes
	3	55.1	380	5.97E-07		No
	4	70.2	115	2.66E-06	Foundation Clay	Yes
	5	85.3	995	1.79E-07		Yes
AECOM-C3	1	7.4	42	9.36E-06	Foundation Silt	No
	2	14.8	1121	1.54E-07		Yes
	3	7.2	18	2.70E-05		Yes
	4	30.2	1033	1.71E-07	Foundation Clay	No
	5	40.7	477	4.49E-07		No
AECOM-C4	1	7.4	Poor Data	Poor Data	Embankment Fill	--
	2	19.5	600	3.37E-07	Foundation Silt	No
	3	24.9	745	2.57E-07		No
	4	30.0	100	3.17E-06		No
	5	49.9	569	3.60E-07	Foundation Clay	Yes
	6	60.4	375	6.07E-07	Foundation Silt	Yes
	7	65.0	14	3.70E-05		Yes
	8	69.9	172	1.61E-06		No
	9	75.6	330	7.12E-07	Foundation Clay	No
AECOM-C5	1	19.9	900	2.03E-07	Embankment Fill	No
	2	24.9	19	2.52E-05		Yes
	3	30.0	47	8.13E-06	Foundation Silt	Yes
	4	34.9	82	4.06E-06		No
	5	40.0	84	3.94E-06		No
	6	45.0	61	5.87E-06		No
	7	49.9	113	2.72E-06		No
	8	55.0	87	3.77E-06	Foundation Clay	No

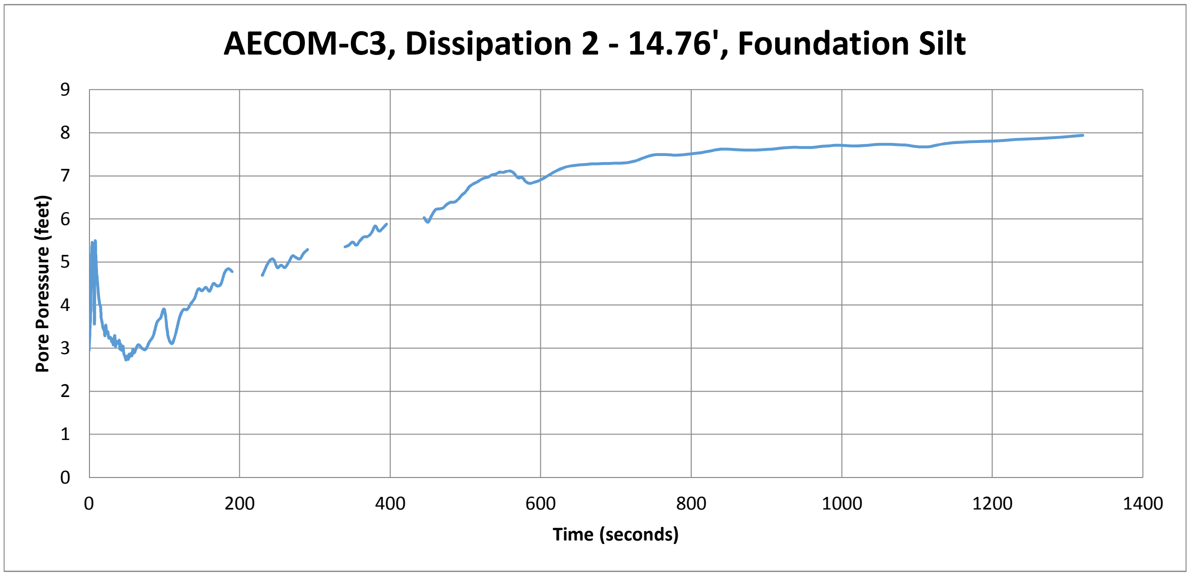

AECOM-C1, Dissipation 1 - 29.36', Embankment Clay
Pore Pressure(feet)
0
2
4
6
8
10
12
14
16
18
20
Time (seconds)
0
200
400
600
800
1000
1200
1400

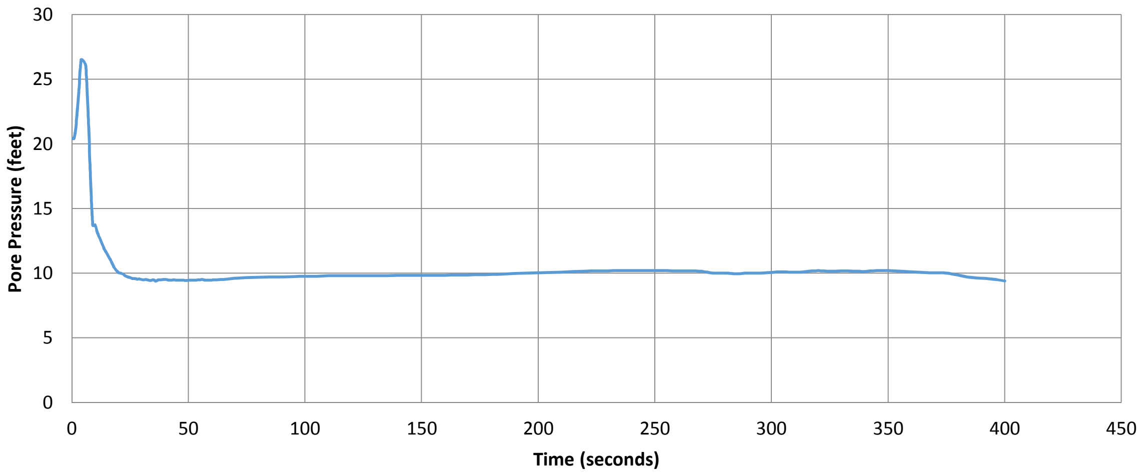

AECOM-C1, Dissipation 2 - 40.52', Embankment Clay
Pore Pressure U (feet)
-15
-10
-5
0
5
10
15
20
25
30
Time (seconds)
0
500
1000
1500
2000
2500
3000
3500
4000


AECOM-C2, Dissipation 2 - 46.75', Embankment Clay
Pore Pressure (feet)
0
5
10
15
20
25
30
35
Time (seconds)
0
200
400
600
800
1000
1200

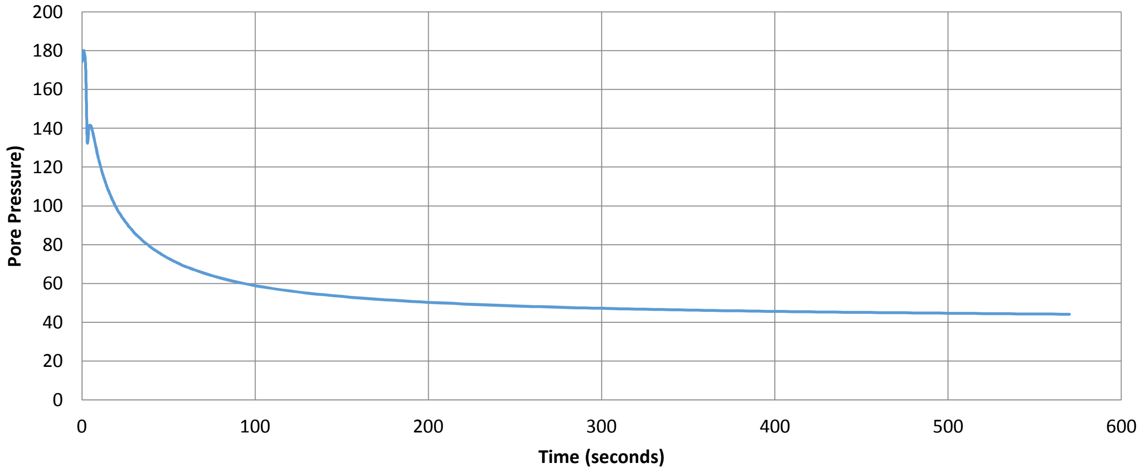

AECOM-C2, Dissipation 4 - 70.21', Foundation Clay


AECOM-C2, Dissipation 5 - 85.3', Foundation Clay

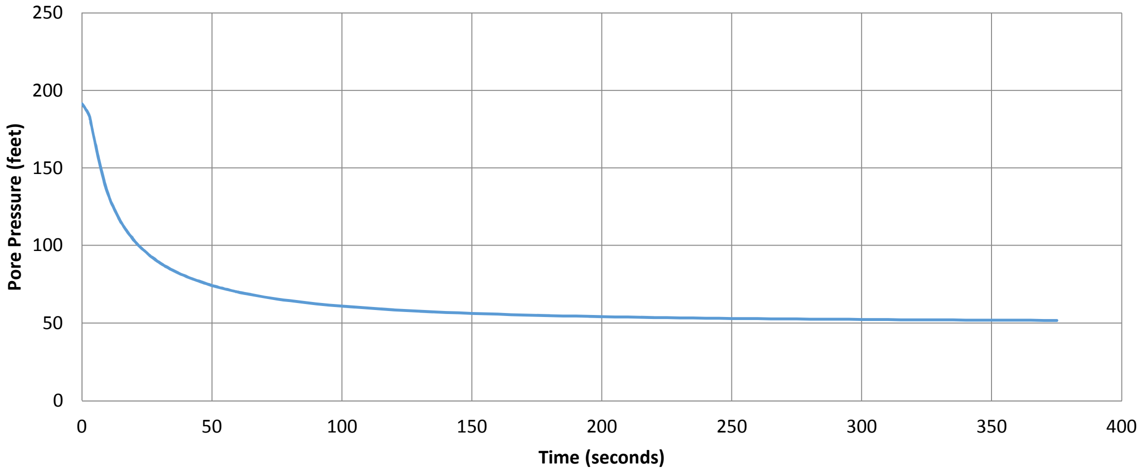

AECOM-C3, Dissipation 2 - 14.76', Foundation Silt



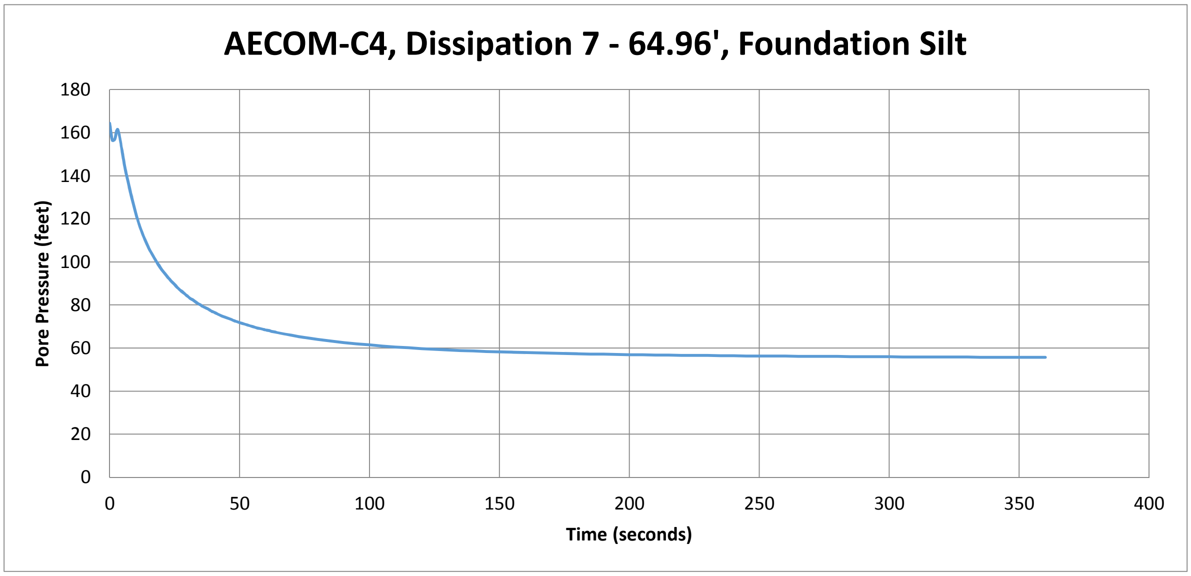
AECOM-C3, Dissipation 3 - 17.22', Foundation Silt

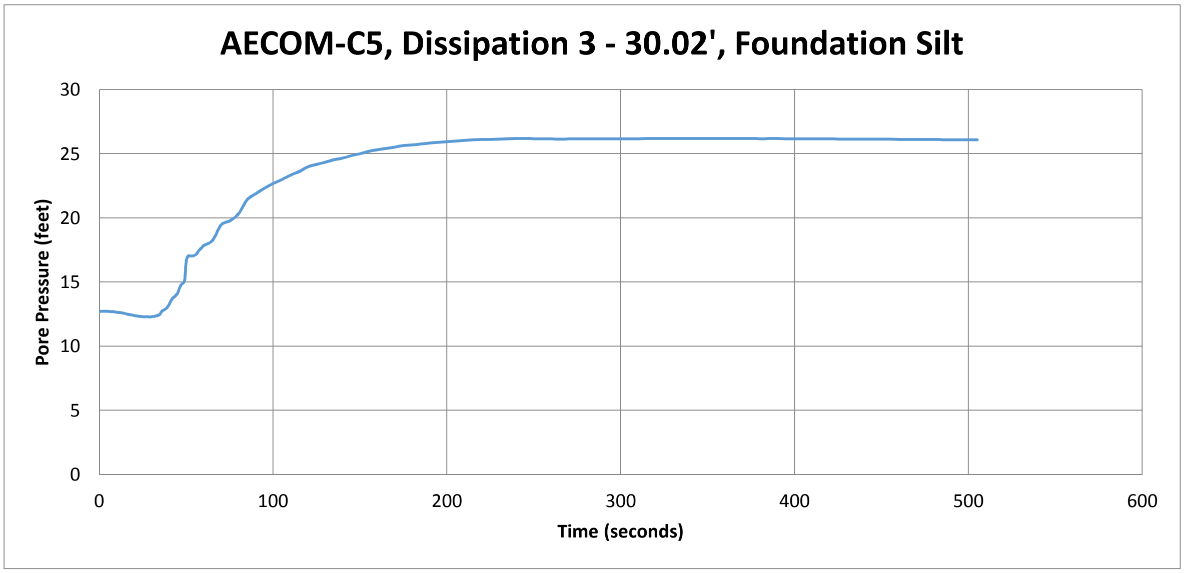


AECOM-C4, Dissipation 5 - 49.87', Foundation Clay



AECOM-C4, Dissipation 6 - 60.04', Foundation Silt


AECOM-C4, Dissipation 7 - 64.96', Foundation Silt
Pore Pressure (feet)
180
160
140
120
100
80
60
40
20
0
0
50
100
150
200
250
300
350
400
Time (seconds)


AECOM-C5, Dissipation 3 - 30.02', Foundation Silt
Pore Pressure (feet)
30
25
20
15
10
5
0
0
100
200
300
400
500
600
Time (seconds)

AECOM-C1

Depth (ft)	U_{peak} (ft)	t-U_{peak} (sec)	U_{eq} (ft)	$U_{average}$ (ft)	t-$U_{average}$ (sec)	t_{50} (sec)	Hydraulic Conductivity, k_h (cm/s)
29.1	0.9	0	17.2	9.0	100	100.0	3.17E-06
40.5	-9.9	0	26.2	8.2	26	26.0	1.70E-05

AECOM-C2

Depth (ft)	U_{peak} (ft)	t-U_{peak} (sec)	U_{eq} (ft)	$U_{average}$ (ft)	t-$U_{average}$ (sec)	t_{50} (sec)	Hydraulic Conductivity, k_h (cm/s)
30.2	4.3	75	30.3	17.3	123	48.0	7.92E-06
70.2	60.6	15	41.6	51.1	130	115.0	2.66E-06
85.3	215.7	5	63.0	139.3	1000	995.0	1.79E-07

AECOM-C3

Depth (ft)	U_{peak} (ft)	t-U_{peak} (sec)	U_{eq} (ft)	$U_{average}$ (ft)	t-$U_{average}$ (sec)	t_{50} (sec)	Hydraulic Conductivity, k_h (cm/s)
14.8	2.7	49	7.8	5.3	1170	1121.0	1.54E-07
17.2	26.5	4	9.9	18.2	22	18.0	2.70E-05

AECOM-C4

Depth (ft)	U_{peak} (ft)	t-U_{peak} (sec)	U_{eq} (ft)	$U_{average}$ (ft)	t-$U_{average}$ (sec)	t_{50} (sec)	Hydraulic Conductivity, k_h (cm/s)
49.9	180.1	1	44.2	112.2	570	569.0	3.60E-07
60.0	191.4	0	51.7	121.6	375	375.0	6.07E-07
65.0	164.4	0	55.8	110.1	14	14.0	3.70E-05

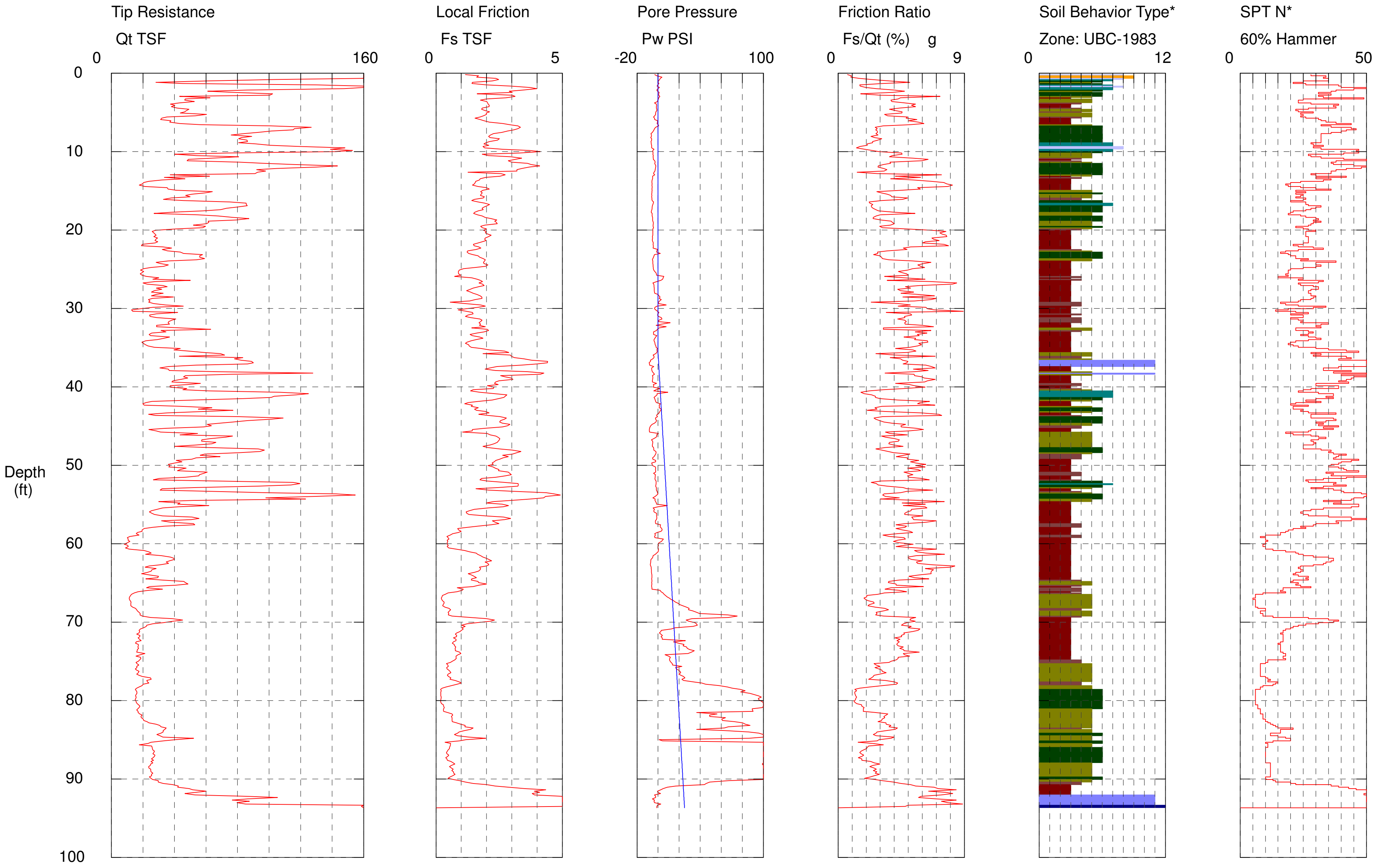
AECOM-C5

Depth (ft)	U_{peak} (ft)	t-U_{peak} (sec)	U_{eq} (ft)	$U_{average}$ (ft)	t-$U_{average}$ (sec)	t_{50} (sec)	Hydraulic Conductivity, k_h (cm/s)
24.9	15.9	0	20.5	18.2	19	19.0	2.52E-05
30.0	12.3	22	26.1	19.2	69	47.0	8.13E-06

CPT-1

Operator: Cardno ATC
Sounding: Elev: 450
Cone Used: DDG1181

CPT Date/Time: 10/2/2015 9:46:26 AM
Location: Vectren-AB Brown
Job Number: 170GC00108



Maximum Depth = 93.67 feet

Depth Increment = 0.164 feet

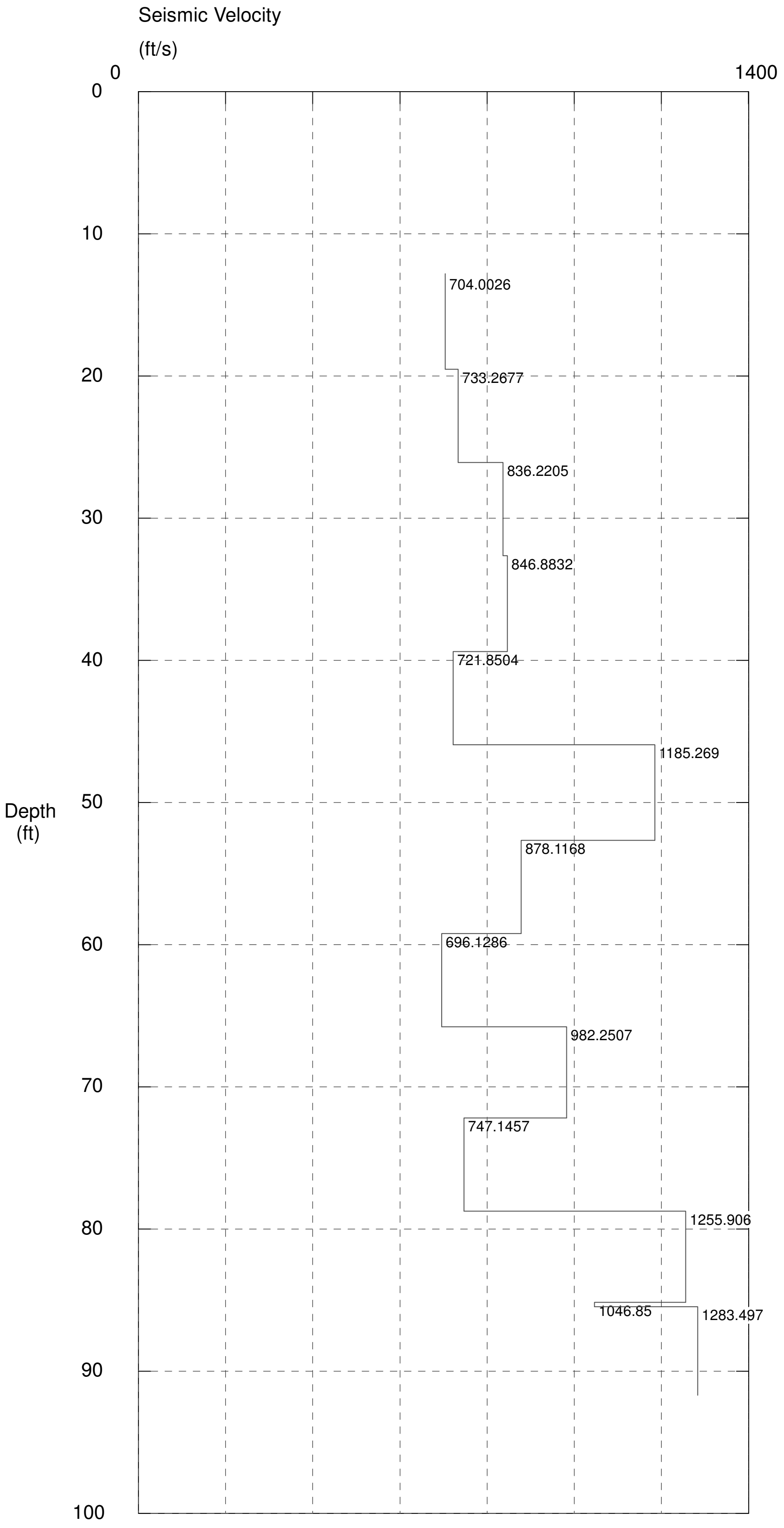
- ▨ 1 sensitive fine grained
- ▨ 2 organic material
- ▨ 3 clay
- ▨ 4 silty clay to clay
- ▨ 5 clayey silt to silty clay
- ▨ 6 sandy silt to clayey silt
- ▨ 7 silty sand to sandy silt
- ▨ 8 sand to silty sand
- ▨ 9 sand
- ▨ 10 gravelly sand to sand
- ▨ 11 very stiff fine grained (*)
- ▨ 12 sand to clayey sand (*)

*Soil behavior type and SPT based on data from UBC-1983

CPT-1

Operator: Cardno ATC
Sounding: Elev: 450
Cone Used: DDG1181

CPT Date/Time: 10/2/2015 9:46:26 AM
Location: Vectren-AB Brown
Job Number: 170GC00108



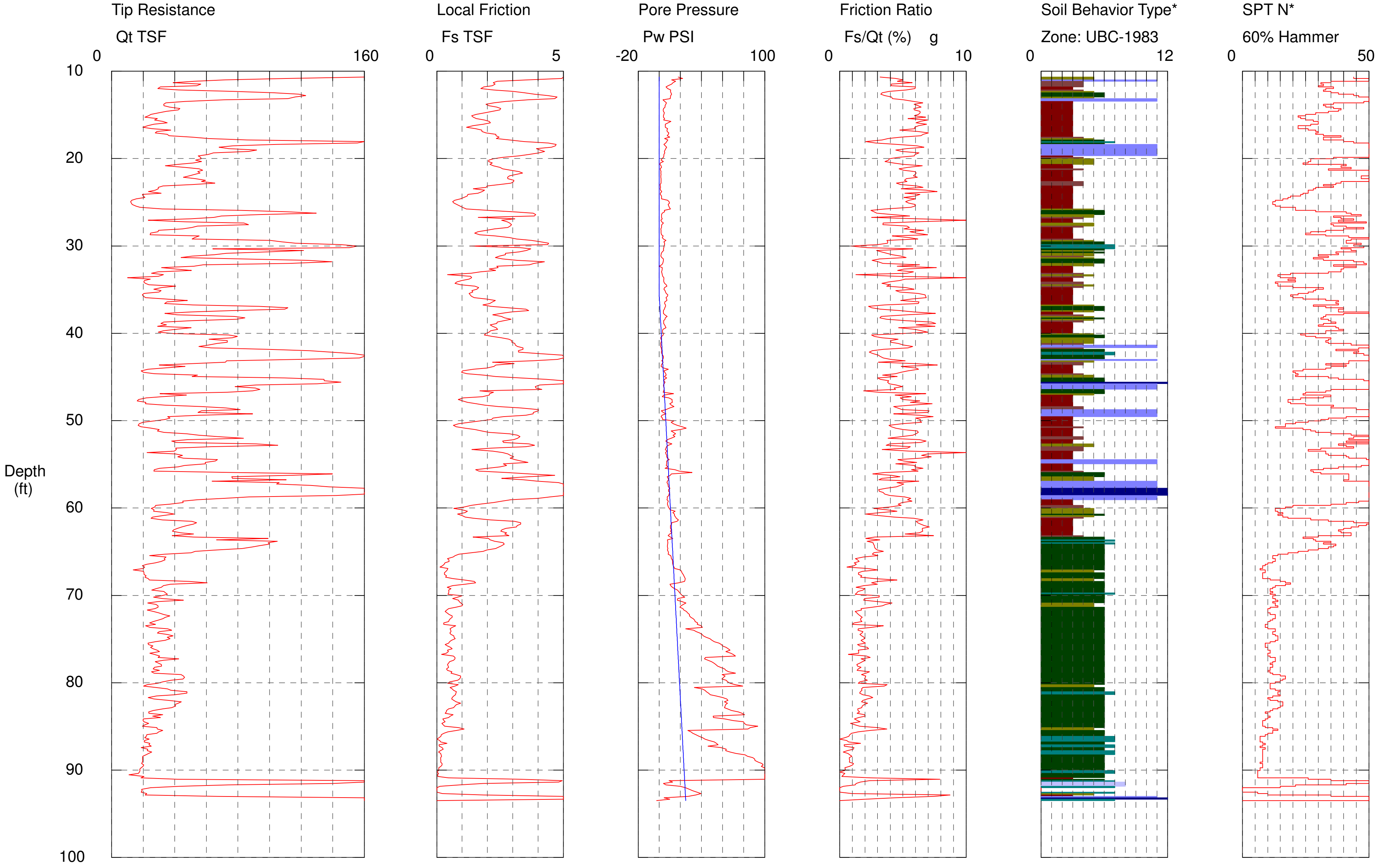
Maximum Depth = 93.67 feet

Depth Increment = 0.164 feet

CPT-2

Operator: Cardno ATC
Sounding: Elev.: 450
Cone Used: DDG1181

CPT Date/Time: 10/2/2015 1:48:58 PM
Location: Vectren-AB Brown
Job Number: 170GC00108



Maximum Depth = 93.50 feet

Depth Increment = 0.164 feet

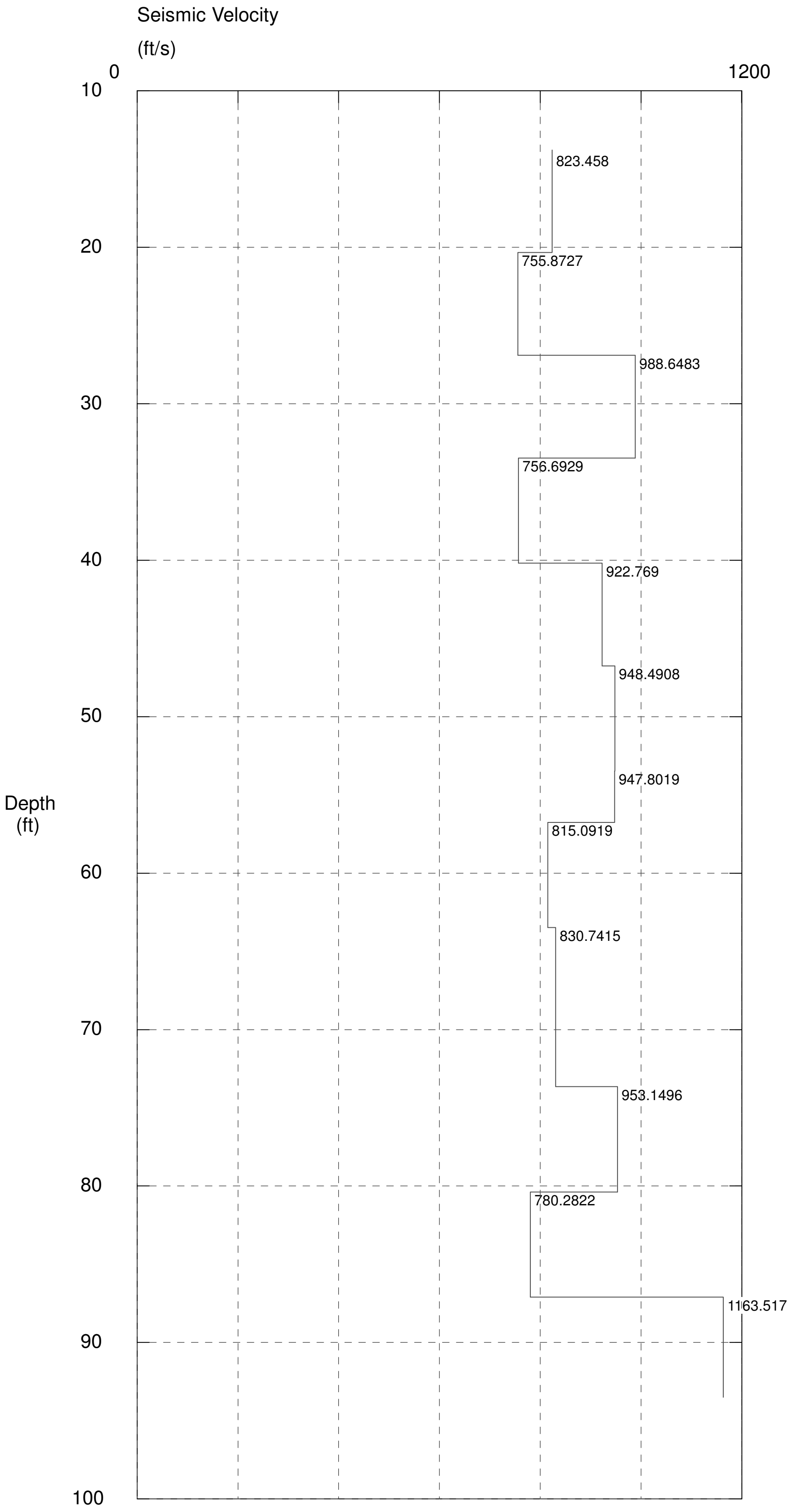
- 1 sensitive fine grained
- 2 organic material
- 3 clay
- 4 silty clay to clay
- 5 clayey silt to silty clay
- 6 sandy silt to clayey silt
- 7 silty sand to sandy silt
- 8 sand to silty sand
- 9 sand
- 10 gravelly sand to sand
- 11 very stiff fine grained (*)
- 12 sand to clayey sand (*)

*Soil behavior type and SPT based on data from UBC-1983

CPT-2

Operator: Cardno ATC
Sounding: Elev.: 450
Cone Used: DDG1181

CPT Date/Time: 10/2/2015 1:48:58 PM
Location: Vectren-AB Brown
Job Number: 170GC00108



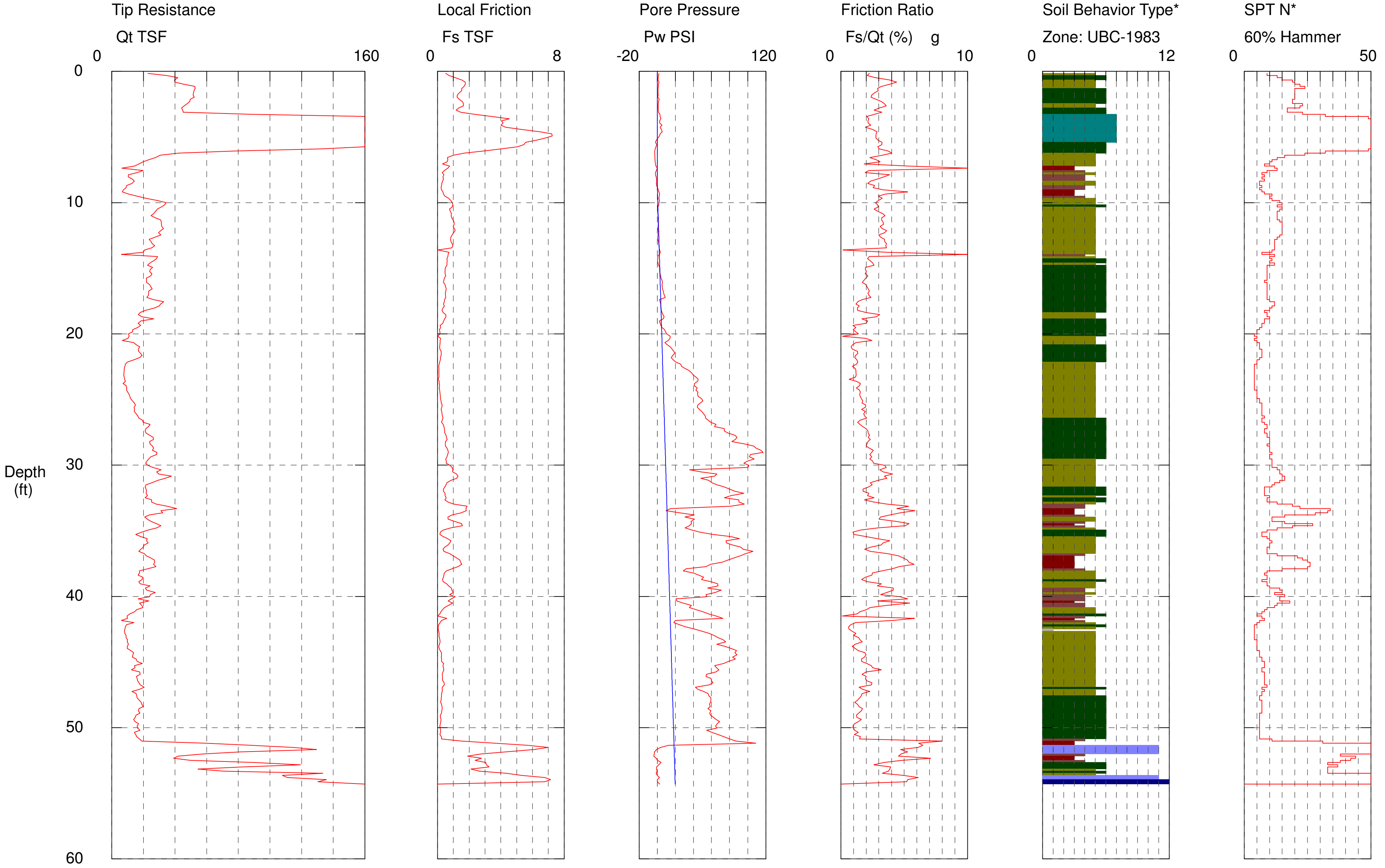
Maximum Depth = 93.50 feet

Depth Increment = 0.164 feet

CPT-3

Operator: Cardno ATC
Sounding: Elev: 415
Cone Used: DDG1181

CPT Date/Time: 10/1/2015 12:04:46 PM
Location: Vectren-AB Brown
Job Number: 170GC00108



Maximum Depth = 54.30 feet

Depth Increment = 0.164 feet

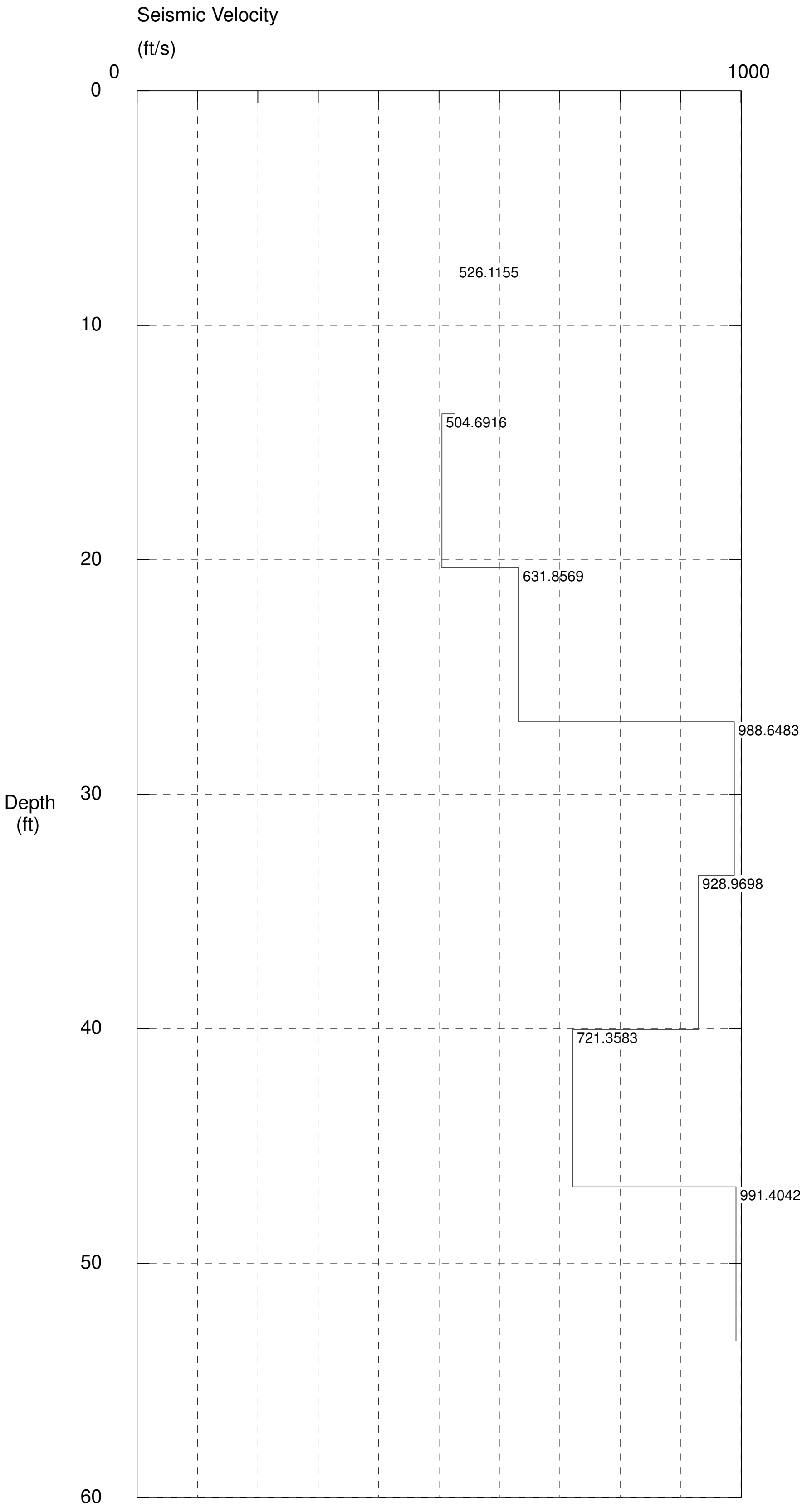
■	1	sensitive fine grained
■	2	organic material
■	3	clay
■	4	silty clay to clay
■	5	clayey silt to silty clay
■	6	sandy silt to clayey silt
■	7	silty sand to sandy silt
■	8	sand to silty sand
■	9	sand
■	10	gravelly sand to sand
■	11	very stiff fine grained (*)
■	12	sand to clayey sand (*)

*Soil behavior type and SPT based on data from UBC-1983

CPT-3

Operator: Cardno ATC
Sounding: Elev: 415
Cone Used: DDG1181

CPT Date/Time: 10/1/2015 12:04:46 PM
Location: Vectren-AB Brown
Job Number: 170GC00108



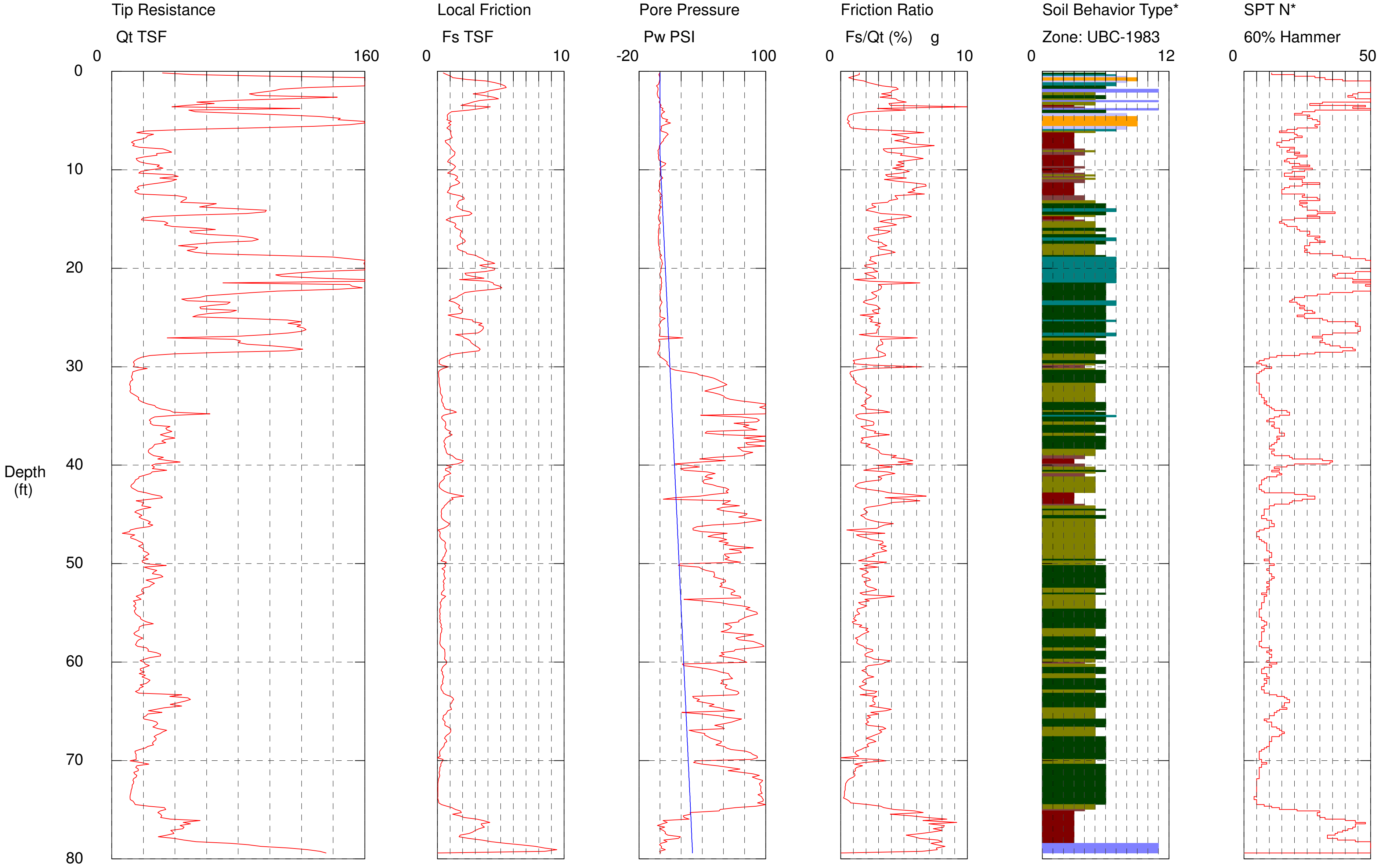
Maximum Depth = 54.30 feet

Depth Increment = 0.164 feet

CPT-4

Operator: Cardno ATC
Sounding: Elev.: 415
Cone Used: DDG1181

CPT Date/Time: 10/1/2015 2:34:16 PM
Location: Vectren-AB Brown
Job Number: 170GC00108



Maximum Depth = 79.40 feet

Depth Increment = 0.164 feet

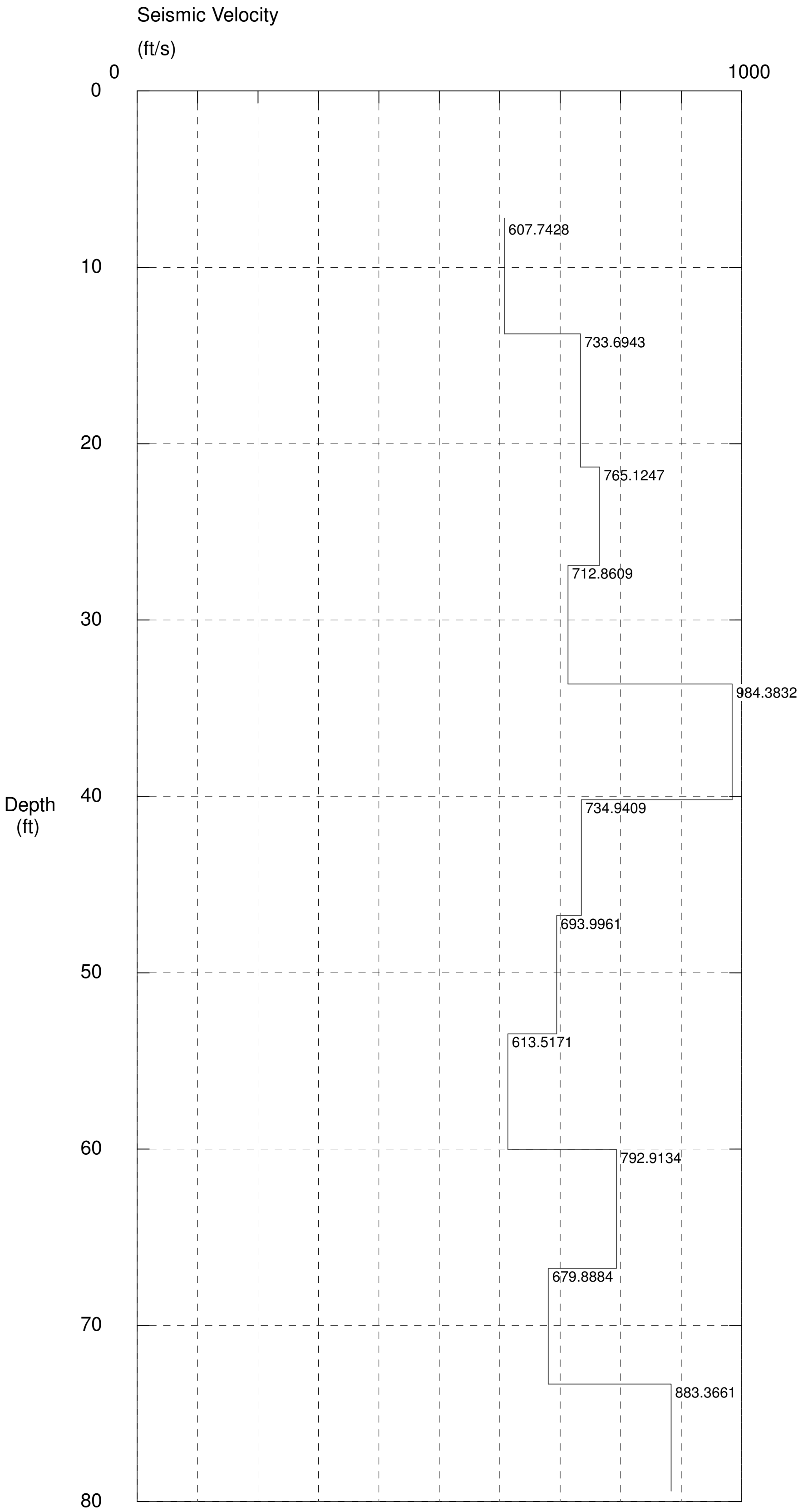
- ⬜ 1 sensitive fine grained
- ⬛ 2 organic material
- 🟥 3 clay
- 🟫 4 silty clay to clay
- 🟫 5 clayey silt to silty clay
- 🟩 6 sandy silt to clayey silt
- 🟦 7 silty sand to sandy silt
- 🟪 8 sand to silty sand
- 🟧 9 sand
- 🟥 10 gravelly sand to sand
- 🟦 11 very stiff fine grained (*)
- 🟦 12 sand to clayey sand (*)

*Soil behavior type and SPT based on data from UBC-1983

CPT-4

Operator: Cardno ATC
Sounding: Elev.: 415
Cone Used: DDG1181

CPT Date/Time: 10/1/2015 2:34:16 PM
Location: Vectren-AB Brown
Job Number: 170GC00108



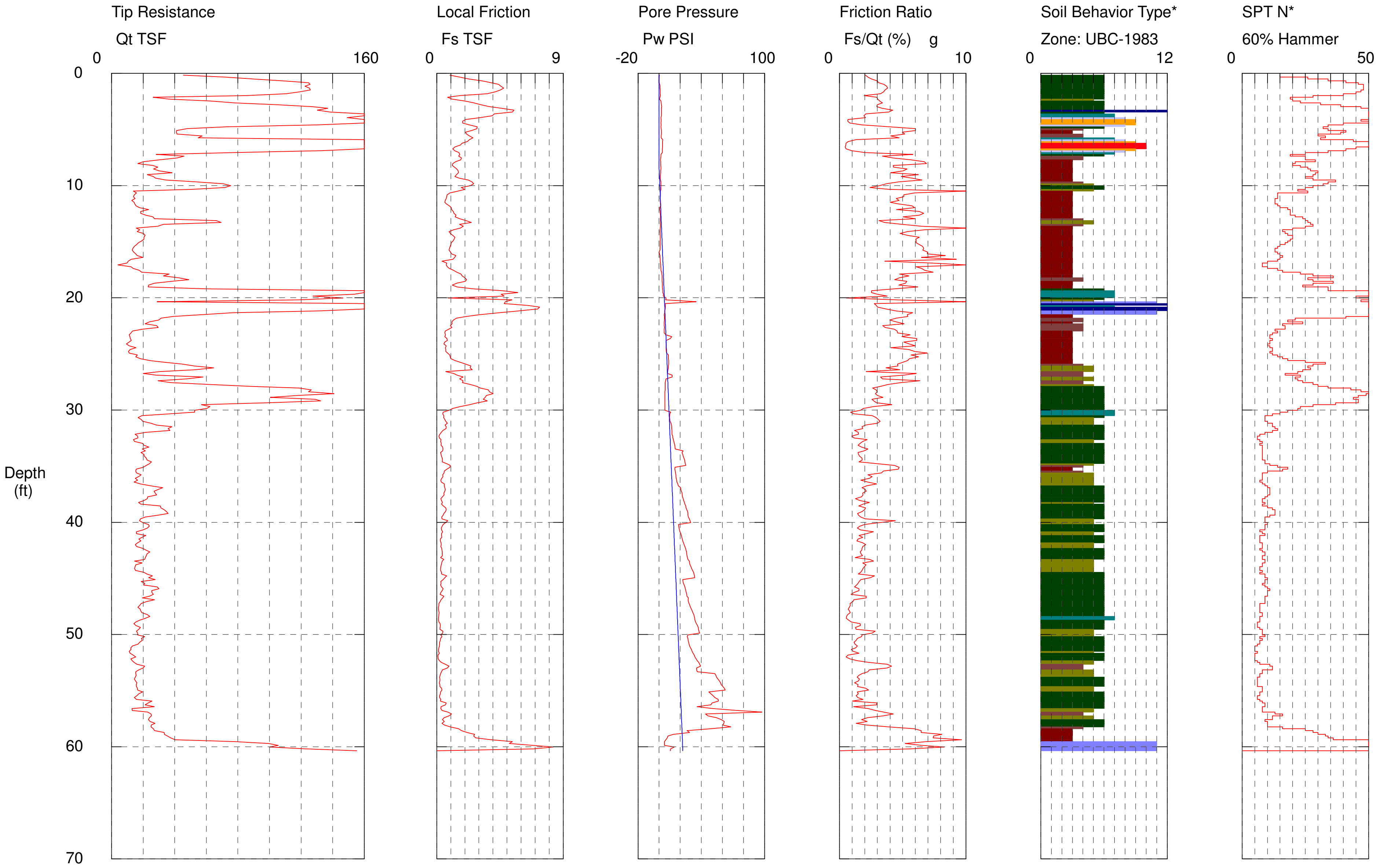
Maximum Depth = 79.40 feet

Depth Increment = 0.164 feet

CPT-5

Operator: Cardno ATC
Sounding: Elev: 415
Cone Used: DDG1181

CPT Date/Time: 10/1/2015 6:00:58 PM
Location: Vectren-AB Brown
Job Number: 170GC00108



Maximum Depth = 60.37 feet

Depth Increment = 0.164 feet

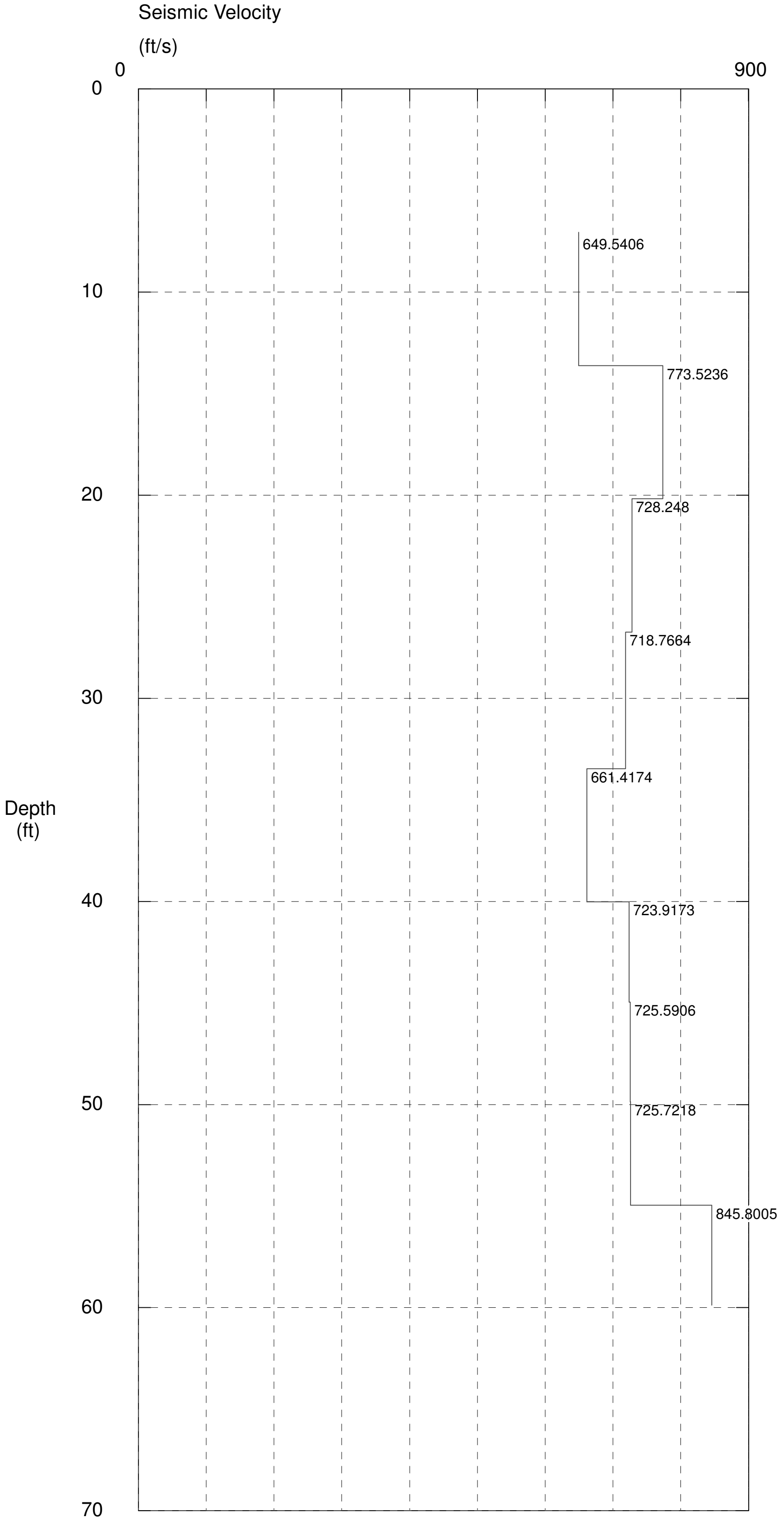
■ 1 sensitive fine grained
■ 2 organic material
■ 3 clay
■ 4 silty clay to clay
■ 5 clayey silt to silty clay
■ 6 sandy silt to clayey silt
■ 7 silty sand to sandy silt
■ 8 sand to silty sand
■ 9 sand
■ 10 gravelly sand to sand
■ 11 very stiff fine grained (*)
■ 12 sand to clayey sand (*)

*Soil behavior type and SPT based on data from UBC-1983

CPT-5

Operator: Cardno ATC
Sounding: Elev: 415
Cone Used: DDG1181

CPT Date/Time: 10/1/2015 6:00:58 PM
Location: Vectren-AB Brown
Job Number: 170GC00108



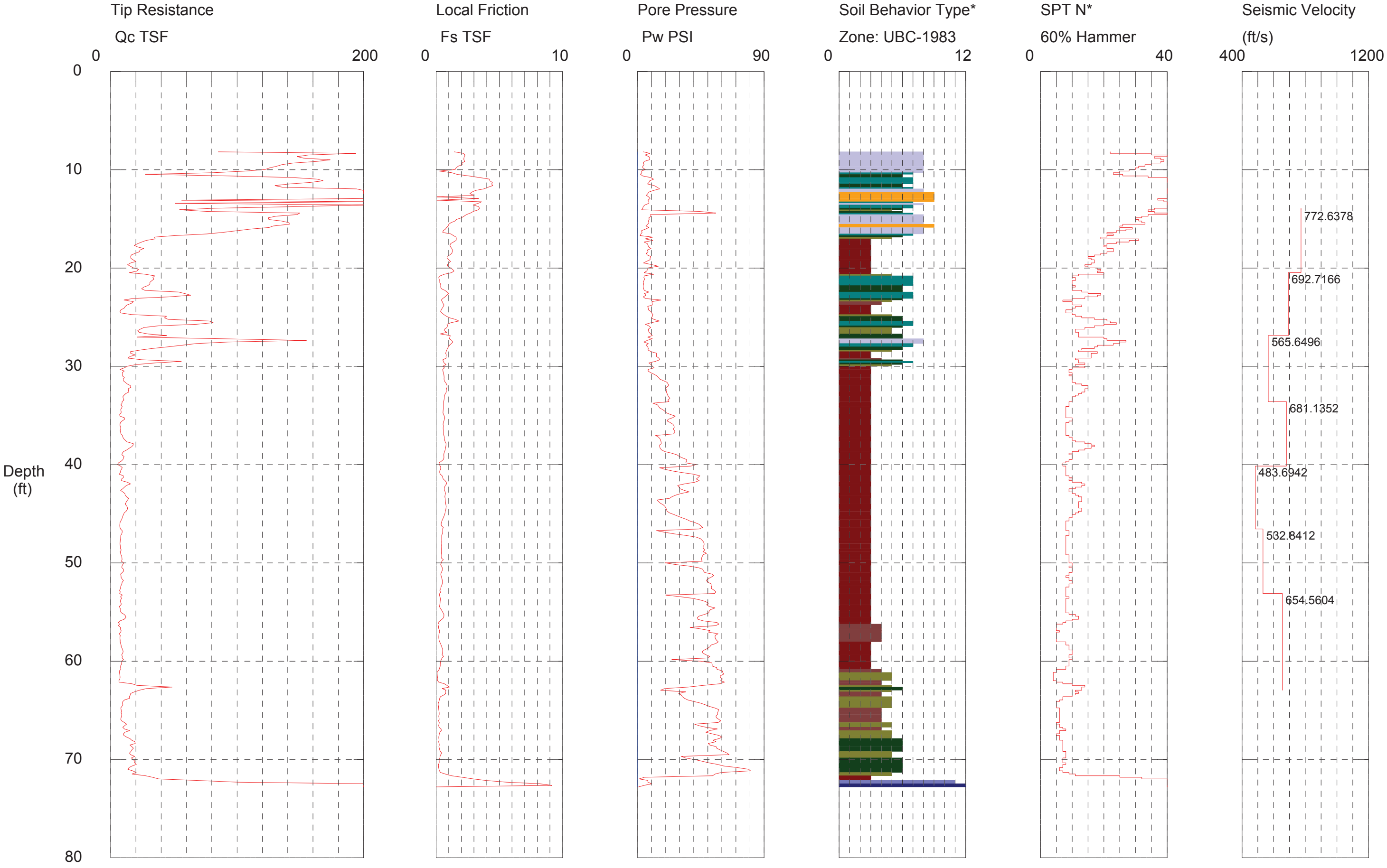
Maximum Depth = 60.37 feet

Depth Increment = 0.164 feet

CPT-101

Operator: Cardno - ZV
Sounding: Elev: 463.5
Cone Used: DDG1181

CPT Date/Time: 4/16/2015 8:48:59 AM
Location: North=968144, East=2772356
Job Number: 170GC00108



Maximum Depth = 72.80 feet

Depth Increment = 0.164 feet

■ 1 sensitive fine grained
■ 2 organic material
■ 3 clay

■ 4 silty clay to clay
■ 5 clayey silt to silty clay
■ 6 sandy silt to clayey silt

■ 7 silty sand to sandy silt
■ 8 sand to silty sand
■ 9 sand

■ 10 gravelly sand to sand
■ 11 very stiff fine grained (*)
■ 12 sand to clayey sand (*)

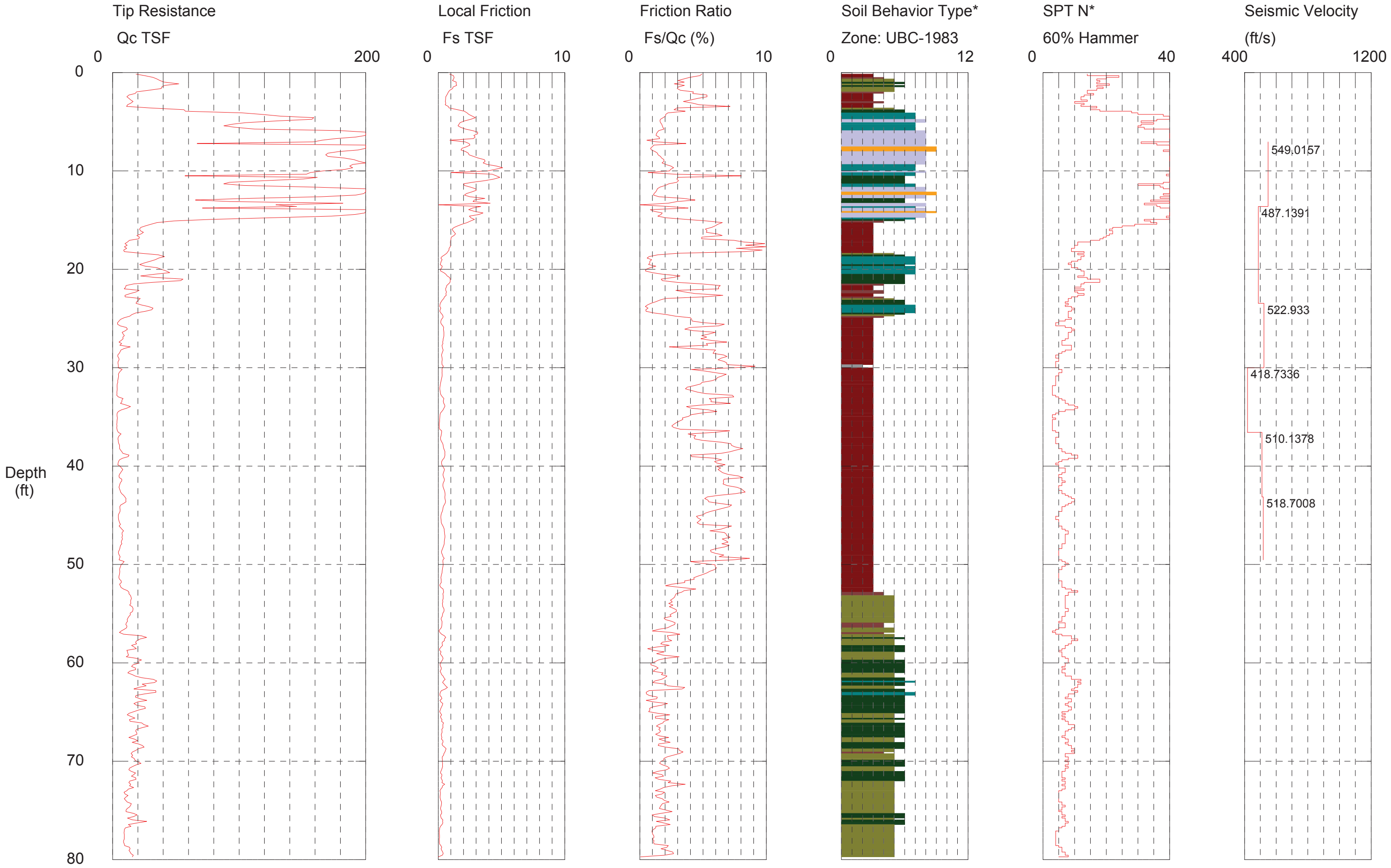
AB-Brown

*Soil behavior type and SPT based on data from UBC-1983

CPT-102

Operator: Cardno - ZV
Sounding: Elev: 463.7
Cone Used: DDG1181

CPT Date/Time: 4/15/2015 1:18:25 PM
Location: North=968474, East=2772217
Job Number: 170GC00108



Maximum Depth = 79.72 feet

Depth Increment = 0.164 feet

■ 1 sensitive fine grained	■ 4 silty clay to clay	■ 7 silty sand to sandy silt	🟥 10 gravelly sand to sand
■ 2 organic material	■ 5 clayey silt to silty clay	■ 8 sand to silty sand	■ 11 very stiff fine grained (*)
■ 3 clay	■ 6 sandy silt to clayey silt	■ 9 sand	■ 12 sand to clayey sand (*)

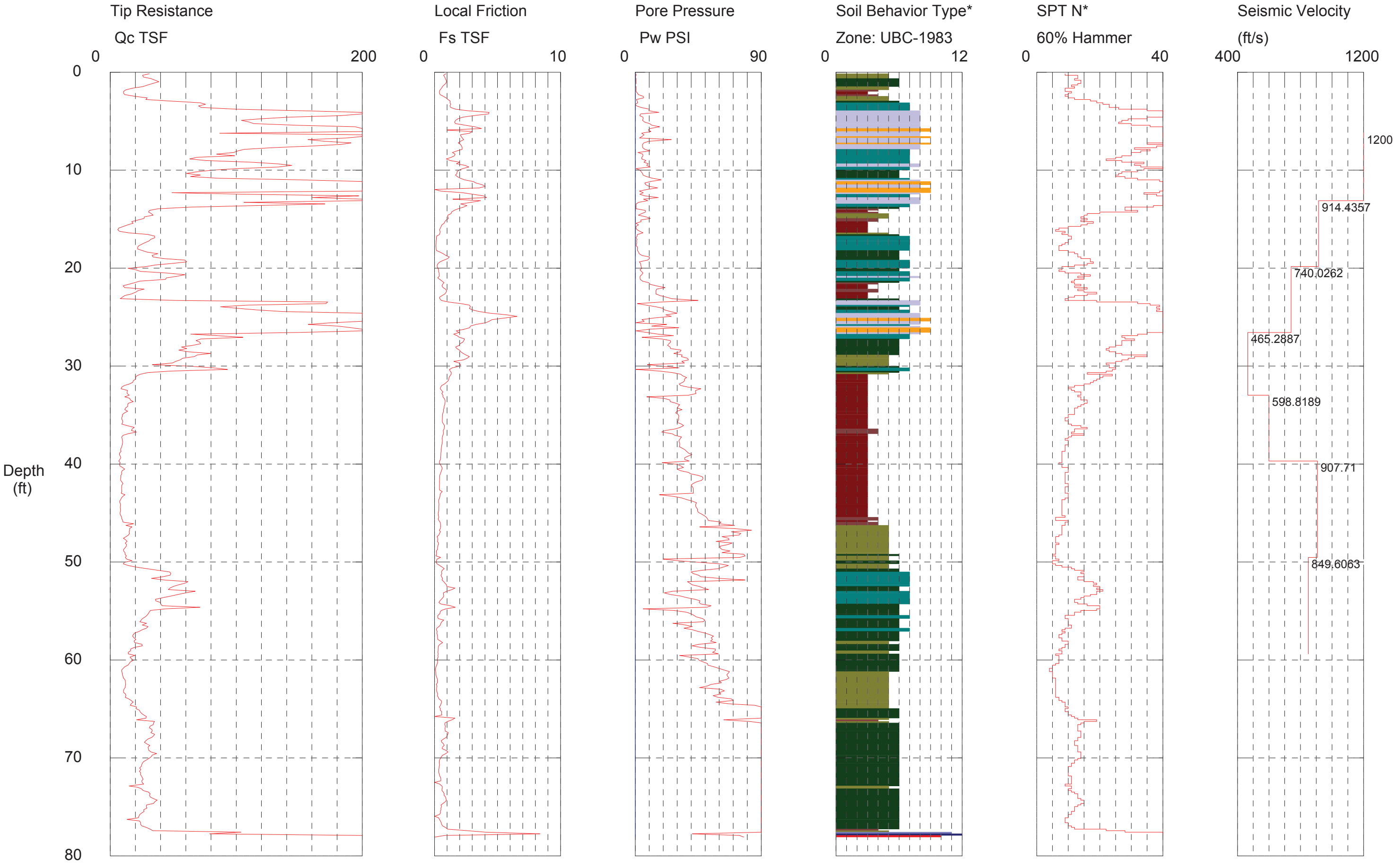
AB Brown

*Soil behavior type and SPT based on data from UBC-1983

CPT-103

Operator: Cardno - ZV
Sounding: Elev: 463.8
Cone Used: DDG1181

CPT Date/Time: 4/15/2015 11:49:19 AM
Location: North=968605, East=2772159
Job Number: 170GC00108



Maximum Depth = 78.08 feet

Depth Increment = 0.164 feet

- ⬜ 1 sensitive fine grained
- ⬛ 2 organic material
- 🟥 3 clay
- 🟫 4 silty clay to clay
- 🟩 5 clayey silt to silty clay
- 🟩 6 sandy silt to clayey silt
- 🟦 7 silty sand to sandy silt
- 🟪 8 sand to silty sand
- 🟧 9 sand
- 🟥 10 gravelly sand to sand
- 🟪 11 very stiff fine grained (*)
- 🟦 12 sand to clayey sand (*)

AB Brown

*Soil behavior type and SPT based on data from UBC-1983

FLOW PROPERTIES from PIEZOCONE DISSIPATION TESTS

Soils exhibit flow properties that control ***hydraulic conductivity*** (k), rates of consolidation, construction behavior, and drainage characteristics in the ground. Field measurements for soil permeability include pumping tests with measured drawdown, slug tests, and packer methods. Laboratory methods include falling head and constant head types in permeameters, controlled gradient, and constant rate of strain consolidation (Leroueil, et al., *Geotechnique*, June 1992). An indirect assessment of permeability can be made from consolidation test data. Results of pressure dissipation readings from piezocone and flat dilatometer and holding tests during pressuremeter testing can be used to determine permeability and the coefficient of consolidation (Jamiolkowski, et al. 1985, *Proc. 11th ICSMFE*, San Francisco, Vol. 1). Herein, only the piezocone approach will be discussed.

The ***permeability*** (k) can be determined from the dissipation test data, either by use of the direct correlative relationship presented earlier, or alternatively by the evaluation of the ***coefficient of consolidation***, c_h. Assuming radial flow, the horizontal permeability (k_h) is obtained from:

$$k_h = \frac{c_h \gamma_w}{D'}$$

where D′ = constrained modulus obtained from oedometer tests. Note: results of high-quality lab testing of natural clays show $k_h \approx 1.1\ k_v$ unless the deposit is highly stratified or consists of varved materials (Tavenas, et al., Nov. 1983, *Canadian Geot. Journal*).

Piezocone Dissipation Tests

In a CPTu test performed in saturated clays and silts, large excess porewater pressures (Δu) are generated during penetration of the piezocone. Soft to firm intact clays will exhibit measured penetration porewater pressures which are 3 to 6 times greater than the hydrostatic water pressure, while values of 10 to 20 times greater than the hydrostatic water pressure will typically be measured in stiff to hard intact clays. In fissured materials, zero or negative porewater pressures will be recorded. Regardless, once penetration is stopped, these excess pressures will decay with time and eventually reach equilibrium conditions which correspond to hydrostatic values. In essence, this is analogous to a push-in type piezometer. In addition to piezometers and piezocones, excess pressures occur during the driving of pile foundations, installation of displacement devices such as vibroflots for stone columns and mandrels for vertical wick-drains, as well as insertion of other in-situ tests including dilatometer, full-displacement pressuremeter, and field vane.

How quickly the porewater pressures decay depends on the permeability of the surrounding medium (k), as well as the horizontal coefficient of consolidation (c_h). In clean sands and gravels that are pervious, essentially drained response is observed at the time of penetration and the measured porewater pressures are hydrostatic. In most other cases, an initial undrained response occurs that is followed by drainage. For example, in silty sands, generated excess pressures can dissipate in 1 to 2 minutes, while in contrast, fat plastic clays may require 2 to 3 days for complete equalization.

Representative dissipation curves from two types of piezocone elements (midface u_1 and shoulder u_2) are presented in Figure F-1. These data were recorded at a depth of 15.2 meters in a deposit of soft varved silty clay at the National Geotechnical Experimentation Site (NGES) in Amherst, MA. Full equalization tohydrostatic conditions is reached in about 1 hour (3600 s). In routine testing, data are recorded to just 50 percent consolidation in order to maintain productivity. In this case, the initial penetration pressures correspond to 0 percent decay and a calculated hydrostatic value (u_0) based on groundwater levels represents the 100 percent completion. Figure F-1 illustrates the procedure to obtain the time to 50% completion (t_{50}).

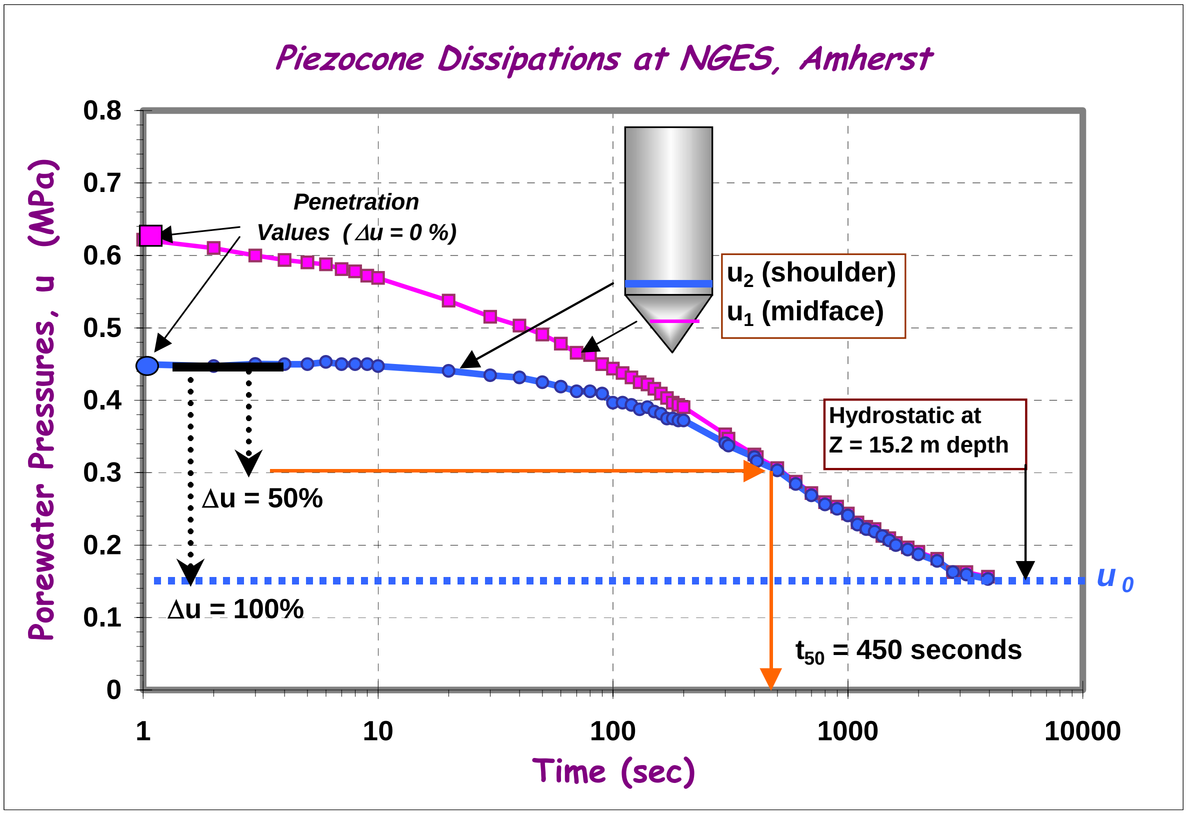


Figure F-1. Porewater Pressure Dissipation Response in Soft Varved Clay at Amherst NGES. (Procedure for t_{50} determination using U_2 readings shown)

The aforementioned approach applies to soils that exhibit monotonic decay of porewater pressures with logarithm of time. For cases involving heavily overconsolidated and fissured geomaterials, a dilatory response can occur whereby the porewater pressures initially rise with time, reach a peak value, and then subsequently decrease with time.

For type 2 piezocones with shoulder filter elements, the t_{50} reading from monotonic responses can be used to evaluate the permeability according to the chart provided in Figure F-2. The average relationship may be approximately expressed by:

$$k \quad (\text{cm/s}) \quad \approx \quad 1/(251 \cdot t_{50})^{1.25}$$

where t_{50} is given in seconds. The interpretation of the coefficient of consolidation from dissipation data is discussed subsequently and includes both monotonic and dilatory porewater pressure behavior.

Monotonic Dissipation

For *monotonic* porewater decays where the readings always decrease with time, these responses are generally are associated with soft to firm clays and silts. For these cases, the strain path method (Teh & Houlsby, 1991, *Geotechnique*) may be used to determine c_h from the expression:

$$c_h = \frac{T * a^2 \sqrt{I}_R}{t_{50}}$$

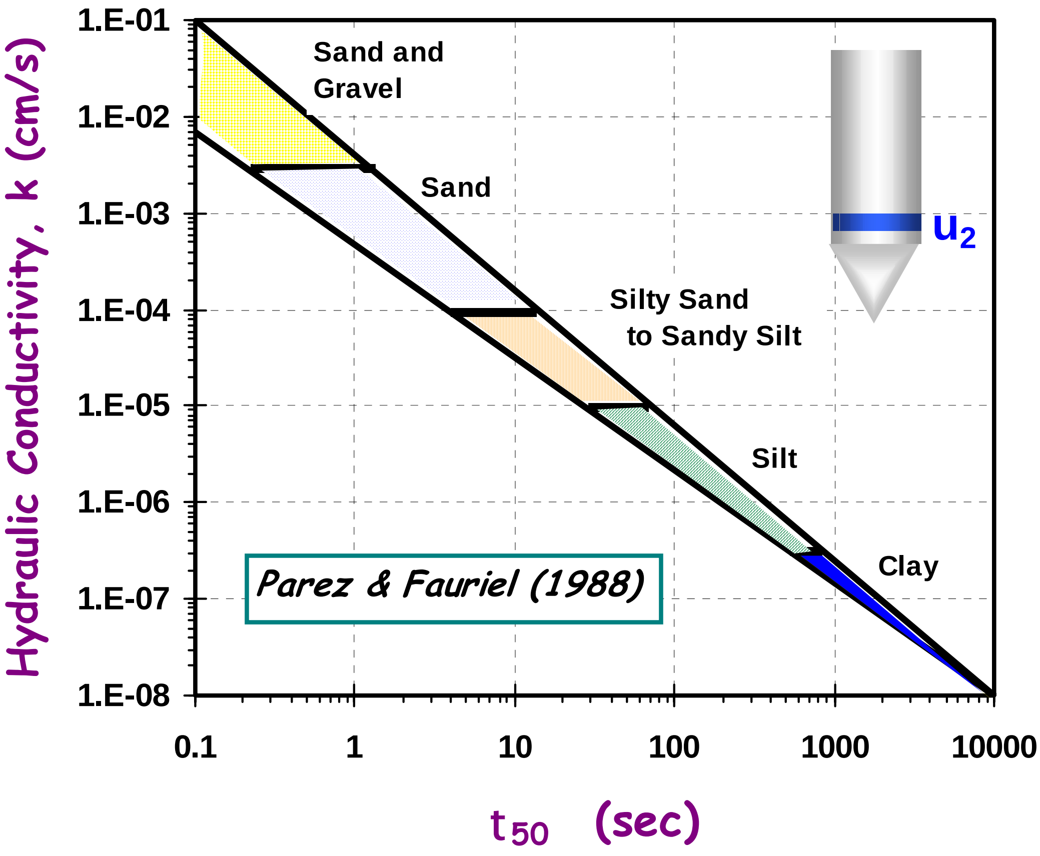


Figure F-2: Coefficient of Permeability (k = Hydraulic Conductivity) from Measured Time to 50% Consolidation (t_{50}) for Monotonic Type 2 Dissipations (from Parez & Fauriel, 1988).

where T* = modified time factor from consolidation theory, a = probe radius, $I_R = G/s_u$ = rigidity index of the soil, and t = measured time on the dissipation record (usually taken at 50% equalization). Several solutions have been presented for the modified time factor T* based on different theories, including cavity expansion, strain path, and dislocation points (Burns & Mayne, 1998, *Can. Geot. J.*). For monotonic dissipation response, the strain path solutions (Teh & Houlsby, 1991, *Geot.*) are presented in Figures F-3 and F-4 for both midface and shoulder type elements, respectively.

The determination of t_{50} from shoulder porewater decays is illustrated by example in Figure F-1. These strain path solutions can be approximately described by the following:

$$\frac{\Delta u}{\Delta u_{initial}} = \left(\frac{1}{1.12 + 30 \cdot T^*}\right)^{0.48}$$

$$\frac{\Delta u_2}{\Delta u_{2-INITIAL}} = \left(\frac{1}{1 + 10 \cdot T^*}\right)^{0.64}$$

For the particular case of 50% consolidation, the respective time factors are T* = 0.118 for the type 1 (midface element) and T* = 0.245 for the type 2 (shoulder element).

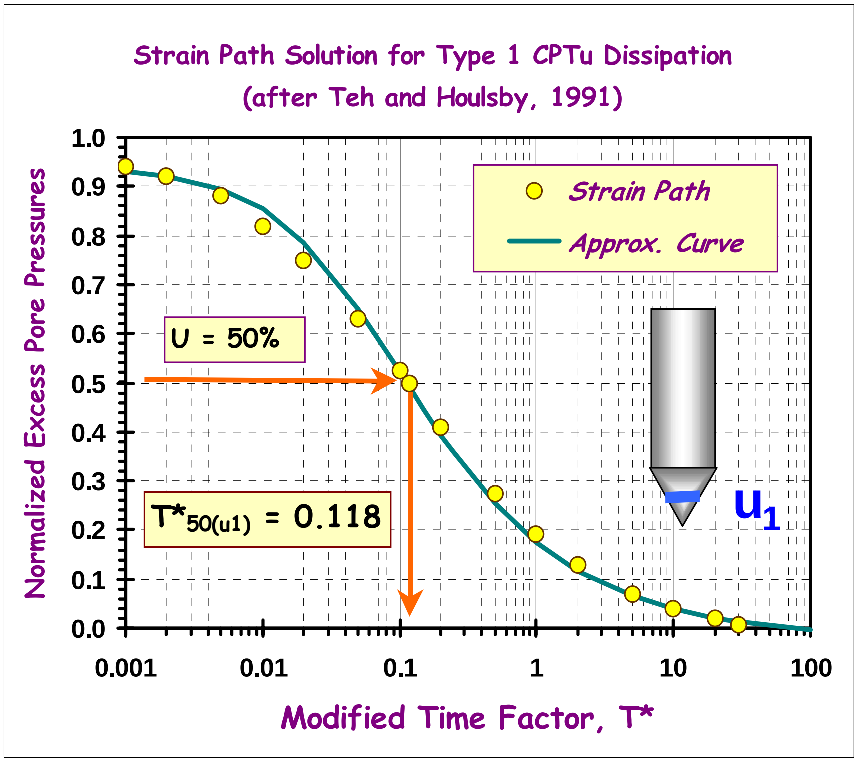


Figure F-3. Modified Time Factors for u_1 Monotonic Porewater Dissipations

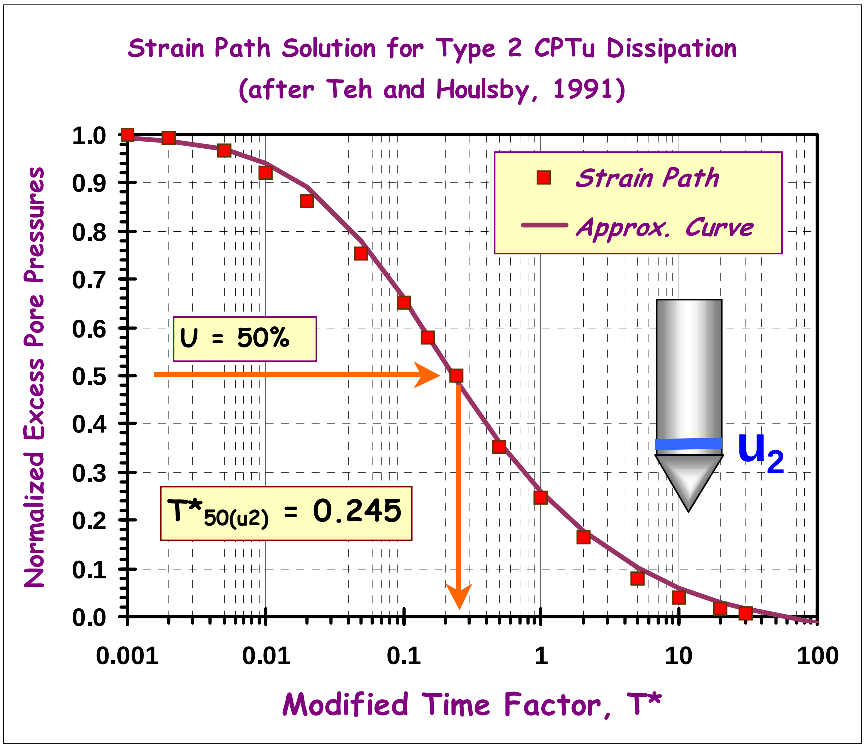


Figure F-4. Modified Time Factors for u_2 Monotonic Porewater Dissipations

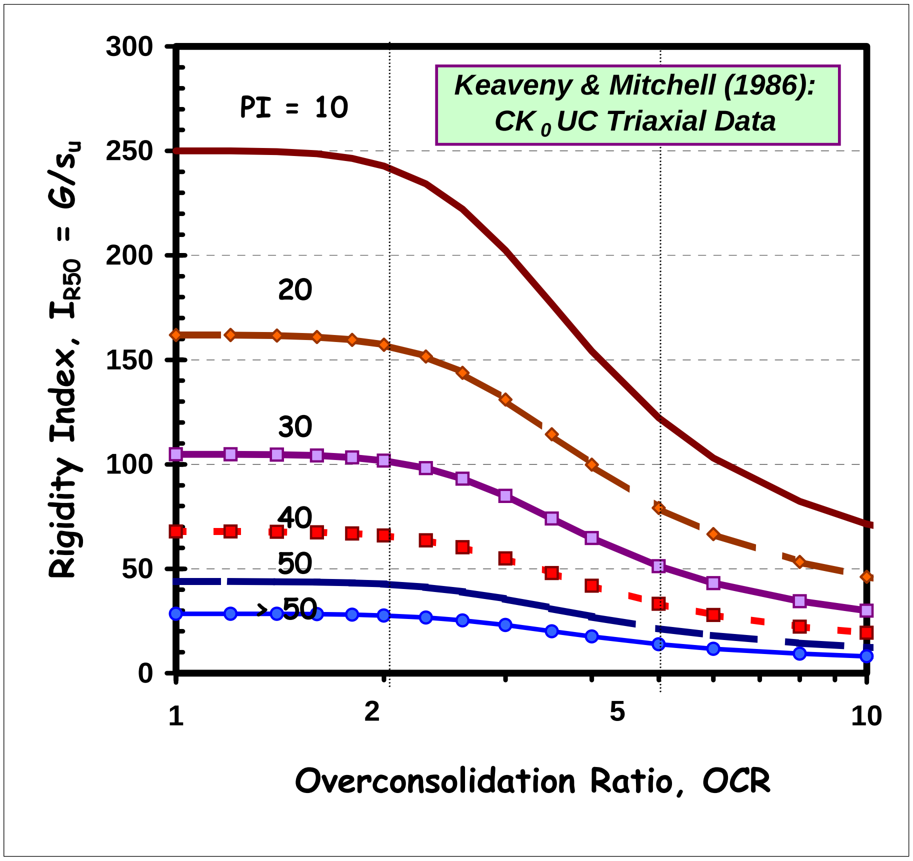


Figure F-5. Estimation of Undrained Rigidity Index of Clays and Silts from OCR and Plasticity Index (Keaveny & Mitchell, 1986).

For clays, the undrained rigidity index (I_R) is the ratio of shear modulus (G) to shear strength (s_u) and may be obtained from a number of different means including: (a) measured triaxial stress-strain curve, (b) measured pressuremeter tests, and (c) empirical correlation. One correlation based on anisotropically-consolidated triaxial compression test data expresses I_R in terms of OCR and plasticity index (PI), as shown in Figure F-5. For spreadsheet use, the empirical trend may be approximated by:

$$I_R \approx \frac{\exp\left[\frac{137 - PI}{23}\right]}{\left[1 + \ln\left\{1 + \frac{(OCR - 1)^{3.2}}{26}\right\}\right]^{0.8}}$$

Additional approaches to estimating the value of I_R are reviewed elsewhere (Mayne, *Proc. In-Situ 2001*, Bali). To facilitate the interpretation of c_h corresponding to t_{50} readings using the standard penetrometer, Figure F-6 presents a graphical plot for various I_R values.

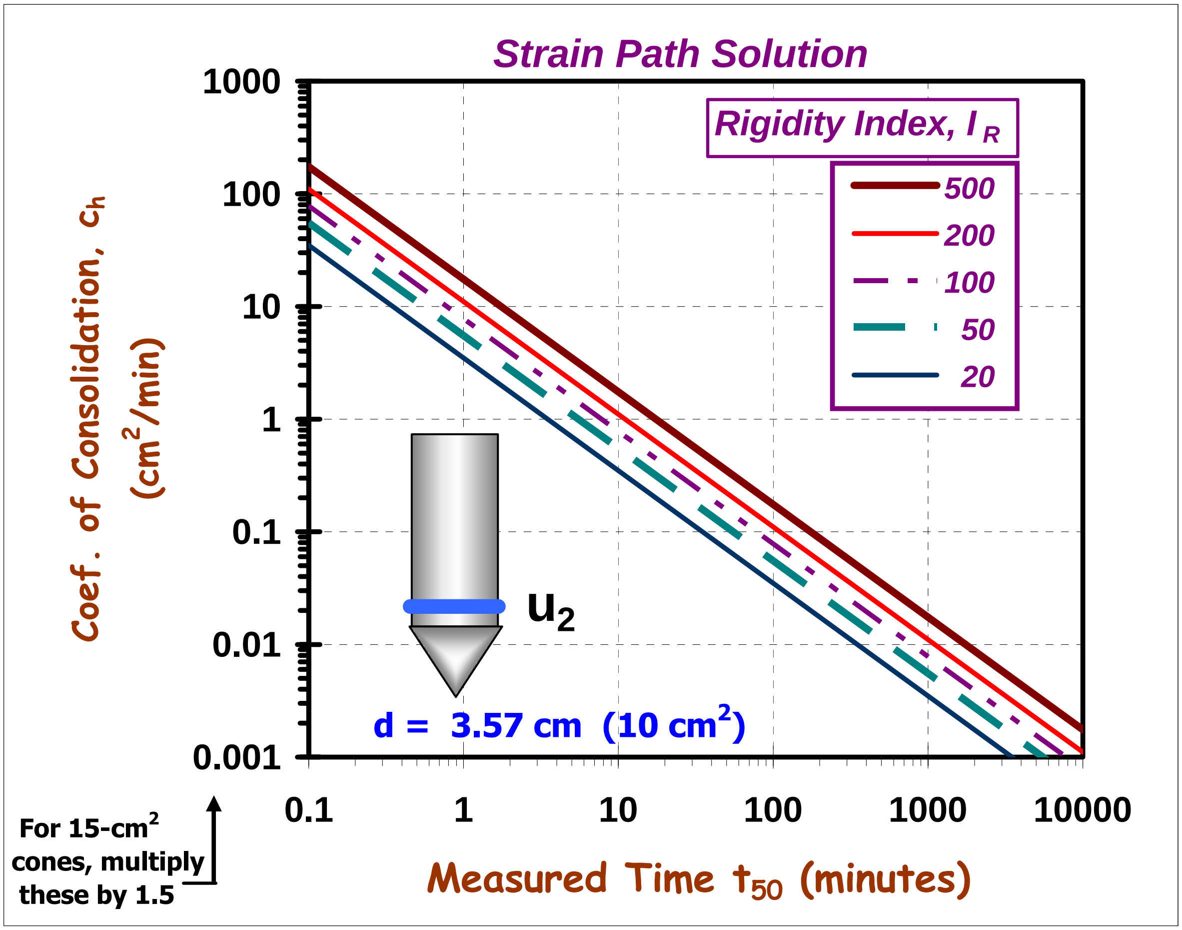


Figure F-6. Coefficient of consolidation at 50% dissipation for shoulder elements.

Dilatory Dissipations

In many overconsolidated and fissured materials, a dissipation test may first show an increase in Δu with time, reaching a peak value, and subsequent decrease in Δu with time (e.g., Lunne, et al. 1997). This type of response is termed *dilatory* dissipation, referring to both the delay in time and cause of the phenomenon (dilation). The dilatory response has been observed during type 2 piezocone tests as well as during installation of driven piles in fine-grained soils. The definition of 50% completion is not clear and thus the previous approach is not applicable.

A rigorous mathematics derivation has been presented elsewhere that provides a cavity expansion-critical state solution to both monotonic and dilatory porewater decay with time (Burns & Mayne, 1998). For practical use, an approximate closed-form expression is presented here. In lieu of merely matching one point on the dissipation curve (i.e, t_{50}), the entire curve is matched to provide the best overall value of c_h. The excess porewater pressures Δu_t at any time t can be compared with the initial values during penetration (Δu_i).

The measured initial excess porewater pressure ($\Delta u_i = u_2 - u_o$) is given by:

$$\Delta u_i \quad = \quad (\Delta u_{oct})_i \quad + \quad (\Delta u_{shear})_i$$

where $(\Delta u_{oct})_i \;=\; \sigma_{vo}'(2M/3)(OCR/2)^{\Lambda} \ln(I_R)$ = the octahedral component during penetration;

and $(\Delta u_{shear})_i \;=\; \sigma_{vo}'[1 - (OCR/2)^{\Lambda}\,]$ is the shear-induced component during penetration.

The porewater pressures at **<u>any</u>** time (t) are obtained in terms of the modified time factor T* from:

$$\Delta u_t \quad = \quad (\Delta u_{oct})_i\,[1 + 50\,T']^{-1} \quad + \quad (\Delta u_{shear})_i\,[1 + 5000\,T']^{-1}$$

where a different modified time factor is defined by: $T' = (c_h\, t)/(a^2\, I_R^{\,0.75})$. On a spreadsheet, a column of assumed (logarithmic) values of T' are used to generate the corresponding time (t) for a given rigidity index (I_R) and probe radius (a). Then, trial & error can be used to obtain the best fit c_h for the measured dissipation data. Series of dissipation curves can be developed for a given set of soil properties. One example set of curves is presented in Figure F-7 for various OCRs and the following parameters: $\Lambda = 0.8$, $I_R = 50$, and $\phi' = 25^{\circ}$, in order to obtain the more conventional time factor, $T = (c_h\, t)/a^2$.

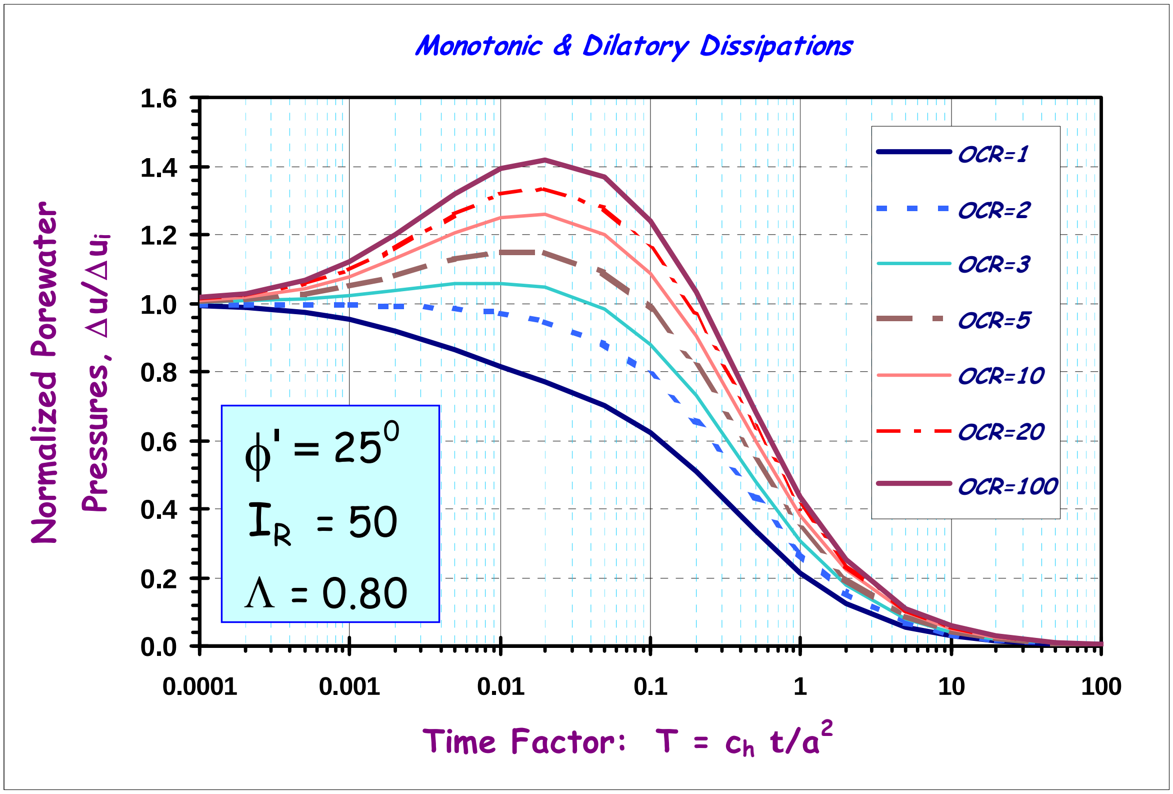


Figure F-7. Representative Solutions for Type 2 Dilatory Dissipation Curves at Various OCRs (after Burns & Mayne, 1998, *Canadian Geotechnical Journal*).

Appendix D
Lab Test Data

Summary of Laboratory Test Results – Embankment Fill

Boring and Sample ID	Ground Surface Elevation	Material Description	Sample Depth	Moisture Content	Dry Unit Weight	Total Unit Weight	Atterberg Limits			Gradations			USCS	Hydraulic Conductivity
							Liquid Limit	Plastic Limit	Plasticity Index	Sieve Analysis (3 inch to #200 Sieve)				
										Gravel	Sand	Fines		
	(ft)		(ft)	(%)	(pcf)	(pcf)	(%)	(%)	(%)	(%)	(%)	(%)		(cm/sec)
AECOM-B1, 1	451.3	Embankment Fill	17.0-19.0	18.0	106.4	125.6	-	-	-	-	-	-	CL	6.2E-07
AECOM-B1, 1A	451.3	Embankment Fill	19.0-21.0	17.0	112	131.0	33	17	16	0.7	3.9	95.4	CL	
AECOM-B1, 2	451.3	Embankment Fill	27.0-29.0	16.3	111	129.1	29	25	4	0.0	2.1	97.9	CL	-
AECOM-B2, 1	451.2	Embankment Fill	30.0-32.0	17.9	108	127.3	-	-	-	0.0	20.4	79.6	CL	-
AECOM-B2, 2	451.2	Embankment Fill	48.0-50.0	15.2	111	127.9	-	-	-	0.0	36.3	63.7	CL	-
AECOM-B4, 1	416.1	Embankment Fill	12.0-14.0	16.4	110	128.0	-	-	-	0.1	17.0	82.9	CL	-
B-201, SS-2	450.3	Embankment Fill	3.5-5.0	15.2	-	-	29	22	7	-	-	99.4	CL	-
B-201, SS-7	450.3	Embankment Fill	16.0-17.5	14.5	-	-	-	-	-	-	-	96.8	CL	-
B-201, SS-11	450.3	Embankment Fill	26.0-27.5	15.5	-	-	29	21	8	-	-	97.4	ML	-
B-202, SS-6	450.7	Embankment Fill	13.5-15.0	14.0	-	-	28	18	10	-	-	95.2	CL	-
B-202, SS-11	450.7	Embankment Fill	26.0-27.5	15.6	-	-	33	15	18	-	-	66.1	CL	-
B-202, SS-16	450.7	Embankment Fill	38.5-40.0	14.7	-	-	32	17	15	-	-	81.7	CL	-
B-203, SS-7	450.5	Embankment Fill	16.0-17.5	16.5	-	-	31	14	17	-	-	71.0	CL	-
B-203, SS-13	450.5	Embankment Fill	31.0-32.5	15.7	-	-	25	18	7	-	-	87.2	CL	-
B-203, SS-22	450.5	Embankment Fill	53.5-55.0	11.7	-	-	26	16	10	-	-	58.7	CL	-
B-204, SS-4	450.5	Embankment Fill	8.5-10.0	18.1	-	-	32	19	13	-	-	98.8	CL	-
B-204, SS-15	450.5	Embankment Fill	36.0-37.5	16.3	-	-	29	22	7	-	-	99.5	CL	-
B-205, SS-5	415.5	Embankment Fill	11.0-12.5	16.3	-	-	33	15	18	-	-	88.5	CL	-
B-209, SS-16	451.0	Embankment Fill	38.5-40	14.7	-	-	25	14	11	-	-	-	CL	-
B-210, SS-13	451.0	Embankment Fill	31-32.5	16.0	-	-	37	17	26	-	-	-	CL	-
B-210, SS-17	451.0	Embankment Fill	41-42.5	17.5	-	-	35	13	22	-	-	-	CL	-
B-210, SS-20	451.0	Embankment Fill	48.5-50	17.8	-	-	27	16	11	-	-	-	CL	-
B-211, SS-11	451.0	Embankment Fill	26-27.5	19.0	-	-	31	17	14	-	-	-	CL	-
B-211, SS-15	451.0	Embankment Fill	36-37.5	17.7	-	-	30	17	30	-	-	-	CL	-
B-212, SS-11	451.0	Embankment Fill	26-27.5	16.2	-	-	38	19	19	-	-	-	CL	-
B-212, SS-14	451.0	Embankment Fill	33.5-35	15.4	-	-	34	14	20	-	-	-	CL	-
B-213, SS-9	451.0	Embankment Fill	21-22.5	14.6	-	-	29	16	13	-	-	-	CL	-
B-213, SS-12	451.0	Embankment Fill	28.5-30	18.4	-	-	24	21	3	-	-	-	ML	-
B-213, SS-17	451.0	Embankment Fill	41-42.5	15.5	-	-	37	16	21	-	-	-	CL	-
B-214, SS-11	451.0	Embankment Fill	21-22.5	16.6	-	-	31	17	14	-	-	-	CL	-
B-214, SS-17	451.0	Embankment Fill	41-42.5	15.6	-	-	29	18	11	-	-	-	CL	-
B-215, SS-4	415.0	Embankment Fill	8.5-10	16.1	-	-	28	12	16	-	-	-	SP	-
B-216, SS-6	415.0	Embankment Fill	13.5-15	21.6	-	-	36	15	21	-	-	-	CL	-
B-216, SS-9	415.0	Embankment Fill	21-22.5	23.9	-	-	30	17	13	-	-	-	CL	-
B-217, SS-8	415.0	Embankment Fill	18.5-20	20.5	-	-	28	17	11	-	-	-	CL	-
HLA-1, SS-3	450.9	Embankment Fill	13.5-15	15.4	119	137.3	-	-	-	-	-	-	CL	-

Summary of Laboratory Test Results – Embankment Fill

Boring and Sample ID	Ground Surface Elevation	Material Description	Sample Depth	Moisture Content	Dry Unit Weight	Total Unit Weight	Atterberg Limits			Gradations			USCS	Hydraulic Conductivity
							Liquid Limit	Plastic Limit	Plasticity Index	Sieve Analysis (3 inch to #200 Sieve)				
										Gravel	Sand	Fines		
	(ft)		(ft)	(%)	(pcf)	(pcf)	(%)	(%)	(%)	(%)	(%)	(%)		(cm/sec)
HLA-1, SS-6	450.9	Embankment Fill	28.5-30	15.5	119	137.4	29	19	10	-	-	-	CL	-
HLA-1, SS-8	450.9	Embankment Fill	38.5-40	18.5	112	132.7	-	-	-	-	-	-	CL	-
HLA-2, SS-1	450.7	Embankment Fill	3.5-5	15.0	114	131.1	-	-	-	-	-	-	CL	-
HLA-2, SS-2	450.7	Embankment Fill	8.5-10	19.2	101	120.4	-	-	-	-	-	-	ML	-
HLA-2, SS-3	450.7	Embankment Fill	8.5-10	23.9	102	126.4	-	-	-	-	-	-	ML	-
HLA-2, ST-6	450.7	Embankment Fill	19-21	17.3	114	133.7	-	-	-	-	-	-	CL	-
HLA-2, ST-7	450.7	Embankment Fill	19-21	17.6	111	130.5	-	-	-	-	-	-	CL	-
HLA-3, ST-3	451.9	Embankment Fill	8.5-10	20.7	108	130.4	28	27	1	-	-	-	ML	-
HLA-3, ST-3	451.9	Embankment Fill	10-12	18.9	107	127.2	-	-	-	-	-	-	ML	-
HLA-3, ST-3	451.9	Embankment Fill	10-12	18.5	114	135.1	-	-	-	-	-	-	ML	-
HLA-3, ST-3	451.9	Embankment Fill	10-12	17.5	110	129.3	-	-	-	-	-	-	ML	-
HLA-5, SS-1	416.1	Embankment Fill	3.5-5	20.2	101	121.4	-	-	-	-	-	-	ML	-
HLA-6, SS-2	416.2	Embankment Fill	3.5-5	15.8	118	136.6	-	-	-	-	-	-	ML	-

Summary of Laboratory Test Results - Foundation Silty Clays

Boring and Sample ID	Ground Surface Elevation	Material Description	Sample Depth	Moisture Content	Dry Unit Weight	Total Unit Weight	Atterberg Limits			Gradations			USCS	Hydraulic Conductivity
							Liquid Limit	Plastic Limit	Plasticity Index	Sieve Analysis (3 inch to #200 Sieve)				
										Gravel	Sand	Fines		
	(ft)		(ft)	(%)	(pcf)	(pcf)	(%)	(%)	(%)	(%)	(%)	(%)		(cm/sec)
AECOM-B3, 3	417.9	Foundation Clay	28.0-30.0	21.2	104.8	127	-	-	-	-	-	-	CL	4.2E-07
B-201, SS-20	450.3	Foundation Clay	48.5-50.0	23.9	-	-	32	17	15	-	-	98.2	CL	-
B-201, SS-21	450.3	Foundation Clay	51-52.5	23.0	-	-	-	-	-	-	-	-	CL	-
B-201, SS-22	450.3	Foundation Clay	53.5-55	22.4	-	-	-	-	-	-	-	-	CL	-
B-201, SS-23	450.3	Foundation Clay	56-57.5	18.6	-	-	-	-	-	-	-	-	CL	-
B-201, SS-24	450.3	Foundation Clay	58.5-60	22.2	-	-	-	-	-	-	-	-	CL	-
B-202, SS-20	450.7	Foundation Clay	48.5-50	14.0	-	-	-	-	-	-	-	-	CL	-
B-202, SS-21	450.7	Foundation Clay	51-52.5	16.3	-	-	-	-	-	-	-	-	CL	-
B-202, SS-22	450.7	Foundation Clay	53.5-55.0	14.0	-	-	42	16	26	-	-	66.1	CL	-
B-202, SS-23	450.7	Foundation Clay	56-57.5	19.1	-	-	-	-	-	-	-	-	CL	-
B-202, SS-24	450.7	Foundation Clay	58.5-60	22.8	-	-	-	-	-	-	-	-	CL	-
B-202, SS-25	450.7	Foundation Clay	61.0-62.5	8.0	-	-	29	19	10	-	-	88.6	CL	-
B-202, ST-26	450.7	Foundation Clay	63.0-65.0	23.2	102.5	126.3	26	20	6	-	-	67.8	CL	-
B-202, SS-27	450.7	Foundation Clay	66-67.5	17.7	-	-	-	-	-	-	-	-	CL	-
B-202, SS-29	450.7	Foundation Clay	71-72.5	24.4	-	-	-	-	-	-	-	-	CL-ML	-
B-202, SS-30	450.7	Foundation Clay	73.5-75.0	24.4	-	-	28	21	7	-	-	99.3	CL-ML	-
B-202, SS-32	450.7	Foundation Clay	78.5-80.0	16.4	-	-	21	13	8	-	-	71.7	CL	-
B-202, SS-34	450.7	Foundation Clay	83.5-85.0	15.3	-	-	31	15	16	-	-	43.6	CL	-
B-203, ST-26	450.5	Foundation Clay	63.0-65.0	19.3	106.4	126.9	30	19	11	-	-	96.6	CL	-
B-203, SS-28	450.5	Foundation Clay	68.5-70.0	21.9	-	-	36	19	17	-	-	99.6	CL	-
B-204, SS-23	450.5	Foundation Clay	56.0-57.5	28.1	-	-	28	21	7	-	-	99.3	CL-ML	-
B-206, SS-13	414.8	Foundation Clay	31.0-32.5	20.9	-	-	32	15	17	-	-	80.3	CL	-
B-206, ST-16	414.8	Foundation Clay	38.0-40.0	24.3	100.5	124.9	29	16	13	-	-	82.3	CL	-
B-206, SS-19	414.8	Foundation Clay	46.0-47.5	40.3	-	-	48	23	25	-	-	99.0	CL	-
B-207, SS-7	395.0	Foundation Clay	16.0-17.5	20.4	-	-	24	19	5	-	-	94.8	CL-ML	-
B-207, ST-8	395.0	Foundation Clay	18.0-20.0	23.4	101.2	124.9	31	16	15	-	-	92.2	CL	-
B-207, SS-16	395.0	Foundation Clay	38.5-40.0	17.6	-	-	30	15	15	-	-	61.7	CL	-
B-208, SS-15	396.7	Foundation Clay	36.0-37.5	18.8	-	-	33	16	17	-	-	84.1	CL	-
B-209, SS-23	451.0	Foundation Clay	56-57.5	29.2	-	-	38	18	20	-	-	-	CL	-
B-210, SS-27	451.0	Foundation Clay	66-67.5	20.2	-	-	26	17	9	-	-	-	CL	-
B-211, SS-21	451.0	Foundation Clay	51-52.5	-	-	-	29	19	10	-	-	-	CL	-
B-211, SS-24	451.0	Foundation Clay	58.5-60	20.7	-	-	30	20	10	-	-	-	CL	-
B-212, SS-22	451.0	Foundation Clay	53.5-55	20.6	-	-	27	17	10	-	-	-	CL	-
B-213, SS-25	451.0	Foundation Clay	61-62.5	23.3	-	-	35	16	19	-	-	-	CL	-
B-214, SS-20	451.0	Foundation Clay	48.5-50	27.4	-	-	28	24	4	-	-	-	CL-ML	-
B-214, SS-23	451.0	Foundation Clay	56-57.5	22.5	-	-	29	16	13	-	-	-	CL	-
B-215, SS-9	415.0	Foundation Clay	21-22.5	27.3	-	-	29	20	9	-	-	-	CL	-

Summary of Laboratory Test Results - Foundation Silty Clays

Boring and Sample ID	Ground Surface Elevation	Material Description	Sample Depth	Moisture Content	Dry Unit Weight	Total Unit Weight	Atterberg Limits			Gradations			USCS	Hydraulic Conductivity
							Liquid Limit	Plastic Limit	Plasticity Index	Sieve Analysis (3 inch to #200 Sieve)				
										Gravel	Sand	Fines		
	(ft)		(ft)	(%)	(pcf)	(pcf)	(%)	(%)	(%)	(%)	(%)	(%)		(cm/sec)
B-217, SS-10	415.0	Foundation Clay	23.5-25	17.8	-	-	29	20	9	-	-	-	CL	-
B-217, SS-15	415.0	Foundation Clay	36-37.5	30.6	-	-	29	21	8	-	-	-	CL-ML	-
B-217, SS-23	415.0	Foundation Clay	56-57.5	23.5	-	-	38	16	22	-	-	-	CL	-
B-218, SS-10	415.0	Foundation Clay	23.5-25	23.4	-	-	24	18	6	-	-	-	CL-ML	-
B-218, SS-17	415.0	Foundation Clay	41-42.5	28.6	-	-	32	25	7	-	-	-	CL-ML	-
B-218, SS-22	415.0	Foundation Clay	53.5-55	20.9	-	-	45	16	29	-	-	-	CL	-
B-219, SS-8	415.0	Foundation Clay	18.5-20	23.9	-	-	28	18	10	-	-	-	CL	-
B-219, SS-11	415.0	Foundation Clay	26-27.5	21.5	-	-	30	13	17	-	-	-	CL	-
B-219, SS-16	415.0	Foundation Clay	38.5-40	26.0	-	-	30	20	10	-	-	-	CL	-
HLA-2, SS-9	450.7	Foundation Clay	33.5-35	20.8	106	127.8	-	-	-	-	-	-	CL	-
HLA-3, SS-11	451.9	Foundation Clay	41-42.5	25.6	99	124.3	-	-	-	-	-	-	CL	-
HLA-4, SS-16	449.6	Foundation Clay	51-52.5	45.5	77	112.0	63	27	36	-	-	-	CL	-
HLA-4, SS-16	449.6	Foundation Clay	51-52.6	32.7	88	116.8	-	-	-	-	-	-	CL	-
HLA-5, SS-8	416.1	Foundation Clay	23.5-25	23.6	-	-	29	20	9	-	-	-	CL	-
HLA-6, SS-6	416.2	Foundation Clay	13.5-15	43.3	-	-	75	27	48	-	-	-	CL	-
HLA-6, SS-8	416.2	Foundation Clay	18.5-20	19.0	111	132.1	-	-	-	-	-	-	CL	-
HLA-6A, SS-2	416.2	Foundation Clay	6-7.5	28.2	94	120.5	-	-	-	-	-	-	CL	-
HLA-6A, ST-3	416.2	Foundation Clay	8-10	28.1	-	-	37	22	15	-	-	-	CL	-
HLA-6A, ST-3	416.2	Foundation Clay	8-10	28.3	97	124.5	-	-	-	-	-	-	CL	-
HLA-6A, ST-3	416.2	Foundation Clay	8-10	28.5	94	120.8	-	-	-	-	-	-	CL	-
HLA-6A, ST-3	416.2	Foundation Clay	8-11	28.6	93	119.6	-	-	-	-	-	-	CL	-

Summary of Laboratory Test Results - Foundation Silts

Boring and Sample ID	Ground Surface Elevation	Material Description	Sample Depth	Moisture Content	Dry Unit Weight	Total Unit Weight	Atterberg Limits			Gradations			USCS	Hydraulic Conductivity
							Liquid Limit	Plastic Limit	Plasticity Index	Sieve Analysis (3 inch to #200 Sieve)				
										Gravel	Sand	Fines		
	(ft)		(ft)	(%)	(pcf)	(pcf)	(%)	(%)	(%)	(%)	(%)	(%)		(cm/sec)
AECOM-B1, 3	451.3	Foundation Silt	39.0-41.0	27.5	-	-	Non-Plastic			0.0	0.4	99.6	ML	-
AECOM-B1, 4	451.3	Foundation Silt	44.0-46.0	26.5	98	124.0	-	-	-	0.0	0.4	99.6	ML	-
AECOM-B1, 5	451.3	Foundation Silt	49.0-51.0	26.8	96.6	122.7	-	-	-	-	-	-	ML	2.6E-07
AECOM-B2, 3	451.2	Foundation Silt	56.0-58.0	25.0	-	-	Non-Plastic			0.0	0.9	99.1	ML	-
AECOM-B2, 4	451.2	Foundation Silt	60.0-62.0	25.9	98.3	123.8	-	-	-	0.0	0.3	99.7	ML	8.70E-07
AECOM-B3, 1	417.9	Foundation Silt	8.0-10.0	30.6	88	115.0	-	-	-	-	-	-	ML	5.20E-06
AECOM-B4, 2	416.1	Foundation Silt	33.0-35.0	38.4	82.7	114.5	31	29	2	0.0	0.3	99.7	ML	-
AECOM-B4, 3	416.1	Foundation Silt	46.0-48.0	26.8	97.4	123.5	-	-	-	0.0	0.1	99.9	ML	-
AECOM-B5, 2	416.4	Foundation Silt	30.0-32.0	33.8	-	-	-	-	-	0.4	28.4	71.2	ML	-
AECOM-B5, 3	416.4	Foundation Silt	34.0-36.0	49.8	71	106.4	-	-	-	-	-	-	ML	7.80E-06
B-201, SS-18	450.3	Foundation Silt	43.5-45.0	29.0	-	-	Non-Plastic			-	-	99.7	ML	-
B-204, SS-20	450.5	Foundation Silt	48.5-50.0	23.5	-	-	27	22	5	-	-	97.2	ML	-
B-205, SS-14	415.5	Foundation Silt	33.5-35.0	36.6	-	-	Non-Plastic			-	-	95.0	ML	-
B-205, SS-19	415.5	Foundation Silt	46.0-47.5	43.5	-	-	Non-Plastic			-	-	92.9	ML	-
B-206, SS-9	414.8	Foundation Silt	21.0-22.5	23.7	-	-	Non-Plastic			-	-	98.3	ML	-
B-206, ST-12	414.8	Foundation Silt	28.0-30.0	21.1	106.2	128.6	23	20	3	-	-	96.6	ML	-
B-206, SS-17	414.8	Foundation Silt	41.0-42.5	24.6	-	-	26	23	3	-	-	94.2	ML	-
B-206, SS-24	414.8	Foundation Silt	58.5-60.0	36.9	-	-	33	31	2	-	-	96.4	ML	-
B-206, SS-25	414.8	Foundation Silt	61.0-62.5	39.8	-	-	38	34	4	-	-	96.3	ML	-
B-207, SS-13	395.0	Foundation Silt	31.0-32.5	26.7	-	-	Non-Plastic			-	-	95.2	ML	-
B-207, ST-15	395.0	Foundation Silt	35.0-37.0	32.0	89.5	118.1	31	25	6	-	-	73.5	ML	-
B-208, SS-7	396.7	Foundation Silt	16.0-17.5	26.3	-	-	26	22	4	-	-	99.7	ML	-
B-208, SS-13	396.7	Foundation Silt	31.0-32.5	27.6	-	-	28	24	4	-	-	99.6	ML	-
B-209, SS-19	451.0	Foundation Silt	46-47.5	29.3	-	-	Non-Plastic			-	-	-	ML	-
B-210, SS-23	451	Foundation Silt	56-57.5	24.5	-	-	26	23	3	-	-	-	ML	-
B-211, SS-28	451.0	Foundation Silt	68.5-70	29.9	-	-	Non-Plastic			-	-	-	ML	-
B-212, SS-27	451.0	Foundation Silt	66-67.5	22.2	-	-	25	22	3	-	-	-	ML	-
B-215, SS-12	415.0	Foundation Silt	28.5-30	36.3	-	-	Non-Plastic			-	-	-	ML	-
B-215, SS-15	415.0	Foundation Silt	36-37.5	35.9	-	-	Non-Plastic			-	-	-	ML	-
B-215, SS-18	415.0	Foundation Silt	43.5-45	26.5	-	-	26	23	3	-	-	-	ML	-
B-216, SS-11	415.0	Foundation Silt	26-27.5	24.3	-	-	Non-Plastic			-	-	-	ML	-
B-216, SS-16	415.0	Foundation Silt	38.5-40	33.9	-	-	Non-Plastic			-	-	-	ML	-
B-217, SS-18	415.0	Foundation Silt	43.5-45	39.5	-	-	37	35	2	-	-	-	ML	-
B-217, SS-21	415.0	Foundation Silt	51-52.5	26.4	-	-	Non-Plastic			-	-	-	ML	-

Summary of Laboratory Test Results - Foundation Silts

Boring and Sample ID	Ground Surface Elevation	Material Description	Sample Depth	Moisture Content	Dry Unit Weight	Total Unit Weight	Atterberg Limits			Gradations			USCS	Hydraulic Conductivity
							Liquid Limit	Plastic Limit	Plasticity Index	Sieve Analysis (3 inch to #200 Sieve)				
										Gravel	Sand	Fines		
	(ft)		(ft)	(%)	(pcf)	(pcf)	(%)	(%)	(%)	(%)	(%)	(%)		(cm/sec)
B-218, SS-4	415.0	Foundation Silt	8.5-10	22.3	-	-	26	25	1	-	-	-	ML	-
B-218, SS-5	415.0	Foundation Silt	11-13.5	30.6	-	-	27	26	1	-	-	-	ML	-
B-218, SS-20	415.0	Foundation Silt	48.5-50	23.0	-	-	Non-Plastic			-	-	-	ML	-
B-219, SS-4	415.0	Foundation Silt	8.5-10	28.9	-	-	Non-Plastic			-	-	-	ML	-
B-219, SS-7	415.0	Foundation Silt	16-17.5	29.9	-	-	Non-Plastic			-	-	-	ML	-
HLA-2, ST-11	450.7	Foundation Silt	43-45	27.2	98	124.7	-	-	-	-	-	-	ML	-
HLA-4, SS-6	449.6	Foundation Silt	26-27.5	32.0	-	-	Non-Plastic			-	-	-	ML	-
HLA-5, SS-4	416.1	Foundation Silt	13.5-15	27.5	98	125.0	Non-Plastic			-	-	-	ML	-
HLA-5, ST-5	416.1	Foundation Silt	15-17	29.6	95	123.1	-	-	-	-	-	-	ML	-

Soil Index Properties Laboratory Test Results

Client:	AECOM				
Project:	Vectran AB Brown Ash Pond Lower Dam				
Location:	Evansville, IN			Project No:	GTX-303915
Boring ID:	AECOM-B1	Sample Type:	tube	Tested By:	jbr
Sample ID:	3	Test Date:	11/17/15	Checked By:	jdt
Depth :	31-41	Test Id:	354184		

Test Comment: ---
Visual Description: Moist, brown silt
Sample Comment: ---

Moisture Content of Soil and Rock - ASTM D2216

Boring ID	Sample ID	Depth	Description	Moisture Content,%
AECOM-B1	3	31-41	Moist, brown silt	27.5

Notes: Temperature of Drying : 110º Celsius

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Client:	AECOM		
Project:	Vectran AB Brown Ash Pond Lower Dam		
Location:	Evansville, IN	Project No:	GTX-303915

Boring ID:	---	Sample Type:	---	Tested By:	GA
Sample ID:	---	Test Date:	12/14/15	Checked By:	emm
Depth :	---	Test Id:	354994		

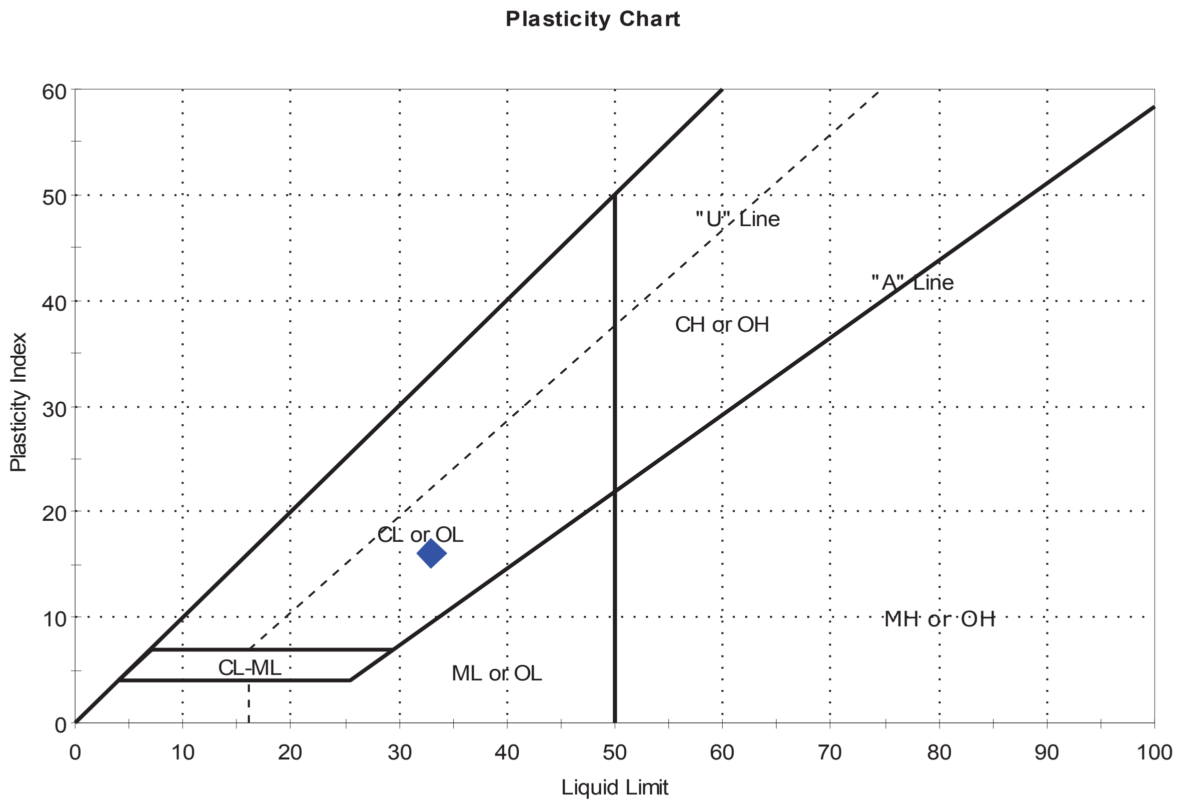
Moisture Content of Soil and Rock - ASTM D2216

Boring ID	Sample ID	Depth	Description	Moisture Content,%
AECOM-B1	1A	19-21	Moist, reddish yellow clay	20.7
AECOM-B1	2	27-29	Moist, dark yellowish brown silt	15.5
AECOM-B2	1	30-32	Moist, reddish yellow clay with sand	16.2
AECOM-B2	2	48-50	Moist, reddish yellow sandy clay	15.3
AECOM-B2	3	56-58	Moist, brown silt	25.0
AECOM-B2	4A	62-64	Moist, gray silt	24.7
AECOM-B4	1	12-14	Moist, yellowish brown clay with sand	16.8
AECOM-B4	2	33-35	Wet, olive silt	37.2
AECOM-B4	3	46-48	Moist, olive silt	29.9
AECOM-B5	2	30-32	Moist, gray silt with sand	33.8

Notes: Temperature of Drying : 110º Celsius

Client: AECOM
Project: Vectran AB Brown Ash Pond Lower Dam
Location: Evansville, IN
Project No: GTX-303915
Boring ID: AECOM- B1
Sample Type: tube
Tested By: GA
Sample ID: 1A
Test Date: 12/14/15
Checked By: emm
Depth : 19-21
Test Id: 354627
Test Comment: ---
Visual Description: Moist, reddish yellow clay
Sample Comment: ---

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	1A	AECOM- B1	19-21	21	33	17	16	0.2	Lean clay (CL)

Sample Prepared using the WET method
2% Retained on #40 Sieve
Dry Strength: HIGH
Dilatancy: NONE
Toughness: MEDIUM

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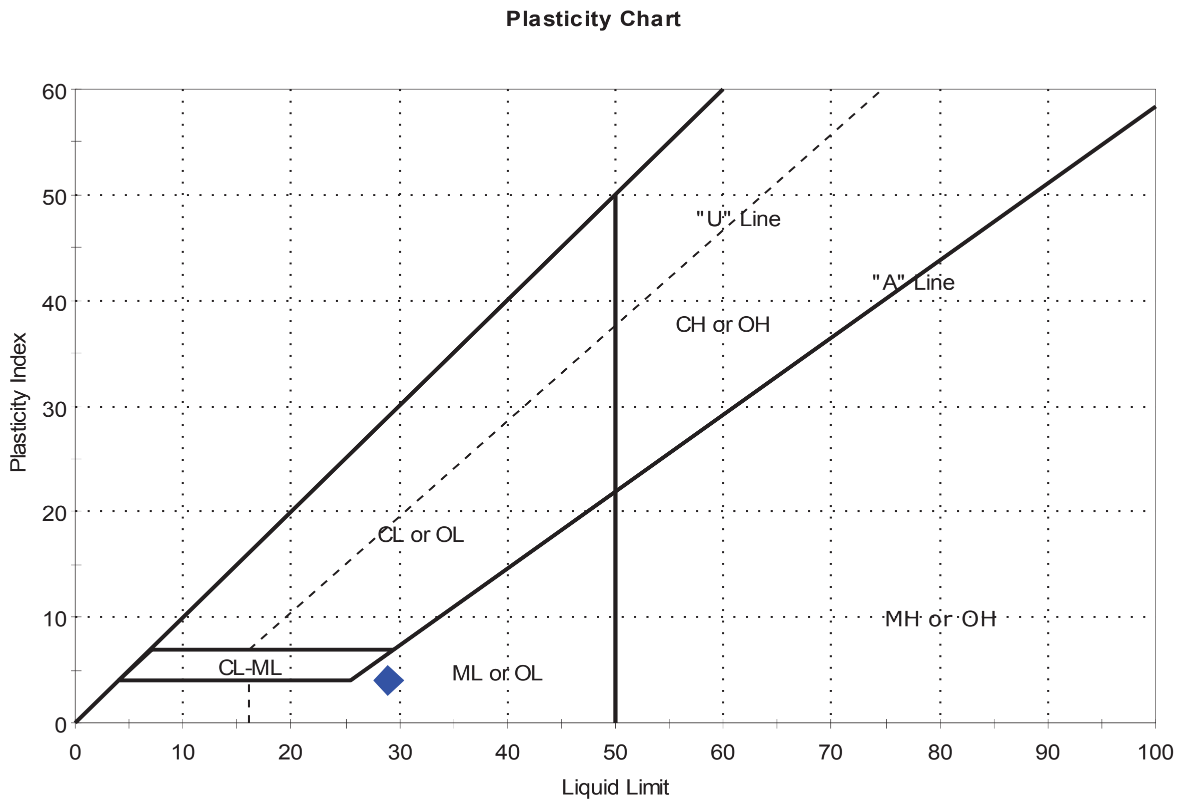
Client: AECOM
Project: Vectran AB Brown Ash Pond Lower Dam
Location: Evansville, IN
Project No: GTX-303915

Boring ID: AECOM- B1
Sample Type: tube
Tested By: GA
Sample ID: 2
Test Date: 12/14/15
Checked By: emm
Depth : 27-29
Test Id: 354626

Test Comment: ---
Visual Description: Moist, dark yellowish brown silt
Sample Comment: ---

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	2	AECOM-B1	27-29	15	29	25	4	-2.4	Silt (ML)

Sample Prepared using the WET method

0% Retained on #40 Sieve

Dry Strength: LOW

Dilatancy: SLOW

Toughness: LOW

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GeoTesting EXPRESS

Client:	AECOM				
Project:	Vectran AB Brown Ash Pond Lower Dam				
Location:	Evansville, IN			Project No:	GTX-303915
Boring ID:	AECOM-B1	Sample Type:	tube	Tested By:	cam
Sample ID:	3	Test Date:	11/17/15	Checked By:	jdt
Depth :	31-41	Test Id:	354183		

Test Comment: ---
Visual Description: Moist, brown silt
Sample Comment: ---

Atterberg Limits - ASTM D4318

Sample Determined to be non-plastic

Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	3	ECOM-B1	31-41	27	n/a	n/a	n/a	n/a	Silt (ML)

0% Retained on #40 Sieve

Dry Strength: MEDIUM

Dilatancy: RAPID

Toughness: n/a

The sample was determined to be Non-Plastic

Client:	AECOM				
Project:	Vectran AB Brown Ash Pond Lower Dam				
Location:	Evansville, IN			Project No:	GTX-303915
Boring ID:	AECOM- B2	Sample Type:	tube	Tested By:	cam
Sample ID:	3	Test Date:	11/23/15	Checked By:	emm
Depth :	56-58	Test Id:	354629		

Test Comment: ---
Visual Description: Moist, brown silt
Sample Comment: ---

Atterberg Limits - ASTM D4318

Sample Determined to be non-plastic

Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	3	AECOM-B2	56-58	25	n/a	n/a	n/a	n/a	Silt (ML)

0% Retained on #40 Sieve
Dry Strength: MEDIUM
Dilatancy: RAPID
Toughness: n/a
The sample was determined to be Non-Plastic

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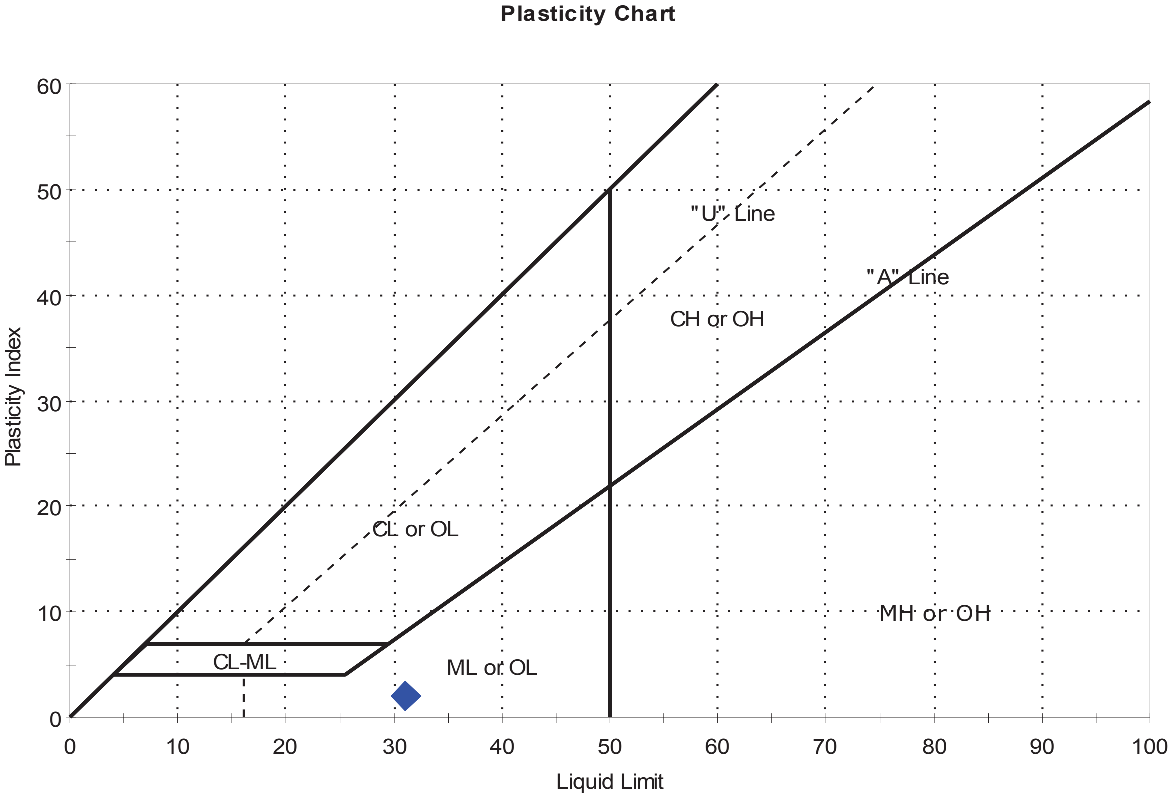
Client: AECOM			
Project: Vectran AB Brown Ash Pond Lower Dam			
Location: Evansville, IN		Project No:	GTX-303915
Boring ID: AECOM- B4	Sample Type: tube	Tested By:	cam
Sample ID: 2	Test Date: 11/24/15	Checked By:	emm
Depth : 33-35	Test Id: 354628		

Test Comment: ---

Visual Description: Wet, olive silt

Sample Comment: ---

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	2	AECOM-B4	33-35	37	31	29	2	4.1	Silt (ML)

Sample Prepared using the WET method

0% Retained on #40 Sieve

Dry Strength: HIGH

Dilatancy: SLOW

Toughness: LOW

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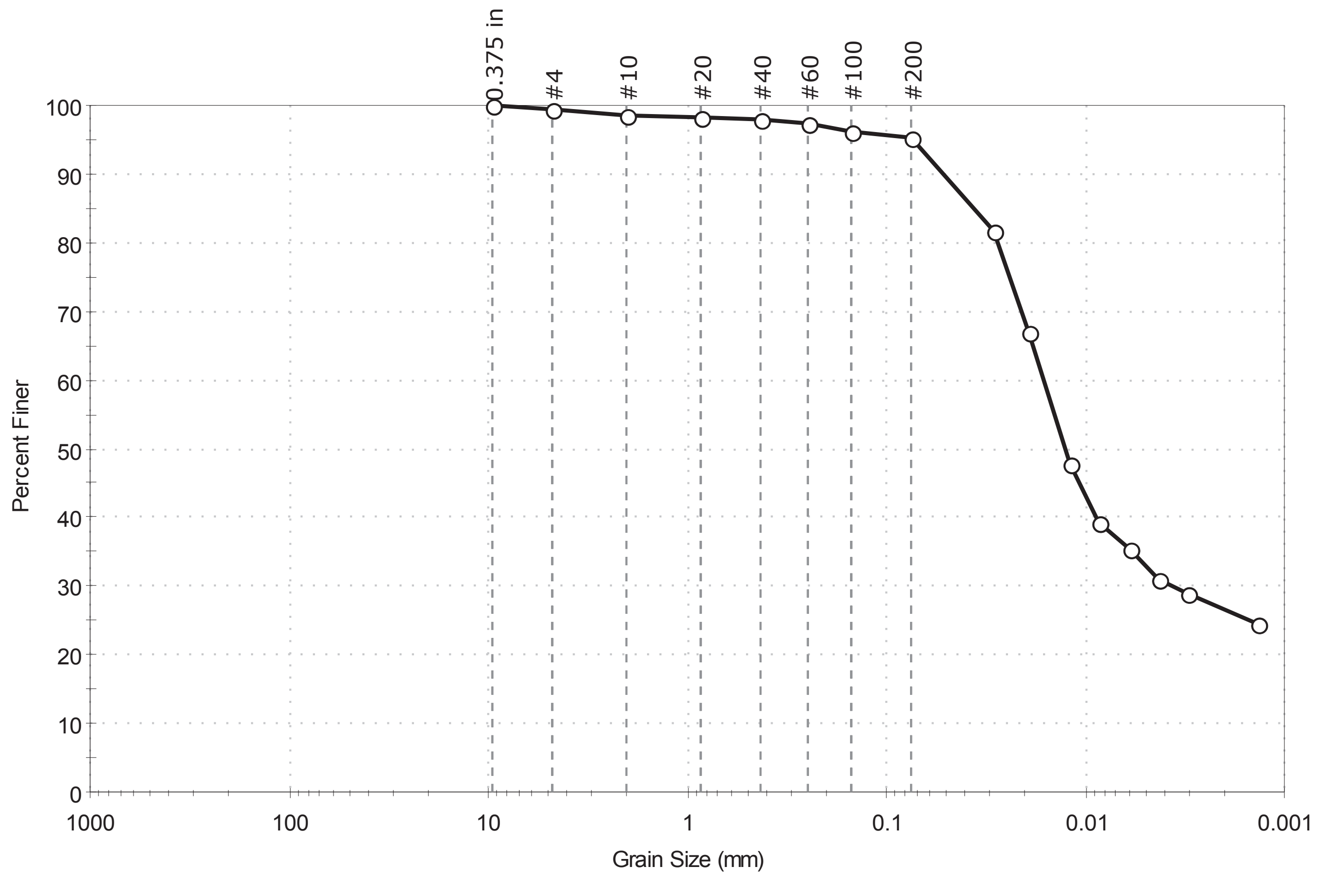
Client: AECOM
Project: Vectran AB Brown Ash Pond Lower Dam
Location: Evansville, IN
Project No: GTX-303915
Boring ID: AECOM-B1
Sample Type: tube
Tested By: GA
Sample ID: 1A
Test Date: 12/14/15
Checked By: emm
Depth : 19-21
Test Id: 354617
Test Comment: ---
Visual Description: Moist, reddish yellow clay
Sample Comment: ---

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.7	3.9	95.4

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.375 in	9.50	100		
#4	4.75	99		
#10	2.00	99		
#20	0.85	98		
#40	0.42	98		
#60	0.25	97		
#100	0.15	96		
#200	0.075	95		
---	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0292	82		
---	0.0194	67		
---	0.0119	48		
---	0.0085	39		
---	0.0059	35		
---	0.0043	31		
---	0.0031	29		
---	0.0014	24		

Coefficients

D_{85} =0.0366 mm
D_{30} =0.0036 mm
D_{60} =0.0162 mm
D_{15} =N/A
D_{50} =0.0125 mm
D_{10} =N/A
C_u =N/A
C_c =N/A

Classification

ASTM	Lean clay (CL)
AASHTO	Clayey Soils (A-6 (15))

Sample/Test Description

Sand/Gravel Particle Shape : ---
Sand/Gravel Hardness : ---
Dispersion Device : Apparatus A - Mech Mixer
Dispersion Period : 1 minute
Specific Gravity : 2.65
Separation of Sample: #200 Sieve

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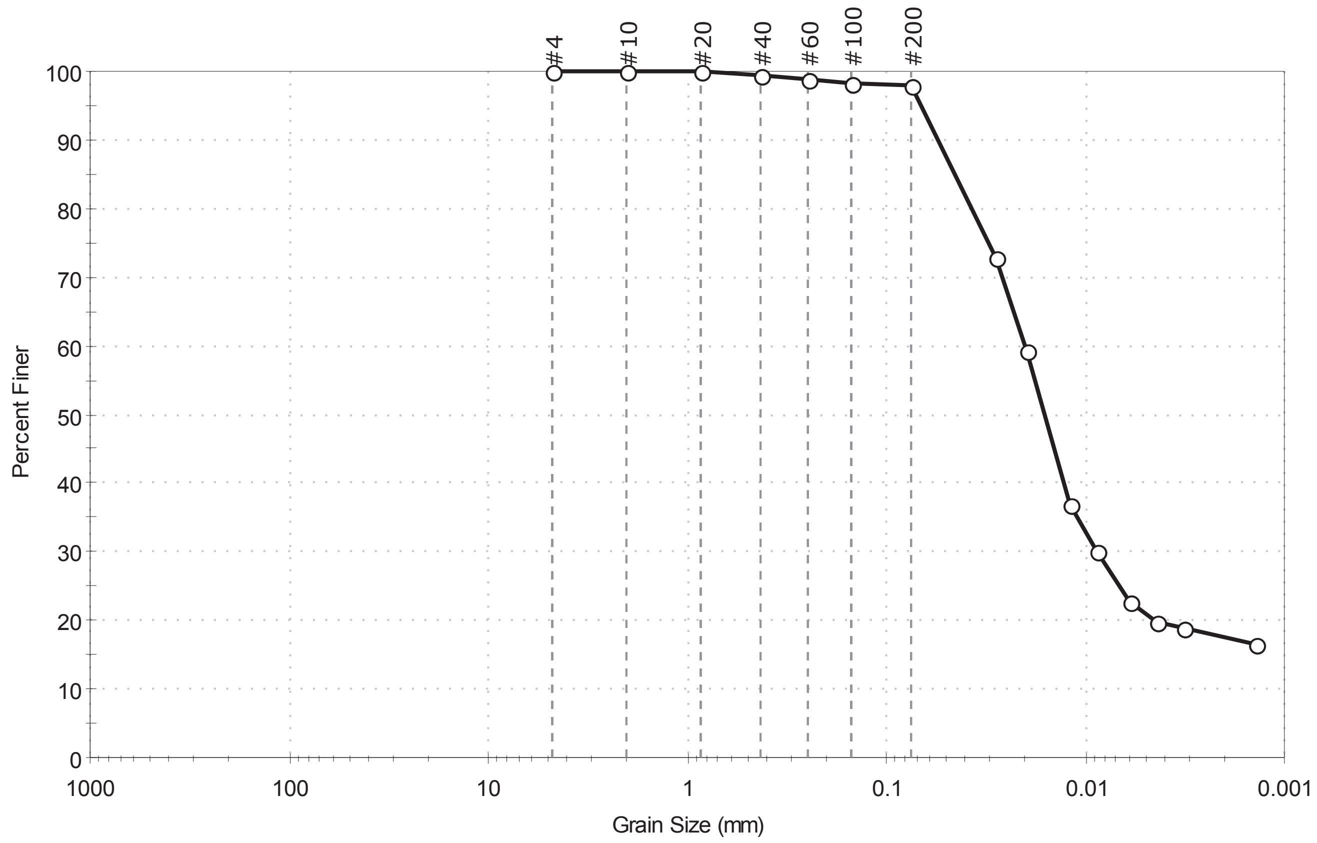
Client: AECOM
Project: Vectran AB Brown Ash Pond Lower Dam
Location: Evansville, IN
Project No: GTX-303915
Boring ID: AECOM-B1
Sample Type: tube
Tested By: GA
Sample ID: 2
Test Date: 12/14/15
Checked By: emm
Depth : 27-29
Test Id: 354616
Test Comment: ---
Visual Description: Moist, dark yellowish brown silt
Sample Comment: ---

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.0	2.1	97.9

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
#4	4.75	100		
#10	2.00	100		
#20	0.85	100		
#40	0.42	100		
#60	0.25	99		
#100	0.15	98		
#200	0.075	98		
---	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0279	73		
---	0.0195	59		
---	0.0121	37		
---	0.0088	30		
---	0.0061	23		
---	0.0044	20		
---	0.0032	19		
---	0.0014	17		

Coefficients

D_{85} =0.0452 mm	D_{30} =0.0087 mm
D_{60} =0.0199 mm	D_{15} =N/A
D_{50} =0.0160 mm	D_{10} =N/A
C_u =N/A	C_c =N/A

Classification

ASTM Silt (ML)

AASHTO Silty Soils (A-4 (4))

Sample/Test Description

Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

Dispersion Device : Apparatus A - Mech Mixer

Dispersion Period : 1 minute

Specific Gravity : 2.65

Separation of Sample: #200 Sieve

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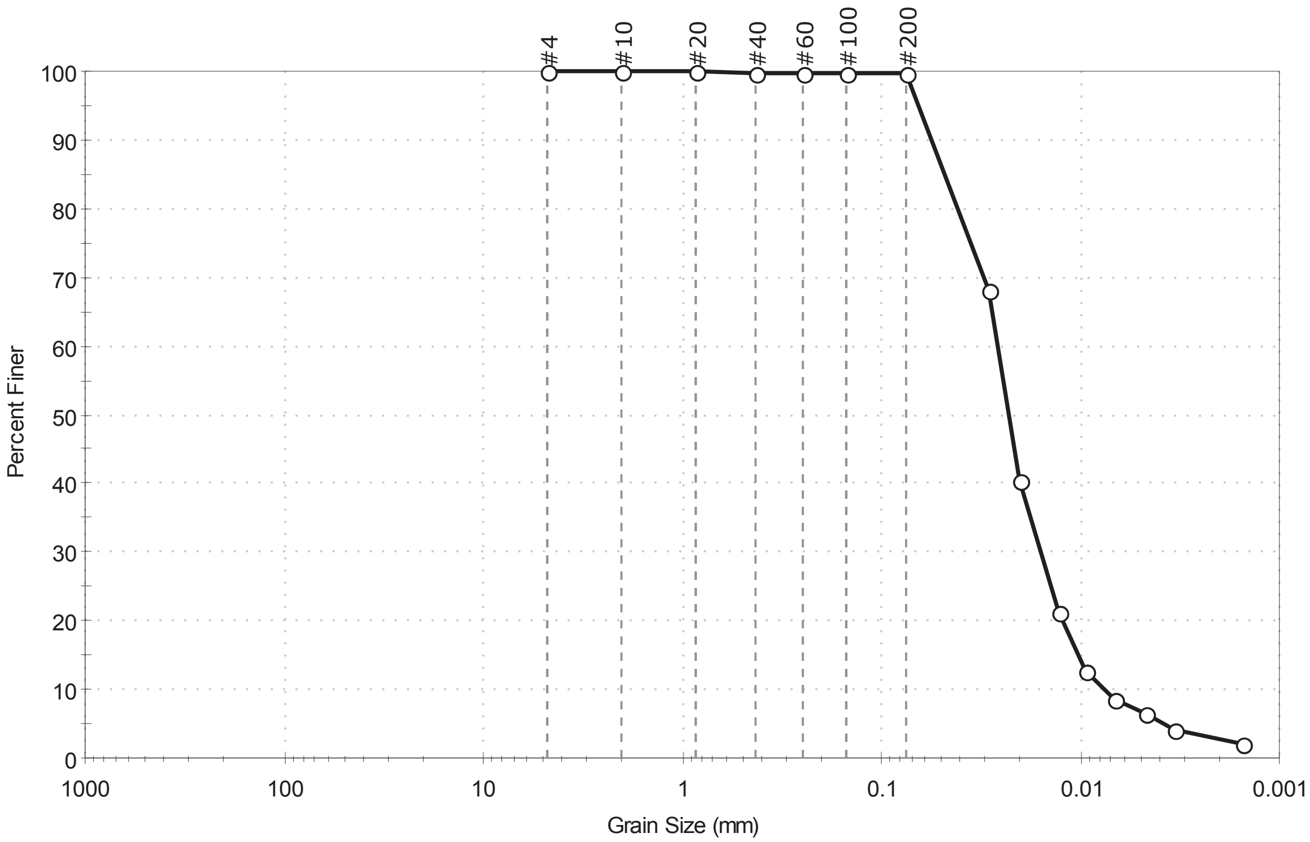
GeoTesting
EXPRESS

Client: AECOM
Project: Vectran AB Brown Ash Pond Lower Dam
Location: Evansville, IN
Project No: GTX-303915
Boring ID: AECOM-B1
Sample Type: tube
Tested By: jbr
Sample ID: 3
Test Date: 11/17/15
Checked By: jdt
Depth : 31-41
Test Id: 354182
Test Comment: ---
Visual Description: Moist, brown silt
Sample Comment: ---

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.0	0.4	99.6

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
#4	4.75	100		
#10	2.00	100		
#20	0.85	100		
#40	0.42	100		
#60	0.25	100		
#100	0.15	100		
#200	0.075	100		
---	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0288	68		
---	0.0203	41		
---	0.0129	21		
---	0.0093	13		
---	0.0067	9		
---	0.0048	6		
---	0.0034	4		
---	0.0015	2		

Coefficients

D_{85} =0.0481 mm
D_{30} =0.0158 mm
D_{60} =0.0259 mm
D_{15} =0.0102 mm
D_{50} =0.0228 mm
D_{10} =0.0075 mm
C_u =3.453
C_c =1.285

Classification

ASTM Silt (ML)

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

Dispersion Device : Apparatus A - Mech Mixer

Dispersion Period : 1 minute

Specific Gravity : 2.65

Separation of Sample: #200 Sieve

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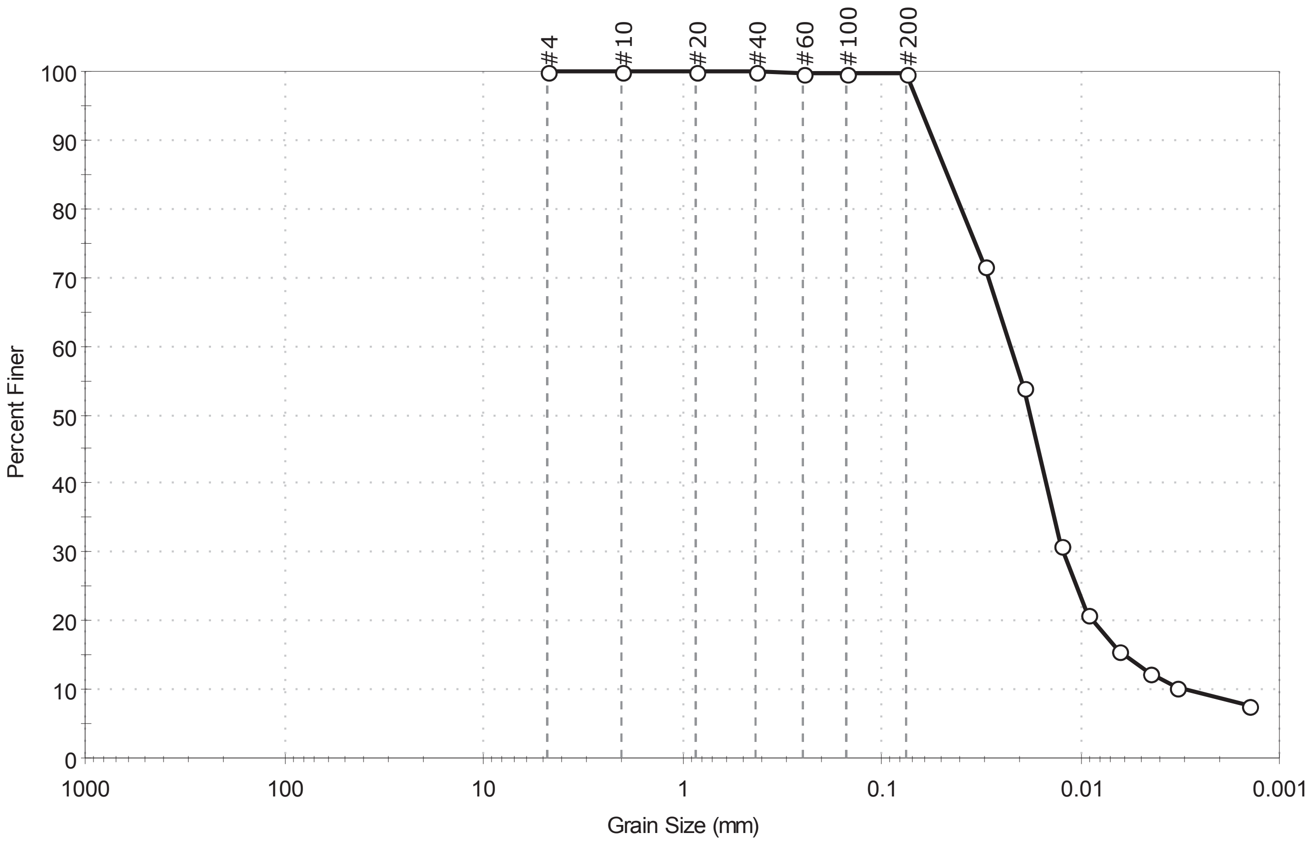
Client: AECOM
Project: Vectran AB Brown Ash Pond Lower Dam
Location: Evansville, IN
Project No: GTX-303915

Boring ID: AECOM-B1
Sample Type: tube
Tested By: GA
Sample ID: 4
Test Date: 12/14/15
Checked By: emm
Depth : 44-46
Test Id: 354630

Test Comment: ---
Visual Description: Moist, dark yellowish brown clay
Sample Comment: ---

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.0	0.4	99.6

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
#4	4.75	100		
#10	2.00	100		
#20	0.85	100		
#40	0.42	100		
#60	0.25	100		
#100	0.15	100		
#200	0.075	100		
---	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0305	72		
---	0.0193	54		
---	0.0126	31		
---	0.0091	21		
---	0.0064	16		
---	0.0045	12		
---	0.0033	10		
---	0.0014	8		

Coefficients

D_{85} =0.0469 mm
D_{30} =0.0122 mm
D_{60} =0.0226 mm
D_{15} =0.0059 mm
D_{50} =0.0179 mm
D_{10} =0.0030 mm
C_u =7.533
C_c =2.195

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ---
Sand/Gravel Hardness : ---
Dispersion Device : Apparatus A - Mech Mixer
Dispersion Period : 1 minute
Specific Gravity : 2.65
Separation of Sample: #200 Sieve

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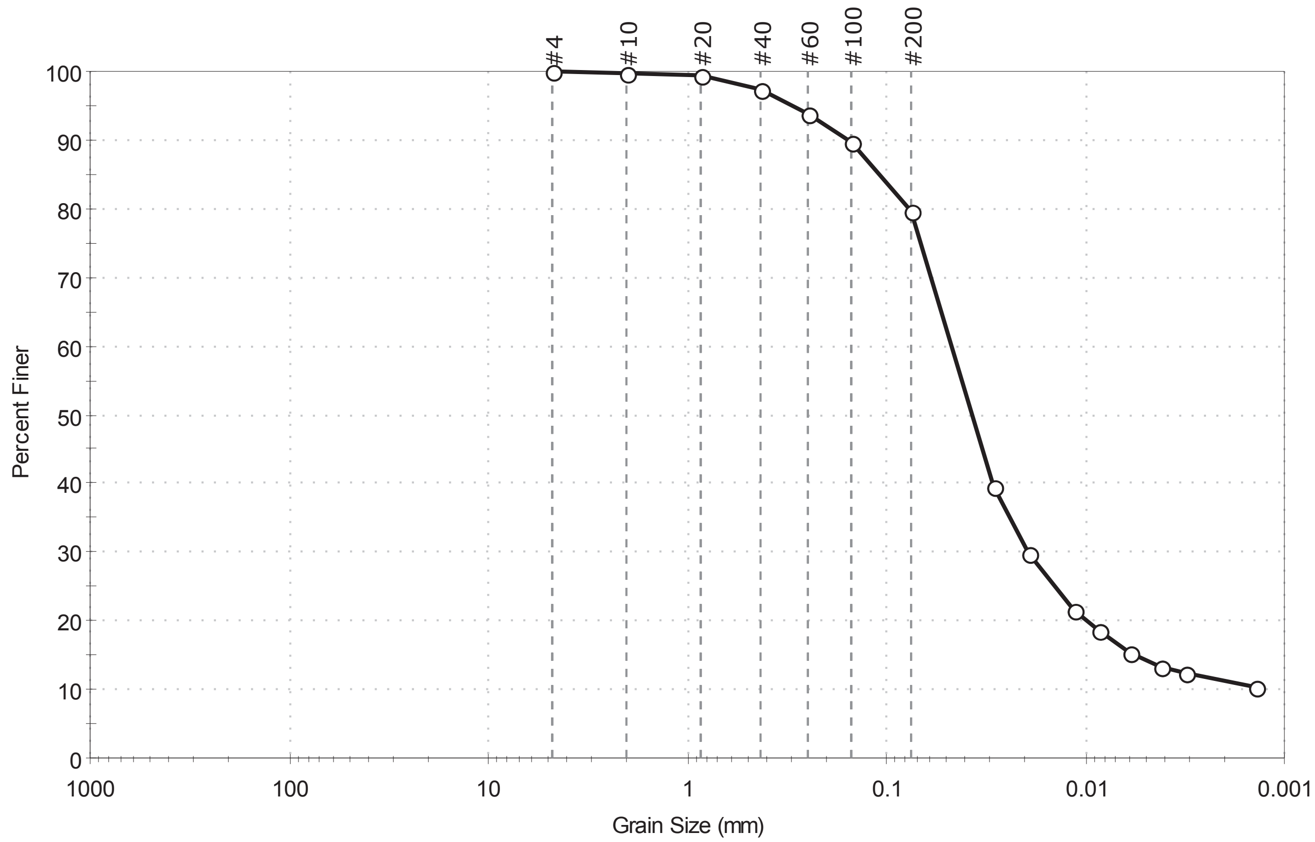
Client:	AECOM				
Project:	Vectran AB Brown Ash Pond Lower Dam				
Location:	Evansville, IN			Project No:	GTX-303915
Boring ID:	AECOM-B2	Sample Type:	tube	Tested By:	GA
Sample ID:	1	Test Date:	12/14/15	Checked By:	emm
Depth :	30-32	Test Id:	354622		

Test Comment: ---
Visual Description: Moist, reddish yellow clay with sand
Sample Comment: ---

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.0	20.4	79.6

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
#4	4.75	100		
#10	2.00	100		
#20	0.85	99		
#40	0.42	97		
#60	0.25	94		
#100	0.15	90		
#200	0.075	80		
---	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0289	40		
---	0.0193	30		
---	0.0114	22		
---	0.0086	19		
---	0.0060	15		
---	0.0042	13		
---	0.0031	12		
---	0.0014	10		

Coefficients

D_{85} =0.1085 mm	D_{30} =0.0194 mm
D_{60} =0.0471 mm	D_{15} =0.0057 mm
D_{50} =0.0371 mm	D_{10} =N/A
C_u =N/A	C_c =N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ---
Sand/Gravel Hardness : ---
Dispersion Device : Apparatus A - Mech Mixer
Dispersion Period : 1 minute
Specific Gravity : 2.65
Separation of Sample: #200 Sieve

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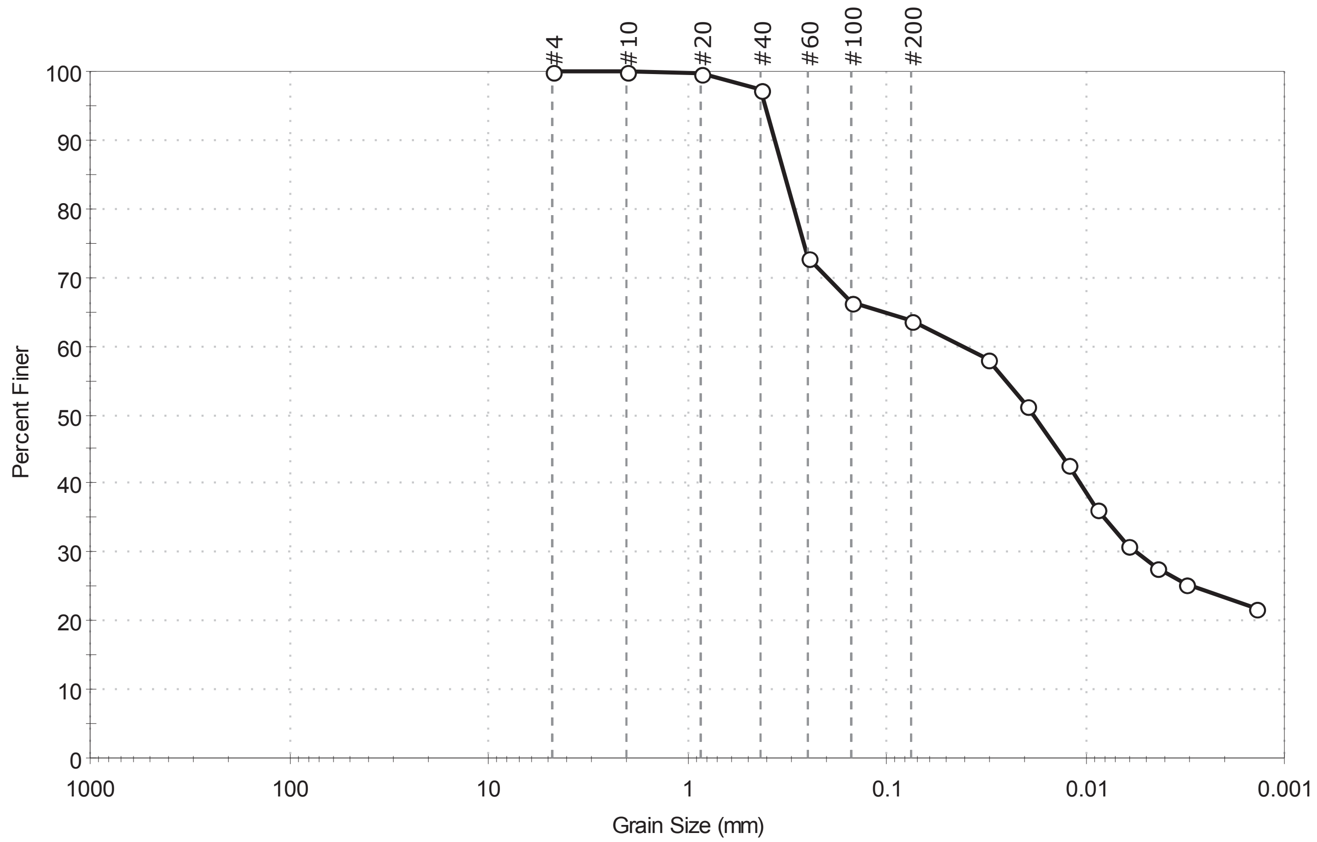
Client:	AECOM			
Project:	Vectran AB Brown Ash Pond Lower Dam			
Location:	Evansville, IN		Project No:	GTX-303915
Boring ID: AECOM-B2		Sample Type: tube	Tested By:	GA
Sample ID: 2		Test Date: 12/14/15	Checked By:	emm
Depth : 48-50		Test Id: 354623		

Test Comment: ---
Visual Description: Moist, reddish yellow sandy clay
Sample Comment: ---

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.0	36.3	63.7

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
#4	4.75	100		
#10	2.00	100		
#20	0.85	100		
#40	0.42	97		
#60	0.25	73		
#100	0.15	66		
#200	0.075	64		
---	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0314	58		
---	0.0199	51		
---	0.0121	43		
---	0.0087	36		
---	0.0061	31		
---	0.0043	28		
---	0.0031	25		
---	0.0014	22		

Coefficients

D_{85} =0.3251 mm	D_{30} =0.0055 mm
D_{60} =0.0425 mm	D_{15} =N/A
D_{50} =0.0183 mm	D_{10} =N/A
C_u =N/A	C_c =N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ---
Sand/Gravel Hardness : ---
Dispersion Device : Apparatus A - Mech Mixer
Dispersion Period : 1 minute
Specific Gravity : 2.65
Separation of Sample: #200 Sieve

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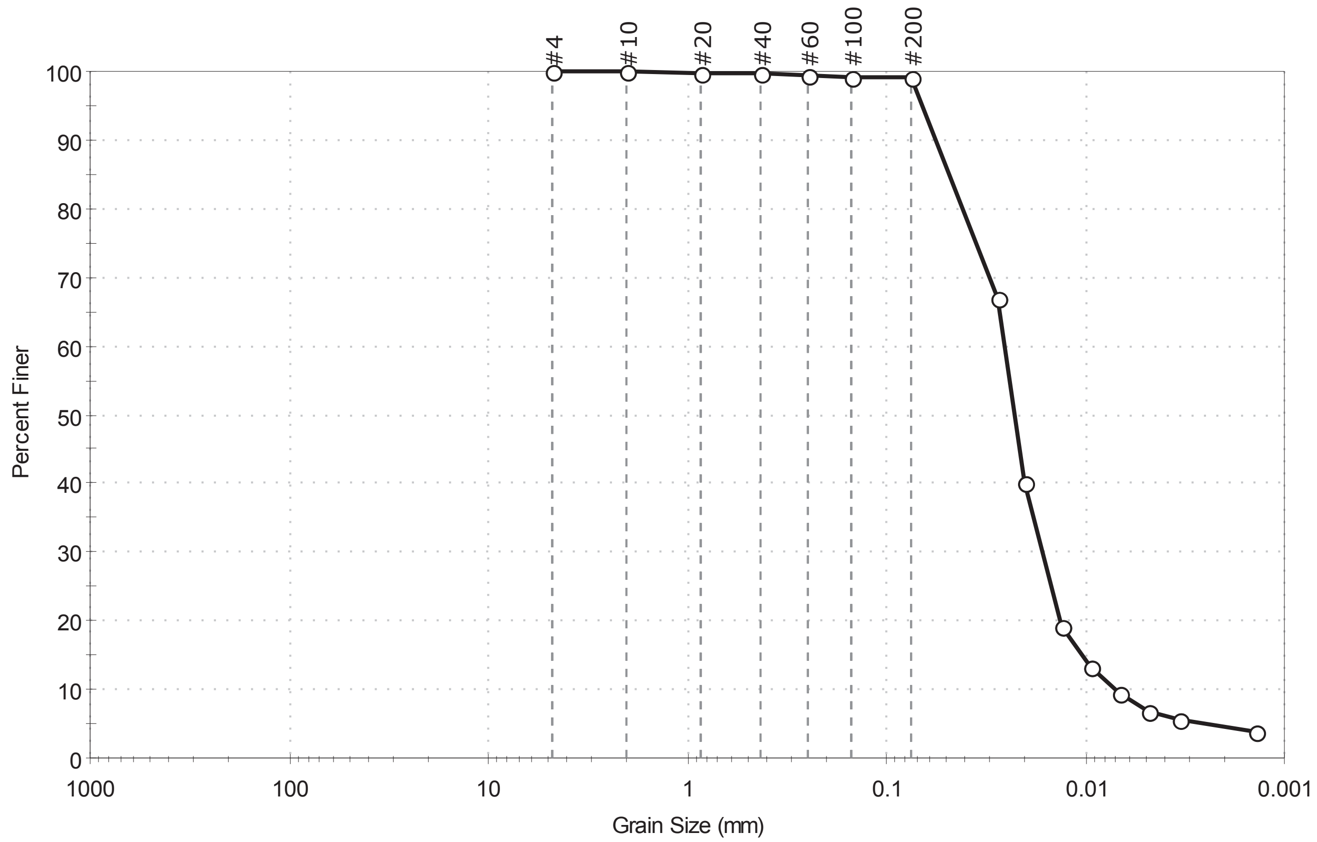
Client:	AECOM				
Project:	Vectran AB Brown Ash Pond Lower Dam				
Location:	Evansville, IN			Project No:	GTX-303915
Boring ID:	AECOM-B2	Sample Type:	tube	Tested By:	jbr
Sample ID:	3	Test Date:	11/24/15	Checked By:	emm
Depth :	56-58	Test Id:	354624		

Test Comment: ---
Visual Description: Moist, brown silt
Sample Comment: ---

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.0	0.9	99.1

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
#4	4.75	100		
#10	2.00	100		
#20	0.85	100		
#40	0.42	100		
#60	0.25	99		
#100	0.15	99		
#200	0.075	99		
---	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0277	67		
---	0.0201	40		
---	0.0130	19		
---	0.0093	13		
---	0.0067	10		
---	0.0048	7		
---	0.0034	6		
---	0.0014	4		

Coefficients

D_{85} =0.0484 mm	D_{30} =0.0163 mm
D_{60} =0.0255 mm	D_{15} =0.0102 mm
D_{50} =0.0226 mm	D_{10} =0.0069 mm
C_u =3.696	C_c =1.510

Classification

ASTM	Silt (ML)
AASHTO	Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

Dispersion Device : Apparatus A - Mech Mixer

Dispersion Period : 1 minute

Specific Gravity : 2.65

Separation of Sample: #200 Sieve

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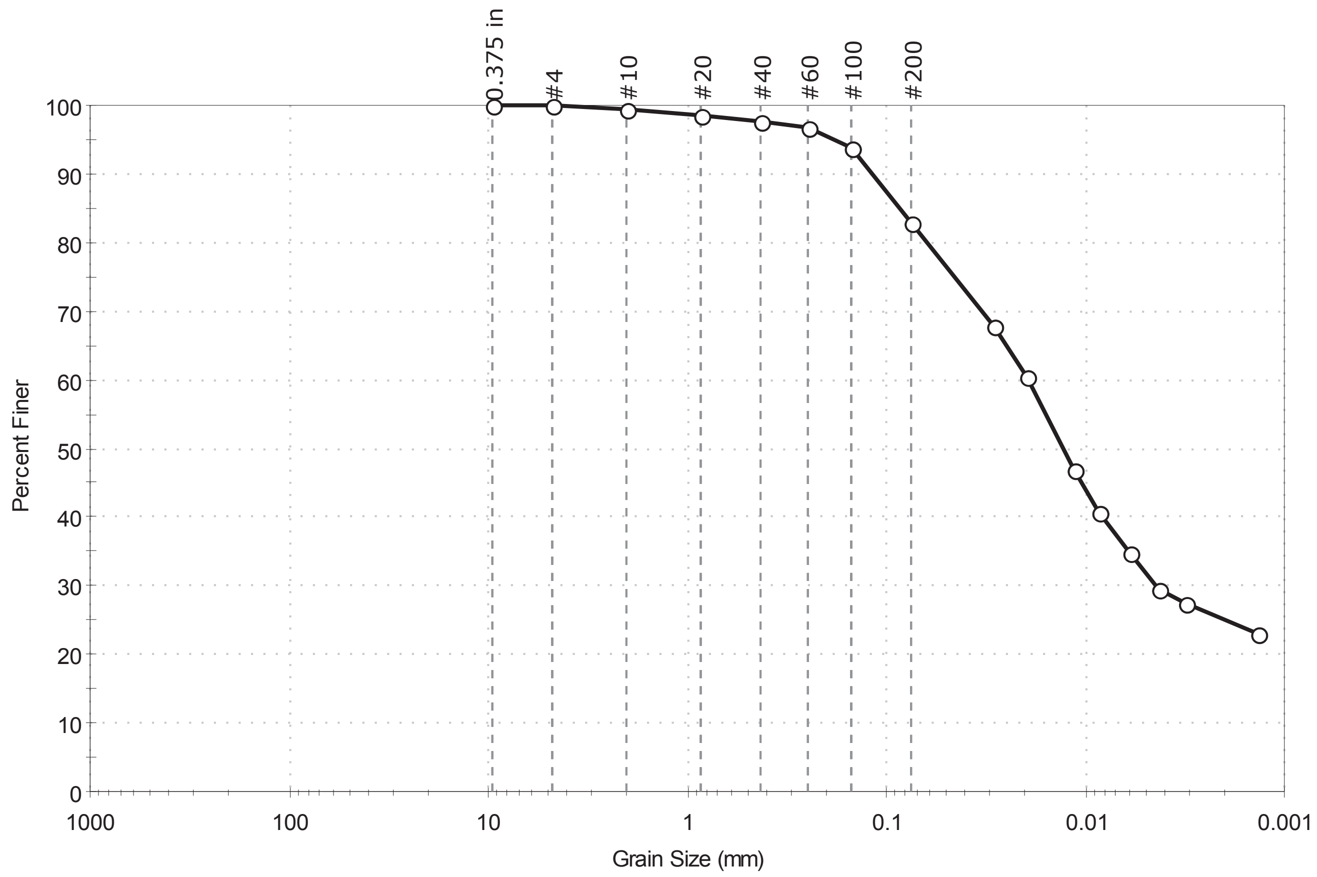
Client: AECOM
Project: Vectran AB Brown Ash Pond Lower Dam
Location: Evansville, IN
Project No: GTX-303915
Boring ID: AECOM-B4
Sample Type: tube
Tested By: GA
Sample ID: 1
Test Date: 12/14/15
Checked By: emm
Depth : 12-14
Test Id: 354618
Test Comment: ---
Visual Description: Moist, yellowish brown clay with sand
Sample Comment: ---

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.1	17.0	82.9

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.375 in	9.50	100		
#4	4.75	100		
#10	2.00	99		
#20	0.85	98		
#40	0.42	98		
#60	0.25	97		
#100	0.15	94		
#200	0.075	83		
---	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0290	68		
---	0.0198	60		
---	0.0115	47		
---	0.0085	41		
---	0.0060	35		
---	0.0042	29		
---	0.0031	27		
---	0.0014	23		

Coefficients

D_{85} =0.0856 mm
D_{30} =0.0044 mm
D_{60} =0.0194 mm
D_{15} =N/A
D_{50} =0.0130 mm
D_{10} =N/A
C_u =N/A
C_c =N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ---
Sand/Gravel Hardness : ---
Dispersion Device : Apparatus A - Mech Mixer
Dispersion Period : 1 minute
Specific Gravity : 2.65
Separation of Sample: #200 Sieve

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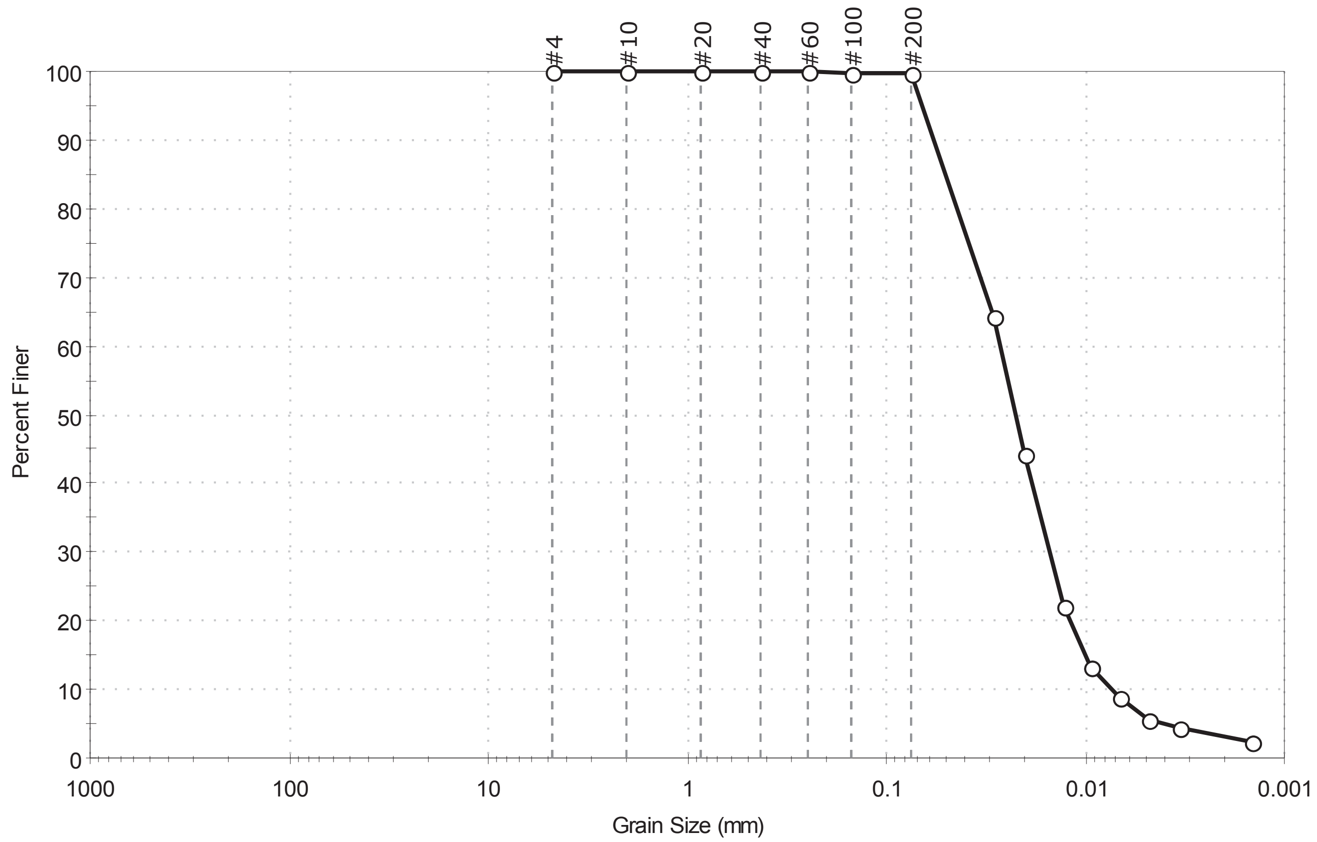
GeoTesting EXPRESS

Client: AECOM
Project: Vectran AB Brown Ash Pond Lower Dam
Location: Evansville, IN
Project No: GTX-303915
Boring ID: AECOM-B4
Sample Type: tube
Tested By: jbr
Sample ID: 2
Test Date: 11/24/15
Checked By: emm
Depth : 33-35
Test Id: 354619
Test Comment: ---
Visual Description: Wet, olive silt
Sample Comment: ---

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.0	0.3	99.7

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
#4	4.75	100		
#10	2.00	100		
#20	0.85	100		
#40	0.42	100		
#60	0.25	100		
#100	0.15	100		
#200	0.075	100		
---	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0286	64		
---	0.0202	44		
---	0.0129	22		
---	0.0094	13		
---	0.0067	9		
---	0.0048	6		
---	0.0034	4		
---	0.0015	2		

Coefficients

D_{85} =0.0503 mm	D_{30} =0.0151 mm
D_{60} =0.0266 mm	D_{15} =0.0100 mm
D_{50} =0.0224 mm	D_{10} =0.0074 mm
C_u =3.595	C_c =1.158

Classification

ASTM	Silt (ML)
AASHTO	Silty Soils (A-4 (3))

Sample/Test Description

Sand/Gravel Particle Shape : ---
Sand/Gravel Hardness : ---
Dispersion Device : Apparatus A - Mech Mixer
Dispersion Period : 1 minute
Specific Gravity : 2.65
Separation of Sample: #200 Sieve

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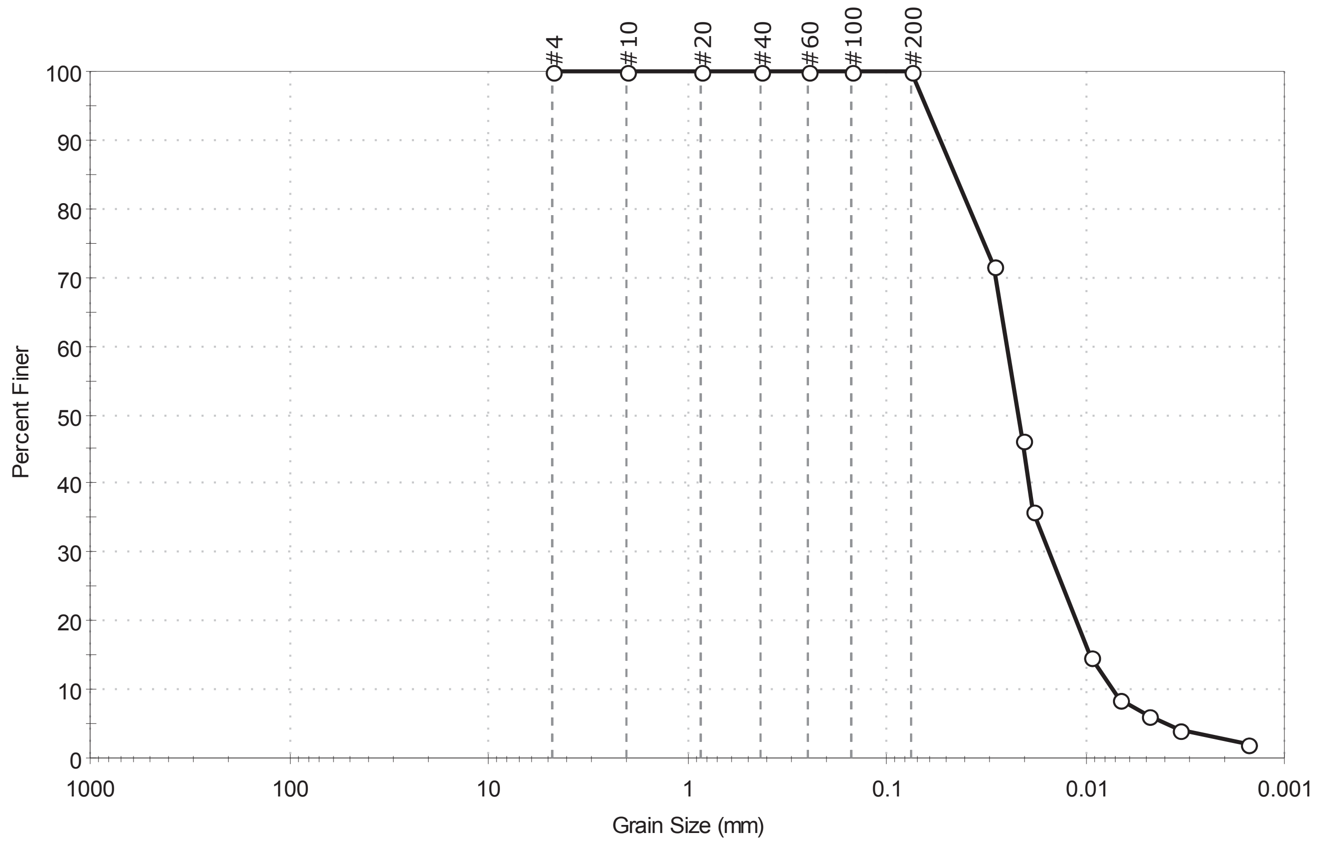
Client: AECOM
Project: Vectran AB Brown Ash Pond Lower Dam
Location: Evansville, IN
Project No: GTX-303915

Boring ID: AECOM-B4
Sample Type: tube
Tested By: jbr
Sample ID: 3
Test Date: 11/25/15
Checked By: emm
Depth : 46-48
Test Id: 354620

Test Comment: ---
Visual Description: Moist, olive silt
Sample Comment: ---

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.0	0.1	99.9

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
#4	4.75	100		
#10	2.00	100		
#20	0.85	100		
#40	0.42	100		
#60	0.25	100		
#100	0.15	100		
#200	0.075	100		
---	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0292	72		
---	0.0208	46		
---	0.0184	36		
---	0.0094	15		
---	0.0067	8		
---	0.0048	6		
---	0.0034	4		
---	0.0015	2		

Coefficients

D_{85} =0.0455 mm	D_{30} =0.0152 mm
D_{60} =0.0250 mm	D_{15} =0.0094 mm
D_{50} =0.0219 mm	D_{10} =0.0073 mm
C_u =3.425	C_c =1.266

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ---
Sand/Gravel Hardness : ---
Dispersion Device : Apparatus A - Mech Mixer
Dispersion Period : 1 minute
Specific Gravity : 2.65
Separation of Sample: #200 Sieve

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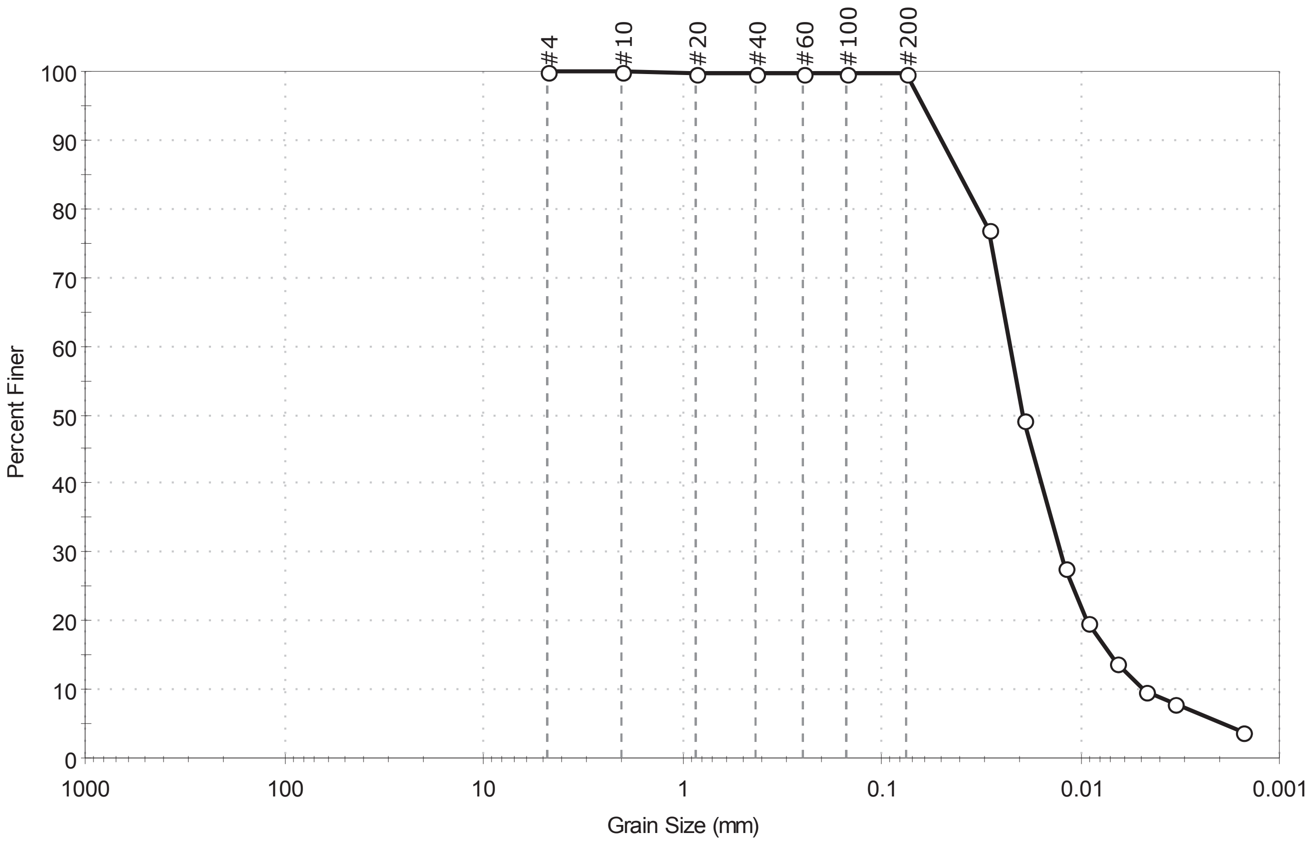
GeoTesting EXPRESS

Client: AECOM
Project: Vectran AB Brown Ash Pond Lower Dam
Location: Evansville, IN
Project No: GTX-303915
Boring ID: AECOM-B2
Sample Type: tube
Tested By: jbr
Sample ID: 4A
Test Date: 11/25/15
Checked By: emm
Depth : 62-64
Test Id: 354625
Test Comment: ---
Visual Description: Moist, gray silt
Sample Comment: ---

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.0	0.3	99.7

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
#4	4.75	100		
#10	2.00	100		
#20	0.85	100		
#40	0.42	100		
#60	0.25	100		
#100	0.15	100		
#200	0.075	100		
---	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0287	77		
---	0.0193	49		
---	0.0119	28		
---	0.0092	20		
---	0.0066	14		
---	0.0047	10		
---	0.0034	8		
---	0.0015	4		

Coefficients

D_{85} =0.0404 mm
D_{30} =0.0126 mm
D_{60} =0.0225 mm
D_{15} =0.0071 mm
D_{50} =0.0195 mm
D_{10} =0.0047 mm
C_u =4.787
C_c =1.501

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ---
Sand/Gravel Hardness : ---
Dispersion Device : Apparatus A - Mech Mixer
Dispersion Period : 1 minute
Specific Gravity : 2.65
Separation of Sample: #200 Sieve

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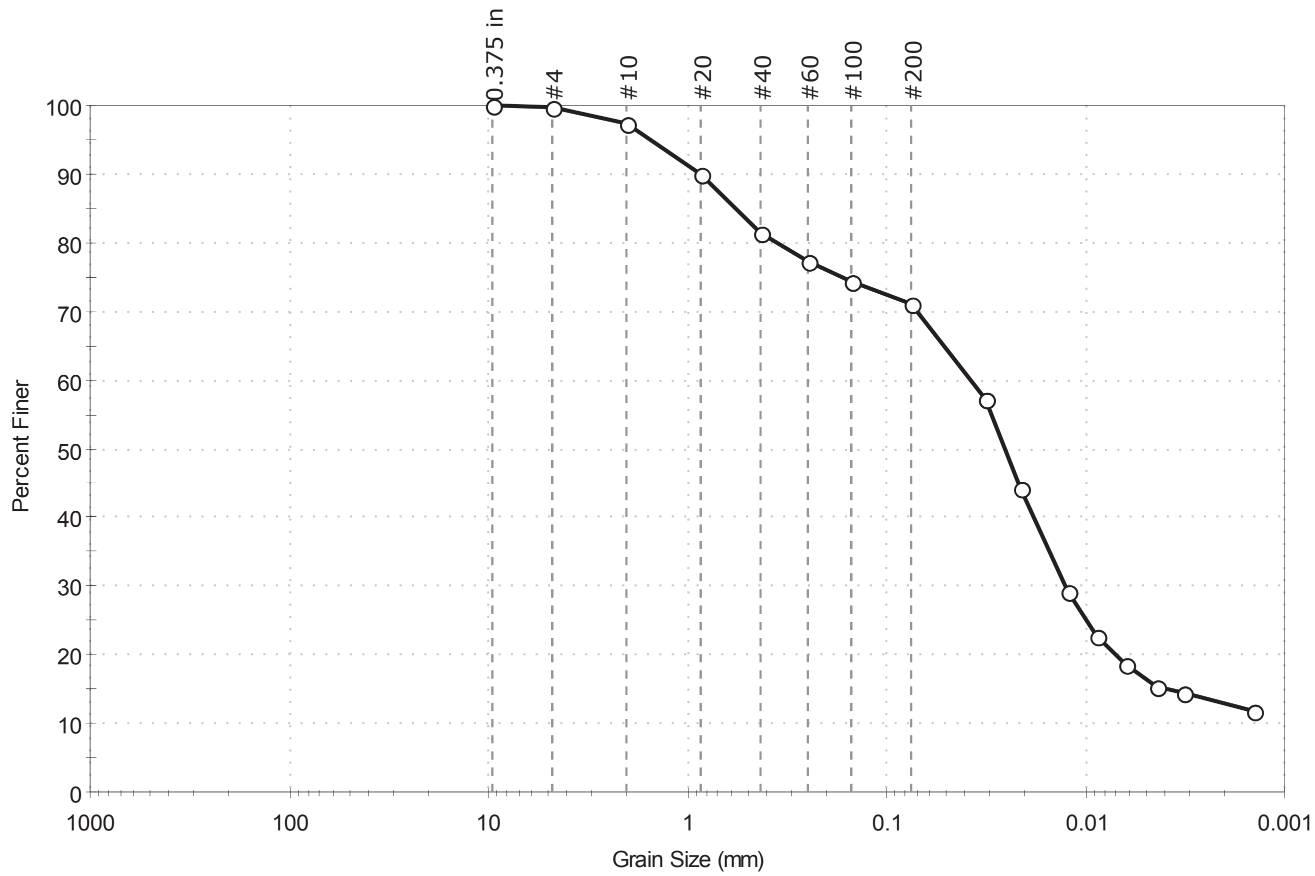
Client:	AECOM				
Project:	Vectran AB Brown Ash Pond Lower Dam				
Location:	Evansville, IN			Project No:	GTX-303915
Boring ID:	AECOM-B5	Sample Type:	tube	Tested By:	GA
Sample ID:	2	Test Date:	12/14/15	Checked By:	emm
Depth :	30-32	Test Id:	354995		
Test Comment:	---				
Visual Description:	Moist, gray silt with sand				
Sample Comment:	---				

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.4	28.4	71.2

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.375 in	9.50	100		
#4	4.75	100		
#10	2.00	97		
#20	0.85	90		
#40	0.42	81		
#60	0.25	77		
#100	0.15	74		
#200	0.075	71		
---	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0321	57		
---	0.0210	44		
---	0.0122	29		
---	0.0089	23		
---	0.0062	19		
---	0.0044	15		
---	0.0032	14		
---	0.0014	12		

Coefficients

D_{85} =0.5703 mm	D_{30} =0.0126 mm
D_{60} =0.0378 mm	D_{15} =0.0039 mm
D_{50} =0.0253 mm	D_{10} =N/A
C_u =N/A	C_c =N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

Dispersion Device : Apparatus A - Mech Mixer

Dispersion Period : 1 minute

Specific Gravity : 2.65

Separation of Sample: #200 Sieve

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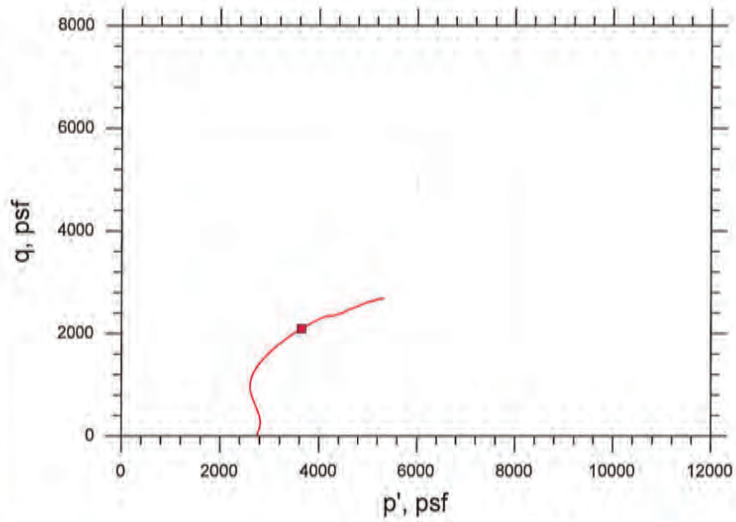
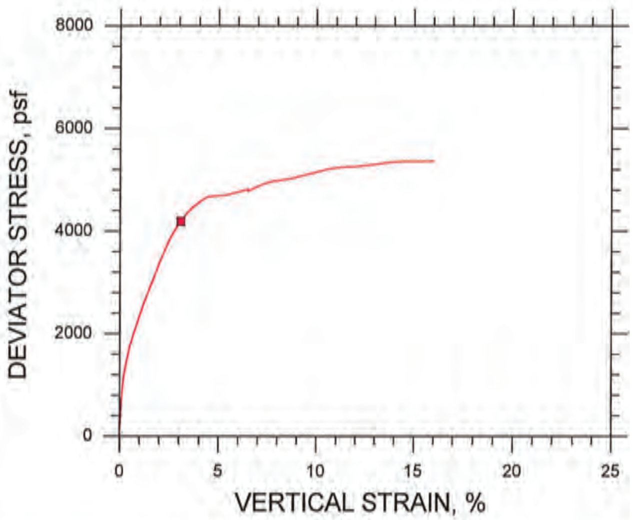
Soil Consolidated Undrained Triaxial Laboratory Test Results

Client: AECOM	
Project Name: Vectran AB Brown Ash Pond	
Project Location: Evansville, IN	
Project Number: GTX-303915	
Tested By: md	Checked By: jdt
Boring ID: AECOM-B1	
Preparation: intact	
Description: Moist, yellow and brown silt with red clay	
Classification: ---	
Group Symbol: ---	
Liquid Limit: ---	Plastic Limit: ---
Plasticity Index: ---	Estimated Specific Gravity: 2.7

CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767





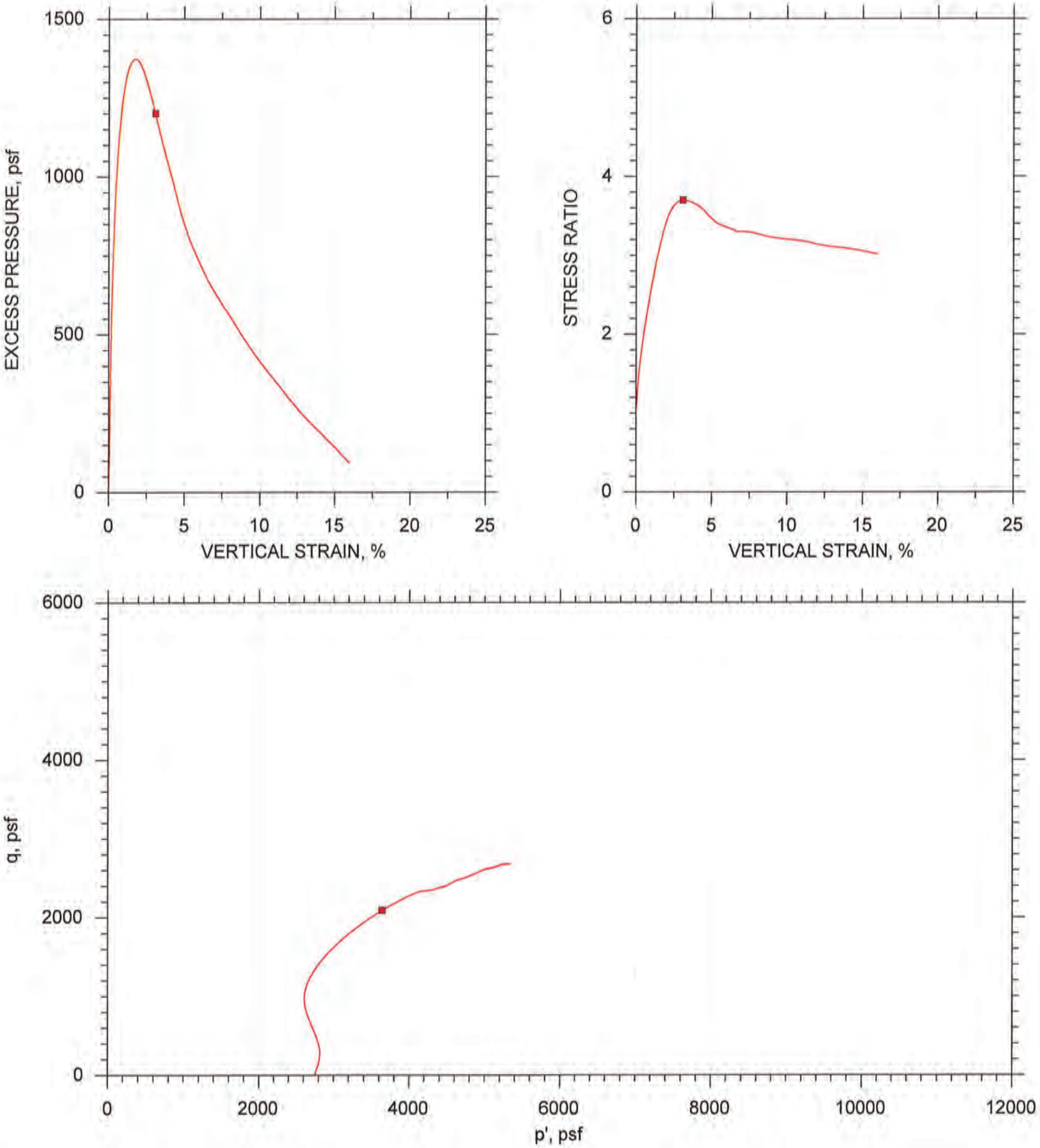
Symbol		🟥			
Sample ID		1A			
Depth, ft		19-21 ft			
Test Number		CU-2-1			
Initial	Height, in	6.200			
	Diameter, in	2.870			
	Moisture Content (from Cuttings), %	17.0			
	Dry Density, pcf	112.			
	Saturation (Wet Method), %	90.9			
	Void Ratio	0.506			
Before Shear	Moisture Content, %	17.2			
	Dry Density, pcf	115.			
	Cross-sectional Area (Method A), in²	6.367			
	Saturation, %	100.0			
	Void Ratio	0.465			
	Back Pressure, psf	1.281e+004			
Vertical Effective Consolidation Stress, psf		2742.			
Horizontal Effective Consolidation Stress, psf		2751.			
Vertical Strain after Consolidation, %		1.085			
Volumetric Strain after Consolidation, %		2.516			
Time to 50% Consolidation, min		12.25			
Shear Strength, psf		2093.			
Strain at Failure, %		3.08			
Strain Rate, %/min		0.01600			
Deviator Stress at Failure, psf		4186.			
Effective Minor Principal Stress at Failure, psf		1550.			
Effective Major Principal Stress at Failure, psf		5736.			
B-Value		0.95			

Notes:
- Before Shear Saturation set to 100% for phase calculation.
- Moisture Content determined by ASTM D2216.
- Deviator Stress includes membrane correction.
- Values for c and φ determined from best-fit straight line for the specific test conditions. Actual strength parameters may vary and should be determined by an engineer for site conditions.

Remarks:

System F

CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767



	Sample No.	Test No.	Depth	Tested By	Test Date	Checked By	Check Date	Test File
🟥	1A	CU-2-1	19-21 ft	md	11/11/15	jdt	11/17/15	303915-CU-2-1n.dat

GeoTesting EXPRESS

Project: Vectran AB Brown Ash Pond	Location: Evansville, IN	Project No.: GTX-303915
Boring No.: AECOM-B1	Sample Type: intact	

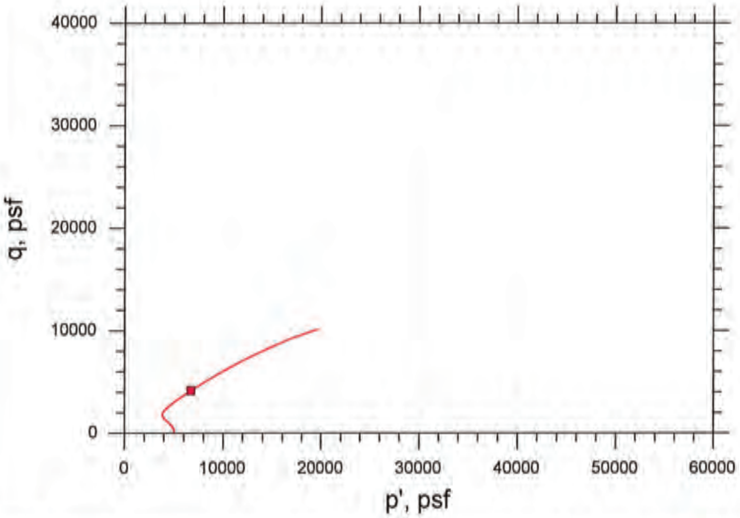
Description: Moist, yellow and brown silt with red clay

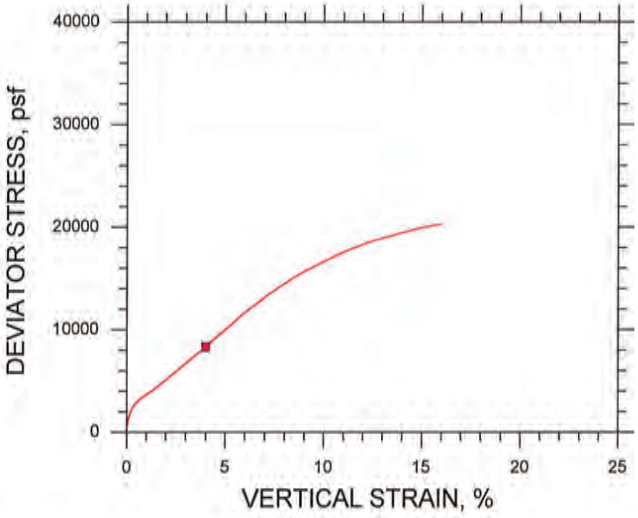
Remarks: System F

GeoTesting
EXPRESS

Client: AECOM
Project Name: Vectran AB Brown Ash Pond
Project Location: Evansville, IN
Project Number: GTX-303915
Tested By: md
Checked By: jdt
Boring ID: AECOM-B1
Preparation: intact
Description: Moist, brown silty clay and sand
Classification: ---
Group Symbol: ---
Liquid Limit: ---
Plastic Limit: ---
Plasticity Index: ---
Estimated Specific Gravity: 2.65

CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767

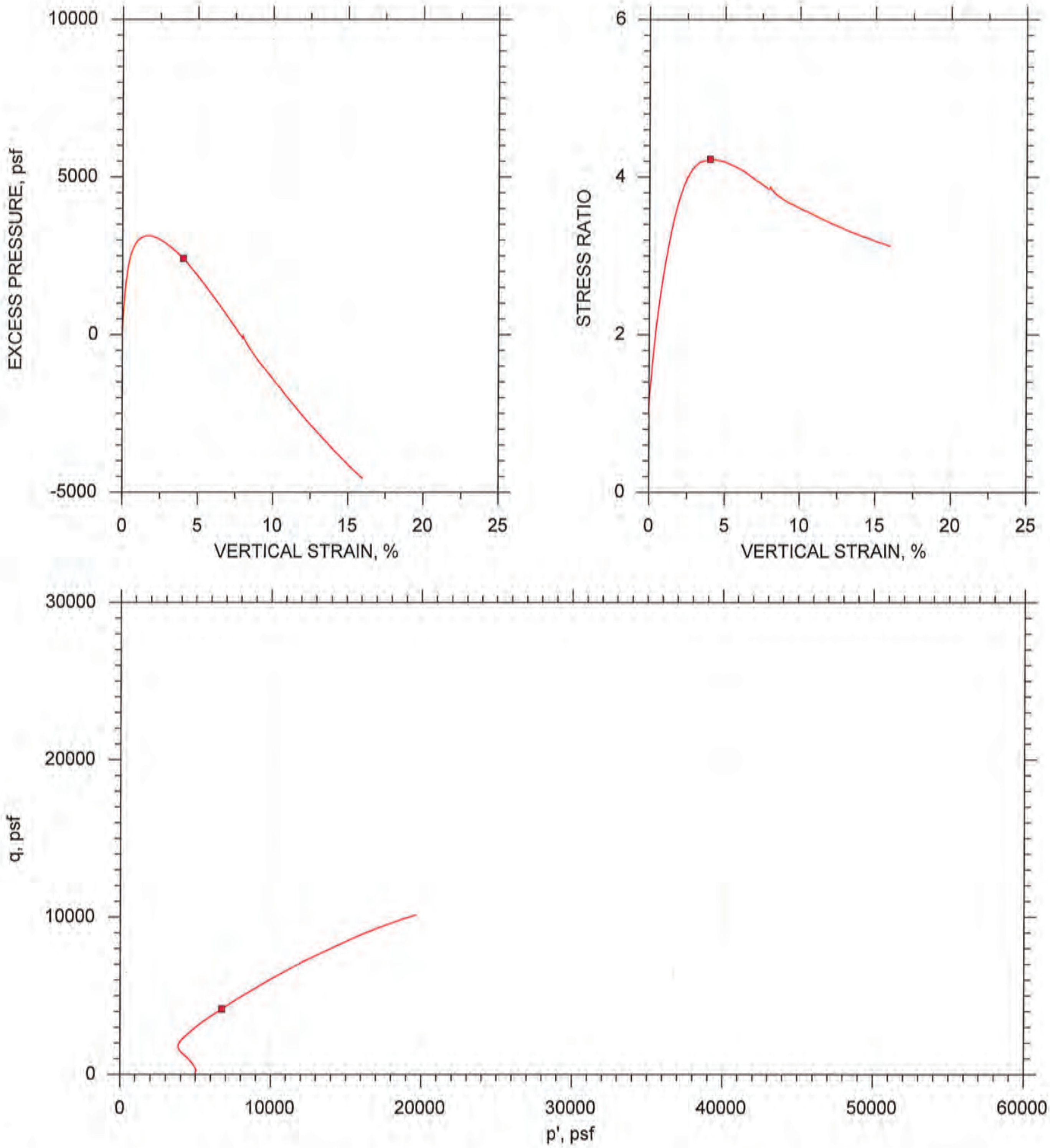




Symbol		🟥			
Sample ID		2			
Depth, ft		27-29 ft			
Test Number		CU-1-1			
Initial	Height, in	6.150			
	Diameter, in	2.870			
	Moisture Content (from Cuttings), %	16.3			
	Dry Density, pcf	111.			
	Saturation (Wet Method), %	87.4			
	Void Ratio	0.494			
Before Shear	Moisture Content, %	17.0			
	Dry Density, pcf	114.			
	Cross-sectional Area (Method A), in²	6.352			
	Saturation, %	100.0			
	Void Ratio	0.449			
	Back Pressure, psf	1.944e+004			
Vertical Effective Consolidation Stress, psf		4991.			
Horizontal Effective Consolidation Stress, psf		4997.			
Vertical Strain after Consolidation, %		1.065			
Volumetric Strain after Consolidation, %		2.671			
Time to 50% Consolidation, min		7.840			
Shear Strength, psf		4160.			
Strain at Failure, %		4.00			
Strain Rate, %/min		0.01600			
Deviator Stress at Failure, psf		8319.			
Effective Minor Principal Stress at Failure, psf		2578.			
Effective Major Principal Stress at Failure, psf		1.090e+004			
B-Value		0.95			
Notes: - Before Shear Saturation set to 100% for phase calculation. - Moisture Content determined by ASTM D2216. - Deviator Stress includes membrane correction. - Values for c and φ determined from best-fit straight line for the specific test conditions. Actual strength parameters may vary and should be determined by an engineer for site conditions. Remarks:					

System E

CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767



	Sample No.	Test No.	Depth	Tested By	Test Date	Checked By	Check Date	Test File
🟥	2	CU-1-1	27-29 ft	md	11/11/15	jdt	11/17/15	303915-CU-1-1n.dat

GeoTesting EXPRESS

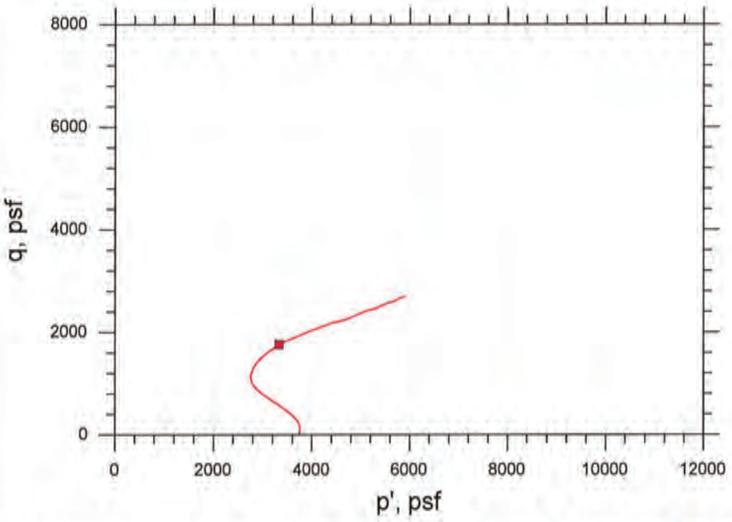
Project: Vectran AB Brown Ash Pond	Location: Evansville, IN	Project No.: GTX-303915
Boring No.: AECOM-B1	Sample Type: intact	

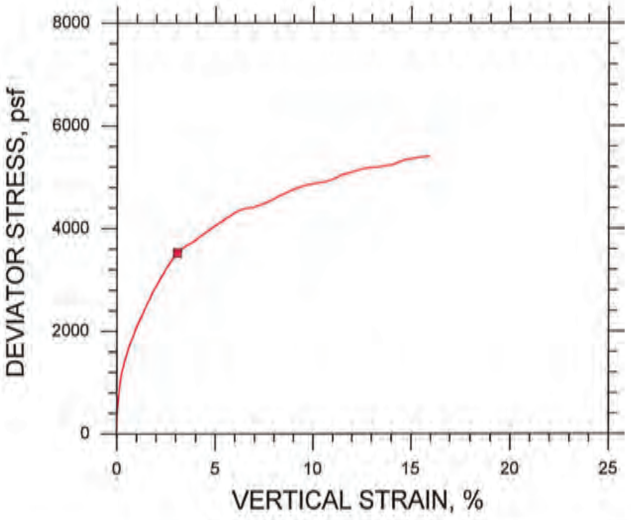
Description: Moist, brown silty clay and sand

Remarks: System E

Client: AECOM
Project Name: Vectran AB Brown Ash Pond
Project Location: Evansville, IN
Project Number: GTX-303915
Tested By: md
Checked By: jdt
Boring ID: AECOM-B2
Preparation: intact
Description: Moist, reddish brown sandy clay
Classification: ---
Group Symbol: ---
Liquid Limit: ---
Plastic Limit: ---
Plasticity Index: ---
Estimated Specific Gravity: 2.7

CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767

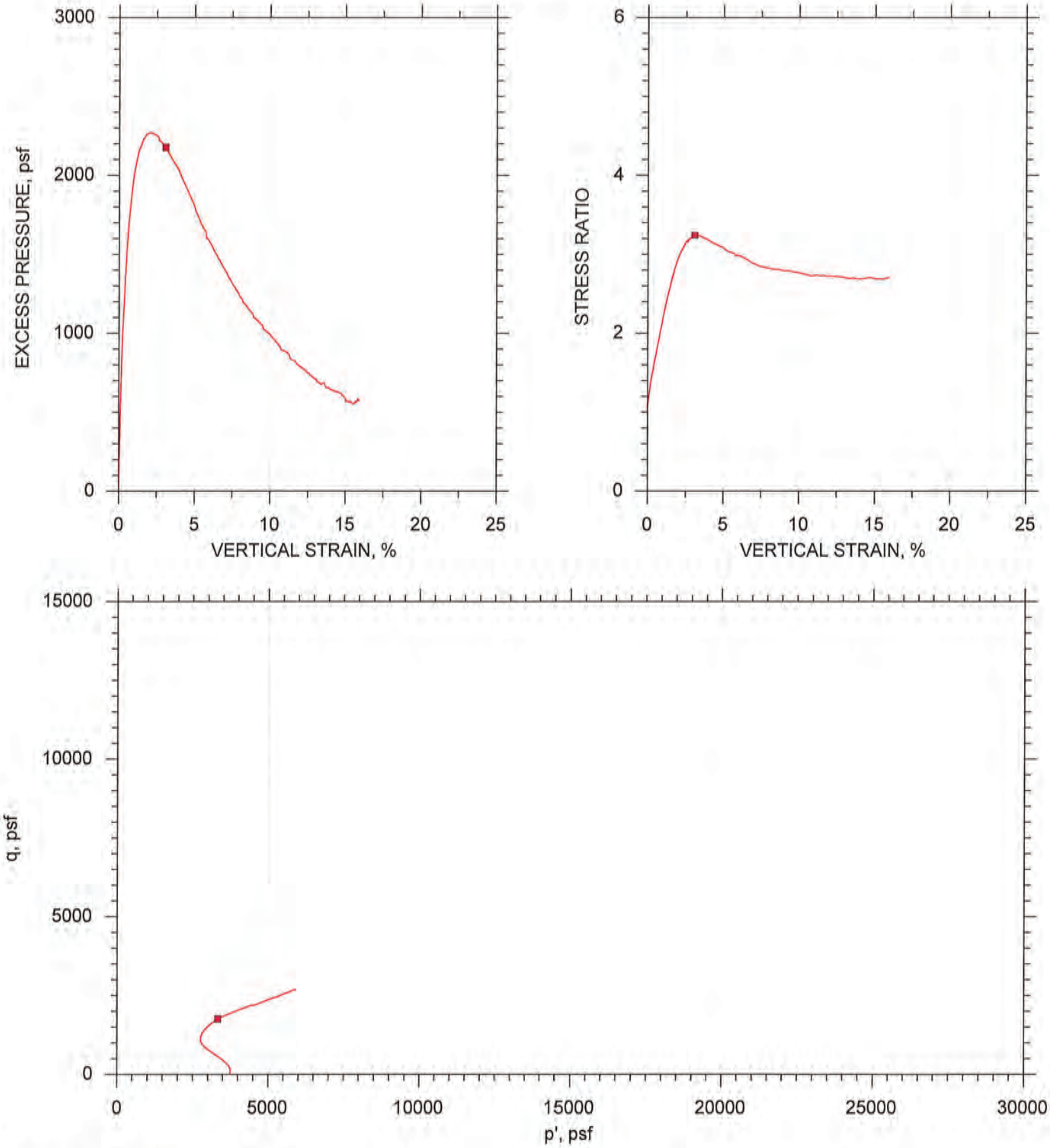




Symbol		🟥			
Sample ID		1			
Depth, ft		30-32 ft			
Test Number		CU-3-1			
Initial	Height, in	6.200			
	Diameter, in	2.870			
	Moisture Content (from Cuttings), %	17.9			
	Dry Density, pcf	108.			
	Saturation (Wet Method), %	87.1			
	Void Ratio	0.556			
Before Shear	Moisture Content, %	19.2			
	Dry Density, pcf	111.			
	Cross-sectional Area (Method A), in²	6.377			
	Saturation, %	100.0			
	Void Ratio	0.517			
	Back Pressure, psf	2.083e+004			
Vertical Effective Consolidation Stress, psf		3744.			
Horizontal Effective Consolidation Stress, psf		3750.			
Vertical Strain after Consolidation, %		0.8401			
Volumetric Strain after Consolidation, %		1.803			
Time to 50% Consolidation, min		56.25			
Shear Strength, psf		1763.			
Strain at Failure, %		3.08			
Strain Rate, %/min		0.01600			
Deviator Stress at Failure, psf		3526.			
Effective Minor Principal Stress at Failure, psf		1571.			
Effective Major Principal Stress at Failure, psf		5097.			
B-Value		0.95			
Notes: - Before Shear Saturation set to 100% for phase calculation. - Moisture Content determined by ASTM D2216. - Deviator Stress includes membrane correction. - Values for c and φ determined from best-fit straight line for the specific test conditions. Actual strength parameters may vary and should be determined by an engineer for site conditions. Remarks:					

System X

CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767



	Sample No.	Test No.	Depth	Tested By	Test Date	Checked By	Check Date	Test File
🟥	1	CU-3-1	30-32 ft	md	11/11/15	jdt	11/17/15	303915-CU-3-1n.dat

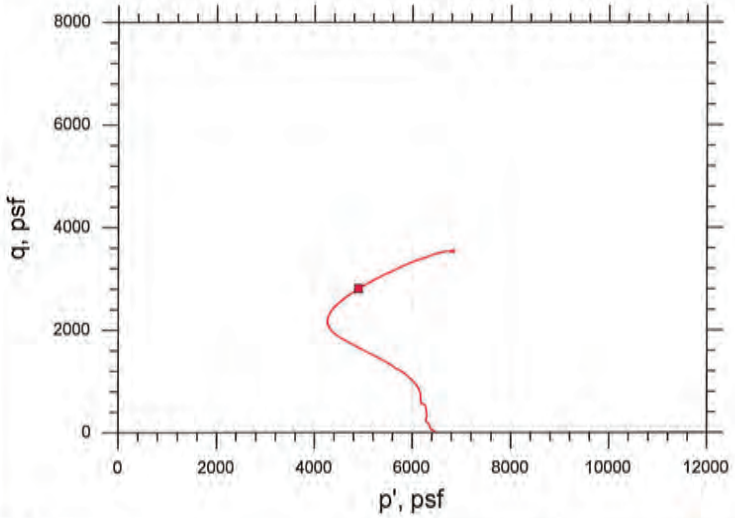
GeoTesting EXPRESS

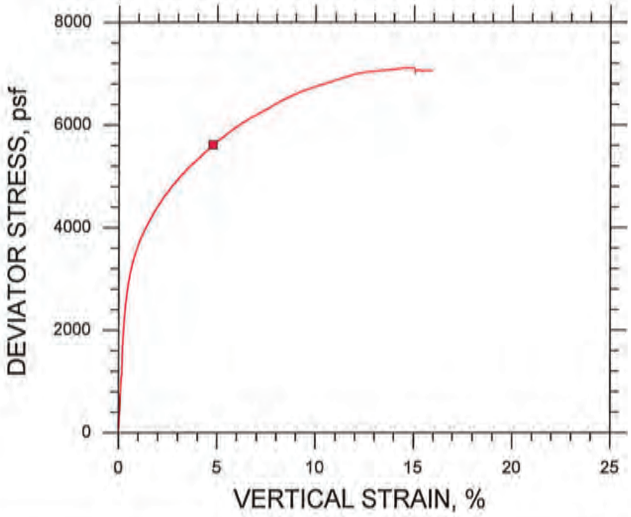
Project: Vectran AB Brown Ash Pond
Location: Evansville, IN
Project No.: GTX-303915
Boring No.: AECOM-B2
Sample Type: intact
Description: Moist, reddish brown sandy clay
Remarks: System X

GeoTesting EXPRESS

Client: AECOM	
Project Name: Vectran AB Brown Ash Pond	
Project Location: Evansville, IN	
Project Number: GTX-303915	
Tested By: md	Checked By: jdt
Boring ID: AECOM-B2	
Preparation: intact	
Description: Moist, reddish brown clay with sand	
Classification: ---	
Group Symbol: ---	
Liquid Limit: ---	Plastic Limit: ---
Plasticity Index: ---	Estimated Specific Gravity: 2.7

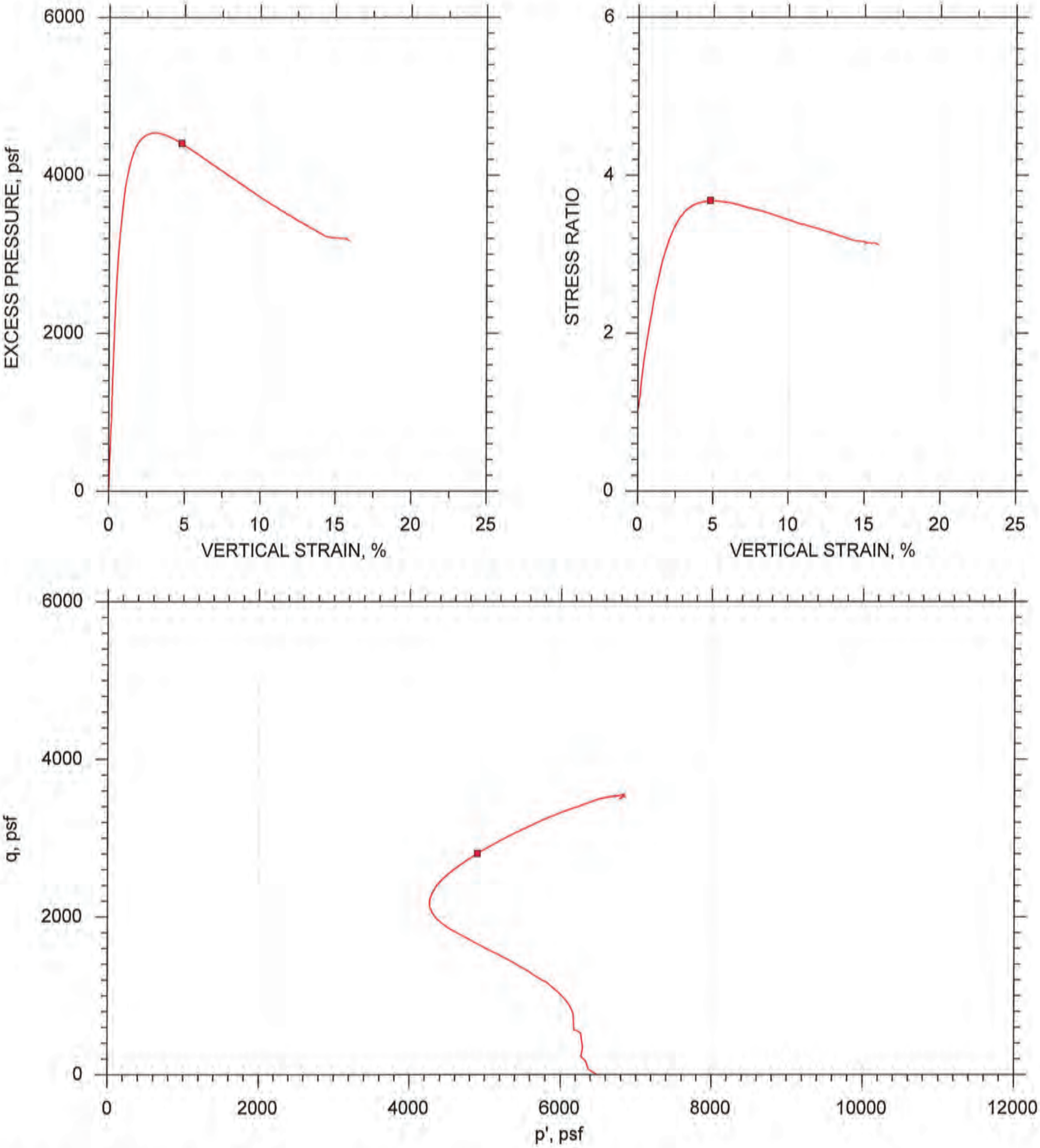
CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767





Symbol		🟥			
Sample ID		2			
Depth, ft		48-50 ft			
Test Number		CU-5-1			
Initial	Height, in	6.180			
	Diameter, in	2.870			
	Moisture Content (from Cuttings), %	15.2			
	Dry Density, pcf	111.			
	Saturation (Wet Method), %	80.2			
	Void Ratio	0.512			
Before Shear	Moisture Content, %	16.1			
	Dry Density, pcf	117.			
	Cross-sectional Area (Method A), in²	6.262			
	Saturation, %	100.0			
	Void Ratio	0.435			
	Back Pressure, psf	1.858e+004			
Vertical Effective Consolidation Stress, psf		6483.			
Horizontal Effective Consolidation Stress, psf		6497.			
Vertical Strain after Consolidation, %		1.739			
Volumetric Strain after Consolidation, %		4.478			
Time to 50% Consolidation, min		100.0			
Shear Strength, psf		2806.			
Strain at Failure, %		4.78			
Strain Rate, %/min		0.01600			
Deviator Stress at Failure, psf		5612.			
Effective Minor Principal Stress at Failure, psf		2092.			
Effective Major Principal Stress at Failure, psf		7703.			
B-Value		0.95			
Notes: - Before Shear Saturation set to 100% for phase calculation. - Moisture Content determined by ASTM D2216. - Deviator Stress includes membrane correction. - Values for c and φ determined from best-fit straight line for the specific test conditions. Actual strength parameters may vary and should be determined by an engineer for site conditions. Remarks:					
System T					

CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767



	Sample No.	Test No.	Depth	Tested By	Test Date	Checked By	Check Date	Test File
🟥	2	CU-5-1	48-50 ft	md	11/11/15	jdt	---	303915-CU-5-1n.dat

GeoTesting EXPRESS

Project: Vectran AB Brown Ash Pond	Location: Evansville, IN	Project No.: GTX-303915
Boring No.: AECOM-B2	Sample Type: intact	

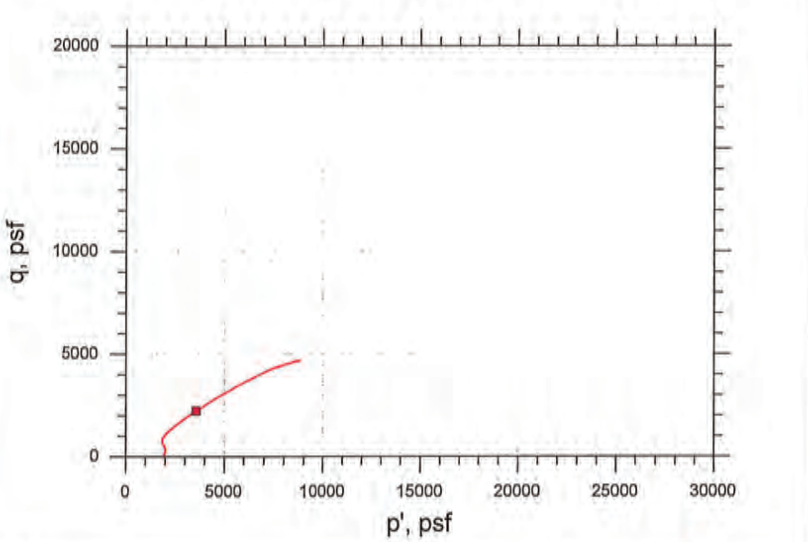
Description: Moist, reddish brown clay with sand

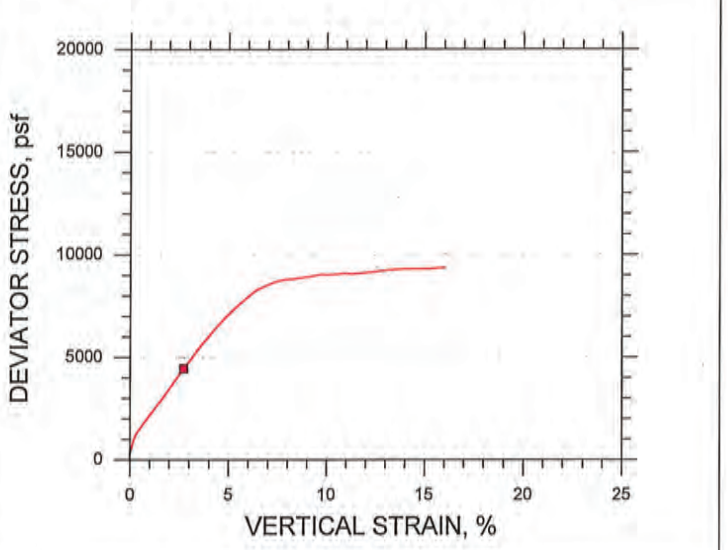
Remarks: System T

GeoTesting EXPRESS

Client: AECOM
Project Name: Vectran AB Brown Ash Pond
Project Location: Evansville, IN
Project Number: GTX-303915
Tested By: md | Checked By: jdt
Boring ID: AECOM-B4
Preparation: intact
Description: Moist, yellowish brown sandy clay
Classification: ---
Group Symbol: ---
Liquid Limit: --- | Plastic Limit: ---
Plasticity Index: --- | Estimated Specific Gravity: 2.7

CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767

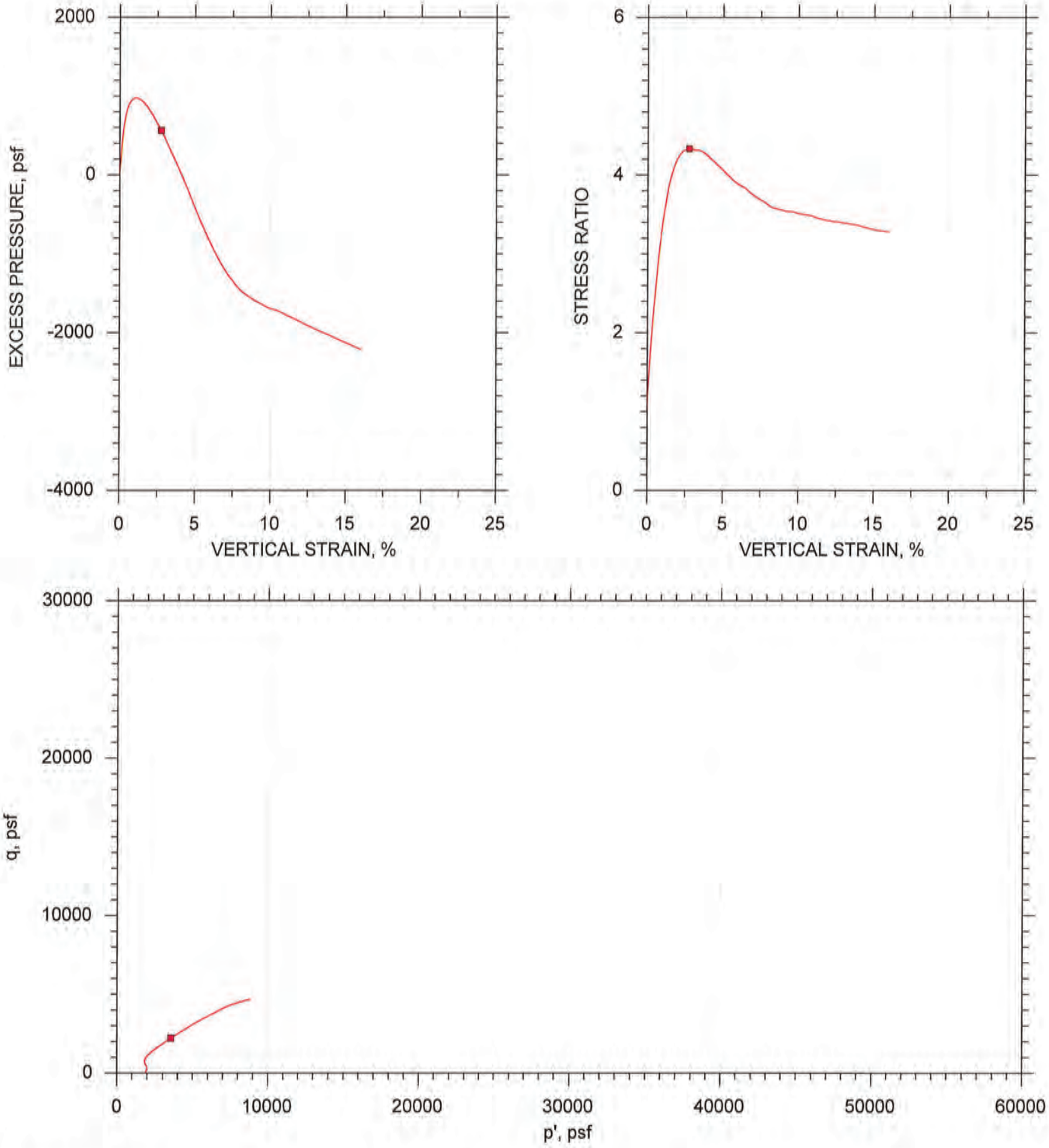




Symbol		🟥			
Sample ID		1			
Depth, ft		12-14 ft			
Test Number		CU-4-1			
Initial	Height, in	6.300			
	Diameter, in	2.870			
	Moisture Content (from Cuttings), %	16.4			
	Dry Density, pcf	110.			
	Saturation (Wet Method), %	82.3			
	Void Ratio	0.538			
Before Shear	Moisture Content, %	18.5			
	Dry Density, pcf	112.			
	Cross-sectional Area (Method A), in²	6.322			
	Saturation, %	100.0			
	Void Ratio	0.500			
	Back Pressure, psf	4175.			
Vertical Effective Consolidation Stress, psf		1898.			
Horizontal Effective Consolidation Stress, psf		1899.			
Vertical Strain after Consolidation, %		0.2220			
Volumetric Strain after Consolidation, %		2.550			
Time to 50% Consolidation, min		25.00			
Shear Strength, psf		2227.			
Strain at Failure, %		2.73			
Strain Rate, %/min		0.01600			
Deviator Stress at Failure, psf		4453.			
Effective Minor Principal Stress at Failure, psf		1336.			
Effective Major Principal Stress at Failure, psf		5789.			
B-Value		1.			
Notes: - Before Shear Saturation set to 100% for phase calculation. - Moisture Content determined by ASTM D2216. - Deviator Stress includes membrane correction. - Values for c and φ determined from best-fit straight line for the specific test conditions. Actual strength parameters may vary and should be determined by an engineer for site conditions. Remarks:					

System Y

CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767



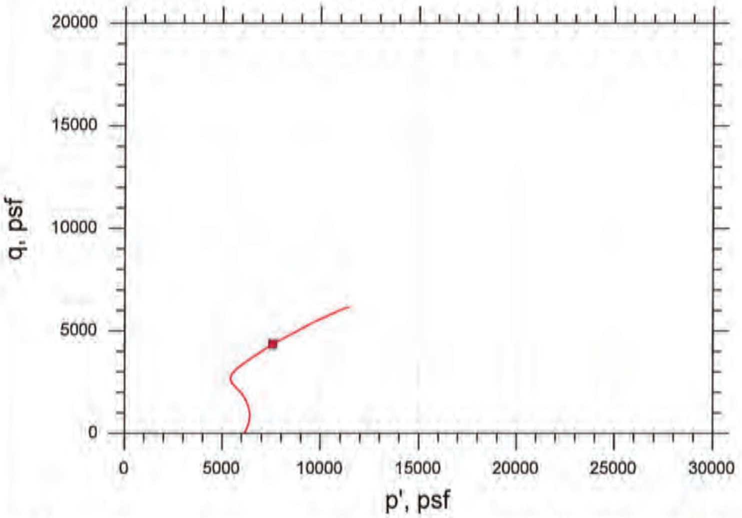
	Sample No.	Test No.	Depth	Tested By	Test Date	Checked By	Check Date	Test File
■	1	CU-4-1	12-14 ft	md	11/11/15	jdt	11/17/15	303915-CU-4-1n.dat

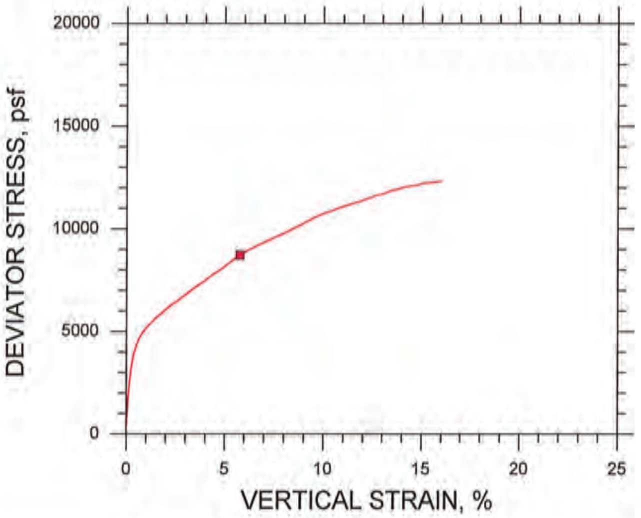
GeoTesting EXPRESS

Project: Vectran AB Brown Ash Pond	Location: Evansville, IN	Project No.: GTX-303915
Boring No.: AECOM-B4	Sample Type: intact	
Description: Moist, yellowish brown sandy clay		
Remarks: System Y		

Client: AECOM	
Project Name: Vectran AB Brown Ash Pond	
Project Location: Evansville, IN	
Project Number: GTX-303915	
Tested By: md	Checked By: njh
Boring ID: AECOM-B1	
Preparation: intact	
Description: Moist, yellowish brown silt	
Classification: ---	
Group Symbol: ---	
Liquid Limit: ---	Plastic Limit: ---
Plasticity Index: ---	Estimated Specific Gravity: 2.7

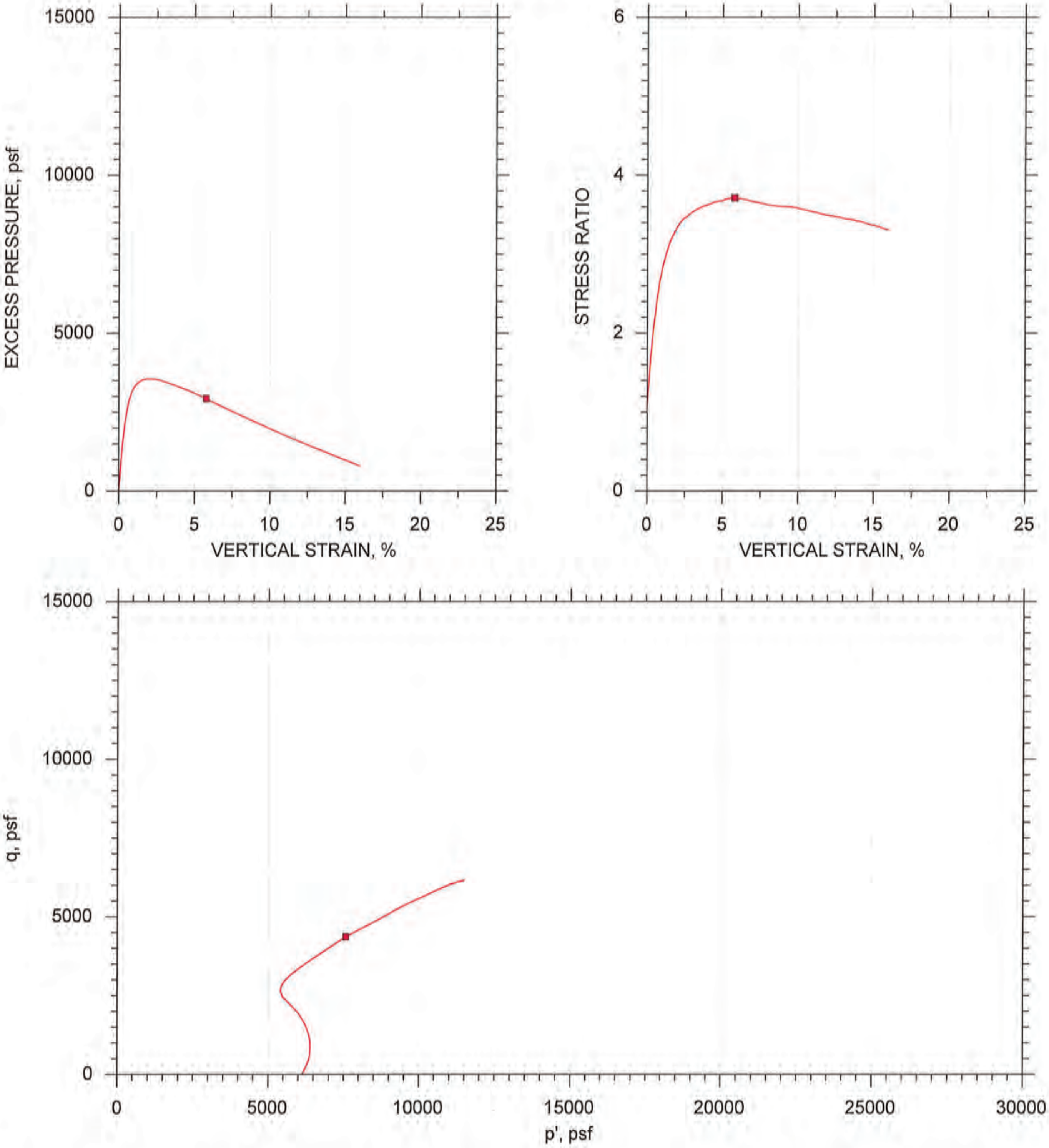
CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767





Symbol		🟥			
Sample ID		4			
Depth, ft		44-46 ft			
Test Number		CU-12-1			
Initial	Height, in	4.500			
	Diameter, in	2.010			
	Moisture Content (from Cuttings), %	26.5			
	Dry Density, pcf	98.0			
	Saturation (Wet Method), %	99.6			
	Void Ratio	0.719			
Before Shear	Moisture Content, %	24.5			
	Dry Density, pcf	101.			
	Cross-sectional Area (Method A), in²	3.090			
	Saturation, %	100.0			
	Void Ratio	0.662			
	Back Pressure, psf	2.172e+004			
Vertical Effective Consolidation Stress, psf		6142.			
Horizontal Effective Consolidation Stress, psf		6148.			
Vertical Strain after Consolidation, %		0.6199			
Volumetric Strain after Consolidation, %		2.959			
Time to 50% Consolidation, min		0.2500			
Shear Strength, psf		4361.			
Strain at Failure, %		5.78			
Strain Rate, %/min		0.01600			
Deviator Stress at Failure, psf		8722.			
Effective Minor Principal Stress at Failure, psf		3213.			
Effective Major Principal Stress at Failure, psf		1.193e+004			
B-Value		0.95			
Notes: - Before Shear Saturation set to 100% for phase calculation. - Moisture Content determined by ASTM D2216. - Deviator Stress includes membrane correction. - Values for c and φ determined from best-fit straight line for the specific test conditions. Actual strength parameters may vary and should be determined by an engineer for site conditions. Remarks:					
System W					

CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767

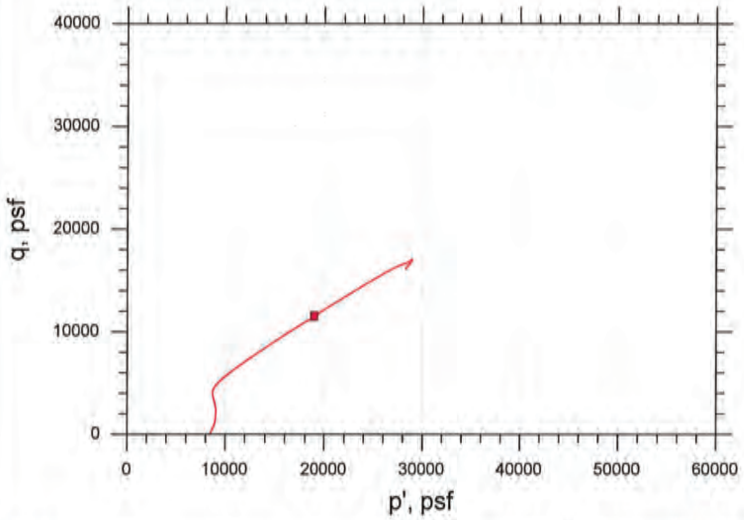


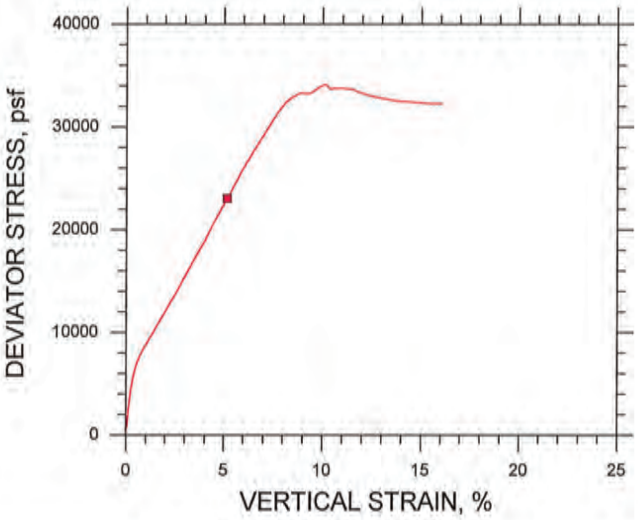
	Sample No.	Test No.	Depth	Tested By	Test Date	Checked By	Check Date	Test File
🟥	4	CU-12-1	44-46 ft	md	2/25/16	njh	3/2/16	303915-CU-12-1n.dat

Project: Vectran AB Brown Ash Pond
Location: Evansville, IN
Project No.: GTX-303915
Boring No.: AECOM-B1
Sample Type: intact
Description: Moist, yellowish brown silt
Remarks: System W

Client: AECOM	
Project Name: Vectran AB Brown Ash Pond	
Project Location: Evansville, IN	
Project Number: GTX-303915	
Tested By: md	Checked By: njh
Boring ID: AECOM-B2	
Preparation: intact	
Description: Moist, light brown silt	
Classification: ---	
Group Symbol: ---	
Liquid Limit: ---	Plastic Limit: ---
Plasticity Index: ---	Estimated Specific Gravity: 2.7

CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767

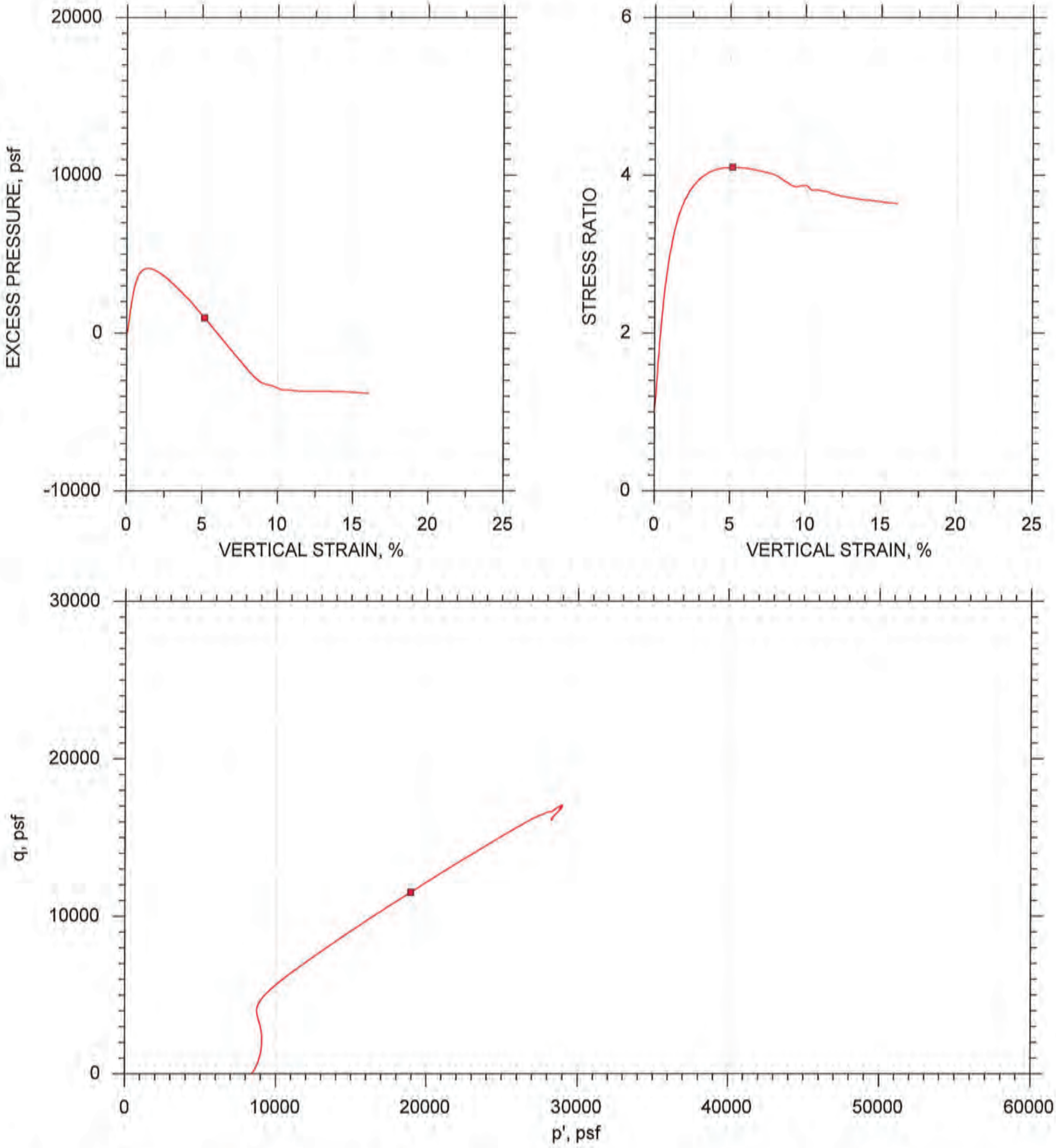




Symbol		🟥			
Sample ID		4			
Depth, ft		60-62 ft			
Test Number		CU-8-1			
Initial	Height, in	4.650			
	Diameter, in	2.040			
	Moisture Content (from Cuttings), %	25.9			
	Dry Density, pcf	98.3			
	Saturation (Wet Method), %	97.9			
	Void Ratio	0.715			
Before Shear	Moisture Content, %	24.9			
	Dry Density, pcf	101.			
	Cross-sectional Area (Method A), in²	3.196			
	Saturation, %	100.0			
	Void Ratio	0.673			
	Back Pressure, psf	2.171e+004			
Vertical Effective Consolidation Stress, psf		8396.			
Horizontal Effective Consolidation Stress, psf		8397.			
Vertical Strain after Consolidation, %		0.4461			
Volumetric Strain after Consolidation, %		3.018			
Time to 50% Consolidation, min		0.6400			
Shear Strength, psf		1.152e+004			
Strain at Failure, %		5.15			
Strain Rate, %/min		0.01600			
Deviator Stress at Failure, psf		2.305e+004			
Effective Minor Principal Stress at Failure, psf		7427.			
Effective Major Principal Stress at Failure, psf		3.048e+004			
B-Value		0.95			
Notes: - Before Shear Saturation set to 100% for phase calculation. - Moisture Content determined by ASTM D2216. - Deviator Stress includes membrane correction. - Values for c and φ determined from best-fit straight line for the specific test conditions. Actual strength parameters may vary and should be determined by an engineer for site conditions. Remarks:					

System Q

CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767



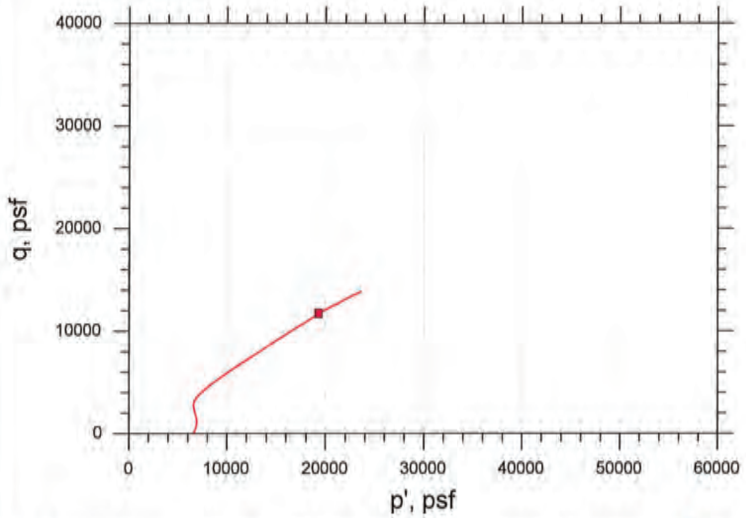
	Sample No.	Test No.	Depth	Tested By	Test Date	Checked By	Check Date	Test File
■	4	CU-8-1	60-62 ft	md	2/25/16	njh	3/2/16	303915-CU-8-1n.dat

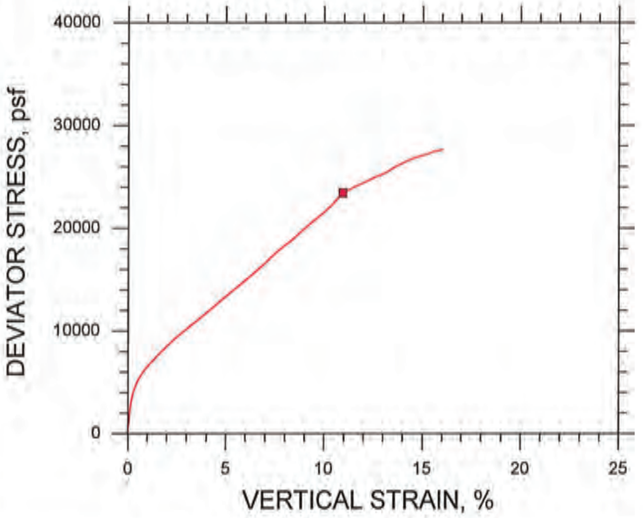
GeoTesting EXPRESS

Project: Vectran AB Brown Ash Pond	Location: Evansville, IN	Project No.: GTX-303915
Boring No.: AECOM-B2	Sample Type: intact	
Description: Moist, light brown silt		
Remarks: System Q		

Client: AECOM
Project Name: Vectran AB Brown Ash Pond
Project Location: Evansville, IN
Project Number: GTX-303915
Tested By: md	Checked By: njh
Boring ID: AECOM-B4
Preparation: intact
Description: Moist, brown silt
Classification: ---
Group Symbol: ---
Liquid Limit: ---	Plastic Limit: ---
Plasticity Index: ---	Estimated Specific Gravity: 2.7

CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767





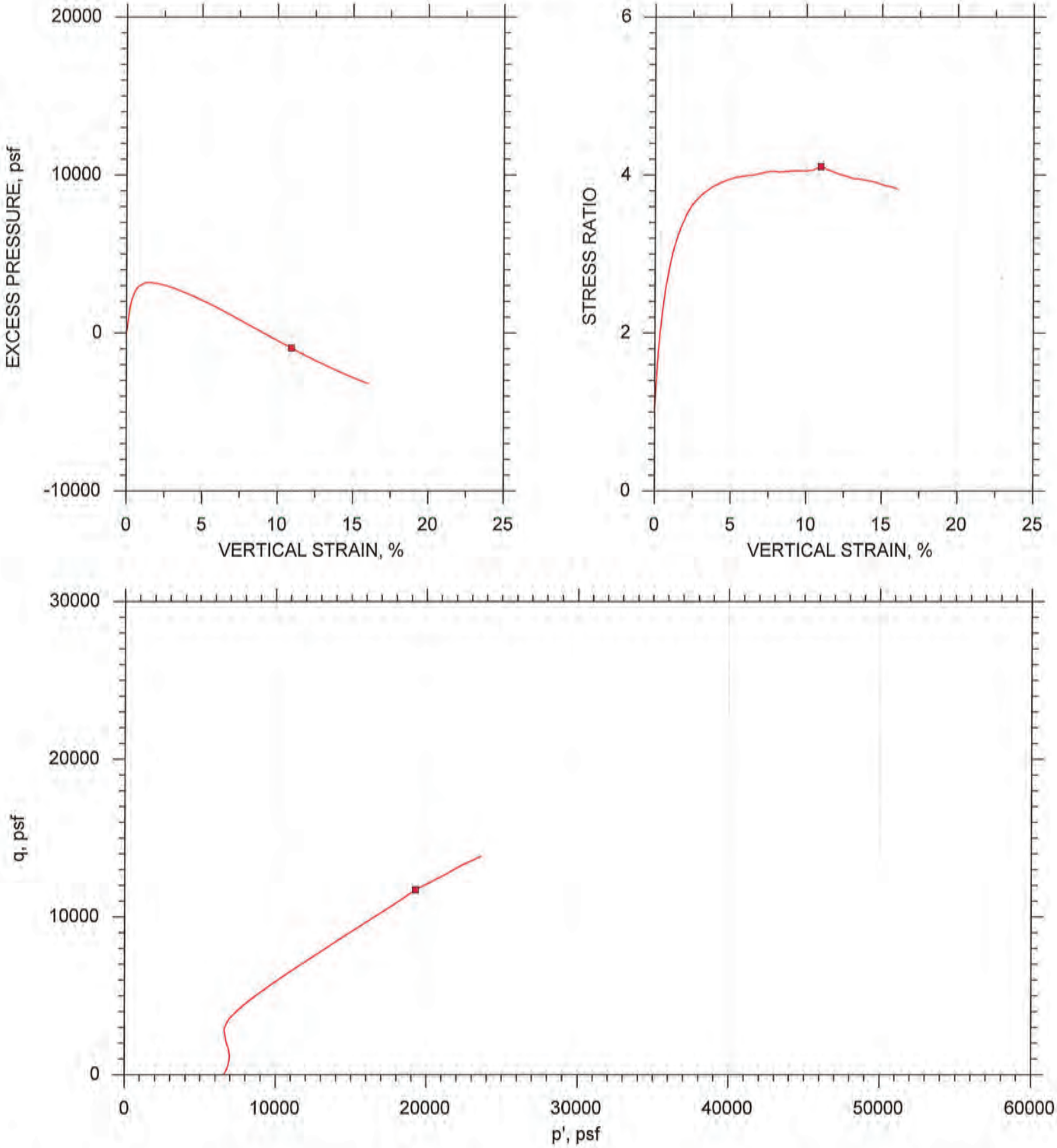
Symbol		🟥			
Sample ID		3			
Depth, ft		46-48 ft			
Test Number		CU-11-1			
Initial	Height, in	4.580			
	Diameter, in	2.020			
	Moisture Content (from Cuttings), %	26.8			
	Dry Density, pcf	97.4			
	Saturation (Wet Method), %	99.0			
	Void Ratio	0.731			
Before Shear	Moisture Content, %	25.0			
	Dry Density, pcf	101.			
	Cross-sectional Area (Method A), in²	3.105			
	Saturation, %	100.0			
	Void Ratio	0.674			
	Back Pressure, psf	2.173e+004			
Vertical Effective Consolidation Stress, psf		6597.			
Horizontal Effective Consolidation Stress, psf		6600.			
Vertical Strain after Consolidation, %		0.3480			
Volumetric Strain after Consolidation, %		3.773			
Time to 50% Consolidation, min		0.2500			
Shear Strength, psf		1.172e+004			
Strain at Failure, %		10.9			
Strain Rate, %/min		0.01600			
Deviator Stress at Failure, psf		2.344e+004			
Effective Minor Principal Stress at Failure, psf		7543.			
Effective Major Principal Stress at Failure, psf		3.098e+004			
B-Value		0.96			

Notes:
- Before Shear Saturation set to 100% for phase calculation.
- Moisture Content determined by ASTM D2216.
- Deviator Stress includes membrane correction.
- Values for c and φ determined from best-fit straight line for the specific test conditions. Actual strength parameters may vary and should be determined by an engineer for site conditions.

Remarks:

System K

CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767



	Sample No.	Test No.	Depth	Tested By	Test Date	Checked By	Check Date	Test File
🟥	3	CU-11-1	46-48 ft	md	2/25/16	njh	3/2/16	303915-CU-11-1n.dat

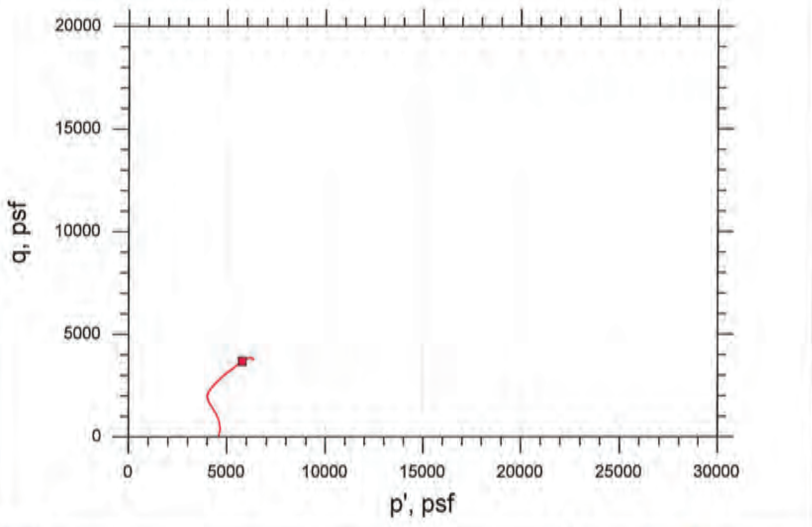
GeoTesting EXPRESS

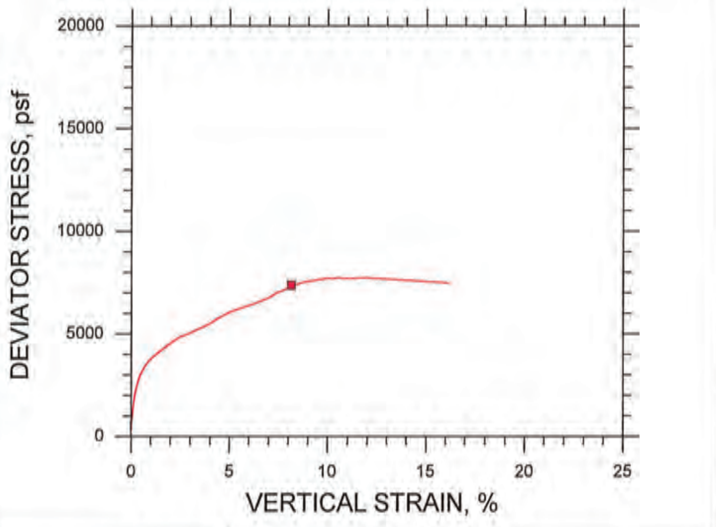
Project: Vectran AB Brown Ash Pond	Location: Evansville, IN	Project No.: GTX-303915
Boring No.: AECOM-B4	Sample Type: intact	
Description: Moist, brown silt		
Remarks: System K		

GeoTesting
EXPRESS

Client: AECOM
Project Name: Vectran AB Brown Ash Pond
Project Location: Evansville, IN
Project Number: GTX-303915
Tested By: md
Checked By: njh
Boring ID: AECOM-B4
Preparation: intact
Description: Moist, gray silty clay
Classification: ---
Group Symbol: ---
Liquid Limit: ---
Plastic Limit: ---
Plasticity Index: ---
Estimated Specific Gravity: 2.7

CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767





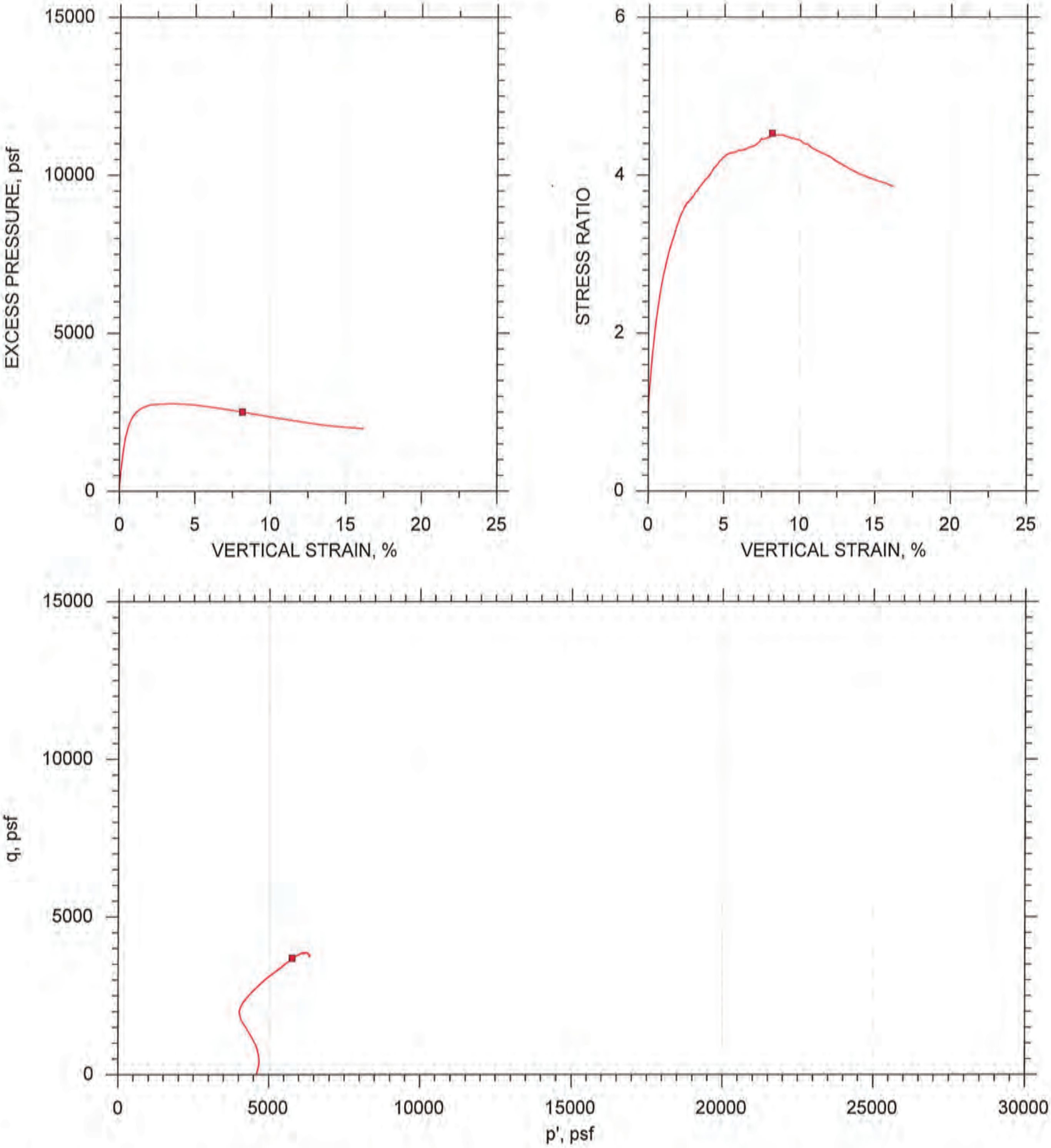
Symbol		🟥			
Sample ID		2			
Depth, ft		33-35 ft			
Test Number		CU-10-1			
Initial	Height, in	4.780			
	Diameter, in	2.020			
	Moisture Content (from Cuttings), %	38.4			
	Dry Density, pcf	82.7			
	Saturation (Wet Method), %	100.0			
	Void Ratio	1.04			
Before Shear	Moisture Content, %	35.7			
	Dry Density, pcf	85.9			
	Cross-sectional Area (Method A), in²	3.120			
	Saturation, %	100.0			
	Void Ratio	0.963			
	Back Pressure, psf	2.173e+004			
Vertical Effective Consolidation Stress, psf		4587.			
Horizontal Effective Consolidation Stress, psf		4599.			
Vertical Strain after Consolidation, %		1.170			
Volumetric Strain after Consolidation, %		3.967			
Time to 50% Consolidation, min		0.2500			
Shear Strength, psf		3690.			
Strain at Failure, %		8.15			
Strain Rate, %/min		0.01600			
Deviator Stress at Failure, psf		7380.			
Effective Minor Principal Stress at Failure, psf		2090.			
Effective Major Principal Stress at Failure, psf		9470.			
B-Value		0.96			

Notes:
- Before Shear Saturation set to 100% for phase calculation.
- Moisture Content determined by ASTM D2216.
- Deviator Stress includes membrane correction.
- Values for c and φ determined from best-fit straight line for the specific test conditions. Actual strength parameters may vary and should be determined by an engineer for site conditions.

Remarks:

System F

CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767



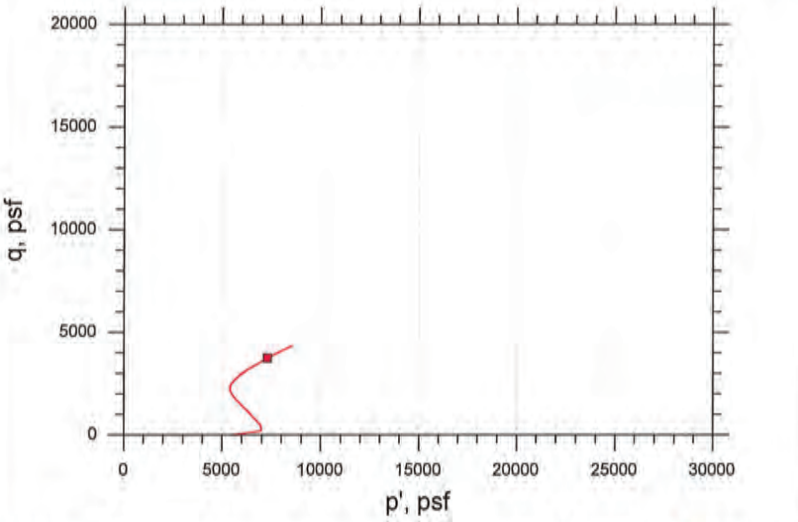
	Sample No.	Test No.	Depth	Tested By	Test Date	Checked By	Check Date	Test File
🟥	2	CU-10-1	33-35 ft	md	2/25/16	njh	2/2/16	303915-CU-10-1n.dat

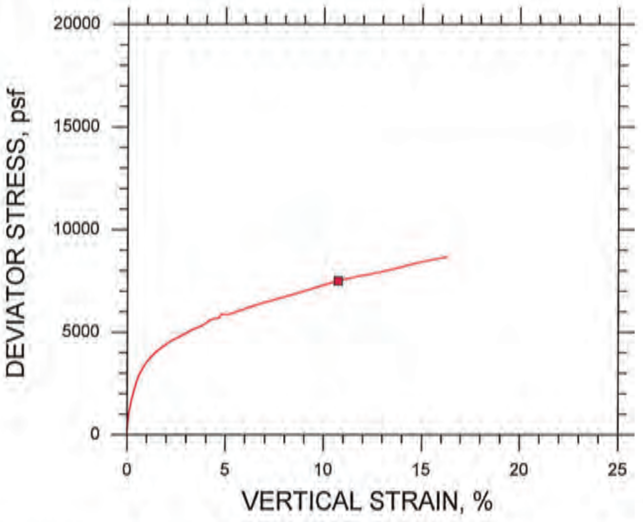
GeoTesting EXPRESS

Project: Vectran AB Brown Ash Pond	Location: Evansville, IN	Project No.: GTX-303915
Boring No.: AECOM-B4	Sample Type: intact	
Description: Moist, gray silty clay		
Remarks: System F		

Client: AECOM	
Project Name: Vectran AB Brown Ash Pond	
Project Location: Evansville, IN	
Project Number: GTX-303915	
Tested By: md	Checked By: njh
Boring ID: AECOM-B5	
Preparation: intact	
Description: Moist, gray silty clay	
Classification: ---	
Group Symbol: ---	
Liquid Limit: ---	Plastic Limit: ---
Plasticity Index: ---	Estimated Specific Gravity: 2.7

CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767





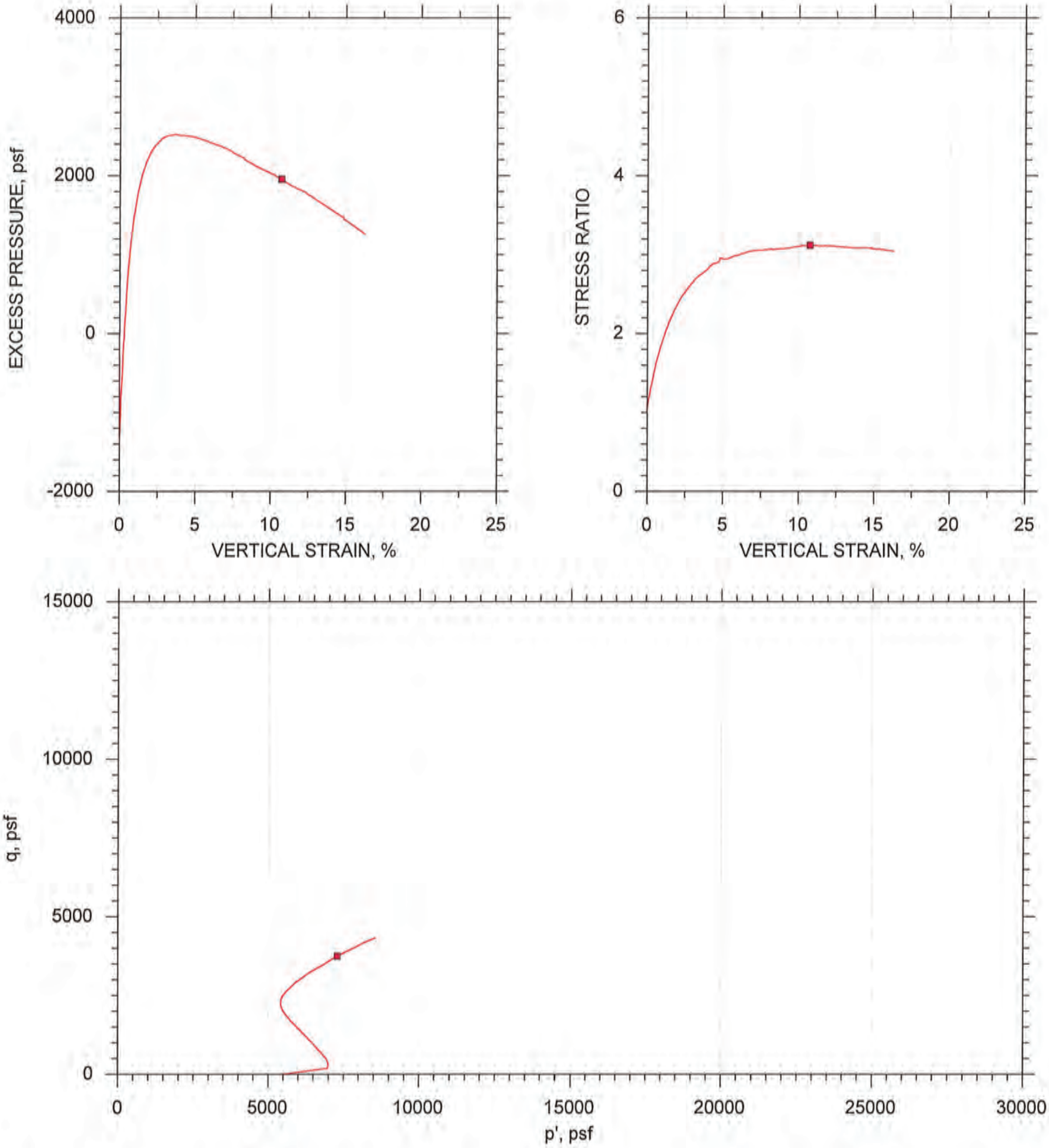
Symbol		🟥			
Sample ID		3			
Depth, ft		34-36 ft			
Test Number		CU-9-1			
Initial	Height, in	4.600			
	Diameter, in	2.040			
	Moisture Content (from Cuttings), %	49.8			
	Dry Density, pcf	71.0			
	Saturation (Wet Method), %	97.7			
	Void Ratio	1.38			
Before Shear	Moisture Content, %	53.9			
	Dry Density, pcf	68.7			
	Cross-sectional Area (Method A), in²	3.434			
	Saturation, %	100.0			
	Void Ratio	1.45			
	Back Pressure, psf	2.084e+004			
Vertical Effective Consolidation Stress, psf		5478.			
Horizontal Effective Consolidation Stress, psf		5499.			
Vertical Strain after Consolidation, %		1.907			
Volumetric Strain after Consolidation, %		-2.500			
Time to 50% Consolidation, min		0.3000			
Shear Strength, psf		3751.			
Strain at Failure, %		10.7			
Strain Rate, %/min		0.01600			
Deviator Stress at Failure, psf		7502.			
Effective Minor Principal Stress at Failure, psf		3539.			
Effective Major Principal Stress at Failure, psf		1.104e+004			
B-Value		0.95			

Notes:
- Before Shear Saturation set to 100% for phase calculation.
- Moisture Content determined by ASTM D2216.
- Deviator Stress includes membrane correction.
- Values for c and φ determined from best-fit straight line for the specific test conditions. Actual strength parameters may vary and should be determined by an engineer for site conditions.

Remarks:

System Y

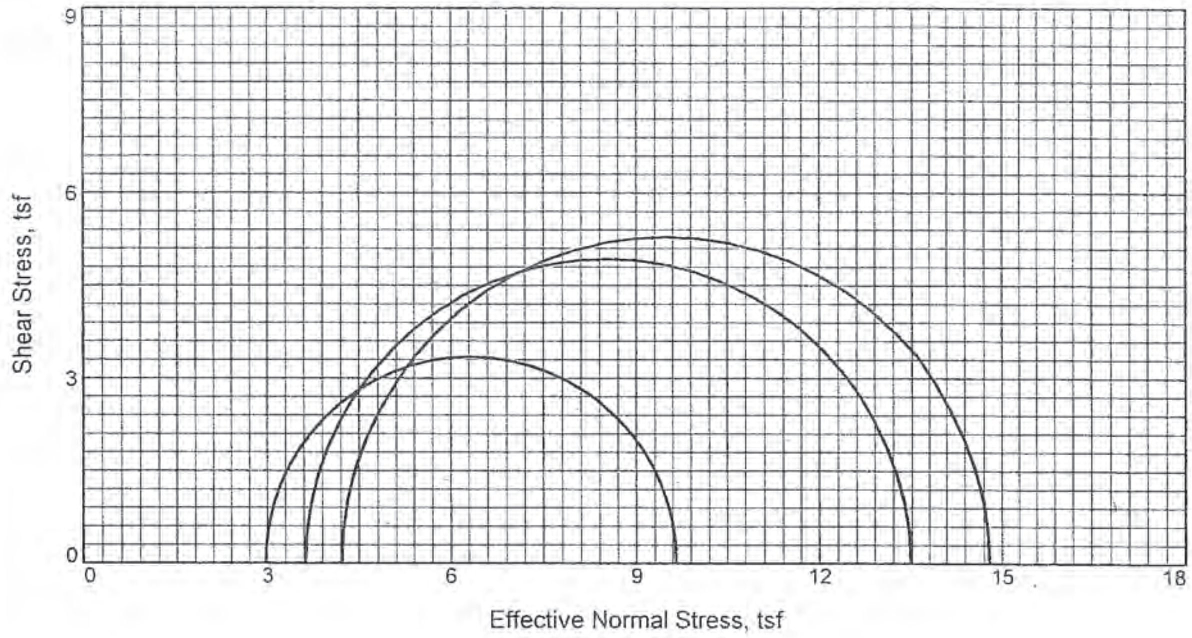
CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767

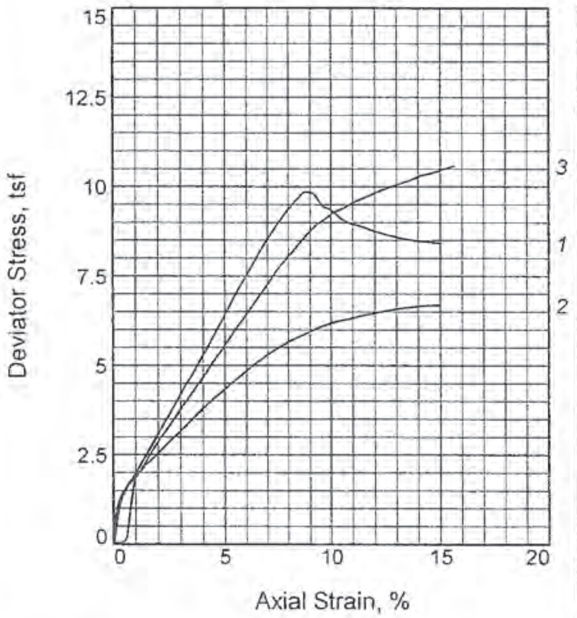


	Sample No.	Test No.	Depth	Tested By	Test Date	Checked By	Check Date	Test File
🟥	3	CU-9-1	34-36 ft	md	2/25/16	njh	3/2/16	303915-CU-9-1n.dat

GeoTesting EXPRESS

Project: Vectran AB Brown Ash Pond	Location: Evansville, IN	Project No.: GTX-303915
Boring No.: AECOM-B5	Sample Type: intact	
Description: Moist, gray silty clay		
Remarks: System Y		





	Sample No.	1	2	3
Initial	Water Content, %	22.7	23.4	23.2
	Dry Density, pcf	103.3	102.0	102.5
	Saturation, %	100.0	100.0	100.0
	Void Ratio	0.6019	0.6212	0.6135
	Diameter, in.	2.87	2.87	2.86
	Height, in.	5.73	5.78	5.69
At Test	Water Content, %	22.7	22.4	22.5
	Dry Density, pcf	103.3	103.8	103.6
	Saturation, %	100.0	100.0	100.0
	Void Ratio	0.6019	0.5944	0.5965
	Diameter, in.	2.87	2.85	2.85
	Height, in.	5.73	5.75	5.67
	Strain rate, %/min.	0.06	0.06	0.06
	Back Pressure, psi	50.00	45.00	40.00
	Cell Pressure, psi	73.00	73.00	73.00
	Fail. Stress, tsf	9.9	6.7	10.6
	Total Pore Pr., tsf	1.6	2.3	1.0
	Ult. Stress, tsf			
	Total Pore Pr., tsf			
	$\bar{\sigma}_1$ Failure, tsf	13.5	9.7	14.8
	$\bar{\sigma}_3$ Failure, tsf	3.6	3.0	4.2

Type of Test:
CU with Pore Pressures

Sample Type: Shelby tube

Description:

Assumed Specific Gravity= 2.65

Remarks:

Figure CU7211C

Client: Vectren

Project: Brown Safety Factor Assessment

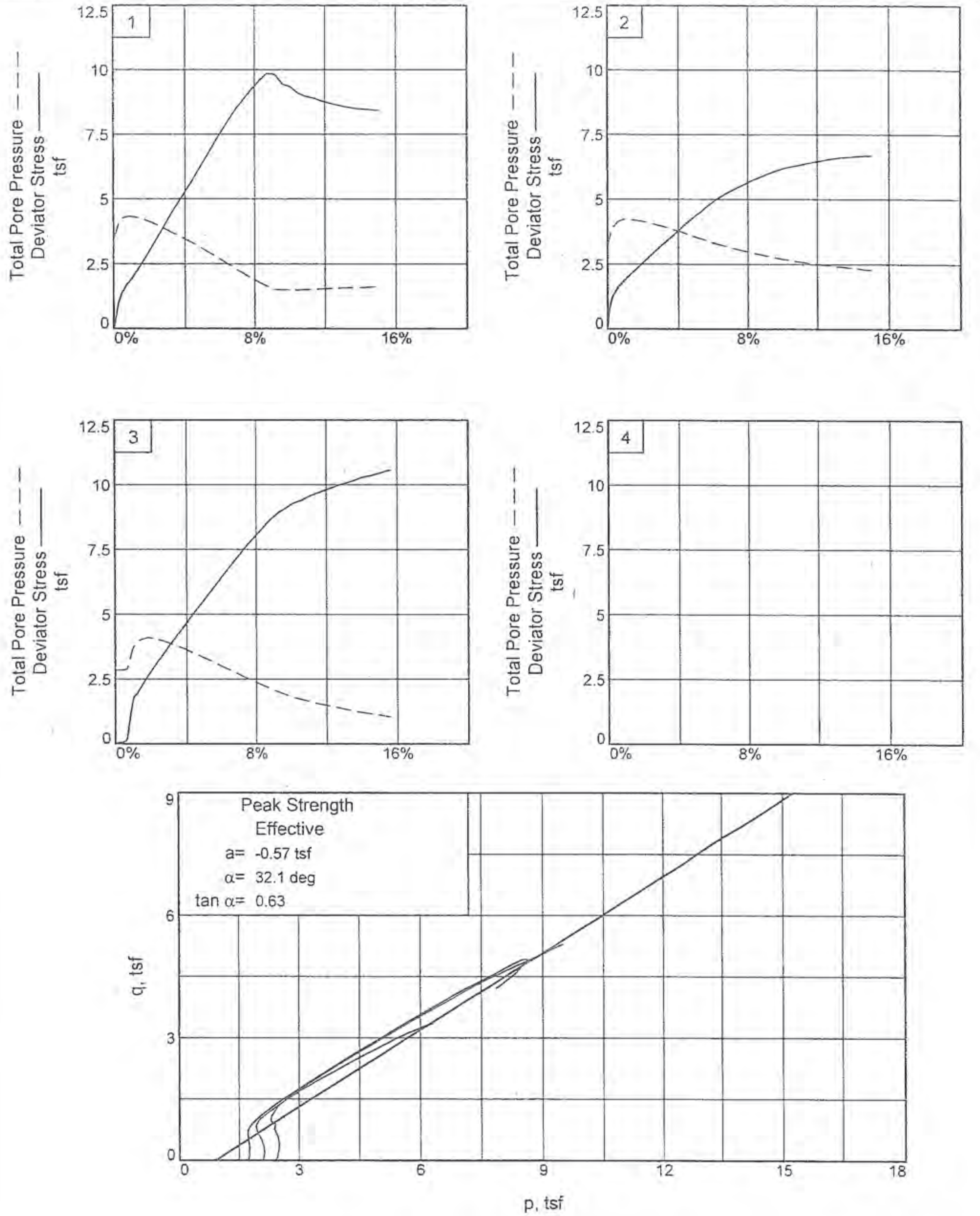
Source of Sample: 7211 **Depth:** 63-65'

Sample Number: B-202

Proj. No.: 170GC00108 **Date Sampled:**

TRIAXIAL SHEAR TEST REPORT
Cardno ATC, INC.
Indianapolis, Indiana

Tested By: MDr



Client: Vectren

Project: Brown Safety Factor Assessment

Source of Sample: 7211 **Depth:** 63-65' **Sample Number:** B-202

Project No.: 170GC00108 **Figure** ________ Cardno ATC, Inc.

Tested By: MDr

TRIAXIAL COMPRESSION TEST
CU with Pore Pressures

6/25/2015
4:38 PM

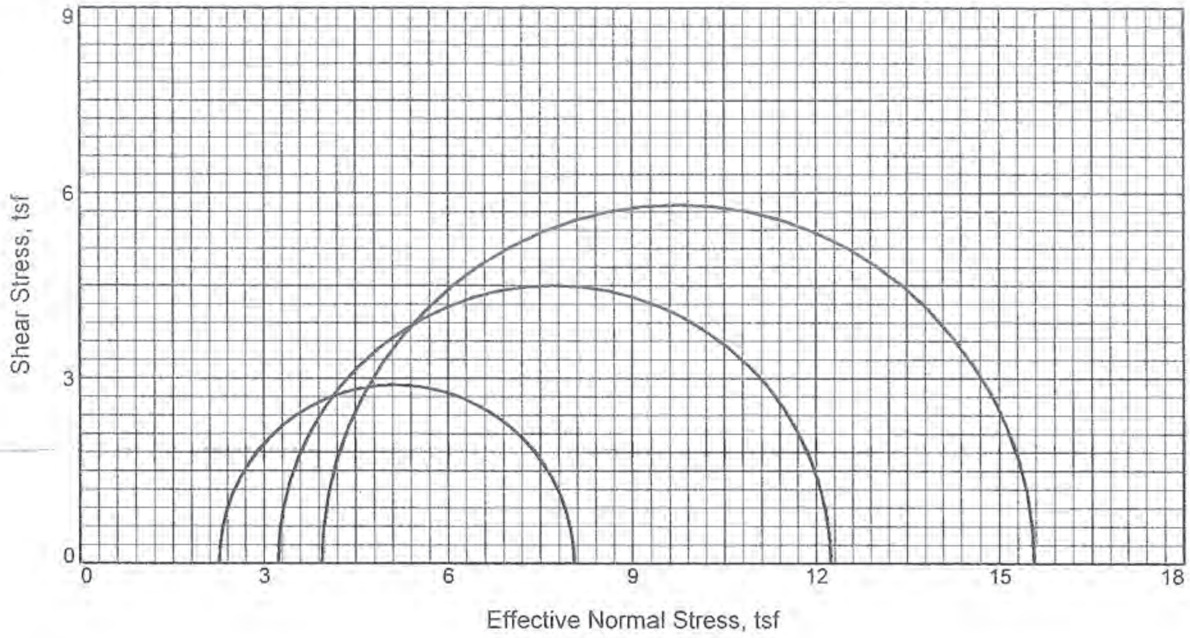
Date:
Client: Vectren
Project: Brown Safety Factor Assessment
Project No.: 170GC00108
Location: 7211
Depth: 63-65' **Sample Number:** B-202
Description:
Remarks:
Type of Sample: Shelby tube
Assumed Specific Gravity=2.65 LL= PL= PI=
Test Method: COE uniform strain

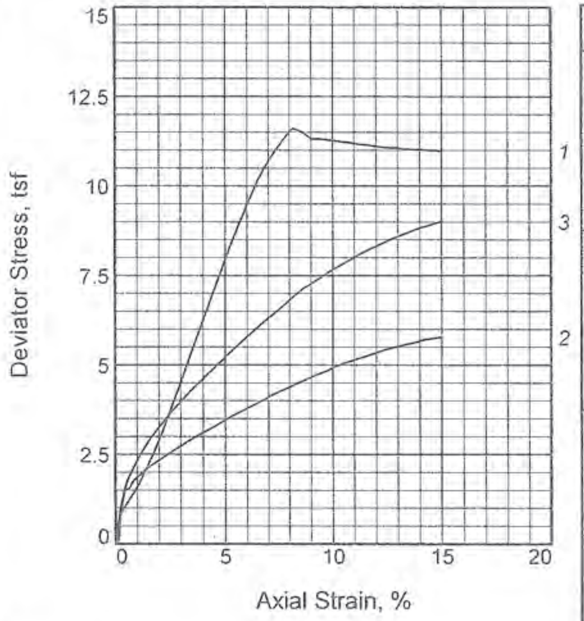
Parameters for Specimen No. 1

Specimen Parameter	Initial	Saturated	Consolidated	Final
Moisture content: Moist soil+tare, gms.	1228.900			1328.080
Moisture content: Dry soil+tare, gms.	1001.380			1113.250
Moisture content: Tare, gms.	0.000			111.870
Moisture, %	22.7	22.8	22.7	21.5
Moist specimen weight, gms.	1228.9			
Diameter, in.	2.87	2.87	2.87	
Area, in.^{2}	6.45	6.46	6.45	
Height, in.	5.73	5.73	5.73	
Net decrease in height, in.		0.00	0.00	
Wet density, pcf	126.7	126.6	126.7	
Dry density, pcf	103.3	103.1	103.3	
Void ratio	0.6019	0.6053	0.6019	
Saturation, %	100.0	100.0	100.0	

Test Readings for Specimen No. 1

Consolidation cell pressure = 73.00 psi (5.256 tsf)
Consolidation back pressure = 50.00 psi (3.600 tsf)
Consolidation effective confining stress = 1.656 tsf
Strain rate, %/min. = 0.06
Fail. Stress = 9.856 tsf at reading no. 34





	Sample No.	1	2	3
Initial	Water Content, %	19.8	19.1	19.0
	Dry Density, pcf	105.8	105.4	108.0
	Saturation, %	93.0	88.6	94.5
	Void Ratio	0.5642	0.5703	0.5315
	Diameter, in.	2.85	2.87	2.85
	Height, in.	5.86	6.03	6.14
At Test	Water Content, %	19.6	19.9	18.6
	Dry Density, pcf	108.9	108.2	110.8
	Saturation, %	100.0	100.0	100.0
	Void Ratio	0.5189	0.5283	0.4928
	Diameter, in.	2.83	2.84	2.82
	Height, in.	5.80	5.97	6.09
	Strain rate, %/min.	0.07	0.07	0.07
	Back Pressure, psi	55.00	45.00	45.00
	Cell Pressure, psi	78.00	73.00	78.00
	Fail. Stress, tsf	11.6	5.8	9.0
	Total Pore Pr., tsf	1.7	3.0	2.4
	Ult. Stress, tsf			
	Total Pore Pr., tsf			
	$\bar{\sigma}_1$ Failure, tsf	15.6	8.0	12.2
	$\bar{\sigma}_3$ Failure, tsf	4.0	2.3	3.2

Type of Test:
CU with Pore Pressures

Sample Type: Shelby tube

Description:

Assumed Specific Gravity= 2.65

Remarks:

Figure CU7211E

Client: Vectren

Project: Brown Safety Factor Assessment

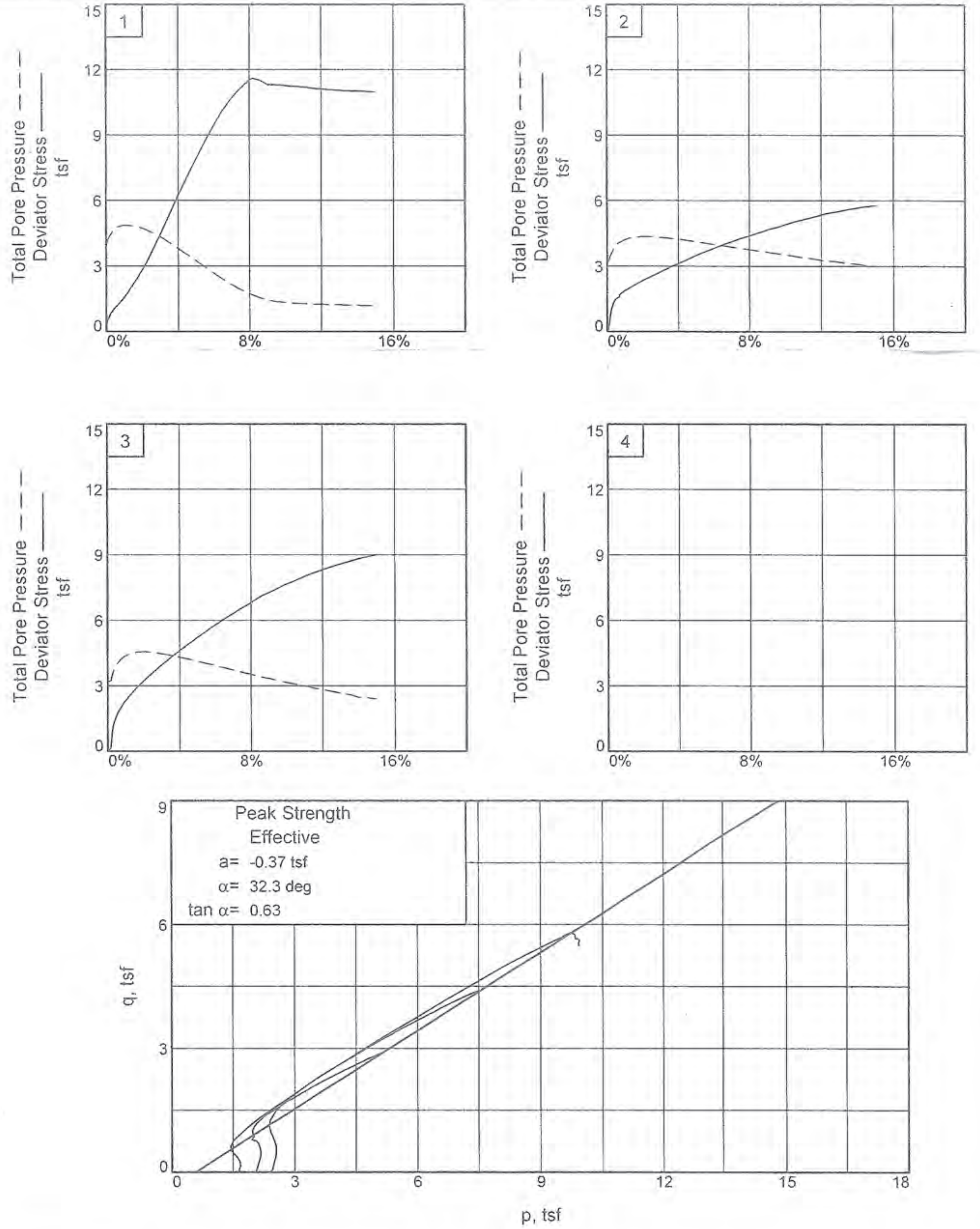
Source of Sample: 7211 **Depth:** 63-65'

Sample Number: B-203

Proj. No.: 170GC00108 **Date Sampled:**

TRIAXIAL SHEAR TEST REPORT
Cardno ATC, INC.
Indianapolis, Indiana

Tested By: MDr



Client: Vectren
Project: Brown Safety Factor Assessment
Source of Sample: 7211 **Depth:** 63-65' **Sample Number:** B-203
Project No.: 170GC00108 **Figure** ________ **Cardno ATC, Inc.**

Tested By: MDr

TRIAXIAL COMPRESSION TEST

CU with Pore Pressures

6/28/2015
2:57 PM

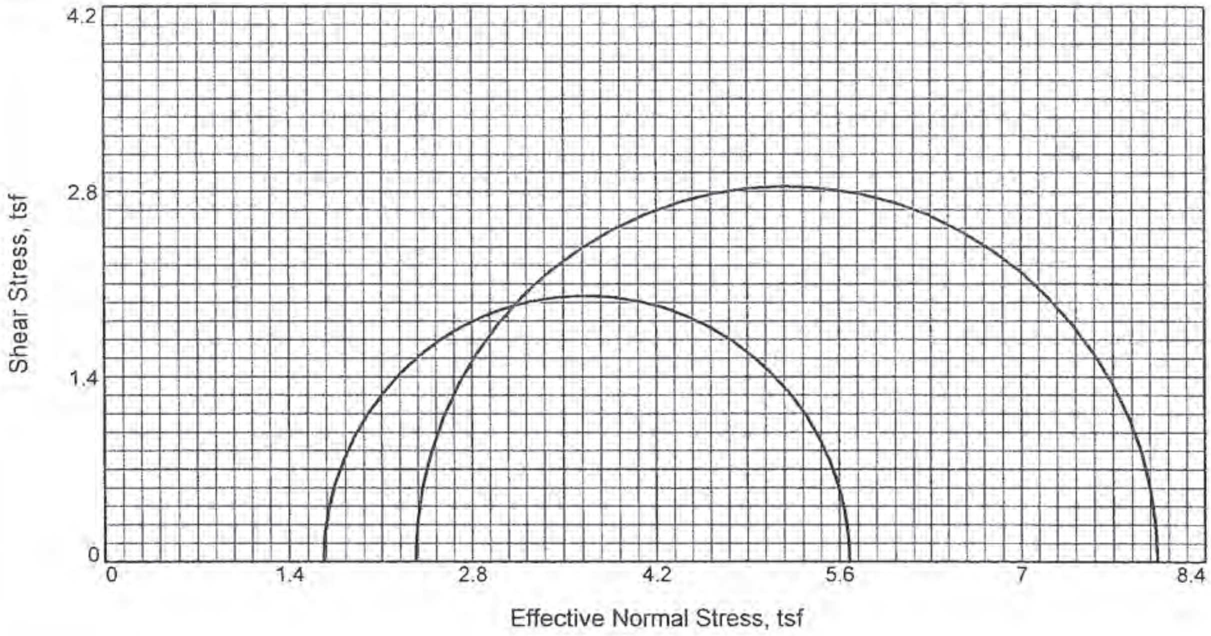
Date:
Client: Vectren
Project: Brown Safety Factor Assessment
Project No.: 170GC00108
Location: 7211
Depth: 63-65' **Sample Number:** B-203
Description:
Remarks:
Type of Sample: Shelby tube
Assumed Specific Gravity=2.65 **LL=** **PL=** **PI=**
Test Method: COE uniform strain

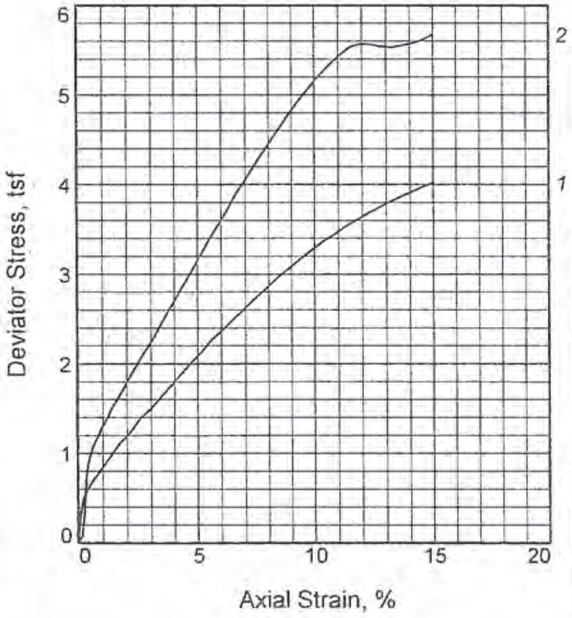
Parameters for Specimen No. 1

Specimen Parameter	Initial	Saturated	Consolidated	Final
Moisture content: Moist soil+tare, gms.	1245.910			1359.100
Moisture content: Dry soil+tare, gms.	1040.020			1148.530
Moisture content: Tare, gms.	0.000			108.510
Moisture, %	19.8	20.3	19.6	20.2
Moist specimen weight, gms.	1245.9			
Diameter, in.	2.85	2.84	2.83	
Area, in.²	6.39	6.32	6.27	
Height, in.	5.86	5.83	5.80	
Net decrease in height, in.		0.03	0.02	
Wet density, pcf	126.7	129.4	130.2	
Dry density, pcf	105.8	107.6	108.9	
Void ratio	0.5642	0.5371	0.5189	
Saturation, %	93.0	100.0	100.0	

Test Readings for Specimen No. 1

Consolidation cell pressure = 78.00 psi (5.616 tsf)
Consolidation back pressure = 55.00 psi (3.960 tsf)
Consolidation effective confining stress = 1.656 tsf
Strain rate, %/min. = 0.07
Fail. Stress = 11.616 tsf **at reading no.** 34





	Sample No.	1	2
Initial	Water Content, %	20.7	21.5
	Dry Density, pcf	106.8	105.5
	Saturation, %	100.0	100.0
	Void Ratio	0.5492	0.5686
	Diameter, in.	2.85	2.85
	Height, in.	5.80	5.65
At Test	Water Content, %	20.1	20.9
	Dry Density, pcf	108.0	106.5
	Saturation, %	100.0	100.0
	Void Ratio	0.5324	0.5528
	Diameter, in.	2.84	2.84
	Height, in.	5.78	5.63
	Strain rate, %/min.	0.06	0.06
	Back Pressure, psi	55.00	45.00
	Cell Pressure, psi	69.00	64.00
	Fail. Stress, tsf	4.02	5.68
	Total Pore Pr., tsf	3.31	2.25
	Ult. Stress, tsf		
	Total Pore Pr., tsf		
	$\bar{\sigma}_1$ Failure, tsf	5.68	8.04
	$\bar{\sigma}_3$ Failure, tsf	1.66	2.36

Type of Test:
CU with Pore Pressures

Sample Type: Shelby tube

Description:

Assumed Specific Gravity= 2.65

Remarks:

Figure CU7211J

Client: Vectren

Project: Brown Safety Factor Assessment

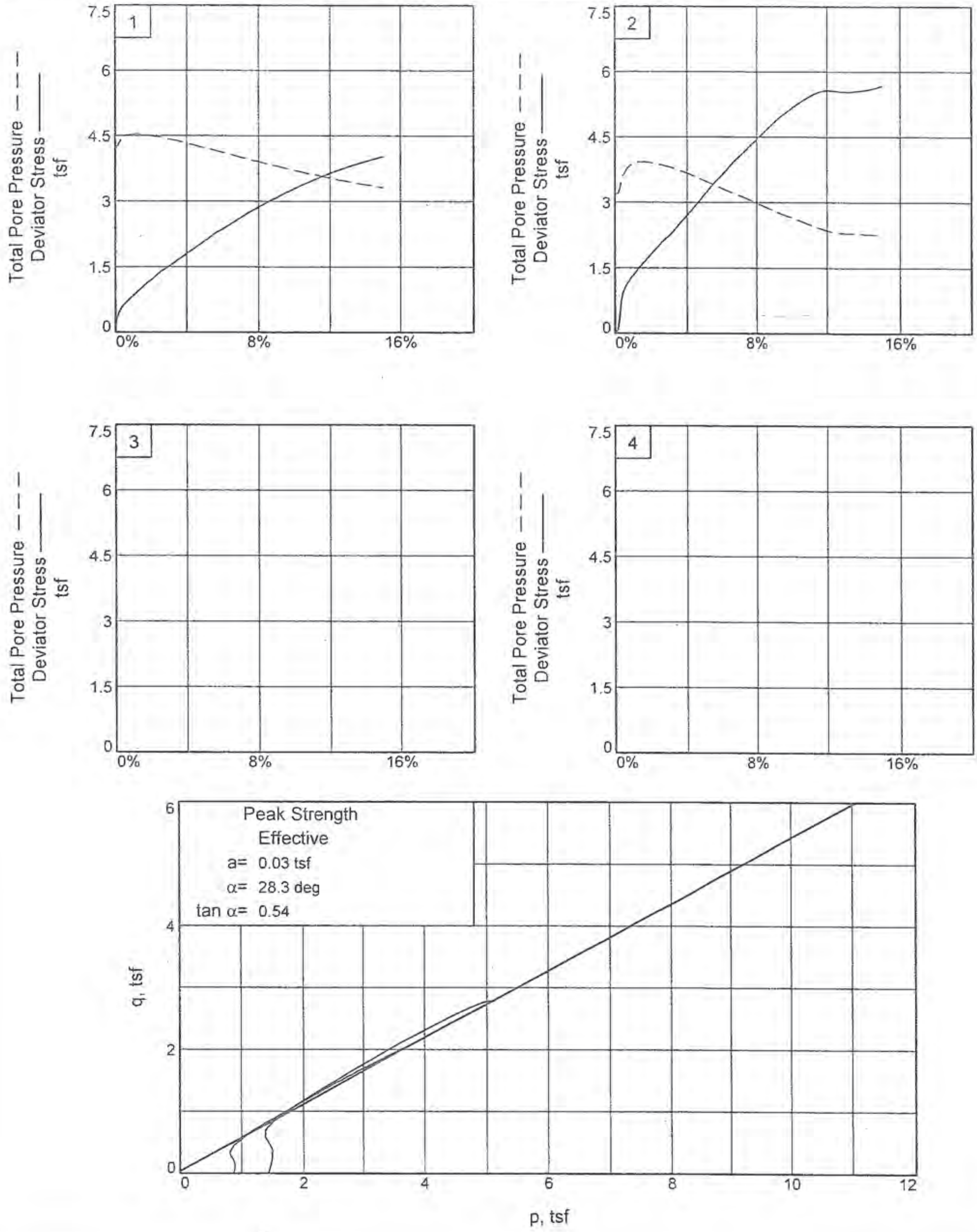
Source of Sample: 7211 **Depth:** 28-30'

Sample Number: B-206

Proj. No.: 170GC00108 **Date Sampled:**

TRIAXIAL SHEAR TEST REPORT
Cardno ATC, INC.
Indianapolis, Indiana

Tested By: MDr



Client: Vectren

Project: Brown Safety Factor Assessment

Source of Sample: 7211 **Depth:** 28-30' **Sample Number:** B-206

Project No.: 170GC00108 **Figure** ________

Cardno ATC, Inc.

Tested By: MDr

TRIAXIAL COMPRESSION TEST
CU with Pore Pressures

5/31/2015
6:43 PM

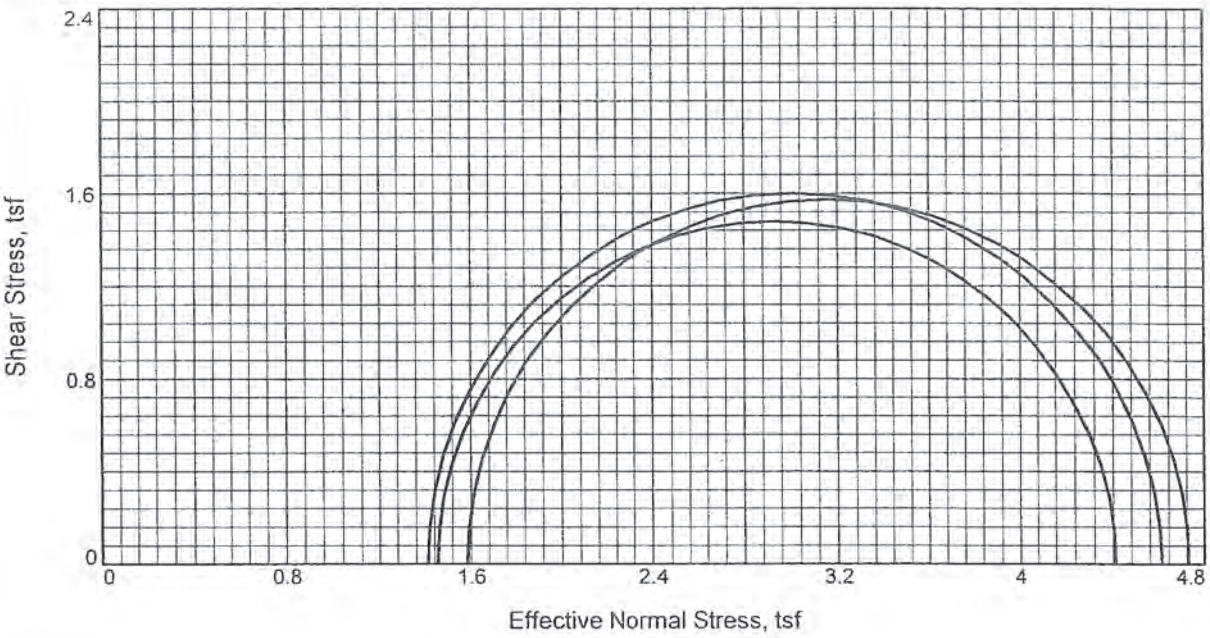
Date:
Client: Vectren
Project: Brown Safety Factor Assessment
Project No.: 170GC00108
Location: 7211
Depth: 28-30' **Sample Number:** B-206
Description:
Remarks:
Type of Sample: Shelby tube
Assumed Specific Gravity=2.65 **LL=** **PL=** **PI=**
Test Method: COE uniform strain

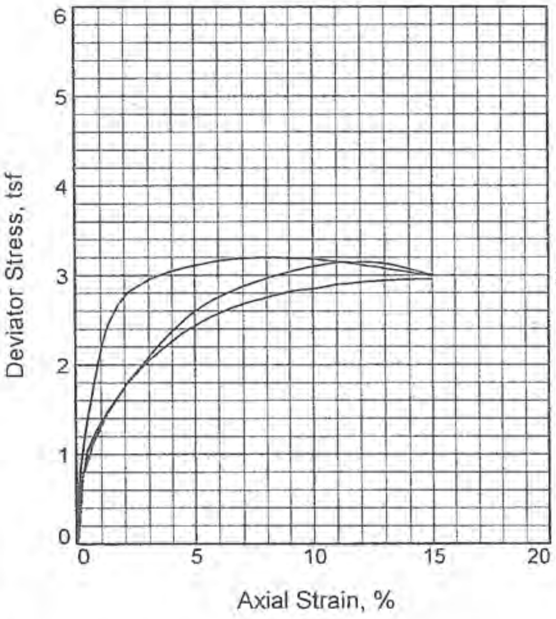
Parameters for Specimen No. 1

Specimen Parameter	Initial	Saturated	Consolidated	Final
Moisture content: Moist soil+tare, gms.	1255.800			1347.270
Moisture content: Dry soil+tare, gms.	1040.240			1150.470
Moisture content: Tare, gms.	0.000			110.230
Moisture, %	20.7	20.5	20.1	18.9
Moist specimen weight, gms.	1255.8			
Diameter, in.	2.85	2.85	2.84	
Area, in.²	6.40	6.38	6.35	
Height, in.	5.80	5.79	5.78	
Net decrease in height, in.		0.01	0.01	
Wet density, pcf	128.9	129.2	129.6	
Dry density, pcf	106.8	107.2	108.0	
Void ratio	0.5492	0.5428	0.5324	
Saturation, %	100.0	100.0	100.0	

Test Readings for Specimen No. 1

Consolidation cell pressure = 69.00 psi (4.968 tsf)
Consolidation back pressure = 55.00 psi (3.960 tsf)
Consolidation effective confining stress = 1.008 tsf
Strain rate, %/min. = 0.06
Fail. Stress = 4.023 tsf **at reading no. 50**





	Sample No.	1	2	3
Initial	Water Content, %	24.7	25.2	22.9
	Dry Density, pcf	99.5	99.1	103.0
	Saturation, %	98.7	99.9	99.9
	Void Ratio	0.6630	0.6690	0.6064
	Diameter, in.	2.85	2.87	2.86
	Height, in.	5.74	5.73	5.72
At Test	Water Content, %	23.7	23.9	21.1
	Dry Density, pcf	101.5	101.3	106.0
	Saturation, %	100.0	100.0	100.0
	Void Ratio	0.6292	0.6325	0.5603
	Diameter, in.	2.83	2.85	2.83
	Height, in.	5.70	5.69	5.66
	Strain rate, %/min.	0.06	0.06	0.06
	Back Pressure, psi	55.00	55.00	50.00
	Cell Pressure, psi	70.00	75.00	75.00
	Fail. Stress, tsf	2.96	3.20	3.15
	Total Pore Pr., tsf	3.59	3.99	3.82
	Ult. Stress, tsf			
	Total Pore Pr., tsf			
	$\overline{\sigma}_1$ Failure, tsf	4.41	4.61	4.73
	$\overline{\sigma}_3$ Failure, tsf	1.45	1.41	1.58

Type of Test:
CU with Pore Pressures

Sample Type: Shelby tube

Description:

Assumed Specific Gravity= 2.65

Remarks:

Figure CU7211K

Client: Vectren

Project: Brown Safety Factor Assessment

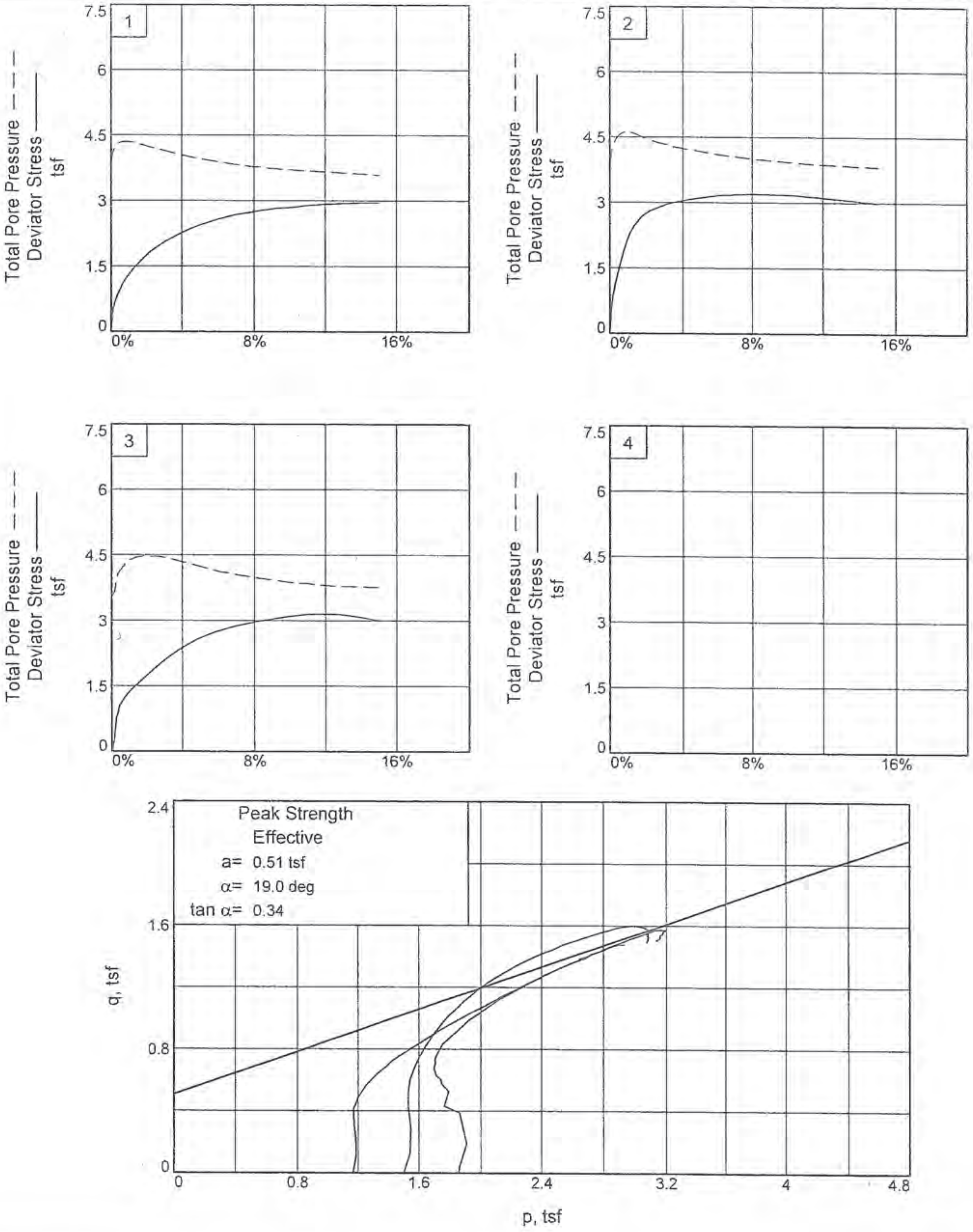
Source of Sample: 7211 **Depth:** 38-40'

Sample Number: B-206

Proj. No.: 170GC00108 **Date Sampled:**

TRIAXIAL SHEAR TEST REPORT
Cardno ATC, INC.
Indianapolis, Indiana

Tested By: MDr



Client: Vectren

Project: Brown Safety Factor Assessment

Source of Sample: 7211 **Depth:** 38-40' **Sample Number:** B-206

Project No.: 170GC00108 **Figure** ________

Cardno ATC, Inc.

Tested By: MDr

TRIAXIAL COMPRESSION TEST
CU with Pore Pressures

6/12/2015
2:18 PM

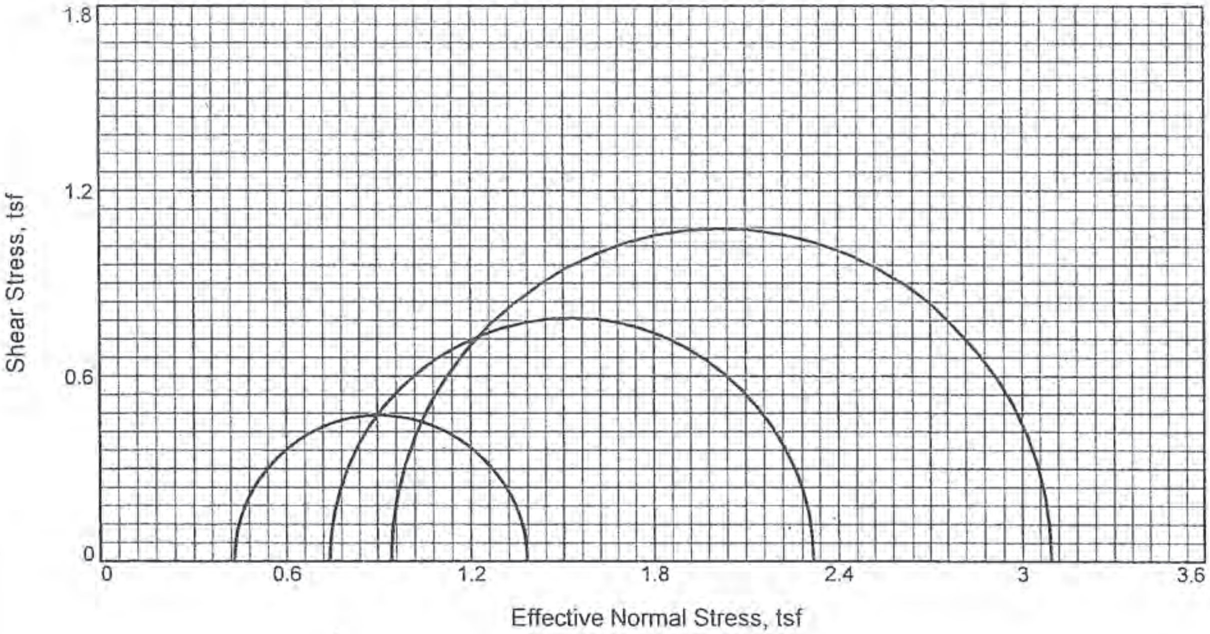
Date:
Client: Vectren
Project: Brown Safety Factor Assessment
Project No.: 170GC00108
Location: 7211
Depth: 38-40' **Sample Number:** B-206
Description:
Remarks:
Type of Sample: Shelby tube
Assumed Specific Gravity=2.65 **LL=** **PL=** **PI=**
Test Method: COE uniform strain

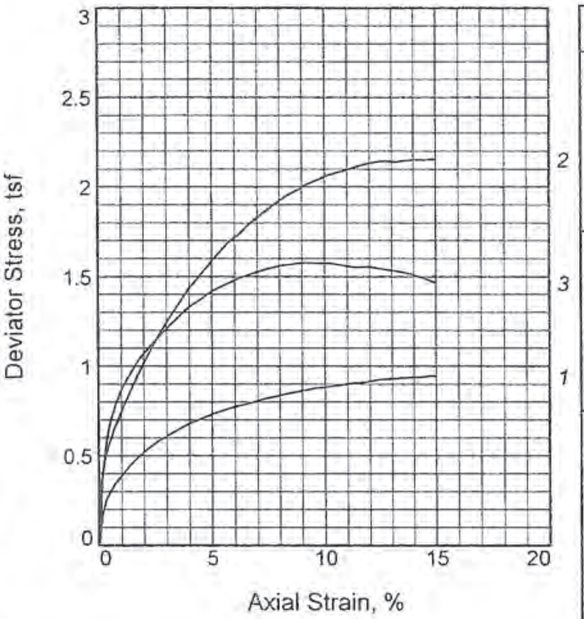
Parameters for Specimen No. 1

Specimen Parameter	Initial	Saturated	Consolidated	Final
Moisture content: Moist soil+tare, gms.	1190.530			1305.400
Moisture content: Dry soil+tare, gms.	954.690			1065.990
Moisture content: Tare, gms.	0.000			111.300
Moisture, %	24.7	24.3	23.7	25.1
Moist specimen weight, gms.	1190.5			
Diameter, in.	2.85	2.84	2.83	
Area, in.²	6.37	6.33	6.29	
Height, in.	5.74	5.71	5.70	
Net decrease in height, in.		0.02	0.02	
Wet density, pcf	124.1	125.1	125.7	
Dry density, pcf	99.5	100.6	101.5	
Void ratio	0.6630	0.6439	0.6292	
Saturation, %	98.7	100.0	100.0	

Test Readings for Specimen No. 1

Consolidation cell pressure = 70.00 psi (5.040 tsf)
Consolidation back pressure = 55.00 psi (3.960 tsf)
Consolidation effective confining stress = 1.080 tsf
Strain rate, %/min. = 0.06
Fail. Stress = 2.957 tsf **at reading no.** 48





	Sample No.	1	2	3
Initial	Water Content, %	24.1	22.5	23.5
	Dry Density, pcf	99.7	102.1	101.9
	Saturation, %	96.8	96.3	100.0
	Void Ratio	0.6596	0.6203	0.6238
	Diameter, in.	2.82	2.84	2.85
	Height, in.	5.76	5.70	5.71
At Test	Water Content, %	23.5	22.9	21.7
	Dry Density, pcf	102.0	103.0	105.1
	Saturation, %	100.0	100.0	100.0
	Void Ratio	0.6226	0.6067	0.5738
	Diameter, in.	2.80	2.84	2.82
	Height, in.	5.71	5.69	5.65
	Strain rate, %/min.	0.07	0.07	0.07
	Back Pressure, psi	50.00	45.00	40.00
	Cell Pressure, psi	55.00	55.00	55.00
	Fail. Stress, tsf	0.95	2.15	1.58
	Total Pore Pr., tsf	3.53	3.02	3.22
	Ult. Stress, tsf			
	Total Pore Pr., tsf			
	$\bar{\sigma}_1$ Failure, tsf	1.38	3.10	2.32
	$\bar{\sigma}_3$ Failure, tsf	0.43	0.94	0.74

Type of Test:
CU with Pore Pressures

Sample Type: Shelby tube

Description:

Assumed Specific Gravity= 2.65

Remarks:

Figure CU7211M

Client: Vectren

Project: Brown Safety Factor Assessment

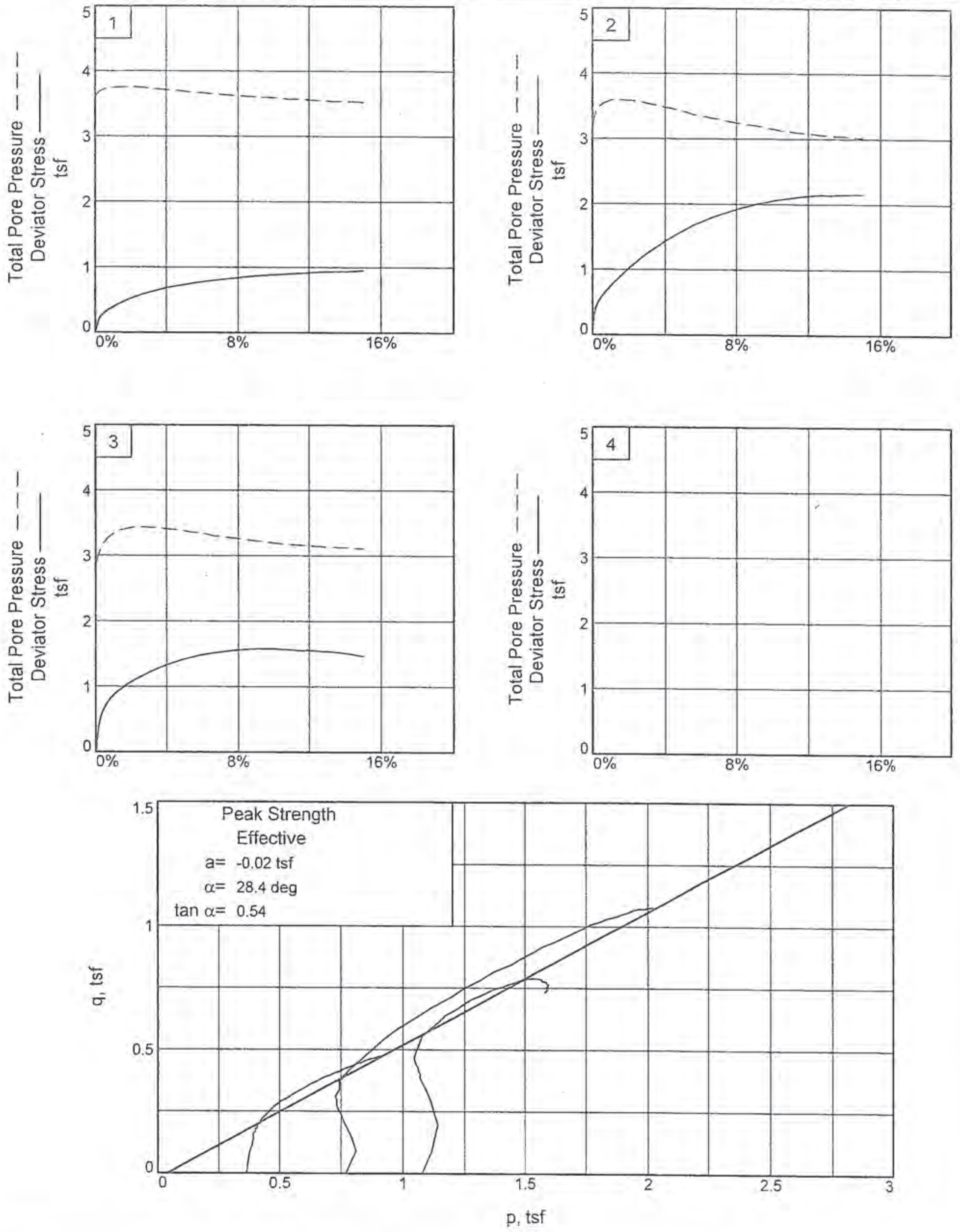
Source of Sample: 7211 **Depth:** 18-20'

Sample Number: B-207

Proj. No.: 170GC00108 **Date Sampled:**

TRIAXIAL SHEAR TEST REPORT
Cardno ATC, INC.
Indianapolis, Indiana

Tested By: MDr



Client: Vectren

Project: Brown Safety Factor Assessment

Source of Sample: 7211 **Depth:** 18-20' **Sample Number:** B-207

Project No.: 170GC00108 **Figure** ______

Cardno ATC, Inc.

Tested By: MDr

TRIAXIAL COMPRESSION TEST

CU with Pore Pressures

6/9/2015
2:48 PM

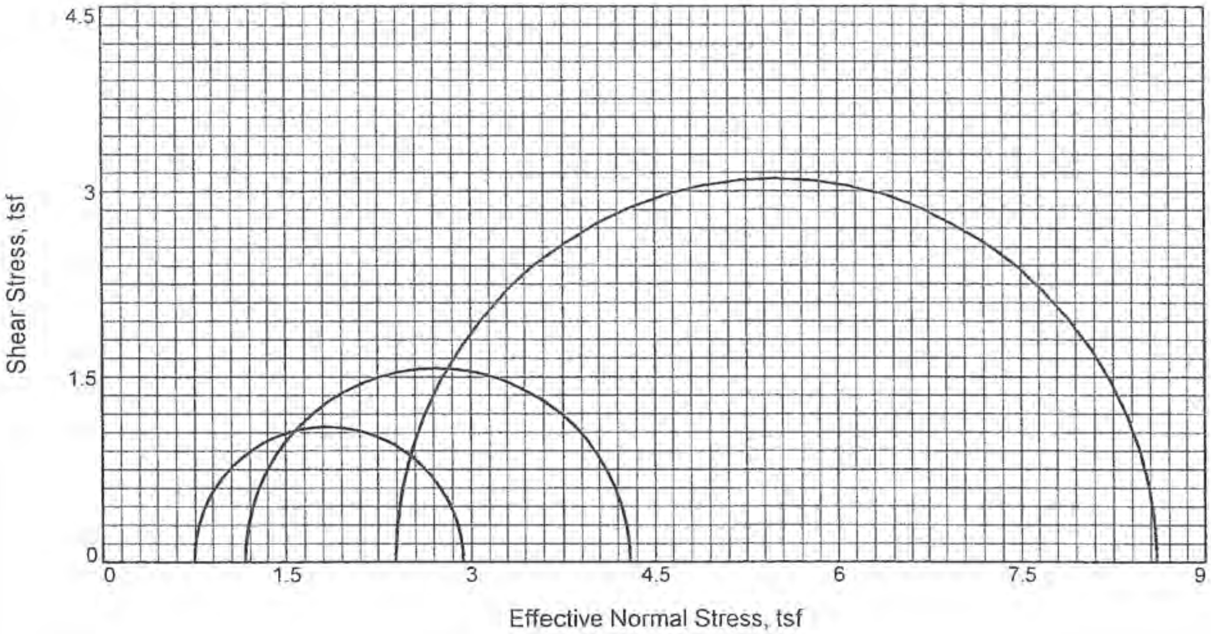
Date:
Client: Vectren
Project: Brown Safety Factor Assessment
Project No.: 170GC00108
Location: 7211
Depth: 18-20' **Sample Number:** B-207
Description:
Remarks:
Type of Sample: Shelby tube
Assumed Specific Gravity=2.65 **LL=** **PL=** **PI=**
Test Method: COE uniform strain

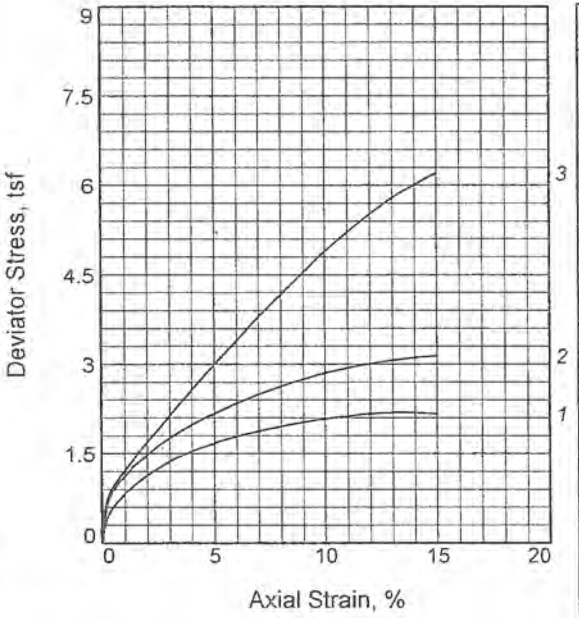
Parameters for Specimen No. 1

Specimen Parameter	Initial	Saturated	Consolidated	Final
Moisture content: Moist soil+tare, gms.	1165.230			1266.050
Moisture content: Dry soil+tare, gms.	939.050			1050.890
Moisture content: Tare, gms.	0.000			111.840
Moisture, %	24.1	23.5	23.5	22.9
Moist specimen weight, gms.	1165.2			
Diameter, in.	2.82	2.80	2.80	
Area, in.²	6.23	6.14	6.14	
Height, in.	5.76	5.71	5.71	
Net decrease in height, in.		0.04	0.00	
Wet density, pcf	123.7	125.9	125.9	
Dry density, pcf	99.7	102.0	102.0	
Void ratio	0.6596	0.6226	0.6226	
Saturation, %	96.8	100.0	100.0	

Test Readings for Specimen No. 1

Consolidation cell pressure = 55.00 psi (3.960 tsf)
Consolidation back pressure = 50.00 psi (3.600 tsf)
Consolidation effective confining stress = 0.360 tsf
Strain rate, %/min. = 0.07
Fail. Stress = 0.946 tsf **at reading no.** 48





	Sample No.	1	2	3
Initial	Water Content, %	33.7	33.3	29.1
	Dry Density, pcf	87.4	87.8	93.4
	Saturation, %	99.9	99.9	100.0
	Void Ratio	0.8927	0.8839	0.7711
	Diameter, in.	2.85	2.86	2.82
	Height, in.	5.71	5.65	5.82
At Test	Water Content, %	31.9	31.9	28.2
	Dry Density, pcf	89.6	89.7	94.7
	Saturation, %	100.0	100.0	100.0
	Void Ratio	0.8461	0.8441	0.7465
	Diameter, in.	2.82	2.84	2.81
	Height, in.	5.66	5.61	5.79
	Strain rate, %/min.	0.07	0.07	0.07
	Back Pressure, psi	50.00	45.00	40.00
	Cell Pressure, psi	60.00	60.00	60.00
	Fail. Stress, tsf	2.19	3.14	6.21
	Total Pore Pr., tsf	3.58	3.16	1.93
	Ult. Stress, tsf			
	Total Pore Pr., tsf			
	$\overline{\sigma}_1$ Failure, tsf	2.93	4.30	8.60
	$\overline{\sigma}_3$ Failure, tsf	0.74	1.16	2.39

Type of Test:
CU with Pore Pressures

Sample Type: Shelby tube

Description:

Assumed Specific Gravity= 2.65

Remarks:

Figure CU7211N

Client: Vectren

Project: Brown Safety Factor Assessment

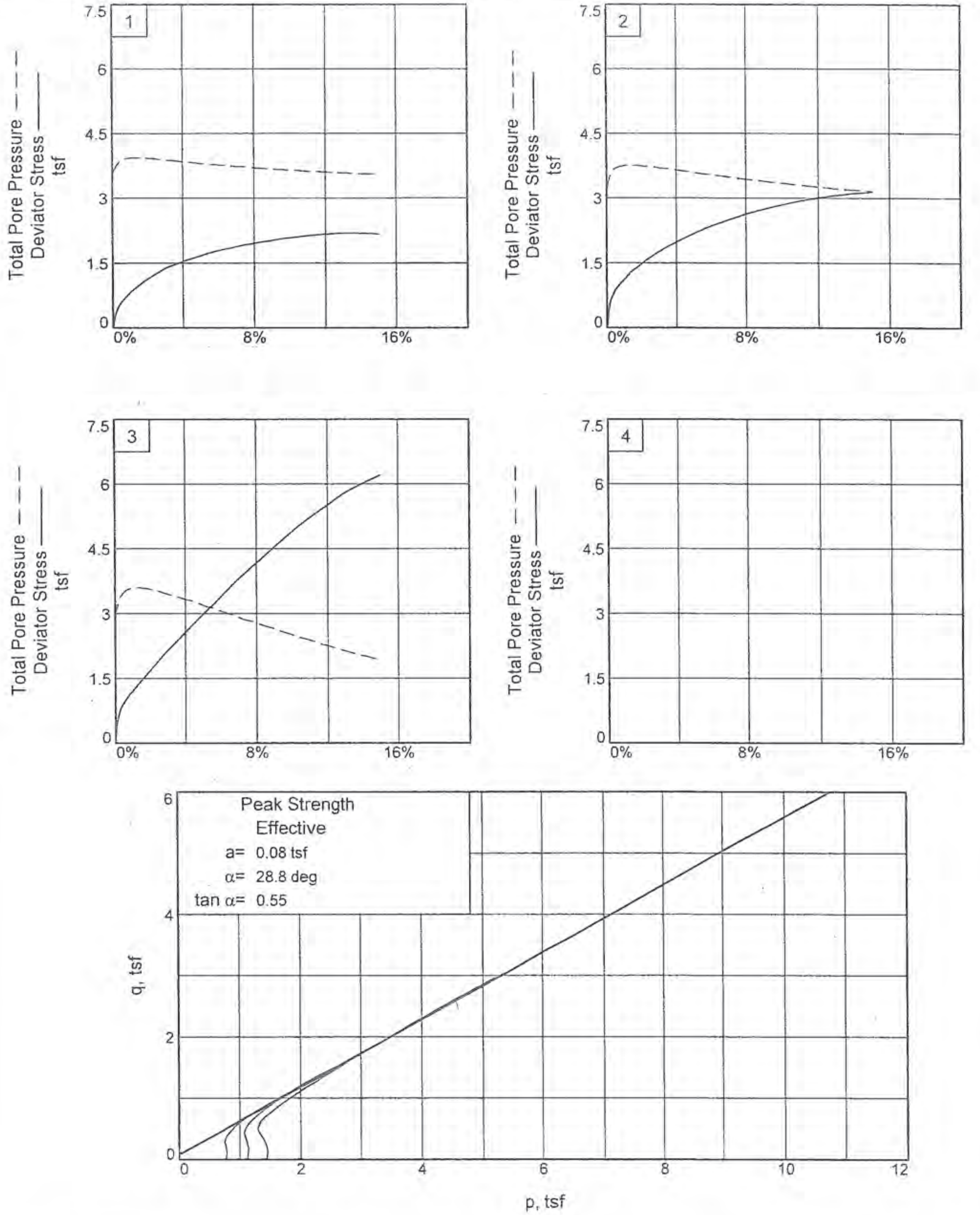
Source of Sample: 7211 **Depth:** 35-37'

Sample Number: B-207

Proj. No.: 170GC00108 **Date Sampled:**

TRIAXIAL SHEAR TEST REPORT
Cardno ATC, INC.
Indianapolis, Indiana

Tested By: MDr



Client: Vectren

Project: Brown Safety Factor Assessment

Source of Sample: 7211 **Depth:** 35-37' **Sample Number:** B-207

Project No.: 170GC00108 **Figure** ________

Cardno ATC, Inc.

Tested By: MDr

TRIAXIAL COMPRESSION TEST
CU with Pore Pressures

6/18/2015
1:43 PM

Date:
Client: Vectren
Project: Brown Safety Factor Assessment
Project No.: 170GC00108
Location: 7211
Depth: 35-37' **Sample Number:** B-207
Description:
Remarks:
Type of Sample: Shelby tube
Assumed Specific Gravity=2.65 **LL=** **PL=** **PI=**
Test Method: COE uniform strain

Parameters for Specimen No. 1

Specimen Parameter	Initial	Saturated	Consolidated	Final
Moisture content: Moist soil+tare, gms.	1114.800			1213.360
Moisture content: Dry soil+tare, gms.	834.020			941.580
Moisture content: Tare, gms.	0.000			107.560
Moisture, %	33.7	32.4	31.9	32.6
Moist specimen weight, gms.	1114.8			
Diameter, in.	2.85	2.83	2.82	
Area, in.^{2}	6.37	6.29	6.27	
Height, in.	5.71	5.67	5.66	
Net decrease in height, in.		0.03	0.01	
Wet density, pcf	116.8	117.8	118.2	
Dry density, pcf	87.4	89.0	89.6	
Void ratio	0.8927	0.8590	0.8461	
Saturation, %	99.9	100.0	100.0	

Test Readings for Specimen No. 1

Consolidation cell pressure = 60.00 psi (4.320 tsf)
Consolidation back pressure = 50.00 psi (3.600 tsf)
Consolidation effective confining stress = 0.720 tsf
Strain rate, %/min. = 0.07
Fail. Stress = 2.193 tsf **at reading no.** 43

Soil Hydraulic Conductivity Laboratory Test Results

Client:	AECOM		
Project Name:	Vectran AB Brown Ash Pond Lower Dam		
Project Location:	Evansville, IN		
GTX #:	303915		
Start Date:	2/25/2016	Tested By:	jcw
End Date:	2/29/2016	Checked By:	emm
Boring #:	AECOM-B1		
Sample #:	1		
Depth:	17-19		
Visual Description:	Moist, dark yellowish brown clay		

Hydraulic Conductivity of Saturated Porous Materials Using a Flexible Wall Permeameter by ASTM D5084

Constant Volume

Sample Type:	Intact	Permeant Fluid:	De-aired Distilled water
Orientation:	Vertical	Cell #:	8/13

Sample Preparation: Extruded from tube, cut, trimmed and placed into permeameter at as-received density and moisture content. Trimmings moisture content = 18.1%.

Assumed Specific Gravity: 2.70

Parameter	Initial	Final
Height, in	3.30	3.21
Diameter, in	2.85	2.85
Area, in^2	6.38	6.38
Volume, in^3	21.1	20.5
Mass, g	695	705
Bulk Density, pcf	125.5	130.9
Moisture Content, %	18.0	19.7
Dry Density, pcf	106.4	109.3
Degree of Saturation, %	83	98

B COEFFICIENT DETERMINATION

Cell Pressure, psi:	89.96	Increased Cell Pressure, psi:	95.04	Cell Pressure Increment, psi:	5.08
Sample Pressure, psi:	84.95	Corresponding Sample Pressure, psi:	89.86	Sample Pressure Increment, psi:	4.91
				B Coefficient:	0.97

FLOW DATA

Date	Trial #	Pressure, psi		Manometer Readings			Elapsed Time, sec	Gradient	Permeability K, cm/sec	Temp, °C	R_t	Permeability K @ 20 °C, cm/sec
		Cell	Sample	Z_1	Z_2	Z_1-Z_2						
2/26	1	90.0	85.0	12.5	12.0	0.5	30	19.3	6.7E-07	20.4	0.991	6.7E-07
2/26	2	90.0	85.0	12.5	12.0	0.5	32	19.3	6.3E-07	20.4	0.991	6.2E-07
2/26	3	90.0	85.0	12.5	12.0	0.5	33	19.3	6.1E-07	20.4	0.991	6.1E-07
2/26	4	90.0	85.0	12.5	12.0	0.5	34	19.3	5.9E-07	20.4	0.991	5.9E-07

PERMEABILITY AT 20° C: 6.2 x 10^{-7} cm/sec (@ 5 psi effective stress)

Client:	AECOM		
Project Name:	Vectran AB Brown Ash Pond Lower Dam		
Project Location:	Evansville, IN		
GTX #:	303915		
Start Date:	2/26/2016	Tested By:	jcw
End Date:	3/1/2016	Checked By:	emm
Boring #:	AECOM-B1		
Sample #:	5		
Depth:	49-51		
Visual Description:	Moist, dark olive brown clay		

Hydraulic Conductivity of Saturated Porous Materials Using a Flexible Wall Permeameter by ASTM D5084

Constant Volume

Sample Type:	Intact	Permeant Fluid:	De-aired Distilled water
Orientation:	Vertical	Cell #:	9/15

Sample Preparation: Extruded from tube, cut, trimmed and placed into permeameter at as-received density and moisture content. Trimmings moisture content = 26.7%.

Assumed Specific Gravity: 2.70

Parameter	Initial	Final
Height, in	2.96	2.93
Diameter, in	2.85	2.85
Area, in^2	6.38	6.38
Volume, in^3	18.9	18.7
Mass, g	610	603
Bulk Density, pcf	122.7	122.6
Moisture Content, %	26.8	25.4
Dry Density, pcf	96.8	97.8
Degree of Saturation, %	98	95

B COEFFICIENT DETERMINATION

Cell Pressure, psi:	89.97	Increased Cell Pressure, psi:	94.87	Cell Pressure Increment, psi:	4.90
Sample Pressure, psi:	84.95	Corresponding Sample Pressure, psi:	89.58	Sample Pressure Increment, psi:	4.63
				B Coefficient:	0.95

FLOW DATA

Date	Trial #	Pressure, psi		Manometer Readings			Elapsed Time, sec	Gradient	Permeability K, cm/sec	Temp, °C	R_t	Permeability K @ 20 °C, cm/sec
		Cell	Sample	Z_1	Z_2	Z_1-Z_2						
2/29	1	90.0	85.0	11.5	11.2	0.3	45	19.5	2.6E-07	20.7	0.983	2.6E-07
2/29	2	90.0	85.0	11.5	11.2	0.3	46	19.5	2.6E-07	20.7	0.983	2.5E-07
2/29	3	90.0	85.0	11.5	11.2	0.3	46	19.5	2.6E-07	20.7	0.983	2.5E-07
2/29	4	90.0	85.0	11.5	11.2	0.3	47	19.5	2.5E-07	20.7	0.983	2.5E-07

PERMEABILITY AT 20° C: 2.6 x 10^{-7} cm/sec (@ 5 psi effective stress)

GeoTesting EXPRESS

Client:	AECOM		
Project Name:	Vectran AB Brown Ash Pond Lower Dam		
Project Location:	Evansville, IN		
GTX #:	303915		
Start Date:	2/26/2016	Tested By:	jcw
End Date:	3/1/2016	Checked By:	emm
Boring #:	AECOM-B2		
Sample #:	4		
Depth:	60-62		
Visual Description:	Moist, light brown silt		

Hydraulic Conductivity of Saturated Porous Materials Using a Flexible Wall Permeameter by ASTM D5084

Constant Volume

Sample Type:	Intact	Permeant Fluid:	De-aired Distilled water
Orientation:	Vertical	Cell #:	6/7

Sample Preparation: Extruded from tube, cut, trimmed and placed into permeameter at as-received density and moisture content. Trimmings moisture content = 23.7%.

Assumed Specific Gravity: 2.70

Parameter	Initial	Final
Height, in	2.37	2.34
Diameter, in	2.85	2.85
Area, in^2	6.38	6.38
Volume, in^3	15.1	14.9
Mass, g	497	492
Bulk Density, pcf	125.0	125.3
Moisture Content, %	24.2	23.0
Dry Density, pcf	100.6	101.9
Degree of Saturation, %	97	95

B COEFFICIENT DETERMINATION

Cell Pressure, psi:	90.04	Increased Cell Pressure, psi:	94.91	Cell Pressure Increment, psi:	4.87
Sample Pressure, psi:	84.97	Corresponding Sample Pressure, psi:	89.72	Sample Pressure Increment, psi:	4.75
				B Coefficient:	0.97

FLOW DATA

Date	Trial #	Pressure, psi		Manometer Readings			Elapsed Time, sec	Gradient	Permeability K, cm/sec	Temp, °C	R_t	Permeability K @ 20 °C, cm/sec
		Cell	Sample	Z_1	Z_2	Z_1-Z_2						
2/29	1	90.0	85.0	5.0	4.6	0.4	33	10.6	9.1E-07	20.7	0.983	9.0E-07
2/29	2	90.0	85.0	5.0	4.6	0.4	34	10.6	8.8E-07	20.7	0.983	8.7E-07
2/29	3	90.0	85.0	5.0	4.6	0.4	34	10.6	8.8E-07	20.7	0.983	8.7E-07
2/29	4	90.0	85.0	5.0	4.6	0.4	35	10.6	8.6E-07	20.7	0.983	8.4E-07

PERMEABILITY AT 20° C: 8.7 x 10^{-7} cm/sec (@ 5 psi effective stress)

GeoTesting EXPRESS

Client:	AECOM		
Project Name:	Vectran AB Brown Ash Pond Lower Dam		
Project Location:	Evansville, IN		
GTX #:	303915		
Start Date:	2/26/2016	Tested By:	jcw
End Date:	3/1/2016	Checked By:	emm
Boring #:	AECOM-B3		
Sample #:	3		
Depth:	28-30		
Visual Description:	Moist, dark yellowish brown clay		

Hydraulic Conductivity of Saturated Porous Materials Using a Flexible Wall Permeameter by ASTM D5084

Constant Volume

Sample Type:	Intact	Permeant Fluid:	De-aired Distilled water
Orientation:	Vertical	Cell #:	15/4

Sample Preparation: Extruded from tube, cut, trimmed and placed into permeameter at as-received density and moisture content. Trimmings moisture content = 21.3%.

Assumed Specific Gravity: 2.70

Parameter	Initial	Final
Height, in	3.03	3.00
Diameter, in	2.85	2.85
Area, in^2	6.38	6.38
Volume, in^3	19.3	19.1
Mass, g	646	644
Bulk Density, pcf	127.0	127.9
Moisture Content, %	21.2	20.8
Dry Density, pcf	104.8	105.8
Degree of Saturation, %	94	95

B COEFFICIENT DETERMINATION

Cell Pressure, psi:	89.98	Increased Cell Pressure, psi:	94.90	Cell Pressure Increment, psi:	4.92
Sample Pressure, psi:	84.95	Corresponding Sample Pressure, psi:	89.65	Sample Pressure Increment, psi:	4.70
				B Coefficient:	0.96

FLOW DATA

Date	Trial #	Pressure, psi		Manometer Readings			Elapsed Time, sec	Gradient	Permeability K, cm/sec	Temp, °C	R_t	Permeability K @ 20 °C, cm/sec
		Cell	Sample	Z_1	Z_2	Z_1-Z_2						
2/29	1	90.0	85.0	11.5	11.1	0.4	34	19.0	4.8E-07	20.7	0.983	4.7E-07
2/29	2	90.0	85.0	11.5	11.1	0.4	38	19.0	4.3E-07	20.7	0.983	4.2E-07
2/29	3	90.0	85.0	11.5	11.1	0.4	40	19.0	4.1E-07	20.7	0.983	4.0E-07
2/29	4	90.0	85.0	11.5	11.1	0.4	43	19.0	3.8E-07	20.7	0.983	3.7E-07

PERMEABILITY AT 20° C: 4.2 x 10^{-7} cm/sec (@ 5 psi effective stress)

Client:	AECOM		
Project Name:	Vectran AB Brown Ash Pond Lower Dam		
Project Location:	Evansville, IN		
GTX #:	303915		
Start Date:	2/26/2016	Tested By:	jcw
End Date:	3/1/2016	Checked By:	emm
Boring #:	AECOM-B3		
Sample #:	1		
Depth:	8-10		
Visual Description:	Moist, yellowish brown silt		

Hydraulic Conductivity of Saturated Porous Materials Using a Flexible Wall Permeameter by ASTM D5084

Constant Volume

Sample Type:	Intact	Permeant Fluid:	De-aired Distilled water
Orientation:	Vertical	Cell #:	8/13

Sample Preparation: Extruded from tube, cut, trimmed and placed into permeameter at as-received density and moisture content. Trimmings moisture content = 31.4%.

Assumed Specific Gravity: 2.70

Parameter	Initial	Final
Height, in	2.84	2.73
Diameter, in	2.85	2.85
Area, in^2	6.38	6.38
Volume, in^3	18.1	17.4
Mass, g	548	543
Bulk Density, pcf	115.0	118.6
Moisture Content, %	30.6	29.5
Dry Density, pcf	88.0	91.6
Degree of Saturation, %	90	95

B COEFFICIENT DETERMINATION

Cell Pressure, psi:	89.96	Increased Cell Pressure, psi:	94.89	Cell Pressure Increment, psi:	4.93
Sample Pressure, psi:	84.95	Corresponding Sample Pressure, psi:	89.77	Sample Pressure Increment, psi:	4.82
				B Coefficient:	0.98

FLOW DATA

	Trial	Pressure, psi		Manometer Readings			Elapsed Time,		Permeability K,	Temp,		Permeability K @ 20 °C,
Date	#	Cell	Sample	Z_1	Z_2	Z_1-Z_2	sec	Gradient	cm/sec	°C	R_t	cm/sec
2/29	1	90.0	85.0	5.5	3.5	2.0	35	10.0	5.5E-06	20.7	0.983	5.4E-06
2/29	2	90.0	85.0	5.5	3.5	2.0	36	10.0	5.3E-06	20.7	0.983	5.2E-06
2/29	3	90.0	85.0	5.5	3.5	2.0	37	10.0	5.2E-06	20.7	0.983	5.1E-06
2/29	4	90.0	85.0	5.5	3.5	2.0	37	10.0	5.2E-06	20.7	0.983	5.1E-06

PERMEABILITY AT 20° C: 5.2 x 10^{-6} cm/sec (@ 5 psi effective stress)

Client:	AECOM
Project Name:	Vectran AB Brown Ash Pond Lower Dam
Project Location:	Evansville, IN
GTX #:	303915

Start Date:	2/26/2016	Tested By:	jcw
End Date:	3/1/2016	Checked By:	emm

Boring #:	AECOM-B5
Sample #:	3
Depth:	34-36
Visual Description:	Moist, gray silty clay

Hydraulic Conductivity of Saturated Porous Materials Using a Flexible Wall Permeameter by ASTM D5084

Constant Volume

Sample Type:	Intact	Permeant Fluid:	De-aired Distilled water
Orientation:	Vertical	Cell #:	2/5

Sample Preparation: Extruded from tube, cut, trimmed and placed into permeameter at as-received density and moisture content. Trimmings moisture content = 39.7%.

Assumed Specific Gravity: 2.70

Parameter	Initial	Final
Height, in	2.53	2.51
Diameter, in	2.85	2.85
Area, in^2	6.38	6.38
Volume, in^3	16.1	16.0
Mass, g	473	468
Bulk Density, pcf	111.4	111.2
Moisture Content, %	39.9	38.5
Dry Density, pcf	79.6	80.3
Degree of Saturation, %	97	95

B COEFFICIENT DETERMINATION

Cell Pressure, psi:	90.03	Increased Cell Pressure, psi:	94.93	Cell Pressure Increment, psi:	4.90
Sample Pressure, psi:	84.95	Corresponding Sample Pressure, psi:	89.72	Sample Pressure Increment, psi:	4.77
				B Coefficient:	0.97

FLOW DATA

Date	Trial #	Pressure, psi		Manometer Readings			Elapsed Time, sec	Gradient	Permeability K, cm/sec	Temp, °C	R_t	Permeability K @ 20 °C, cm/sec
		Cell	Sample	Z_1	Z_2	Z_1-Z_2						
2/29	1	90.0	85.0	4.5	3.0	1.5	20	8.9	7.9E-06	20.7	0.983	7.8E-06
2/29	2	90.0	85.0	4.5	3.0	1.5	20	8.9	7.9E-06	20.7	0.983	7.8E-06
2/29	3	90.0	85.0	4.5	3.0	1.5	20	8.9	7.9E-06	20.7	0.983	7.8E-06
2/29	4	90.0	85.0	4.5	3.0	1.5	20	8.9	7.9E-06	20.7	0.983	7.8E-06

PERMEABILITY AT 20° C: 7.8 x 10^{-6} cm/sec (@ 5 psi effective stress)

Soil Cyclic Direct Simple Shear Laboratory Test Results

Consolidated Undrained Cyclic Direct Simple Shear Test of Cohesive Soils

Client:	AECOM	GTX#:	303915
Project Name:	Vectran AB Brown Ash Pond Lower Dam	Test Date:	10/28/15
Project Location:	Evansville, IN		

Boring ID: AECOM-B1
Sample ID: 3
Depth, ft: 31-41

Visual Description: Moist, greenish brown silt with clay

Test Equipment: Top and bottom box (circular) = 2.5 in diameter. Load cells and LVDT's connected to data acquisition system for shear force, normal load, horizontal and vertical displacement; surface area = 4.91 in^2, soil height = 1 inch.
Stacked Teflon Rings set-up used, which included porous stones with pins.

Test Condition: Inundated prior to consolidation.

Sample Type and Preparation: Extruded from tube, cut, trimmed and placed into apparatus at as-received density and moisture content.

Parameter	Point 1	Point 2	Point 3	Point 4	Point 5
Test No.	CDSS-1A				
Initial Moisture Content, %	30.8				
Initial Dry Density, pcf	89.5				
Vertical Consolidation Stress, psf	4275				
Cyclic Stress Ratio	0.25				
Number of cycles completed	21				
Frequency, Hz	1				
Final Moisture Content, %	25.4				
Measured Post-Cyclic Peak Shear Stress, psf	---				
Shear Strain at Post-Cyclic Peak shear Stress, %	---				
Membrane Correction, psf	---				
Corrected Post-Cyclic Peak Shear Stress, psf	---				
S_r/σ'_{vc}	---				

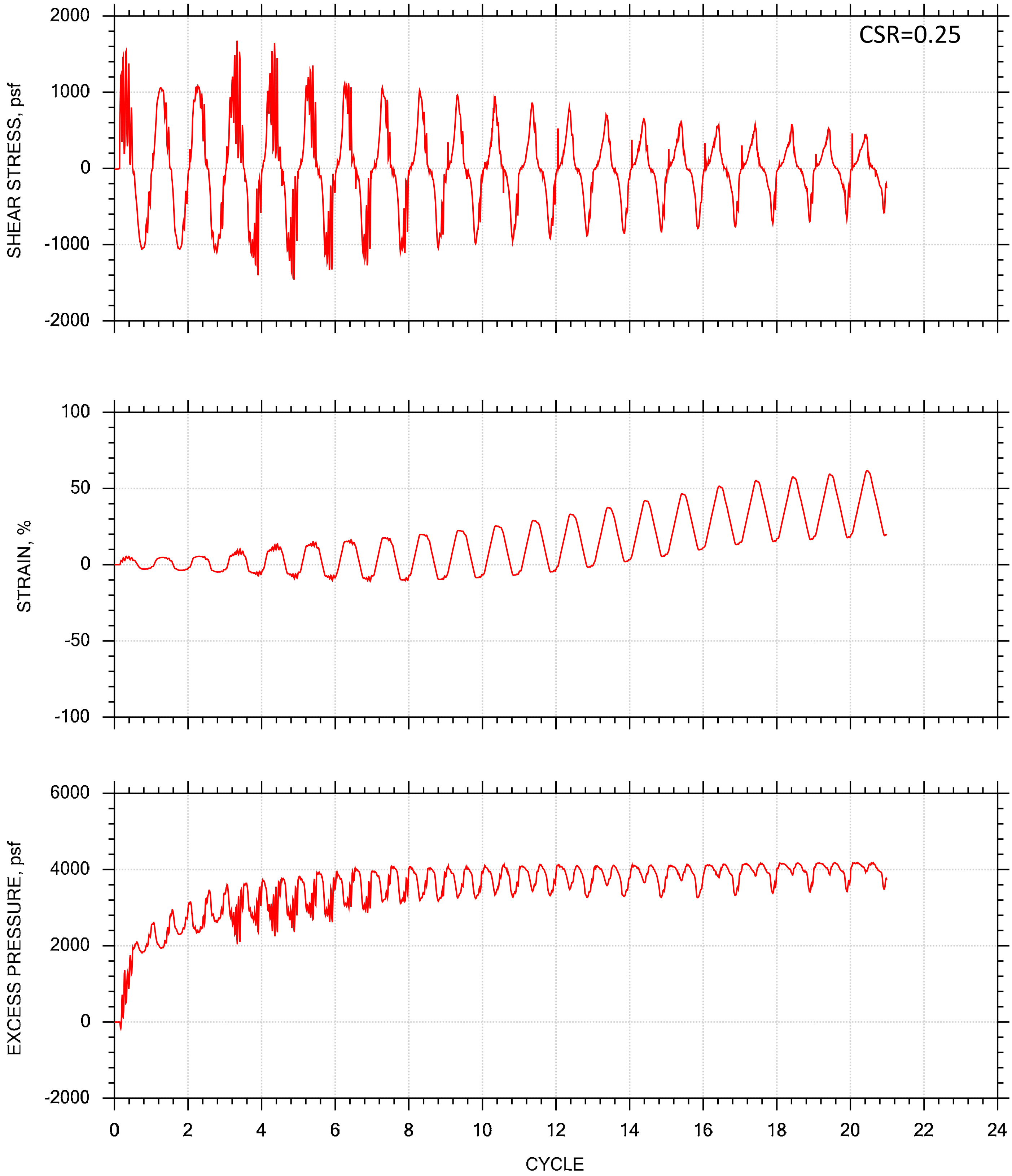
Comments: 500 cycles were requested. Specimen reached a 40% peak-to-peak strain, which is excessive, at 21 cycles which terminated the test. Shear strains higher than 10% peak-to-peak caused the sample to drift and the equipment had trouble keeping up with the target loading. There was no strength left to measure in the post cyclic condition.

Tested By: md/njh Checked By: jdt

Notes: These results apply only to the sample tested for the specific test conditions. The test procedures employed follow accepted industry practice and the indicated test method. GeoTesting Express has no specific knowledge as to conditioning, origin, sampling procedure or intended use of the material.

CYCLIC SIMPLE SHEAR DATA

Step 1 of **1**

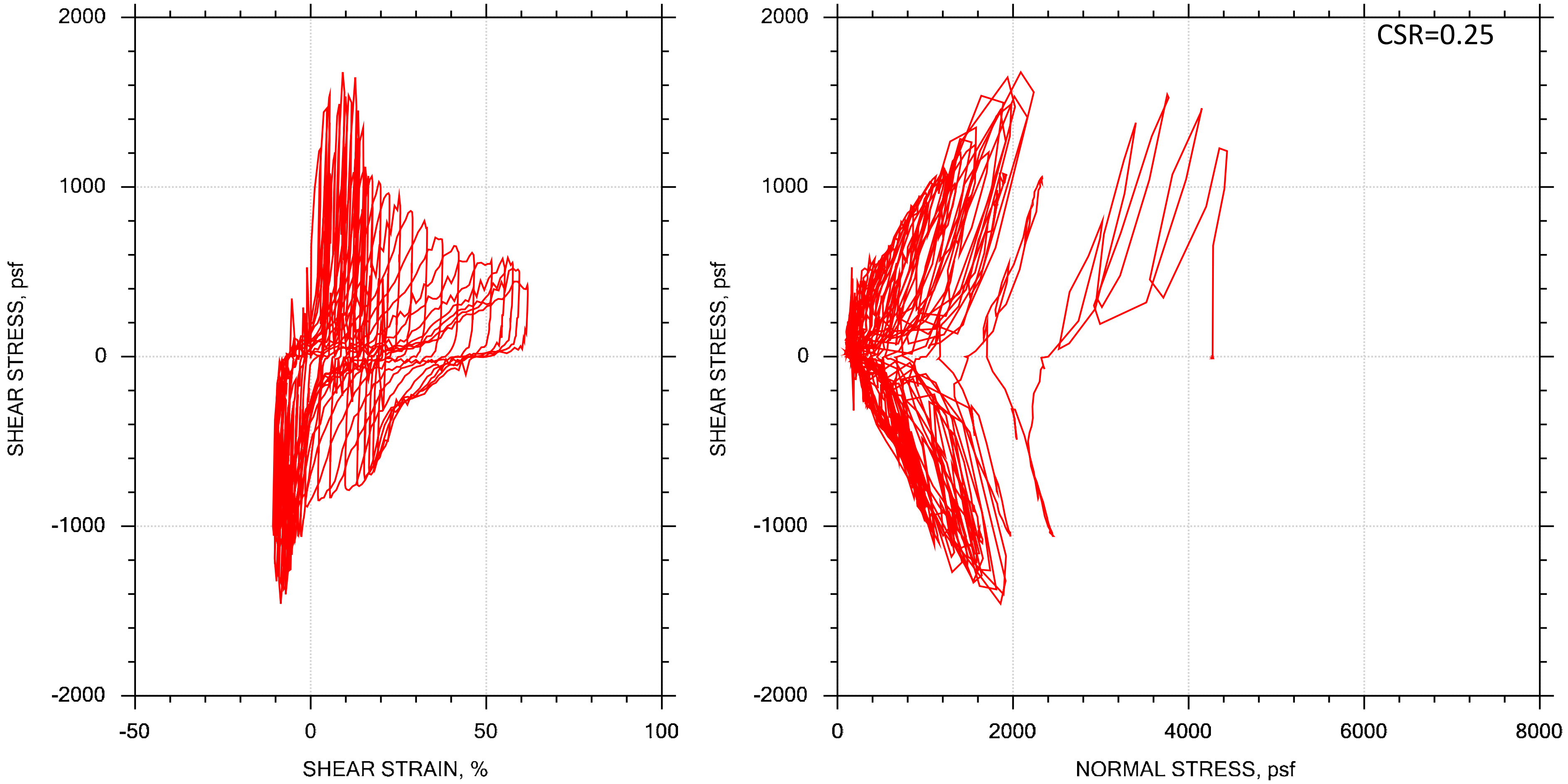


GeoTesting EXPRESS	Project: Vectran AB Brown Ash Pond Lower	Location: Evansville, IN	Project No.: GTX-303915
	Boring No.: AECOM-B1	Tested By: md/njh	Checked By: jdt
	Sample No.: 3	Test Date: 10/30/15	Test No.: CDSS-1A
	Depth: 31-41 ft	Sample Type: intact	Elevation: ---
	Description: Moist greenish Brown silt with clay		
	Remarks: System GG		Page 2 of 3
	File: \\hal1\Projects\GTX303915\6 Lab Testing\Soil\CDSS\303915-CDSS-1A.dat		

CYCLIC SIMPLE SHEAR STRESS DATA

Step 1 of **1**

Cycle: 0.0 to 21.0



GeoTesting EXPRESS	Project: Vectran AB Brown Ash Pond Lower	Location: Evansville, IN	Project No.: GTX-303915
	Boring No.: AECOM-B1	Tested By: md/njh	Checked By: jdt
	Sample No.: 3	Test Date: 10/30/15	Depth: 31-41 ft
	Test No.: CDSS-1A	Sample Type: intact	Elevation: ---
	Description: Moist greenish Brown silt with clay		
	Remarks: System GG		Page 3 of 3
	File: \\hal1\Projects\GTX303915\6 Lab Testing\Soil\CDSS\303915-CDSS-1A.dat		

Consolidated Undrained Cyclic Direct Simple Shear Test of Cohesive Soils

Client: AECOM
Project Name: Vectran AB Brown Ash Pond Lower Dam
Project Location: Evansville, IN

GTX#: 303915
Test Date: 11/18/15

Boring ID: AECOM-B2
Sample ID: 3
Depth, ft: 56-58

Visual Description: Moist, brown silt

Test Equipment: Top and bottom box (circular) = 2.5 in diameter. Load cells and LVDT's connected to data acquisition system for shear force, normal load, horizontal and vertical displacement; surface area = 4.91 in^2, soil height = 1 inch.
Stacked Teflon Rings set-up used, which included porous stones with pins.

Test Condition: Inundated prior to consolidation.

Sample Type and Preparation: Extruded from tube, cut, trimmed and placed into apparatus at as-received density and moisture content.

Parameter	Point 1	Point 2	Point 3	Point 4	Point 5
Test No.	CDSS-2A				
Initial Moisture Content, %	23.3				
Initial Dry Density, pcf	99.2				
Vertical Consolidation Stress, psf	4950				
Cyclic Stress Ratio	0.15				
Number of cycles completed	29				
Frequency, Hz	1				
Final Moisture Content, %	23.5				
Delay before shearing, min	60				
Nominal Rate of Shear Strain, %/hr	5.0				
Measured Post-Cyclic Peak Shear Stress, psf	2918				
Shear Strain at Post-Cyclic Peak shear Stress, %	20.0				
Membrane Correction, psf	49				
Corrected Post-Cyclic Peak Shear Stress, psf	2869				
S_r/σ'_{vc}	0.58				

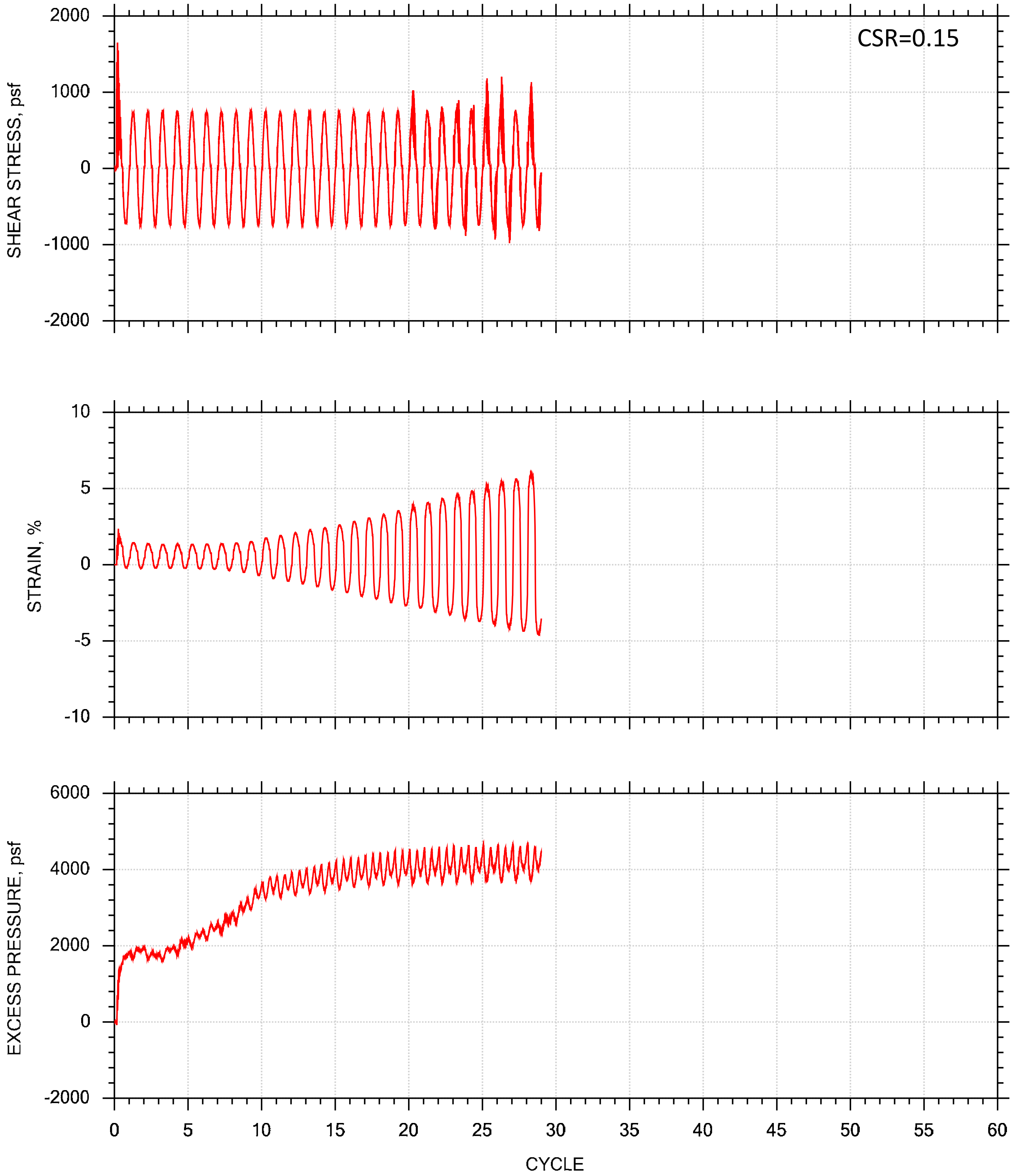
Comments: The cyclic portion of the test resulted in an R value approaching 1, and terminated the test at a 10% peak-to-peak axial strain. Actual post cyclic strength parameters should be determined by an engineer familiar with dynamic testing data.

Tested By: md

Checked By: jdt

CYCLIC SIMPLE SHEAR DATA

Step 1 of **1**

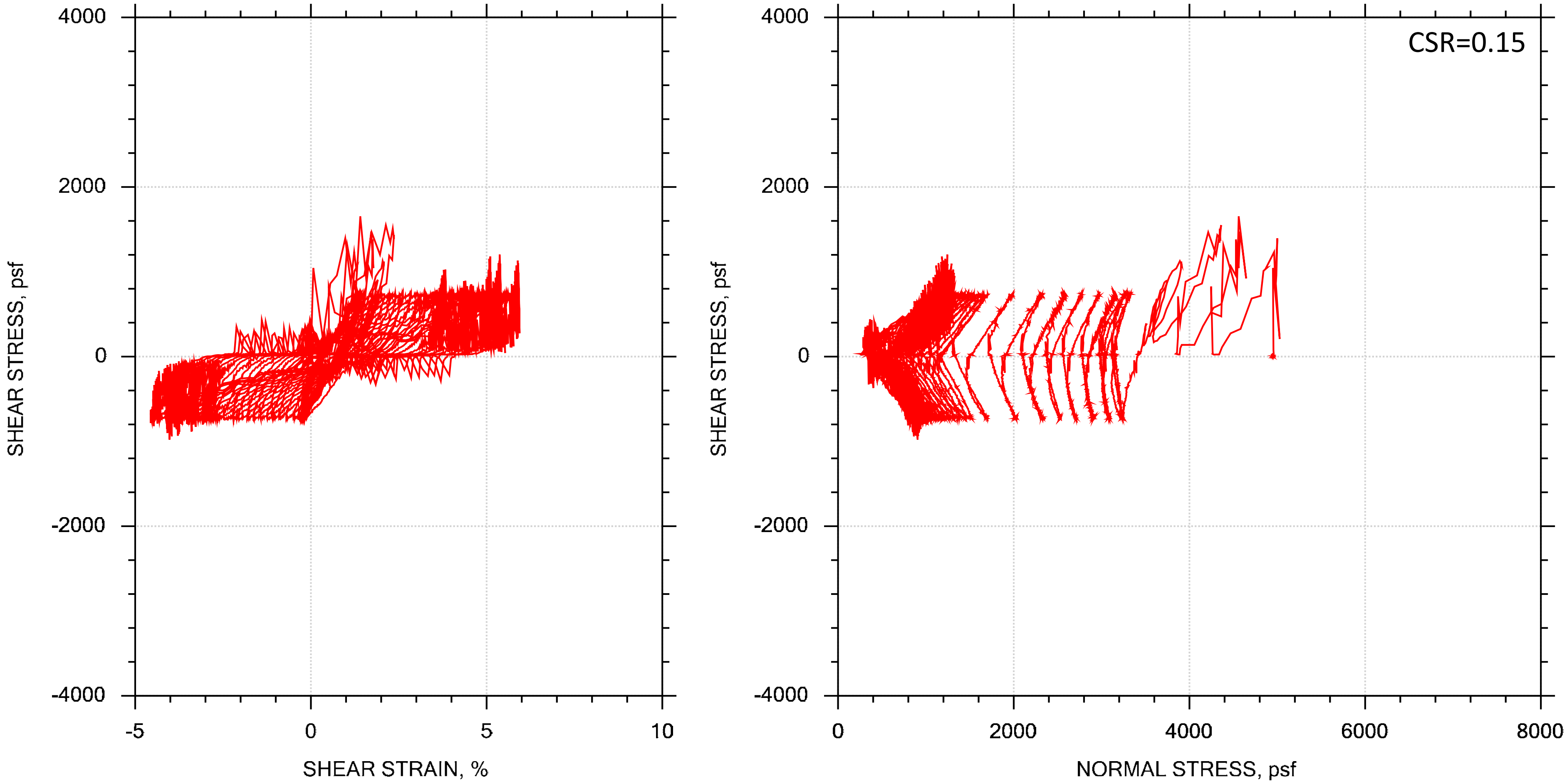


GeoTesting EXPRESS	Project: Vectran AB Brown Ash Pond	Location: Evansville, IN	Project No.: GTX-303915
	Boring No.: AECOM-B2	Tested By: md	Checked By: jdt
	Sample No.: 3	Test Date: 11/23/15	Test No.: CDSS-2A
	Depth: 56-58 ft	Sample Type: intact	Elevation: ---
	Description: Moist, brown silt		
	Remarks: System GG		
	File: \\hal1\Projects\GTX303915 - AECOM - Vectran AB Brown Ash Pond\6 Lab Testing\Soil\CDSS\303915-CDSS-2A.dat		

CYCLIC SIMPLE SHEAR STRESS DATA

Step 1 of **1**

Cycle: 0.0 to 29.0



GeoTesting EXPRESS	Project: Vectran AB Brown Ash Pond	Location: Evansville, IN	Project No.: GTX-303915
	Boring No.: AECOM-B2	Tested By: md	Checked By: jdt
	Sample No.: 3	Test Date: 11/23/15	Depth: 56-58 ft
	Test No.: CDSS-2A	Sample Type: intact	Elevation: ---
	Description: Moist, brown silt		
	Remarks: System GG		Page 3 of 5
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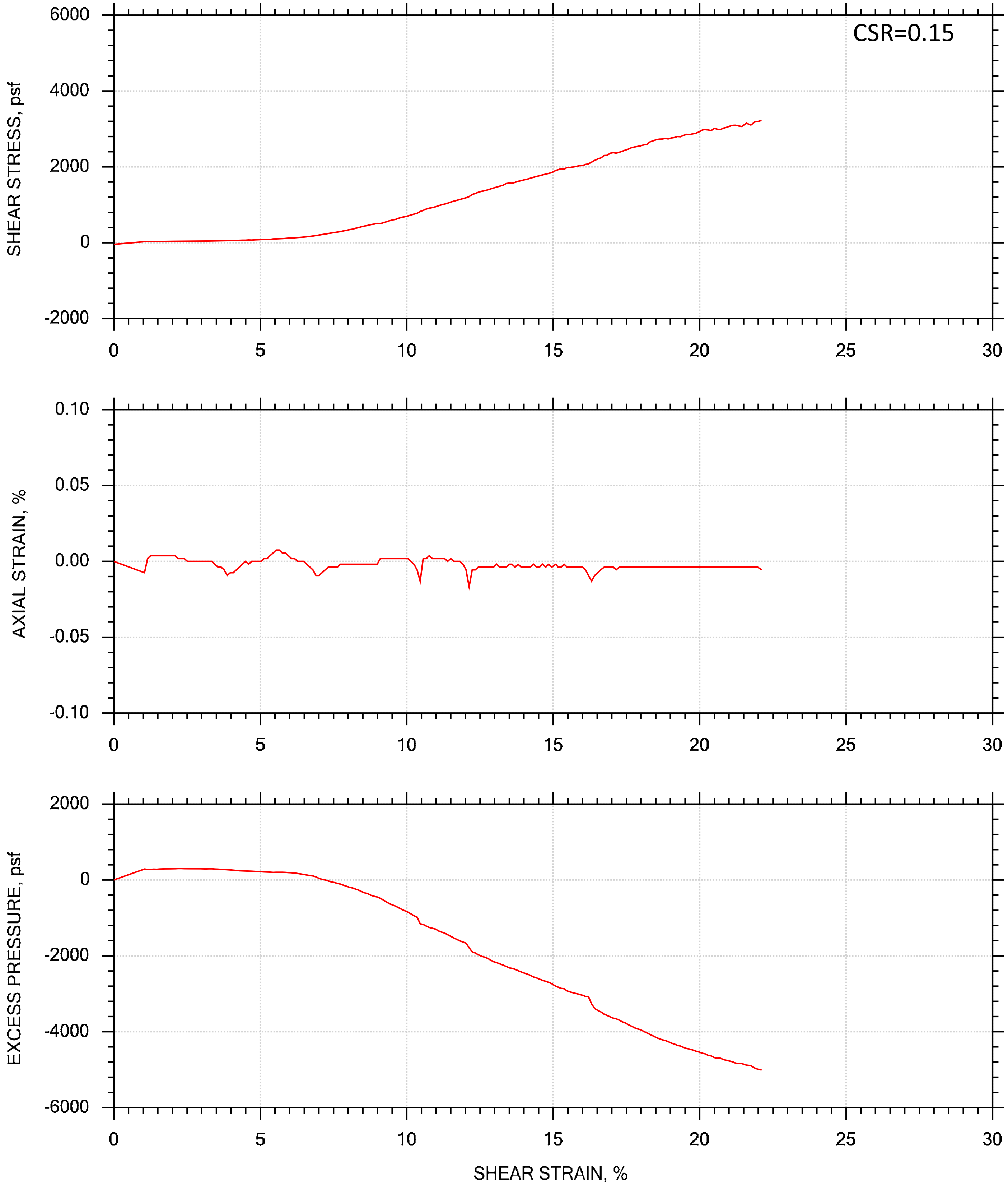
DIRECT SIMPLE SHEAR TEST by ASTM D6528

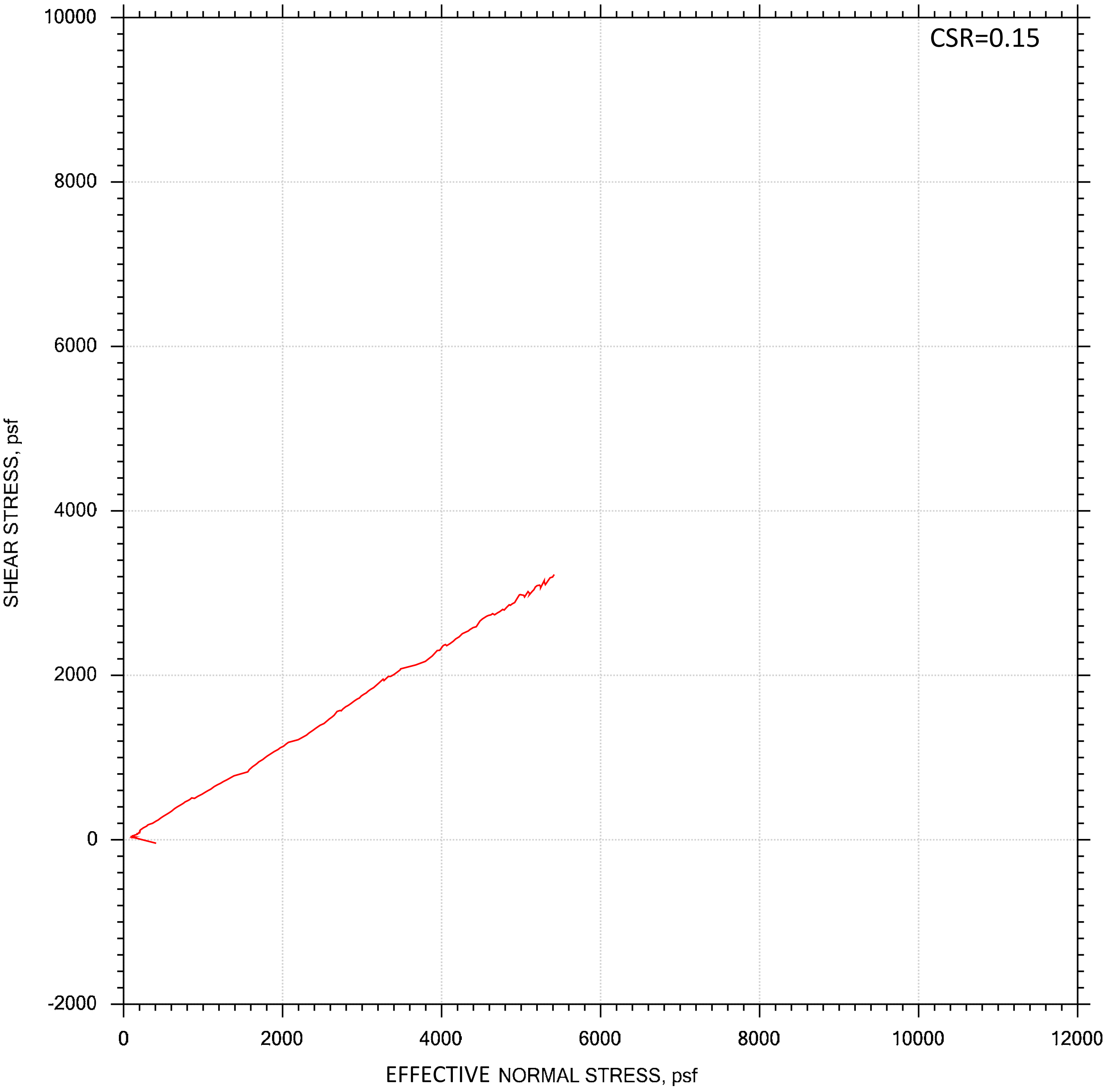
POST CYCLIC



Project: Vectran AB Brown Ash Pond	Location: Evansville, IN	Project No.: GTX-303915
Boring No.: AECOM-B2	Tested By: md	Checked By: jdt
Sample No.: 3	Test Date: 11/23/15	Test No.: CDSS-2A
Depth: 56-58 ft	Sample Type: intact	Elevation: ---
Description: Moist, brown silt		
Remarks: System GG		Page 4 of 5
File: \\hal1\Projects\GTX303915 - AECOM - Vectran AB Brown Ash Pond\6 Lab Testing\Soil\CDSS\303915-CDSS-2A.dat		

DIRECT SIMPLE SHEAR TEST by ASTM D6528

POST CYCLIC



GeoTesting EXPRESS	Project: Vectran AB Brown Ash Pond	Location: Evansville, IN	Project No.: GTX-303915
	Boring No.: AECOM-B2	Tested By: md	Checked By: jdt
	Sample No.: 3	Test Date: 11/23/15	Test No.: CDSS-2A
	Depth: 56-58 ft	Sample Type: intact	Elevation: ---
	Description: Moist, brown silt		
	Remarks: System GG		Page 5 of 5
	File: \\hal1\Projects\GTX303915 - AECOM - Vectran AB Brown Ash Pond\6 Lab Testing\Soil\CDSS\303915-CDSS-2A.dat		

Consolidated Undrained Cyclic Direct Simple Shear Test of Cohesive Soils

Client:	AECOM	GTX#:	303915
Project Name:	Vectran AB Brown Ash Pond Lower Dam	Test Date:	11/20/15
Project Location:	Evansville, IN		

Boring ID: AECOM-B2
Sample ID: 4A
Depth, ft: 62-64

Visual Description: Moist, gray silt

Test Equipment: Top and bottom box (circular) = 2.5 in diameter. Load cells and LVDT's connected to data acquisition system for shear force, normal load, horizontal and vertical displacement; surface area = 4.91 in^2, soil height = 1 inch.
Stacked Teflon Rings set-up used, which included porous stones with pins.

Test Condition: Inundated prior to consolidation.
Sample Type and Preparation: Extruded from tube, cut, trimmed and placed into apparatus at as-received density and moisture content.

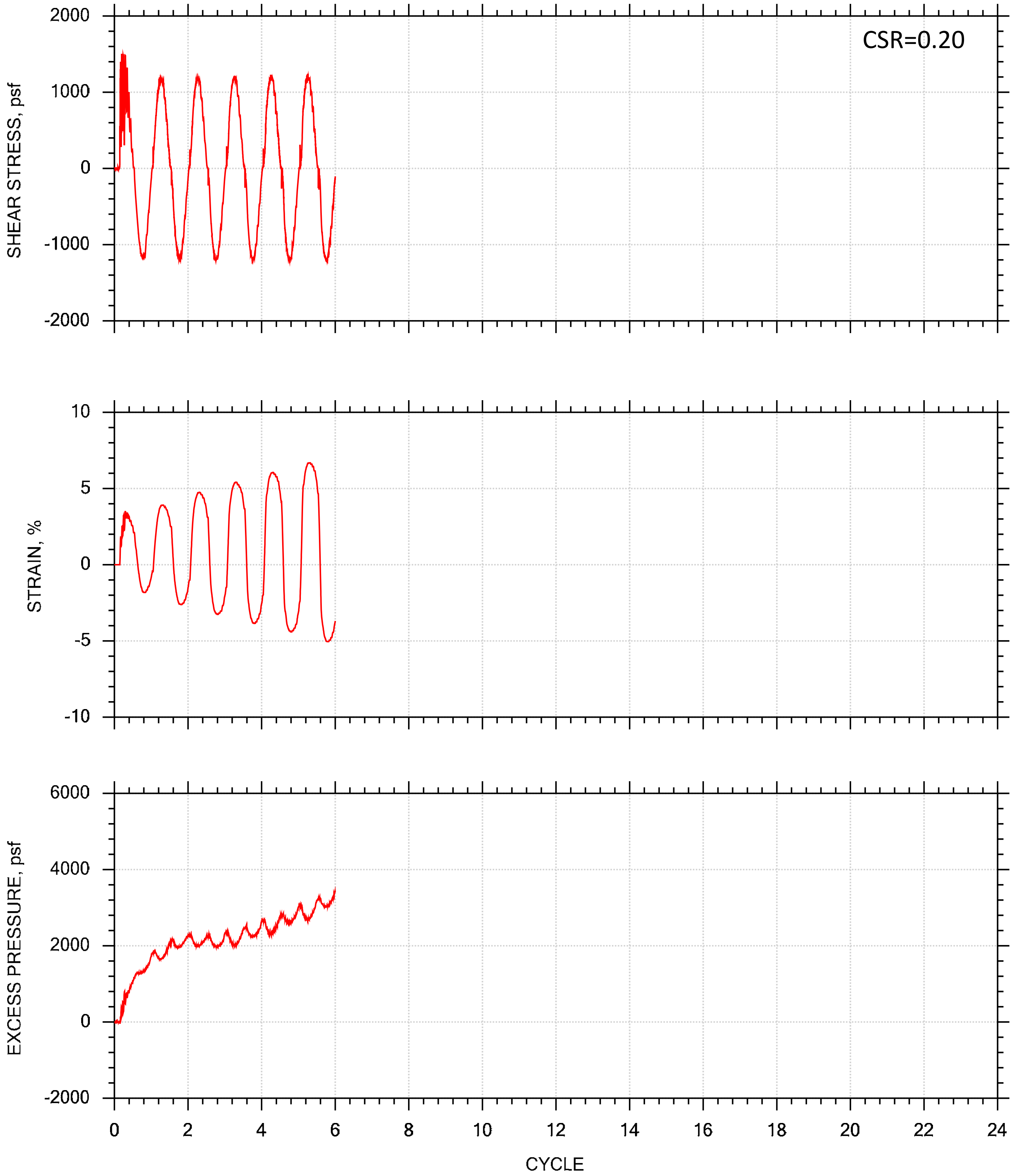
Parameter	Point 1	Point 2	Point 3	Point 4	Point 5
Test No.	CDSS-5				
Initial Moisture Content, %	24.5				
Initial Dry Density, pcf	99.0				
Vertical Consolidation Stress, psf	6040				
Cyclic Stress Ratio	0.20				
Number of cycles completed	6				
Frequency, Hz	1				
Final Moisture Content, %	22.6				
Delay before shearing, min	60				
Nominal Rate of Shear Strain, %/hr	5.0				
Measured Post-Cyclic Peak Shear Stress, psf	2215				
Shear Strain at Post-Cyclic Peak shear Stress, %	20.0				
Membrane Correction, psf	49				
Corrected Post-Cyclic Peak Shear Stress, psf	2166				
S_r/σ'_{vc}	0.36				

Comments: The cyclic portion of the test was terminated at a 10% peak-to-peak axial strain. Actual post cyclic strength parameters should be determined by an engineer familiar with dynamic testing data.

Tested By: md
Checked By: jdt

CYCLIC SIMPLE SHEAR DATA

Step 1 of **1**

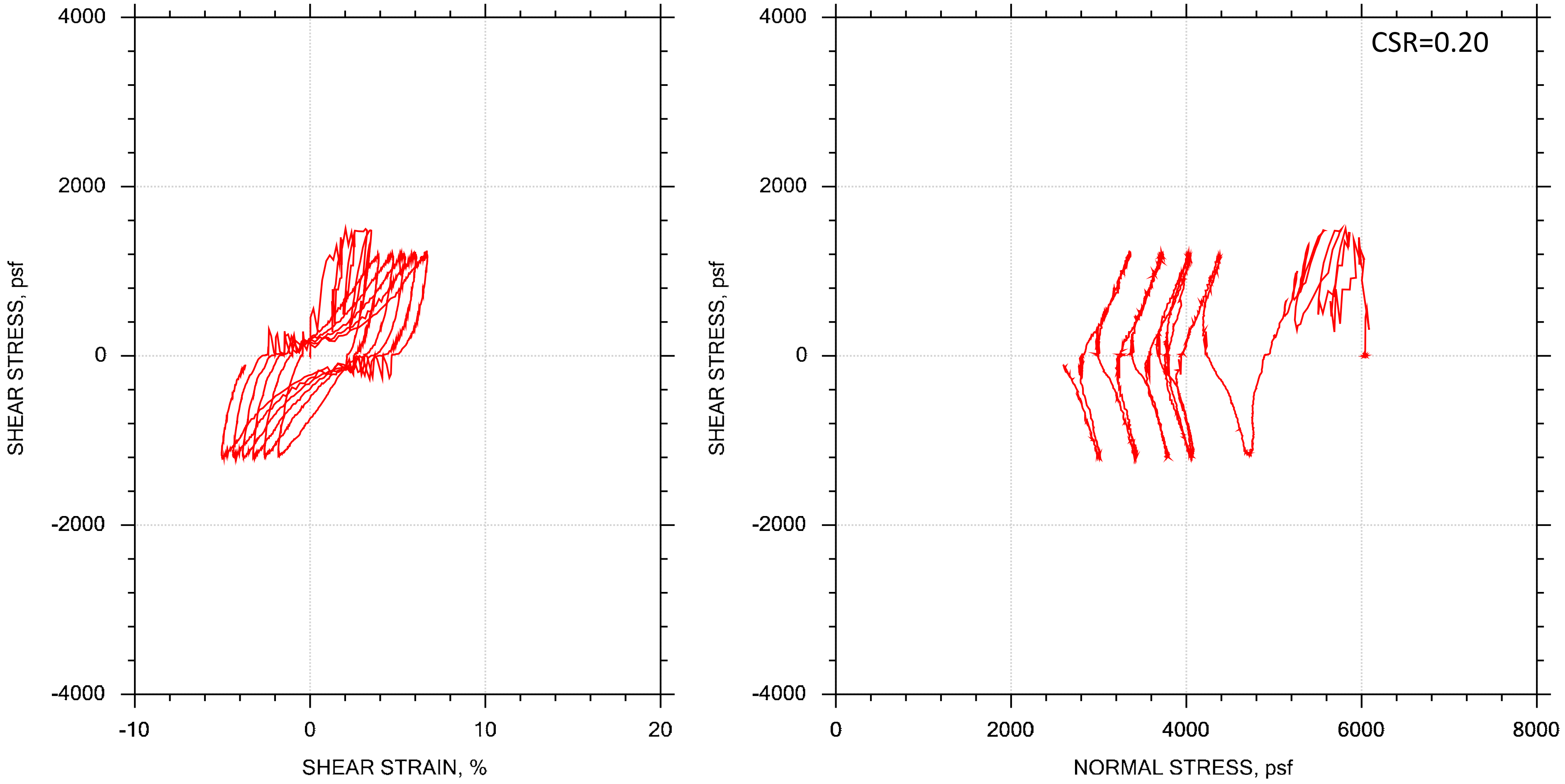


GeoTesting EXPRESS	Project: Vectran AB Brown Ash Pond	Location: Evansville, IN	Project No.: GTX-303915
	Boring No.: AECOM-B2	Tested By: md	Checked By: jdt
	Sample No.: 4A	Test Date: 11/20/15	Test No.: CDSS-5
	Depth: 62-64 ft	Sample Type: intact	Elevation: ---
	Description: Moist, gray silt		
	Remarks: System GG		Page 2 of 5
	File: \\hal1\Projects\GTX303915 - AECOM - Vectran AB Brown Ash Pond\6 Lab Testing\Soil\CDSS\303915-CDSS-5n.dat		

CYCLIC SIMPLE SHEAR STRESS DATA

Step 1 of 1

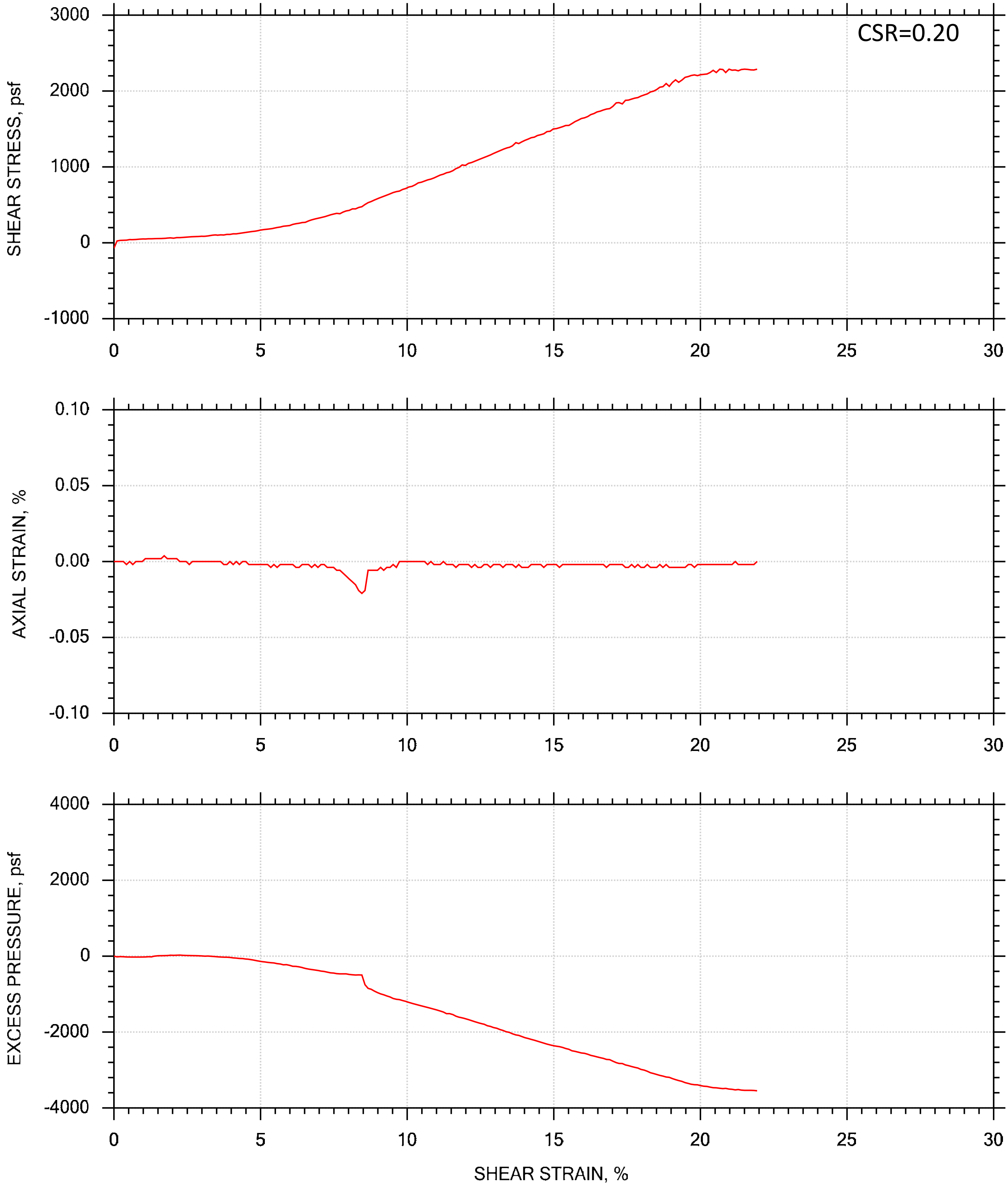
Cycle: 0.0 to 6.0



GeoTesting EXPRESS	Project: Vectran AB Brown Ash Pond	Location: Evansville, IN	Project No.: GTX-303915
	Boring No.: AECOM-B2	Tested By: md	Checked By: jdt
	Sample No.: 4A	Test Date: 11/20/15	Depth: 62-64 ft
	Test No.: CDSS-5	Sample Type: intact	Elevation: ---
	Description: Moist, gray silt		
	Remarks: System GG		Page 3 of 5
	File: \\hal1\Projects\GTX303915 - AECOM - Vectran AB Brown Ash Pond\6 Lab Testing\Soil\CDSS\303915-CDSS-5n.dat		

DIRECT SIMPLE SHEAR TEST by ASTM D6528

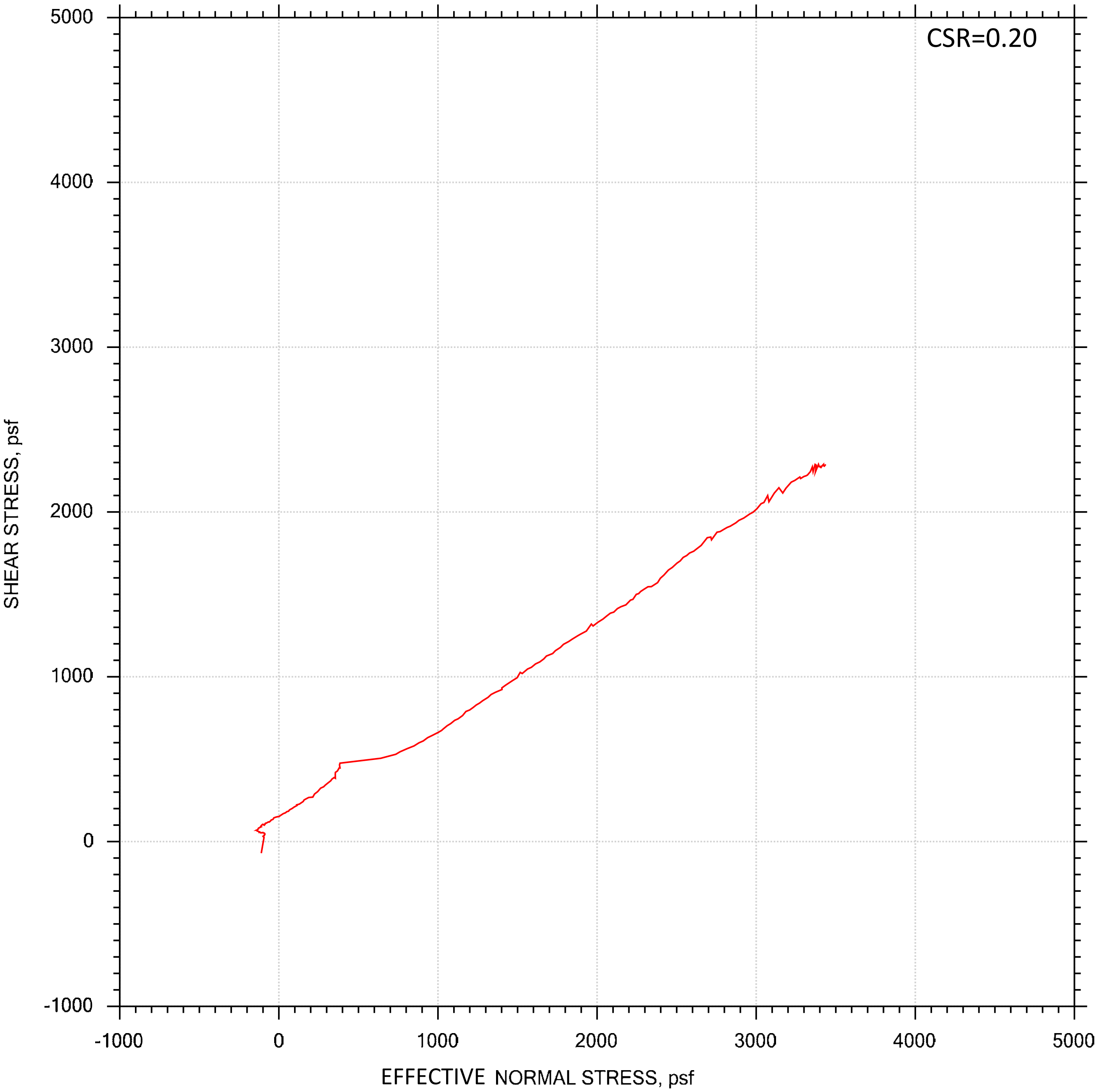
POST CYCLIC



GeoTesting EXPRESS	Project: Vectran AB Brown Ash Pond	Location: Evansville, IN	Project No.: GTX-303915
	Boring No.: AECOM-B2	Tested By: md	Checked By: jdt
	Sample No.: 4A	Test Date: 11/20/15	Test No.: CDSS-5
	Depth: 62-64 ft	Sample Type: intact	Elevation: ---
	Description: Moist, gray silt		
	Remarks: System GG		Page 4 of 5
	File: \\hal1\Projects\GTX303915 - AECOM - Vectran AB Brown Ash Pond\6 Lab Testing\Soil\CDSS\303915-CDSS-5n.dat		

DIRECT SIMPLE SHEAR TEST by ASTM D6528

POST CYCLIC



Project: Vectran AB Brown Ash Pond	Location: Evansville, IN	Project No.: GTX-303915
Boring No.: AECOM-B2	Tested By: md	Checked By: jdt
Sample No.: 4A	Test Date: 11/20/15	Test No.: CDSS-5
Depth: 62-64 ft	Sample Type: intact	Elevation: ---
Description: Moist, gray silt		
Remarks: System GG		Page 5 of 5
File: \\hal1\Projects\GTX303915 - AECOM - Vectran AB Brown Ash Pond\6 Lab Testing\Soil\CDSS\303915-CDSS-5n.dat		

Consolidated Undrained Cyclic Direct Simple Shear Test of Cohesive Soils

Client:	AECOM	GTX#:	303915
Project Name:	Vectran AB Brown Ash Pond Lower Dam	Test Date:	11/18/15
Project Location:	Evansville, IN		

Boring ID: AECOM-B4
Sample ID: 2
Depth, ft: 33-35

Visual Description: Wet, olive silt

Test Equipment: Top and bottom box (circular) = 2.5 in diameter. Load cells and LVDT's connected to data acquisition system for shear force, normal load, horizontal and vertical displacement; surface area = 4.91 in^2, soil height = 1 inch.
Stacked Teflon Rings set-up used, which included porous stones with pins.

Test Condition: Inundated prior to consolidation.

Sample Type and Preparation: Extruded from tube, cut, trimmed and placed into apparatus at as-received density and moisture content.

Parameter	Point 1	Point 2	Point 3	Point 4	Point 5
Test No.	CDSS-3				
Initial Moisture Content, %	27.8				
Initial Dry Density, pcf	85.8				
Vertical Consolidation Stress, psf	2965				
Cyclic Stress Ratio	0.08				
Number of cycles completed	50				
Frequency, Hz	1				
Final Moisture Content, %	36.1				
Delay before shearing, min	60				
Nominal Rate of Shear Strain, %/hr	5.0				
Measured Post-Cyclic Peak Shear Stress, psf	1722				
Shear Strain at Post-Cyclic Peak shear Stress, %	20.0				
Membrane Correction, psf	49				
Corrected Post-Cyclic Peak Shear Stress, psf	1673				
S_r/σ'_{vc}	0.56				

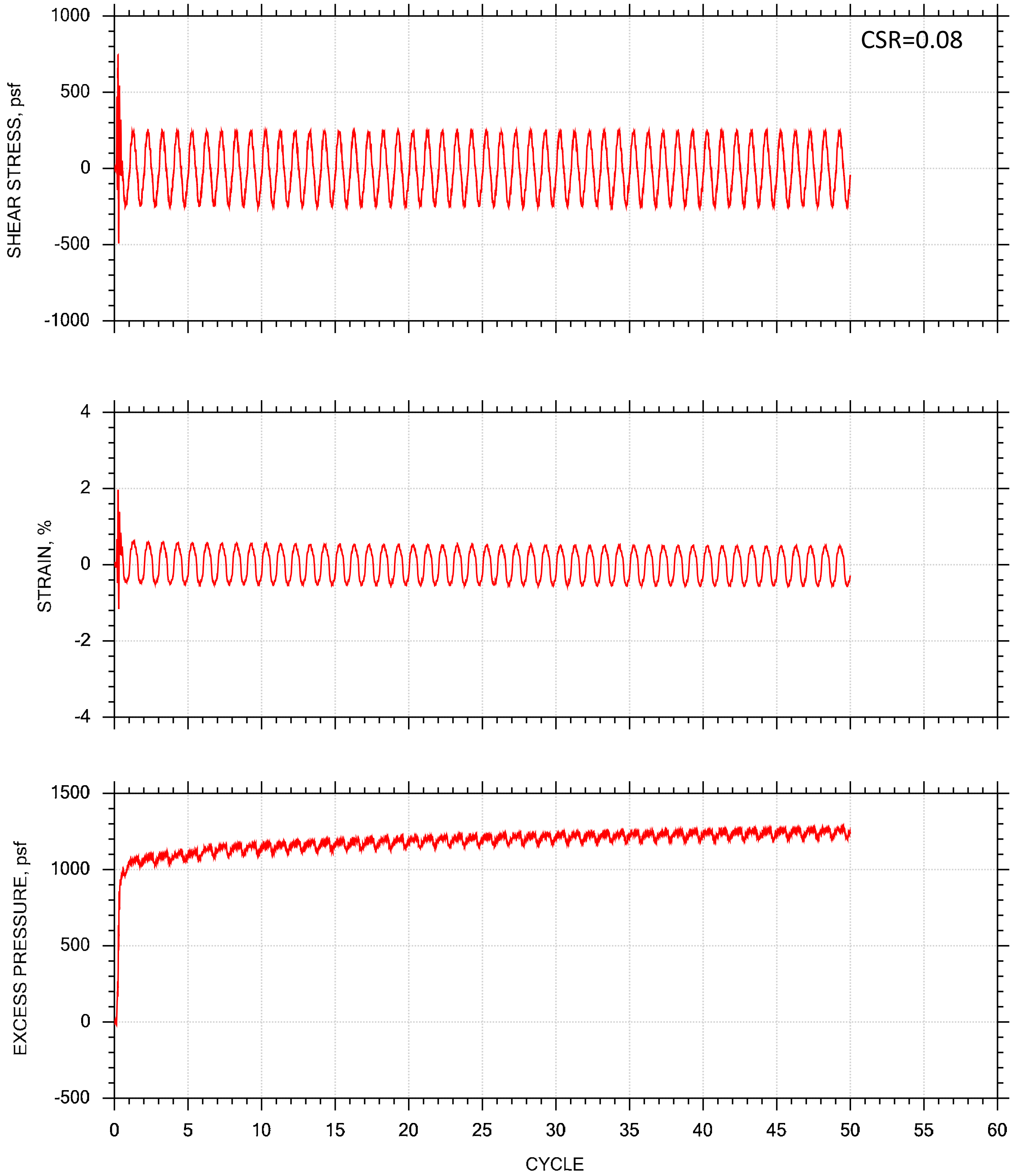
Comments: Actual post cyclic strength parameters should be determined by an engineer familiar with dynamic testing data.

Tested By: md

Checked By: jdt

CYCLIC SIMPLE SHEAR DATA

Step 1 of **1**

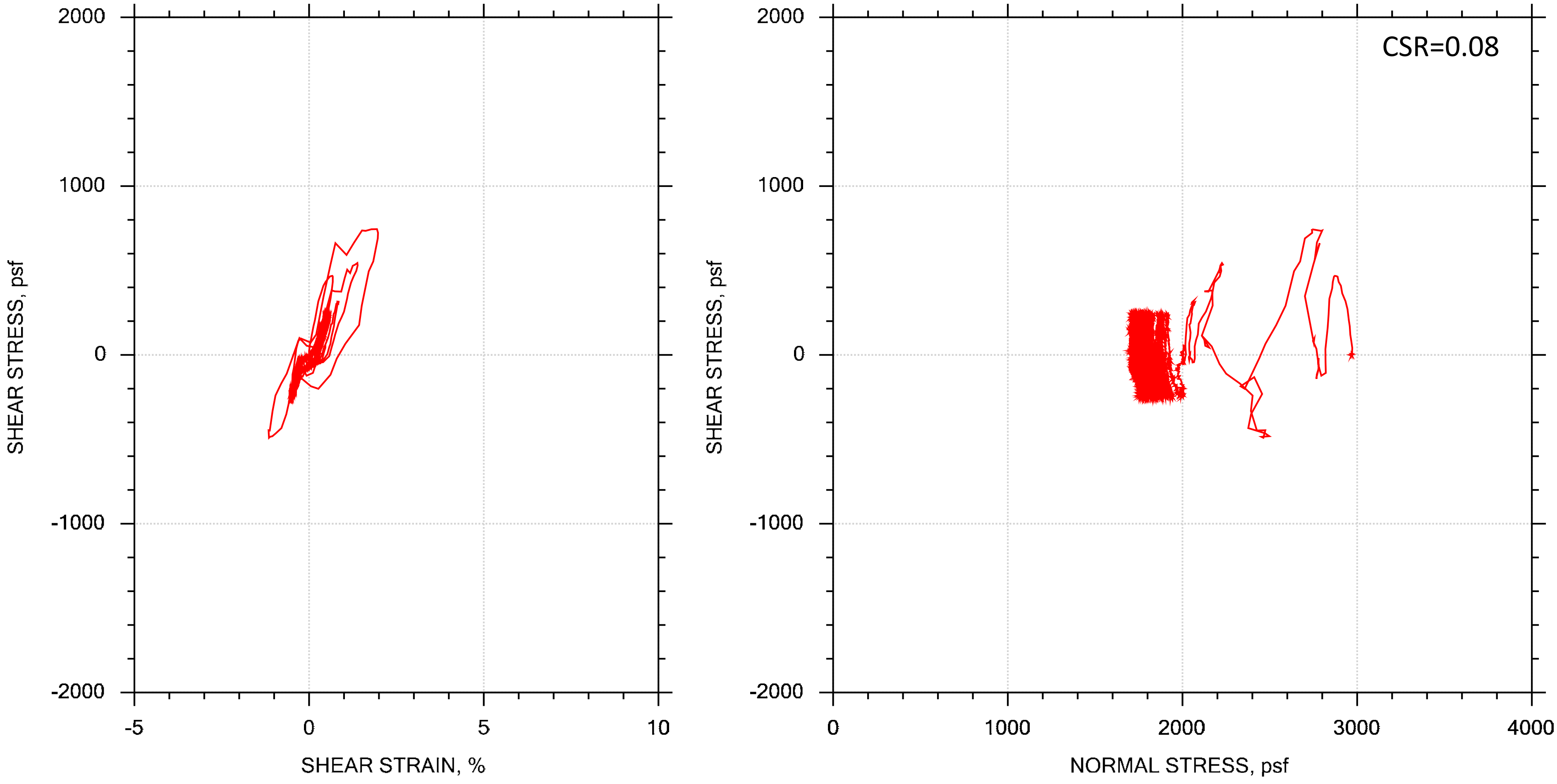


Project: Vectran AB Brown Ash Pond	Location: Evansville, IN	Project No.: GTX-303915
Boring No.: AECOM-B4	Tested By: md	Checked By: jdt
Sample No.: 2	Test Date: 11/18/15	Test No.: CDSS-3
Depth: 33-35 ft	Sample Type: intact	Elevation: ---
Description: Wet, olive silt		
Remarks: System HH		Page 2 of 5
File: \\hal1\Projects\GTX303915 - AECOM - Vectran AB Brown Ash Pond\6 Lab Testing\Soil\CDSS\303915-CDSS-3n.dat		

CYCLIC SIMPLE SHEAR STRESS DATA

Step 1 of 1

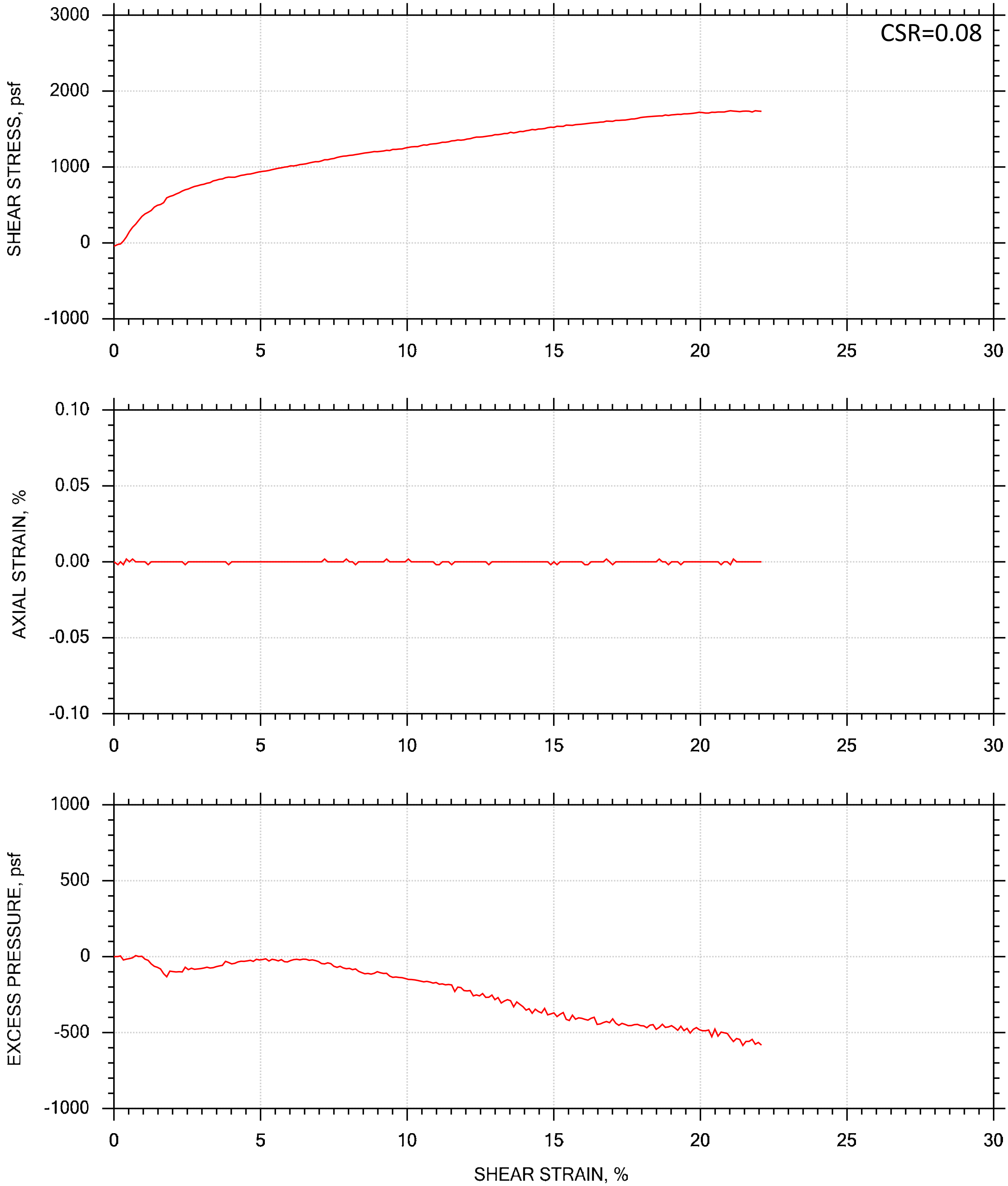
Cycle: 0.0 to 50.0



GeoTesting EXPRESS	Project: Vectran AB Brown Ash Pond	Location: Evansville, IN	Project No.: GTX-303915
	Boring No.: AECOM-B4	Tested By: md	Checked By: jdt
	Sample No.: 2	Test Date: 11/18/15	Depth: 33-35 ft
	Test No.: CDSS-3	Sample Type: intact	Elevation: ---
	Description: Wet, olive silt		
	Remarks: System HH		Page 3 of 5
	File: \\hal1\Projects\GTX303915 - AECOM - Vectran AB Brown Ash Pond\6 Lab Testing\Soil\CDSS\303915-CDSS-3n.dat		

DIRECT SIMPLE SHEAR TEST by ASTM D6528

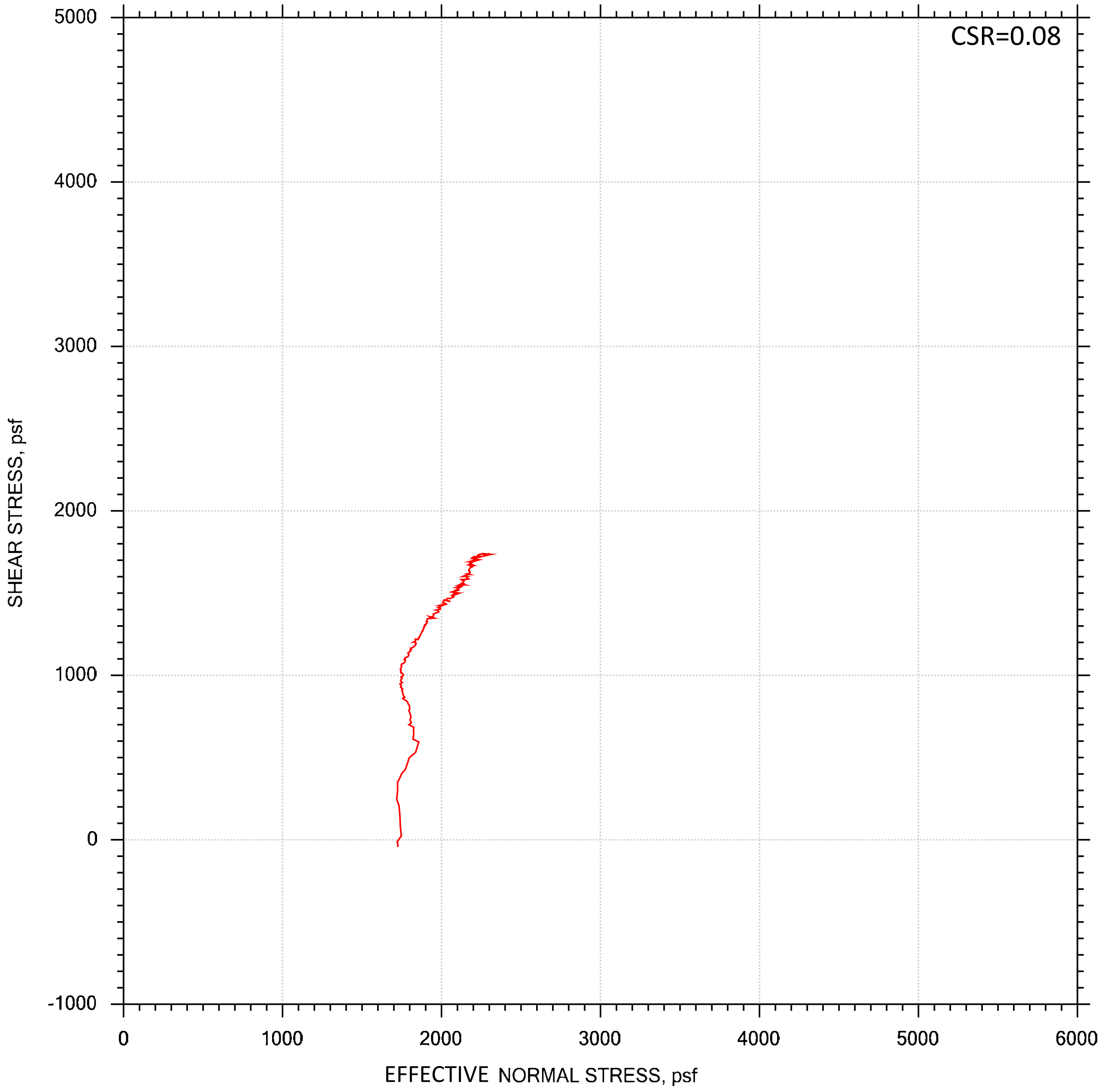
POST CYCLIC



GeoTesting EXPRESS	Project: Vectran AB Brown Ash Pond	Location: Evansville, IN	Project No.: GTX-303915
	Boring No.: AECOM-B4	Tested By: md	Checked By: jdt
	Sample No.: 2	Test Date: 11/18/15	Test No.: CDSS-3
	Depth: 33-35 ft	Sample Type: intact	Elevation: ---
	Description: Wet, olive silt		
	Remarks: System HH		Page 4 of 5
	File: \\hal1\Projects\GTX303915 - AECOM - Vectran AB Brown Ash Pond\6 Lab Testing\Soil\CDSS\303915-CDSS-3n.dat		

DIRECT SIMPLE SHEAR TEST by ASTM D6528

POST CYCLIC



GeoTesting EXPRESS	Project: Vectran AB Brown Ash Pond	Location: Evansville, IN	Project No.: GTX-303915
	Boring No.: AECOM-B4	Tested By: md	Checked By: jdt
	Sample No.: 2	Test Date: 11/18/15	Test No.: CDSS-3
	Depth: 33-35 ft	Sample Type: intact	Elevation: ---
	Description: Wet, olive silt		
	Remarks: System HH		Page 5 of 5
	File: \\hal1\Projects\GTX303915 - AECOM - Vectran AB Brown Ash Pond\6 Lab Testing\Soil\CDSS\303915-CDSS-3n.dat		

Consolidated Undrained Cyclic Direct Simple Shear Test of Cohesive Soils

Client:	AECOM	GTX#:	303915
Project Name:	Vectran AB Brown Ash Pond Lower Dam	Test Date:	11/20/15
Project Location:	Evansville, IN		

Boring ID: AECOM-B4
Sample ID: 3
Depth, ft: 46-48

Visual Description: Moist, olive silt

Test Equipment: Top and bottom box (circular) = 2.5 in diameter. Load cells and LVDT's connected to data acquisition system for shear force, normal load, horizontal and vertical displacement; surface area = 4.91 in^2, soil height = 1 inch.
Stacked Teflon Rings set-up used, which included porous stones with pins.

Test Condition: Inundated prior to consolidation.

Sample Type and Preparation: Extruded from tube, cut, trimmed and placed into apparatus at as-received density and moisture content.

Parameter	Point 1	Point 2	Point 3	Point 4	Point 5
Test No.	CDSS-4				
Initial Moisture Content, %	29.1				
Initial Dry Density, pcf	91.8				
Vertical Consolidation Stress, psf	3830				
Cyclic Stress Ratio	0.20				
Number of cycles completed	9				
Frequency, Hz	1				
Final Moisture Content, %	26.8				
Delay before shearing, min	60				
Nominal Rate of Shear Strain, %/hr	5.0				
Measured Post-Cyclic Peak Shear Stress, psf	1516				
Shear Strain at Post-Cyclic Peak shear Stress, %	20.0				
Membrane Correction, psf	49				
Corrected Post-Cyclic Peak Shear Stress, psf	1467				
S_r/σ'_{vc}	0.38				

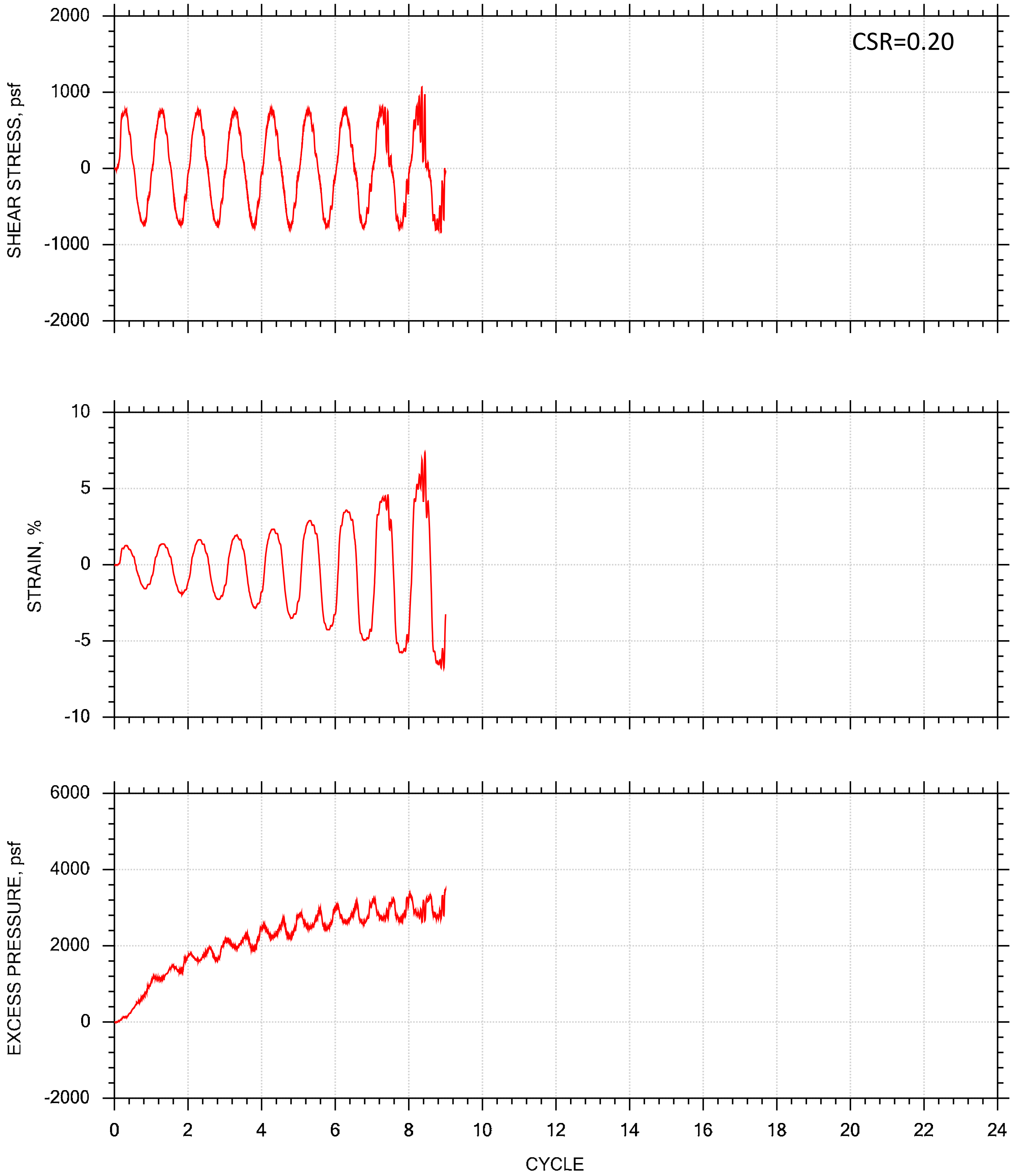
Comments: The cyclic portion of the test resulted in an R value approaching 1, and terminated the test at a 10% peak-to-peak axial strain. Actual post cyclic strength parameters should be determined by an engineer familiar with dynamic testing data.

Tested By: md

Checked By: jdt

CYCLIC SIMPLE SHEAR DATA

Step 1 of **1**

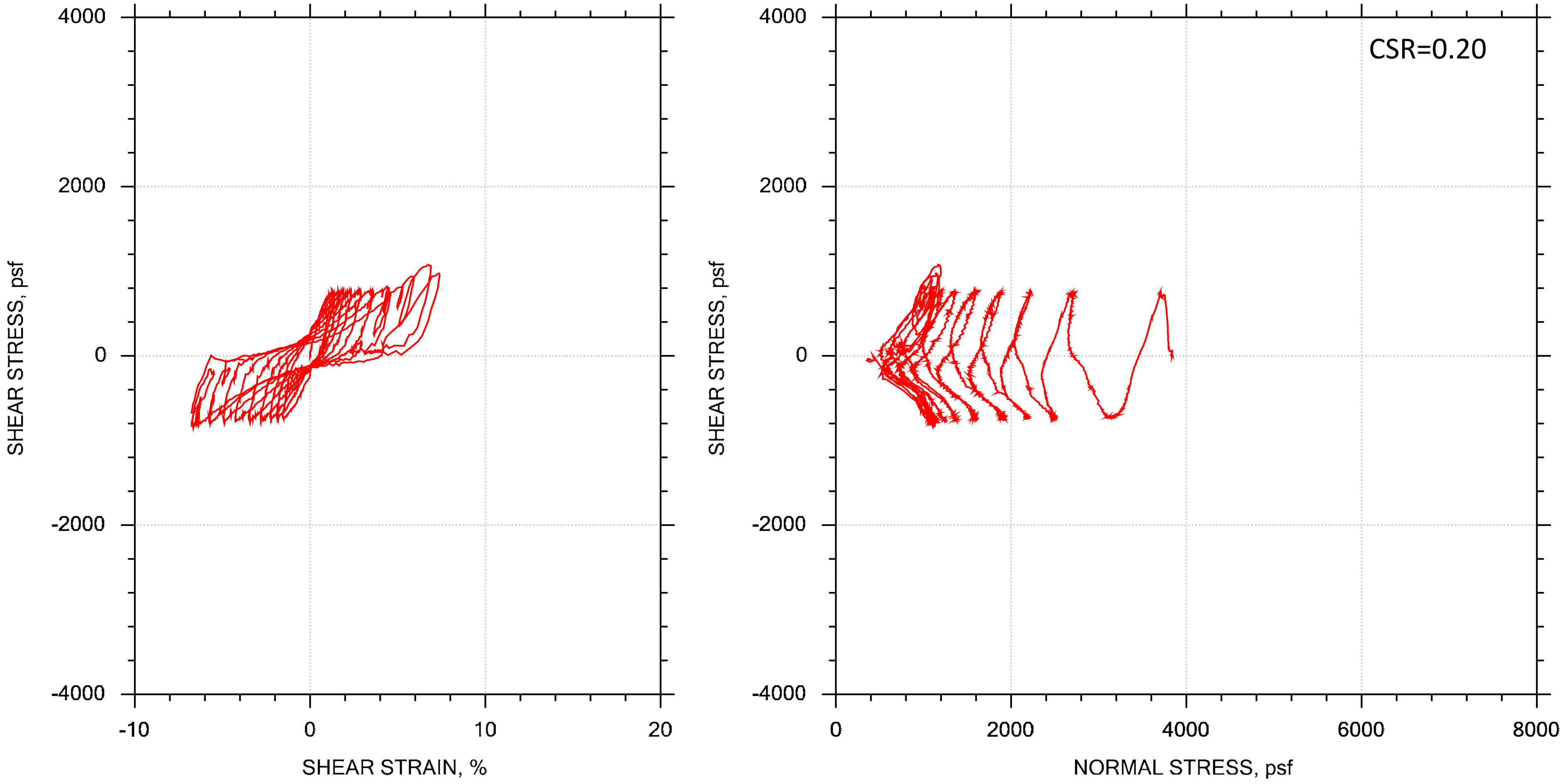


GeoTesting EXPRESS	Project: Vectran AB Brown Ash Pond	Location: Evansville, IN	Project No.: GTX-303915
	Boring No.: AECOM-B4	Tested By: md	Checked By: jdt
	Sample No.: 3	Test Date: 11/20/15	Test No.: CDSS-4
	Depth: 46-48 ft	Sample Type: intact	Elevation: ---
	Description: Moist, olive silt		
	Remarks: System HH		Page 2 of 5
	File: \\hal1\Projects\GTX303915 - AECOM - Vectran AB Brown Ash Pond\6 Lab Testing\Soil\CDSS\303915-CDSS-4n.dat		

CYCLIC SIMPLE SHEAR STRESS DATA

Step 1 of **1**

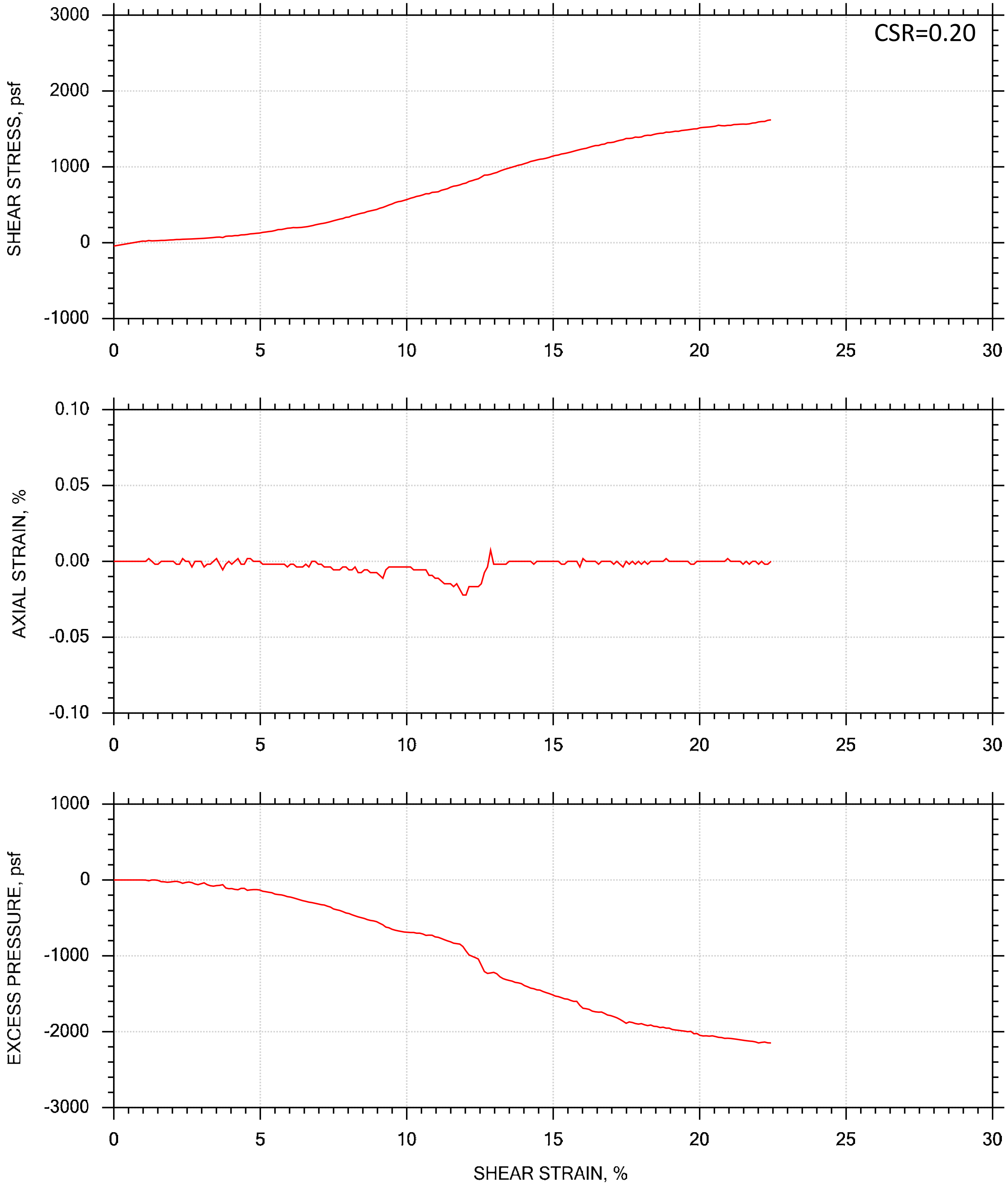
Cycle: 0.0 to 9.0



GeoTesting EXPRESS	Project: Vectran AB Brown Ash Pond	Location: Evansville, IN	Project No.: GTX-303915
	Boring No.: AECOM-B4	Tested By: md	Checked By: jdt
	Sample No.: 3	Test Date: 11/20/15	Depth: 46-48 ft
	Test No.: CDSS-4	Sample Type: intact	Elevation: ---
	Description: Moist, olive silt		
	Remarks: System HH		Page 3 of 5
	File: \\hal1\Projects\GTX303915 - AECOM - Vectran AB Brown Ash Pond\6 Lab Testing\Soil\CDSS\303915-CDSS-4n.dat		

DIRECT SIMPLE SHEAR TEST by ASTM D6528

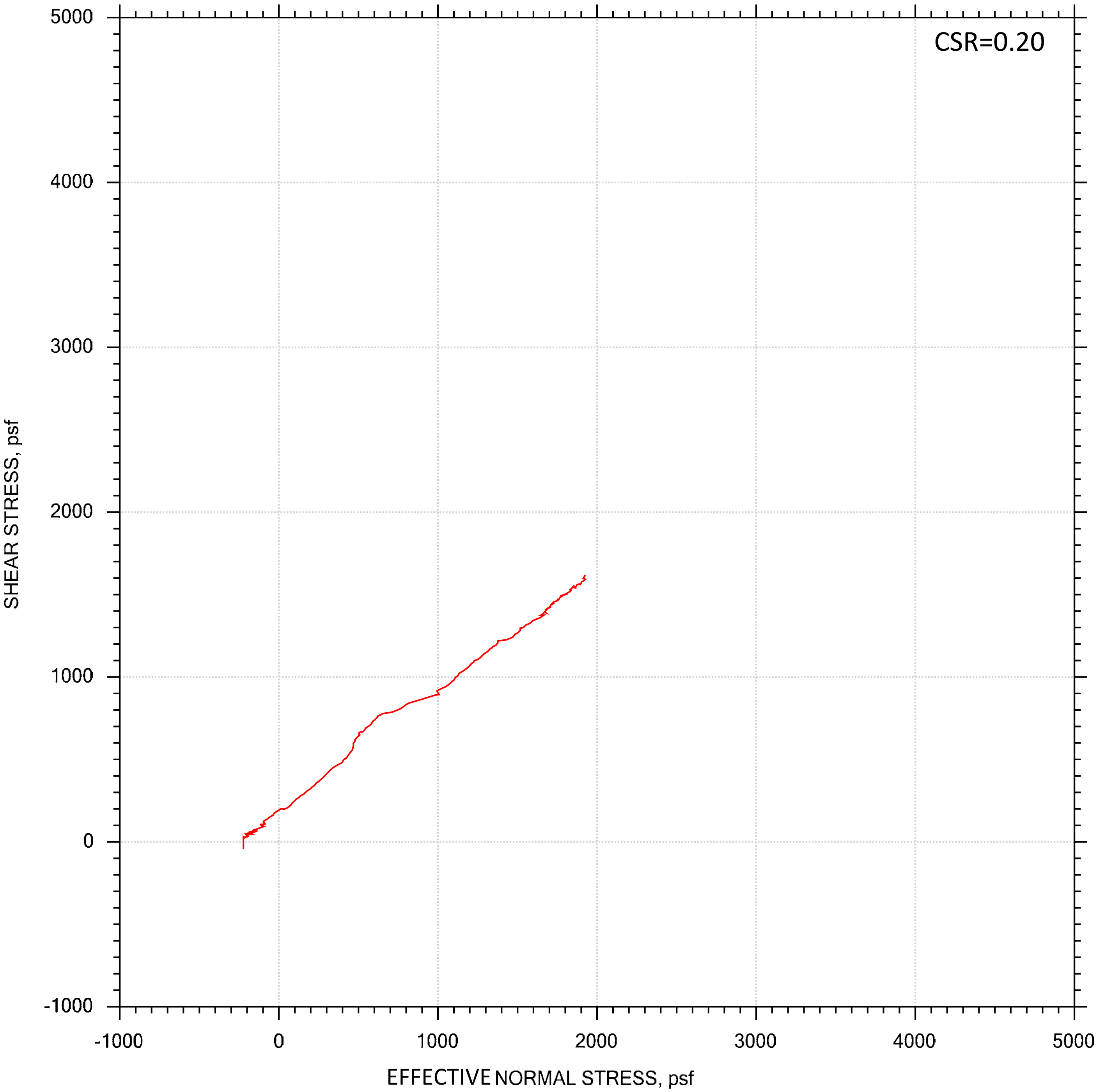
POST CYCLIC



Project: Vectran AB Brown Ash Pond	Location: Evansville, IN	Project No.: GTX-303915
Boring No.: AECOM-B4	Tested By: md	Checked By: jdt
Sample No.: 3	Test Date: 11/20/15	Test No.: CDSS-4
Depth: 46-48 ft	Sample Type: intact	Elevation: ---
Description: Moist, olive silt		
Remarks: System HH		Page 4 of 5
File: \\hal1\Projects\GTX303915 - AECOM - Vectran AB Brown Ash Pond\6 Lab Testing\Soil\CDSS\303915-CDSS-4n.dat		

DIRECT SIMPLE SHEAR TEST by ASTM D6528

POST CYCLIC



	Project: Vectran AB Brown Ash Pond	Location: Evansville, IN	Project No.: GTX-303915
	Boring No.: AECOM-B4	Tested By: md	Checked By: jdt
GeoTesting EXPRESS	Sample No.: 3	Test Date: 11/20/15	Test No.: CDSS-4
	Depth: 46-48 ft	Sample Type: intact	Elevation: ---
	Description: Moist, olive silt		
	Remarks: System HH		Page 5 of 5
	File: \\hal1\Projects\GTX303915 - AECOM - Vectran AB Brown Ash Pond\6 Lab Testing\Soil\CDSS\303915-CDSS-4n.dat		

Consolidated Undrained Cyclic Direct Simple Shear Test of Cohesive Soils

Client: AECOM
Project Name: Vectran AB Brown Ash Pond Lower Dam
Project Location: Evansville, IN

GTX#: 303915
Test Date: 12/7/15

Boring ID: AECOM-B5
Sample ID: 2
Depth, ft: 30-32

Visual Description: Moist, gray silt with sand

Test Equipment: Top and bottom box (circular) = 2.5 in diameter. Load cells and LVDT's connected to data acquisition system for shear force, normal load, horizontal and vertical displacement; surface area = 4.91 in^2, soil height = 1 inch.
Stacked Teflon Rings set-up used, which included porous stones with pins.

Test Condition: Inundated prior to consolidation.
Sample Type and Preparation: Extruded from tube, cut, trimmed and placed into apparatus at as-received density and moisture content.

Parameter	Point 1	Point 2	Point 3	Point 4	Point 5
Test No.	CDSS-6				
Initial Moisture Content, %	32.2				
Initial Dry Density, pcf	86.3				
Vertical Consolidation Stress, psf	2660				
Cyclic Stress Ratio	0.15				
Number of cycles completed	50				
Frequency, Hz	1				
Final Moisture Content, %	30.6				
Delay before shearing, min	60				
Nominal Rate of Shear Strain, %/hr	5.0				
Measured Post-Cyclic Peak Shear Stress, psf	1222				
Shear Strain at Post-Cyclic Peak shear Stress, %	20.0				
Membrane Correction, psf	49				
Corrected Post-Cyclic Peak Shear Stress, psf	1173				
S_r/σ'_{vc}	0.44				

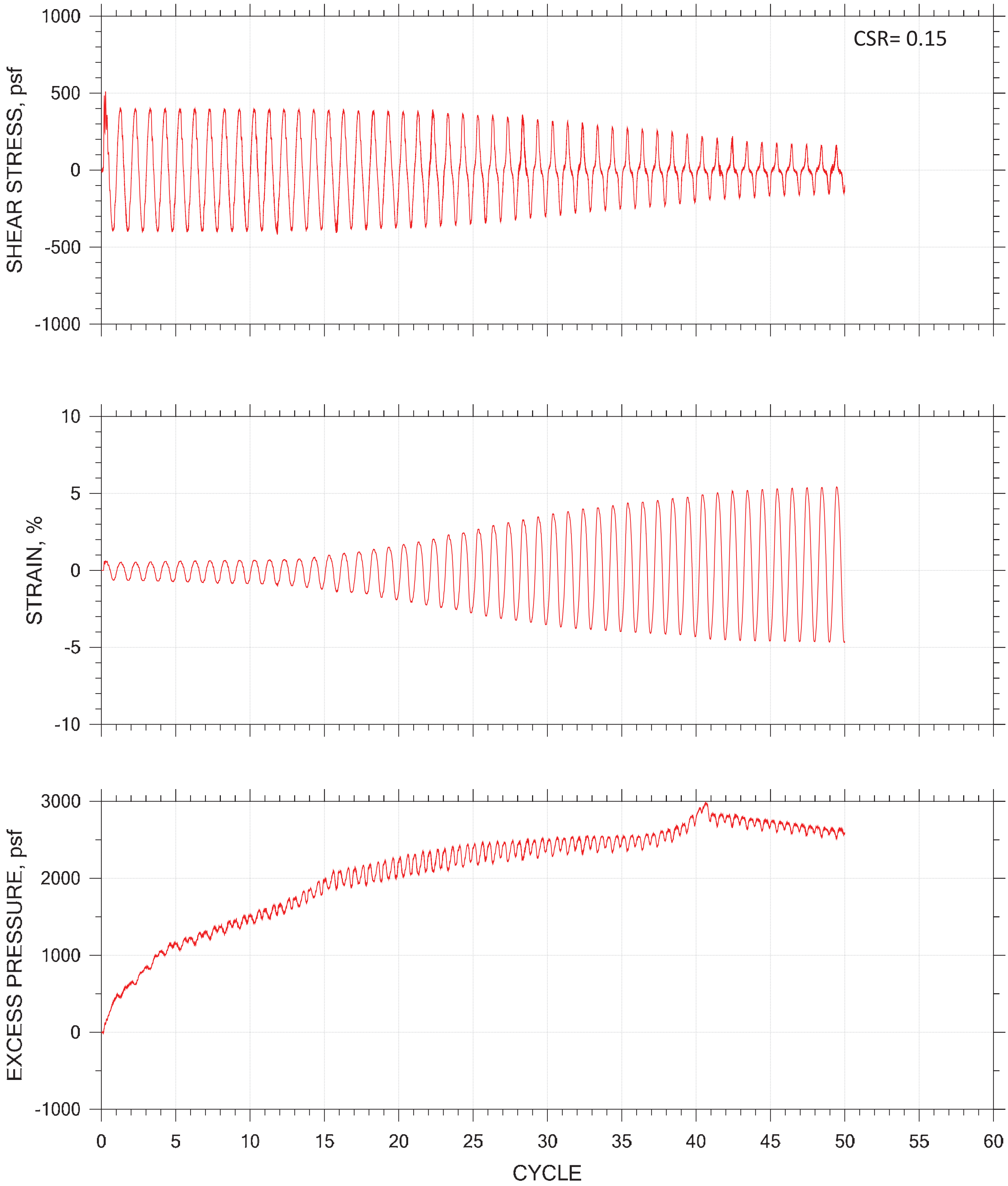
Comments: Actual post cyclic strength parameters should be determined by an engineer familiar with dynamic testing data.

Tested By: md

Checked By: njh

CYCLIC SIMPLE SHEAR DATA

Step 1 of 1

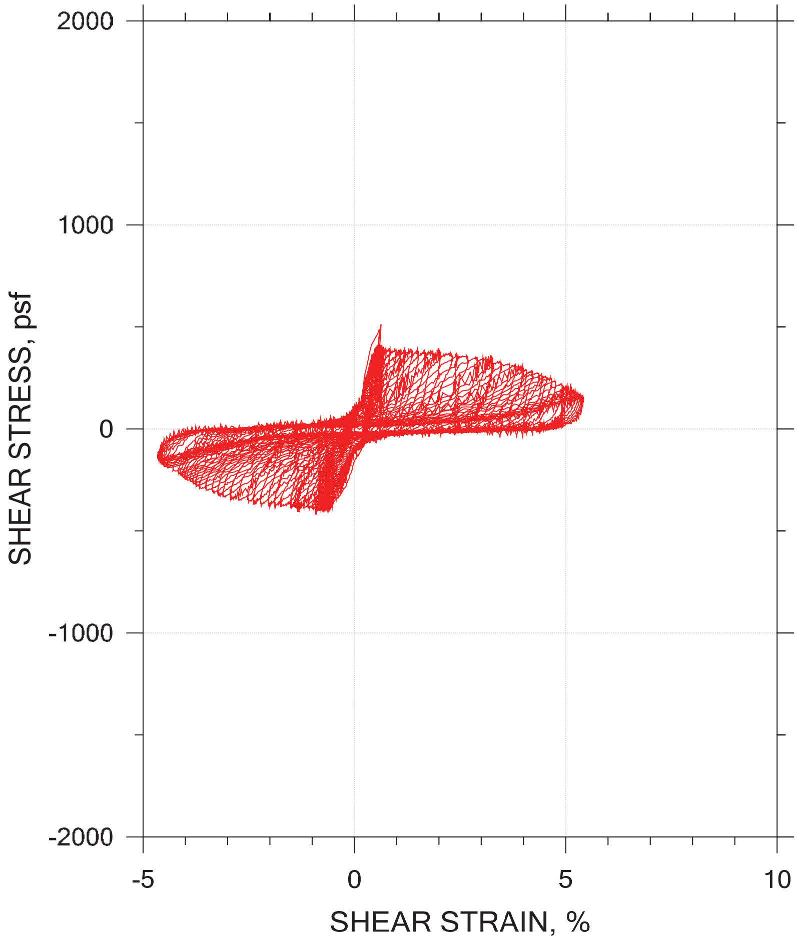


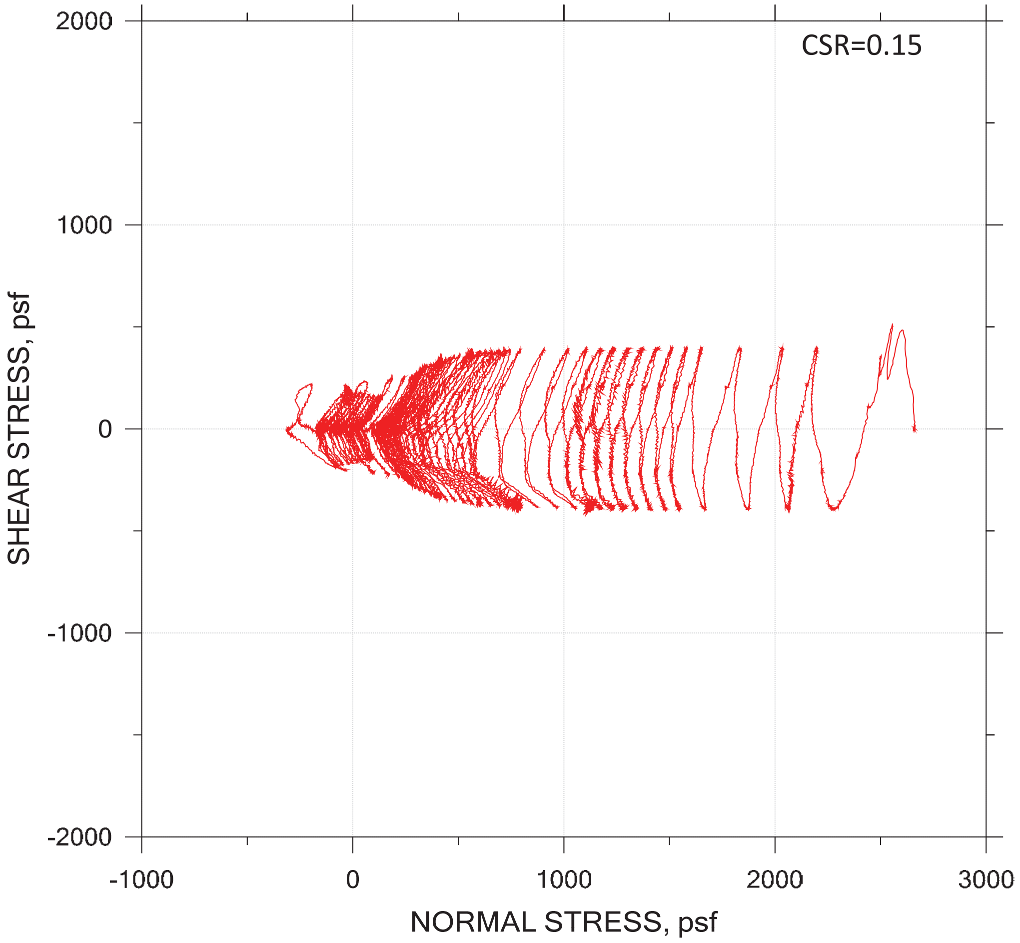
GeoTesting EXPRESS	Project: Vectran AB Brown Ash Pond	Location: Evansville, IN	Project No.: GTX-303915
	Boring No.: AECOM-B5	Tested By: md	Checked By: njh
	Sample No.: 2	Test Date: 12/07/15	Test No.: CDSS-6
	Depth: 30-32 ft	Sample Type: intact	Elevation: ---
	Description: Moist, gray silt with sand		
	Remarks: System HH		Page 2 of 5
	File: \\hal1\Projects\GTX303915 - AECOM - Vectran AB Brown Ash Pond\6 Lab Testing\Soil\CDSS\303915-CDSS-6n.dat		

CYCLIC SIMPLE SHEAR STRESS DATA

Step 1 of 1

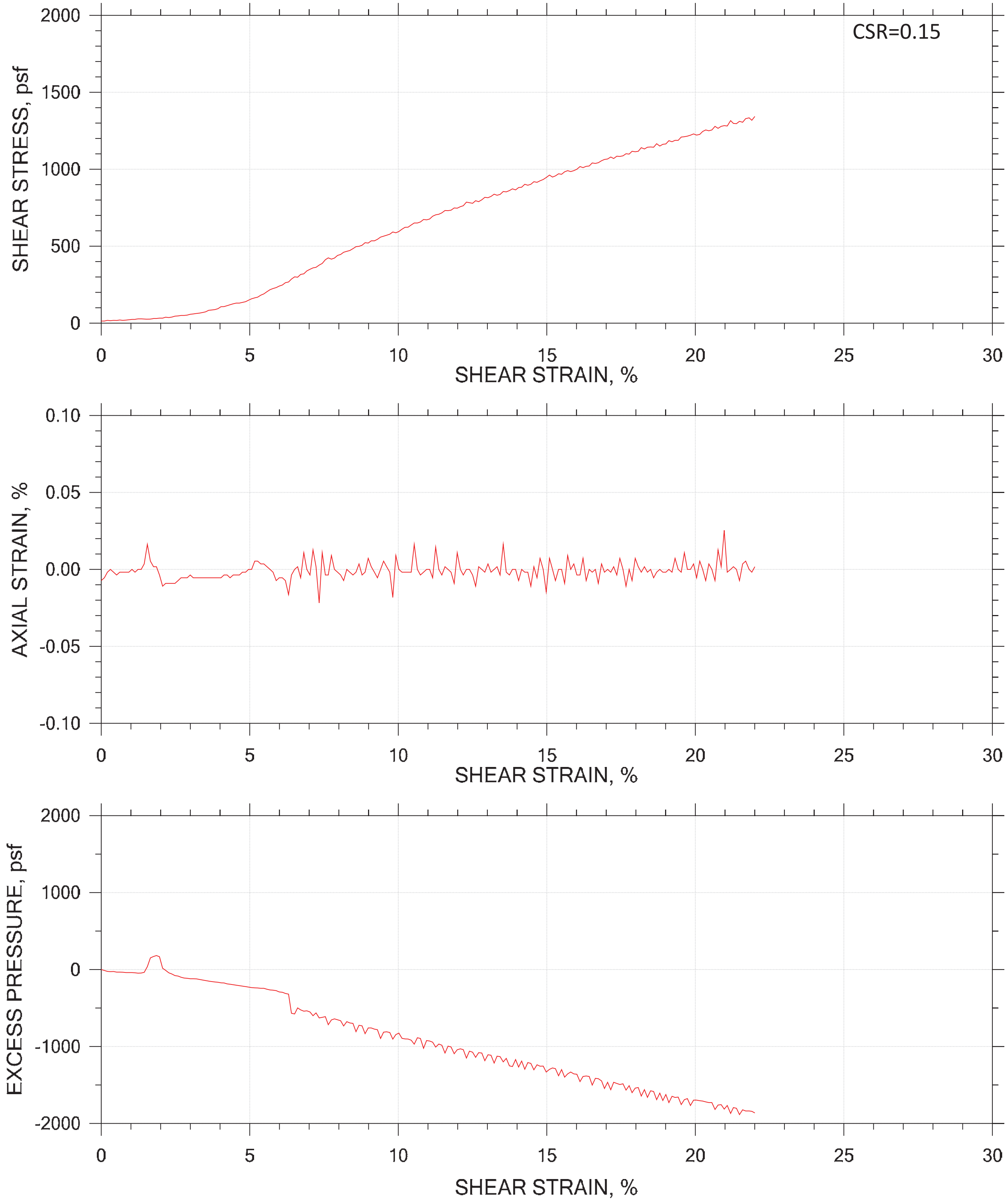
Cycle: 0.0 to 50.0





Project: Vectran AB Brown Ash Pond	Location: Evansville, IN	Project No.: GTX-303915
Boring No.: AECOM-B5	Tested By: md	Checked By: njh
Sample No.: 2	Test Date: 12/07/15	Depth: 30-32 ft
Test No.: CDSS-6	Sample Type: intact	Elevation: ---
Description: Moist, gray silt with sand		
Remarks: System H		Page 3 of 5
File: \\hal1\Projects\GTX303915 - AECOM - Vectran AB Brown Ash Pond\6 Lab Testing\Soil\CDSS\303915-CDSS-6n.dat		

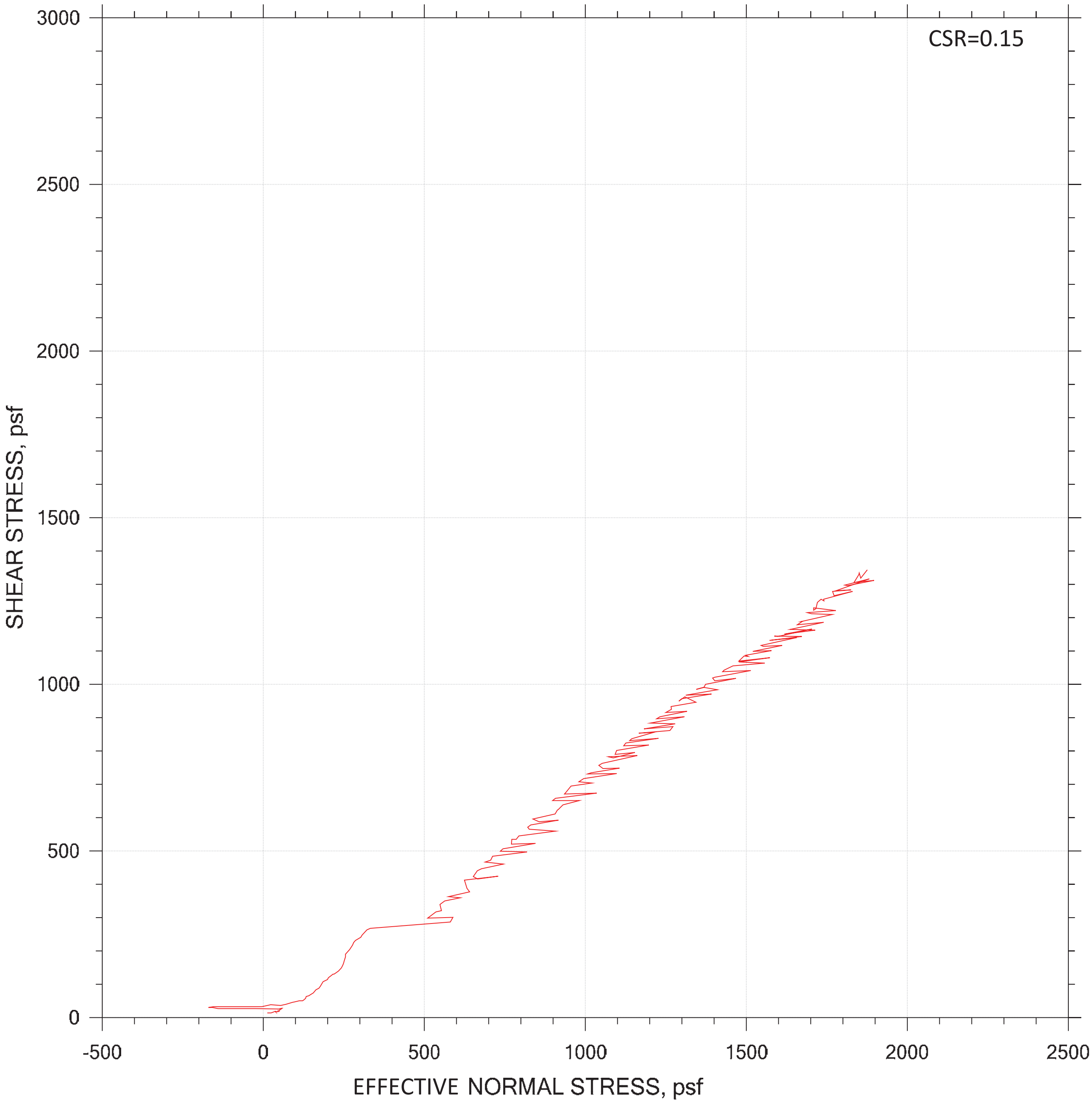
DIRECT SIMPLE SHEAR TEST by ASTM D6528
POST CYCLIC



GeoTesting EXPRESS

Project: Vectran AB Brown Ash Pond	Location: Evansville, IN	Project No.: GTX-303915
Boring No.: AECOM-B5	Tested By: md	Checked By: njh
Sample No.: 2	Test Date: 12/07/15	Test No.: CDSS-6
Depth: 30-32 ft	Sample Type: intact	Elevation: ---
Description: Moist, gray silt with sand		
Remarks: System HH		
File: \\hal1\Projects\GTX303915 - AECOM - Vectran AB Brown Ash Pond\6 Lab Testing\Soil\CDSS\303915-CDSS-6n.dat		

DIRECT SIMPLE SHEAR TEST by ASTM D6528
POST CYCLIC



GeoTesting EXPRESS	Project: Vectran AB Brown Ash Pond	Location: Evansville, IN	Project No.: GTX-303915
	Boring No.: AECOM-B5	Tested By: md	Checked By: njh
	Sample No.: 2	Test Date: 12/07/15	Test No.: CDSS-6
	Depth: 30-32 ft	Sample Type: intact	Elevation: ---
	Description: Moist, gray silt with sand		
	Remarks: System HH		Page 5 of 5
	File: \\hal1\Projects\GTX303915 - AECOM - Vectran AB Brown Ash Pond\6 Lab Testing\Soil\CDSS\303915-CDSS-6n.dat		

Ash Material Index Properties Laboratory Test Results

Summary of Laboratory Test Results - Impounded Ash

Boring and Sample ID	Ground Surface Elevation (ft)	Material Description	Sample Depth (ft)	Moisture Content (%)	Atterberg Limits: Liquid Limit (%)	Atterberg Limits: Plastic Limit (%)	Atterberg Limits: Plasticity Index (%)	Gradations: Sieve Analysis (3 inch to #200 Sieve): Gravel (%)	Sand (%)	Fines (%)
B-101, SS-11	463.7	Ash	26.0-27.5	24.1	33	20	13	-	-	97.2
B-102, SS-10	463.4	Ash	23.5-25.0	56.5	Non-Plastic			-	-	74.5
B-102, SS-13	463.4	Ash	31.0-32.5	71.2	Non-Plastic			-	-	74.4
B-102, SS-16	463.4	Ash	38.5-40.0	57.7	Non-Plastic			-	-	78.9
B-102, SS-20	463.4	Ash	48.5-50.0	54.8	Non-Plastic			-	-	94.9
B-103, SS-10	463.7	Ash	23.5-25.0	62.9	Non-Plastic			-	-	97.3
B-103, SS-15	463.7	Ash	36.0-37.5	72.4	Non-Plastic			-	-	96.0

Appendix E
Material Characterization Calculations

Job	Vectren A.B. Brown – Ash Pond System CCR Certification Report	Project No.	60442627	Sheet	1 of 13
Description	Appendix E	Computed by	ACI	Date	09/01/2016
	Strength Characterization Calculations	Checked by	VKG	Date	09/02/2016

I. Objective

This calculation package summarizes the interpretations and analyses performed to select material properties for use in the slope stability analyses of the Lower Dam at Vectren's A.B. Brown power station.

II. Subsurface Conditions

Various modern and historical subsurface investigations were performed at the Lower Dam, including in 2015/2016 and 1982. Collectively, a total of 32 borings and 5 cone penetration test soundings (with pore pressure dissipation testing and seismic shear wave velocity measurements) were performed. A full set of AECOM's boring logs, including soil descriptions, types of sampling, and choice laboratory test results, is provided in **Appendix B** of the report. A CPT data report is provided in **Appendix C** and complete laboratory testing results are provided in **Appendix D**.

Based on the results of the investigation, five stratigraphic materials were identified at the site. These are listed below and briefly summarized:

Dam Embankment Fill: Embankment Fill materials were encountered from the ground surface and extending to depths ranging from approximately 37 to 58 feet below ground surface (bgs) from the crest boring and 5.5 to 26.5 feet bgs from the bench borings. Embankment Fill materials were typically a mixture of lean clays (CL) and silty clays (CL-ML) with varying amounts of sand. Visual classifications were most often described as slightly moist to moist, reddish brown to brown, silty clay to sandy lean clay.

Table E-1 summarizes the field data obtained within the Embankment Fill.

Table E-1: Embankment Fill Material Field Data Summary

Category	Min.	Max.	Average
SPT-N	3	50	16
Pocket Penetrometer (tsf)	0.5	4.5	2.6
Cone Tip Resistance (tsf)	56.6	111.7	71.3
Cone Sleeve Resistance (tsf)	1.8	3.0	2.3
SCPTu Shear Wave Velocity (ft/sec)	670	878	815

The field results in the Embankment Fill reflect a material with stiff to very stiff consistency, and indicate that the fill is well-compacted.

Foundation Silt Materials: Natural, alluvial silt deposits were encountered in most borings drilled in the lower bench area and beyond the toe of the dam. Silts were not encountered at any of the borings drilled at the crest of the dam, indicating that the deposit grades out moving from west to east across the width of the dam and buttress structures. The deposits consisted of a moist to wet, brown to gray, very soft to very stiff silt (ML) with occasion traces of fine sand. The silts were generally non-plastic or had very low plasticity indices. Silts varied in thickness from approximately 2.0 feet to 27.5 feet.

Table E-2 summarizes the field data obtained within the Foundation Silt deposit.

Table E-2: Foundation Silt Material Field Data Summary

Category	Min.	Max.	Average
SPT-N	0	23	7
Pocket Penetrometer (tsf)	NA	NA	NA
Cone Tip Resistance (tsf)	23.9	50.3	34.0
Cone Sleeve Resistance (tsf)	0.64	1.32	0.90
SCPTu Shear Wave Velocity (ft/sec)	533	737	692

Foundation Silty Clay Materials: Native lean clays make up much of the foundation materials of the Lower Dam, especially at the eastern regions of the dam footprint and below the crest. These clays consisted primarily of moist to wet, light brown to gray, very soft to very stiff lean clays (CL) to silty clays (CL-ML) with varying amounts of sand. In some locations, the clays are interbedded with the foundation silts described previously. The thickness of the clays varied widely, becoming more interbedded with silt layers to the west towards the bench and downstream toe of the embankment.

Table E-3 summarizes the field data obtained within the Foundation Clay deposit.

Table E-3: Foundation Clay Material Field Data Summary

Category	Min.	Max.	Average
SPT-N	0	33	10
Pocket Penetrometer (tsf)	0.25	4.0	1.4
Cone Tip Resistance (tsf)	17.5	38.4	26.6
Cone Sleeve Resistance (tsf)	0.46	1.43	0.91
SCPTu Shear Wave Velocity (ft/sec)	804	984	882

Buttress Fill Materials: The buttress fill was obtained from near-site borrow sources, and consists of fine-grained soils most typically classified as lean clay (CL). Plasticity indices of the fill material generally range from 6 to 14, with an average of about 12. To a much lesser extent, the buttress fill includes materials classified as silt (ML). The fill was placed and compacted in lifts (construction was to 95% of the Standard Proctor Maximum Dry Density), and density testing of each lift using nuclear methods was performed. Field SPT and CPT data are not available for the buttress, because construction of this structure occurred after the field investigations for the project were completed.

Sluiced Ash Materials: No ash materials were present in the Lower Dam. Bottom ash materials were encountered in historical borings drilled in the area east of the dam. The material was generally classified as fine- to coarse-grained sand, silly clay, sandy silt. The materials were generally very loose to loose, moist to wet, and brown to black.

III. Laboratory Strength Testing Program

Representative samples were collected at regular intervals from the borings and were utilized for laboratory index and strength testing. Strength testing included isotropically consolidated-undrained triaxial tests with pore pressure measurements (CIU) on the Embankment Fill, Foundation Silt, and Foundation Silty Clay materials, and cyclic direct simple shear (CDSS) tests on the Foundation Silt materials. Table E-4 summarizes the strength testing performed.

Table E-4: Laboratory Strength Testing Program for Lower Dam

Test	ASTM Method	Number of Test Points		
		Embankment Fill	Foundation Silt	Foundation Clay
Unit Weight		6	18	14
Consolidated Undrained (CIU)	D4767	5	10	12
Cyclic Direct Simple Shear (CDSS)	GTX S1085	-	6	-

IV. Material Properties For Stability Analyses

Material properties for slope stability analyses were developed using both laboratory testing data (index and strength testing) and strength correlations from SPT and CPT data.

The following specific material properties were developed for each material, for use in the various stability analyses performed as part of this study:

- Unit Weight
- Drained and Undrained Shear Strength of Fine-Grained Soil Strata
- Post-Earthquake Shear Strengths For Foundation Silts

Material properties for the coal ash materials were conservatively estimated based on experience with similar materials. It is noted that the impounded ash layer has little to no influence on the stability analysis.

Unit Weight

Unit weight for the embankment fill and the foundation silts and silty clays were evaluated using measured results from samples collected. **Table E-5** below summarizes the unit weights as measured from samples collected:

Job	**Vectren A.B. Brown – Ash Pond System CCR Certification Report**	Project No.	**60442627**	Sheet	**4** of **13**
Description	**Appendix E**	Computed by	**ACI**	Date	**09/01/2016**
	Strength Characterization Calculations	Checked by	**VKG**	Date	**09/02/2016**

Table E-5: Total Unit Weight from Laboratory Testing Program

Strata	No. Tests	Min. (pcf)	Max. (pcf)	Average (pcf)
Embankment Fill	6	125.6	131.0	128.2
Foundation Silts	18	106.4	128.9	118.9
Foundation Silty Clays	14	122.7	128.5	123.5

The buttress fill materials were constructed in a controlled manner and density testing of each lift of fill was performed using nuclear methods. The results of all the field testing were reviewed and found to have very little variation. The average total unit weight among all test data points is approximately 123 pcf.

Drained and Undrained Shear Strength Fine-Grained Soil Strata

Shear Strength From Laboratory Triaxial Testing

Multiple laboratory triaxial tests were performed for the embankment fill, foundation silt and foundation silty clay soils over a range of confining pressures. In analyzing the test results, a number of definitions of failure were considered, including the point of peak deviator stress during the test, the deviator stress corresponding to an axial strain of 12% and 15%, and the point of the test with the maximum effective principle stress ratio (obliquity) from the tabulated CU test data. For both effective and total strength conditions, defining the failure point to coincide with the deviator stress corresponding to 15% strain was selected to establish the shear strength parameters.

As a result of having multiple laboratory CU tests, a failure envelope was defined for each material by plotting the failure points on a Modified Mohr-Coulomb plot (a p-q and p'-q plot), as described in Appendix D of the United States Corps of Engineers Engineer Manual EM-1110-2-1902 "Slope Stability."

For A.B. Brown, p-q and p'-q plots were constructed for each of the following materials based on multiple CU laboratory test data:

- Embankment Fill
- Foundation Silty Clay
- Foundation Silt

The p-q relationship is as follows:

$p = ½ (\sigma_3 + \sigma_1)$

$p' = ½ (\sigma'_3 + \sigma'_1)$

$q = ½ (\sigma_1 - \sigma_3)$

Where:

σ_1 = total major principal stress at failure (axial stress)
σ'_1 = effective major principal stress at failure (axial stress)
σ_3 = total minor principal stress at failure (confining stress)
σ'_3 = effective minor principal stress at failure (confining stress)
p = mean total normal stress at failure
p' = mean effective normal stress at failure
q = shear stress at failure

A fit line through the p-q and p'-q failure points will have an intercept of d and a slope of tangent α (or d' and α' for effective stress conditions). Equivalent Mohr-Coulomb parameters can then be computed as follows:

$\sin \phi = \tan y$ or $\sin \phi' = \tan y'$
$c = (d / \cos \phi)$ or $c' = (d' / \cos \phi')$

In fitting strength parameters to multiple test results, the US Army Corps of Engineers recommends selecting design parameters such that about two thirds of the total tests are above the failure envelope. As considered appropriate, occasional test points which were outliers to the high (stronger) side were removed from consideration on the plots.

Total and effective stress p-q plots for the embankment fill, foundation silty clay and foundation silt materials are shown on **Figures E-1 through E-6** below. The calculated shear strength parameters are also shown.

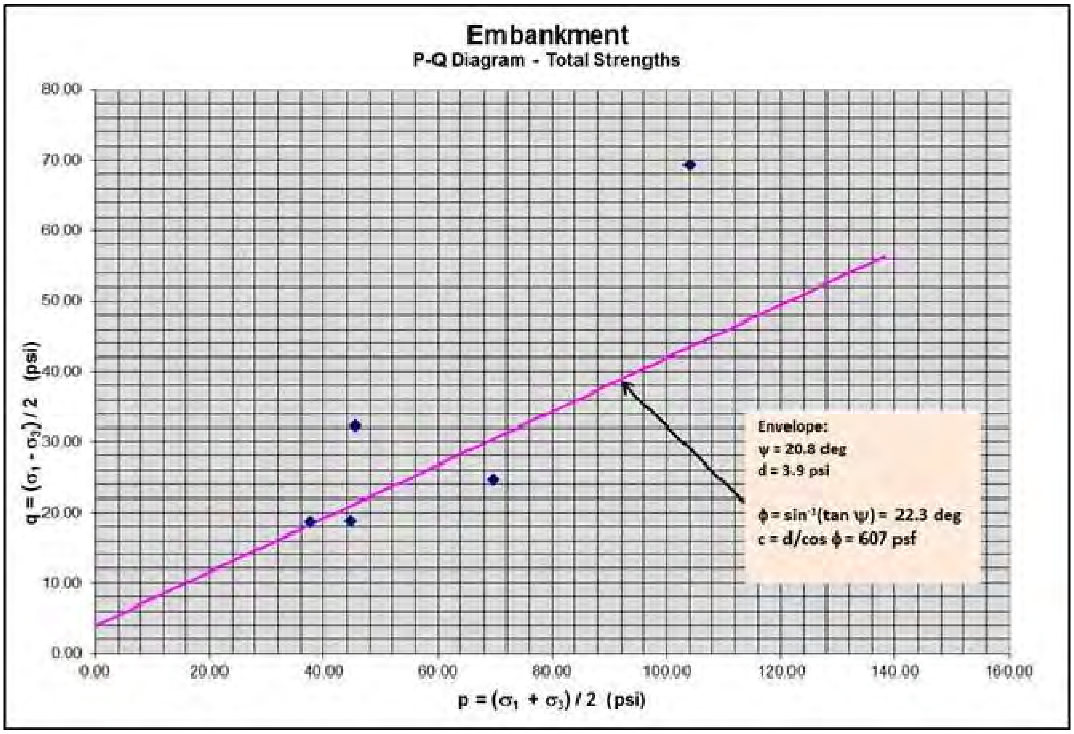


Figure E-1. Total Strength P-Q Plot for Embankment FIll at A.B. Brown

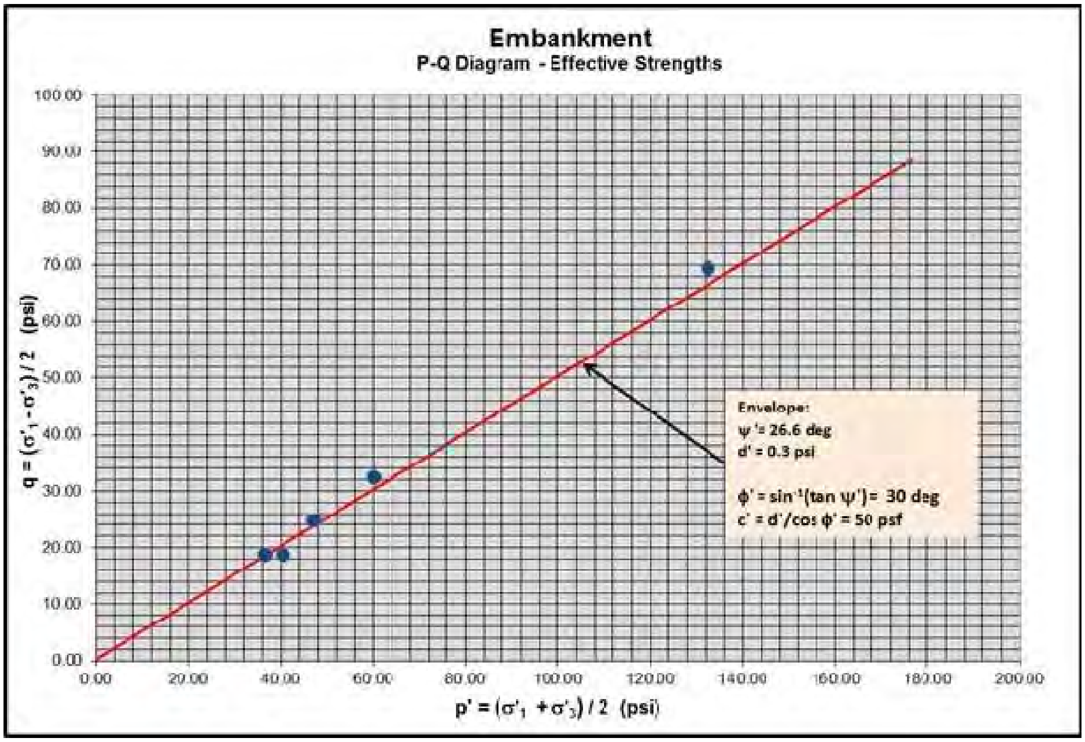


Figure E-2. Effective Strength P-Q Plot for Embankment Fill at A.B. Brown

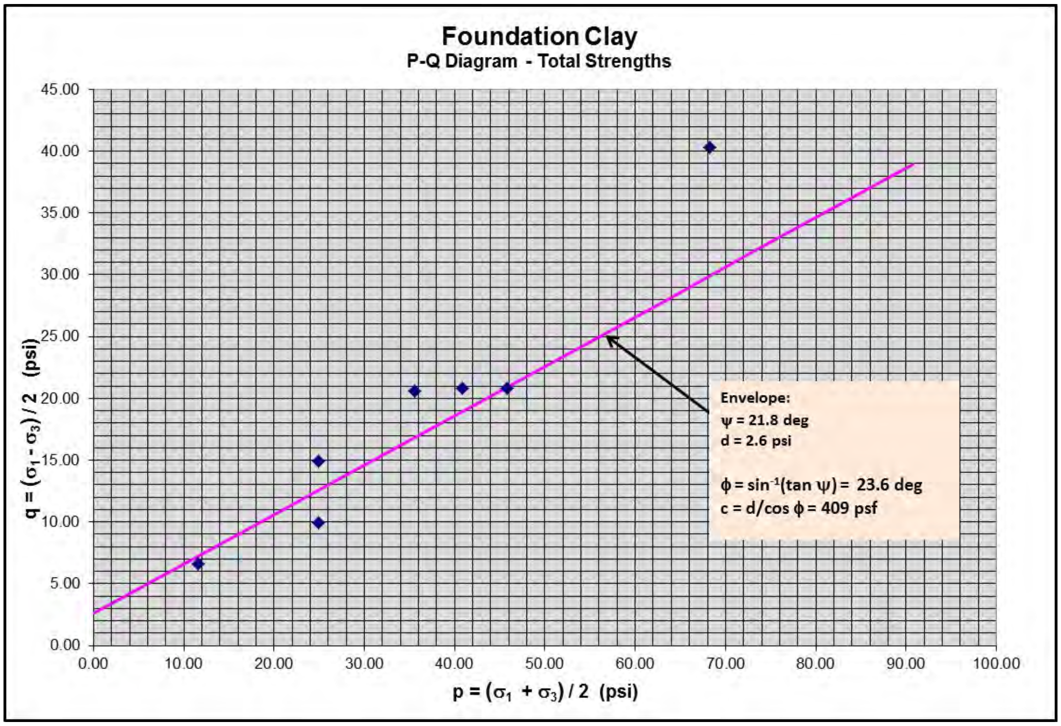


Figure E-3. Total Strength P-Q Plot for Foundation Silty Clay at A.B. Brown

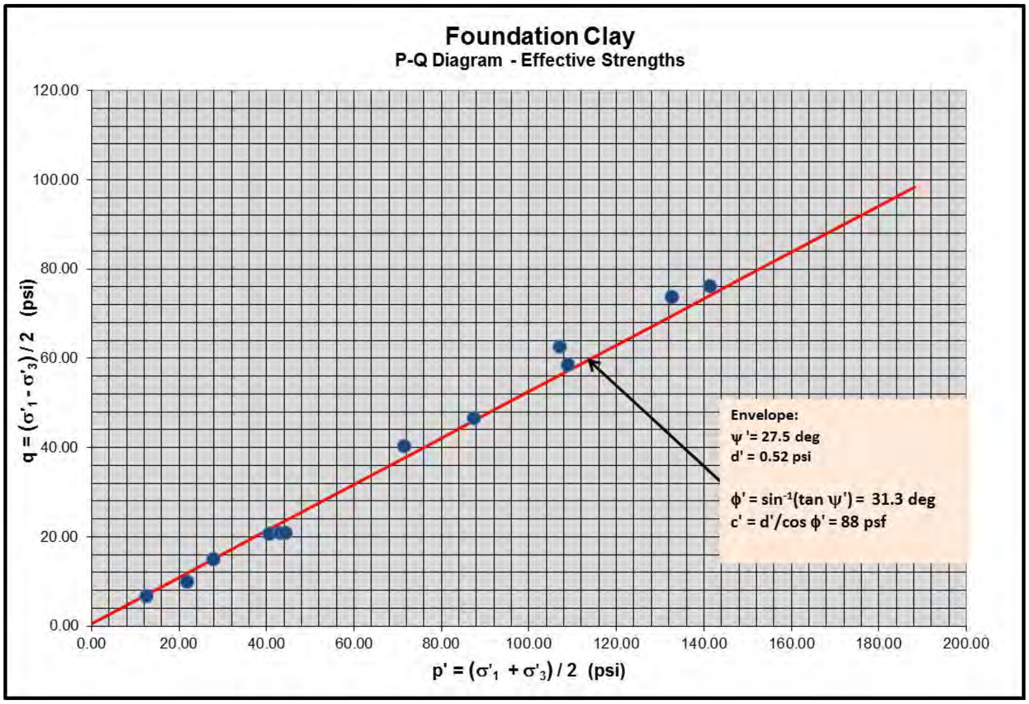


Figure E-4. Effective Strength P-Q Plot for Foundation Silty Clay at A.B. Brown

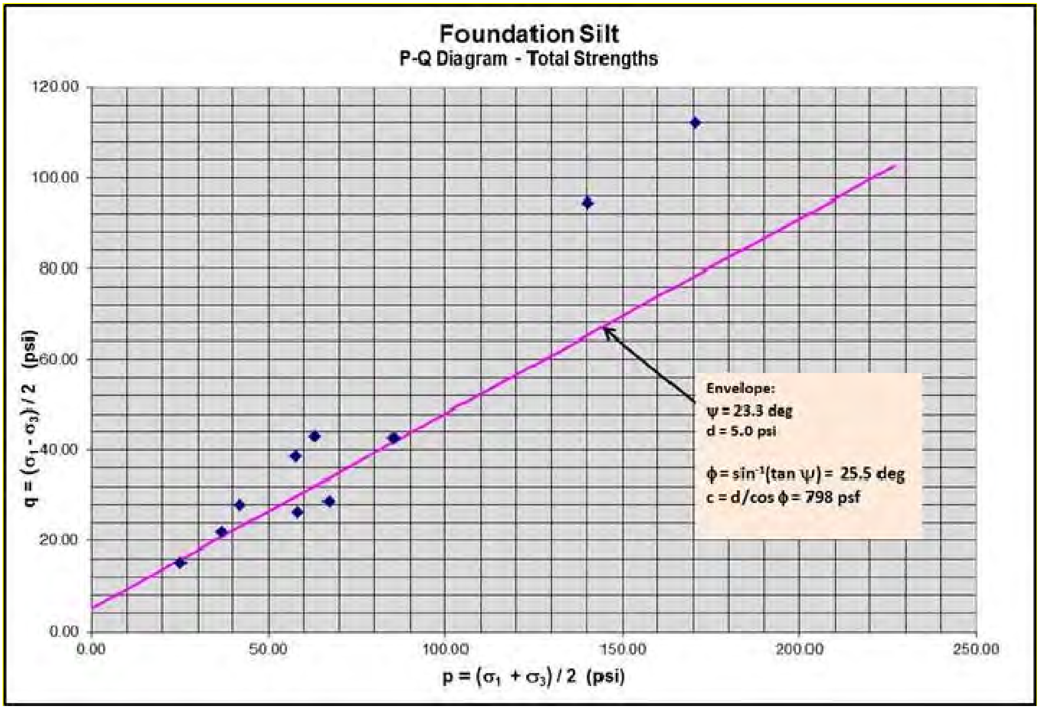


Figure E-5. Total Strength P-Q Plot for Foundation Silt at A.B. Brown

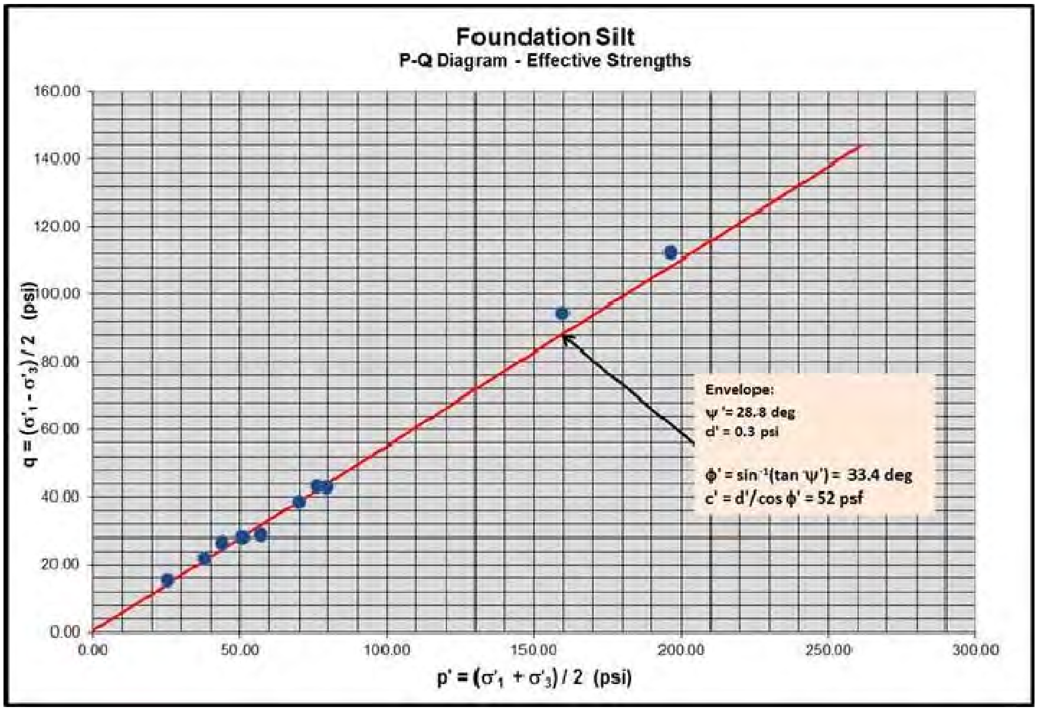


Figure E-6. Effective Strength P-Q Plot for Foundation Silt at A.B. Brown

Post-Earthquake Shear Strength for Foundation Silts

The liquefied strength (residual strength) of the foundation silts was estimated following procedures in Idriss and Boulanger (2008, 2014). Strength estimates presented in those references are based on empirical observations and back-analyses made at actual sites that have experienced liquefaction in past earthquakes and is based on correlations with SPT and CPT results. It relates the residual strength of a liquefied sand or silt (non- or low-plasticity material) to the normalized, fines-corrected resistance (SPT N-value or CPT tip resistance). Specifically, the method relates the equivalent fines-corrected clean sand SPT blow count, $(N_1)_{60CS\text{-}Sr}$, and CPT tip resistance $q_{c1Ncs\text{-}Sr}$ to the steady-state (post-liquefaction) shear strength. The strength is expressed as a ratio of the existing vertical overburden stress at any point in the layer, i.e., S_r / σ'_v.

The analyses performed as part of the SPT-based liquefaction screening analysis utilizes the fines-corrected blow count, $(N_1)_{60CS\text{-}Sr}$, and this parameter is calculated for each sample of silt within the spreadsheets that were created for that purpose. Cardno furnished the most recent hammer calibration data of the drill rig used on the site which was determined to be 81% efficient; this efficiency was used in determining the corrected N-values. These data were used to select the steady-state strength of the silt deposit, as follows:

- The $(N_1)_{60CS\text{-}Sr}$ for each silt sample among all borings were taken from the liquefaction screening analysis spreadsheet, and combined in a single graph. This is shown in **Figure E-7** below.
- The mean $(N_1)_{60CS\text{-}Sr}$ was determined from graph, and this value was selected for analysis purposes, to represent the silt deposit as a whole. From **Figure E-7**, mean $(N_1)_{60CS\text{-}Sr} = 12$.
- **Figure E-8** was then used to estimate the shear strength ratio, that corresponds to $(N_1)_{60CS\text{-}Sr} = 12$. As shown on the figure, the shear strength ratio of the silt was determined to be $Sr / \sigma'_v = 0.11$.

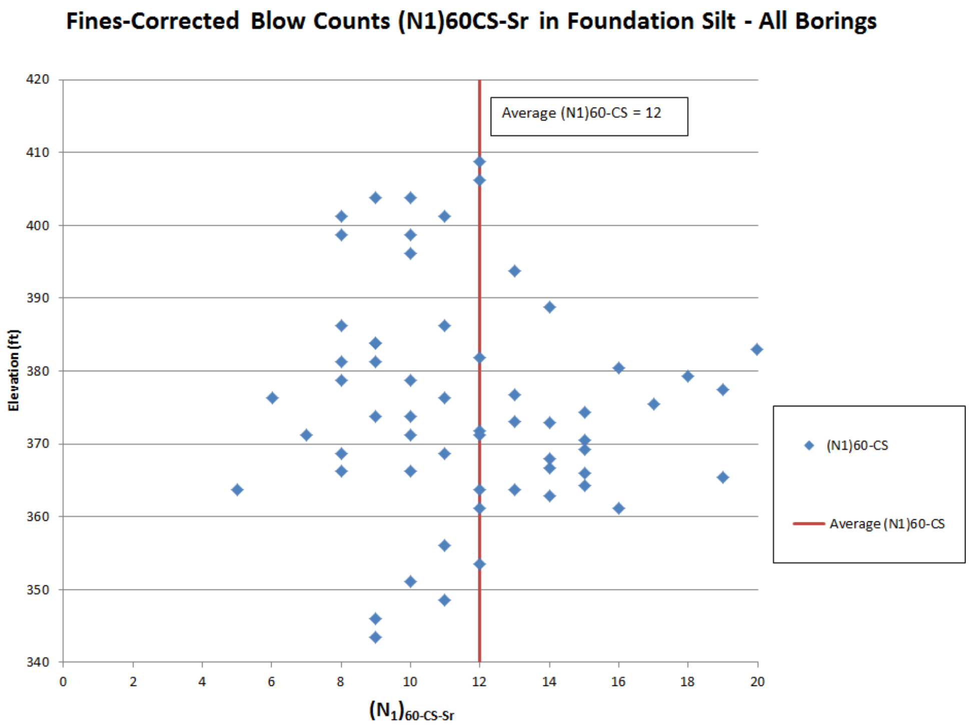
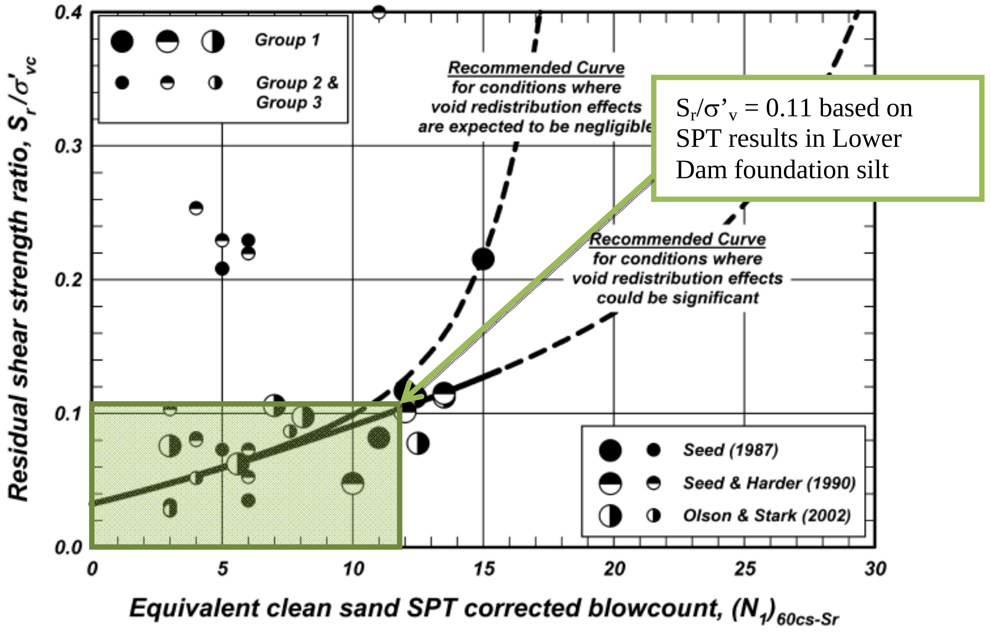


Figure E-7: Compilation of Fines Corrected Blow Counts in Foundation Silts



$S_r/\sigma'_v = 0.11$ based on SPT results in Lower Dam foundation silt

Figure E-8: Steady-State Strength Ratio vs.Equivalent Clean Sand Blow Count (Idriss and Boulanger, 2008)

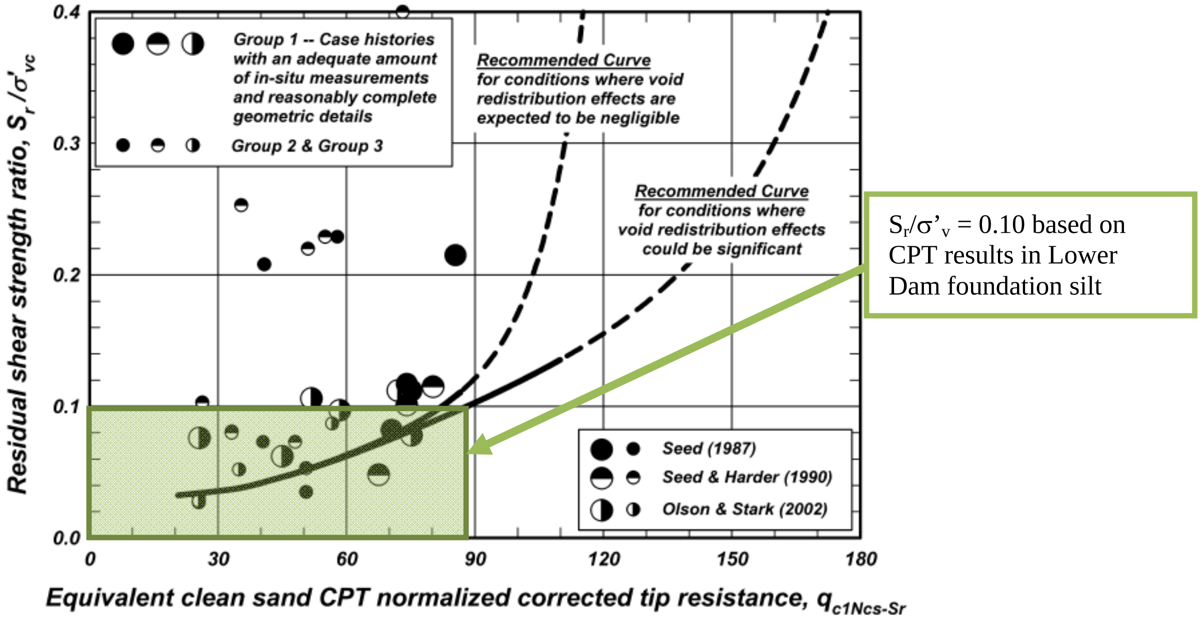
The analyses performed as part of the CPT-based liquefaction screening analysis utilized the fines-corrected tip resistance $q_{c1Ncs\text{-}Sr}$; this parameter is calculated for each tip-resistance data point within the silt deposits. CPT data points were taken every 0.05 meter (0.16 foot), essentially creating a continuous profile of data which were used to select the residual strength of the silt deposit.

The equivalent fines-corrected and normalized clean sand tip resistance, $q_{c1Ncs\ \text{-}Sr}$ taken from each CPT data point were calculated for all intervals within the silt layer. The average values from each CPT Sounding were tabulated, as shown in **Table E-6** below and an overall average tip resistance was determined ($q_{c1Ncs\ \text{-}Sr}$ = 87.1). This value was conservatively selected as the basis for determining the residual strength of silt for modeling purposes.

Table E-6: Summary of Equivalent Clean Sand Normalized CPT Tip Resistance q_{c1Ncs}

CPT Sounding	Adjacent Cardno Boring	Top of Silt Horizon Examined	Bottom of Silt Horizon Examined	Average q_{c1Ncs}	Overall Average q_{c1Ncs} (tsf)
		Elevation (ft)	Elevation (ft)	Tons per square foot (tsf)	
AECOM-C1	B-202	382.7	372.7	62.7	87.1
AECOM-C2	B-203	--	--	--	
AECOM-C3	B-219	409.5	397.0	98.0	
AECOM-C4	B-206	396.8	384.3	154.1	
		374.8	371.8	67.1	
		356.8	341.8	65.8	
AECOM-C5	B-205	389.0	361.5	74.6	

Figure E-9, reproduced from Idriss and Boulanger (2008), relates $q_{c1Ncs\ \text{-}Sr}$ to the residual shear strength. The strength is expressed as a ratio of the existing vertical overburden stress at any point in the layer, i.e., Sr / σ'_V. For $q_{c1Ncs\ \text{-}Sr}$ of 87.1, the estimated strength ratio is 0.10. This strength was selected to represent that portion of the foundation silt material that is anticipated to liquefy, for use in the post-liquefaction stability analyses.



S_r/σ'_v = 0.10 based on CPT results in Lower Dam foundation silt

Figure E-9. Steady-State Strength Ratio vs. Equivalent Clean Sand CPT Tip Resistance (Idriss and Boulanger, 2008)

V. Material Properties for Analysis

The table below summarizes the material parameters used as the basis for slope stability analysis, based on the analysis and strength selection procedures and considerations presented in the preceding sections.

Table E-7: Summary of Material Parameters used in Stability Analysis

Material	Unit Weight (pcf)	Effective (drained) Shear Strength Parameters		Total (undrained) Shear Strength Parameters		Post-Earthquake Shear Strength Parameters		
		c' (psf)	Φ' (°)	c (psf)	Φ (°)	c (psf)	Φ (°)	S_{ur}/σ'_{vc}
Embankment Fill	128	50	30	600	22	475	18	-
Foundation Silt	119	0	33	650	22	-	-	0.10
Foundation Clay	126	80	31	400	23	320	19	
Buttress Fill	123	45	27	540	20	425	16	-
Sluiced Ash	100	0	32	100	12	-	-	0.12
Bedrock	Assumed to be impenetrable in the slope stability models							

The following additional considerations were made in selecting the above parameters:

- As stated above, drilling and sampling in the Buttress Fill was not performed, because construction of the buttress occurred after the end of the geotechnical investigations for this project. The buttress is comprised of engineered fill material, similar to the Embankment Fill, and constructed using modern techniques. The buttress is therefore expected to have material parameters equal to or better than the dam embankment. As a conservative judgment, shear strength of the buttress is assumed to be 90% that of the Embankment Fill for analysis purposes.
- For impounded sluiced materials, strength properties were selected based on past experience and conservative engineering judgment. Furthermore, liquefaction was conservatively assumed by inspection, and steady-state strengths were also assigned based on conservative engineering judgment. It is noted that the impounded ash has little to no influence in the stability analyses.
- The total (undrained) strength parameters of the foundation silt layer used for analysis were reduced by 15% with respect to the values resulting from the P-Q diagrams, as a conservative engineering judgment.
- The fine-grained Foundation Silty Clay and Embankment Fill soils are generally stiff to very stiff materials. The laboratory triaxial strength test results did not indicate significant post-peak softening in these materials, which indicates low susceptibility to cyclic softening. Furthermore, the Embankment Fill was a mechanically compacted material.

  It is considered unlikely that the Embankment Fill and Foundation Silty Clay deposits will undergo strength loss as a result of cyclic loading in an earthquake, as these materials have stiff consistency and generally did not exhibit significant post-peak loss of strength in the triaxial tests. However, as a conservative consideration, a 20% strength loss has been assumed for analysis purposes for these materials, for the post-liquefaction analysis condition – i.e., the strengths in **Table E-7** for these materials for the post-earthquake condition correspond to 80% of the static undrained shear strength.

Appendix F
Slope Stability Analysis Calculations

Job	**A.B. Brown Generating Station – Ash Pond System CCR Certification Report**	Project No.	**60442627**	Sheet	**1** of **5**
Description	**Appendix F**	Computed by	**SAM**	Date	**08/25/2016**
	Slope Stability Analysis Calculations	Checked by	**VKG**	Date	**09/02/2016**

This calculation package summarizes the limit equilibrium slope stability analyses for both the static and seismic loading conditions performed in support of certifications of the Ash Pond Complex at Vectren's A.B. Brown Generating Station. The analyses pertain to the Lower Dam, which impounds the pond system. The methodology of the analyses are presented herein, along with figures, calculations and computer program outputs.

I. Objective

The objective for the slope stability analysis is to determine factors of safety (FoS) at critical cross section locations across the Lower Dam for the following loading cases:

- Static, Steady-State, Normal Pool Conditions;
- Static, Maximum Pool Surcharge Conditions;
- Seismic Slope Stability Analysis; and
- Post-Liquefaction Condition.

The factors of safety determined from each of these loading conditions will be utilized to determine if the requirements outlined by the United States Environmental Protection Agency (USEPA) CCR Rule under 40 Code of Federal Regulations (CFR) §257.73 (e) are met. The methodology used to perform the slope stability analyses and the results of the analyses are summarized in the subsequent sections listed below.

II. Development of Cross-Sections for Analysis

Five cross sections were identified for the stability evaluation of the Lower Dam. The analysis sections were selected based on factors including the height and steepness of the downstream embankment slope and subsurface conditions in the foundation of the embankment as revealed by the borings. Taken together, the five analysis sections are considered to comprehensively represent the Lower Dam. Each of the five analysis cross-sections are briefly summarized below:

- **Cross-Section A:** This section is located in the northern half of the dam and is representative of the surface and subsurface conditions in that area.
- **Cross-Section B:** This section is located central to the axis of the dam and models the tallest height (vertical difference between crest of the embankment and the toe of the embankment fill) of the dam embankment. The Foundation Silt layer (which is of interest because it is prone to liquefaction after a strong earthquake) featured most prominently within this cross section.
- **Cross-Section C:** This section is located along the southern half of the dam and is roughly in line with an existing pump house structure. The embankment is relatively tall at this section, similar to Section B.
- **Cross-Section D:** This section is representative of the southern end of the dam.
- **Cross-Section E:** This section is representative of the northern end of the dam, where bedrock rises sharply in elevation and the groundwater level at and beyond the toe of the dam is higher than at other areas.

The section locations are shown on **Figure F-1**.

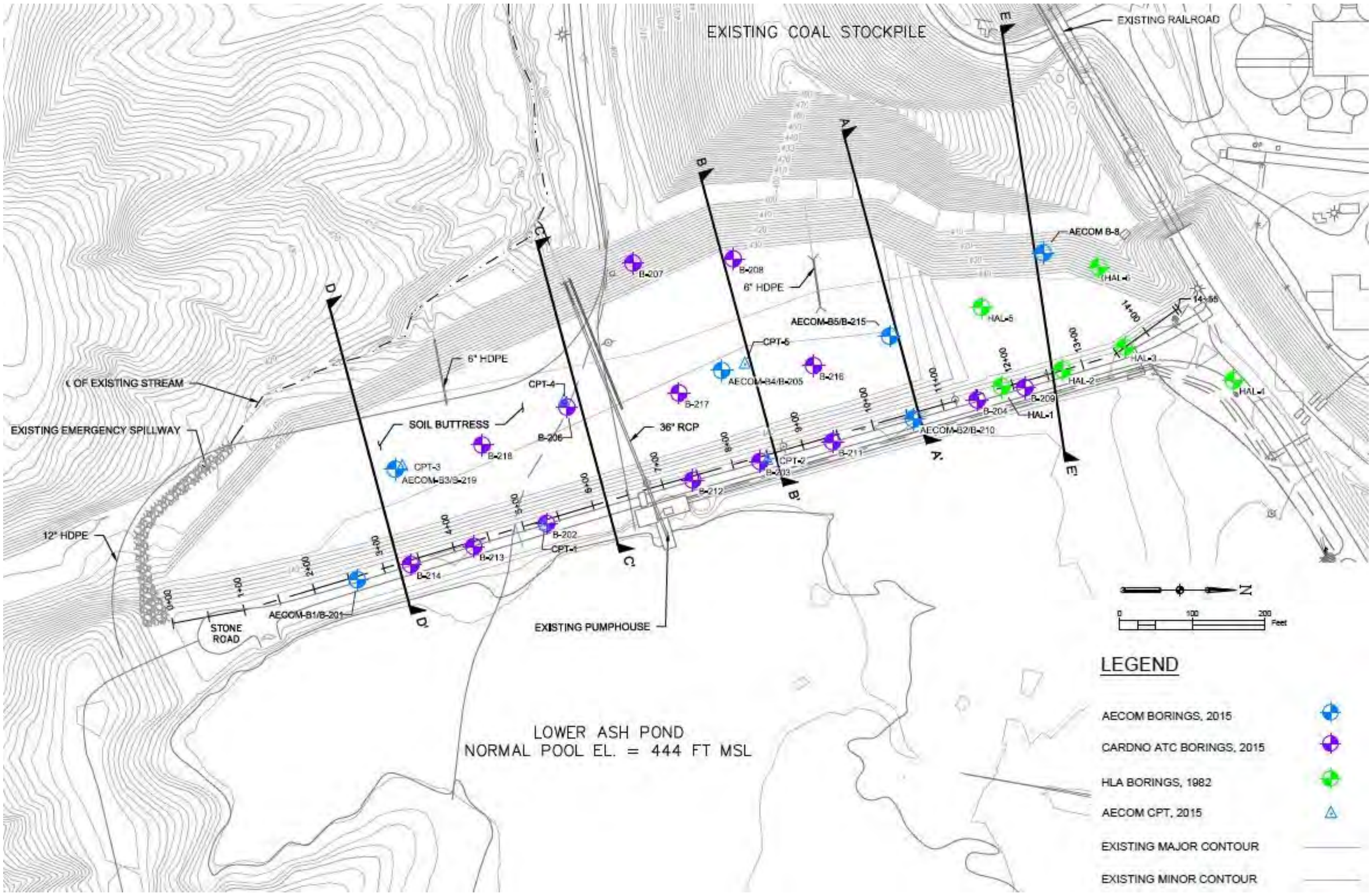


Figure F-1: Analysis Cross-Section Location Plan

III. Interpretation of Topography and Stratigraphy

Subsurface materials and extents (stratigraphy) at each cross section were developed by utilizing nearby subsurface explorations (CPTs and borings) from the various geotechnical investigations performed at the site. The subsurface strata generally encountered across the exploration locations can be generalized into five typical layers:

- Sluiced Ash
- Embankment Fill
- Foundation Silty Clay
- Foundation Silt
- Buttress Fill

These layers are described in detail in **Appendix E – Shear Strength Characterization**.

The topography for each analysis cross-section was determined based on ground surveys performed to support this project (for Cross-Section A thru D) or from the aerial basemapping provided by Vectren (for Section E). It is noted that the generating station's coal storage area lies directly to the west of cross-sections A, B, and E. The coal pile rises above the natural grade in this area and would act as a stabilizing surcharge against very large failure surfaces, such as are calculated under the post-liquefaction loading condition (described below). While the coal pile is a permanent feature of the station, the size of the pile can vary, depending on production needs at any given time. In the slope stability models for these sections, the surface grade at the toe of the gravity buttress (which will not change) was carried as constant to the west. This assumption conservatively eliminates any stabilizing effect of the coal pile on the stability models.

Stratigraphy was established from the subsurface information indicated by the borings and CPT soundings. The relevant CPT soundings and test borings that were used to develop subsurface stratigraphy at the five analysis sections are shown in **Table F-1**:

Table F-1: Summary of Geotechnical Explorations at Cross Sectional Locations

Cross-Section	Geotechnical Explorations Used
A-A	AECOM-B5, AECOM-B2, B-215, B-210
B-B	AECOM-B4, AECOM-C5, AECOM-C2, B-203, B-205, B-208
C-C	AECOM-C4, AECOM-C1, B-206, B-207, B-217
D-D	AECOM-B1, AECOM-B3, AECOM-C3, B-219, B-214, B-201
E-E	HLA-2, HLA-3, HLA-5, and HLA-6

A full set of AECOM's boring logs, including soil descriptions, types of sampling, and choice laboratory test results, is provided in **Appendix B** of the report. A CPT data report is provided in **Appendix C**, and complete laboratory testing results are provided in **Appendix D**.

IV. Groundwater Conditions

The phreatic surface under normal conditions was established using the water levels in the piezometers installed near the centerline of the dam (at boring location B-212 and B-217). Long term water levels in these piezometers are shown in **Table F-2**.

Table F-2: Long-Term Water Levels in Piezometers

Piezometer	Water El. (ft NAD 83)
B-212	424
B-217	406

Depths and elevations of free water as indicated in the borings and observations of water flow in the streams and ditches that lie to the west of the dam were also used to compare against the piezometer data for areas located away from the centerline (especially to estimate groundwater elevations in the far field beyond the toe of the dam). The available data and observations indicate that the static groundwater table beyond the toe of the dam lies at around El. 390 at the northern area of the dam, and at or below El. 380 at the central and southern areas.

The water elevations were drawn into the stability models as piezometric lines with straight line interpolation between the pool elevation and piezometer locations. AECOM reviewed the water elevations and cross-checked the interpolated phreatic surface with finite element seepage analysis using GeoStudio's SEEP/W software. Phreatic surfaces calculated in SEEP/W were in reasonable agreement with the straight-line interpolations from the available field groundwater measurements, but generally resulted in a lower phreatic level than the field measurements. Therefore, the straight-line interpolation was conservatively selected for the slope stability models.

V. Analysis Methodology

Analyses were performed using Spencer's Method which is a limit equilibrium slope stability analysis procedure satisfying both force and moment equilibrium. The computer program SLOPE/W 2007 by Geo-Slope International was utilized. The program analyzes a large number of potential slip surface geometries and identifies the geometry that results in a critical (i.e. lowest) factor of safety (FS). Additional information on the program is available at http://www.geo-slope.com/. Both circular and plane (block) shaped failure surfaces were analyzed, for the each of the loading cases considered.

Each section was analyzed for the following cases, which are in accordance with USEPA CCR Rule requirements:

- **Static, Steady-State, Normal Pool Condition:** This case models the embankment and connected buttress under static, long-term conditions, at normal water level within the impoundment. The USEPA CCR Rule requires a maximum storage pool factor of safety greater than or equal to 1.50.

  The steady-state condition used a normal pool elevation of 444.0 feet in the impoundment, which corresponds to the inlet elevation of the gooseneck outlet structure at the dam. This is the highest elevation that water can pool in the impoundment under normal conditions. The phreatic surface was modeled using piezometric lines and the straight-line interpolation between the pool level and the groundwater elevations in the reference piezometers and borings, as described in Section IV above.

- **Static, Maximum Surcharge Pool Condition:** This case models the conditions under short-term surcharge pool conditions, with the water level in the pond corresponding to the anticipated level during the design flood condition (which is a 1,000 year recurrence interval flood event for this site). This condition requires a minimum Factor of Safety greater than or equal to 1.40.

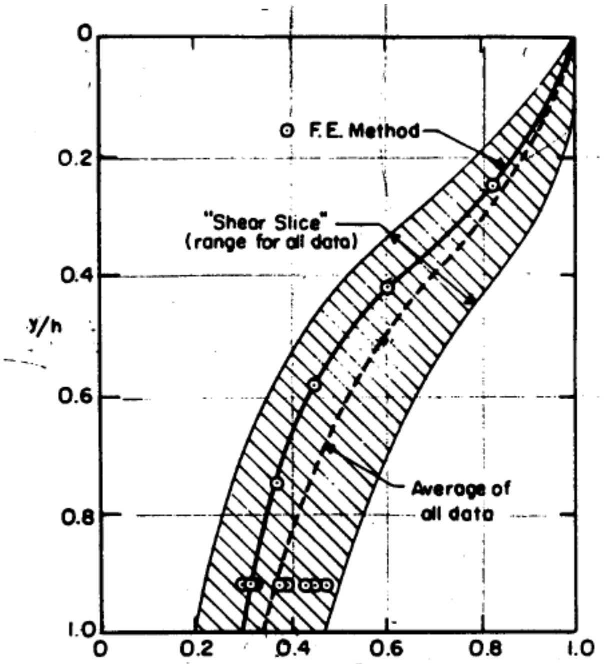
  The maximum surcharge pool elevation for this condition was set at El. 446.8 feet. This corresponds to the anticipated water level in the pond during the design flood event (which is a 1,000 year recurrence interval flood event for this site), as provided by the Hydraulics Engineer. For the maximum surcharge pool condition, the pool level in the pond was raised to the design flood level. The straight-line interpolation described above was adjusted accordingly to the raised water level. Therefore, the phreatic surface used for this loading condition corresponds to steady-state seepage to the raised pool level. This is a conservative representation, as the maximum storage pool water level is likely to be a short-term event and steady state seepage conditions through the dam are unlikely to develop.

- **Seismic Stability Condition:** These analyses incorporate a horizontal seismic coefficient k_h selected to be representative of expected loading during the design earthquake event (i.e., a “pseudostatic” analysis). The design earthquake event is one with a 2% probability of exceedance in 50 years (approximately 2,500 year recurrence interval), as required by the CCR Rule. The seismic coefficient was selected on the basis of the results of the site-specific Probabilistic Seismic Hazard Analysis (PSHA) and dynamic response analysis (See **Appendices G and H**). The analyses utilized peak undrained strength parameters for soils that are not considered to be rapidly draining materials (including the dam embankment and buttress soils, silty clay foundation stratum, and silt foundation stratum). The phreatic surface and pore water pressures corresponding to the steady state pool from the static analyses were utilized. This condition requires a minimum Factor of Safety greater than or equal to 1.00.

  Pool elevation in the pond and the phreatic surface for the seismic loading condition were the same as utilized in the steady-state normal pool loading condition.

  The pseudostatic coefficient was selected using the simplified procedure outlined by Makdisi and Seed (1977), and based on earthquake ground motions established from the probabilistic seismic hazard analysis (PSHA) and dynamic response analyses performed for the site (see **Attachment G** and **H**). Specifically, the pseudostatic coefficient was taken as the parameter k_{max}, which represents the peak average acceleration along the failure surface. As shown in **Figure F-2** below (excerpted from the above reference), the ratio k_{max}/u_{max} (where u_{max} is the peak acceleration at the crest of the embankment) for a full height failure surface (y/H = 1.0) is 0.34. The value for full-height failure surfaces is pertinent to the slope stability analyses, as these analyses are focused on global failure surfaces that could release the contents of the impoundment, if mobilized.

  Peak ground accelerations at the crest of the dam were determined in the dynamic response analysis (see **Attachment H**), for each of four reference time histories generated from the PSHA. The results from the QUAD4M model representing the existing condition of the dam (with the stabilizing soil buttress in place) were used to establish the crest PGA. The average crest PGA among the time histories from this model was 0.53g. Therefore, the pseudostatic coefficient k_h was estimated as k_h= 0.34*0.53g = 0.18g. This value was input as the seismic coefficient in the slope stability models.



VARIATION OF "MAXIMUM ACCELERATION RATIO" WITH DEPTH OF SLIDING MASS

Figure F-2: Determination of Maximum Average Acceleration Along Failure Surface

- **Post-Liquefaction Condition:** These analyses were performed at each stability cross section where liquefaction triggering analysis indicates potential liquefaction of non-plastic materials or cyclic softening of fine-grained soils. The purpose of the post-liquefaction stability analysis is to assess stability conditions immediately following the design seismic event. No horizontal seismic coefficient is included in these analyses, but selection of strength parameters for the analyses takes into account the potential for the softening/weakening of the soils as a result of pore pressures generated in sand-like materials, or cyclic softening in clay-like materials due to the earthquake shaking. Liquefaction potential analysis was performed on the foundation silt deposits, using cyclic stress ratios (CSRs) determined from finite element dynamic response analysis, and cyclic resistance ratios (CRRs) determined from the results of cyclic direct simple shear testing. The liquefaction potential analysis is presented in **Appendix I**. As discussed in subsequent sections, these analyses predict that the silt deposit will liquefy as a result of the design earthquake. In the post-liquefaction stability analyses, steady state (liquefied) strength was therefore assigned to the silt.

  Pool elevation in the pond and the phreatic surface for the post-liquefaction loading condition were the same as utilized in the steady-state normal pool loading condition.

The CCR Rule requires a minimum Factor of Safety greater than or equal to 1.20 for the post-liquefaction slope stability analysis.

VI. Material Properties for Analysis

Material properties for slope stability analyses were developed using both laboratory testing data (index and strength testing) and strength correlations from CPT and SPT data. Details of the material characterization and strength parameter selection for each stratum are provided in **Appendix E** of this report. The properties used in the stability analysis are summarized in the table below:

Table F-3: Summary of Material Parameters used in Stability Analysis

Material	Unit Weight (pcf)	Effective (drained) Shear Strength Parameters		Total (undrained) Shear Strength Parameters		Post-Earthquake Shear Strength Parameters		
		c' (psf)	Φ' (°)	c (psf)	Φ (°)	c (psf)	Φ (°)	S_{ur}/σ'_{vc}
Embankment Fill	128	50	30	600	22	475	18	-
Foundation Silt	119	0	33	650	22	-	-	0.10
Foundation Clay	126	80	31	400	23	320	19	
Buttress Fill	123	45	27	540	20	425	16	-
Sluiced Ash	100	0	32	100	12	-	-	0.12
Bedrock	Assumed to be impenetrable in the slope stability models							

VII. Results

Table F-4 summarizes the results of the stability analyses for each section, and output figures from the SLOPE/W models are provided at the back of this appendix.

Table F-4: Summary of Minimum Slope Stability Factors

Load Case	CCR Rule Criteria	Failure Geometry	A-A	B-B	C-C	D-D	E-E
Steady State (Normal Pool)	FS ≥ 1.50	Circular	3.43	3.42	3.21	3.32	3.65
		Block	3.48	3.72	3.36	3.38	3.36
Surcharge Pool (Flood)	FS ≥ 1.40	Circular	3.33	3.32	3.06	3.22	3.61
		Block	3.57	3.48	3.24	3.30	3.36
Seismic (Pseudostatic)	FS ≥ 1.00	Circular	1.51	1.56	1.32	1.49	1.56
		Block	1.62	1.64	1.38	1.58	1.65
Post-Liquefaction	FS ≥ 1.20	Circular	1.61	1.61	1.55	2.17	1.69
		Block	1.23	1.25	1.32	1.25	1.32

VIII. Conclusions

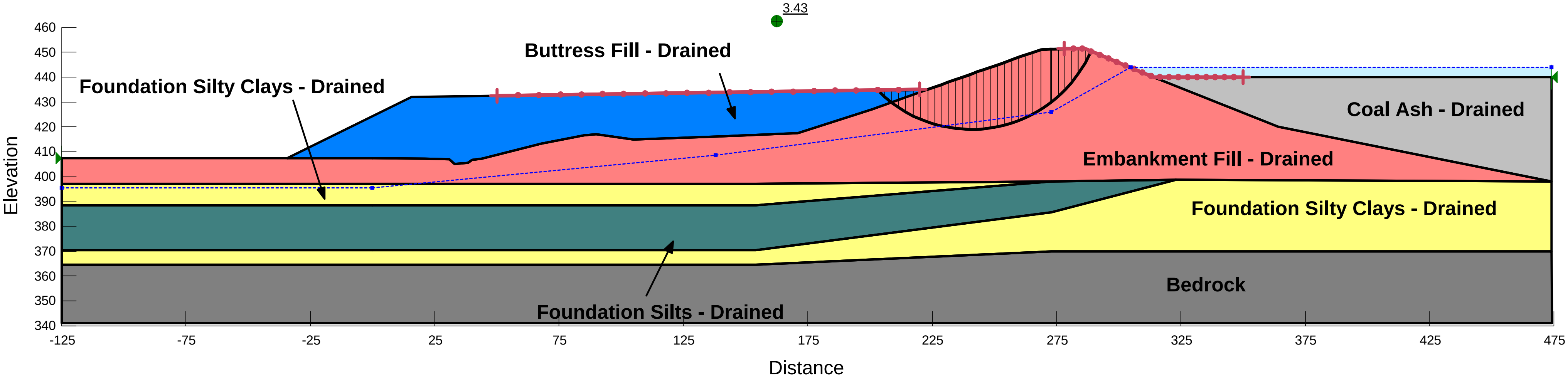
Calculated factors of safety at all cross-sections are greater than or equal to the minimum values required in USEPA CCR Rule §257.73(e), for all loading conditions considered.

IX. References

Makdisi, F.I. and Seed, B. H., August, 1977. “A Simplified Procedure for Estimating Earthquake-Induced Deformations in Dams and Embankments”, Earthquake Engineering Research Center Report No. UCB/EERC-77/19, University of California, Berkeley, CA.

Ash Pond Lower Dam Buttress Evaluation
Vectren A.B. Brown Station

CCR Rule Safety Factor Assessment
Static Storage Pool - Critical Circular Surface Failure Geometry
Cross-Section A
Date: 10/8/2016



Material Properties

Embankment Fill - Drained
Unit Weight: 128 pcf
Cohesion: 50 psf
Phi: 30 °

Coal Ash - Drained
Unit Weight: 100 pcf
Cohesion: 0 psf
Phi: 32 °

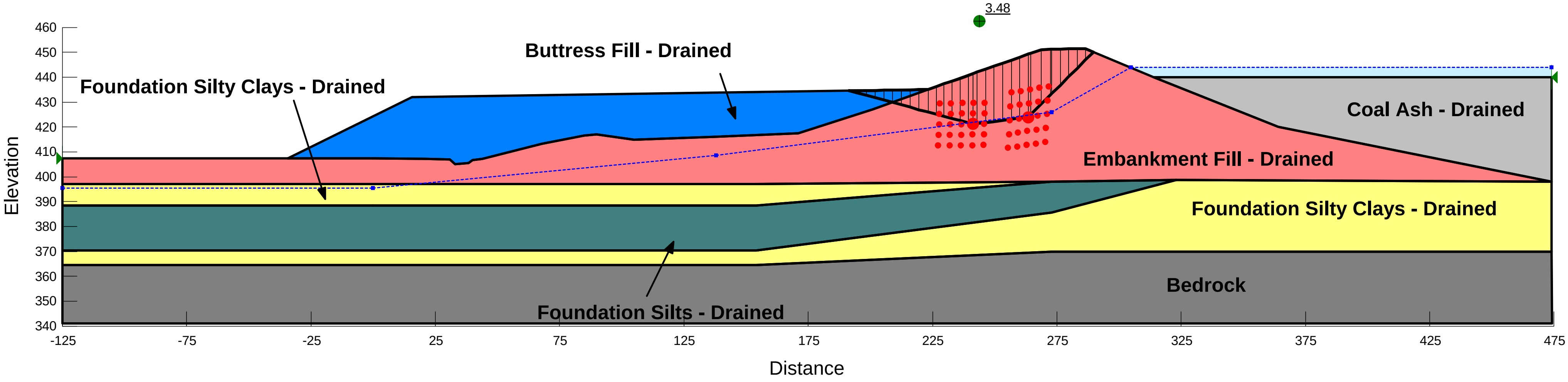
Foundation Silty Clays - Drained
Unit Weight: 126 pcf
Cohesion: 80 psf
Phi: 31 °

Foundation Silts - Drained
Unit Weight: 119 pcf
Cohesion: 0 psf
Phi: 33 °

Buttress Fill - Drained
Unit Weight: 123 pcf
Cohesion: 45 psf
Phi: 27 °

Ash Pond Lower Dam Buttress Evaluation
Vectren A.B. Brown Station

CCR Rule Safety Factor Assessment
Static Storage Pool - Critical Block Failure Surface Geometry
Cross-Section A
Date: 10/8/2016



Material Properties

Embankment Fill - Drained
Unit Weight: 128 pcf
Cohesion: 50 psf
Phi: 30 °

Coal Ash - Drained
Unit Weight: 100 pcf
Cohesion: 0 psf
Phi: 32 °

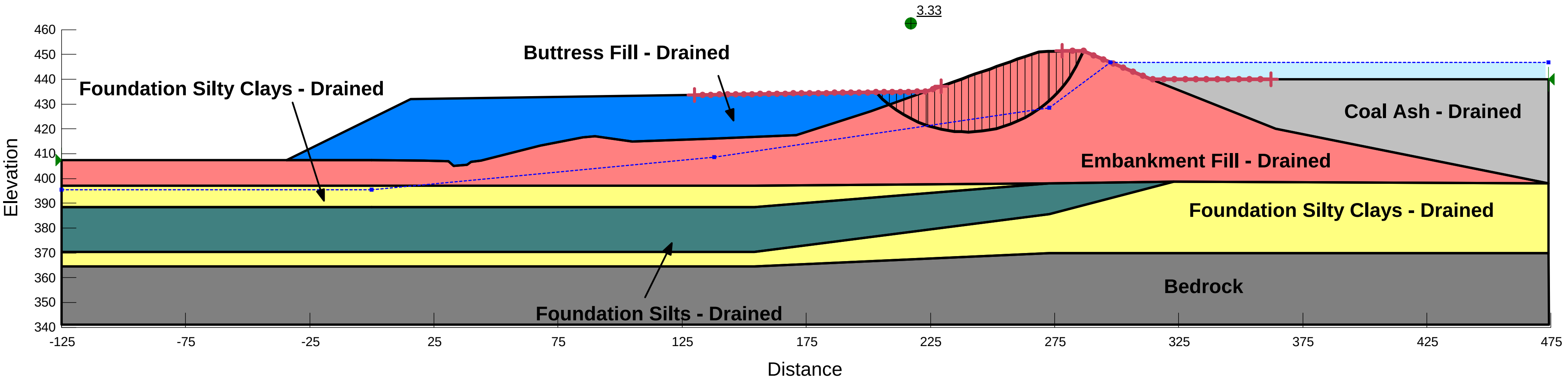
Foundation Silty Clays - Drained
Unit Weight: 126 pcf
Cohesion: 80 psf
Phi: 31 °

Foundation Silts - Drained
Unit Weight: 119 pcf
Cohesion: 0 psf
Phi: 33 °

Buttress Fill - Drained
Unit Weight: 123 pcf
Cohesion: 45 psf
Phi: 27 °

Ash Pond Lower Dam Buttress Evaluation
Vectren A.B. Brown Station

CCR Rule Safety Factor Assessment
Static Surcharge Pool - Critical Circular Failure Geometry
Cross-Section A
Date: 10/8/2016



Material Properties

Embankment Fill - Drained
Unit Weight: 128 pcf
Cohesion: 50 psf
Phi: 30 °

Coal Ash - Drained
Unit Weight: 100 pcf
Cohesion: 0 psf
Phi: 32 °

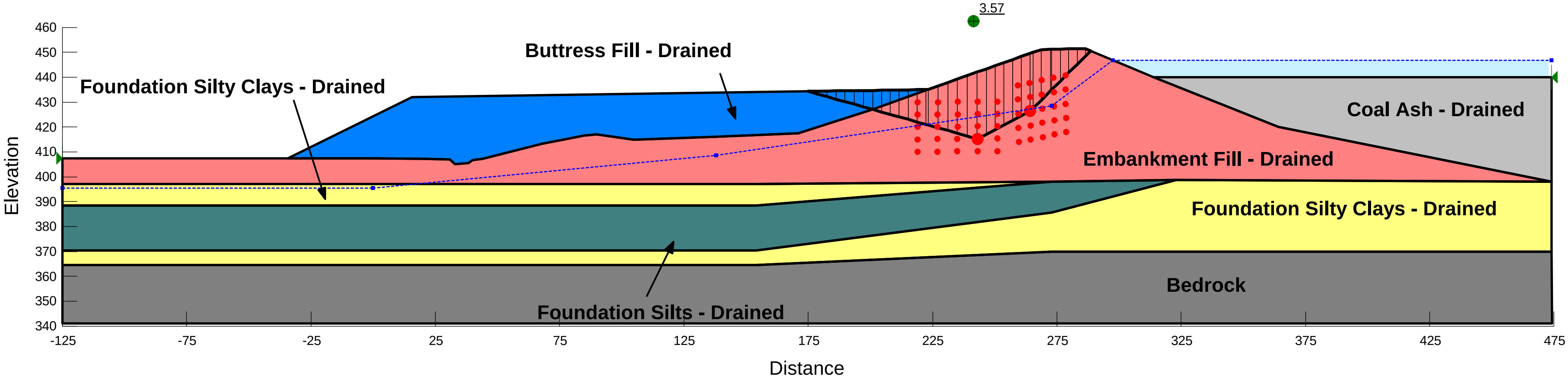
Foundation Silty Clays - Drained
Unit Weight: 126 pcf
Cohesion: 80 psf
Phi: 31 °

Foundation Silts - Drained
Unit Weight: 119 pcf
Cohesion: 0 psf
Phi: 33 °

Buttress Fill - Drained
Unit Weight: 123 pcf
Cohesion: 45 psf
Phi: 27 °

Ash Pond Lower Dam Buttress Evaluation
Vectren A.B. Brown Station

CCR Rule Safety Factor Assessment
Static Surcharge Pool - Critical Block Failure Surface Geometry
Cross-Section A
Date: 10/8/2016



Material Properties

Embankment Fill - Drained
Unit Weight: 128 pcf
Cohesion: 50 psf
Phi: 30 °

Coal Ash - Drained
Unit Weight: 100 pcf
Cohesion: 0 psf
Phi: 32 °

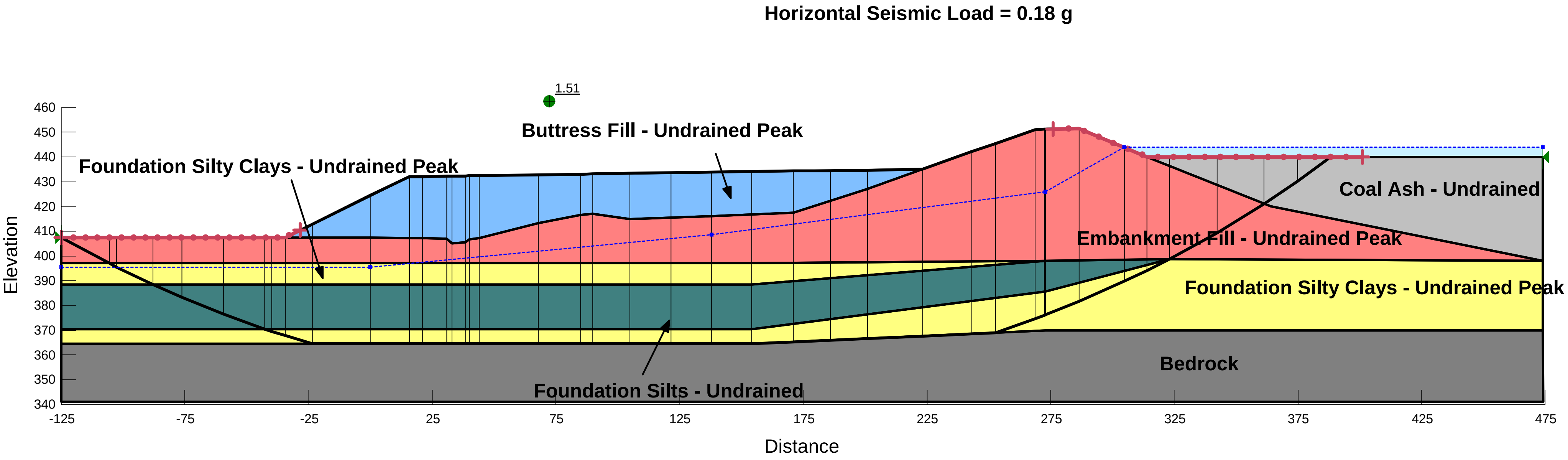
Foundation Silty Clays - Drained
Unit Weight: 126 pcf
Cohesion: 80 psf
Phi: 31 °

Foundation Silts - Drained
Unit Weight: 119 pcf
Cohesion: 0 psf
Phi: 33 °

Buttress Fill - Drained
Unit Weight: 123 pcf
Cohesion: 45 psf
Phi: 27 °

Ash Pond Lower Dam Buttress Evaluation
Vectren A.B. Brown Station

CCR Rule Safety Factor Assessment
Seismic - Critical Circular Buttress Failure Geometry
Cross-Section A
Date: 10/10/2016



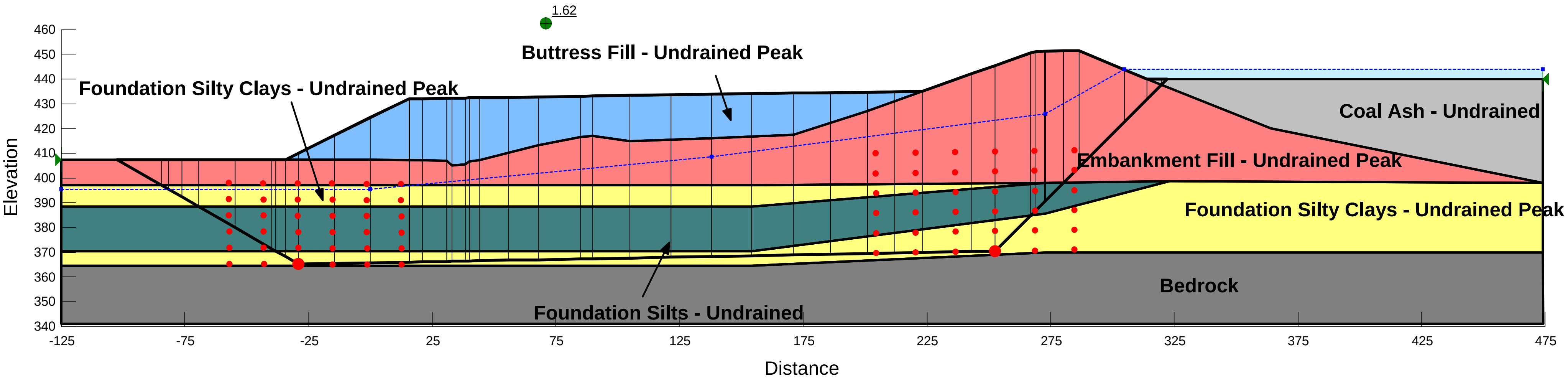
Material Properties

Embankment Fill - Undrained Peak	**Coal Ash - Undrained**	**Foundation Silty Clays - Undrained Peak**	**Foundation Silts - Undrained**	**Buttress Fill - Undrained Peak**
Unit Weight: 128 pcf	**Unit Weight: 100 pcf**	**Unit Weight: 126 pcf**	**Unit Weight: 119 pcf**	**Unit Weight: 123 pcf**
Cohesion: 600 psf	**Cohesion: 100 psf**	**Cohesion: 400 psf**	**Cohesion: 650 psf**	**Cohesion: 540 psf**
Phi: 22 °	**Phi: 12 °**	**Phi: 23 °**	**Phi: 22 °**	**Phi: 20 °**

Ash Pond Lower Dam Buttress Evaluation
Vectren A.B. Brown Station

CCR Rule Safety Factor Assessment
Seismic - Critical Block Failure Surface Geometry
Cross-Section A
Date: 10/10/2016

Horizontal Seismic Load = 0.18 g

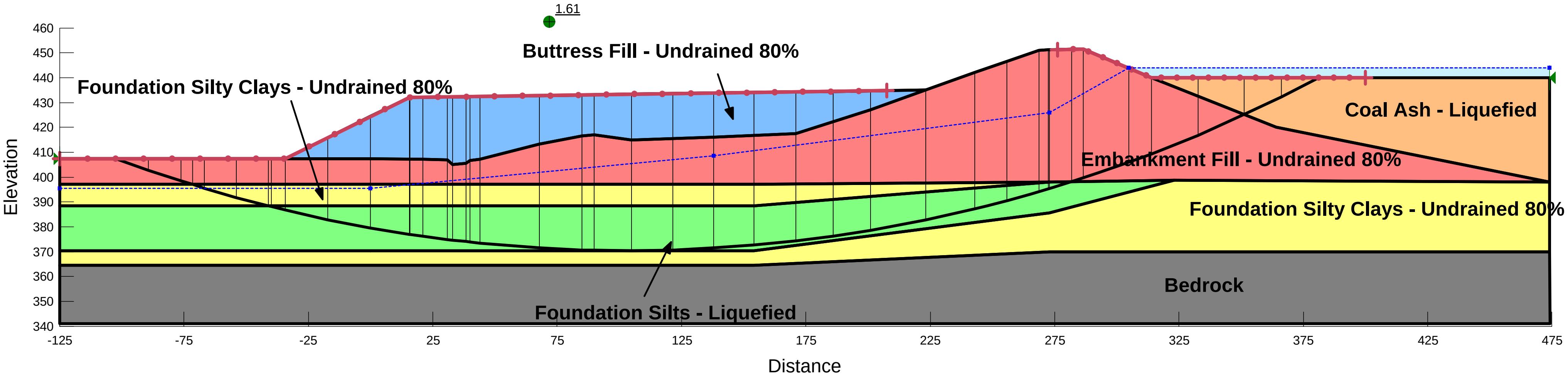


Material Properties

Embankment Fill - Undrained Peak	**Coal Ash - Undrained**	**Foundation Silty Clays - Undrained Peak**	**Foundation Silts - Undrained**	**Buttress Fill - Undrained Peak**
Unit Weight: 128 pcf	**Unit Weight: 100 pcf**	**Unit Weight: 126 pcf**	**Unit Weight: 119 pcf**	**Unit Weight: 123 pcf**
Cohesion: 600 psf	**Cohesion: 100 psf**	**Cohesion: 400 psf**	**Cohesion: 650 psf**	**Cohesion: 540 psf**
Phi: 22 °	**Phi: 12 °**	**Phi: 23 °**	**Phi: 22 °**	**Phi: 20 °**

Ash Pond Lower Dam Buttress Evaluation
Vectren A.B. Brown Station

CCR Rule Safety Factor Assessment
Post-Liquefaction - Critical Circular Surface Failure Geometry
Cross-Section A
Date: 9/13/2016



Material Properties

Embankment Fill - Undrained 80%
Unit Weight: 128 pcf
Cohesion: 475 psf
Phi: 18 °

Coal Ash - Liquefied
Unit Weight: 100 pcf
Tau/Sigma Ratio: 0.12
Minimum Strength: 0

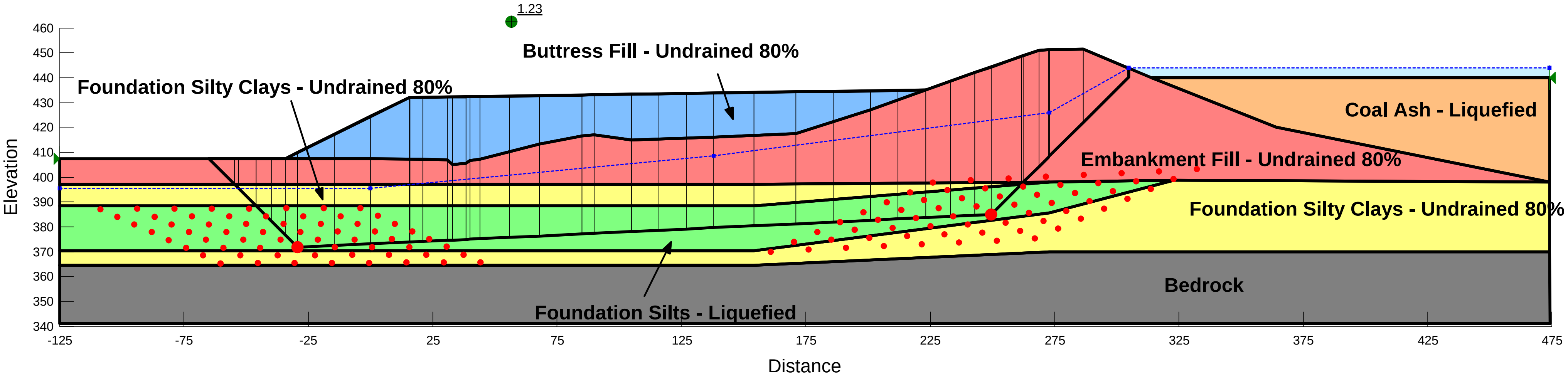
Foundation Silty Clays - Undrained 80%
Unit Weight: 126 pcf
Cohesion: 320 psf
Phi: 19 °

Foundation Silts - Liquefied
Unit Weight: 119 pcf
Tau/Sigma Ratio: 0.1
Minimum Strength: 100

Buttress Fill - Undrained 80%
Unit Weight: 123 pcf
Cohesion: 425 psf
Phi: 16 °

Ash Pond Lower Dam Buttress Evaluation
Vectren A.B. Brown Station

CCR Rule Safety Factor Assessment
Post-Liquefaction - Critical Block Failure Surface Geometry
Cross-Section A
Date: 9/13/2016



Material Properties

Embankment Fill - Undrained 80%
Unit Weight: 128 pcf
Cohesion: 475 psf
Phi: 18 °

Coal Ash - Liquefied
Unit Weight: 100 pcf
Tau/Sigma Ratio: 0.12
Minimum Strength: 0

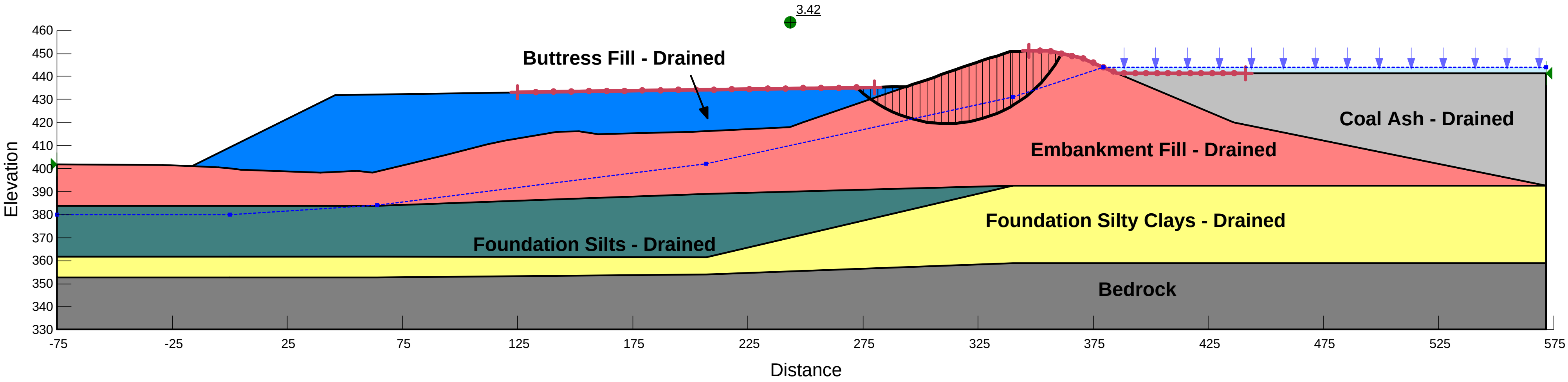
Foundation Silty Clays - Undrained 80%
Unit Weight: 126 pcf
Cohesion: 320 psf
Phi: 19 °

Foundation Silts - Liquefied
Unit Weight: 119 pcf
Tau/Sigma Ratio: 0.1
Minimum Strength: 100

Buttress Fill - Undrained 80%
Unit Weight: 123 pcf
Cohesion: 425 psf
Phi: 16 °

Ash Pond Lower Dam Buttress Evaluation
Vectren A.B. Brown Station

CCR Rule Safety Factor Assessment
Static Storage Pool - Critical Circular Surface Failure Geometry
Cross-Section B
Date: 10/8/2016



Material Properties

Embankment Fill - Drained
Unit Weight: 128 pcf
Cohesion: 50 psf
Phi: 30 °

Coal Ash - Drained
Unit Weight: 100 pcf
Cohesion: 0 psf
Phi: 32 °

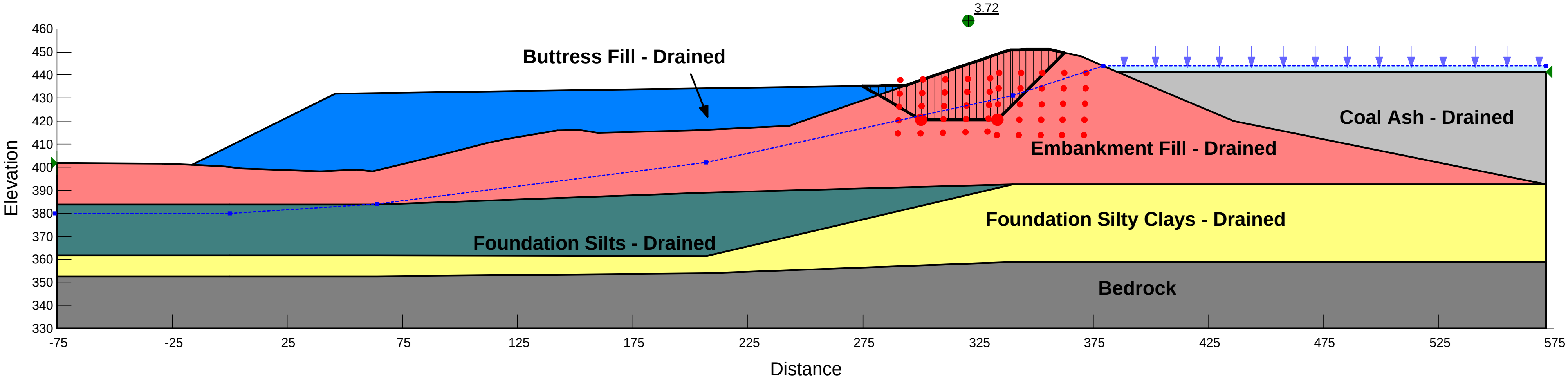
Foundation Silty Clays - Drained
Unit Weight: 126 pcf
Cohesion: 80 psf
Phi: 31 °

Foundation Silts - Drained
Unit Weight: 119 pcf
Cohesion: 0 psf
Phi: 33 °

Buttress Fill - Drained
Unit Weight: 123 pcf
Cohesion: 45 psf
Phi: 27 °

Ash Pond Lower Dam Buttress Evaluation
Vectren A.B. Brown Station

CCR Rule Safety Factor Assessment
Static Storage Pool - Critical Block Failure Surface Geometry
Cross-Section B
Date: 10/8/2016



Material Properties

Embankment Fill - Drained
Unit Weight: 128 pcf
Cohesion: 50 psf
Phi: 30 °

Coal Ash - Drained
Unit Weight: 100 pcf
Cohesion: 0 psf
Phi: 32 °

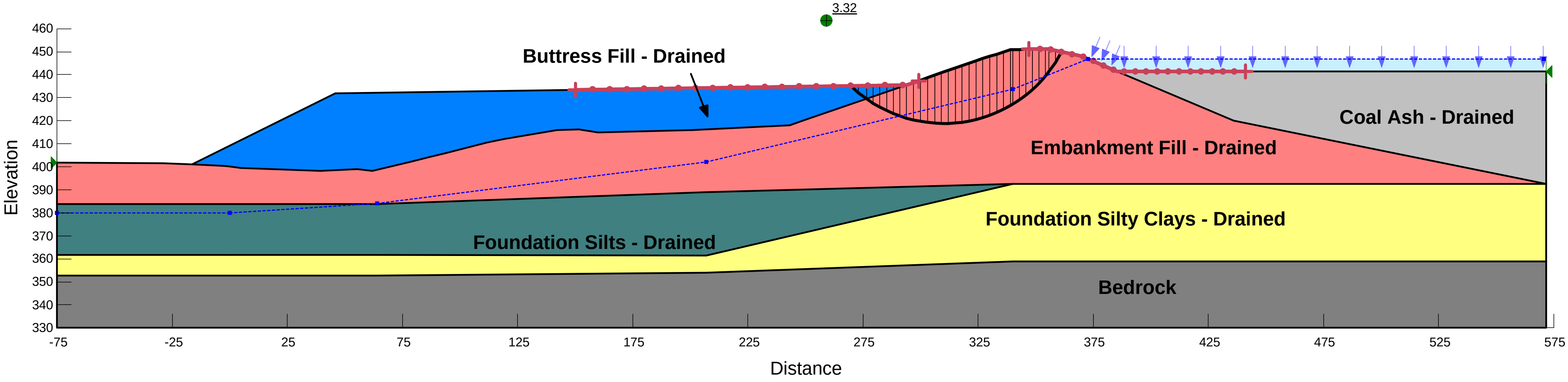
Foundation Silty Clays - Drained
Unit Weight: 126 pcf
Cohesion: 80 psf
Phi: 31 °

Foundation Silts - Drained
Unit Weight: 119 pcf
Cohesion: 0 psf
Phi: 33 °

Buttress Fill - Drained
Unit Weight: 123 pcf
Cohesion: 45 psf
Phi: 27 °

Ash Pond Lower Dam Buttress Evaluation
Vectren A.B. Brown Station

CCR Rule Safety Factor Assessment
Static Surcharge Pool - Critical Circular Surface Failure Geometry
Cross-Section B
Date: 10/8/2016



Material Properties

Embankment Fill - Drained
Unit Weight: 128 pcf
Cohesion: 50 psf
Phi: 30 °

Coal Ash - Drained
Unit Weight: 100 pcf
Cohesion: 0 psf
Phi: 32 °

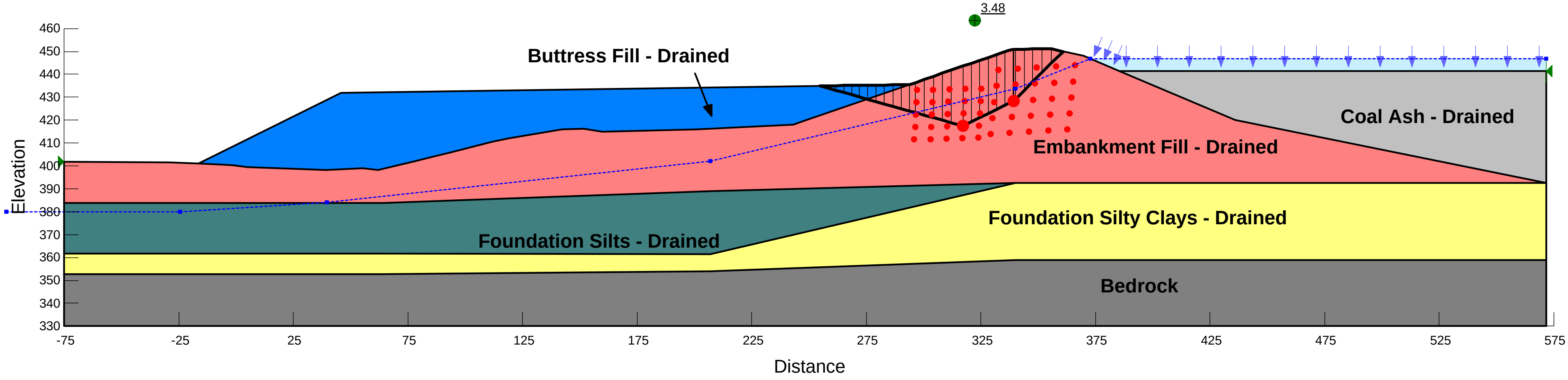
Foundation Silty Clays - Drained
Unit Weight: 126 pcf
Cohesion: 80 psf
Phi: 31 °

Foundation Silts - Drained
Unit Weight: 119 pcf
Cohesion: 0 psf
Phi: 33 °

Buttress Fill - Drained
Unit Weight: 123 pcf
Cohesion: 45 psf
Phi: 27 °

Ash Pond Lower Dam Buttress Evaluation
Vectren A.B. Brown Station

CCR Rule Safety Factor Assessment
Static Surcharge Pool - Critical Block Failure Surface Geometry
Cross-Section B
Date: 10/8/2016



Material Properties

Embankment Fill - Drained
Unit Weight: 128 pcf
Cohesion: 50 psf
Phi: 30 °

Coal Ash - Drained
Unit Weight: 100 pcf
Cohesion: 0 psf
Phi: 32 °

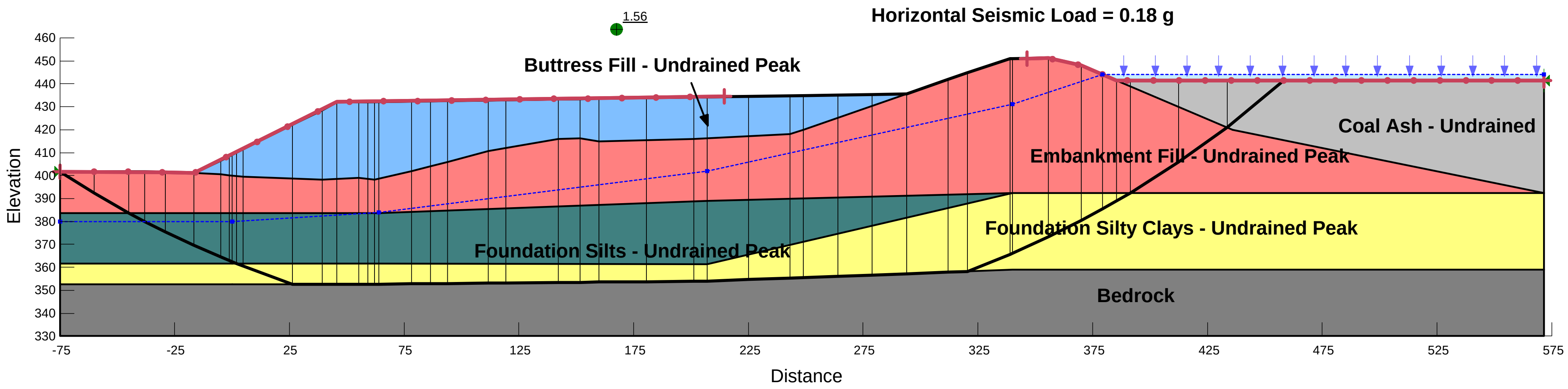
Foundation Silty Clays - Drained
Unit Weight: 126 pcf
Cohesion: 80 psf
Phi: 31 °

Foundation Silts - Drained
Unit Weight: 119 pcf
Cohesion: 0 psf
Phi: 33 °

Buttress Fill - Drained
Unit Weight: 123 pcf
Cohesion: 45 psf
Phi: 27 °

Ash Pond Lower Dam Buttress Evaluation
Vectren A.B. Brown Station

CCR Rule Safety Factor Assessment
Seismic - Critical Circular Surface Failure Geometry
Cross-Section B
Date: 10/7/2016



Material Properties

Embankment Fill - Undrained Peak
Unit Weight: 128 pcf
Cohesion: 600 psf
Phi: 22 °

Coal Ash - Undrained
Unit Weight: 100 pcf
Cohesion: 100 psf
Phi: 12 °

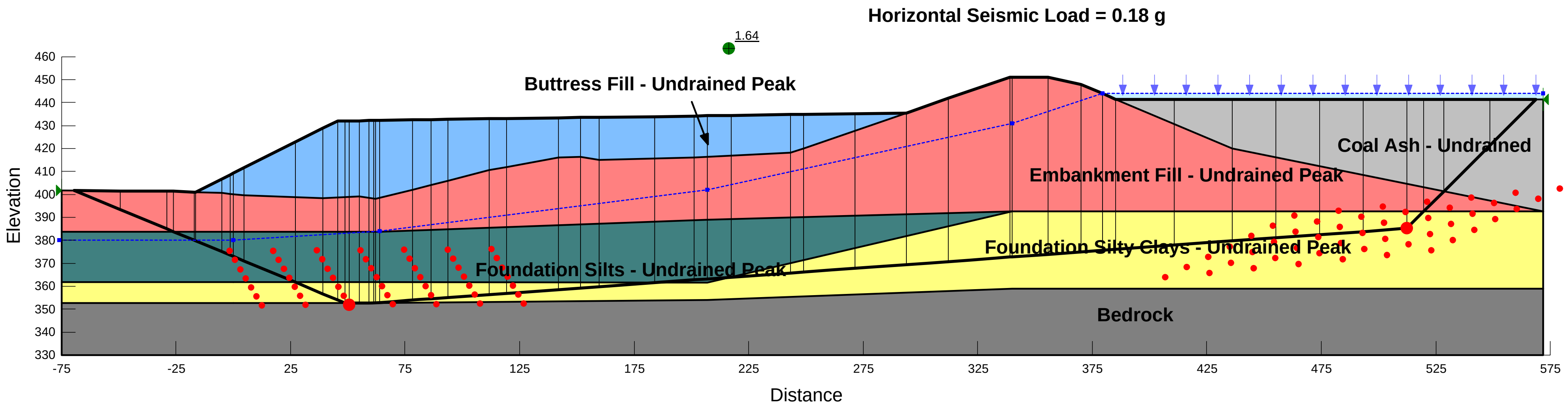
Foundation Silty Clays - Undrained Peak
Unit Weight: 126 pcf
Cohesion: 400 psf
Phi: 23 °

Foundation Silts - Undrained Peak
Unit Weight: 119 pcf
Cohesion: 650 psf
Phi: 22 °

Buttress Fill - Undrained Peak
Unit Weight: 123 pcf
Cohesion: 540 psf
Phi: 20 °

Ash Pond Lower Dam Buttress Evaluation
Vectren A.B. Brown Station

CCR Rule Safety Factor Assessment
Seismic - Critical Block Failure Surface Geometry
Cross-Section B
Date: 10/7/2016



Material Properties

Embankment Fill - Undrained Peak
Unit Weight: 128 pcf
Cohesion: 600 psf
Phi: 22 °

Coal Ash - Undrained
Unit Weight: 100 pcf
Cohesion: 100 psf
Phi: 12 °

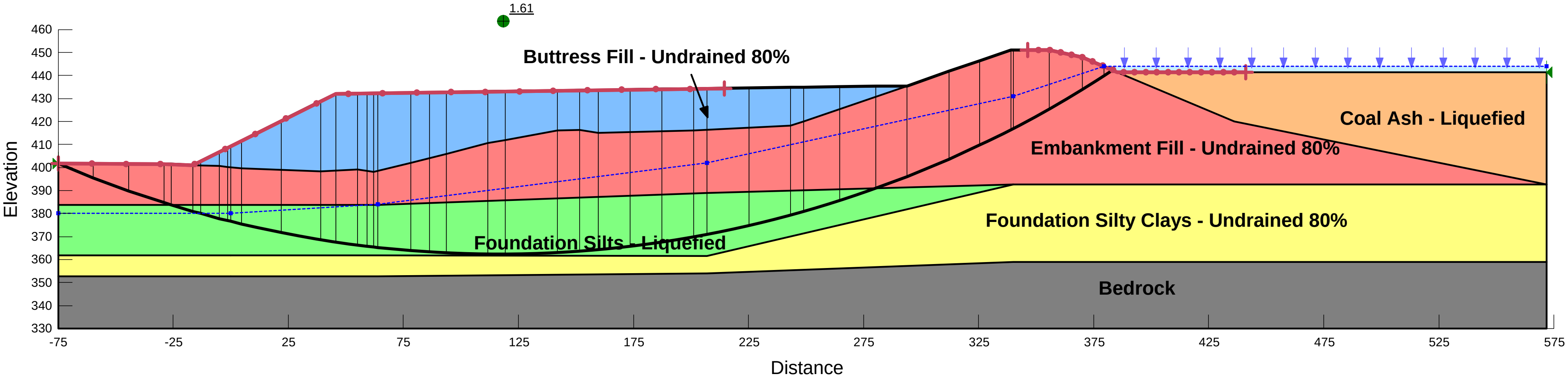
Foundation Silty Clays - Undrained Peak
Unit Weight: 126 pcf
Cohesion: 400 psf
Phi: 23 °

Foundation Silts - Undrained Peak
Unit Weight: 119 pcf
Cohesion: 650 psf
Phi: 22 °

Buttress Fill - Undrained Peak
Unit Weight: 123 pcf
Cohesion: 540 psf
Phi: 20 °

Ash Pond Lower Dam Buttress Evaluation
Vectren A.B. Brown Station

CCR Rule Safety Factor Assessment
Post-Liquefaction - Critical Circular Surface Failure Geometry
Cross-Section B
Date: 9/13/2016



Material Properties

Embankment Fill - Undrained 80%
Unit Weight: 128 pcf
Cohesion: 475 psf
Phi: 18 °

Coal Ash - Liquefied
Unit Weight: 100 pcf
Tau/Sigma Ratio: 0.12
Minimum Strength: 0

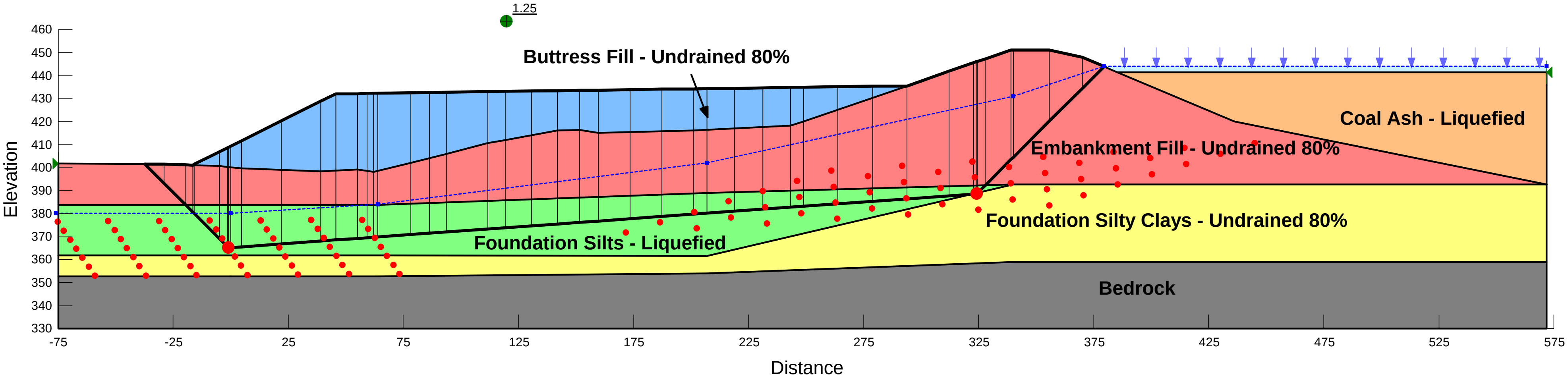
Foundation Silty Clays - Undrained 80%
Unit Weight: 126 pcf
Cohesion: 320 psf
Phi: 19 °

Foundation Silts - Liquefied
Unit Weight: 119 pcf
Tau/Sigma Ratio: 0.1
Minimum Strength: 100

Buttress Fill - Undrained 80%
Unit Weight: 123 pcf
Cohesion: 425 psf
Phi: 16 °

Ash Pond Lower Dam Buttress Evaluation
Vectren A.B. Brown Station

CCR Rule Safety Factor Assessment
Post-Liquefaction - Critical Block Failure Surface Geometry
Cross-Section B
Date: 9/13/2016



Material Properties

Embankment Fill - Undrained 80%
Unit Weight: 128 pcf
Cohesion: 475 psf
Phi: 18 °

Coal Ash - Liquefied
Unit Weight: 100 pcf
Tau/Sigma Ratio: 0.12
Minimum Strength: 0

Foundation Silty Clays - Undrained 80%
Unit Weight: 126 pcf
Cohesion: 320 psf
Phi: 19 °

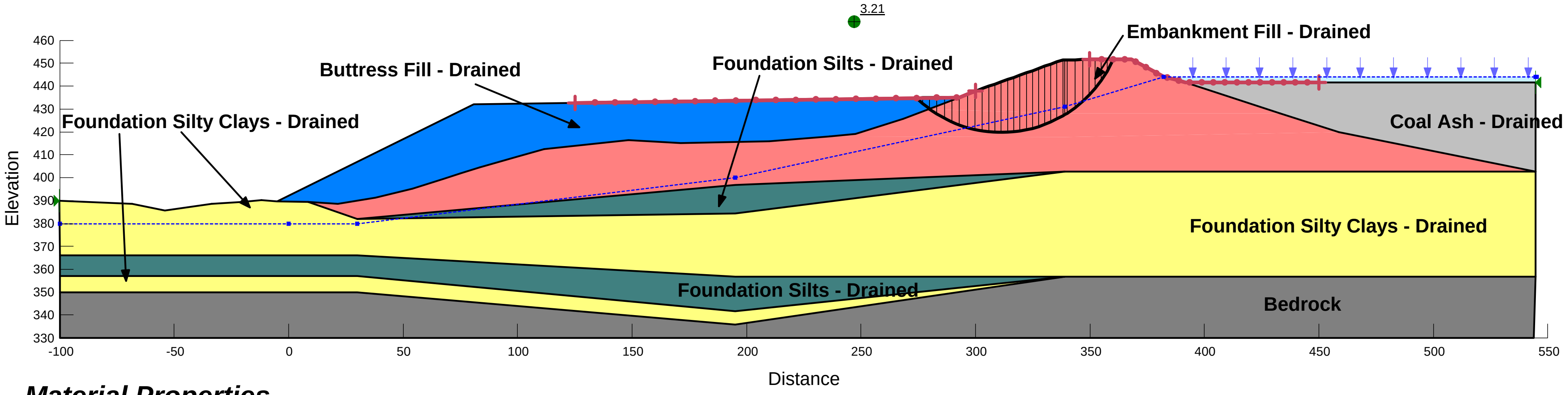
Foundation Silts - Liquefied
Unit Weight: 119 pcf
Tau/Sigma Ratio: 0.1
Minimum Strength: 100

Buttress Fill - Undrained 80%
Unit Weight: 123 pcf
Cohesion: 425 psf
Phi: 16 °

Ash Pond Lower Dam Buttress Evaluation
Vectren A.B. Brown Station

CCR Rule Safety Factor Assessment
Static Storage Pool - Critical Circular Failure Surface Geometry
Cross-Section C

Date: 10/10/2016



Material Properties

Embankment Fill - Drained
Unit Weight: 128 pcf
Cohesion: 50 psf
Phi: 30 °

Coal Ash - Drained
Unit Weight: 100 pcf
Cohesion: 0 psf
Phi: 32 °

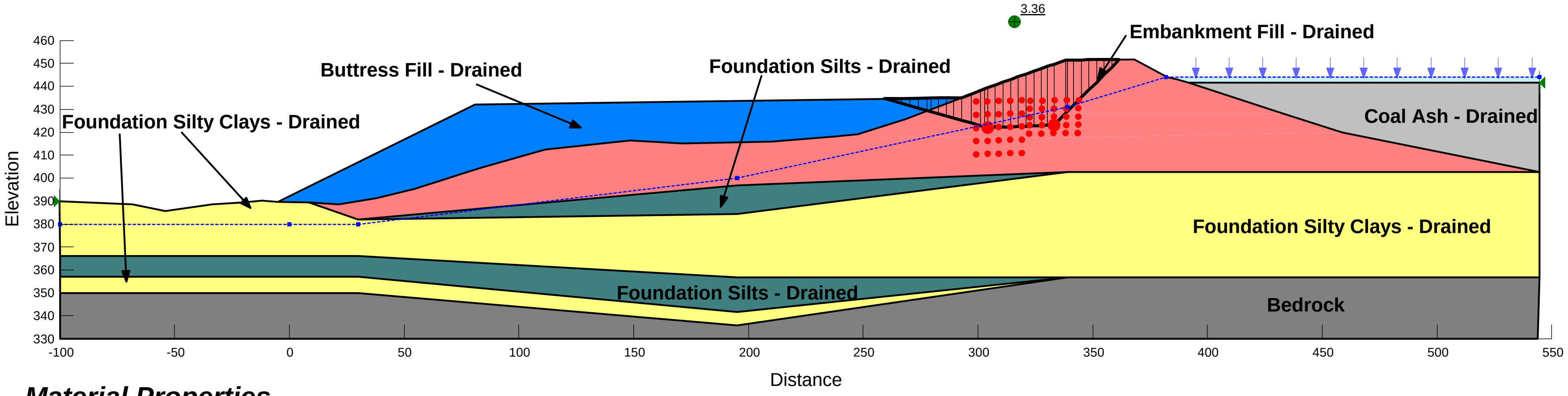
Buttress Fill - Drained
Unit Weight: 123 pcf
Cohesion: 45 psf
Phi: 27 °

Foundation Silts - Drained
Unit Weight: 119 pcf
Cohesion: 0 psf
Phi: 33 °

Foundation Silty Clays - Drained
Unit Weight: 126 pcf
Cohesion: 80 psf
Phi: 31 °

Ash Pond Lower Dam Buttress Evaluation
Vectren A.B. Brown Station

CCR Rule Safety Factor Assessment
Static Storage Pool - Critical Block Failure Surface Geometry
Cross-Section C
Date: 10/10/2016



Material Properties

Embankment Fill - Drained
Unit Weight: 128 pcf
Cohesion: 50 psf
Phi: 30 °

Coal Ash - Drained
Unit Weight: 100 pcf
Cohesion: 0 psf
Phi: 32 °

Buttress Fill - Drained
Unit Weight: 123 pcf
Cohesion: 45 psf
Phi: 27 °

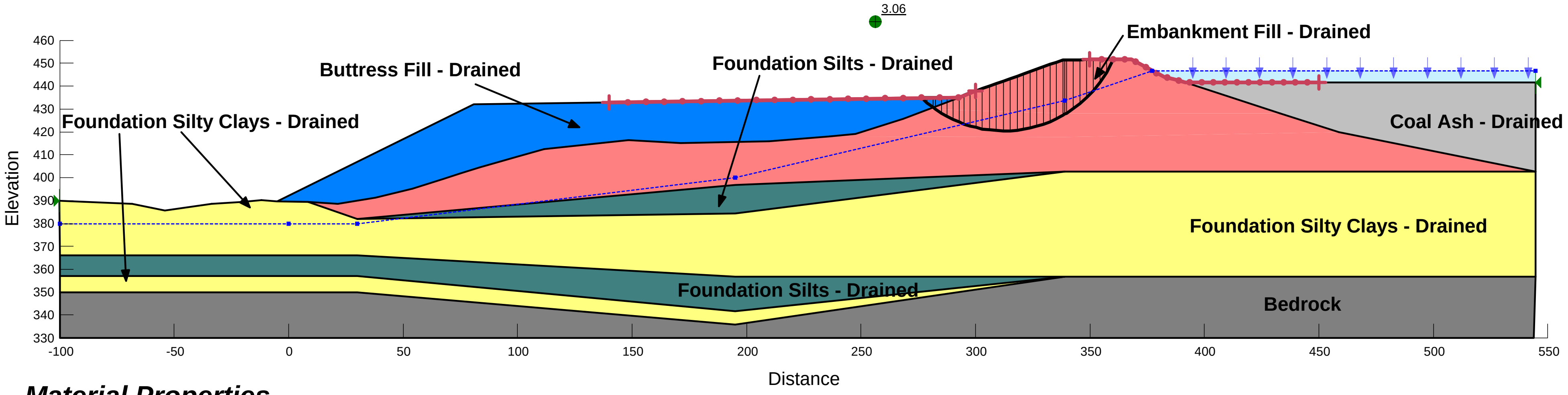
Foundation Silts - Drained
Unit Weight: 119 pcf
Cohesion: 0 psf
Phi: 33 °

Foundation Silty Clays - Drained
Unit Weight: 126 pcf
Cohesion: 80 psf
Phi: 31 °

Ash Pond Lower Dam Buttress Evaluation
Vectren A.B. Brown Station

CCR Rule Safety Factor Assessment
Static Surcharge Pool - Critical Circular Failure Surface Geometry
Cross-Section C

Date: 10/10/2016



Material Properties

Embankment Fill - Drained
Unit Weight: 128 pcf
Cohesion: 50 psf
Phi: 30 °

Coal Ash - Drained
Unit Weight: 100 pcf
Cohesion: 0 psf
Phi: 32 °

Buttress Fill - Drained
Unit Weight: 123 pcf
Cohesion: 45 psf
Phi: 27 °

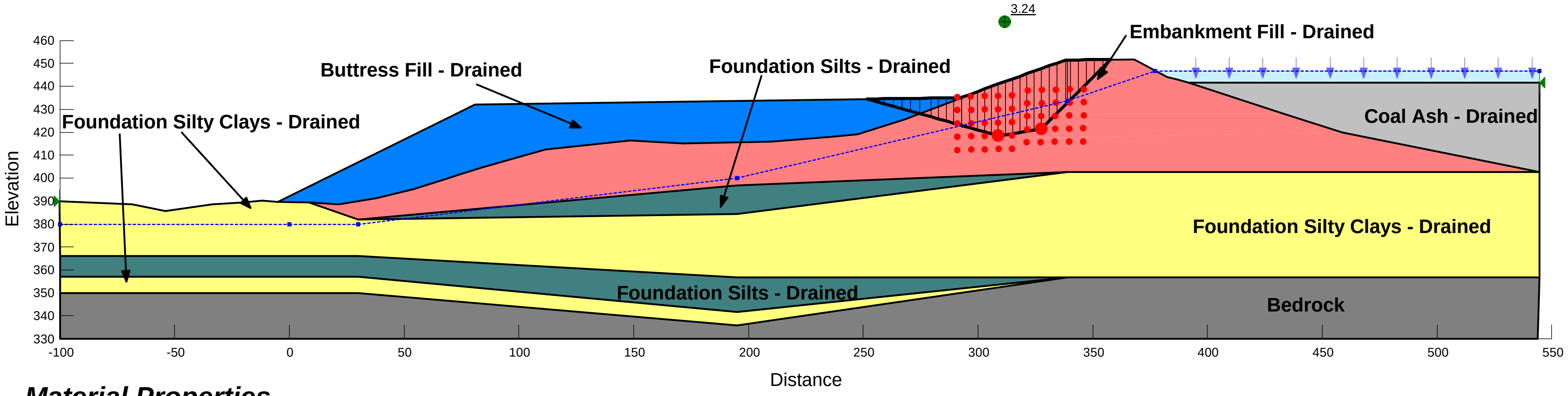
Foundation Silts - Drained
Unit Weight: 119 pcf
Cohesion: 0 psf
Phi: 33 °

Foundation Silty Clays - Drained
Unit Weight: 126 pcf
Cohesion: 80 psf
Phi: 31 °

Ash Pond Lower Dam Buttress Evaluation
Vectren A.B. Brown Station

CCR Rule Safety Factor Assessment
Static Surcharge Pool - Critical Block Failure Surface Geometry
Cross-Section C

Date: 10/10/2016



Material Properties

Embankment Fill - Drained
Unit Weight: 128 pcf
Cohesion: 50 psf
Phi: 30 °

Coal Ash - Drained
Unit Weight: 100 pcf
Cohesion: 0 psf
Phi: 32 °

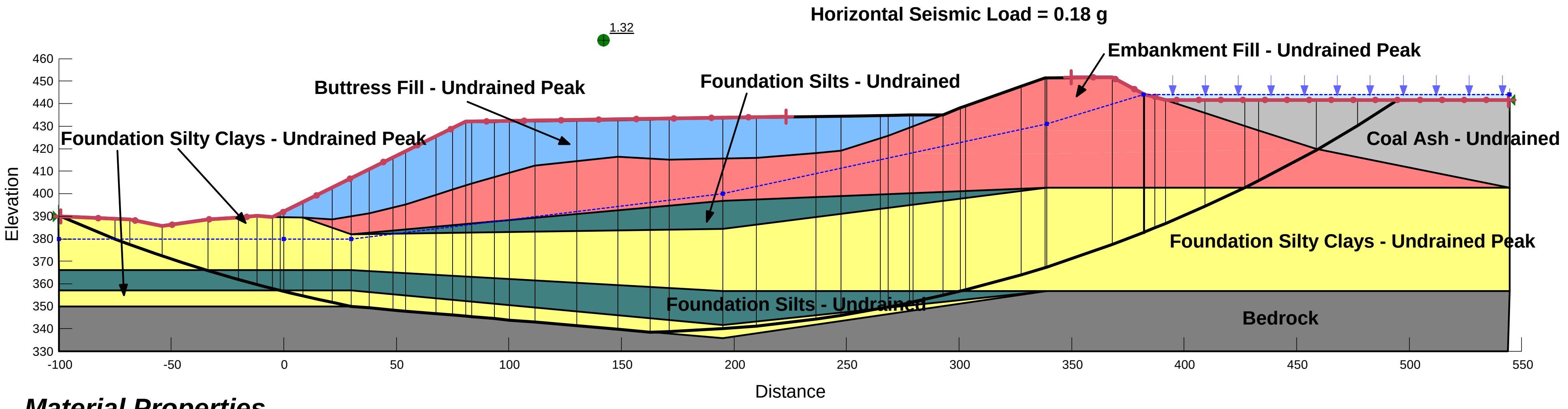
Buttress Fill - Drained
Unit Weight: 123 pcf
Cohesion: 45 psf
Phi: 27 °

Foundation Silts - Drained
Unit Weight: 119 pcf
Cohesion: 0 psf
Phi: 33 °

Foundation Silty Clays - Drained
Unit Weight: 126 pcf
Cohesion: 80 psf
Phi: 31 °

Ash Pond Lower Dam Buttress Evaluation
Vectren A.B. Brown Station

CCR Rule Safety Factor Assessment
Seismic - Critical Circular Failure Surface Geometry
Cross-Section C
Date: 10/7/2016



Material Properties

Embankment Fill - Undrained Peak
Unit Weight: 128 pcf
Cohesion: 600 psf
Phi: 22 °

Coal Ash - Undrained
Unit Weight: 100 pcf
Cohesion: 100 psf
Phi: 12 °

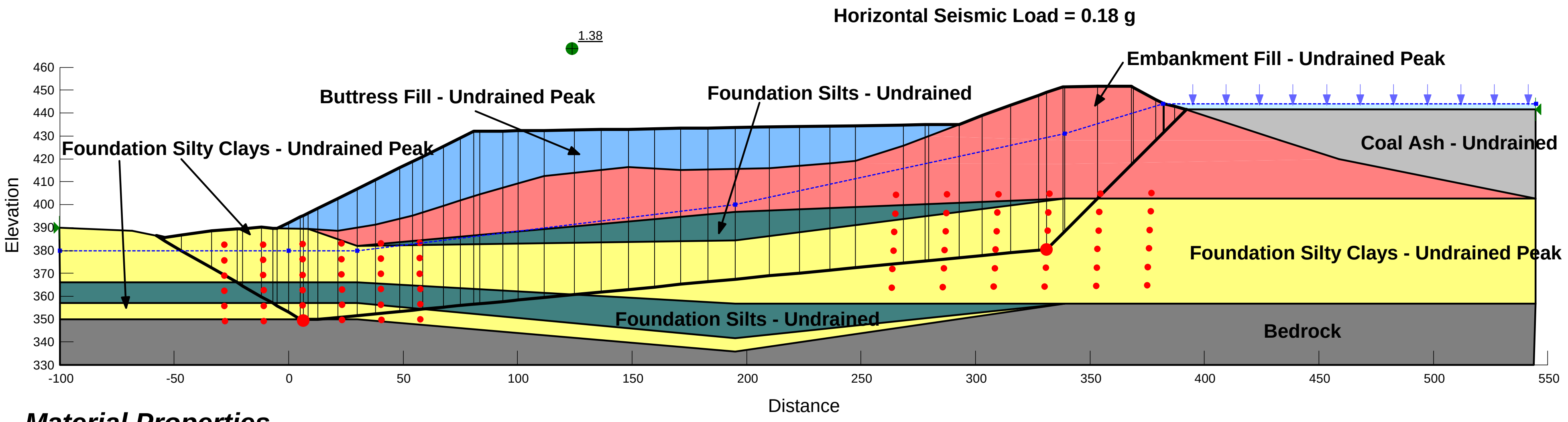
Buttress Fill - Undrained Peak
Unit Weight: 123 pcf
Cohesion: 540 psf
Phi: 20 °

Foundation Silts - Undrained
Unit Weight: 119 pcf
Cohesion: 650 psf
Phi: 22 °

Foundation Silty Clays - Undrained Peak
Unit Weight: 126 pcf
Cohesion: 400 psf
Phi: 23 °

Ash Pond Lower Dam Buttress Evaluation
Vectren A.B. Brown Station

CCR Rule Safety Factor Assessment
Seismic - Critical Block Failure Surface Geometry
Cross-Section C
Date: 10/7/2016



Material Properties

Embankment Fill - Undrained Peak
Unit Weight: 128 pcf
Cohesion: 600 psf
Phi: 22 °

Coal Ash - Undrained
Unit Weight: 100 pcf
Cohesion: 100 psf
Phi: 12 °

Buttress Fill - Undrained Peak
Unit Weight: 123 pcf
Cohesion: 540 psf
Phi: 20 °

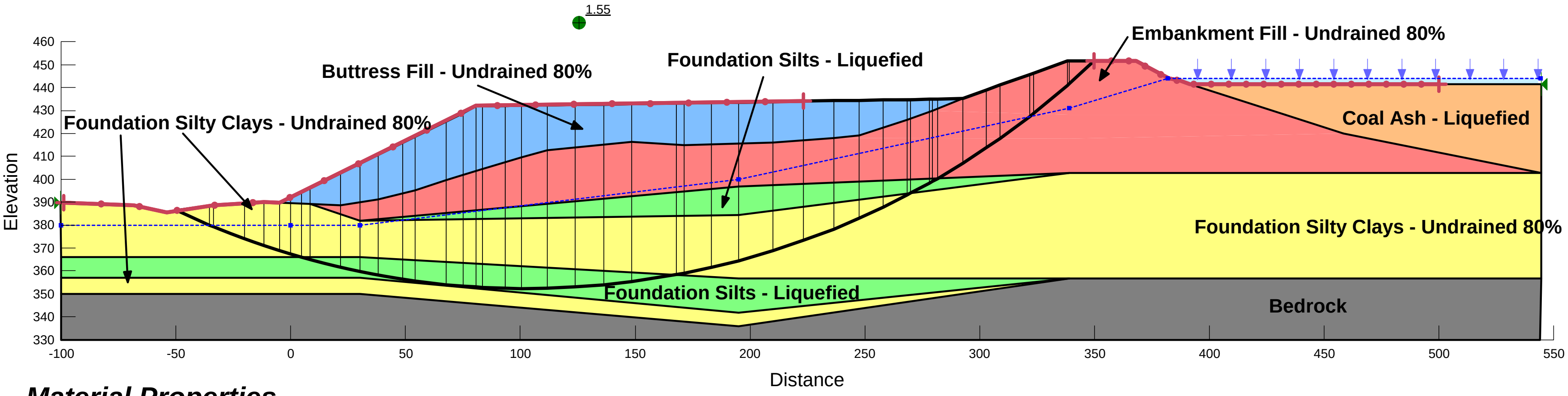
Foundation Silts - Undrained
Unit Weight: 119 pcf
Cohesion: 650 psf
Phi: 22 °

Foundation Silty Clays - Undrained Peak
Unit Weight: 126 pcf
Cohesion: 400 psf
Phi: 23 °

Ash Pond Lower Dam Buttress Evaluation
Vectren A.B. Brown Station

CCR Rule Safety Factor Assessment
Post-Liquefaction - Critical Circular Failure Surface Geometry
Cross-Section C

Date: 9/13/2016



Material Properties

Embankment Fill - Undrained 80%
Unit Weight: 128 pcf
Cohesion: 475 psf
Phi: 18 °

Coal Ash - Liquefied
Unit Weight: 100 pcf
Tau/Sigma Ratio: 0.12
Minimum Strength: 0

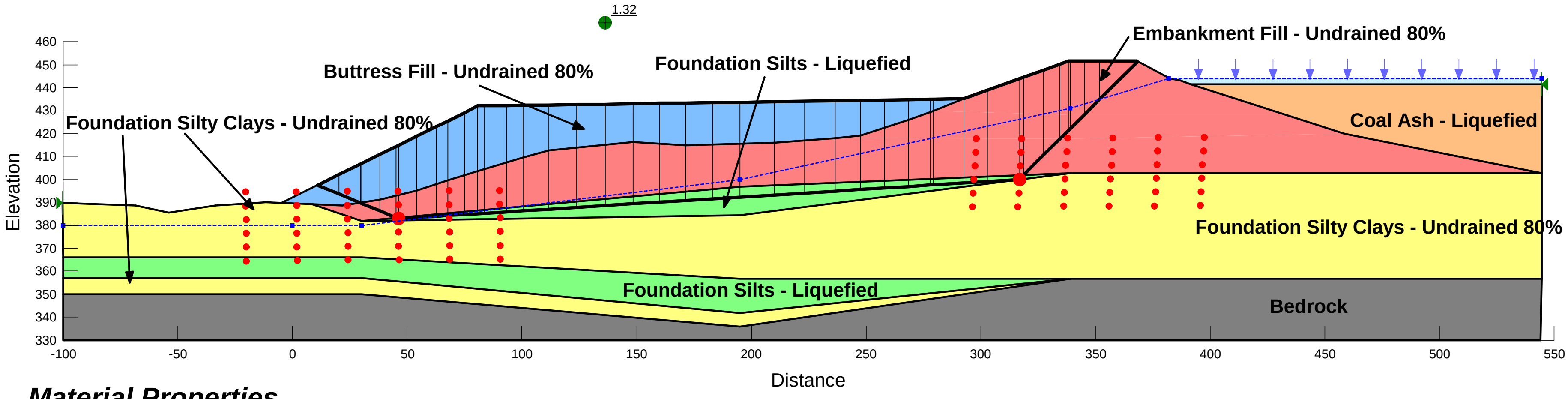
Buttress Fill - Undrained 80%
Unit Weight: 123 pcf
Cohesion: 425 psf
Phi: 16 °

Foundation Silts - Liquefied
Unit Weight: 119 pcf
Tau/Sigma Ratio: 0.1
Minimum Strength: 100

Foundation Silty Clays - Undrained 80%
Unit Weight: 126 pcf
Cohesion: 320 psf
Phi: 19 °

Ash Pond Lower Dam Buttress Evaluation
Vectren A.B. Brown Station

CCR Rule Safety Factor Assessment
Post-Liquefaction - Critical Block Failure Surface Geometry
Cross-Section C
Date: 9/13/2016



Material Properties

Embankment Fill - Undrained 80%
Unit Weight: 128 pcf
Cohesion: 475 psf
Phi: 18 °

Coal Ash - Liquefied
Unit Weight: 100 pcf
Tau/Sigma Ratio: 0.12
Minimum Strength: 0

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Unit Weight: 123 pcf
Cohesion: 425 psf
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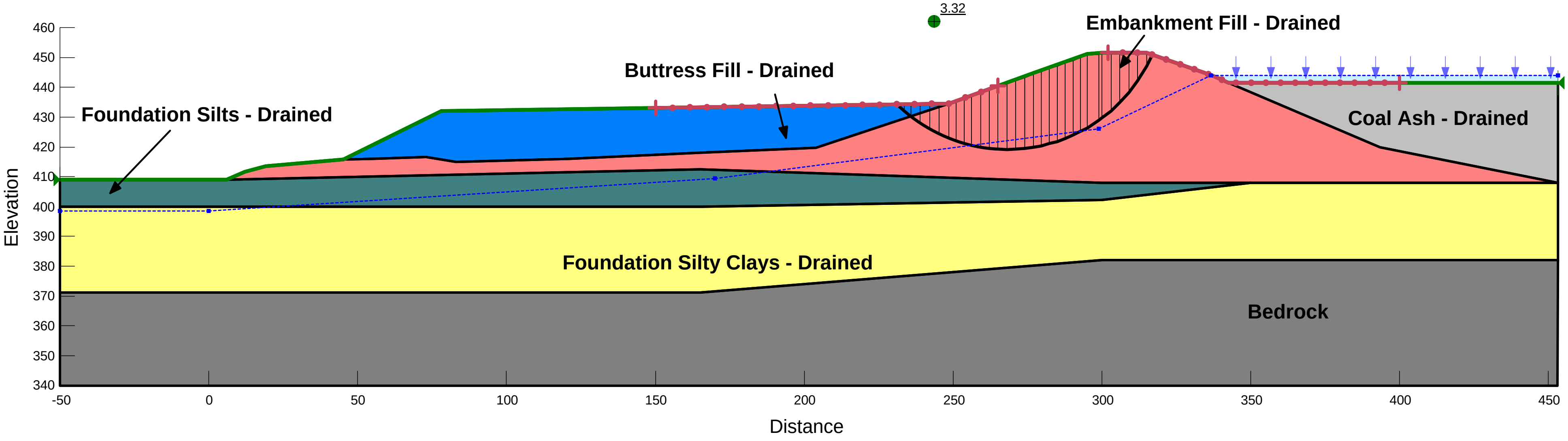
Foundation Silts - Liquefied
Unit Weight: 119 pcf
Tau/Sigma Ratio: 0.1
Minimum Strength: 100

Foundation Silty Clays - Undrained 80%
Unit Weight: 126 pcf
Cohesion: 320 psf
Phi: 19 °

Ash Pond Lower Dam Buttress Evaluation
Vectren A.B. Brown Station

CCR Rule Safety Factor Assessment
Static Storage Pool - Critical Circular Failure Surface Geometry
Cross-Section D
Date: 10/10/2016



Material Properties

Embankment Fill - Drained
Unit Weight: 128 pcf
Cohesion: 50 psf
Phi: 30 °

Foundation Silts - Drained
Unit Weight: 119 pcf
Cohesion: 0 psf
Phi: 33 °

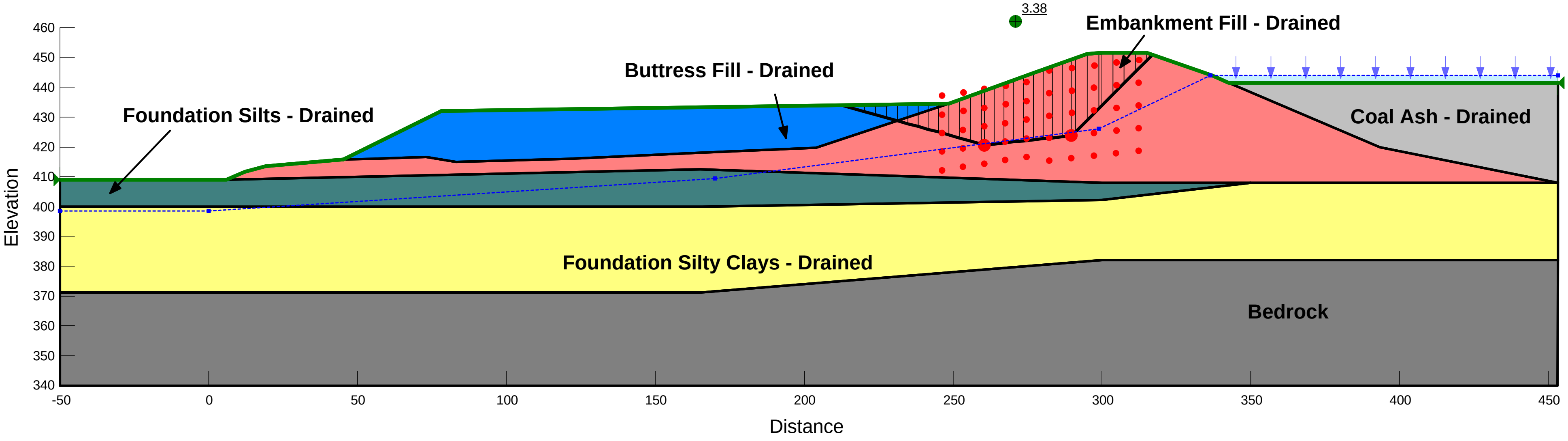
Foundation Silty Clays - Drained
Unit Weight: 126 pcf
Cohesion: 80 psf
Phi: 31 °

Coal Ash - Drained
Unit Weight: 100 pcf
Cohesion: 0 psf
Phi: 32 °

Buttress Fill - Drained
Unit Weight: 123 pcf
Cohesion: 45 psf
Phi: 27 °

Ash Pond Lower Dam Buttress Evaluation
Vectren A.B. Brown Station

CCR Rule Safety Factor Assessment
Static Storage Pool - Critical Block Failure Surface Geometry
Cross-Section D
Date: 10/10/2016



Material Properties

Embankment Fill - Drained
Unit Weight: 128 pcf
Cohesion: 50 psf
Phi: 30 °

Foundation Silts - Drained
Unit Weight: 119 pcf
Cohesion: 0 psf
Phi: 33 °

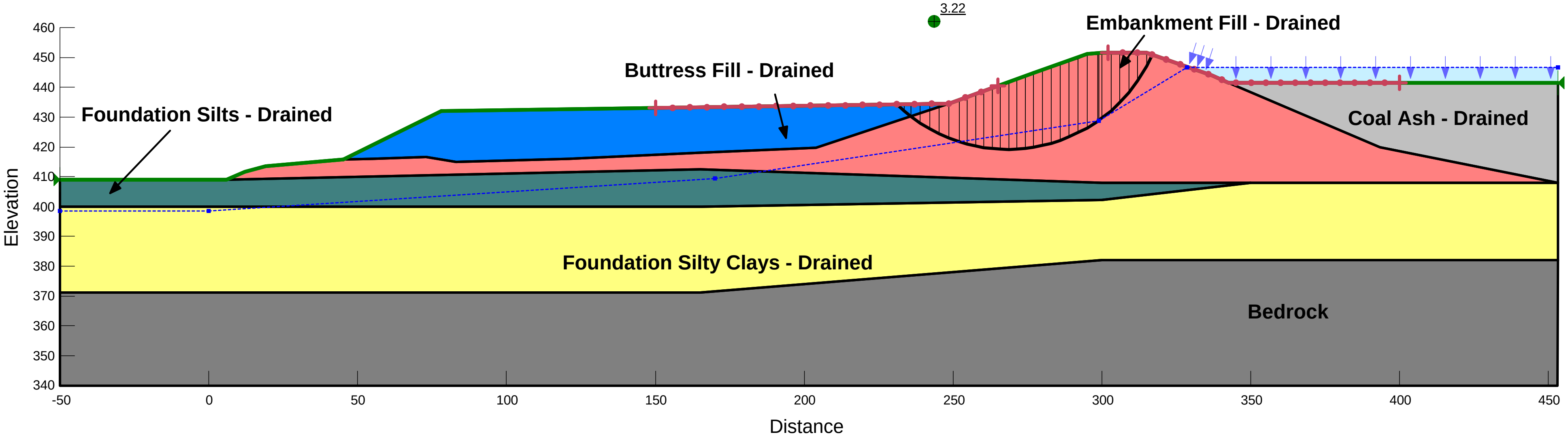
Foundation Silty Clays - Drained
Unit Weight: 126 pcf
Cohesion: 80 psf
Phi: 31 °

Coal Ash - Drained
Unit Weight: 100 pcf
Cohesion: 0 psf
Phi: 32 °

Buttress Fill - Drained
Unit Weight: 123 pcf
Cohesion: 45 psf
Phi: 27 °

Ash Pond Lower Dam Buttress Evaluation
Vectren A.B. Brown Station

CCR Rule Safety Factor Assessment
Static Surcharge Pool - Critical Circular Failure Surface Geometry
Cross-Section D
Date: 10/10/2016



Material Properties

Embankment Fill - Drained
Unit Weight: 128 pcf
Cohesion: 50 psf
Phi: 30 °

Foundation Silts - Drained
Unit Weight: 119 pcf
Cohesion: 0 psf
Phi: 33 °

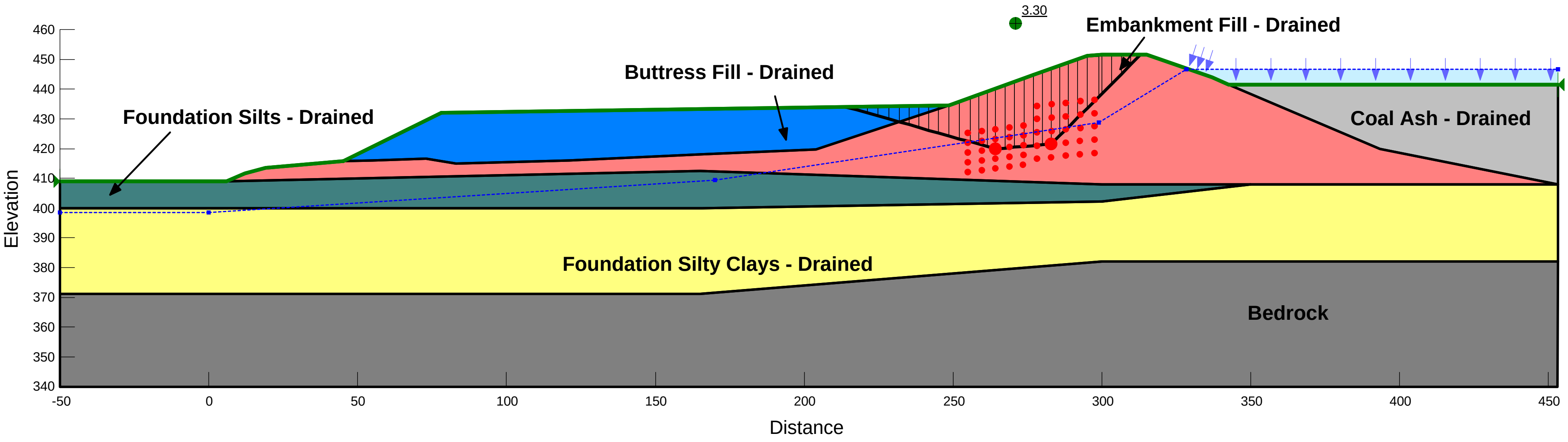
Foundation Silty Clays - Drained
Unit Weight: 126 pcf
Cohesion: 80 psf
Phi: 31 °

Coal Ash - Drained
Unit Weight: 100 pcf
Cohesion: 0 psf
Phi: 32 °

Buttress Fill - Drained
Unit Weight: 123 pcf
Cohesion: 45 psf
Phi: 27 °

Ash Pond Lower Dam Buttress Evaluation
Vectren A.B. Brown Station

CCR Rule Safety Factor Assessment
Static Surcharge Pool - Critical Block Failure Surface Geometry
Cross-Section D
Date: 10/10/2016



Material Properties

Embankment Fill - Drained
Unit Weight: 128 pcf
Cohesion: 50 psf
Phi: 30 °

Foundation Silts - Drained
Unit Weight: 119 pcf
Cohesion: 0 psf
Phi: 33 °

Foundation Silty Clays - Drained
Unit Weight: 126 pcf
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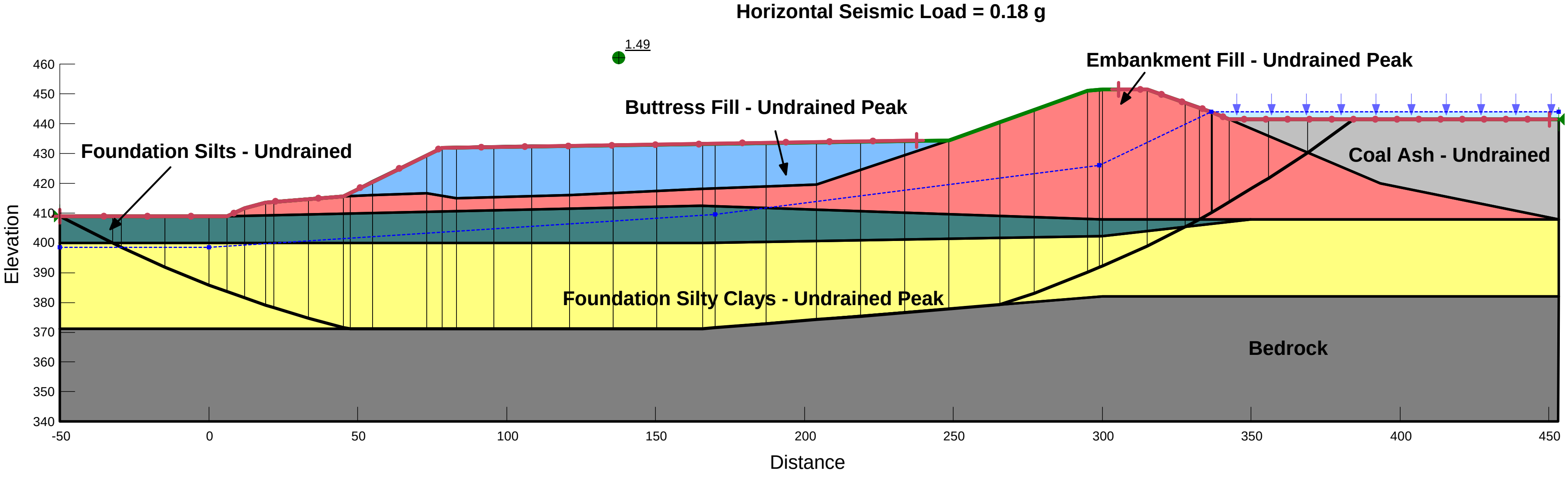
Coal Ash - Drained
Unit Weight: 100 pcf
Cohesion: 0 psf
Phi: 32 °

Buttress Fill - Drained
Unit Weight: 123 pcf
Cohesion: 45 psf
Phi: 27 °

Ash Pond Lower Dam Buttress Evaluation
Vectren A.B. Brown Station

CCR Rule Safety Factor Assessment
Seismic - Critical Circular Failure Surface Geometry
Cross-Section D
Date: 10/7/2016



Material Properties

Embankment Fill - Undrained Peak
Unit Weight: 128 pcf
Cohesion: 600 psf
Phi: 22 °

Foundation Silts - Undrained
Unit Weight: 119 pcf
Cohesion: 650 psf
Phi: 22 °

Foundation Silty Clays - Undrained Peak
Unit Weight: 126 pcf
Cohesion: 400 psf
Phi: 23 °

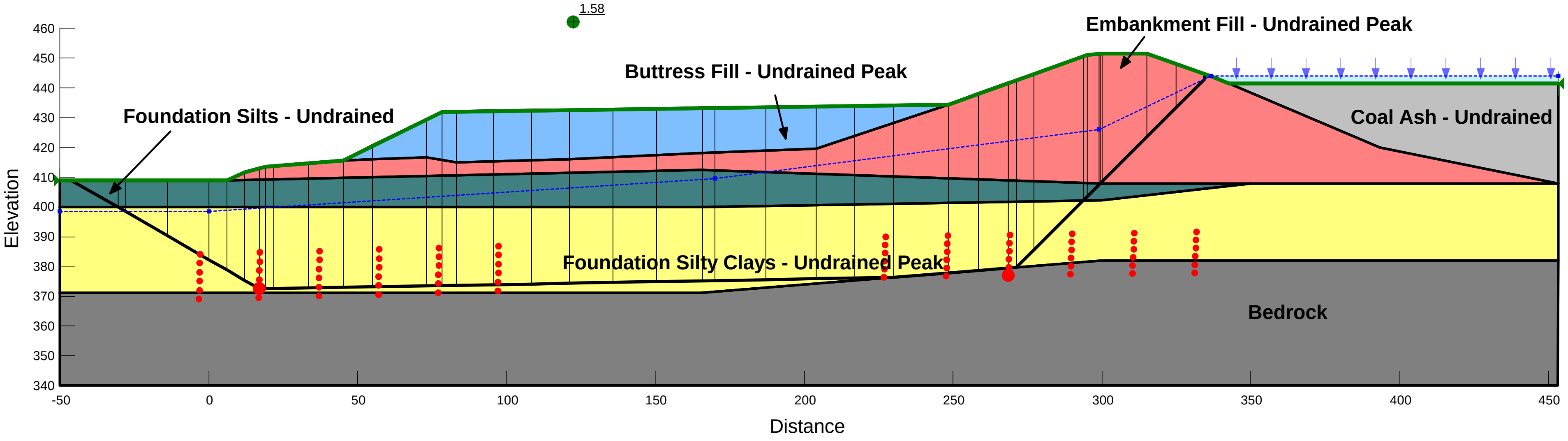
Coal Ash - Undrained
Unit Weight: 100 pcf
Cohesion: 100 psf
Phi: 12 °

Buttress Fill - Undrained Peak
Unit Weight: 123 pcf
Cohesion: 540 psf
Phi: 20 °

Ash Pond Lower Dam Buttress Evaluation
Vectren A.B. Brown Station

CCR Rule Safety Factor Assessment
Seismic - Critical Block Failure Surface Geometry
Cross-Section D
Date: 10/7/2016

Horizontal Seismic Load = 0.18 g



Material Properties

Embankment Fill - Undrained Peak
Unit Weight: 128 pcf
Cohesion: 600 psf
Phi: 22 °

Foundation Silts - Undrained
Unit Weight: 119 pcf
Cohesion: 650 psf
Phi: 22 °

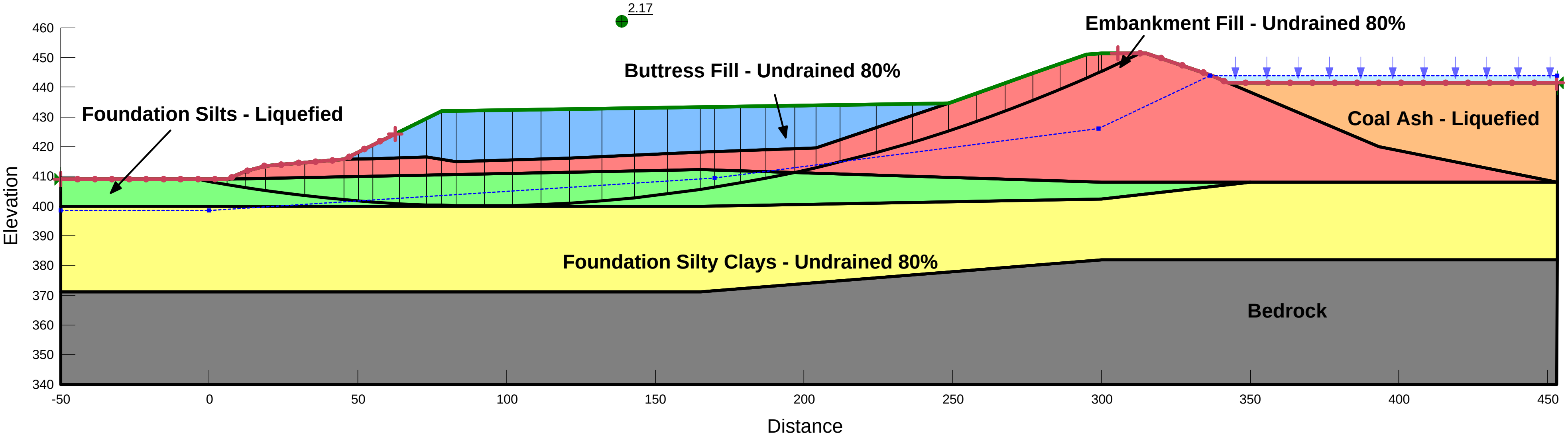
Foundation Silty Clays - Undrained Peak
Unit Weight: 126 pcf
Cohesion: 400 psf
Phi: 23 °

Coal Ash - Undrained
Unit Weight: 100 pcf
Cohesion: 100 psf
Phi: 12 °

Buttress Fill - Undrained Peak
Unit Weight: 123 pcf
Cohesion: 540 psf
Phi: 20 °

Ash Pond Lower Dam Buttress Evaluation
Vectren A.B. Brown Station

CCR Rule Safety Factor Assessment
Post-Liquefaction - Critical Circular Failure Surface Geometry
Cross-Section D
Date: 9/13/2016



Material Properties

Embankment Fill - Undrained 80%
Unit Weight: 128 pcf
Cohesion: 475 psf
Phi: 18 °

Coal Ash - Liquefied
Unit Weight: 100 pcf
Tau/Sigma Ratio: 0.12
Minimum Strength: 0

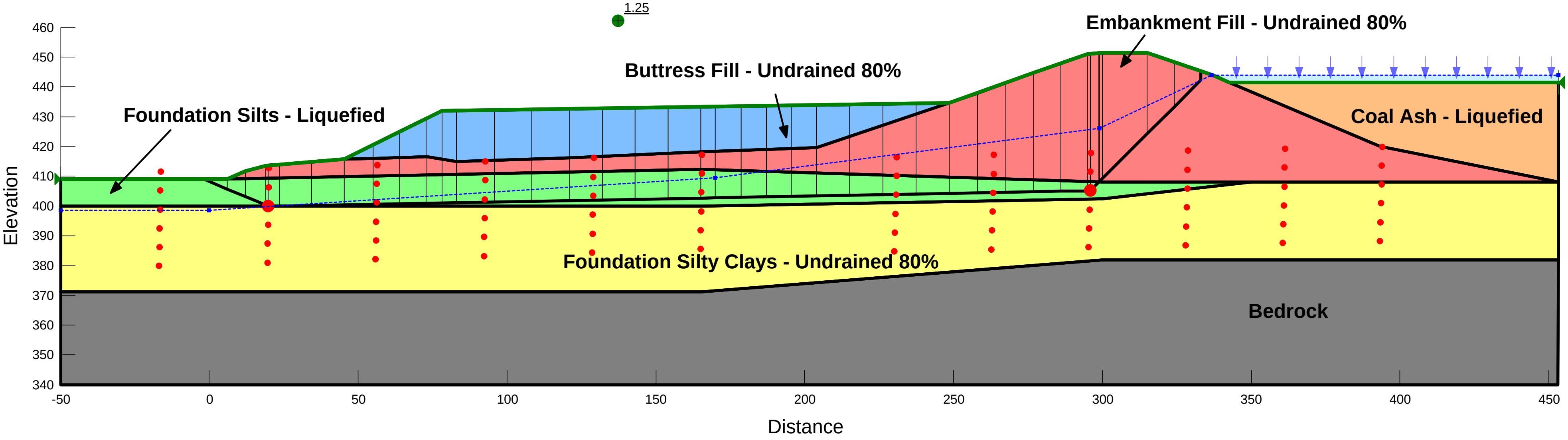
Foundation Silty Clays - Undrained 80%
Unit Weight: 126 pcf
Cohesion: 320 psf
Phi: 19 °

Foundation Silts - Liquefied
Unit Weight: 119 pcf
Tau/Sigma Ratio: 0.1
Minimum Strength: 100

Buttress Fill - Undrained 80%
Unit Weight: 123 pcf
Cohesion: 425 psf
Phi: 16 °

Ash Pond Lower Dam Buttress Evaluation
Vectren A.B. Brown Station

CCR Rule Safety Factor Assessment
Post-Liquefaction - Critical Block Failure Surface Geometry
Cross-Section D
Date: 9/13/2016



Material Properties

Embankment Fill - Undrained 80%
Unit Weight: 128 pcf
Cohesion: 475 psf
Phi: 18 °

Coal Ash - Liquefied
Unit Weight: 100 pcf
Tau/Sigma Ratio: 0.12
Minimum Strength: 0

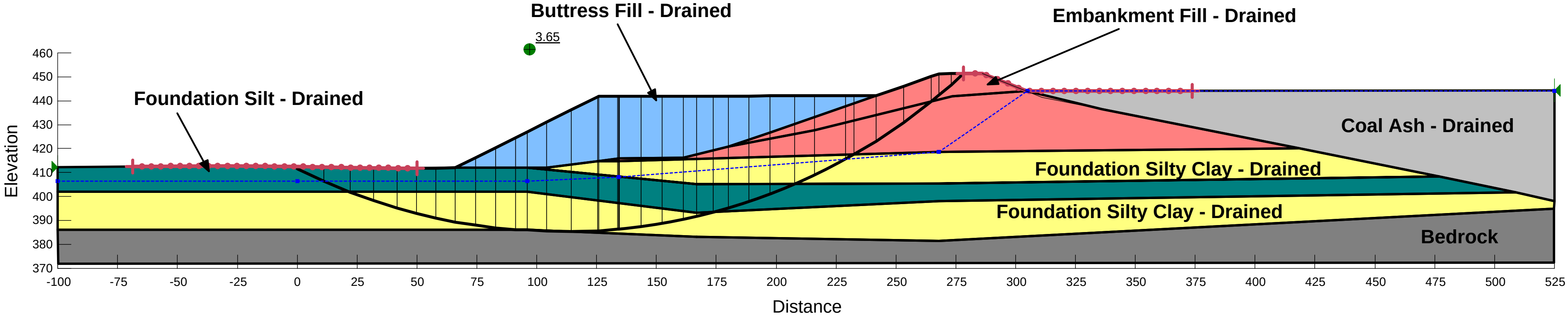
Foundation Silty Clays - Undrained 80%
Unit Weight: 126 pcf
Cohesion: 320 psf
Phi: 19 °

Foundation Silts - Liquefied
Unit Weight: 119 pcf
Tau/Sigma Ratio: 0.1
Minimum Strength: 100

Buttress Fill - Undrained 80%
Unit Weight: 123 pcf
Cohesion: 425 psf
Phi: 16 °

Ash Pond Lower Dan Buttress Evaluation
Vectren A.B. Brown Station

CCR Rule Safety Factor Assessment
Static Storage Pool - Critical Circular Failure Surface Geometry
Cross-Section E
Date: 10/10/2016



Material Properties

Embankment Fill - Drained
Unit Weight: 128 pcf
Cohesion: 50 psf
Phi: 30 °

Coal Ash - Drained
Unit Weight: 100 pcf
Cohesion: 0 psf
Phi: 32 °

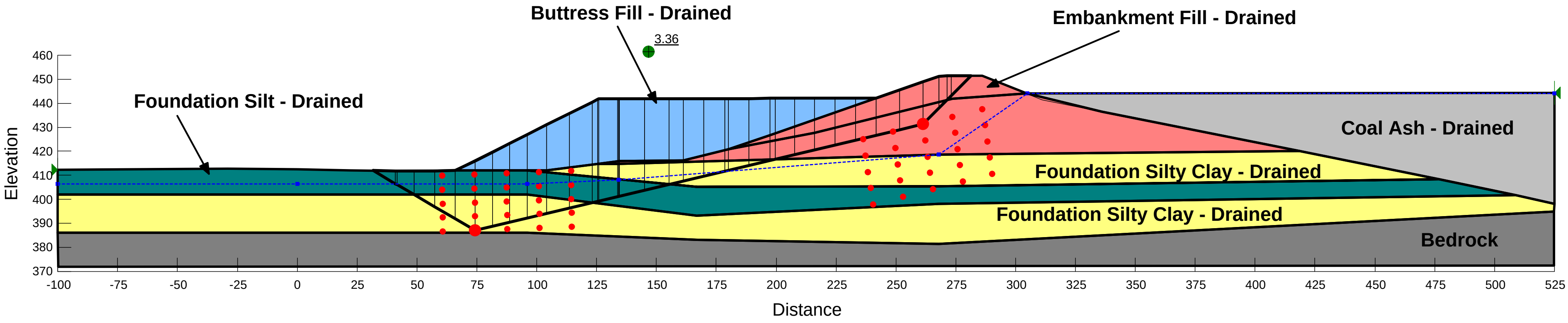
Foundation Silty Clay - Drained
Unit Weight: 126 pcf
Cohesion: 80 psf
Phi: 31 °

Foundation Silt - Drained
Unit Weight: 119 pcf
Cohesion: 0 psf
Phi: 33 °

Buttress Fill - Drained
Unit Weight: 123 pcf
Cohesion: 45 psf
Phi: 27 °

Ash Pond Lower Dan Buttress Evaluation
Vectren A.B. Brown Station

CCR Rule Safety Factor Assessment
Static Storage Pool - Critical Block Failure Surface Geometry
Cross-Section E
Date: 10/10/2016



Material Properties

Embankment Fill - Drained
Unit Weight: 128 pcf
Cohesion: 50 psf
Phi: 30 °

Coal Ash - Drained
Unit Weight: 100 pcf
Cohesion: 0 psf
Phi: 32 °

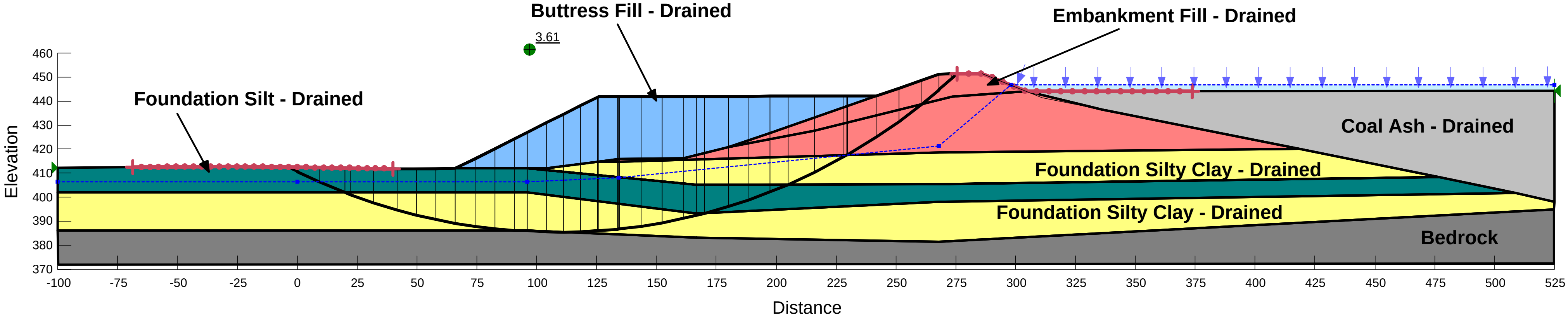
Foundation Silty Clay - Drained
Unit Weight: 126 pcf
Cohesion: 80 psf
Phi: 31 °

Foundation Silt - Drained
Unit Weight: 119 pcf
Cohesion: 0 psf
Phi: 33 °

Buttress Fill - Drained
Unit Weight: 123 pcf
Cohesion: 45 psf
Phi: 27 °

Ash Pond Lower Dan Buttress Evaluation
Vectren A.B. Brown Station

CCR Rule Safety Factor Assessment
Static Surcharge Pool - Critical Circular Failure Surface Geometry
Cross-Section E
Date: 10/10/2016



Material Properties

Embankment Fill - Drained
Unit Weight: 128 pcf
Cohesion: 50 psf
Phi: 30 °

Coal Ash - Drained
Unit Weight: 100 pcf
Cohesion: 0 psf
Phi: 32 °

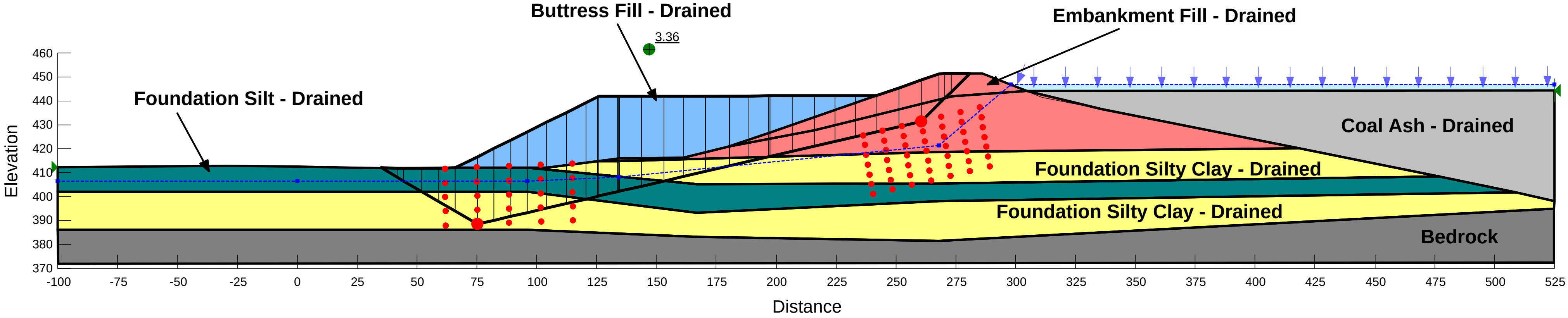
Foundation Silty Clay - Drained
Unit Weight: 126 pcf
Cohesion: 80 psf
Phi: 31 °

Foundation Silt - Drained
Unit Weight: 119 pcf
Cohesion: 0 psf
Phi: 33 °

Buttress Fill - Drained
Unit Weight: 123 pcf
Cohesion: 45 psf
Phi: 27 °

Ash Pond Lower Dan Buttress Evaluation
Vectren A.B. Brown Station

CCR Rule Safety Factor Assessment
Static Surcharge Pool - Critical Block Failure Surface Geometry
Cross-Section E
Date: 10/10/2016



Material Properties

Embankment Fill - Drained
Unit Weight: 128 pcf
Cohesion: 50 psf
Phi: 30 °

Coal Ash - Drained
Unit Weight: 100 pcf
Cohesion: 0 psf
Phi: 32 °

Foundation Silty Clay - Drained
Unit Weight: 126 pcf
Cohesion: 80 psf
Phi: 31 °

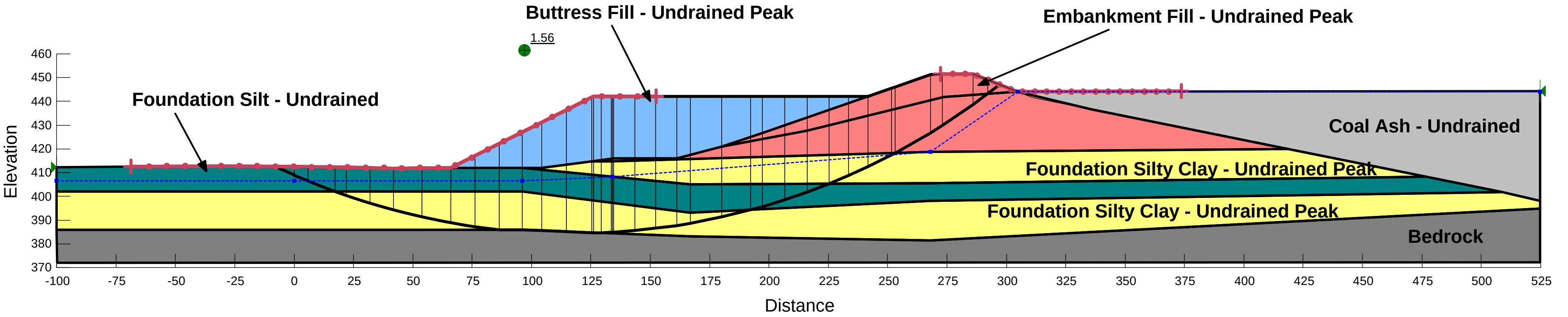
Foundation Silt - Drained
Unit Weight: 119 pcf
Cohesion: 0 psf
Phi: 33 °

Buttress Fill - Drained
Unit Weight: 123 pcf
Cohesion: 45 psf
Phi: 27 °

Ash Pond Lower Dan Buttress Evaluation
Vectren A.B. Brown Station

CCR Rule Safety Factor Assessment
Seismic - Critical Circular Failure Surface Geometry
Cross-Section E
Date: 10/7/2016

Horizontal Seismic Load: 0.18 g



Material Properties

Embankment Fill - Undrained Peak
Unit Weight: 128 pcf
Cohesion: 600 psf
Phi: 22 °

Coal Ash - Undrained
Unit Weight: 100 pcf
Cohesion: 100 psf
Phi: 12 °

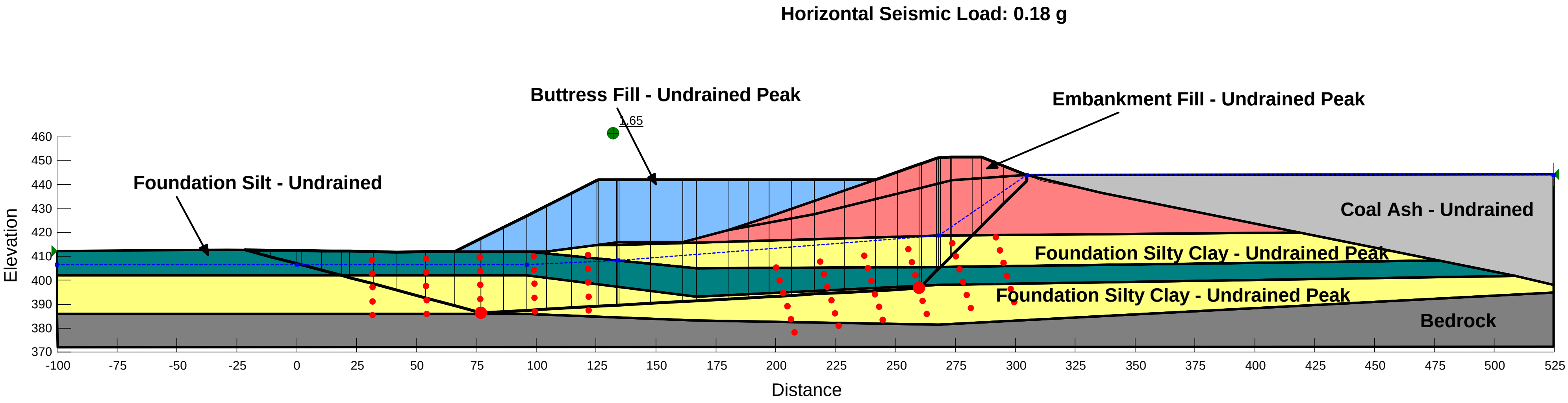
Foundation Silty Clay - Undrained Peak
Unit Weight: 126 pcf
Cohesion: 400 psf
Phi: 23 °

Foundation Silt - Undrained
Unit Weight: 119 pcf
Cohesion: 650 psf
Phi: 22 °

Buttress Fill - Undrained Peak
Unit Weight: 123 pcf
Cohesion: 540 psf
Phi: 20 °

Ash Pond Lower Dan Buttress Evaluation
Vectren A.B. Brown Station

CCR Rule Safety Factor Assessment
Seismic - Critical Block Failure Surface Geometry
Cross-Section E
Date: 10/7/2016



Material Properties

Embankment Fill - Undrained Peak
Unit Weight: 128 pcf
Cohesion: 600 psf
Phi: 22 °

Coal Ash - Undrained
Unit Weight: 100 pcf
Cohesion: 100 psf
Phi: 12 °

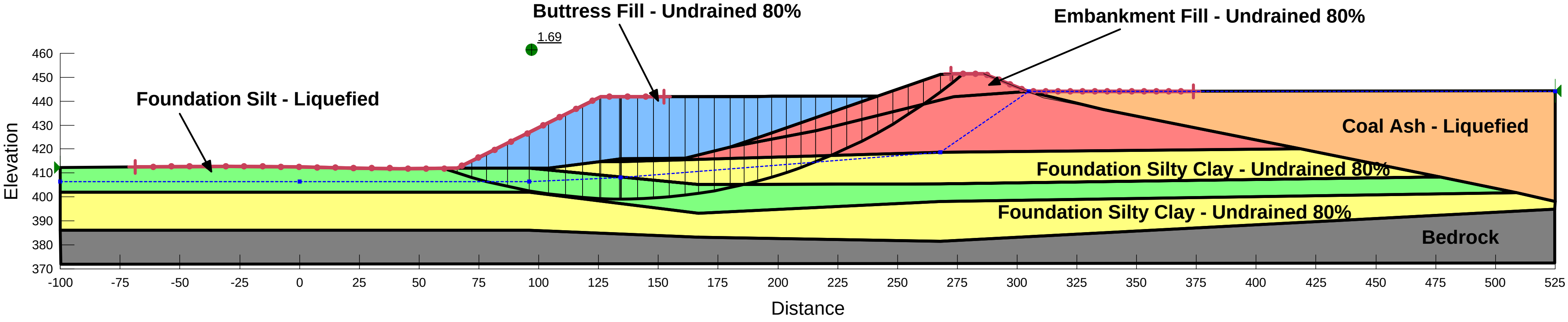
Foundation Silty Clay - Undrained Peak
Unit Weight: 126 pcf
Cohesion: 400 psf
Phi: 23 °

Foundation Silt - Undrained
Unit Weight: 119 pcf
Cohesion: 650 psf
Phi: 22 °

Buttress Fill - Undrained Peak
Unit Weight: 123 pcf
Cohesion: 540 psf
Phi: 20 °

Ash Pond Lower Dan Buttress Evaluation
Vectren A.B. Brown Station

CCR Rule Safety Factor Assessment
Post-Liquefaction - Critical Circular Failure Surface Geometry
Cross-Section E
Date: 9/13/2016



Material Properties

Embankment Fill - Undrained 80%
Unit Weight: 128 pcf
Cohesion: 475 psf
Phi: 18 °

Coal Ash - Liquefied
Unit Weight: 100 pcf
Tau/Sigma Ratio: 0.12
Minimum Strength: 0

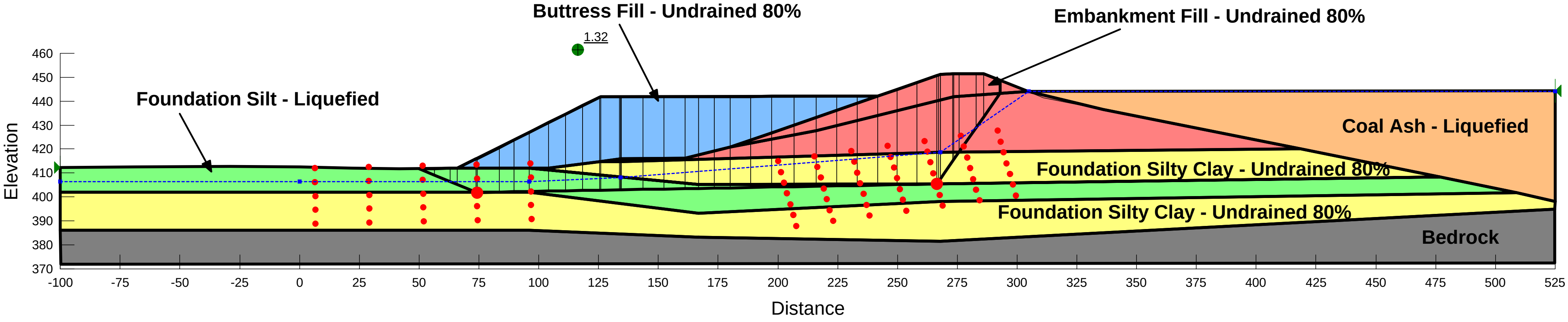
Foundation Silty Clay - Undrained 80%
Unit Weight: 126 pcf
Cohesion: 320 psf
Phi: 19 °

Foundation Silt - Liquefied
Unit Weight: 119 pcf
Tau/Sigma Ratio: 0.1
Minimum Strength: 100

Buttress Fill - Undrained 80%
Unit Weight: 123 pcf
Cohesion: 425 psf
Phi: 16 °

Ash Pond Lower Dan Buttress Evaluation
Vectren A.B. Brown Station

CCR Rule Safety Factor Assessment
Post- Liquefaction - Critical Block Failure Surface Geometry
Cross-Section E
Date: 9/13/2016



Material Properties

Embankment Fill - Undrained 80%
Unit Weight: 128 pcf
Cohesion: 475 psf
Phi: 18 °

Coal Ash - Liquefied
Unit Weight: 100 pcf
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Unit Weight: 119 pcf
Tau/Sigma Ratio: 0.1
Minimum Strength: 100

Buttress Fill - Undrained 80%
Unit Weight: 123 pcf
Cohesion: 425 psf
Phi: 16 °

Appendix G
Probabilistic Seismic Hazard Analysis Report

Site-Specific Probabilistic Seismic Hazard Analysis and Development of Time Histories for A.B. Brown Generating Station in Southwestern Indiana

Prepared for

Vectren Corporation

14 December 2015

Prepared by

AECOM

Patricia Thomas, Melanie Walling, Mark Dober, and Ivan Wong
Seismic Hazards Group
AECOM
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Oakland, CA 94612

TABLE OF CONTENTS

TABLE OF CONTENTS

Tables

Figures

At the request of Vectren Corporation, a site-specific probabilistic seismic hazard analysis (PSHA) has been performed for A.B. Brown Generating Station in southwestern Indiana (Figure 1) for a hard rock site condition. The hard rock hazard results and period-dependent amplification factors were used to compute a 2,500-yr return period Uniform Hazard Spectrum (UHS) for a firm rock site condition characterized by a time-averaged shear-wave velocity in the top 30 m (V_S30) of 760 m/sec (NEHRP B/C boundary). Horizontal acceleration time histories were developed consistent with the firm rock 2,500-yr return period UHS. The firm rock acceleration time histories will be used in liquefaction and deformation analysis of the Lower Ash Pond Dam at the A.B. Brown Generating Station. This report presents the results of the site-specific PSHA and the development of the horizontal acceleration time histories

A.B. Brown Generating Station is located in the Midcontinent region of the U.S. away from active plate boundaries in a region that has exhibited a moderate level of historical seismicity (Figure 1). There have been seven known earthquakes larger than moment magnitude (**M**) 5.0 within 200 km of the site. However, the region is capable of experiencing strong ground motions from moderate to large earthquakes (**M** > 6) particularly from the Wabash Seismic Zone and the New Madrid Seismic Zone to the southwest of the site (Figure 1).

1.1 PURPOSE

As stated in the Statement of Work, the following is the scope of work and deliverables.

Develop mean hazard curves based on performing a PSHA for the site utilizing the 2012 EPRI/DOE/NRC Central and Eastern U.S. (CEUS) Seismic Source Characterization (CEUS-SSC) model and the EPRI (2013) ground motion prediction models. Compute the Uniform Hazard Spectra (UHS) corresponding to horizontal motion in hard rock (shear-wave velocity [V_S] 9,200 ft/sec [2,804 m/sec]) outcrop conditions for an annual frequency of exceedance of 1 in 2,500 at 5% damping. Develop three sets of horizontal acceleration time histories consistent with the 2,500-year hard rock UHS.

Current ground motion prediction models for the CEUS are only available for hard rock conditions, hence the PSHA must be performed for hard rock conditions. However, the depth to hard rock at A.B. Brown Generating Station is estimated to be more than 60 m (200 ft). In order to limit the size of the model used in deformation analyses, acceleration time histories consistent with a 2,500-year UHS for a firm rock site condition (V_S of 760 m/sec) were developed using amplification factors to convert the hard rock UHS to a firm rock site condition.

The PSHA methodology used in this study allows for the explicit inclusion of the range of possible interpretations in components of the model, including seismic source characterization and ground motion estimation. Uncertainties in models and parameters are incorporated into the PSHA through the use of logic trees. This report describes the seismic source model, the ground motion prediction models used in the PSHA, the hard rock hazard results and the development of a 2,500-yr UHS for firm rock and associated time histories.

1.2 ACKNOWLEDGMENTS

The seismic hazard analysis of A.B. Brown Generating Station was performed by Melanie Walling, Mark Dober, Patricia Thomas, and Ivan Wong of the Seismic Hazards Group of

AECOM. Our appreciation to Rajendram Arulnathan for project management support and Melinda Lee for her assistance in the preparation of this report.

The PSHA approach used in this study is based on the model developed principally by Cornell (1968). The occurrence of earthquakes on a fault is assumed to be a Poisson process. The Poisson model is widely used and is a reasonable assumption in regions where data are sufficient to provide only an estimate of average recurrence rate (Cornell, 1968). The occurrence of ground motions at the site in excess of a specified level is also a Poisson process, if (1) the occurrence of earthquakes is a Poisson process, and (2) the probability that any one event will result in ground motions at the site in excess of a specified level is independent of the occurrence of other events.

The probability that a ground motion parameter "Z" exceeds a specified value "z" in a time period "t" is given by:

$$p(Z > z) = 1\text{-}e^{-\nu(z)\bullet t} \qquad (2\text{-}1)$$

where $\nu(z)$ is the annual mean number (or rate) of events in which Z exceeds z. It should be noted that the assumption of a Poisson process for the number of events is not critical. This is because the mean number of events in time t, $\nu(z) \bullet t$, can be shown to be a close upper bound on the probability $p(Z > z)$ for small probabilities (less than 0.10) that generally are of interest for engineering applications. The annual mean number of events is obtained by summing the contributions from all sources, that is:

$$\nu(z) = \sum_{n} \nu_n(z) \qquad (2\text{-}2)$$

where $\nu_n(z)$ is the annual mean number (or rate) of events on source n for which Z exceeds z at the site. The parameter $\nu_n(z)$ is given by the expression:

$$\nu_n(z) = \sum_{i}\sum_{j} ß_n(m_i) \bullet p(R{=}r_j|m_i) \bullet p(Z{>}z|m_i,r_j) \qquad (2\text{-}3)$$

where:

$ß_n(m_i)$ = annual mean rate of recurrence of earthquakes of magnitude increment m_i on source n;

$p(R=r_j|m_i)$ = probability that given the occurrence of an earthquake of magnitude m_i on source n, r_j is the closest distance increment from the rupture surface to the site;

$p(Z > z|m_i,r_j)$ = probability that given an earthquake of magnitude m_i at a distance of r_j, the ground motion exceeds the specified level z.

The calculations were made using the computer program HAZ38CEUS. The basic program (HAZ38) has been validated in the Pacific Earthquake Engineering Research (PEER) Center-sponsored "Validation of PSHA Computer Programs" Project (Thomas *et al.*, 2010). Modifications were made to HAZ38 to incorporate the CEUS-SSC model and the resulting revision, HAZ38CEUS, was validated by comparing hazard results with the test case results contained in EPRI/DOE/NRC (2012).

The following is a general overview of PSHA methodology used by AECOM. For this study, we have adopted the EPRI/DOE/NRC (2012) seismic source model, which required modifications to our general approach. For a detailed description, see EPRI/DOE/NRC (2012). A sample logic tree is shown on Figure 2. Logic trees such as shown on Figure 3 are used in the EPRI/DOE/NRC (2012) model.

2.1 SEISMIC SOURCE CHARACTERIZATION

Three types of earthquake sources are characterized in the CEUS-SSC model: (1) known fault sources; (2) seismotectonic zones; and (3) Mmax zones. Fault sources are modeled as three-dimensional fault surfaces and details of their behavior are incorporated into the source characterization. The inventory of fault sources in the CEUS is small and undoubtedly incomplete. Given this shortcoming, the historical seismicity is used as a proxy to address the hazard from those buried or unknown faults. The spatial density of the historical seismicity was assumed to be stationary; in this model the recurrence rates per area for each small area were smoothed using a Gaussian filter. The resulting seismotectonic and Mmax zones are areal source zones in which earthquakes are modeled as point sources.

The geometric source parameters for faults include fault location, segmentation model, dip, and thickness of the seismogenic zone (Figure 2). The recurrence parameters include recurrence model, recurrence rate (slip rate or average recurrence interval for the maximum event), slope of the recurrence curve (*b*-value), and maximum magnitude. Clearly, the geometry and recurrence are not totally independent. For example, if a fault is modeled with several small segments instead of large segments, the maximum magnitude is lower, and a given slip rate requires many more small earthquakes to accommodate a cumulative seismic moment. For areal source zones, only the area, seismogenic thickness, maximum magnitude, and recurrence parameters (based on the historical earthquake record) need to be defined.

Uncertainties in the CEUS-SSC source parameters are modeled using logic trees. In this procedure, values of the source parameters are represented by the branches of logic trees with weights that define the distribution of values. Sample logic trees are shown on Figures 2 and 3. In general, three or five values for each parameter were weighted and used in the analysis. Note that the weights associated with the percentiles are not equivalent to probabilities for these values, but rather are weights assigned to define the distribution.

2.1.1 Source Geometry

In the PSHA, it is assumed that earthquakes of a certain magnitude may occur randomly along the length of a given fault or segment. The distance from an earthquake to the site is dependent on the source geometry, the size and shape of the rupture on the fault plane, and the likelihood of the earthquake occurring at different points along the fault length. The distance to the fault is defined to be consistent with the specific ground motion prediction model used to calculate the ground motions. The distance, therefore, is dependent on both the dip and depth of the fault plane, and a separate distance function is calculated for each geometry and each ground motion prediction model. The size and shape of the rupture on the fault plane are dependent on the magnitude of the earthquake, with larger events rupturing longer and wider portions of the fault plane. For a given magnitude, the associated rupture surface is uniformly distributed along the fault length and width. Ruptures are constrained to occur entirely on the defined fault plane.

The rupture dimensions can be modeled using magnitude-rupture area and rupture width relationships.

2.1.2 Fault Recurrence

The recurrence relationships for faults are generally modeled using the exponentially truncated Gutenberg-Richter, characteristic earthquake, and the maximum moment (magnitude) recurrence models (Figure 2). These models are weighted to represent judgment on their applicability to the sources. For the areal source zones, only a truncated exponential recurrence relationship is assumed appropriate.

The general approach of Molnar (1979) and Anderson (1979) is often used to arrive at the recurrence for the exponentially truncated model. The number of events exceeding a given magnitude, *N(m)*, for the truncated exponential relationship is

$$N(m) = \alpha(m^o)\frac{10^{-b(m-m^o)} - 10^{-b(m^u-m^o)}}{1 - 10^{-b(m^u-m^o)}} \qquad (2\text{-}4)$$

where $\alpha(m^o)$ is the annual frequency of occurrence of earthquake greater than the minimum magnitude, m^o; b is the Gutenberg-Richter parameter defining the slope of the recurrence curve; and m^u is the upper-bound magnitude event that can occur on the source. A m^o of **M** 5.0 was used for the hazard calculations; this value is also used by the USGS in the National Hazard Maps (Frankel *et al*., 1996; Petersen *et al*., 2008).

A popular model often used in PSHA is where faults rupture with a "characteristic" magnitude on specific segments; this model is described by Aki (1983) and Schwartz and Coppersmith (1984). For the characteristic model, the numerical model of Youngs and Coppersmith (1985) is often used. In the characteristic model, the number of events exceeding a given magnitude is the sum of the characteristic events and the non-characteristic events. The characteristic events are distributed uniformly over a ± 0.25 magnitude unit around the characteristic magnitude and the remainder of the moment rate is distributed exponentially up to the characteristic range using the above equation (Youngs and Coppersmith, 1985).

The maximum moment model can be regarded as an extreme version of the characteristic model. The model proposed by Wesnousky (1986) is often used when there is no exponential portion of the recurrence curve, i.e., no events can occur between the minimum magnitude of **M** 5.0 and the distribution about the maximum magnitude.

The recurrence rates for the fault sources are defined by either the slip rate or the average return time for the maximum or characteristic event and the recurrence *b*-value. The slip rate is used to calculate the moment rate on the fault using the following equation defining the seismic moment:

$$M_o = \mu A D \qquad (2\text{-}5)$$

where M_o is the seismic moment, μ is the shear modulus, A is the area of the rupture plane, and D is the slip on the plane. Dividing both sides of the equation by time results in the moment rate as a function of slip rate:

$$\dot{M}_o = \mu\, A\, S \qquad (2\text{-}6)$$

where $\dot{M}_o$ is the moment rate and S is the slip rate. M_o has been related to moment magnitude, **M**, by Hanks and Kanamori (1979):

$$\mathbf{M} = 2/3 \log M_o - 10.7 \qquad (2\text{-}7)$$

Using this relationship and the relative frequency of different magnitude events from the recurrence model, the slip rate can be used to estimate the absolute frequency of different magnitude events.

The average return time for the characteristic or maximum magnitude event defines the high magnitude (low likelihood) end of the recurrence curve. When combined with the relative frequency of different magnitude events from the recurrence model, the recurrence curve is established.

2.2 GROUND MOTION PREDICTION

To characterize the ground motions at a specified site as a result of the seismic sources considered in the PSHA, we used ground motion prediction models for spectral accelerations (Figure 2; Section 4.2). Ground motion prediction models have at a minimum the variables of magnitude, distance, and site condition (e.g., rock, soil).

The uncertainty in ground motion models was included in the PSHA by using the log-normal distribution about the median values as defined by the standard deviation associated with each model. This distribution was truncated at five standard deviations above the median value predicted by the each model. We have tested our approach using the five sigma truncation against the test cases contained in EPRI/DOE/NRC (2012) where sigma was untruncated. The differences are insignificant.

In this section, we describe the seismotectonic and geologic setting and historical seismicity of the site region.

3.1 SEISMOTECTONIC SETTING

A.B. Brown Generating Station is located in southwestern Indiana, within the Wabash Valley Seismic Zone and about 140 km northeast of the New Madrid Seismic Zone (NMSZ) (Figure 4). Although the site is located within the continental interior and far from active plate boundaries, the preexisting structures formed in earlier tectonic settings are still capable of generating seismicity that can pose a hazard to the region. This seismicity has included several large historical earthquakes in the area (**M** > 7), e.g., the 1811 and 1812 New Madrid earthquakes (Figure 1).

The Wabash Valley Seismic Zone is a region of southwestern Indiana and southeastern Illinois that contains the Wabash Valley fault system (WVFS; see below). Numerous Holocene paleoliquefaction features have been mapped along river valleys within the Wabash Valley Seismic Zone and other regions of southern Indiana and Illinois and have been interpreted as having been caused by paleoearthquakes (e.g., Obermeier *et al.*, 1993). Munson *et al*. (1997) reported that at least eight paleoearthquakes had occurred in the area in the past 20,000 years. However, the faults of the WVFS have been mapped as pre-Quaternary, and no fault has been identified as the causative structure for the liquefaction nor been explicitly correlated with historic or paleoseismicity.

The CEUS is part of a broad mid-plate compressive stress province that also includes most of Canada (Zoback and Zoback, 1991). Over this large region, the stress field is oriented with a relatively uniform east-northeast direction of maximum horizontal compression. This compression direction corresponds well to the direction of absolute plate motion of the North American Plate, which suggests that a far-field tectonic source such as ridge-push or basal drag at the Mid-Atlantic Ridge may be the primary source of stress in the mid-plate region (Zoback and Zoback, 1991).

3.2 HISTORICAL SEISMICITY

The following is a discussion of the historical seismicity and significant earthquakes in the region surrounding A.B. Brown Generating Station.

3.2.1 Catalog

A historical seismicity catalog was derived mainly from the Central and Eastern United States Seismic Source Characterization (CEUS-SSC) catalog (EPRI/NRC/DOE, 2012). This catalog includes data primarily from the catalog compiled by the U.S. Geological Survey (USGS) for the National Seismic Hazard Mapping Project (Mueller *et al*., 1997; Petersen *et al*., 2008) and from the Geological Survey of Canada (GSC) catalog for seismic hazard analyses (Adams and Halchuk, 2003). The main source for the USGS catalog was the NCEER-91 catalog (Seeber and Ambruster, 1991) which updated the original EPRI-SOG (EPRI 1988) catalog. The catalog was then updated using the National Earthquake Information Center's (NEIC) Preliminary Determination of Epicenters (PDE) and data from the National Earthquake Database (NEDB) of Canada. Researchers reviewed original catalogs and special earthquake studies to verify and if needed update original entries, and regional catalogs were incorporated into the continental scale catalogs described above (see EPRI/NRC/DOE, 2012 for details of special study references and list of regional catalogs used). The CEUS-SSC catalog spans the time period of 1568 to 2008. We updated this catalog with more recent data (through 6 March 2013) from the Advanced National Seismic System (ANSS) and NEIC PDE catalogs (Figure 1).

All of the events in the USGS catalog used to compile the CEUS-SSC catalog have body-wave (m_b) magnitude values, which were converted to **M** using the equations of Atkinson and Boore (1995):

$\mathbf{M} = -0.39 + 0.98Mn$ for magnitudes ≤ 5.5

$\mathbf{M} = 2.715 - 0.277Mn + 0.127(Mn^2)$ for magnitudes > 5.5

and Johnston (1996):

$\mathbf{M} = 1.14 + 0.24\ m_b + 0.0933\ m_b^2$

Mn (Nuttli magnitude) was considered to be equivalent to m_b. All events in the PDE catalog that we used to update the CEUS-SSC catalog were Mn or M_D. We converted the PDE Mn magnitudes to **M** using the average of Atkinson and Boore (1995) and Johnston (1996). For the M_D values, we used the same conversion used in the CEUS-SSC catalog to convert them to **M** values for the Mid-Continent U.S. east of 100º W (EPRI/DOE/NRC, 2012).

$\mathbf{M} = 0.869 + 0.762\ M_D$

3.2.2 Significant Earthquakes

The most significant earthquakes to have occurred in the CEUS are the 1811-1812 **M** 7 to 8 New Madrid earthquake sequence and the 1886 **M** 6.8 Charleston, South Carolina, earthquake (Figure 1). The New Madrid earthquake sequence occurred over the winter of 1811-1812 in southeastern Missouri/northeastern Arkansas. This sequence, which was felt as far away as the East Coast (Figure 5), consisted of three principal events on 16 December 1811, 23 January 1812, and 7 February 1812 (referred to as NM1, NM2, and NM3, respectively in Hough *et al.*, 2000) (Figure 6). Because the epicentral region was sparsely populated at the time of the events, little structural damage occurred, and the maximum Modified Mercalli (MM) intensity is IX (NM1) as reinterpreted by Hough *et al.* (2000). The A.B. Brown Generating Station site probably underwent strong ground shaking of MM VII to VIII in the 16 December 1811 mainshock (Figure 5). The NMSZ is currently the most seismically active area in the CEUS (Figure 1).

The most damaging earthquake to have occurred in the southeast U.S. is the 31 August 1886 **M** 6.8 Charleston, South Carolina earthquake. Sixty people were killed and many buildings in the old city of Charleston were damaged or destroyed and estimated property damage was on the order of $23 million (Stover and Coffman, 1993). Liquefaction was extensive with cratering, sand ejecta and fissuring over an area of 1,300 km^2. No surface-faulting was observed. The maximum intensity reported was MM X within an elliptical area trending northeasterly between Charleston and Jedburg (Stover and Coffman, 1993) (Figure 7). The earthquake affected an area of over 5 million km^2 and the site may have been subjected to moderate ground shaking of MM IV even though it is located 880 km northwest of the epicenter (Figure 7).

The Wabash Valley has historically been seismically active with several earthquakes of **M** 4.5 and larger (Figure 1). Hence, the site has been strongly shaken numerous times after the 1811-1812 and 1886 earthquakes. An event on 27 September 1891 occurred near Mt. Vernon, Illinois, which caused chimney damage in the epicentral area (Stover and Coffman, 1993). The size of the earthquake was estimated to be a body-wave magnitude (m_b) 5.8 and the event was felt widely in several states (Figure 8). Shaking at the site could have been as strong as MM V.

On 31 October 1895, an earthquake of estimated surface wave magnitude (M_S) 6.7 struck the northern end of the NMSZ (Figure 9). This is the largest earthquake to have occurred in the central Mississippi Valley since 1811-1812 (Stover and Coffman, 1993). The event caused extensive damage in the town of Charleston, Missouri. Sand blows due to liquefaction were also reported in the epicentral area (Stover and Coffman, 1993). In the area of the site, the ground shaking was probably at a MM VII level (Figure 9).

On 9 November 1968, a m_b 5.5 earthquake struck southern Illinois and neighboring states with a maximum reported MM VII (Figure 10). Damage consisted of damaged chimneys, broken windows, cracked or fallen plaster, cracked foundations, and scattered instances of collapsed parapets (Stover and Coffman, 1993). The site was probably subjected to MM VI to VII ground shaking from this event. Another notable earthquake was the 18 April 2008 **M** 5.4 Southern Illinois earthquake south of the site (Figure 1).

On 27 July 1980, a **M** 5.1 earthquake struck the area near Sharpsburg, Kentucky. This event, the strongest in the history of Kentucky, occurred approximately 340 km east of the site and caused over $1 million in property damage (Stover and Coffman, 1993). The site was probably subjected to intensities of MM II to III (Figure 11).

The 23 August 2011 **M** 5.8 Mineral, Virginia, earthquake occurred within the Central Virginia Seismic Zone and is the largest reported event in this zone. The previous largest event in this zone was an event of estimated **M** 4.8 in 1875. The 2011 earthquake occurred at a shallow depth of 6 km but it was felt throughout the eastern U.S. from central Georgia to central Maine and as far west as Detroit, Michigan and Chicago, Illinois (Figure 12). It may possibly have been lightly felt at the site more than 875 km away, based on the USGS Did You Feel It (DYFI) map (Figure 12).

The following discusses the two major inputs into the PSHA: the seismic source model and the ground motion prediction models.

4.1 SEISMIC SOURCE MODEL

Seismic source characterization is concerned with three fundamental elements: (1) the location, geometry, and characteristics of significant sources of future earthquakes; (2) the maximum size of these earthquakes; and (3) the rate at which different size earthquakes occur. Two types of seismic sources were considered in this PSHA: discrete fault or fault zone sources and regional seismic source zones.

The seismic source characterization presented here is adopted from the comprehensive seismic source characterization of the CEUS, developed for nuclear facilities by EPRI/DOE/NRC (2012). Two zonation models, account for earthquakes associated with buried or generally unknown faults (background), were characterized and included in the PSHA; these models include multiple zones, many having alternative geometries (Figures 13 and 14). In addition, the source parameters for several fault sources or RLMEs (repeated large magnitude earthquakes) were characterized for input into the PSHA (Figure 13).

A major challenge in understanding the earthquake potential in the CEUS has been associating the observed seismicity with specific geologic structures. Few active faults are known east of the Rocky Mountains. Thus the traditional approach in addressing the seismic hazard in the CEUS has been to rely on the historical earthquake record in conjunction with seismic source zones that separate regions of different seismotectonic characteristics and hence possibly different earthquake potential. Each seismic source zone is defined and characterized according to geologic, tectonic, and seismicity data. The zones comprise regions having a common geologic history that distinguishes them from neighboring areas. They may have a similar structure (e.g., faults or fractures of similar age, type, orientation), a similar pattern of seismicity, and/or a homogeneous stress regime. The EPRI/DOE/NRC (2012) model retains this methodology by dividing the CEUS into numerous “seismotectonic zones”, defined by differences in various seismic source assessment criteria such as style of faulting, earthquake recurrence, maximum magnitude, seismogenic thickness, etc. The model includes an alternative approach to dividing the CEUS into source zones, which is based solely on the expected maximum magnitude in the zone. This alternative zonation approach divides the study area into “Mmax zones” (Figure 14). The seismotectonic zone approach receives slightly higher weight, 0.6, than the Mmax zone approach, 0.4.

Figures 13 and 14 show the locations of the seismotectonic and Mmax zones, respectively. There are three Mmax zones and 12 seismotectonic zones in the EPRI/DOE/NRC model. The Mmax zones and some seismotectonic zones have one or more alternate geometries. Table 1 summarizes the source zone parameters used in the analysis. (Not all seismic source zones are shown on Figure 13.) A.B. Brown Generating Station lies in the Illinois Basin Extended Basin Zone (IBEB) zone and near the boundary of the Wabash Valley RLME zone (Figure 13).

Table 1
Seismic Source Zones Incorporated Into Analysis

Source Zone	Symbol	Mmax (M)[1]	Seismogenic Depth[2] (km)	Area (km^2)
Seismotectonic Zones				
Atlantic Highly Extended Crust	AHEX	6.0 6.7 7.2 7.7 8.1	8 (0.5) 15 (0.5)	177683
Extended Continental Crust–Atlantic Margin Zone	ECC-AM	6.0 6.7 7.2 7.7 8.1	13 (0.4) 17 (0.4) 22 (0.2)	881480
Extended Continental Crust–Gulf Coast	ECC-GC	6.0 6.7 7.2 7.7 8.1	13 (0.4) 17 (0.4) 22 (0.2)	1239288
Gulf Highly Extended Crust	GHEX	6.0 6.7 7.2 7.7 8.1	8 (0.5) 15 (0.5)	509090
Great Meteor Hotspot Zone	GMH	6.0 6.7 7.2 7.7 8.1	25 (0.5) 30 (0.5)	32250
Illinois Basin Extended Basin Zone	IBEB	6.5 6.9 7.4 7.8 8.1	13 (0.4) 17 (0.4) 22 (0.2)	114526
Midcontinent Craton Zone (all alternatives)	MidC	5.6 6.1 6.6 7.2 8.0	13 (0.4) 17 (0.4) 22 (0.2)	4258598 4246625 4025001 4013028
Northern Appalachian Zone	NAP	6.1 6.7 7.2 7.7 8.1	13 (0.4) 17 (0.4) 22 (0.2)	378331
Oklahoma Aulacogen Zone	OKA	5.8 6.4 6.9 7.4 8.0	15 (0.5) 20 (0.5)	53583

Source Zone	**Symbol**	**Mmax (M)[1]**	**Seismogenic Depth[2] (km)**	**Area (km^2)**
Paleozoic Extended Crust (Narrow and Wide alternatives)	PEZ	5.9 6.4 6.8 7.2 7.9	13 (0.4) 17 (0.4) 22 (0.2)	365395 598992
Reelfoot Rift Zone	RR	6.2 6.7 7.2 7.7 8.1	13 (0.4) 15 (0.4) 17 (0.2)	69479
Reelfoot Rift with Rough Creek Graben Zone	RR and RR_RCG	6.1 6.6 7.1 7.6 8.1	13 (0.4) 15 (0.4) 17 (0.2)	81452
St. Lawrence Rift Zone	SLR	6.2 6.8 7.3 7.7 8.1	25 (0.5) 30 (0.5)	329322
Mmax Zones				
Mesozoic and Younger Extended Crust - Narrow	MESE-N	6.4 6.8 7.2 7.7 8.1	13 (0.4) 17 (0.4) 22 (0.2)	3616923
Mesozoic and Younger Extended Crust - Wide	MESE-W	6.5 6.9 7.3 7.7 8.1	13 (0.4) 17 (0.4) 22 (0.2)	4342413
Non-Mesozoic and Younger Extended Crust - Narrow	NMESE-N	6.4 6.8 7.1 7.5 8.0	13 (0.4) 17 (0.4) 22 (0.2)	4792101
Non-Mesozoic and Younger Extended Crust - Wide	NMESE-W	5.7 6.1 6.6 7.2 7.9	13 (0.4) 17 (0.4) 22 (0.2)	4066611
Study Region	Study Region	6.5 6.9 7.2 7.7 8.1	13 (0.4) 17 (0.4) 22 (0.2)	8409024

Notes:

[1] Weights for all magnitude distributions are 0.101/0.244/0.310/0.244/0.101, a discrete five-point approximation to an arbitrary continuous distribution (EPRI/DOE/NRC, 2012).

[2] Weights for depth in parentheses

The EPRI/DOE/NRC (2012) model includes sources defined based on RLMEs rather than only fault sources. Many of the RLMEs correlate with identified geologic faults, but some are defined solely by geographically clustered paleoliquefaction events that suggest a localized source even if the responsible fault has not been identified and characterized. The site is adjacent to the Wabash Valley RLME zone and the New Madrid fault system (NMFS) lies approximately 200 km to the south of the site (Figures 6 and 13). Although quite distant from the site, we include the Charleston source and the NMFS and its associated elements (Figures 6 and 13) in the PSHA because their maximum earthquakes and relatively high activity rates often dominate the hazard in the CEUS, particularly at long-period ground motions. The Reelfoot Rift-Eastern Rift Margin (ERM) fault, the Reelfoot Rift-Marianna fault, and the Reelfoot-Commerce fault zone, to the southwest were also included in the PSHA (Figure 6). Tables 2 and 3 summarize the RLME (fault) source parameters used in the analysis.

4.1.1 Seismotectonic Zones

This section describes the seismotectonic characteristics of the most significant seismotectonic zones to the site, the basis for delineating the zone and for defining the model values for style of faulting, geometry, seismogenic depth, and Mmax. Recurrence for the zones is discussed in Section 4.1.3.

Illinois Basin Extended Basement Zone (IBEB)

The site lies within the IBEB zone, which encompasses southwestern Indiana and southeastern Illinois (Figure 13). Southern Indiana and southern Illinois are characterized by several moderate-sized paleoearthquakes and by higher rates of seismicity than adjacent craton regions (Figure 4). Several characteristics combine to support the delineation of IBEB as a separate seismotectonic zone. The southern part of the Illinois basin is one of the most structurally complex areas of the Midcontinent (McBride *et al*., 2002), with a crust distinct from that of the neighboring craton. Numerous moderately dipping reflectors interpreted to be faults are present in the basement. Moderate-sized historical earthquakes that appear to be spatially associated with Precambrian basement faults and with Paleozoic faults suggest continued reactivation of older basement features as well as younger Paleozoic structures (McBride *et al*., 2002). Stresses induced by Mesozoic rifting possibly extend into the southern Illinois basin causing the reactivation of deep structures (Braile *et al*., 1984). The IBEB source zone is defined to characterize sources of moderate- to large-magnitude earthquakes (excluding those attributed to the Wabash Valley RLME source) that may occur on deep structures in the Precambrian basement and as Paleozoic faults that extend into the overlying Paleozoic sedimentary rocks (EPRI/DOE/NRC 2012).

Fault dips are generalized based on sense of slip, with strike-slip ruptures assigned steep dips between 70° and 90° and reverse ruptures assigned moderate dips between 40° and 70°. Seismogenic thickness ranges from 13 to 22 km, the default values for the entire study area (EPRI/NRC/DOE, 2012). The seismogenic thickness is based on reported depths of seismicity within the IBEB. The deepest well-constrained earthquake hypocenters in the deep part of the Illinois basin, are located at depths of 20 to 22 km (McBride *et al*., 2002; Yang *et al*., 2009). However, the average depth throughout the IBEB zone based on other historical earthquakes may be less (EPRI/DOE/NRC, 2012).

The largest earthquakes in the IBEB zone include an August 1891 **M** 5.5 event, a September 1891 **M** 5.0 event in eastern Nebraska, and a 2008 **M** 5.3 event. Four prehistoric earthquakes inferred from the paleoliquefaction studies have estimated magnitudes (**M** 6.2 to 6.3) that are larger than the historical earthquakes (EPRI/DOE/NRC, 2012). Maximum magnitudes modeled in the IBEB range from **M** 6.5 to 8.1, with a value of **M** 7.4 being preferred.

Midcontinent-Craton Zone (MidC)

The MidC zone occupies most of the CEUS study area, dominating the central United States and encompassing most of the Great Plains area (Figure 13). The MidC zone includes those regions of the continent that have not occupied the Phanerozoic continental margin, specifically Precambrian basement rocks of the Canadian shield and the platform (EPRI/DOE/NRC, 2012). The craton was formed by Paleoproterozoic accretion and now forms a cold, strong crustal core to the continent. Two orthogonal sets of structures, northeast-striking ductile shear zones and northwest-striking brittle-ductile faults dominate the Precambrian basement structure (Sims *et al*., 2005). Numerous geophysical anomalies have been observed within the MidC zone and may represent zones of crustal weakness that could localize future seismicity. Seismicity in the MidC zone is spatially variable and includes a few concentrations of activity that constitute seismic zones within the greater seismotectonic zone, such as the Anna seismic zone and Northeast Ohio seismic zone in Ohio, and the Nehama Ridge seismic zone in Kansas.

The fundamental distinguishing characteristic of the MidC zone is that it contains crust that has not experienced Mesozoic or younger extension, and generally not Paleozoic extension either. The characterization of the seismotectonic zone includes four alternative geometries, based on the inclusion or exclusion of smaller Mid-Continent regions. These smaller zones include a northeast-trending band of crust along the Appalachian Mountains that is included either within the PEZ or within the MidC zone, and the Rough Creek Graben, which is included either in the Reelfoot Ridge zone (RR) or in the MidC zone (Figure 13).

The largest earthquakes in the MidC zone include a 1909 **M** 5.7 event in eastern Montana, an 1877 **M** 5.5 event in eastern Nebraska, and a 1964 **M** 4.8 earthquake in eastern Ontario. Maximum magnitudes have a broader distribution in the MidC than most other seismotectonic zones, ranging from **M** 5.6 to 8.0, with a value of **M** 6.6 being preferred.

Few data exist to characterize independently the deep Precambrian structures within the intracratonic MidC region on which future earthquakes might be preferentially located. Thus the characterization of the MidC region is equivalent to what EPRI/DOE/NRC (2012) calls the "default" seismotectonic characteristics, representative of the entire study region. Thus both strike-slip and reverse mechanisms are included, with a 2/3 weight on strike-slip, reflecting the occurrence of both mechanisms in focal mechanism data, the state of stress, and the orientation of existing geologic structures in the region. Strikes include northwest, north-south, northeast and east-west orientations, determined based on focal mechanism data, tectonic stress, and structural grain within the study area. The dips are generalized based on sense of slip, with strike-slip ruptures assigned steep dips between 60° and 90° and reverse ruptures assigned moderate dips between 30° and 60°. Seismogenic thickness ranges from 13 to 22 km.

4.1.2 Mmax Zones

The Mmax zones are based on the observation that within the global catalogue of earthquakes within stable continental regions, there is little to distinguish any of them in a statistically significant way except that larger earthquakes seem to occur more commonly within those parts of the stable continental regions that have undergone extension, especially Mesozoic or younger extension (Johnston *et al*., 1994). Consequently, the zonation model is based on using global analogues to characterize the maximum magnitudes, with regions divided into extended and cratonic categories, each with a different distribution of maximum magnitudes. We adopt the zone boundaries and maximum magnitude distribution of EPRI/DOE/NRC (2012). The maximum magnitude distributions are used for the background seismicity.

The EPRI/DOE/NRC statistical analysis of the global database of earthquakes in stable continental regions (SCR) showed that the distinction between Mesozoic extended crust and non-extended crust noted by Johnston *et al*. (1994), while present, is only marginally significant. Therefore, within the Mmax zonation approach, two models are included: 1) the CEUS is divided into two Mmax zones, each with its own Mmax distribution, based on the presence or absence of Mesozoic-extended crust, and 2) the CEUS can be described by a single Mmax zone with a single Mmax distribution. The former model has slightly higher weight because of the marginally significant difference observed in the statistical analyses.

Mesozoic and Younger Extended Crust (MESE)

The Mesozoic extended zone (MESE) includes areas that underwent Paleozoic and Mesozoic or younger extension and includes the Atlantic and Gulf coastal regions as well as the failed rifts in the central U.S. (including the Reelfoot Rift and southern Oklahoma aulocogen) (Figure 14).

Non-Mesozoic and Younger Extended Crust (NMESE)

The Non-Mesozoic and Younger extended crust (NMESE) includes that part of the CEUS stable continental region that has not undergone Mesozoic or younger extension. This includes primarily interior cratonic regions and overlaps significantly with the MidC seismotectonic zone.

The boundaries between the extended and non-extended Mmax zones have two alternatives, reflecting uncertainty in the geographic extent of extended crust (Figure 14). The MESE-N (N = “narrow”) includes regions that have definitively experienced Mesozoic extension as inferred based on the presence of certain distinguishing characteristics. These may include: Mesozoic grabens and rift basin, Mesozoic and younger plutons, Mesozoic and younger uplift and unroofing associated with normal faulting (EPRI/DOE/NRC, 2012). Generally, regions that meet most of these criteria are considered to be extended and are assigned to the MESE-N zone. Regions with less compelling evidence, such as localized Mesozoic and younger reactivation of older structures or the presence of structures favorably oriented for reactivation, are less certainly extended and are assigned to the MESE-W (W = “wide”) zone. The NMESE-N and NMESE-W zones include the rest of the CEUS region outside the MESE-N and MESE-W zones, respectively. The narrow boundary, dividing definitively extended crust from the rest of the craton receives most of the weight (0.8) due to the lack of clear evidence for extension in the MESE-W zone.

The narrow and wide geometry for each zone has its own maximum magnitude distribution for this region, based on the largest historical earthquake known in each zone. These appear in Table 1 (Table 6.3.2-1 in EPRI/DOE/NRC, 2012).

Study Region

The single-zone alternative of the Mmax zone model includes the Study Region (StudyR) source zone (Figure 14), which encompasses the entire study area, which is represented by a single Mmax distribution. The distributions for seismogenic depth and Mmax for this zone appear in Table 1.

4.1.3 Recurrence for Seismic Zonation

The CEUS-SSC model is based on the spatial stationarity of seismicity, which is defined from small- to moderate-magnitude earthquakes that have occurred during a relatively short historical and instrumental record (EPRI/DOE/NRC, 2012).

For the seismotectonic and Mmax source zones, the seismicity rates are determined from the historical seismicity catalog. All dependent earthquakes were removed from the catalog, and earthquakes associated with the RLME sources were also removed to avoid double-counting. The cell size for all seismotectonic source zones except MidC was 0.25 degrees; the cell size for MidC was set to 0.5 degrees. The spatial smoothing operation, a penalized-likelihood function, is based on calculations of earthquake recurrence within each cell. Both *a*- and *b*- values are allowed to vary, but the degree of variation has been optimized such that *b*-values vary little across the study region, and the *a*-values are neither too smooth or spikey. Also, the recurrence calculations consider weighting of magnitudes in the recurrence rate calculations, with moderate events assigned more weight than smaller events.

Five alternative cases were considered for weights, which affect the degree of smoothing, for various magnitude bins; Cases A, B, C, D, and E (EPRI/DOE/NRC, 2012). Case C was dropped as it is very similar to Case B, and Case D was considered too extreme. Thus for each source zone three magnitude weighted cases were used: A, B, and E, with weights of 0.3, 0.3, and 0.4, respectively.

Furthermore, more than point estimates of the recurrence parameters are needed as modern PSHA requires an assessment of the epistemic uncertainty associated with these estimates, including correlations between the recurrence parameters of cells in the same geographical region, which may jointly affect the hazard at one site. The approach used to generate alternative maps of the recurrence parameters uses a technique known as Markov Chain Monte Carlo (MCMC) (EPRI/DOE/NRC, 2012).

This resulted in eight alternative maps representing the uncertainty in recurrence parameters that result from the limited duration of the catalog. If the smoothing parameters are treated as uncertain and estimated objectively from the data, the eight alternative maps also include the uncertainty about the appropriate values of the smoothing parameters. The eight realizations are equally weighted. For computational efficiency, the mean of the eight realizations was utilized in these calculations.

4.1.4 RLME

The following describes the Wabash Valley and NMFS RLMEs, which are the most significant RLMEs to the site.

Wabash Valley Fault Zone

The north-northeast-trending WVFS consists of numerous high-angle oblique-slip faults that comprise a broad 80-km-long zone located within the limits of the Grayville graben (Figure 6). The Wabash Valley RLME as configured in the CEUS-SSC model is significantly longer than the WVFS proper and extends north to include the Vincennes, Indiana area (Figures 6 and 13). The Grayville graben formed during Iapetan rifting (Hildenbrand and Ravat, 1997; EPRI/DOE/NRC, 2012). Direct evidence for neotectonic activity, including exposures of Quaternary displacement, was documented along the WVFS by Woolery (2005). He interpreted offset of a reflector, identified as a late Quaternary (ca 37,000 years old) sand, revealed in high-resolution seismic reflection profiles as due to displacement across the Hovey Lake fault at the south end of the WVFS. More recent work by Counts *et al*. (2009) and Van Arsdale *et al*. (2009) has identified Holocene deformation across the Uniontown scarp, part of the Hovey Lake fault. Van Arsdale *et al*. (2009) excavated a trench exposing 3500-year-old Ohio River alluvium that had been folded in a monocline with a 3-m amplitude, and also observed fractures within a younger unit that indicate possible activity within the last 295 years. For the most part, activity of the WVFS is indicated by historical seismicity and the aforementioned paleoliquefaction features. The historic seismicity includes five slightly damaging earthquakes of mb 5.0 to 5.8 during 200 years of historical time (Figure 1).

The maximum magnitude estimates adopted from the EPRI/DOE/NRC (2012) CEUS source characterization of the Wabash Valley source are based on analysis of paleoliquefaction features in the vicinity of the lower Wabash Valley of southern Illinois and Indiana. The magnitude of the largest paleoearthquake in the lower Wabash Valley (the Vincennes-Bridgeport earthquake), which occurred 6,011 ± 200 yr BP, was estimated to be ≥ **M** 7.5 using the magnitude-bound method (Obermeier, 1998). Use of a more recently developed magnitude-bound curve for the CEUS gives a lower estimate of **M** 7.1 to 7.3 (Olsen *et al*. (2005). The lower-bound relationship developed by Castilla and Audermard (2007) from a worldwide database gives a range of **M** 7.0 to 7.3. Estimates based on asuite of geotechnical analyses (cyclic stress and energy stress methods) range from **M** 7.5 to 7.8 (summarized in Obermeier *et al*., 1993). The next largest earthquake, the Skelton paleoearthquake, occurred 12,000 ± 1,000 yr BP (Obermeier, 1998). Lower and upperbound magnitude range from **M** 6.3 to 7.3 based on estimates by Munson *et al*. 1997, Olsen *et al*., 2005 and Castilla and Audemard (2007). The magnitude distribution of the EPRI/DOE/NRC (2012) CEUS source model (Table 2) incorporates the range of estimated sizes of the Vincennes-Bridgeport and Skelton paleoearthquakes as representative of both the aleatory variability in the size of individual Wabash Valley RLMEs and the epistemic uncertainty in the approaches and data used to estimate the magnitudes of prehistoric earthquakes.

The recurrence rates for the Wabash Valley RLME (Table 2) are based on the estimated ages for the Vincennes-Bridgeport and Skeleton paleoearthquakes using a Poisson model (EPRI/DOE/NRC, 2012).

Table 2
RLME Sources Incorporated Into Analysis

Fault	Geometry	Style of Faulting[1]	Mmax (M)	Dip (deg)	Seismogenic Thickness (km)	Recurrence Data[2]	Recurrence Interval (yr)[3]
Reelfoot Rift - Eastern Rift Margin Fault (ERM)							
ERM-N	ERM-N (1.0)	SS	6.7 (0.3) 6.9 (0.3) 7.1 (0.3) 7.4 (0.1)	90	13 (0.3) 15 (0.5) 17 (0.2)	1 event in 12-35 kyr (0.9)	3448 6667 12500 25000 71429
						2 events in 12-35 kyr (0.1)	2564 4545 7692 13889 31250
ERM-S	ERM-SCC (0.6)	SS	6.7 (0.15) 6.9 (0.2) 7.1 (0.2) 7.3 (0.2) 7.5 (0.2) 7.7 (0.05)	90	same as above	2 events in 17.7-21.7 kyr (0.333)	2857 4762 7143 12500 27778
						3 events in 17.7-21.7 kyr (0.334)	2326 3571 5263 8333 16129
						4 events in 17.7-21.7 kyr (0.333)	2000 2941 4167 6250 11111
	ERM-SRP (0.4)	same as above	same as above	same as above	same as above	same as above	same as above
Reelfoot Rift-Marianna In cluster (0.5) [Out of cluster (0.5) - default to background]	Marianna NW-strike (0.5)	SS	6.7 (0.15) 6.9 (0.2) 7.1 (0.2) 7.3 (0.2) 7.5 (0.2) 7.7 (0.05)	90	13 (0.3) 15 (0.5) 17 (0.2)	3 events in 9.6-10.2 kyr	1449 2381 3704 6250 13889
						4 events in 9.6-10.2 kyr	1190 1818 2703 4167 8333
	Marianna NE-strike (0.5)	same as above	same as above	same as above	same as above	same as above	same as above

Fault	Geometry	Style of Faulting[1]	Mmax (M)	Dip (deg)	Seismogenic Thickness (km)	Recurrence Data[2]	Recurrence Interval (yr)[3]
Reelfoot Rift - Commerce Fault Zone	Commerce fault (1.0)	SS	6.7 (0.15) 6.9 (0.35) 7.1 (0.35) 7.3 (0.1) 7.7 (0.05)	90	13 (0.3) 15 (0.5) 17 (0.2)	2 events in 18.9-23.6 kyr	4000 7143 12500 25000 71429
						3 events in 18.9-23.6 kyr	3030 5000 7692 13158 29412
Wabash Valley	Wabash Valley zone (1.0)	SS	6.75 (0.05) 7 (0.25) 7.25 (0.35) 7.5 (0.35)	90		2 events in 11-13 kyr	2273 4000 7143 13889 41667
Charleston	Local (0.5)	SS	6.7 (0.1) 6.9 (0.25) 7.1 (0.3) 7.3 (0.25) 7.5 (0.1)	90	13 (0.4) 17 (0.4) 22 (0.2)	2,000-yr record (0.8) 4 events in 2 kyr (1.0)	213 323 476 769 1471
						5,500-yr record (0.2) 4 events in 5.5 kyr (0.2)	213 323 476 769 1471
						5 events in 5.5 kyr (0.3)	370 526 769 1136 2000
						5 events in 5.5 kyr (0.2)	526 769 1086 1562 2941
						6 events in 5.5 kyr (0.3)	455 667 909 1282 2174
	Narrow (0.3)	SS	same as above	90	same as above	same as above	same as above
	Regional (0.2)	SS	same as above	90	same as above	same as above	same as above
New Madrid Fault System (NMFS)	see Table 3						

Note: Values in parentheses are weights. All faults are modeled with the Characteristic recurrence model

[1] SS Strike-slip

[2] "Recurrence Data" describes datasets used to calculate recurrence intervals.

[3] Weights for all distributions are: 0.101/0.244/0.310/0.244/0.101.

New Madrid Fault System (NMFS) RLME

The NMSZ is the most likely site of the 1811-1812 New Madrid earthquake sequence, which includes three of the largest earthquakes to have occurred within the North American plate in historical times (Johnston and Shedlock, 1992) (Figure 6). The pattern of seismicity and surface uplift is generally interpreted as delineating a left-stepping, right-lateral, strike-slip fault system (Cox *et al*., 2001; Johnston and Schweig, 1996). Johnston and Schweig (1996) developed faulting models for the 1811-1812 sequence based on geological, geophysical, seismological, and historical data. They concur with the commonly held assumption that the current seismicity is illuminating the most active faults; i.e., those that ruptured in 1811–1812 and also prior to 1811.

Schweig and Ellis (1994) and Johnston and Schweig (1996) provide summaries of the seismological, geodetic, and paleoseismologic data that have been used to assess the repeat times of large-magnitude events in the New Madrid region. In addition, Wheeler and Perkins (2000) provide additional information from the 2002 USGS National Hazard Maps for the CEUS. Correlation of dated liquefaction features suggest that widespread liquefaction occurred within the zone in A.D. 1811-1812, 1450, 900, 300 as well as about 2350 B.C. (Tuttle *et al*., 2005). Liquefaction deposits can constrain the ages of prehistoric events but not the causative faults. However, several of the prehistoric liquefaction deposits are composite, indicating they were formed in multiple episodes within a short period and thus may have occurred in a rapid sequence of large earthquakes similar to the 1811-1812 sequence.

The occurrence of two large events in A.D. ~900 and 2500-1400 B.C. is supported by recent studies of Mississippi River channel morphology that suggest that the Mississippi River changed its course in response to a sudden localized change in base level at those times (Holbrook *et al*., 2006). That change in base level is attributed to uplift of the downstream side of the channel across the Reelfoot reverse fault (described below).

These paleoseismic results indicate a recurrence interval of about 500 years for large earthquakes or earthquake sequences in the NMSZ over the past 2,000 years. The absence of paleoseismic evidence for earthquakes between 300 A.D. and 2200-2350 B.C. has been cited as indicative of temporal clustering of earthquakes in the NMSZ, with large earthquakes or earthquake sequences happening every few hundred years over a period of time followed by a long hiatus in activity (Holbrook *et al*., 2006). However, at this point it remains uncertain if the lack of events documented between A.D. 300 and 2200 B.C. in New Madrid is due to clustering or an incomplete paleoseismic record.

The possibly clustered behavior in the NMSZ, coupled with the discovery of paleoliquefaction features in the Reelfoot Rift (RR) southwest of the New Madrid zone (indicative of large earthquakes between about 5,000 and 7,000 years ago but not during the New Madrid cycles), has led to the suggestion that the locus of earthquake activity moves around the RR, on time scales of 5 to 15 kyr. In this model, the New Madrid region is the current, or most recent, locus of activity, but other areas have been so in the past, and the locus may shift again.

In the seismic source model, the elevated seismicity in the NMSZ is included in the RR seismotectonic zone, whereas large historical and paleoseismic events that likely occurred on the structures that ruptured in 1811-1812 are modeled as part of the NMFS RLME, in keeping with the EPRI/DOE/NRC (2012) model. The source zone accommodates the hazard from background seismicity; the NMFS contributes an additional hazard (Tables 1 and 2). In the seismic source model, the NMFS comprises three distinct fault zones, located within the NMSZ source zone (Figure 6). The three NMFS faults, defined after the models of Van Arsdale (2000) and Johnston and Schwieg (1996), include: 1) the southern section (NMS), comprising the Blytheville arch (BA), extending into the Blytheville fault zone (BFZ) and Bootheel lineament (BL) area, 2) the central section, comprising the Reelfoot reverse fault (RFT), and 3) the northern section, comprising the New Madrid North fault and the Northwestern Seismicity Arm (NMN) (Figure 6; Table 3). Each of these sections ruptured to produce the 1811 and 1812 earthquakes.

The faults of the NMFS are defined primarily based on concentrations of seismicity as geomorphic expression of faulting is poor; only the Reelfoot reverse fault is well expressed as a definitively tectonic feature. Several different geologic faults have been postulated as the source of the events but there remains considerable uncertainty in defining the causative faults. The southern and northern sections of the fault system are northeast-striking features that are probably ancient faults related to rifting that have been reactivated in the modern stress regime as primarily right-lateral strike-slip faults. Focal mechanisms from these areas are consistent with predominantly dextral motion. The Reelfoot reverse fault strikes northwest and dips southwest; earthquakes associated with it have a variety of focal mechanisms. The fault has been described as a cross-structure in a compressional left step between right-lateral strike-slip faults.

Van Arsdale (2000) reports that the first of the 1811 and 1812 earthquakes, the NM1 event in December 1811, occurred on the southern section (NMS), which extends about 110 km (69 mi) from northeastern Arkansas to the southeastern bootheel of Missouri (EOI, 2008). The rupture occurred along the Blytheville arch, a 10 to 15-km wide northeast-trending Paleozoic upwarp that lies along the axis of the RR, and extended northeast of the arch proper. Van Arsdale (2000) considers that the event may have resulted from rupture of the 65-km long, steeply dipping to vertical, dextral-oblique Cottonwood Grove-Ridgely fault. Johnston and Schweig (1996) assign the northern extension of the rupture to the Blytheville fault, a 55-km long structure that continues on trend with the Blytheville arch and lies about 4 km east of the Cottonwood Grove fault. However, they suggest the Blytheville fault and the Cottonwood Grove fault may be essentially the same structure.

In contrast, Schweig and Ellis (1994) and Johnston and Schweig (1996) have proposed that the 1811 rupture did not follow the northeastern trend of seismicity along the Blytheville and/or Cottonwood Grove fault but rather branched onto the more northerly trending Bootheel lineament to the west of the Cottonwood Grove fault (Figure 6). This structure extends 135 km south-southwest from the western edge of the Reelfoot fault, crossing the Blytheville Arch. It was originally defined only as a lineament based on a linear alignment of *en echelon* fissures and sandblows, but has since been identified as a fault based on observations of Holocene surface faulting (Guccione *et al.*, 2005). Unlike the Cottonwood Grove-Ridgely fault, the Bootheel lineament is not associated with a significant amount of seismicity, yet it is considered a candidate for the source of the December 1811 main event because of the numerous liquefaction features that occurred along it (Schweig and Marple, 1991).

Johnston and Schweig (1996) propose two alternative rupture scenarios for the December earthquake: 1) the Blytheville Arch region ruptured along with its extension to the northeast, the Blytheville fault (NMS: BA-BFZ) and 2) the Blytheville Arch ruptured, but the rupture branched onto the Bootheel lineament and ruptured the northernmost 70 km of that structure (NMS: BA-BL) (Figure 6). In each scenario, the structure that did not rupture in the main event was the source of one of more of the large aftershocks, which have been proposed as smaller mainshocks (Johnston and Schweig, 1996). In other words, the Bootheel lineament and Blytheville fault sustained the aftershocks in the first and second scenarios, respectively.

The second mainshock of the New Madrid 1811-1812 sequence was the NM2 earthquake, in January 1812, on the northern margin of the fault system (NMN; Figure 6). The source of this event is also uncertain. The region is delineated by a line of seismicity, the Northwestern Seismicity Arm. Concentrated seismicity extends about 40 km (25 mi), with more sparse seismicity extending another 20 km to near the Illinois border. This seismicity has been postulated to be correlated with the New Madrid North fault (sometimes the East Prairie fault), which has been seen in the subsurface, geomorphically, and in trench exposures (Baldwin *et al.*, 2005; Johnston and Schweig, 1996). That fault is at least 30 km long; the seismicity extends beyond the known fault. Wheeler (1997) postulated that the structure continued still farther north to merge with the Rough Creek graben in western Kentucky; he considered this extent, about 100 km , to be the maximum extent of RR faults. There is little in the sparse distribution of seismicity and lack of significant Quaternary faulting in the northern extent to support that assertion, and based on surface and subsurface expression as well as focal mechanisms, this fault is likely a steeply dipping dextral fault (DTEE, 2011).

The last of the three 1811-1812 mainshocks, NM3, occurred in February 1812, on the central section, the Reelfoot reverse fault, the proposed cross-structure in a compressional step-over between the dextral southern and northern sections of the system (Figure 6). The Reelfoot fault is a south-dipping blind reverse fault that has a dip that varies laterally and down dip. The dip can be as steep as 45°-75° in the upper few kilometers and as shallow as 25°-30° at depth (Mueller and Pujol 2001; Csontos and Van Arsdale, 2008). This fault is well-expressed geomorphically with a pronounced scarp, but its extent is also uncertain because seismicity extends beyond the scarp in both directions, beyond the strike-slip faults of the postulated stepover. Johnston and Schweig (1996) define three distinct fault segments: 1) the central Reelfoot fault, defined by its mapped surface extent of about 32 km (Van Arsdale *et al.*, 1995); 2) the Reelfoot South seismicity trend, extending 35 km east of the Reelfoot fault; and 3) the New Madrid West seismicity trend, extending about 40 km west of the Reelfoot fault. Their proposed rupture scenarios include rupture of the Reelfoot fault with one or the other of the flanking seismicity trends in the NM3 mainshock.

Table 3
New Madrid Fault System RLME Source Model

Cluster?	wt	Localizing Structures	Southern Fault Geometry	wt	Northern Fault Geometry	wt	Central Fault Geometry	wt	Thickness (km)	wt	Mmax	wt	Recurrence method	wt	Recurrence Data	wt	Earthquake Recurrence Model	wt	Repeat Time Coefficient of Variation	wt	Rate (yrs)	wt
All In	0.9	NMS NMN RFT	BA-BL	0.6	NMN-S	0.7	RFT-S	0.7	13	0.4	NMS, RFT, NMN 7.9, 7.8, 7.6	0.167	Intervals	1.0	1811-1812, 1450, and 900 AD	1.0	Poisson	0.75	NA		167	0.101
																					270	0.244
																					417	0.310
																					714	0.244
																					1613	0.101
											7.8, 7.7, 7.5	0.167	same as above				Renewal	0.25	0.3	0.2	286	0.101
											7.6, 7.8, 7.5	0.25									909	0.244
											7.2, 7.4, 7.2	0.085									3125	0.310
											6.9, 7.3, 7.0	0.25									15625	0.244
											6.7, 7.1, 6.8	0.085									212766	0.101
									15	0.4	same as above								0.5	0.5	208	0.101
									17	0.2											455	0.244
							RFT-L	0.3	same as above												1124	0.310
					NMN-L	0.3	same as above														3846	0.244
			BA-BFZ	0.4	same as above																32258	0.101
																			0.7	0.3	227	0.101
																					455	0.244
																					1000	0.310
																					2941	0.244
																					21277	0.101
All out except RFT	0.05	RFT	NA		NA		RFT-S	0.7	13	0.4	7.8	0.167	Intervals	1.0	2000 BC and 1000 AD	1.0	Poisson	1.0	NA		769	0.101
																					1389	0.244
																					2381	0.310
																					4545	0.244
																					12500	0.101
											7.7	0.167	same as above									
											7.8	0.25										
											7.4	0.085										
											7.3	0.25										
											7.1	0.085										
									15	0.4	same as above											
									17	0.2												
							RFT-L	0.3	same as above													
All Out	0.05	None	Revert to background																			

The third event may have served to accommodate the strain produced by the previous two bounding events (Van Arsdale, 2000). Van Arsdale (2000) also suggests that this sequence of multiple, temporally-clustered events may not be unusual for the NMFS. He cites evidence from subsurface analyses that suggests that these three faults may have identical displacement histories since the Late Cretaceous. Thus, he suggests that the paleoseismic history for the Reelfoot reverse fault can serve as a proxy for the other two faults. Trench exposures of the Reelfoot fault indicate that deformation occurs primarily as folding rather than faulting at the surface and that the structure has experienced at least three earthquakes in the past 2400 years at times consistent with those determined from regional paleoliquefaction studies (Kelson *et al*., 1996). This interpretation is supported by paleoliquefaction studies, which indicate that large magnitude earthquakes on the faults of the New Madrid system have occurred in clusters like those of 1811-1812 (e.g., Tuttle *et al*., 2002; 2005).

There is significant uncertainty regarding the exact identification and geometry of the faults that ruptured in the 1811-1812 and earlier earthquakes, and some models of rupture (e.g., EPRI/DOE/NRC, 2012; STNOC 2011; USNRC, 2006) include weighted alternative geometries for each of the three faults. We adopt the characterization of EPRI/DOE/NRC (2012; Table 3). We include two alternative geometries for the northern extent of the southern section, the Blytheville fault zone (NMS: BA-BL), weighted 0.4, and the Bootheel Lineament (NMS: BA-BFZ), weighted 0.6. For the central and northern sections, we include two alternatives: short and long (RFT-S, RFT-L, NMN-S, MNM-L). The short central section (RFT-S) includes only that part of the Reelfoot reverse fault that is defined by the Reelfoot scarp and extends from the Blytheville fault to the New Madrid North fault; the long alternative (RFT-L) extends both east and west, based on continued seismicity. The short alternative for the New Madrid north fault (NMN-S) is the fault as defined by Johnston and Schweig (1996); the long alternative (NMN-L) extends the source along northward continuations of seismicity identified by Wheeler (1997). Because the causative faults are not well understood, the dips are not well constrained. The northern and southern sections of the system are modeled as vertical. The Reelfoot fault is modeled with a 40-degree southwest dip.

The EPRI/DOE/NRC (2012) characterization also addresses the apparent clustering of activity along the NMFS faults using the approach of Toro and Silva (2001). The rate of earthquakes and geomorphic expression of faulting on the Reelfoot fault in the late Holocene suggests that the system is or has recently been in a cluster. However, geodetic data gathered over the last decade or so suggest that little or no interseismic deformation is occurring across the NMSZ, which some researchers have interpreted as evidence that the system is shutting down and entering an inter-cluster period of quiescence (e.g., Calais *et al*., 2005; Calais and Stein, 2009). The EPRI/DOE/NRC model strongly favors the interpretation that the system is currently in a cluster (0.9), based on the recent history of activity and the unlikelihood that we have just happened upon the exact moment the system is shutting down. However, they, and we, give some weight to two alternative models: 1) only the Reelfoot faultis currently in a cluster, and the other faults are quiescent (0.05), and 2) the entire system is out of a cluster (0.05) (Table 3). In the former case, the Reelfoot faultis active, but at a lower rate than the in-cluster case; in the latter case, no faults are active and the system defaults to the RR background zone characterization.

Several recent hazard analyses have developed source characterizations for the NMFS. The USGS National Seismic Hazard Maps (Petersen *et al*., 2008) compiled recent data to develop a model with lower weighted mean magnitudes for the faults than in previous models, and with a recurrence model reflecting possibly clustered timing of events. Their magnitudes range from **M** 7.3 to 8.0 for the southern and central sections, with a preferred magnitude of **M** 7.7 and weighted mean of **M** 7.6, and from **M** 7.1 to 7.8 for the northern section, with a preferred value of **M** 7.5 and weighted mean of **M** 7.4. Models developed for the Site Safety Analysis for Exelon Generation Company in Illinois (USNRC, 2006) include a lower magnitude distribution, with **M** 7.2 to 7.9 (weighted mean **M** 7.5), **M** 7.4 to 7.8 (weighted mean of **M** 7.6), and **M** 7.0 to 7.6 (weighted mean of **M** 7.3) for the southern, central, and northern faults, respectively. EPRI/DOE/NRC (2012) include distributions for the NMS, Reelfoot reverse fault, and NMN sections of the NMFS of **M** 6.7 to 7.9, **M** 7.1 to 7.8, and **M** 6.8 to 7.6, respectively. In our model, we adopt the EPRI/DOE/NRC distribution of maximum magnitudes. The preferred values and weighted means are similar to those developed in the nuclear studies described above.

4.2 EPRI GROUND MOTION PREDICTION MODELS

Several factors control the level and character of earthquake ground shaking. These factors are in general: (1) rupture dimensions, geometry, and orientation of the causative fault; (2) distance from the causative fault; (3) magnitude of the earthquake; (4) the rate of attenuation of the seismic waves along the propagation path from the source to site; and (5) site factors, including the effects of near-surface geology, particularly from soils and unconsolidated sediments. Other factors, which vary in their significance depending on specific conditions, include slip distribution along the fault, rupture process, footwall/hanging-wall effects, and the effects of crustal structure such as basin effects.

Several parameters may be used to characterize earthquake ground motions. The common parameters include: peak ground acceleration, velocity, and displacement; response spectral accelerations or velocities, duration, and time histories in acceleration, velocity, or displacement. In this analysis, we have estimated peak horizontal ground acceleration (PGA) and horizontal spectral accelerations (SA) at 0.04, 0.1, 0.2, 0.4, 1.0, and 2.0 sec.

Crustal ground motion prediction models for tectonically active regions like the western U.S. are empirical in nature and derived from strong motion data from such areas as California, Taiwan, Japan, and Italy. In contrast, few useable strong motion records exist for earthquakes in the Central and Eastern North America (CENA). Thus ground motion prediction models for the CENA have been developed, in large part, using seismological-based numerical models. During the past decade, ground motion models for the CENA have been derived using three different approaches: the stochastic method, the Green's function method, and the complex/empirical source method.

Recent efforts have been made to update the ground motion models for the CENA. One project is called the Next Generation of Attenuation (NGA) – East sponsored by Pacific Earthquake Engineering Research (PEER) Center. The objective of the project is to develop a new suite of ground motion prediction model for the CENA. The median ground motion models were just released but no standard deviations for the models were specified. There are 20 new NGA-East models and we expect it will be several months before the models become vetted.

In a second project, EPRI (2013) updated the 2004/2006 EPRI models in the near-term so that preliminary Ground Motion Response Spectra (GMRS) could be developed for existing nuclear power plant sites as required by the NRC's Recommendation 2.1 pending completion of the NGA East Project. The models were used in this study. The EPRI Ground-Motion Model (GMM) Review Project (EPRI, 2013), an enhanced SSHAC Level 2 assessment process, established a methodology to evaluate the existing 2004 EPRI GMM and determine if it should be updated. After reviewing the current literature and conducting interviews and convening a workshop with ground-motion experts and seismologists it was decided to update the 2004 GMM because (1) seven of the 13 developers of the 2004 EPRI GMM recommended that their models be replaced; (2) three new models have been developed for the CENA by ground-motion experts; (3) 80% of the earthquake records in a new ground-motion database provided by the NGA-East Project are from earthquakes that occurred after the development of the 2004 EPRI GMM; (4) comparisons to the updated CENA database indicate the 2004 EPRI GMM overpredicts ground motions at some magnitude-distance and structural frequency ranges that are important to nuclear power plant PSHA; and (5) the models used to develop the aleatory portion of the 2006 EPRI GMM have been superseded.

The 2013 EPRI GMM retains the structure of the 2004 EPRI GMM, grouping the candidate individual models into four clusters according to their seismological characteristics, weighting the models within each cluster according to their consistency with the data, representing each cluster by three fitted relationships (5th percentile, median, and 95th percentile), and assessing cluster weights based on consistency with observed data and seismological attributes of the models within each cluster. The GMM Review Project identified new candidate models for the updated GMM clusters, models and weights, as shown in Table 4 and a summary of the overall elements of the model are listed in Table 5.

For reference, the ground motion prediction models used by the USGS to develop the 2014 National Seismic Hazard Maps include Toro *et al.* (1997), Frankel *et al.* (1996), Silva *et al.* (2002), Atkinson and Boore (2006), Atkinson (2008), Campbell (2003), Tavakoli and Pezeshk (2005), Pezeshk *et al.* (2011), and Somerville *et al*. (2001). The versions of Atkinson and Boore (2006) and Atkinson (2008) in the EPRI study have been updated with Atkinson and Boore (2011). All the ground motion prediction models are for hard rock characterized by a V_S30 of 2,800 m/sec.

Comparisons indicate that the 2013 GMM is somewhat lower than 2004 EPRI GMM when the two models are taken as a whole, but these differences are moderate, given the broad uncertainty range spanned by both GMMs. The greater differences occur at low frequencies. For PGA the bulk of the curves are consistent between the two GMMs. In addition, there is a substantial overlap in the 10 to 200 km range indicating that the updated GMM does not represent a radical departure from the 2004 EPRI GMM. The observed differences are the result of possessing and using substantially more data and having acquired additional insights from other regions over a period of nearly 10 years.

The 2006 EPRI model for aleatory uncertainty (sigma) was based on preliminary NGA-West 1 models for sigma from active tectonic regions, adjusted to account for differences in properties of the earth's crust between active (western North America [WNA]) and stable tectonic regions (i.e., CENA) (EPRI, 2006). The EPRI GMM Review Project updated the model to incorporate the nearly final NGA-West 2 aleatory models, with the same adjustments for differences between WNA and CENA. The updated sigma model is frequency and magnitude dependent, with inter-event and intra-event components. There is additional aleatory variability for distances of R_{JB} <

20 km. The updated aleatory variability model has higher values of total sigma than the 2006 EPRI model for **M** 5 earthquakes, and lower values for **M** 6 and 7 earthquakes for motions at 2.5 Hz and higher. At 1 Hz, the values of sigma are comparable in the two models and at 0.5 Hz, the updated GMM has slightly higher sigma than the 2006 EPRI model.

Table 4

EPRI (2013) GMM Clusters and Models

Cluster	Model Types and Cluster Weights (repeated large-magnitude earthquake sources/area earthquake sources)	Models
1	Single-corner Brune source (0.15/0.185)	Silva *et al*. (2002) – SC-CS-Sat[1] Silva *et al*. (2002) – SC-VS[1] Toro *et al*. (1997) Frankel et al. (1996)
2	Complex/Empirical Source ~R^{-1} geometrical spreading (0.31/0.383)	Silva *et al*. (2002) – DC-Sat Atkinson (2008) with 2011 modifications (A08′)
3	Complex/Empirical Source ~$R^{-1.3}$ geometrical spreading (0.35/0.432)	Atkinson-Boore (2006) with 2011 modifications (AB06′) Pezeshk *et al*. (2011)
4	Finite-source /Green’s function (0.19/0)	Somerville *et al*. (2001); slightly different models for rifted and nonrifted (not used for distributed seismicity sources with large contribution from **M** < 6)

SC = single-corner; DC = double-corner; CS = constant stress; VS = variable stress; Sat = saturation.

[1] Treated as one model for calculation of weights.

Table 5

Elements of the CENA Ground Motion Models

Feature	Attribute
Ground Motion Measure	Peak ground acceleration Spectral acceleration at frequencies of 0.5, 1, 2.5, 5, 10, 25 Hz
Site Conditions	Hard rock (V_S 2.8 km/sec, 9200 ft/sec)
Regions	Midcontinent (includes east coast) Gulf Coast
Ground Motion Model Types	Four types included: • Single-corner Brune source • Complex/empirical source ~R^{-1} geometrical spreading • Complex/empirical source ~$R^{-1.3}$ geometrical spreading • Finite-source/Green's function
Aleatory Variability	Magnitude and frequency dependent Includes additional variability for distances of R_{JB} < 20 km

The hard rock PSHA results are presented below including comparisons with the National Seismic Hazard Maps.

5.1 PSHA RESULTS

The results of the PSHA are presented in terms of ground motion for hard rock site conditions as a function of annual frequency of exceedance (AFE). AFE is the reciprocal of the average return period. Figure 15 shows the mean, median (50th percentile), 5th, 15th, 85th, and 95th percentile hazard curves for PGA. These fractiles indicate the range of epistemic uncertainties about the mean hazard. The uncertainties are very large due to both the large uncertainties in the ground motion prediction models and the source parameters of the controlling seismic source. The 0.4 sec and 1.0 sec horizontal spectral acceleration (SA) hazard are shown in Figures 16 and 17. The 2,500 year return period mean PGA for hard rock is 0.35 g (Table 6).

The contributions of the various seismic sources to the mean PGA hazard are shown on Figure 18. The major contributors to the hazard at the site for a return period of 2,500 years are the IBEB zone and the Wabash Valley RLME. The distributed seismicity contributes just over 70 percent of the PGA hazard at 2,500-year return period with the Wabash Valley and New Madrid RLMEs contributing approximately 15 percent each (Figure 19). At longer periods (0.4 and 1.0 sec SA), the New Madrid RLME relative contribution increases to up to 75 percent of the hazard at 2,500 years (Figures 20 through 23).

By deaggregating the PGA, 0.4 and 1.0 sec SA hazard by magnitude, distance and epsilon bins, we can illustrate the contributions by events at a return period of 2,500 years (Figures 24 through 26). Epsilon is the difference between the logarithm of the ground motion amplitude and the mean logarithm of ground motion (for that M and R) measured in units of the standard deviation (σ) of the logarithm of the ground motion. As shown on Figure 24, a majority of the PGA hazard at the site is coming from nearby distributed seismicity of **M** 5.0 to 6.0 within 25 km and the Wabash Valley RLME (**M** 7.0 to 7.75 within 25 km). The 0.4 sec SA hazard is bimodal with significant contributions from nearby events from both distributed seismicity (**M** 5.0 to 6.0 within 25 km) and the Wabash Valley RLME (**M** 7.0 to 7.75 within 25 km) and from more distant events from the NMFS RLME (**M** 7.0 to 8.25 at 150 to 250 km) (Figure 25). At 1.0 sec SA, the hazard is dominated by the NMFS RLME (Figure 26).

The deaggregation shown in Figures 24 through 26 also provides the modal magnitude M*, modal distance D*, and modal epsilon ε*, which represent the largest contributor to the hazard at the defined return period. The M* and D* for the 2,500-year return period for PGA, 0.4 and 1.0 sec horizontal SA are listed in Table 7. Because the 0.4 sec hazard is bimodal (Figure 25), Table 7 lists the modes for both peaks.

A horizontal UHS on hard rock computed for the 2,500-year return period is shown on Figure 27. A UHS shows the hazard across all periods for the same annual exceedance probability or return period. The SA hazard has been calculated at 0.01 (PGA), 0.04, 0.1, 0.2, 0.4, 1.0 and 2.0 sec. These are the spectral periods specified in the EPRI (2013) ground motion models.

To obtain a smooth spectrum at very short and longer periods, interpolation and extrapolation were required. For periods between PGA and 0.04 sec, linear or log-linear interpolation of the ground motions defined at those frequencies is not ideal. More recent ground motion models indicate that the UHS in the CEUS peak in this period range. The spectral accelerations in this range were determined using the shape predicted by recent ground motion models for the modal magnitude and distances controlling the UHS at 0.04 sec. The median acceleration response spectra were computed for the controlling M and D using the Silva *et al*. (2002) and Pezeshk *et al*. (2011) ground motion models. Each of these spectra were then scaled to their respective 0.04 sec SA to compute scale factors (ratios of 0.02 sec SA to 0.04 sec SA and 0.03 sec SA to 0.04 sec SA). The scale factors from the two ground motion models were then weighted equally. The weighted mean scale factors were then applied to the 0.04 sec value from the UHS to obtain the 0.02 and 0.03 sec SA values.

Similarly, the 3.0, 4.0, 5.0 7.5, and 10.0 sec SA values were computed by using the long-period spectral shape predicted by available CEUS ground motion models that are defined at these long periods. The Silva *et al*. (2002) and Pezeshk *et al.* (2011) ground motion models were equally weighted. Scale factors were computed relative to the 2.0 sec SA using the controlling M and D for the 2.0 sec hazard.

Given the large depth to hard rock at the site, ground motions consistent with firm rock (V_S of 760 m/sec) were requested for input into finite element deformation analyses. The hard rock UHS was adjusted to firm rock using the generic amplification factors developed by David Boore (Frankel *et al.*, 1996). These factors are used in the development of the National Seismic Hazard Maps (NSHMs) by the USGS. They are not site-specific and therefore are highly uncertain, but are probably adequate in lieu of performing a site response analysis. Figure 28 shows the firm rock 2,500-year UHS. The mean firm rock PGA is 0.53 g (Table 8).

5.2 COMPARISON WITH USGS NATIONAL HAZARD MAPS

In 1996, the USGS released a “landmark” set of NSHMs for earthquake ground shaking, which was a significant improvement from previous maps they had developed (Frankel *et al.*, 1996). These maps were the result of the most comprehensive analyses of seismic sources and ground motion prediction ever undertaken on a national scale. The maps are the basis for the NEHRP Maximum Considered Earthquake (MCE_R) maps, which are used in the International Building Code. The maps are for NEHRP site class B/C (firm rock) (V_S30 760 m/sec).

For a 2,500-year return period, the 2014 NSHMs indicate firm rock (site class B/C) PGA, 0.2 sec SA and 1.0 sec SA values of 0.33, 0.57, and 0.17 g, respectively (USGS website). The site-specific firm rock values of 0.53, 0.68, and 0.14 g for PGA, 0.2 and 1.0 sec SA. The site-specific values are higher at short periods and slightly lower at long periods. These differences are likely due to the differences in the seismic source model and/or the ground motion prediction models. Note that the EPRI (2013) ground motion models were not available at the time the 2014 USGS NSHMs began development. As noted in the documentation of these maps, the EPRI (2013) suite of ground motion models and weights produce higher short-period and lower long-period ground motions than the suite of models implemented in the 2014 USGS NSHM (Petersen *et al.*, 2014). Also the 2014 NSHMs simplified the EPRI/DOE/NRC (2012) CEUS-SSC model for use in their PSHA and weighted this model in addition to the previous USGS model for Wabash Valley and New Madrid RLMEs.

Table 6

2,500-Year Return Period UHS for Hard Rock

Period (sec)	SA (g)
0.01	0.35
0.04	0.73
0.10	0.58
0.20	0.39
0.40	0.24
1.00	0.10
2.00	0.058

Table 7

Modal M* and D* at 2,500-year Return Period

	M*	D*
PGA	5.1	12.5 km
0.4 Sec SA (bimodal)	7.1 7.6	12.5 km 238 km
1.0 Sec SA	7.6	238 km

Table 8

2,500-Year Return Period UHS for Firm Rock (V_S of 760 m/sec)

Period (sec)	SA (g)
0.01	0.53
0.02	0.96
0.03	1.16
0.04	1.21
0.10	1.02
0.20	0.68
0.40	0.40
1.0	0.14
2.0	0.070
3.0	0.041
4.0	0.028
5.0	0.021

Four sets of two-component time histories were spectrally-matched to the firm rock 2,500-year UHS. At short periods, the 2,500-year hazard is from large events from the Wabash Valley RLME (**M** 7.0 to 7.75) and from moderate events (**M** 5.0 to 6.0) return period both within 25 km (Figure 24). At longer periods (0.4 and 1.0 sec), the hazard is bimodal with contribution from large events from the Wabash Valley RLME (**M** 7.0 to 7.75 within 25 km) and from large events of the New Madrid RLME (**M** 7.25 to 8.25 at 150 to 250 km) (Figures 25 and 26). Hence, two sets of seed time histories were selected consistent with a **M** 7.0 to 7.5 event within 25 km and two sets of seed time histories consistent with a larger, distant event (Table 9).

Because the response spectrum of a time history has peaks and valleys that deviate from the design response spectrum (target spectrum), it is necessary to modify the motion to improve its response spectrum compatibility. The procedure proposed by Lilhanand and Tseng (1988), as modified by Al Atik and Abrahamson (2010) and contained in the computer code RSPMatch09 (Fouad and Rathje, 2012), was used to develop the acceleration time histories through spectral matching to the target (seed) spectrum. This time-domain procedure has been shown to be superior to previous frequency-domain approaches because the adjustments to the time history are only done at the time at which the spectral response occurs resulting in only localized perturbations on both the time history and the spectra (Lilhanand and Tseng, 1988).

To match the design (target) spectrum, seed time histories should be from events of similar magnitude and distance (for duration) and most importantly, spectral shape as the earthquake dominating the spectrum. Figure 29 shows the spectra from the seed time histories scaled to the target spectrum at PGA. The spectral shapes of the seed time histories peak at about 0.1 sec typical of earthquakes in tectonically active regions compared to the 0.4 sec peak in the 2,500-Year UHS. The lack of strong motion records in stable continental interiors such as CEUS necessitates use of records from active regions.

The seed acceleration time history series are shown on Figures 30 to 33. The spectral matches and resulting time histories are shown on Figures 34 to 49. Arias intensities and durations of the spectrally-matched time histories are provided in Table 10. There are currently no predictive models available for the CEUS for Arias intensity or 5-95% duration.

Table 9

Seed Time Histories

Record Sequence Number	Year	Earthquake Name	Station Name	Earthquake Magnitude (M)	ClstD (km)	V_S30 (m/sec)	Comp	PGA(g)	PGV (cm/sec)	PGD (cm)	5-95% AI (m/sec)	5-95% Dur (sec)
1404	1999	Chi-Chi, Taiwan	PNG	7.6	110	466	E	0.03	1.5	0.47	0.027	31.99
							N	0.03	2.3	0.66	0.030	28.10
2112	2002	Denali, Alaska	TAPS Pump Station #8	7.9	105	425	049°	0.07	10.0	7.13	0.245	75.93
							319°	0.09	14.6	11.12	0.337	73.40
5804	2008	Iwate	Yamauchi Tsuchibuchi Yokote	6.9	28	562	E	0.26	10.5	7.76	0.648	9.18
							N	0.29	17.1	6.97	0.874	9.94
6928	2010	Darfield, NZ	LPCC	7.0	26	650	080°	0.24	17.7	3.82	0.613	12.91
							170°	0.36	30.3	21.27	0.618	11.37

ClstD	Closest distance
Comp	Component
PGV	Peak horizontal ground velocity
PGD	Peak horizontal ground displacement
AI	Arias intensity
Dur	Duration

Table 10

Spectrally-Matched Time Histories

Record Sequence Number	Year	Earthquake Name	Station Name	Earthquake Magnitude (M)	ClstD (km)	V_S30 (m/sec)	Comp	PGA(g)	PGV (cm/sec)	PGD (cm)	5-95% AI (m/sec)	5-95% Dur (sec)
1404	1999	Chi-Chi, Taiwan	PNG	7.6	110	466	E	0.54	13.9	3.3	4.69	35.3
							N	0.54	12.5	3.6	3.82	31.7
2112	2002	Denali, Alaska	TAPS Pump Station #8	7.9	105	425	049°	0.55	13.4	6.1	2.76	39.4
							319°	0.52	15.5	8.2	4.16	41.4
5804	2008	Iwate	Yamauchi Tsuchibuchi Yokote	6.9	28	562	E	0.55	19.0	9.7	1.79	10.2
							N	0.54	13.9	5.5	1.70	12.3
6928	2010	Darfield, NZ	LPCC	7.0	26	650	080°	0.53	18.8	9.8	1.80	17.1
							170°	0.53	20.4	8.3	1.07	12.6

ClstD	Closest distance
Comp	Component
PGV	Peak horizontal ground velocity
PGD	Peak horizontal ground displacement
AI	Arias intensity
Dur	Duration

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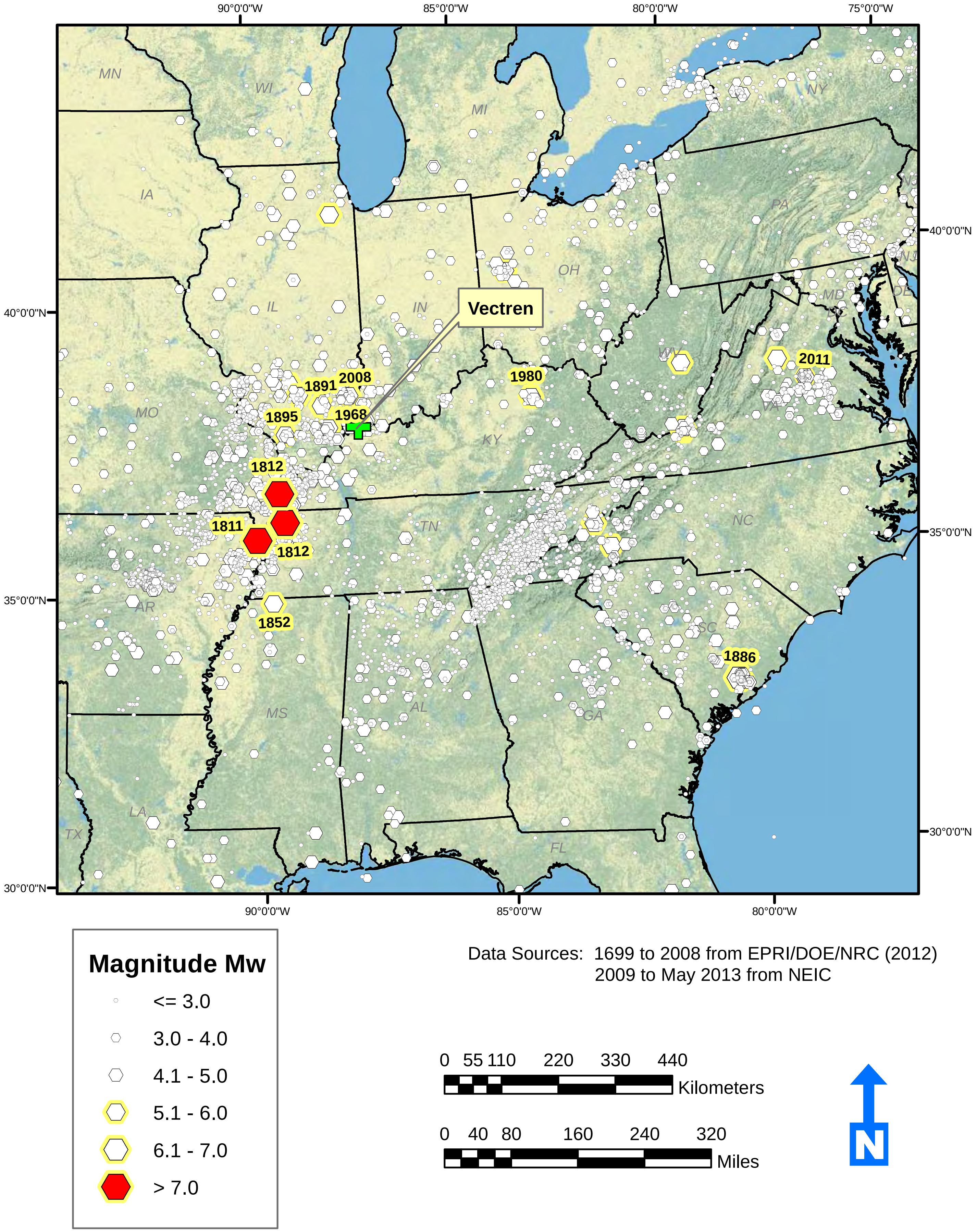

Data Sources: 1699 to 2008 from EPRI/DOE/NRC (2012)
2009 to May 2013 from NEIC

AECOM

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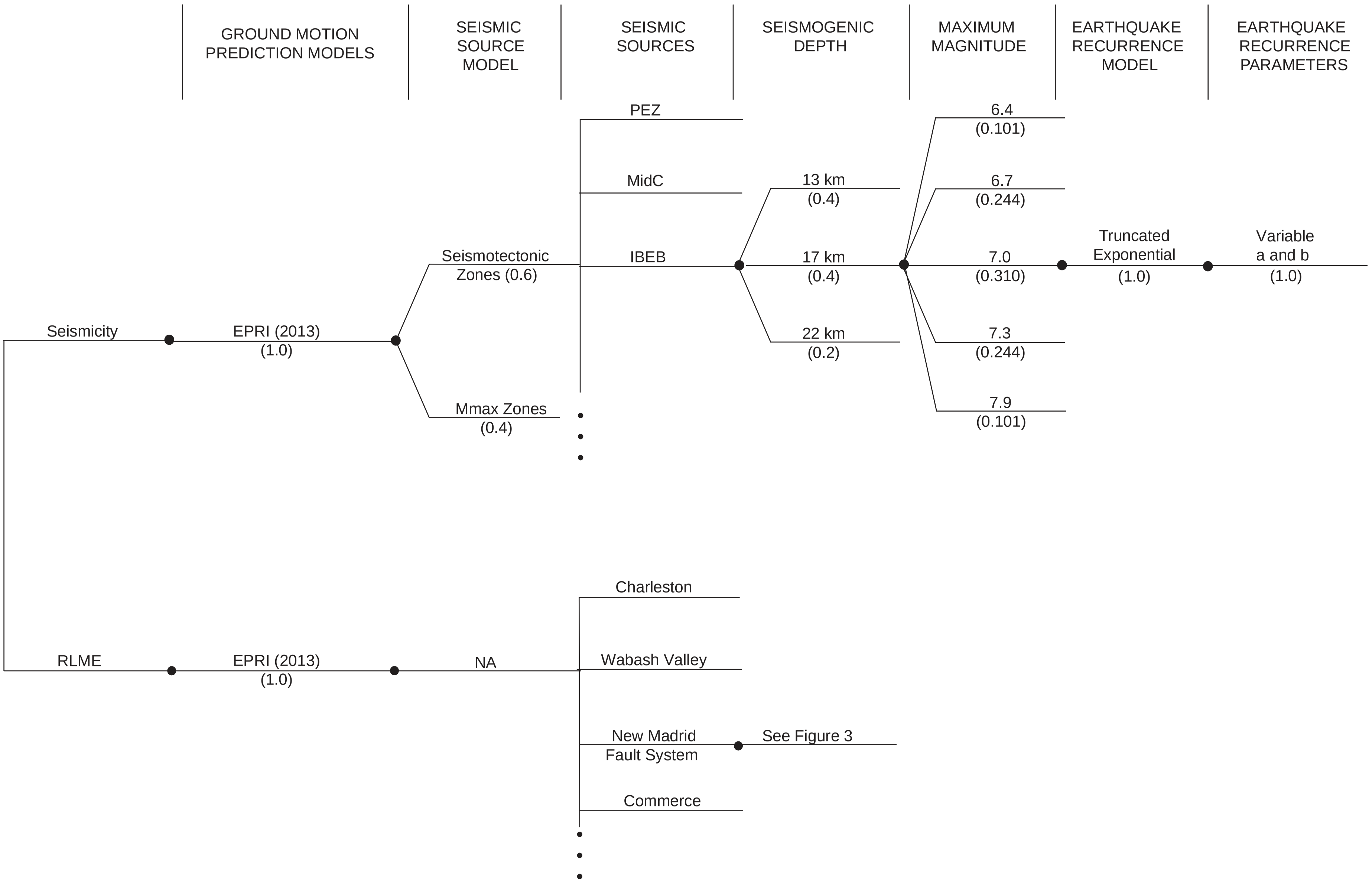
Generating Station
Vectren Corporation

HISTORICAL SEISMICITY OF CENTRAL
AND EASTERN UNITED STATES
(1699 - 2013)

Figure

1



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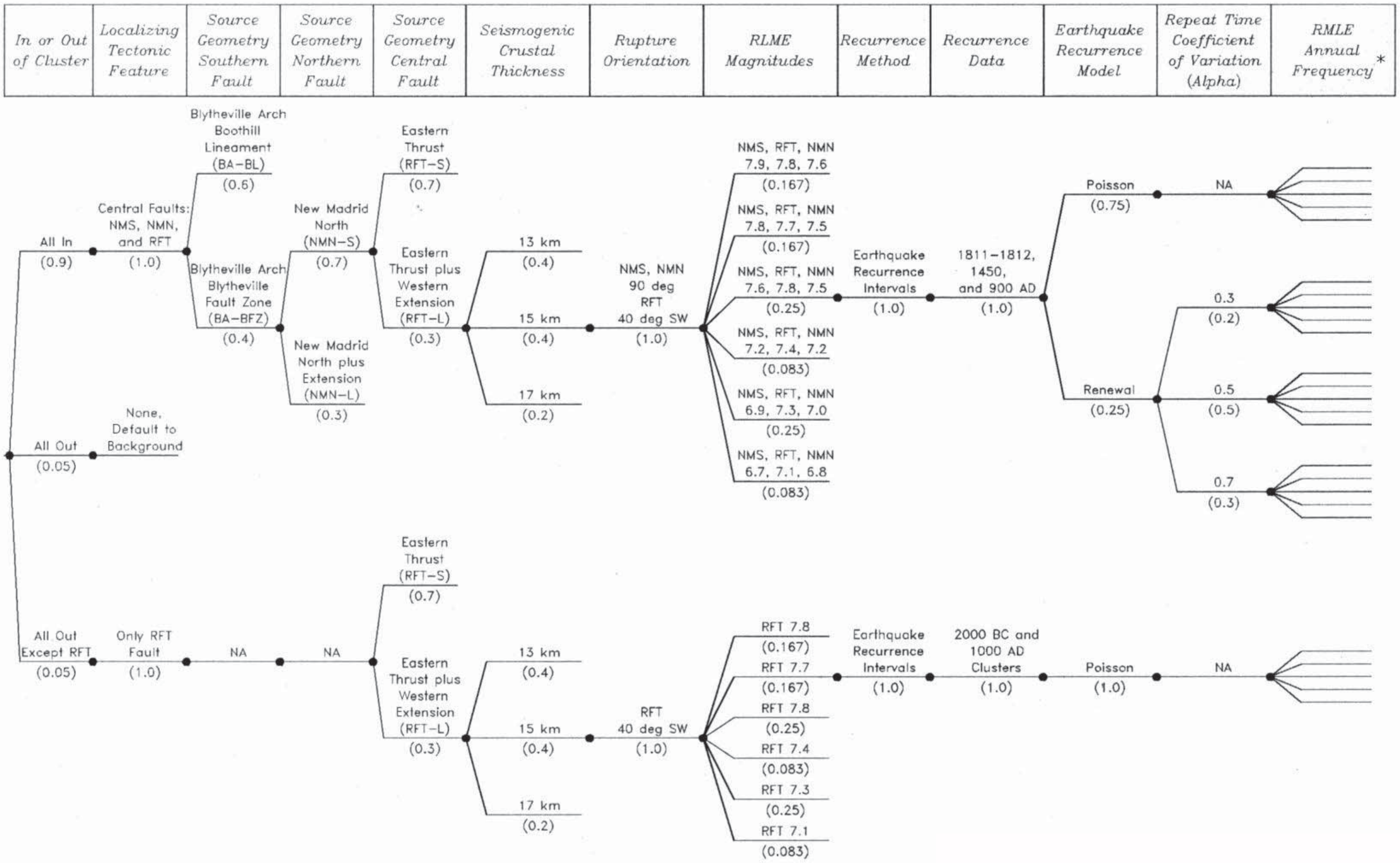
EXAMPLE SEISMIC HAZARD MODEL LOGIC TREE

Figure 2



* See EPRI/DOE/NRC (2012)

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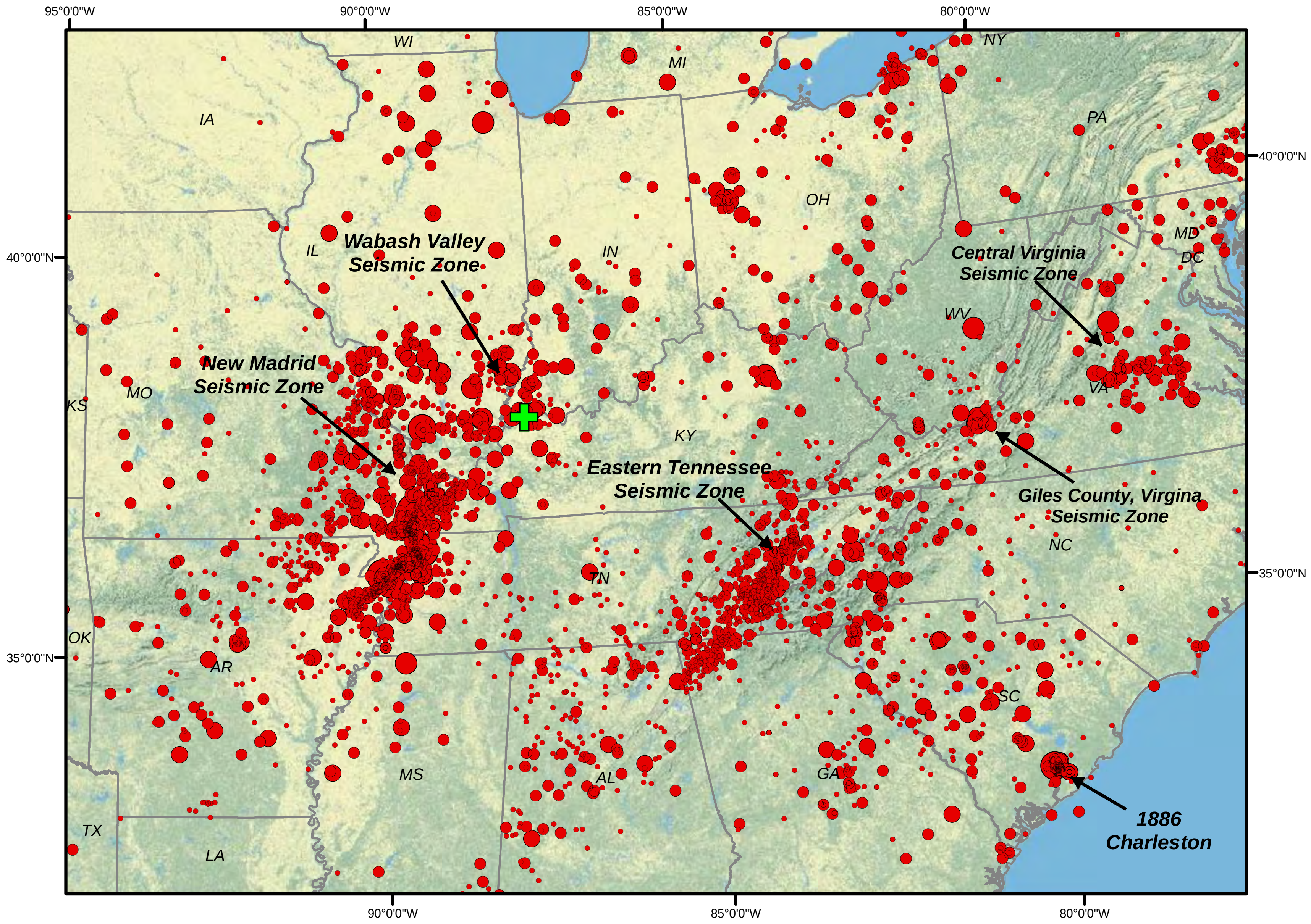
NEW MADRID RLME LOGIC TREE

Figure
3



✚ Project Site

● Earthquake Epicenters

Seismicity from:
EPRI/DOE/NRC (2012)

Project No. 60442676

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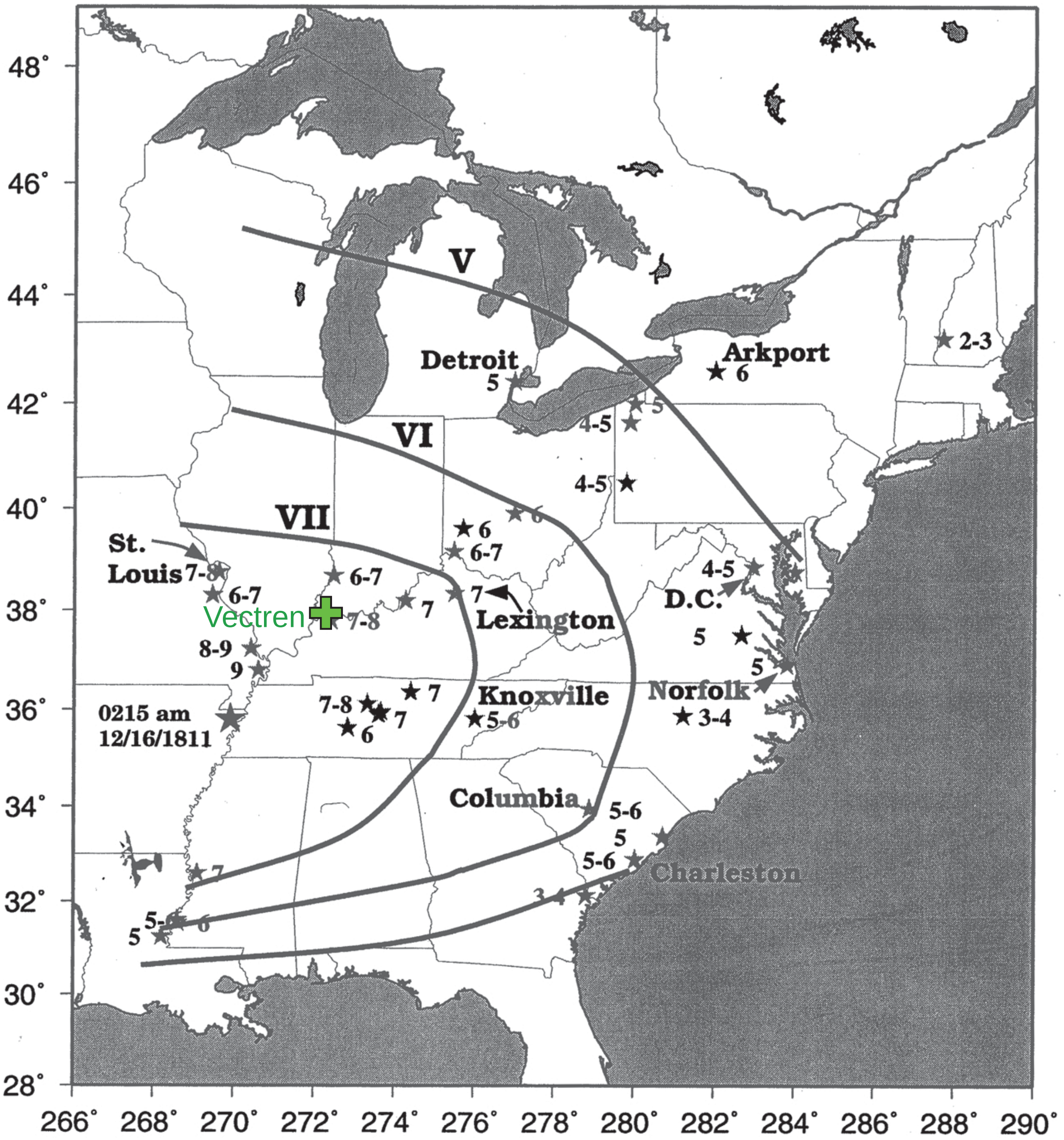
HISTORICAL SEISMICITY AND SEISMIC ZONES
IN THE CENTRAL AND EASTERN U.S.

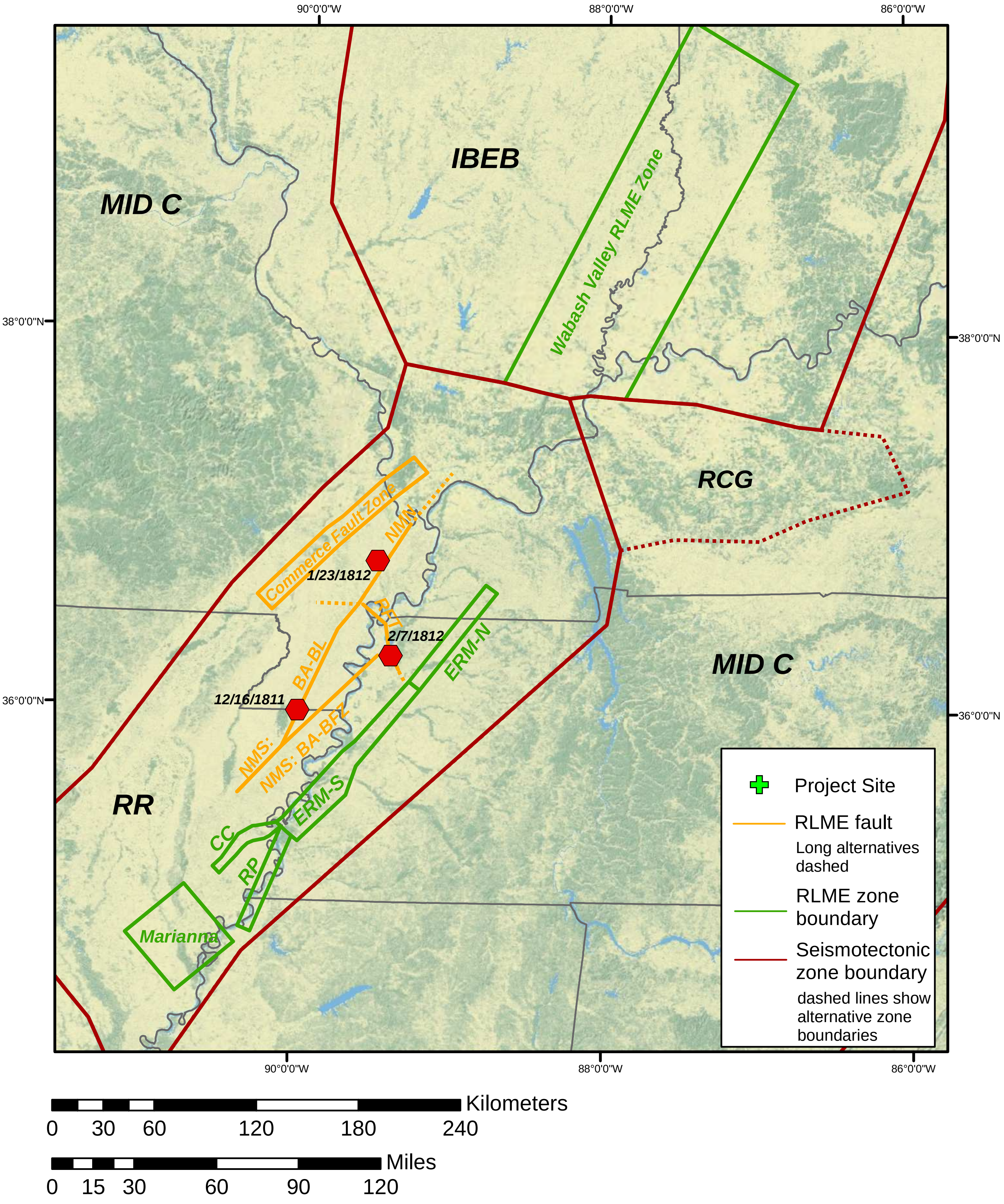
Figure

4



Source: Hough *et al*. (2000)

	Project No. 60442676	ISOSEISMAL MAP OF THE 16 DECEMBER 1811 **M** 7.2-7.3 NEW MADRID EARTHQUAKE	Figure 5
AECOM	A.B. Brown Generating Station Vectren Corporation		



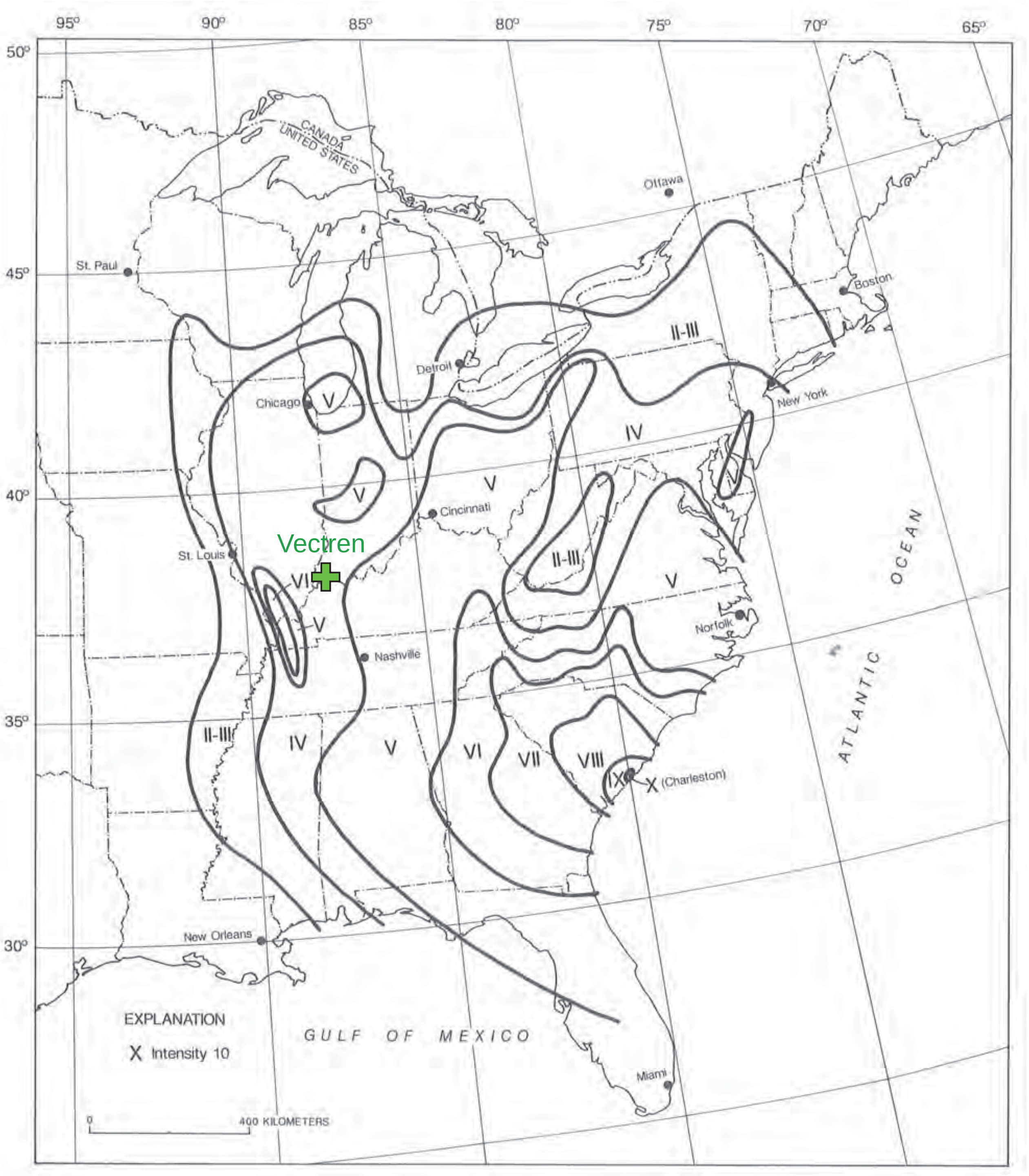
Project No. 60442676

A.B. Brown
Generating Station
Vectren Corporation

NEW MADRID FAULT SYSTEM,
1811-1812 NMFS EARTHQUAKES,
AND NEIGHBORING RLMEs

Figure

6



Source: Stover and Coffman (1993)

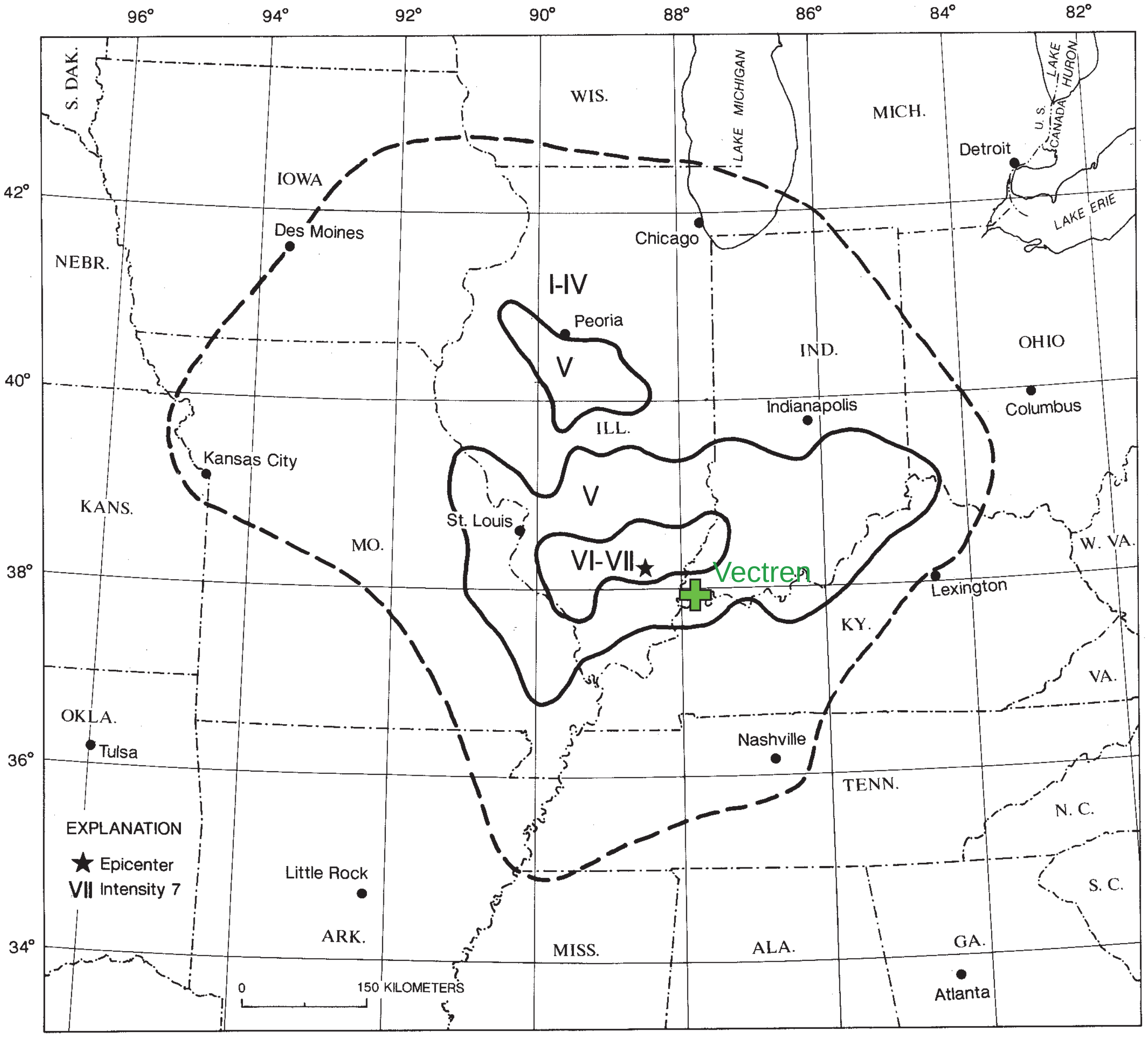
AECOM

Project No. 60442676

A.B. Brown
Generating Station
Vectren Corporation

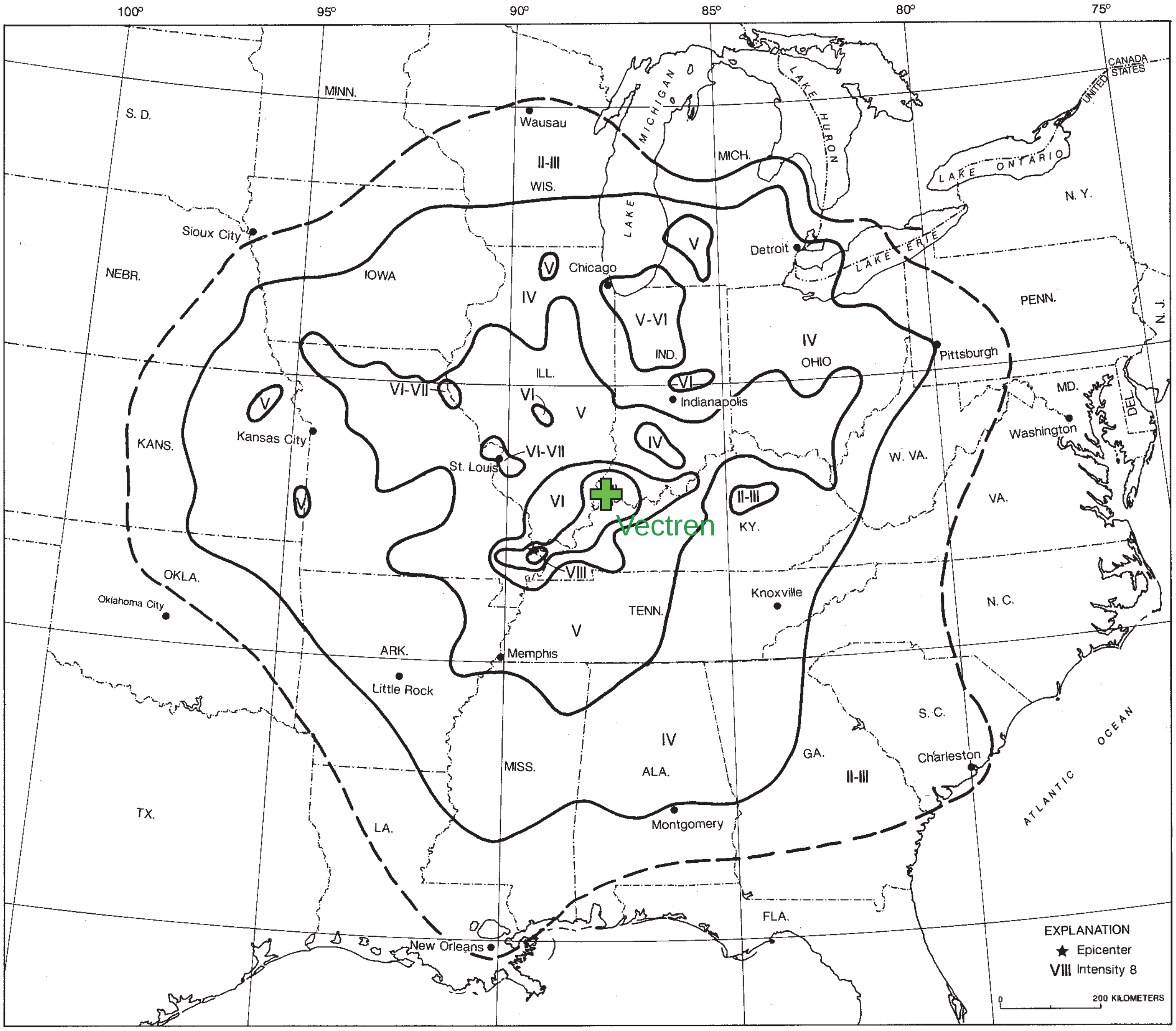
ISOSEISMAL MAP FOR THE 1 SEPTEMBER 1886 **M**~7 CHARLESTON EARTHQUAKE

Figure
7



Source: Stover and Coffman (1993)

AECOM	Project No. 60442676	ISOSEISMAL MAP OF THE 27 SEPTEMBER 1891 m_b 5.8 SOUTHERN ILLINOIS EARTHQUAKE	Figure 8
	A.B. Brown Generating Station Vectren Corporation		



Source: Stover and Coffman (1993)

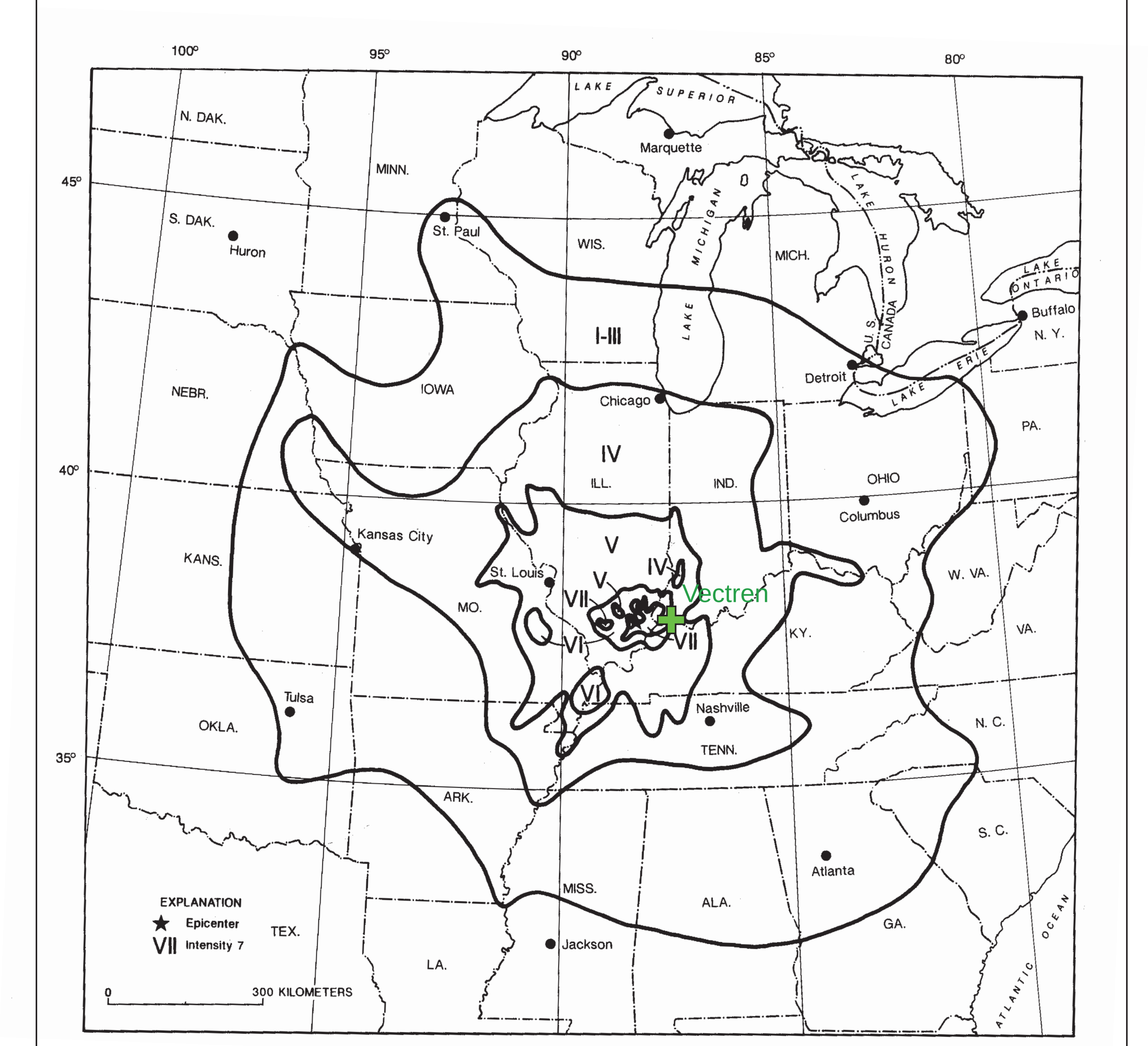
AECOM

Project No. 60442676

A.B. Brown
Generating Station
Vectren Corporation

ISOSEISMAL MAP OF THE
31 OCTOBER 1895 M_S 6.7
CHARLESTON, MISSOURI EARTHQUAKE

Figure
9



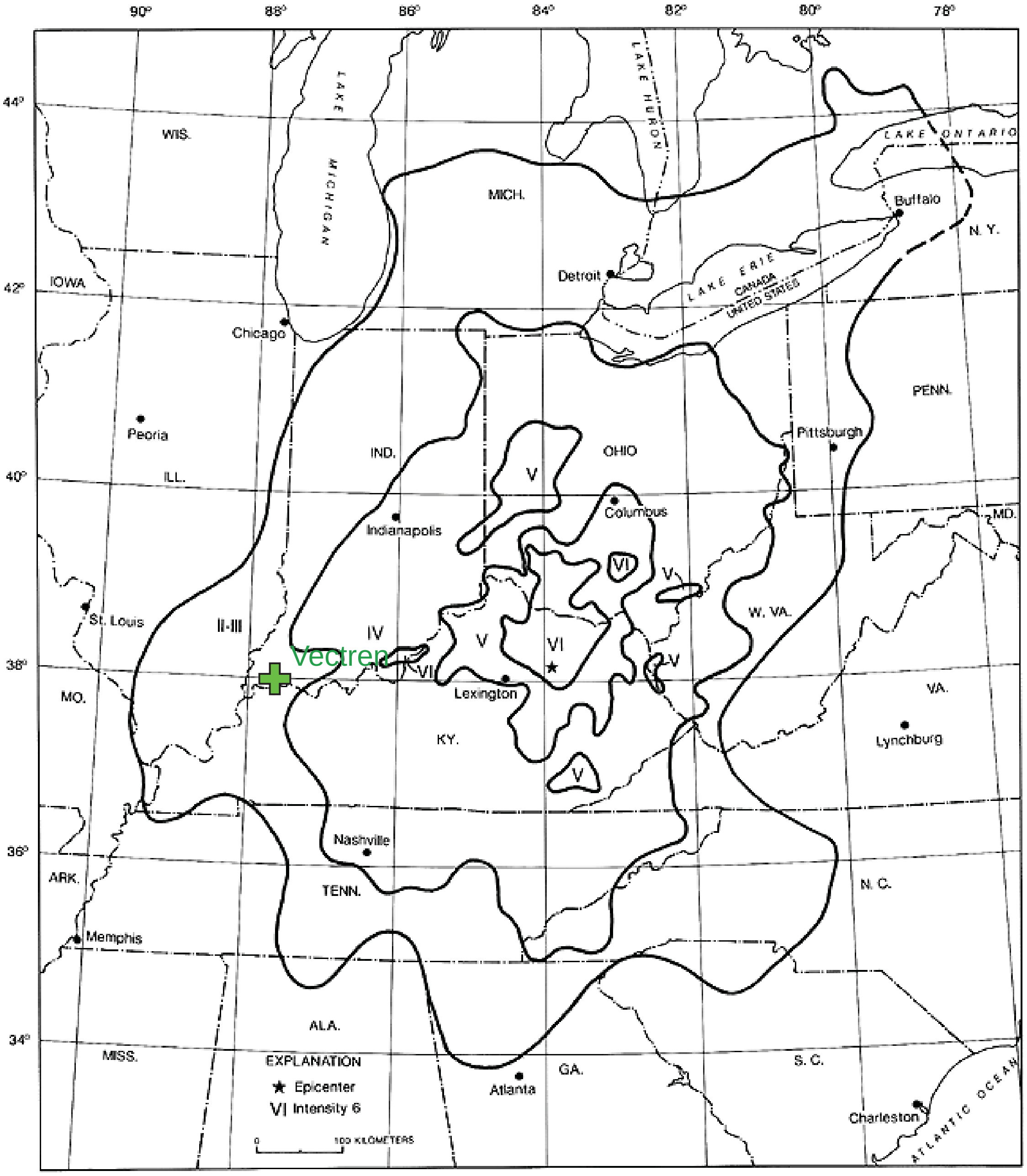
Source: Stover and Coffman (1993)

Project No. 60442676

A.B. Brown
Generating Station
Vectren Corporation

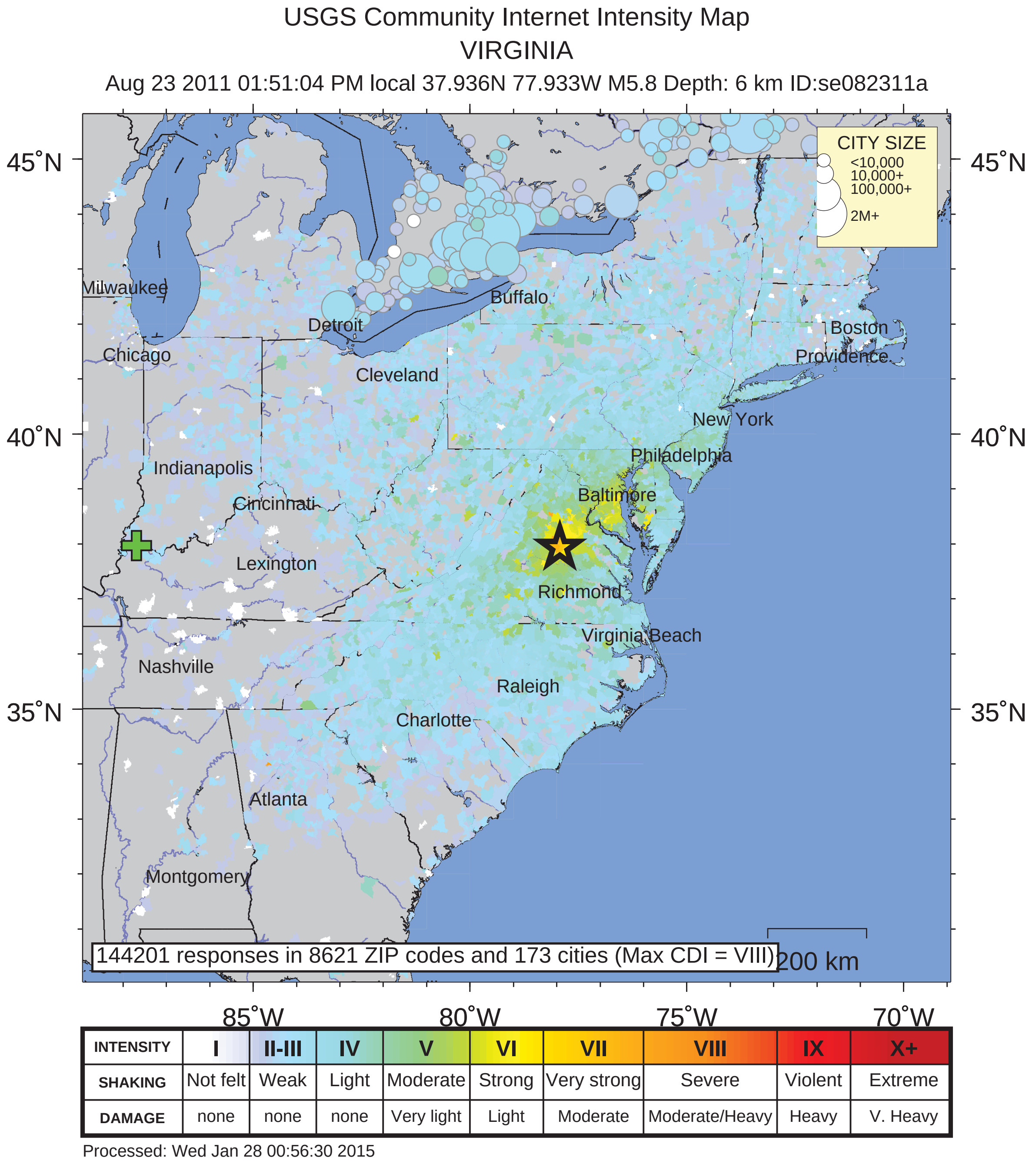
ISOSEISMAL MAP OF THE
9 NOVEMBER 1968 m_b 5.5
SOUTHERN ILLINOIS EARTHQUAKE

Figure
10



Source: Stover and Coffman (1993)

AECOM	Project No. 60442676	ISOSEISMAL MAP FOR THE 27 JULY 1980 M 5.1 SHARPSBURG, KENTUCKY EARTHQUAKE	Figure 11
	A.B. Brown Generating Station Vectren Corporation		



INTENSITY	I	II-III	IV	V	VI	VII	VIII	IX	X+
SHAKING	Not felt	Weak	Light	Moderate	Strong	Very strong	Severe	Violent	Extreme
DAMAGE	none	none	none	Very light	Light	Moderate	Moderate/Heavy	Heavy	V. Heavy

Processed: Wed Jan 28 00:56:30 2015

Source: http://earthquake.usgs.gov/earthquakes/dyfi/events/se/082311a/us/index.html

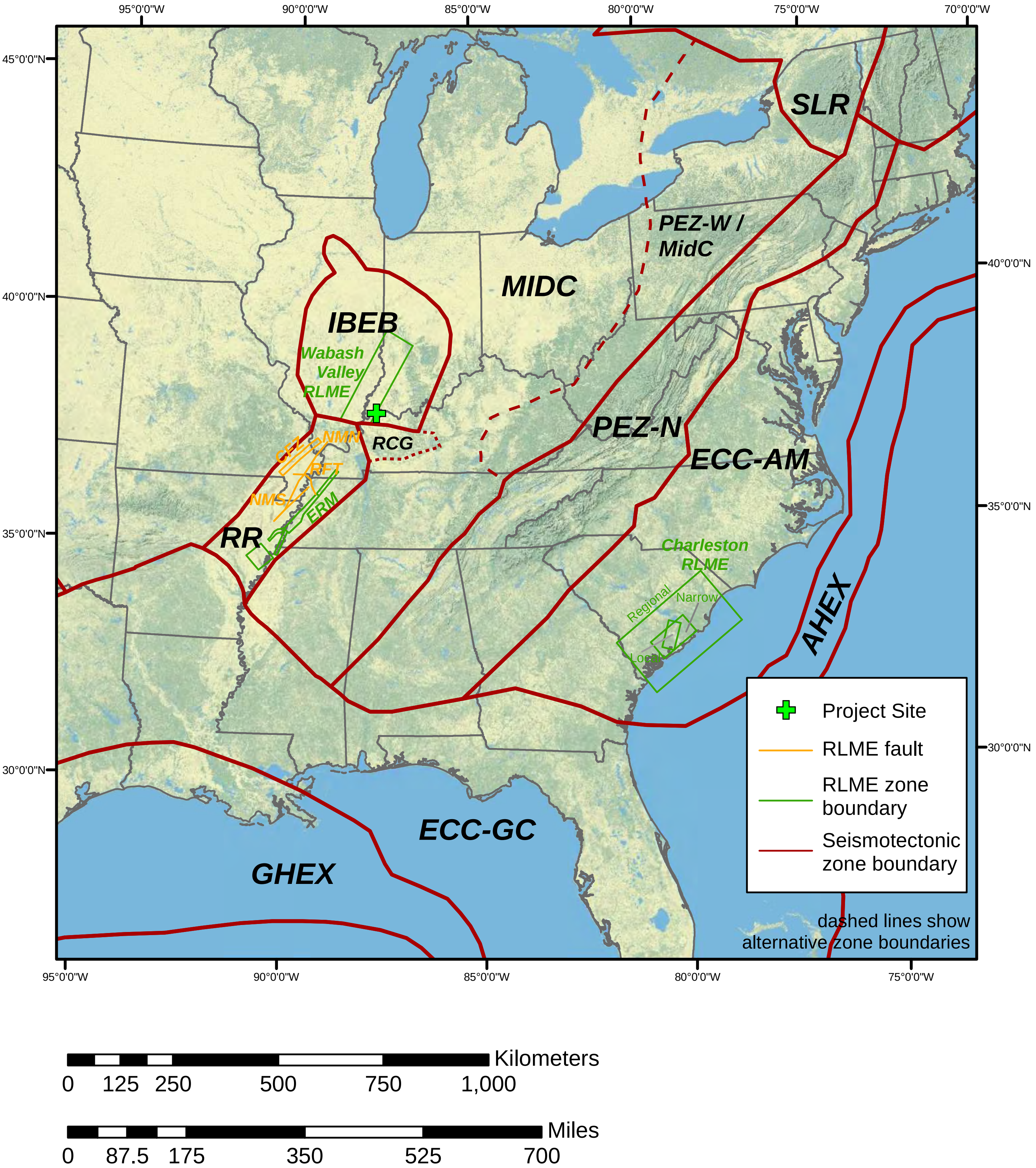
AECOM

Project No. 60442676

A.B. Brown
Generating Station
Vectren Corporation

DYFI MAP
FOR THE 23 AUGUST 2011
M5.8 MINERAL, VIRGINA EARTHQUAKE

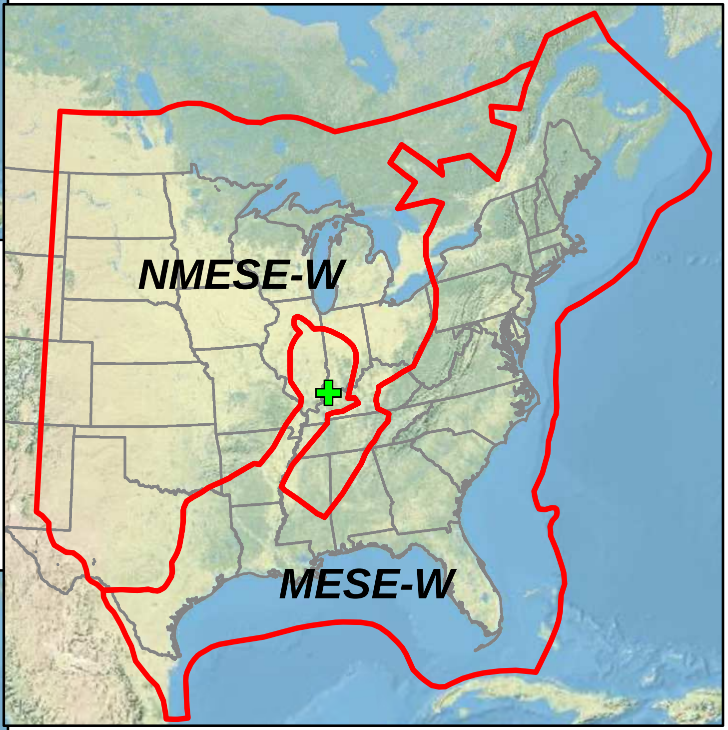
Figure
12

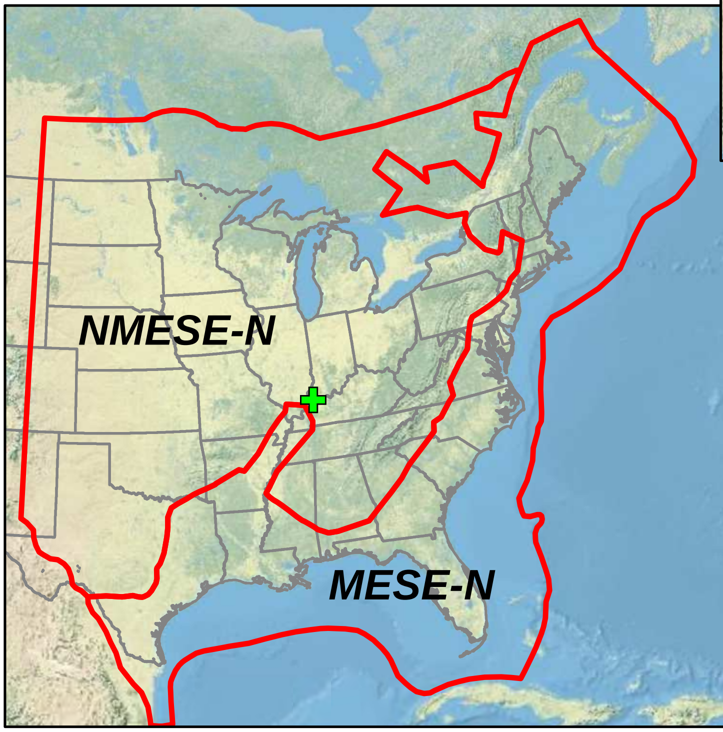


Project No. 60442676 A.B. Brown Generating Station Vectren Corporation	SEISMOTECTONIC ZONES AND RLMES	Figure 13

1-Zone Model

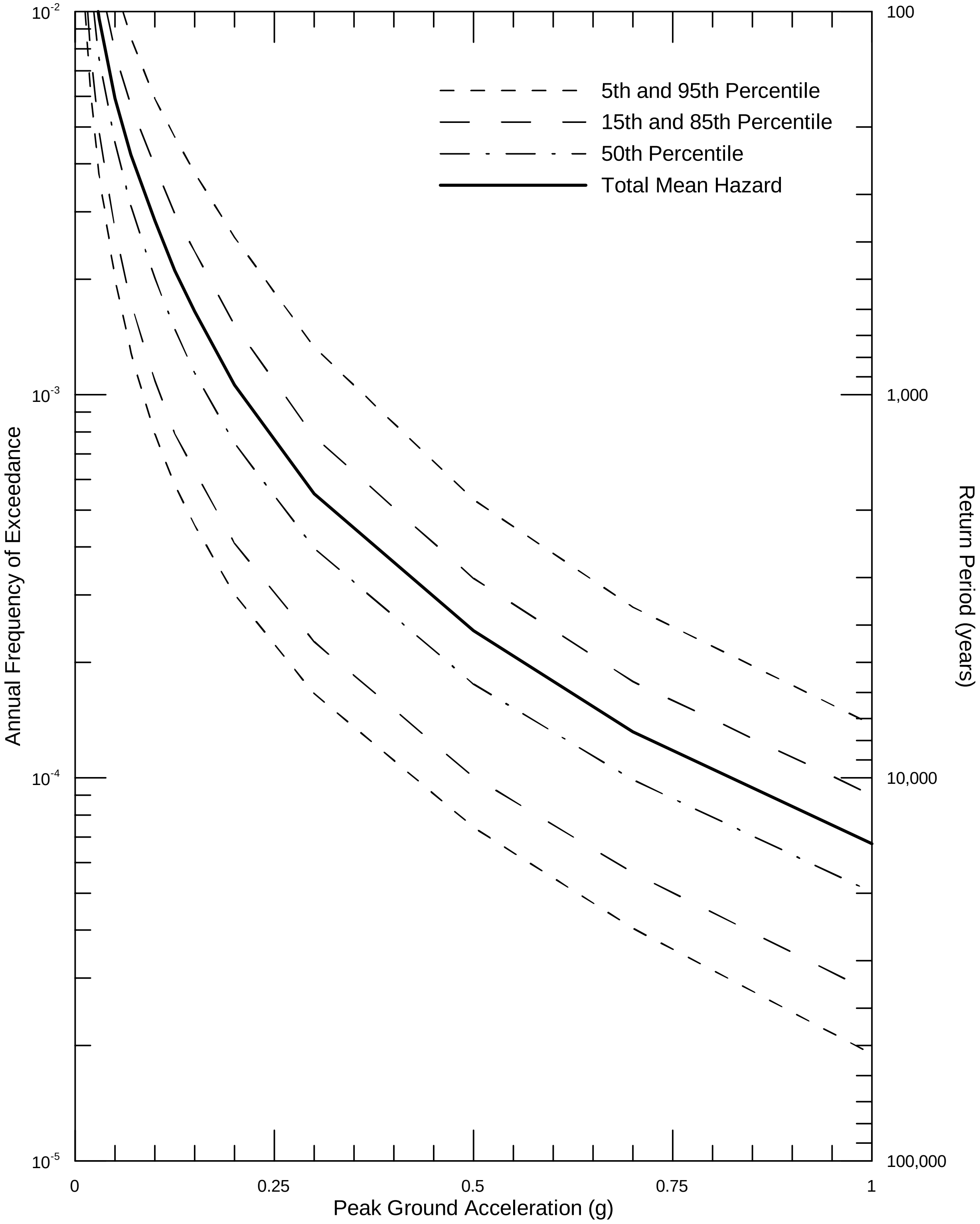
2-Zone Model Wide



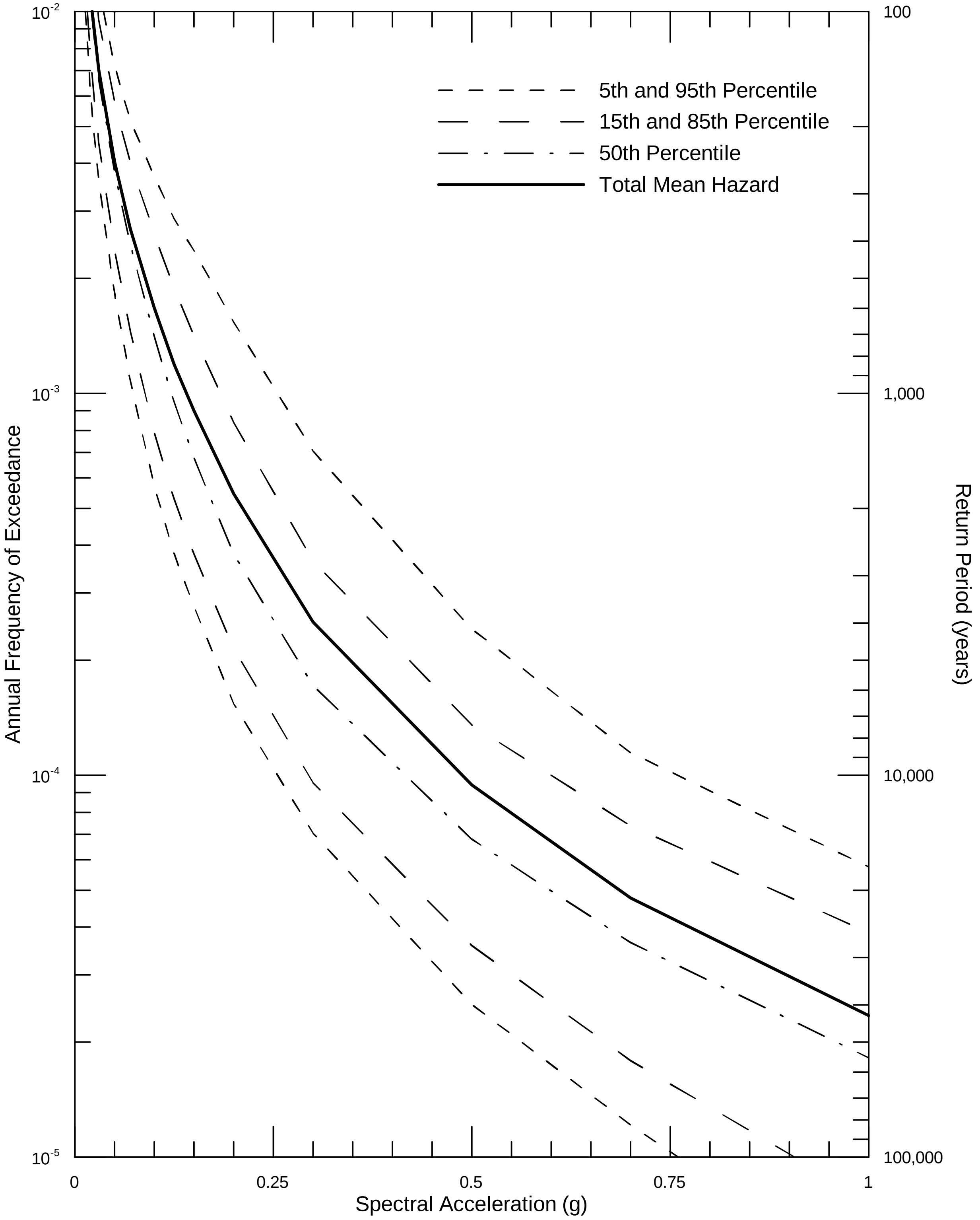


2-Zone Model Narrow

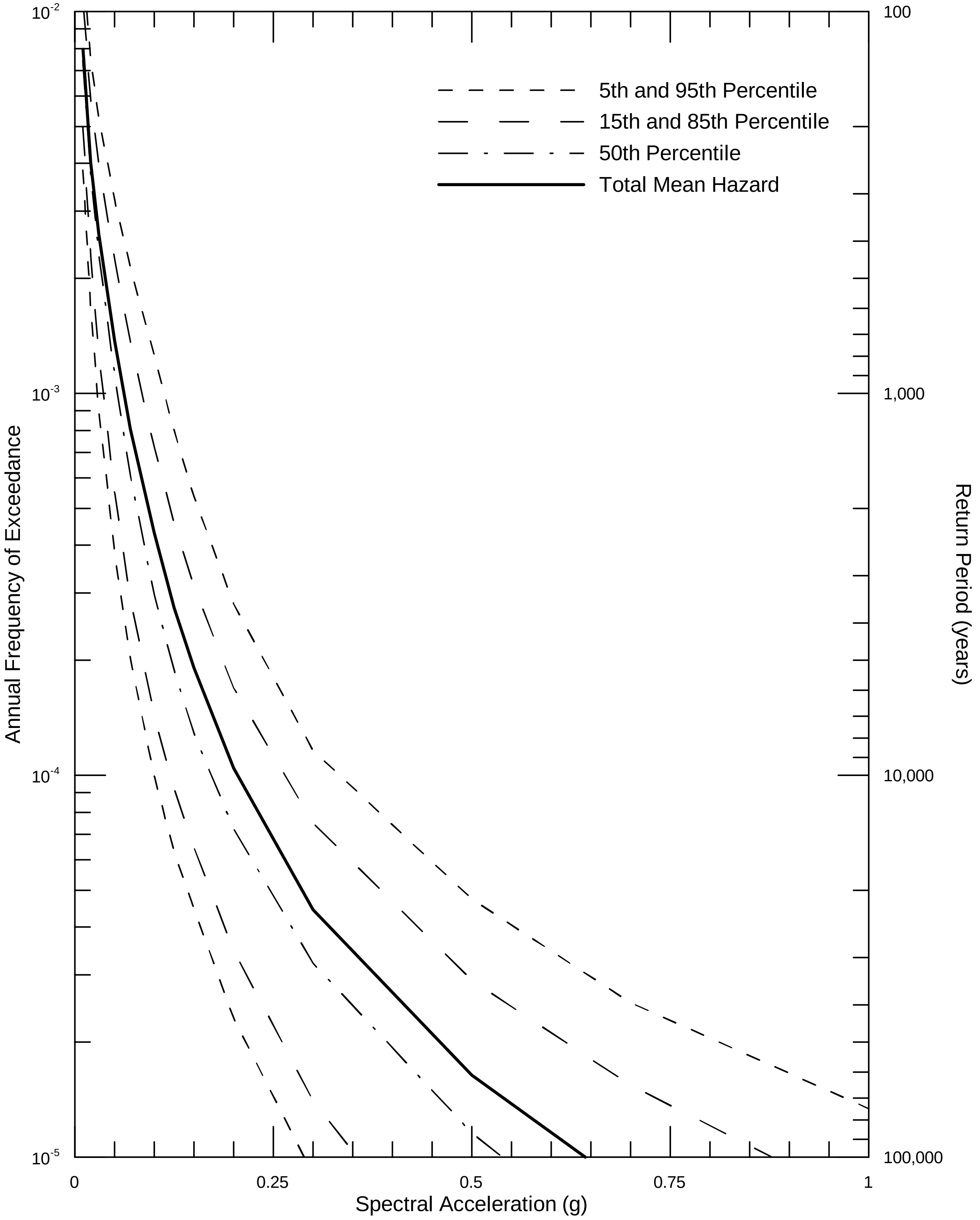
Project No. 60442676	Mmax ZONES	Figure 14
A.B. Brown Generating Station Vectren Corporation		



AECOM	Project No. 60442676	SEISMIC HAZARD CURVES FOR PEAK HORIZONTAL ACCELERATION ON HARD ROCK	Figure 15
	A.B. Brown Generating Station Vectren Corporation		



AECOM	Project No. 60442676 A.B. Brown Generating Station Vectren Corporation	SEISMIC HAZARD CURVES FOR 0.4 SEC HORIZONTAL SPECTRAL ACCELERATION ON HARD ROCK	Figure 16



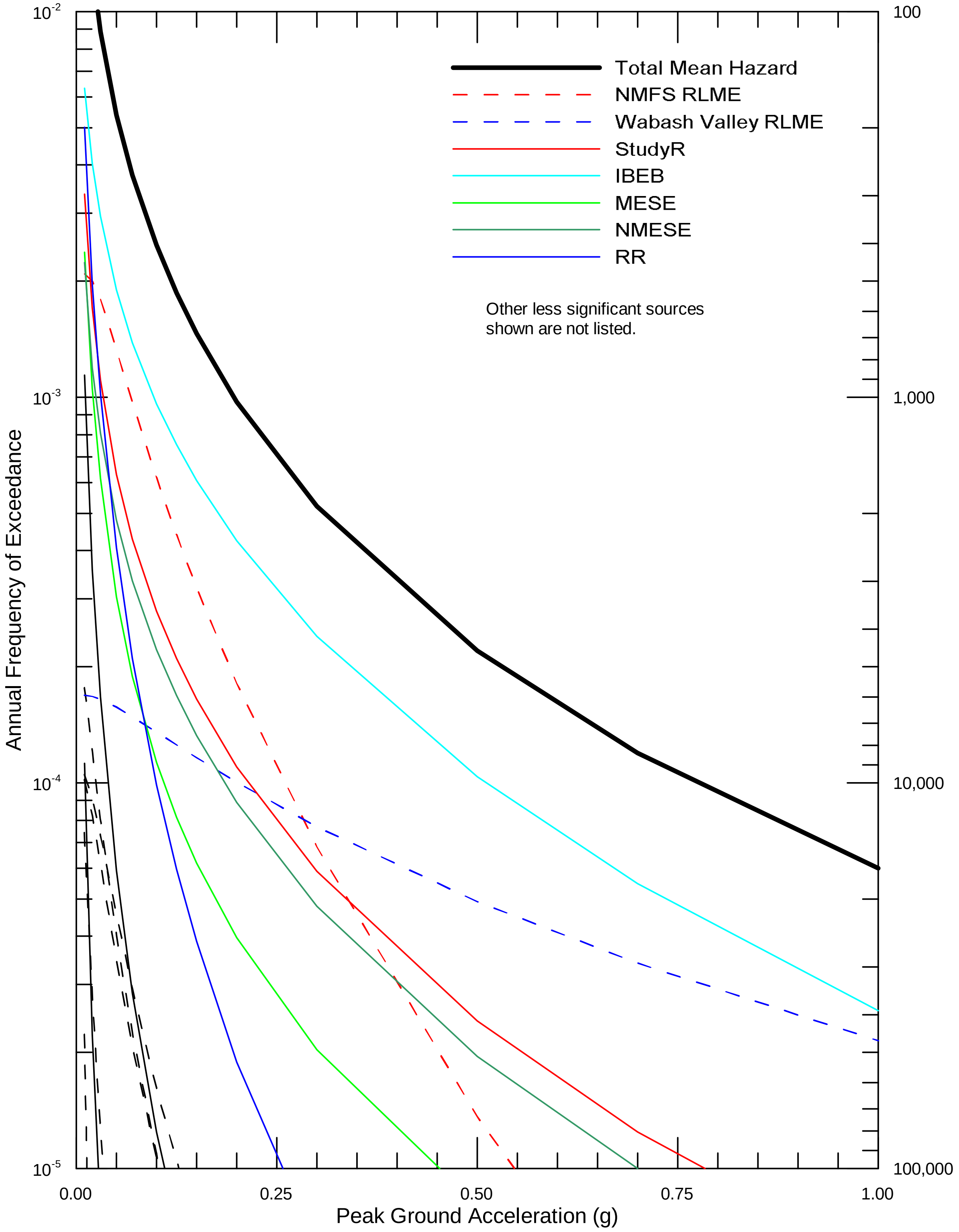
AECOM

Project No. 60442676

A.B. Brown
Generating Station
Vectren Corporation

SEISMIC HAZARD CURVES FOR 1.0 SEC
HORIZONTAL SPECTRAL ACCELERATION
ON HARD ROCK

Figure
17



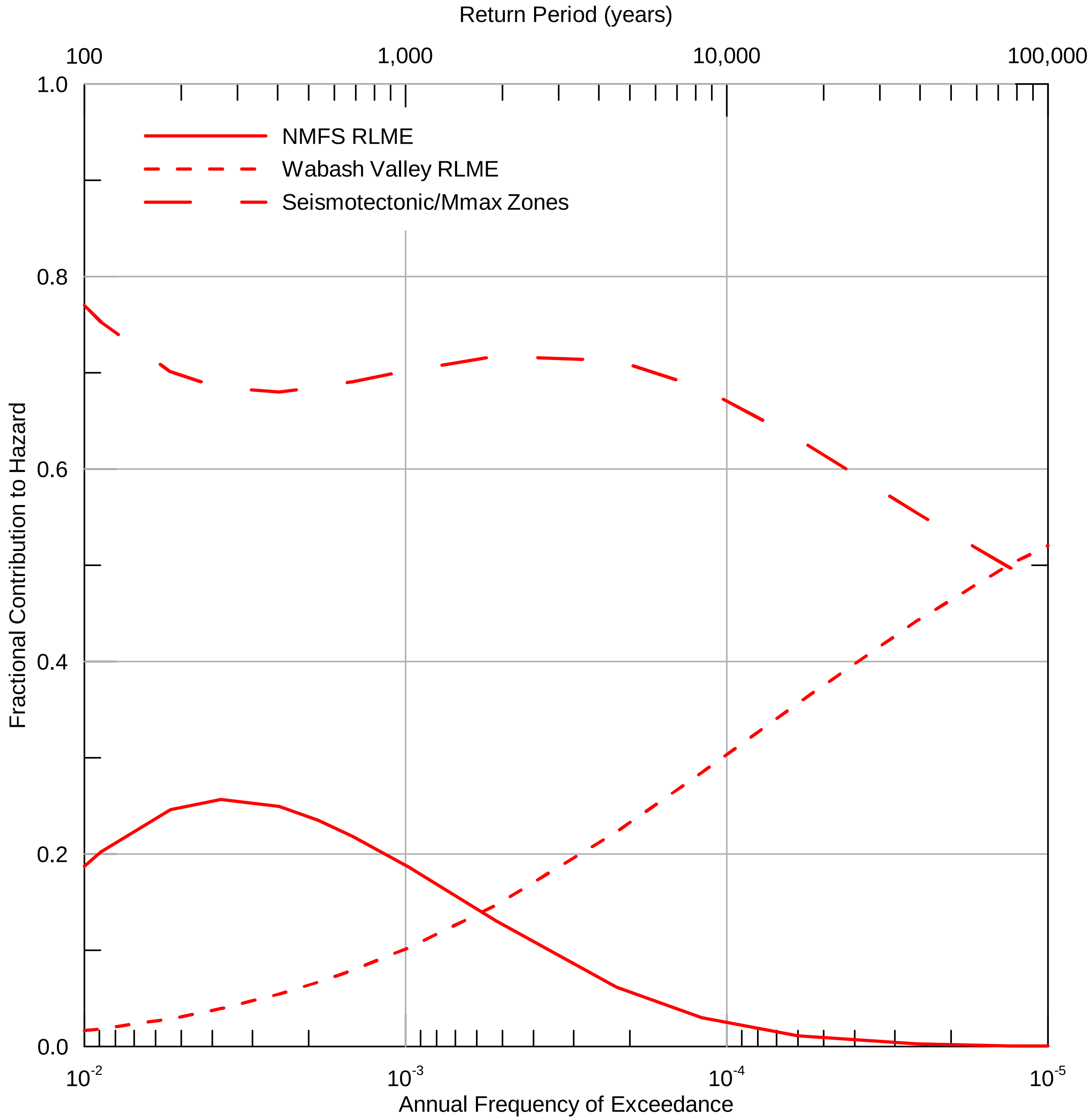
AECOM

Project No. 60442676

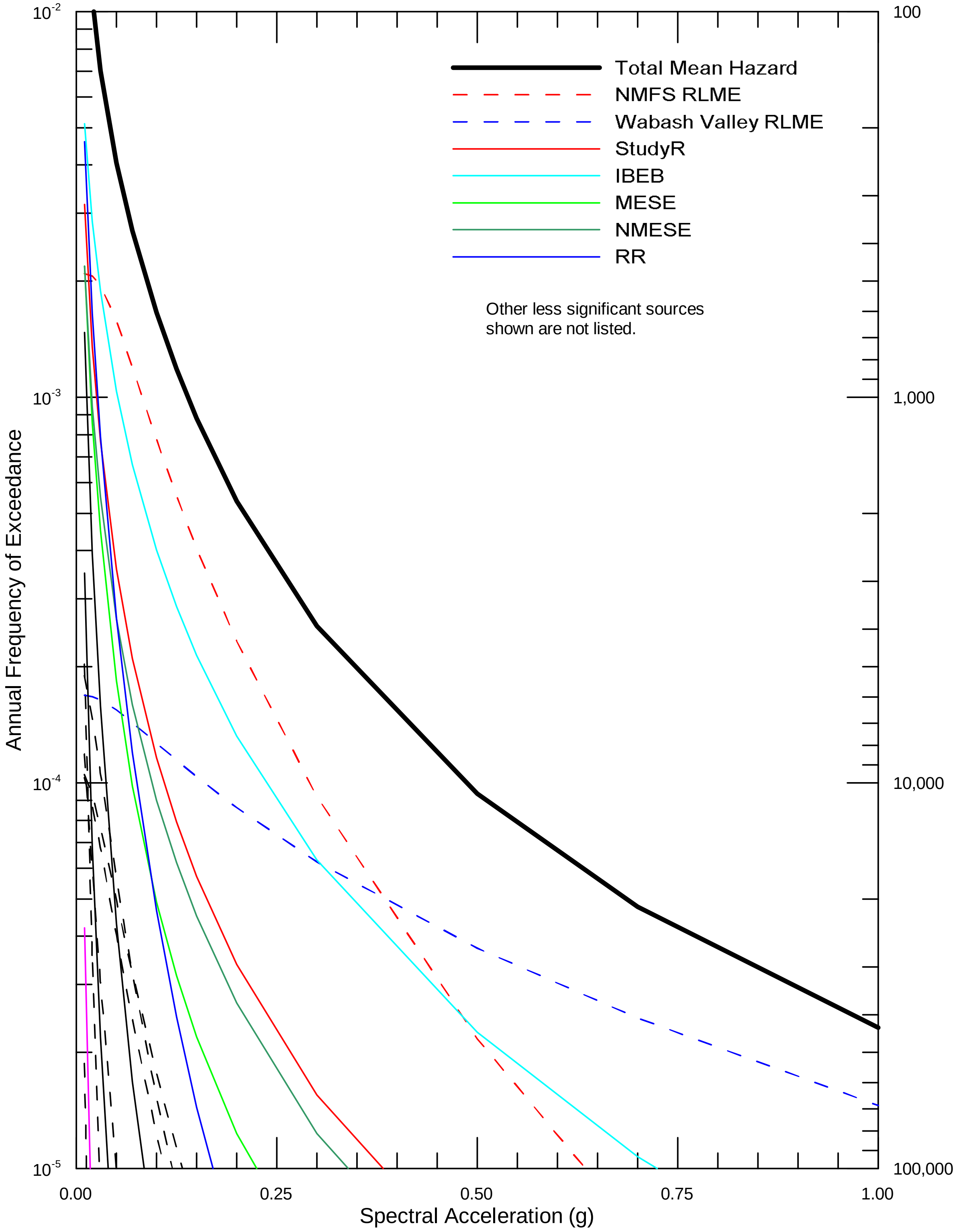
A.B. Brown
Generating Station
Vectren Corporation

SEISMIC SOURCE CONTRIBUTIONS TO MEAN
PEAK HORIZONTAL ACCELERATION HAZARD
ON HARD ROCK

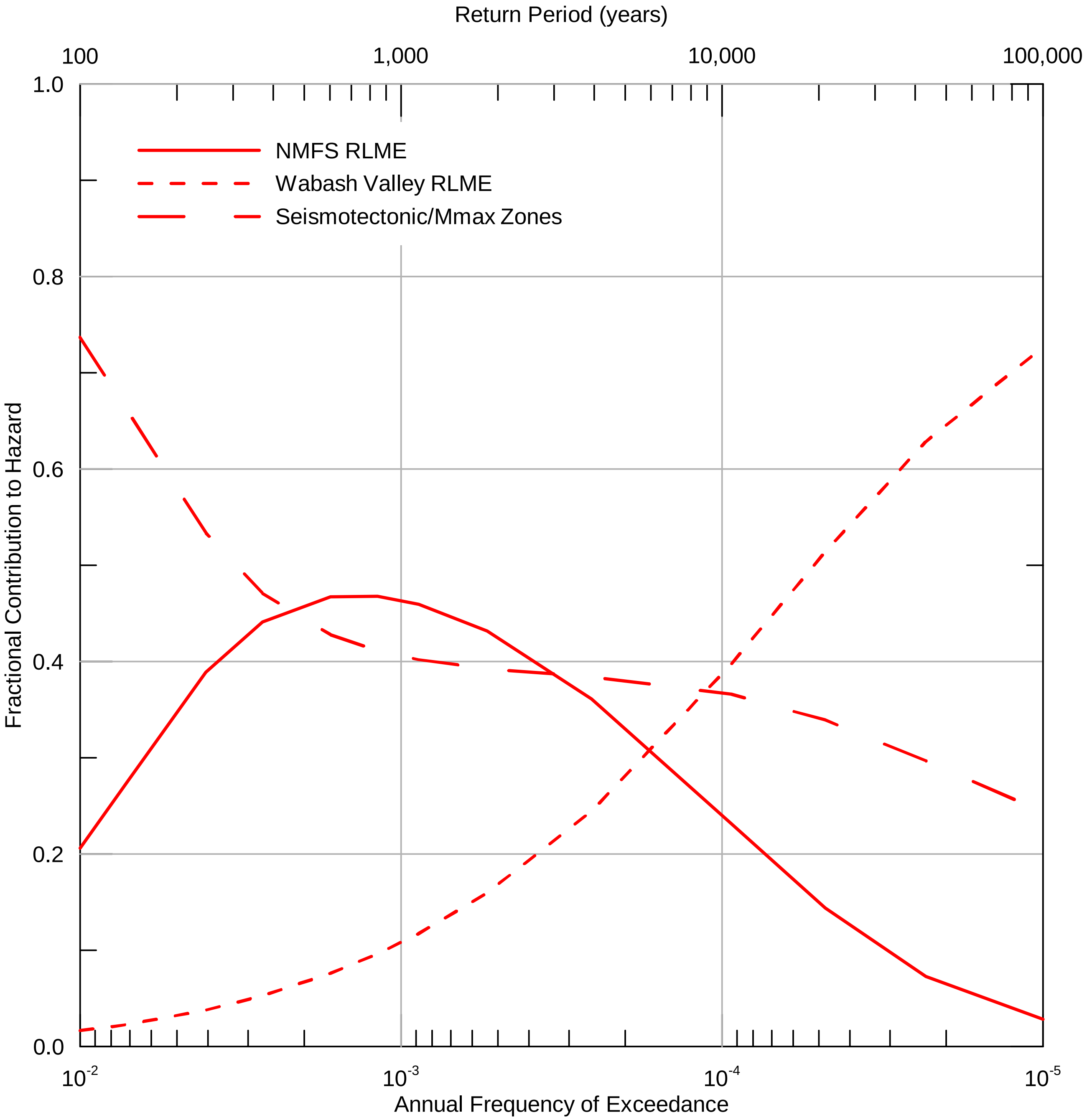
Figure
18



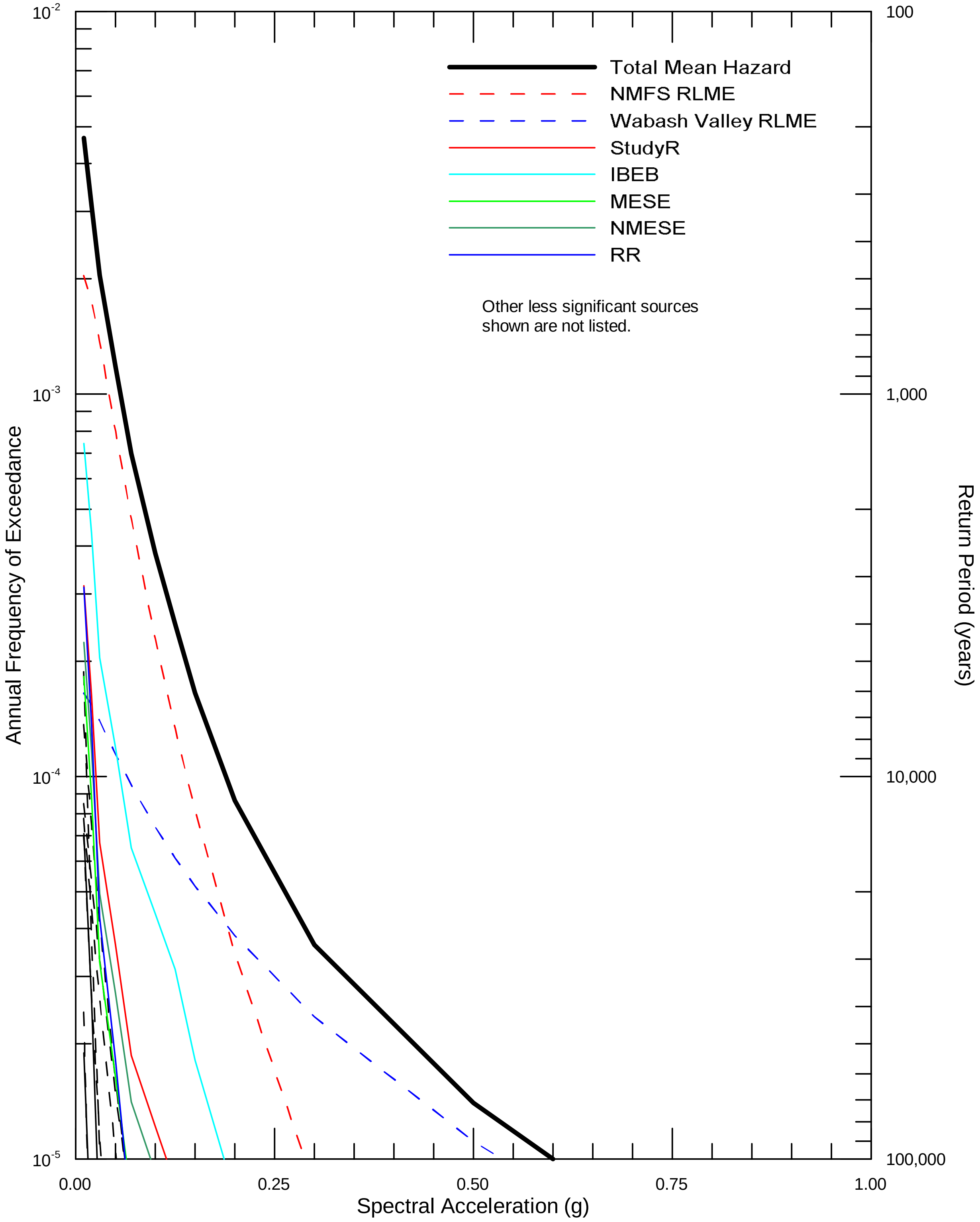
AECOM	Project No. 60442676 A.B. Brown Generating Station Vectren Corporation	SEISMIC SOURCE FRACTIONAL CONTRIBUTION TO MEAN PEAK HORIZONTAL ACCELERATION HAZARD ON HARD ROCK	Figure 19



AECOM	Project No. 60442676	SEISMIC SOURCE CONTRIBUTIONS TO MEAN 0.4 SEC HORIZONTAL SPECTRAL ACCELERATION HAZARD ON HARD ROCK	Figure 20
	A.B. Brown Generating Station Vectren Corporation		



AECOM	Project No.60442676 A.B. Brown Generating Station Vectren Corporation	SEISMIC SOURCE FRACTIONAL CONTRIBUTION TO MEAN 0.4 SEC HORIZONTAL SPECTRAL ACCELERATION HAZARD ON HARD ROCK	Figure 21



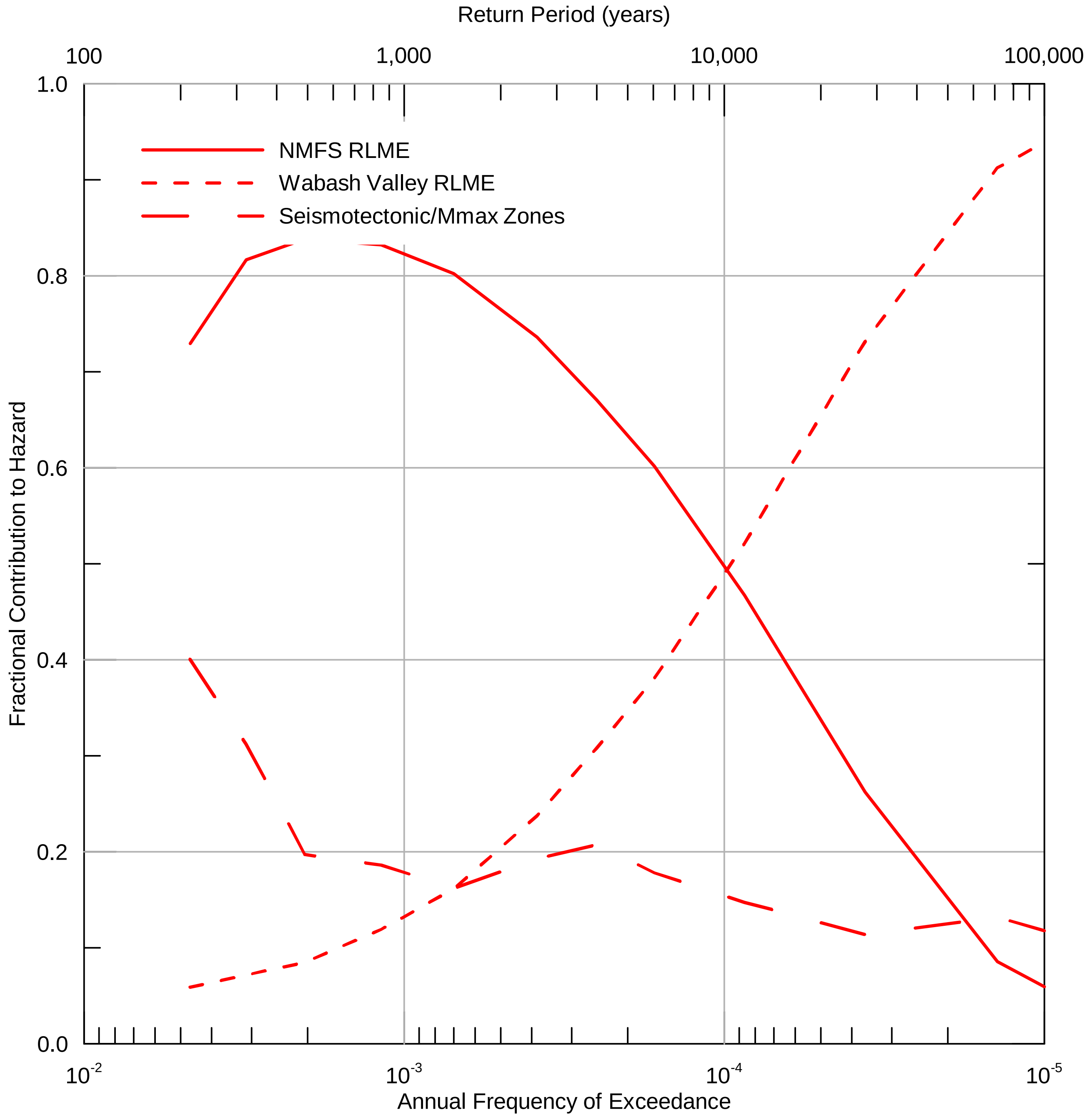
AECOM

Project No. 60442676

A.B. Brown
Generating Station
Vectren Corporation

SEISMIC SOURCE CONTRIBUTIONS TO MEAN
1.0 SEC HORIZONTAL SPECTRAL ACCELERATION
HAZARD ON HARD ROCK

Figure
22



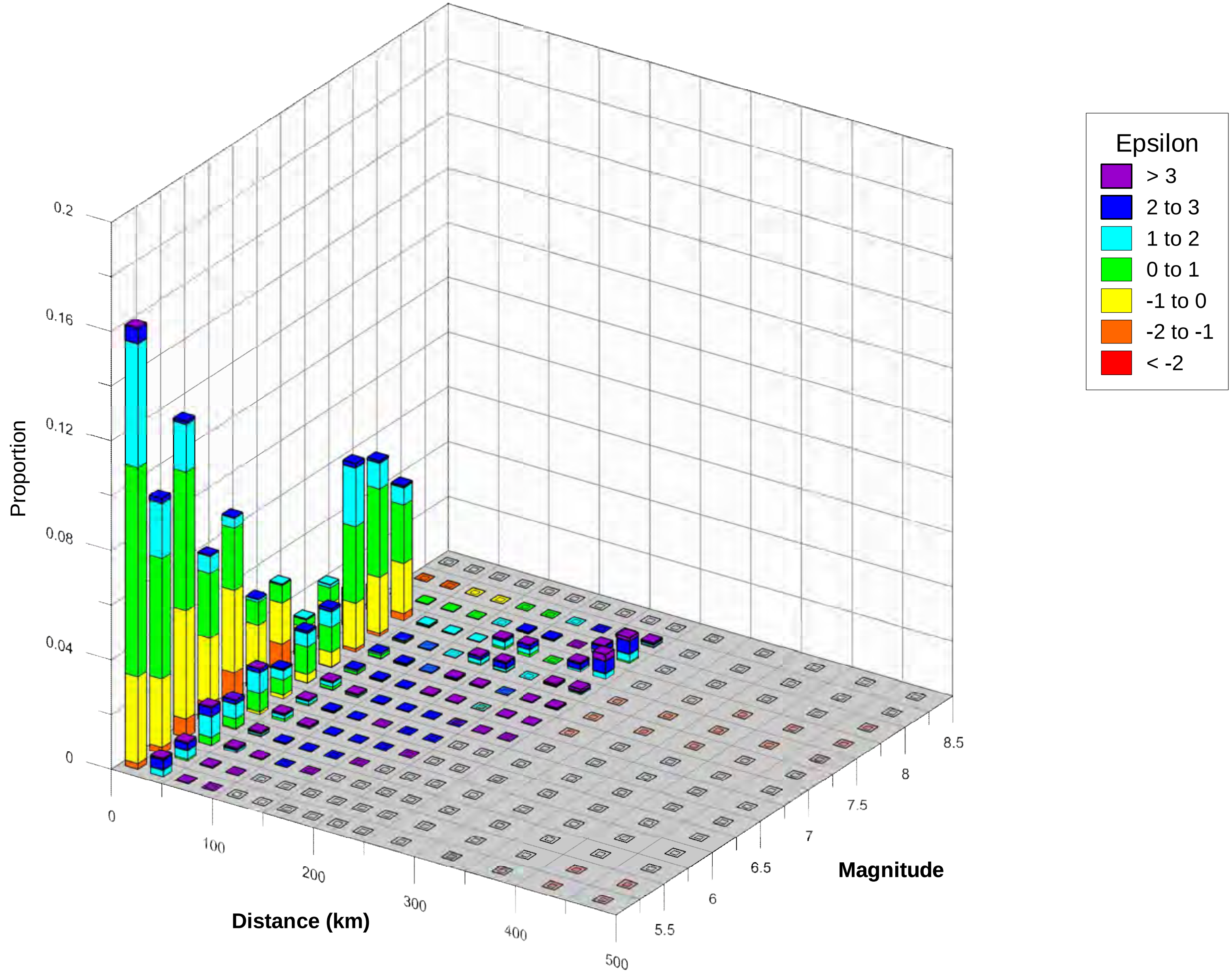
AECOM

Project No. 60442676

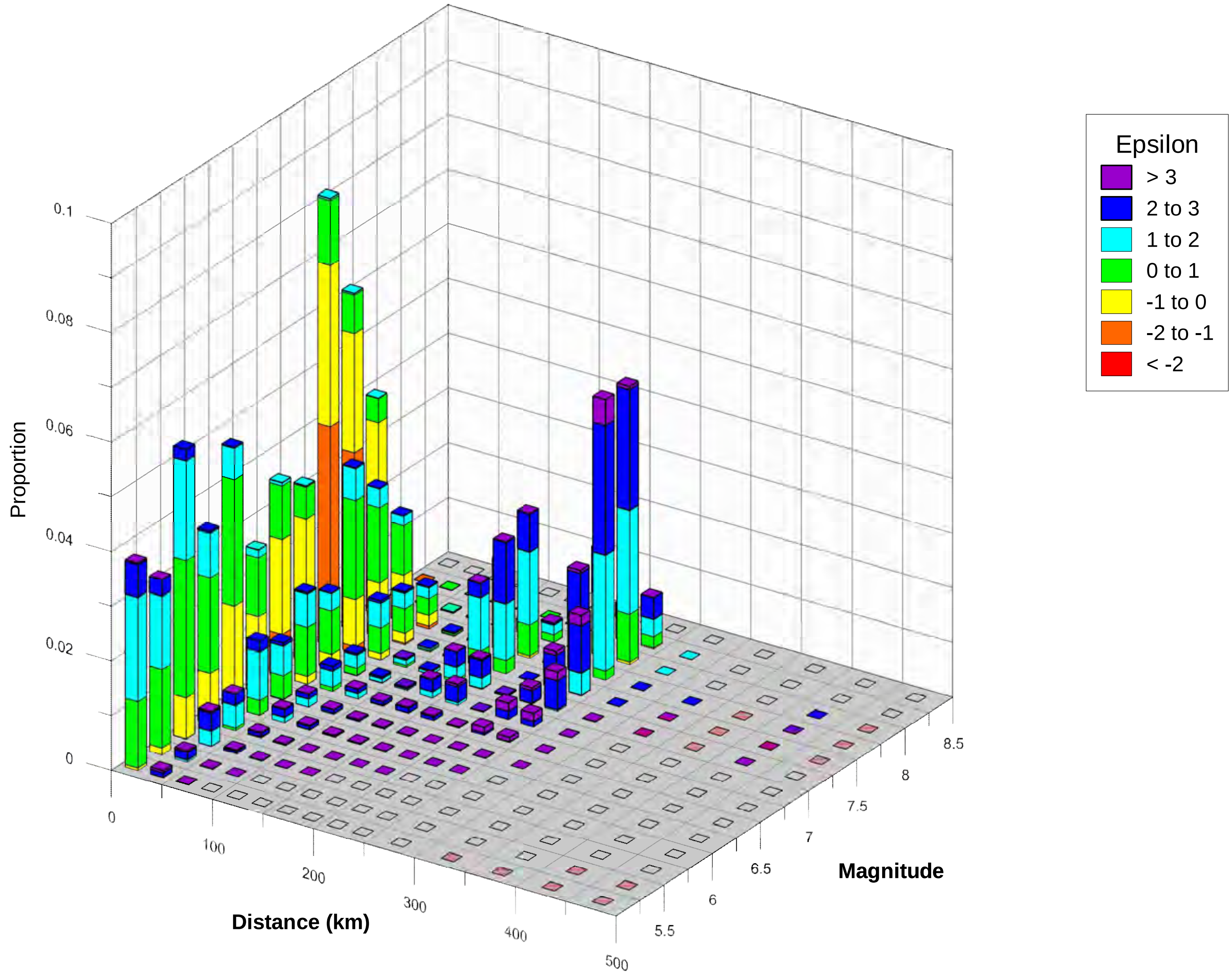
A.B. Brown
Generating Station
Vectren Corporation

SEISMIC SOURCE FRACTIONAL CONTRIBUTION
TO MEAN 1.0 SEC HORIZONTAL SPECTRAL
ACCELERATION HAZARD ON HARD ROCK

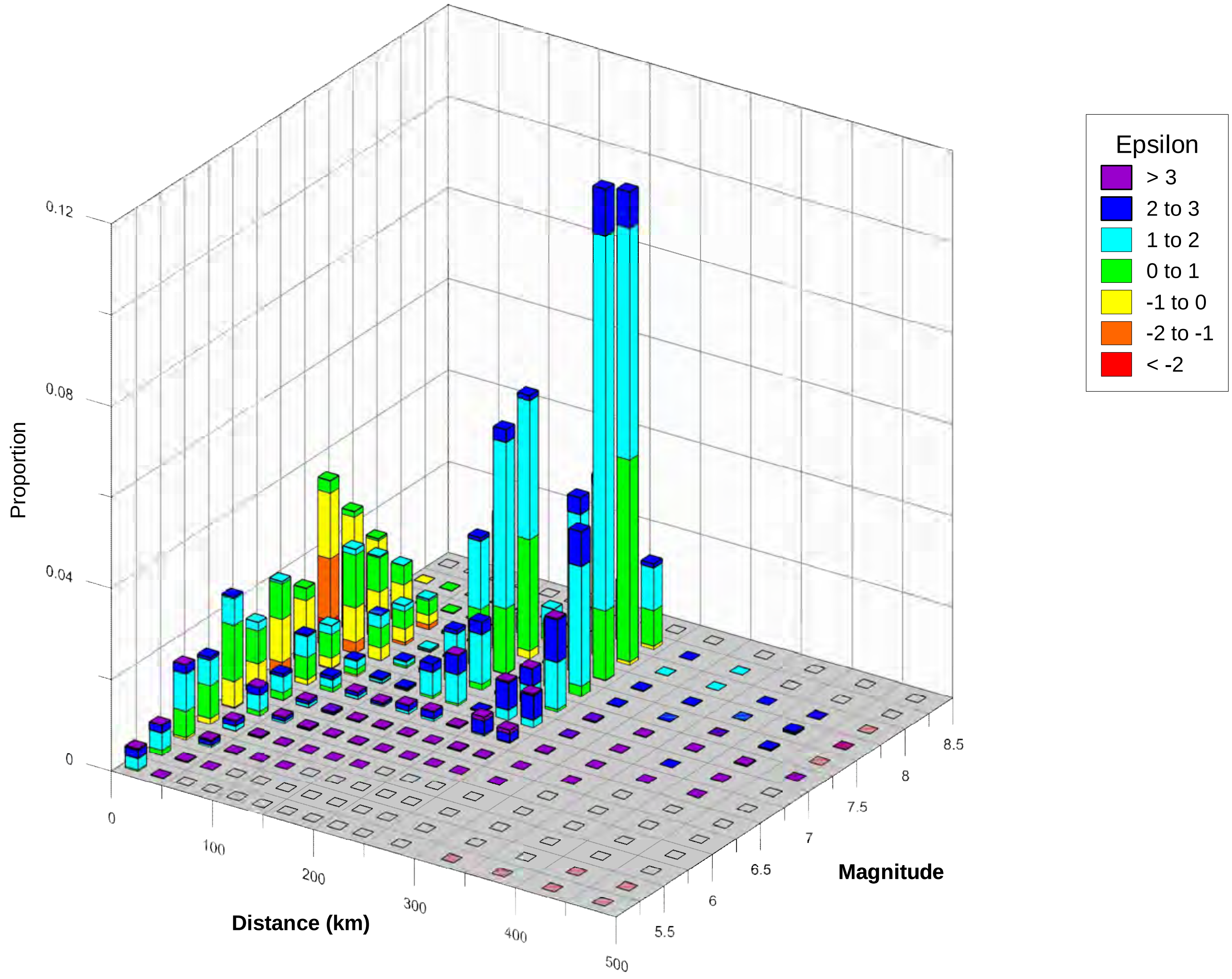
Figure
23



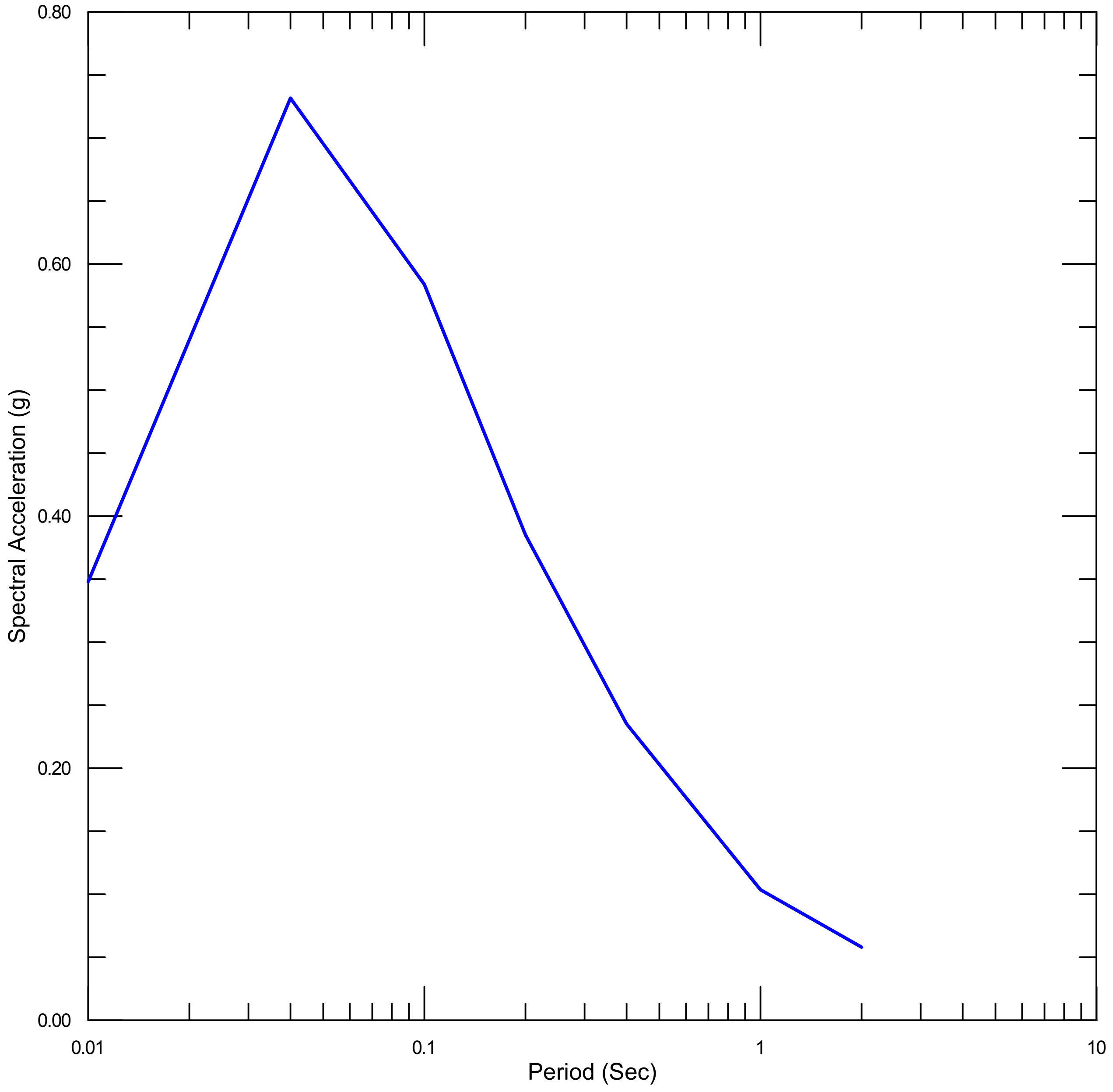
AECOM	Project No. 60442676 A.B. Brown Generating Station Vectren Corporation	MAGNITUDE, DISTANCE AND EPSILON CONTRIBUTIONS TO THE MEAN PEAK HORIZONTAL ACCELERATIONHAZARD AT 2,500-YEAR RETURN PERIOD ON HARD ROCK	Figure 24



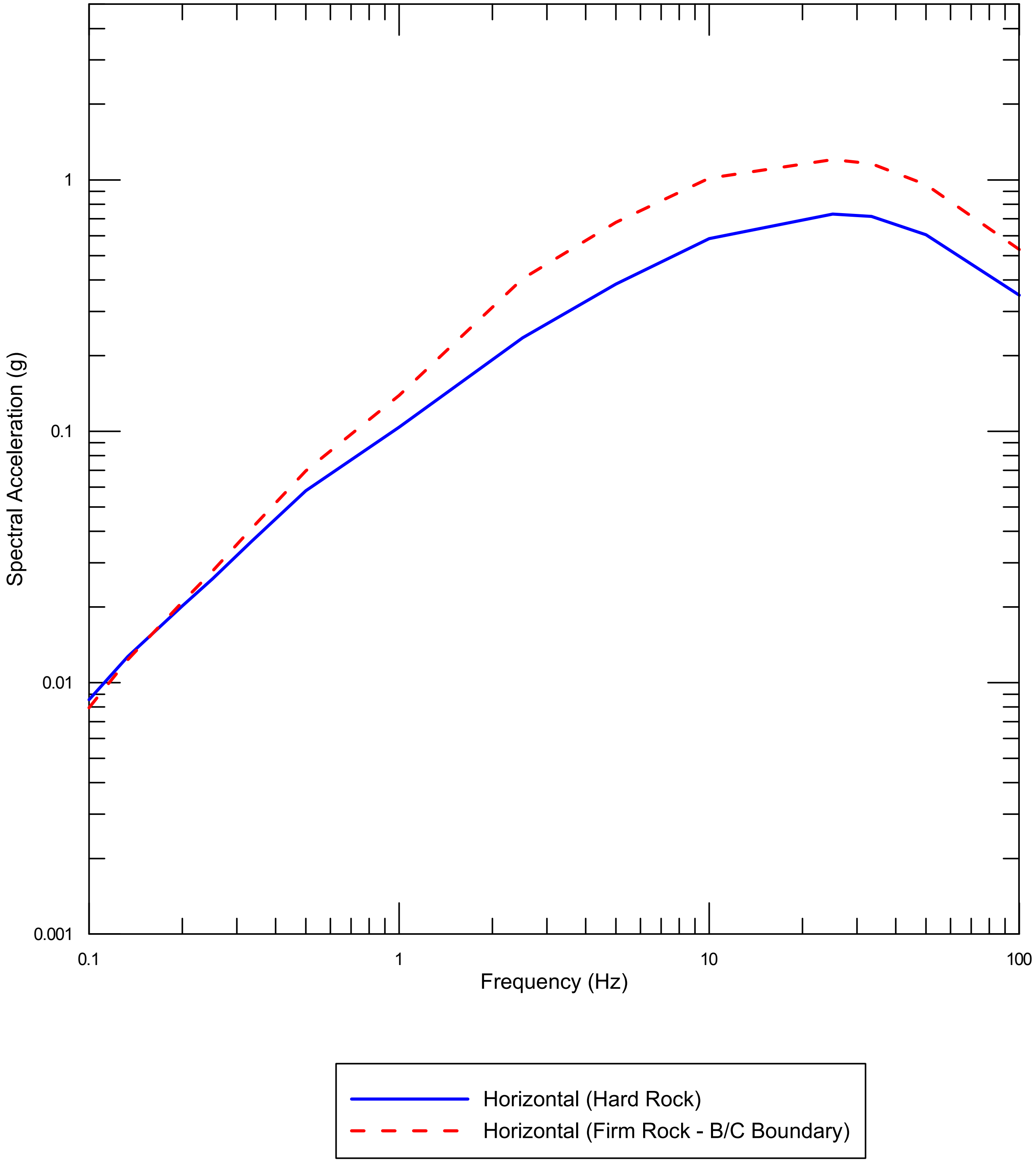
AECOM	Project No.60442676 A.B. Brown Generating Station Vectren Corporation	MAGNITUDE,DISTANCE AND EPSILON CONTRIBUTIONS TO THE MEAN 0.4 SEC HORIZONTAL SPECTRALACCELERATION HAZARD AT 2,500-YEAR RETURN PERIOD ON HARD ROCK	Figure 25



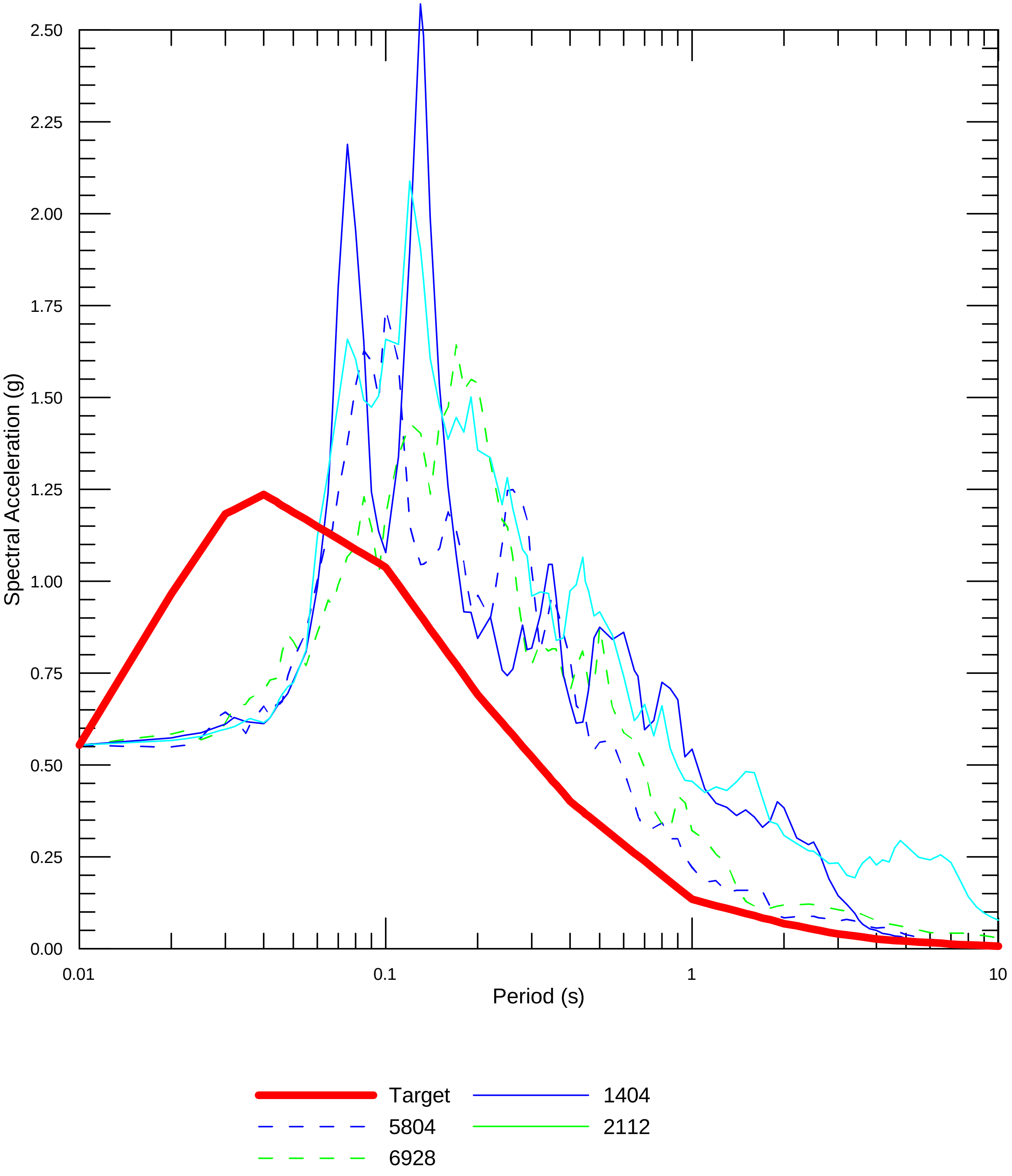
AECOM	Project No. 60442676 A.B. Brown Generating Station Vectren Corporation	MAGNITUDE,DISTANCE AND EPSILON CONTRIBUTIONS TO THE MEAN 1.0 SEC HORIZONTAL SPECTRALACCELERATION HAZARD AT 2,500-YEAR RETURN PERIOD ON HARD ROCK	Figure 26



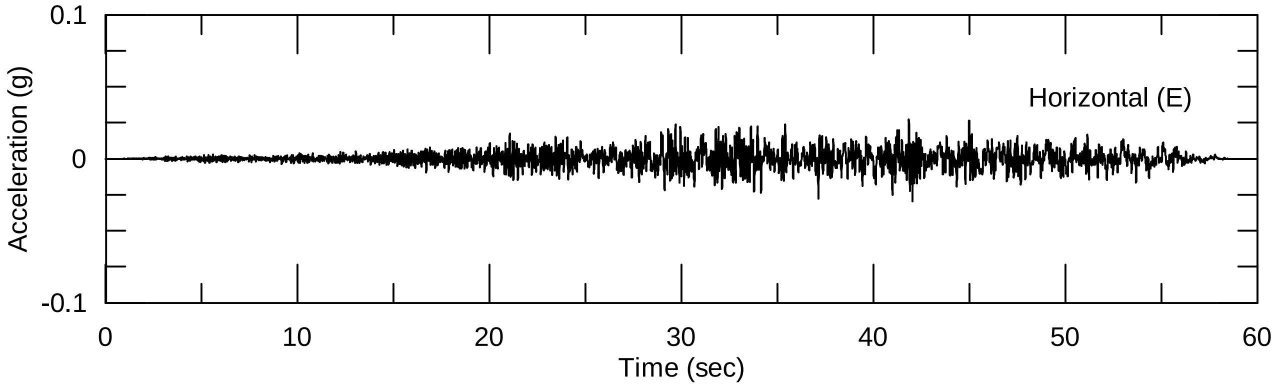
Project No. 60442676	HORIZONTAL 5%-DAMPED MEAN UHS AT 2,500-YEAR RETURN PERIOD ON HARD ROCK	Figure 27
A.B. Brown Generating Station Vectren Corporation		

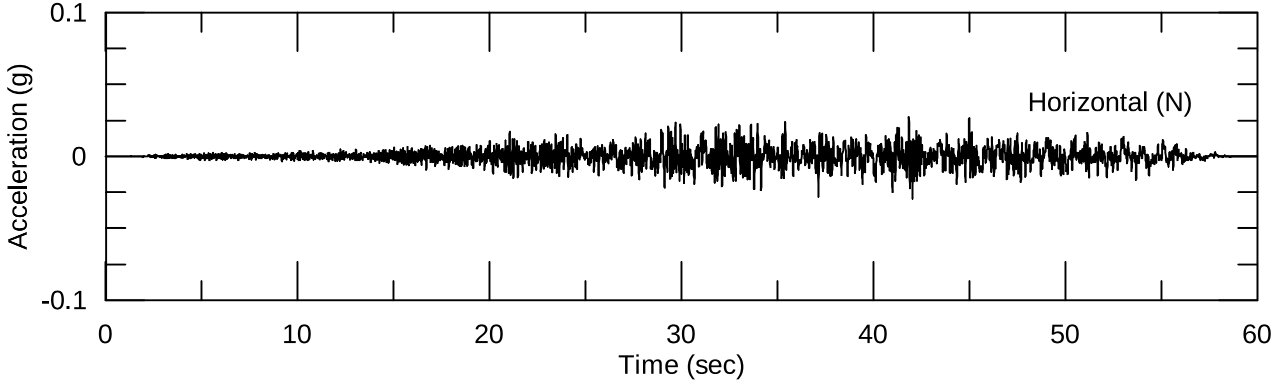


AECOM	Project No. 60442676	COMPARISON OF HORIZONTAL 5%-DAMPED MEAN UHS AT 2,500-YEAR RETURN PERIOD ON HARD AND FIRM ROCK	Figure 28
	A.B. Brown Generating Station Vectren Corporation		



AECOM	Project No.60442676	HORIZONTAL TARGET AND SELECTED SEED RESPONSE SPECTRA	Figure 29
	A.B. Brown Generating Station Vectren Corporation		





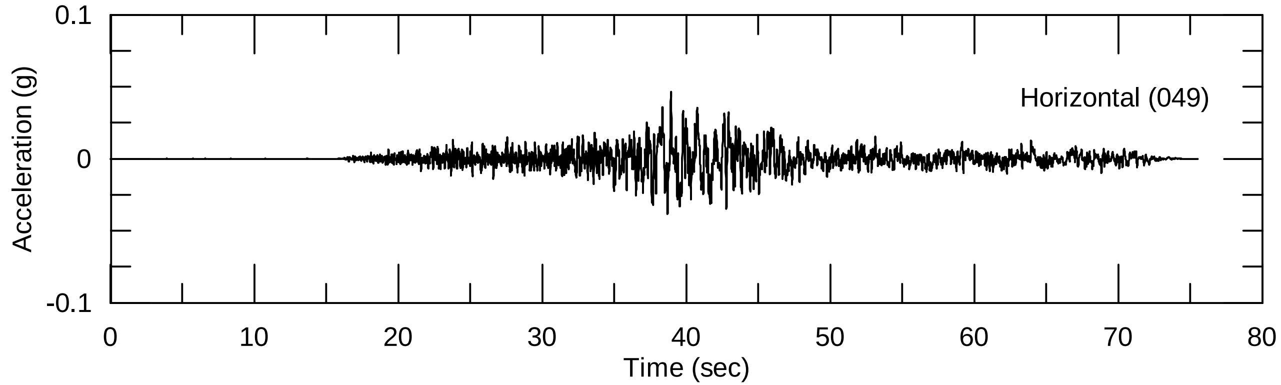
AECOM

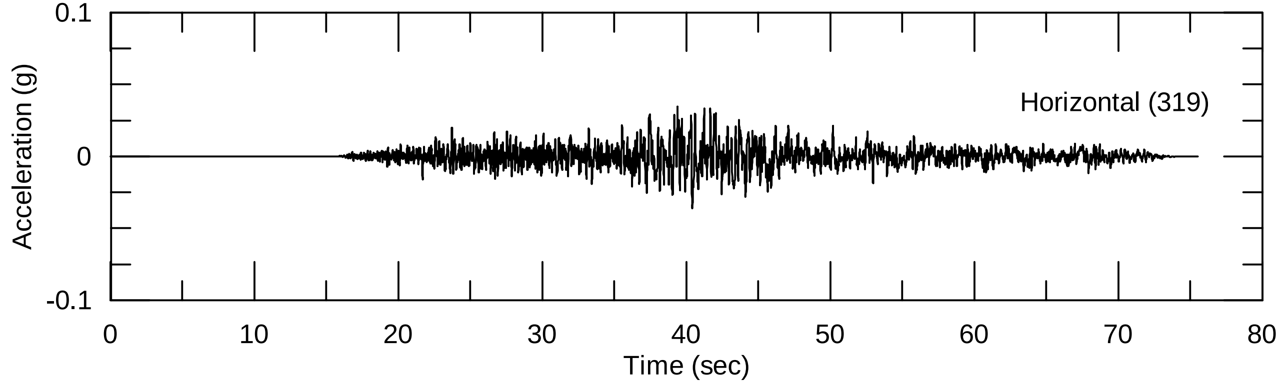
Project No. 60442676

A.B. Brown
Generating Station
Vectren Corporation

SEED TIME HISTORIES
RSN1404 - 1999 CHI CHI
PNG

Figure
30



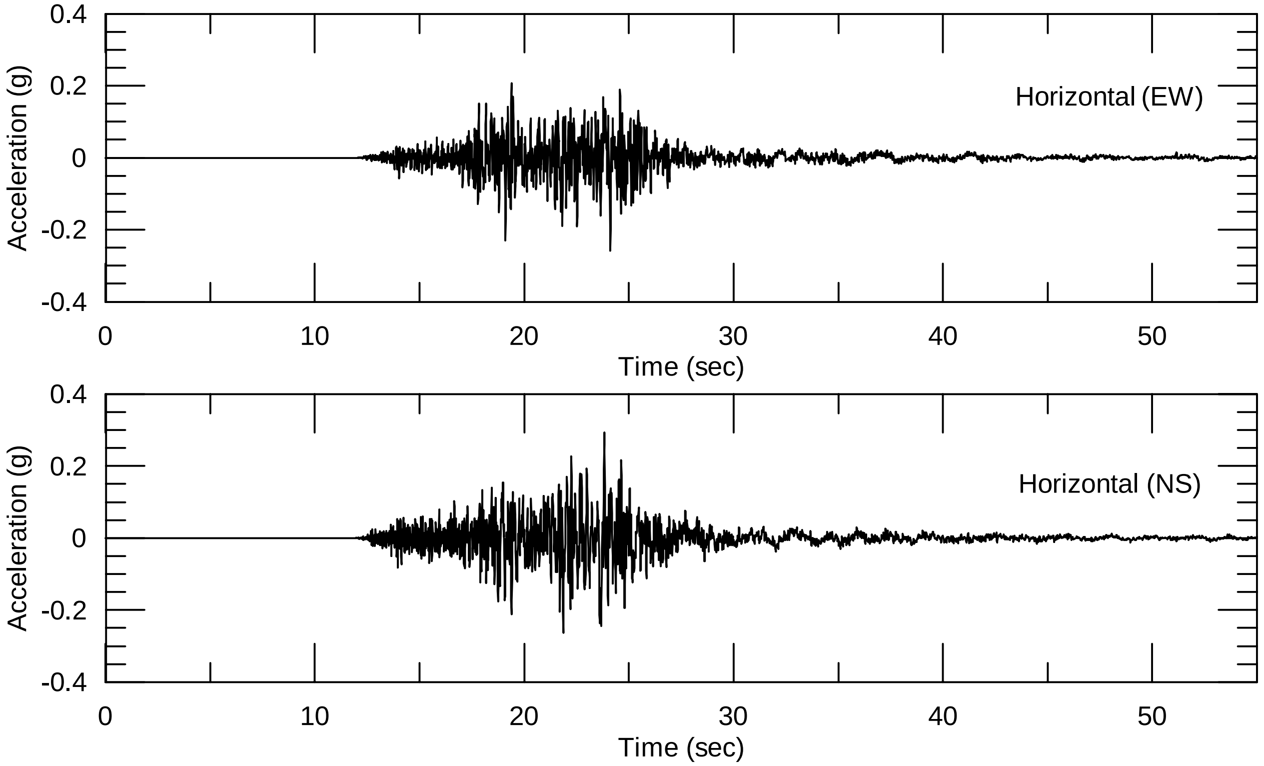


Project No. 60442676

A.B. Brown
Generating Station
Vectren Corporation

SEED TIME HISTORIES
RSN2112 - 2002 DENALI
TAPS PUMP STATION #8

Figure
31



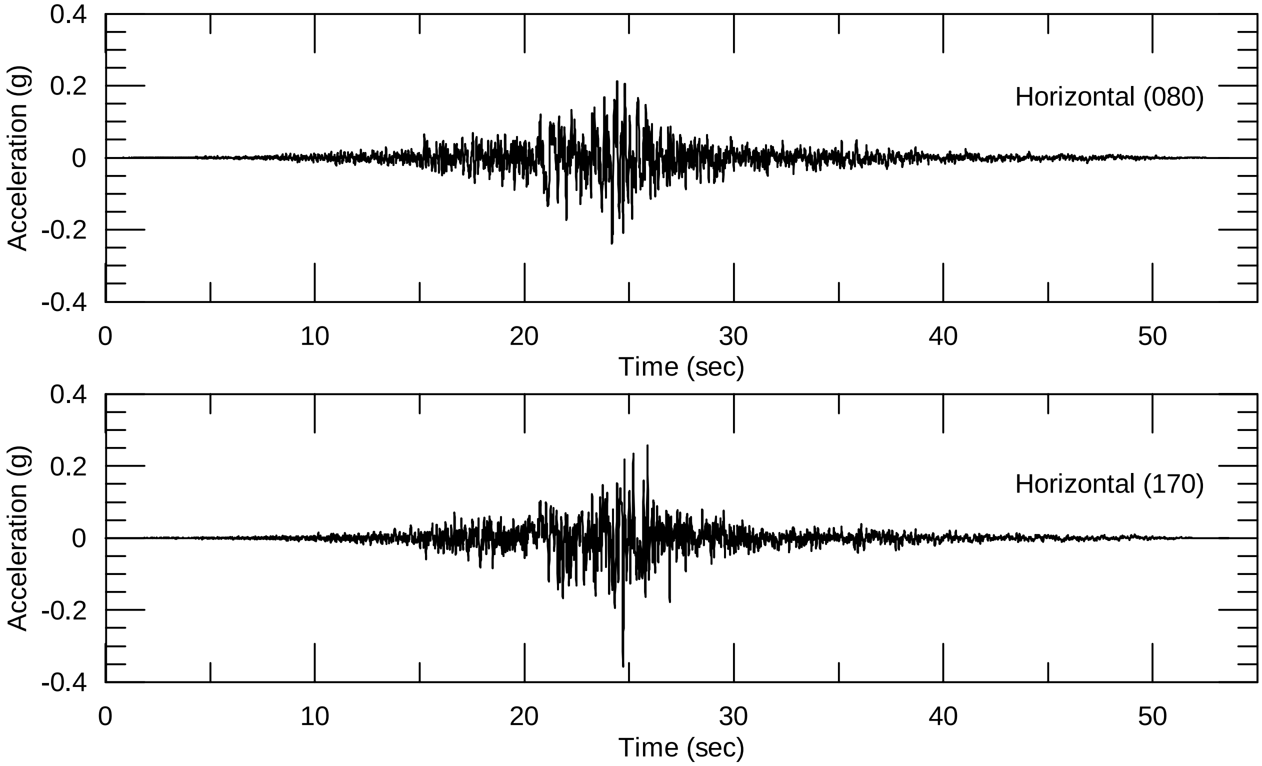
AECOM

Project No. 60442676

A.B. Brown
Generating Station
Vectren Corporation

SEED TIME HISTORIES
RSN5804 - IWATE
55446

Figure
32



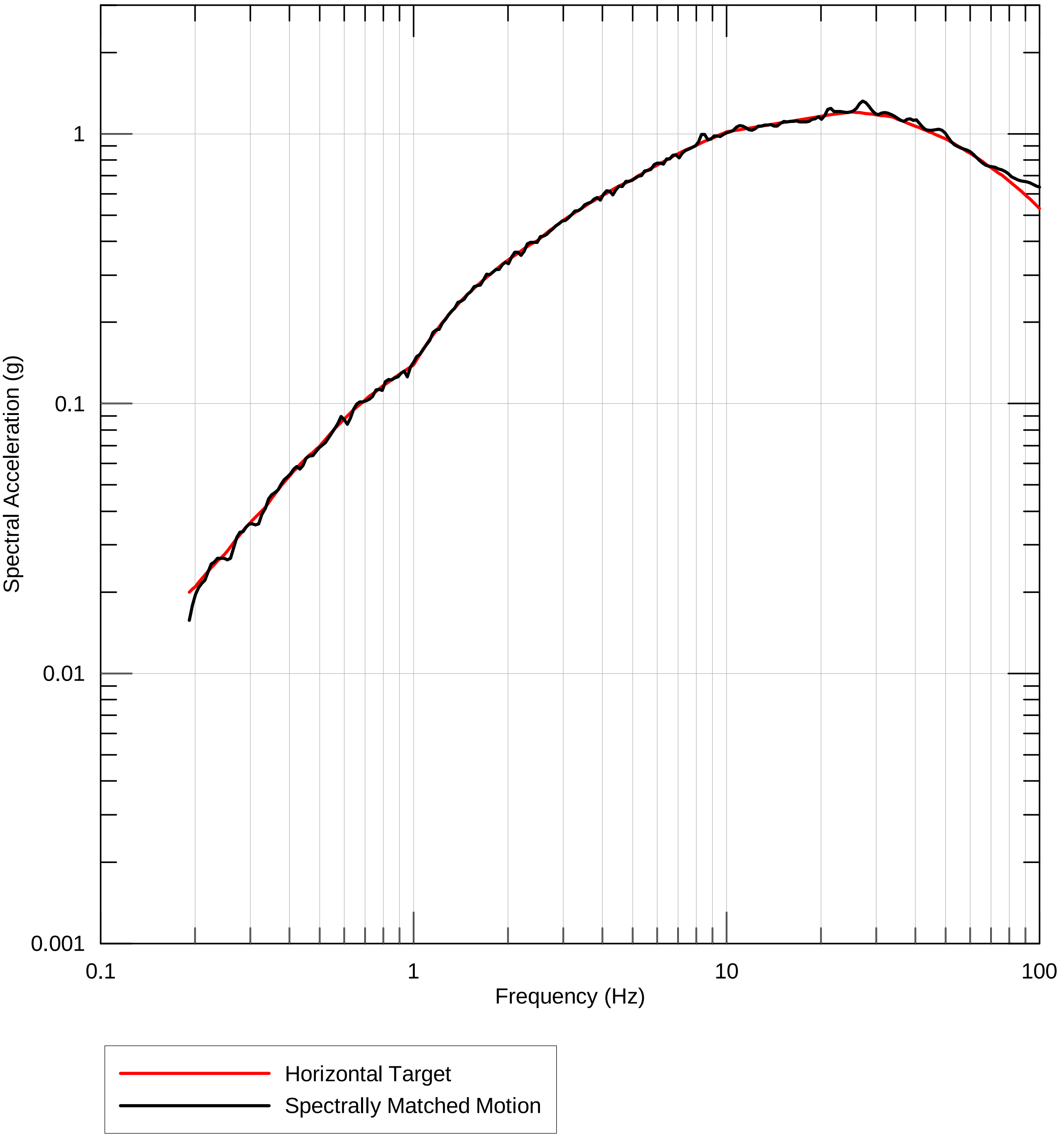
AECOM

Project No. 60442676

A.B. Brown
Generating Station
Vectren Corporation

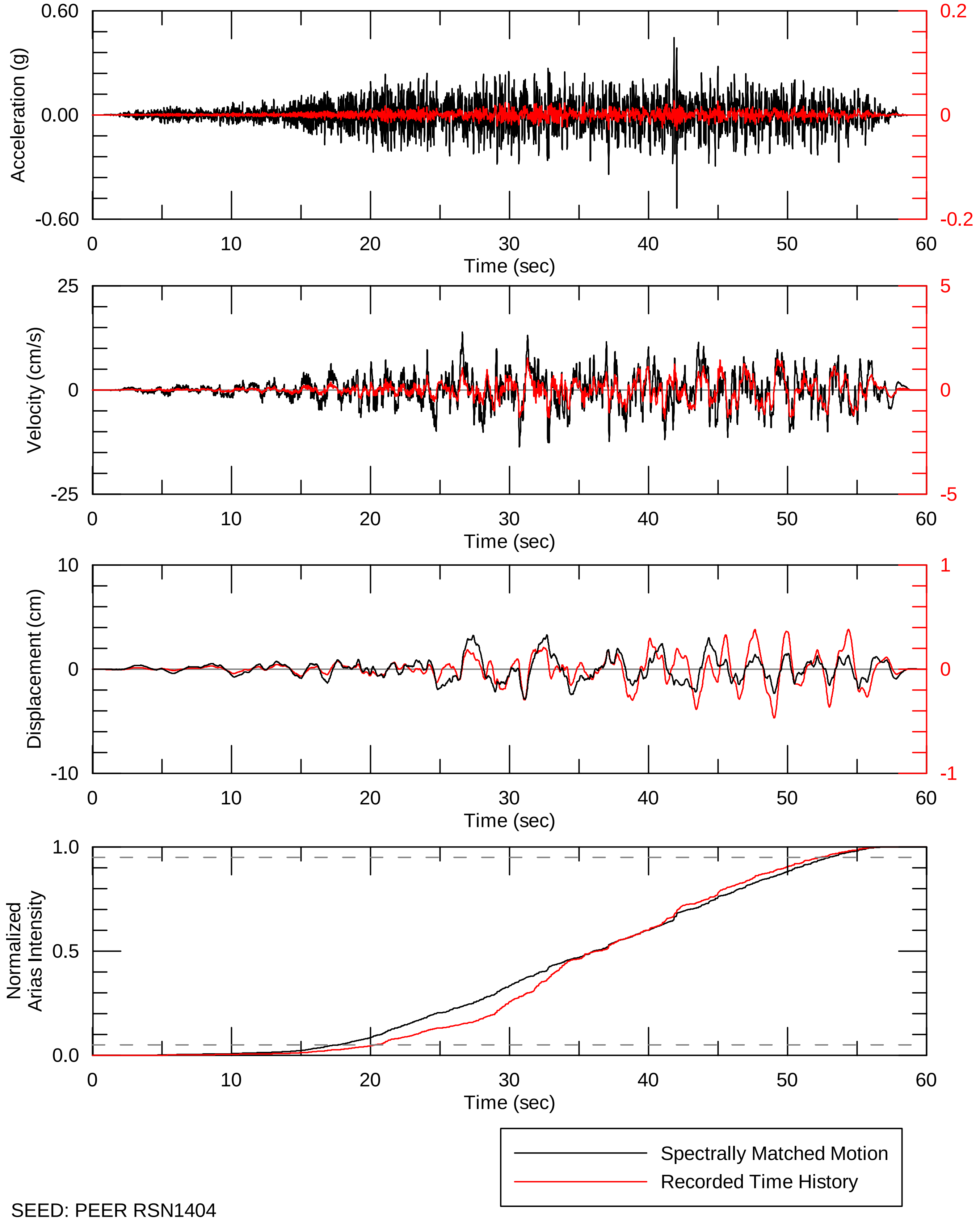
SEED TIME HISTORIES
RSN6928 - DARFIELD
LPCC

Figure
33



SEED: PEER RSN1404

AECOM	Project No. 60442676 A.B. Brown Generating Station Vectren Corporation	RESPONSE SPECTRUM FOR TIME HISTORY SPECTRALLY MATCHED TO 2,500-YEAR RETURN PERIOD UHS HORIZONTAL TARGET 1999 CHI CHI - PNG (E) SEED	Figure 34



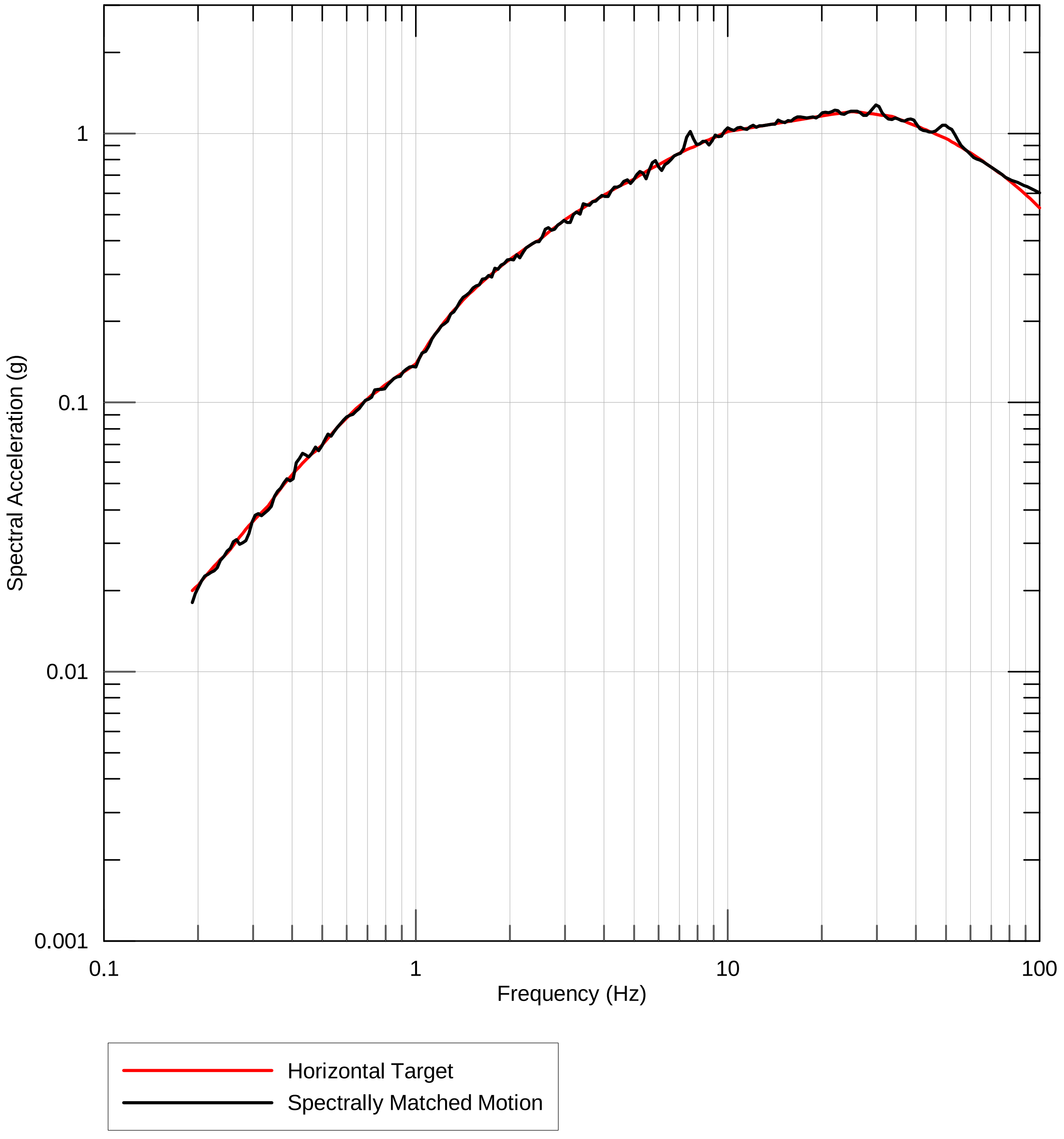
SEED: PEER RSN1404

Project No. 60442676

A.B. Brown
Generating Station
Vectren Corporation

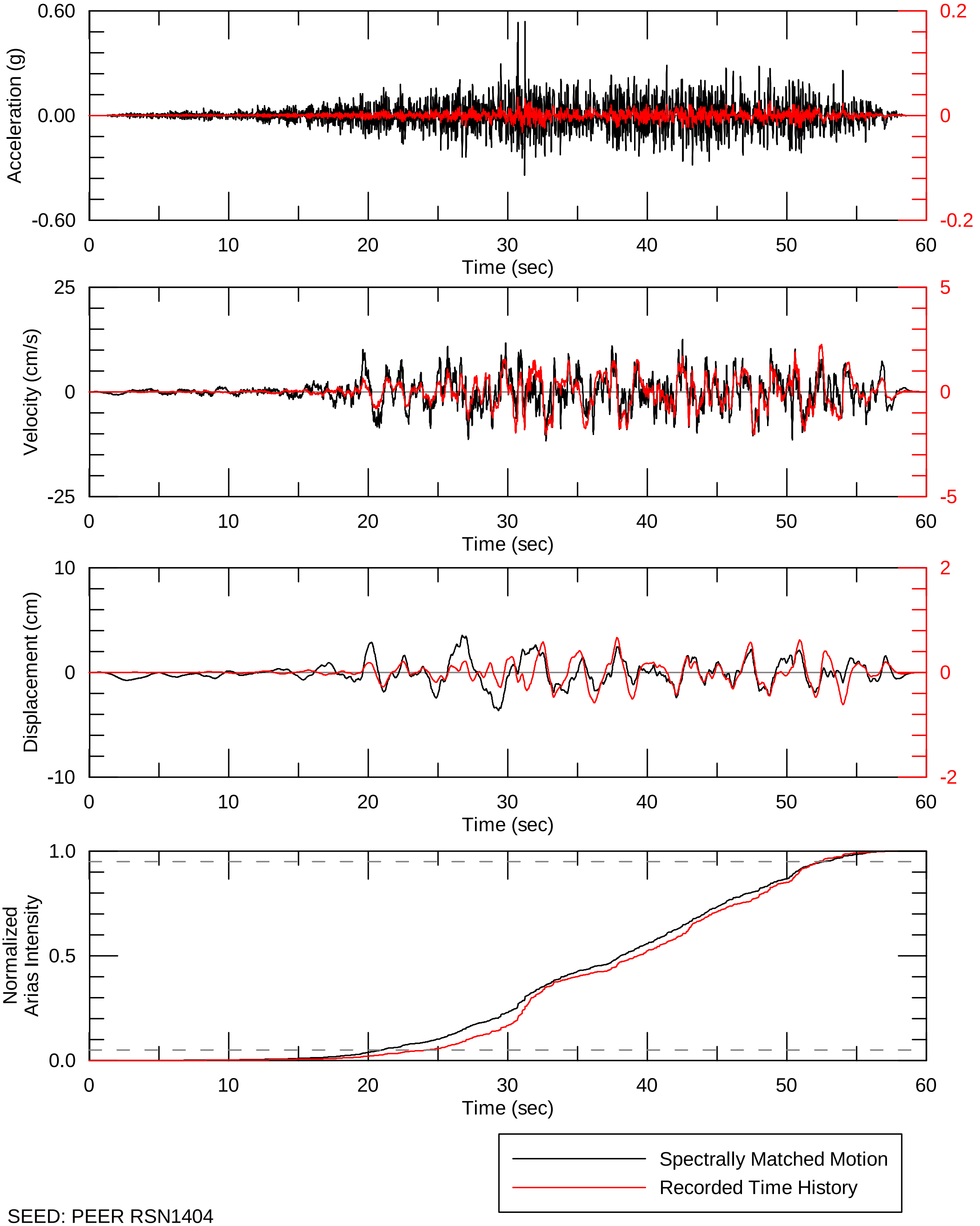
TIME HISTORY SPECTRALLY MATCHED TO
2,500-YEAR RETURN PERIOD UHS
HORIZONTAL TARGET
1999 CHI CHI - PNG (E) SEED

Figure
35



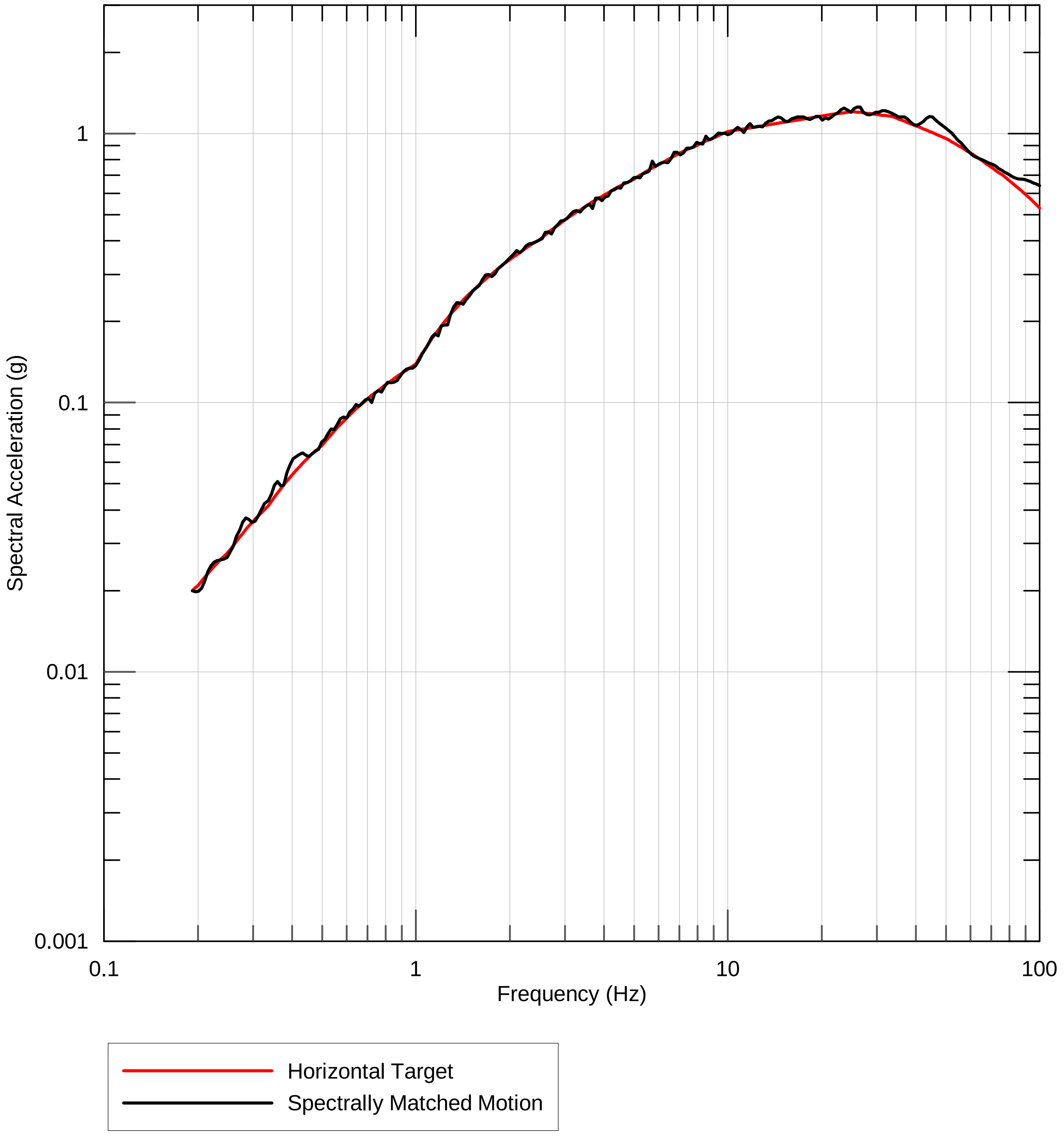
SEED: PEER RSN1404

AECOM	Project No. 60442676 A.B. Brown Generating Station Vectren Corporation	RESPONSE SPECTRUM FOR TIME HISTORY SPECTRALLY MATCHED TO 2,500-YEAR RETURN PERIOD UHS HORIZONTAL TARGET 1999 CHI CHI - PNG (N) SEED	Figure 36



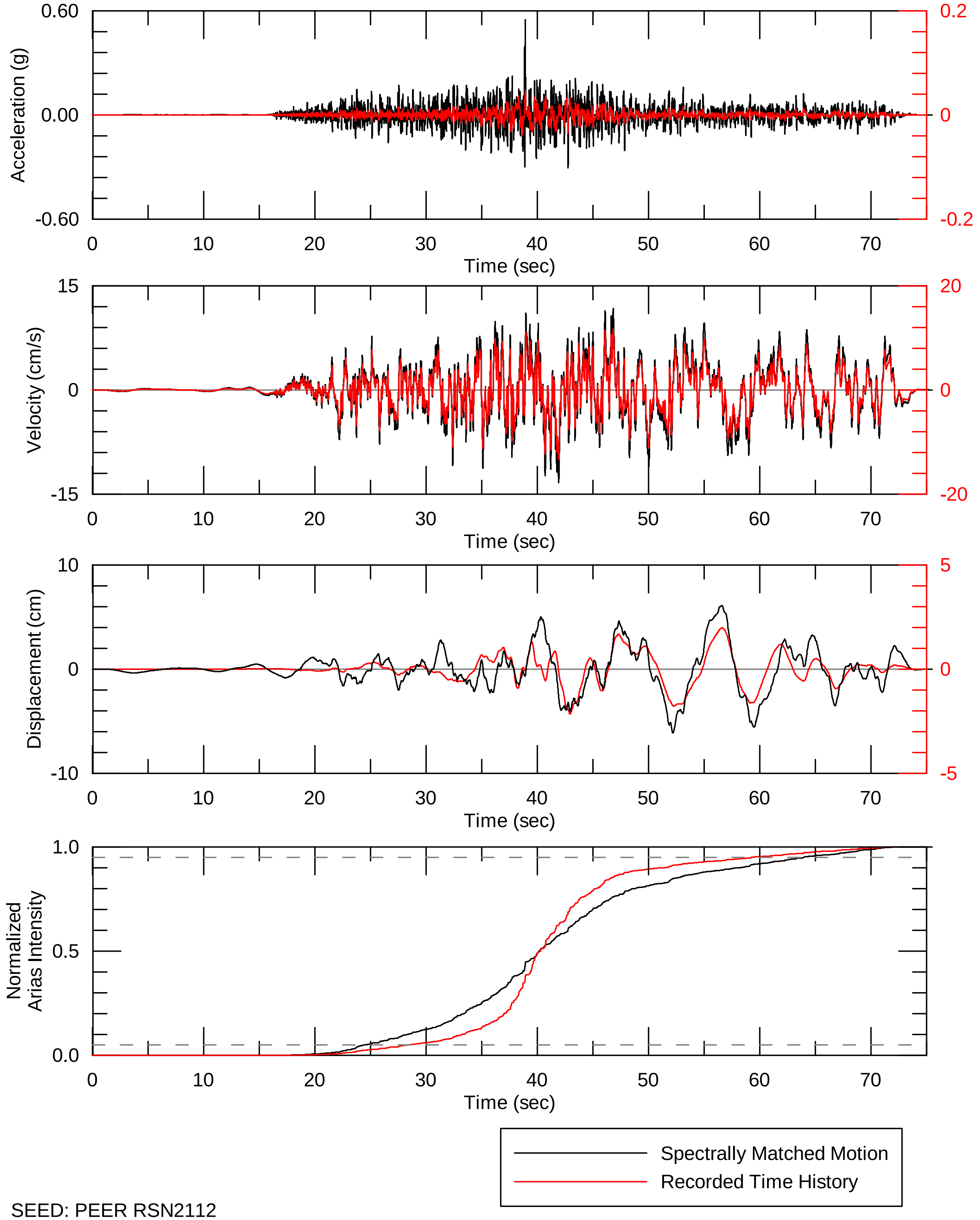
SEED: PEER RSN1404

AECOM	Project No. 60442676 A.B. Brown Generating Station Vectren Corporation	TIME HISTORY SPECTRALLY MATCHED TO 2,500-YEAR RETURN PERIOD UHS HORIZONTAL TARGET 1999 CHI CHI - PNG (N) SEED	Figure 37



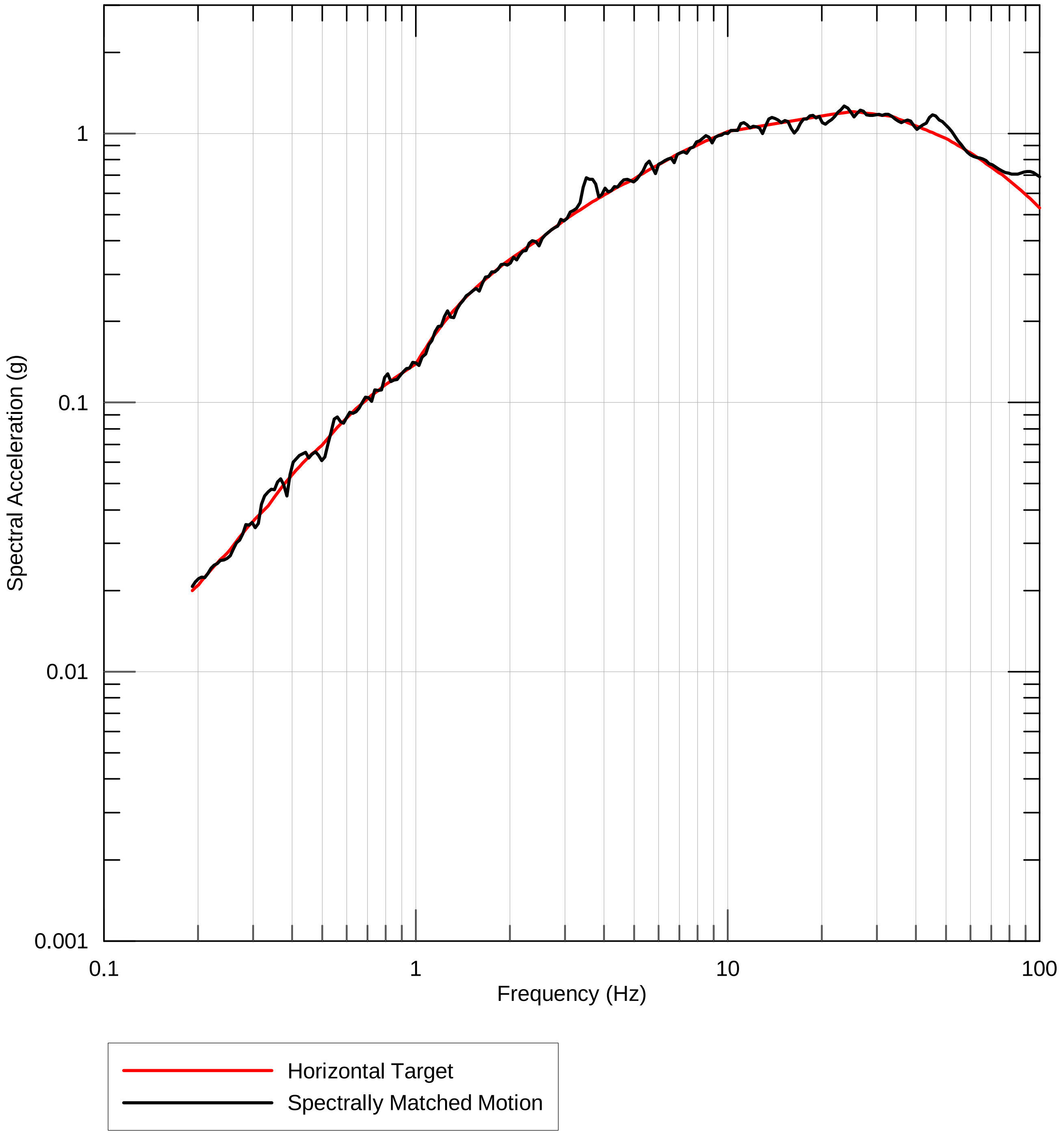
SEED: PEER RSN2112

AECOM	Project No. 60442676 A.B. Brown Generating Station Vectren Corporation	RESPONSE SPECTRUM FOR TIME HISTORY SPECTRALLY MATCHED TO 2,500-YEAR RETURN PERIOD UHS HORIZONTAL TARGET 2002 DENALI - TAPS PUMP STATION #8 (049) SEED	Figure 38



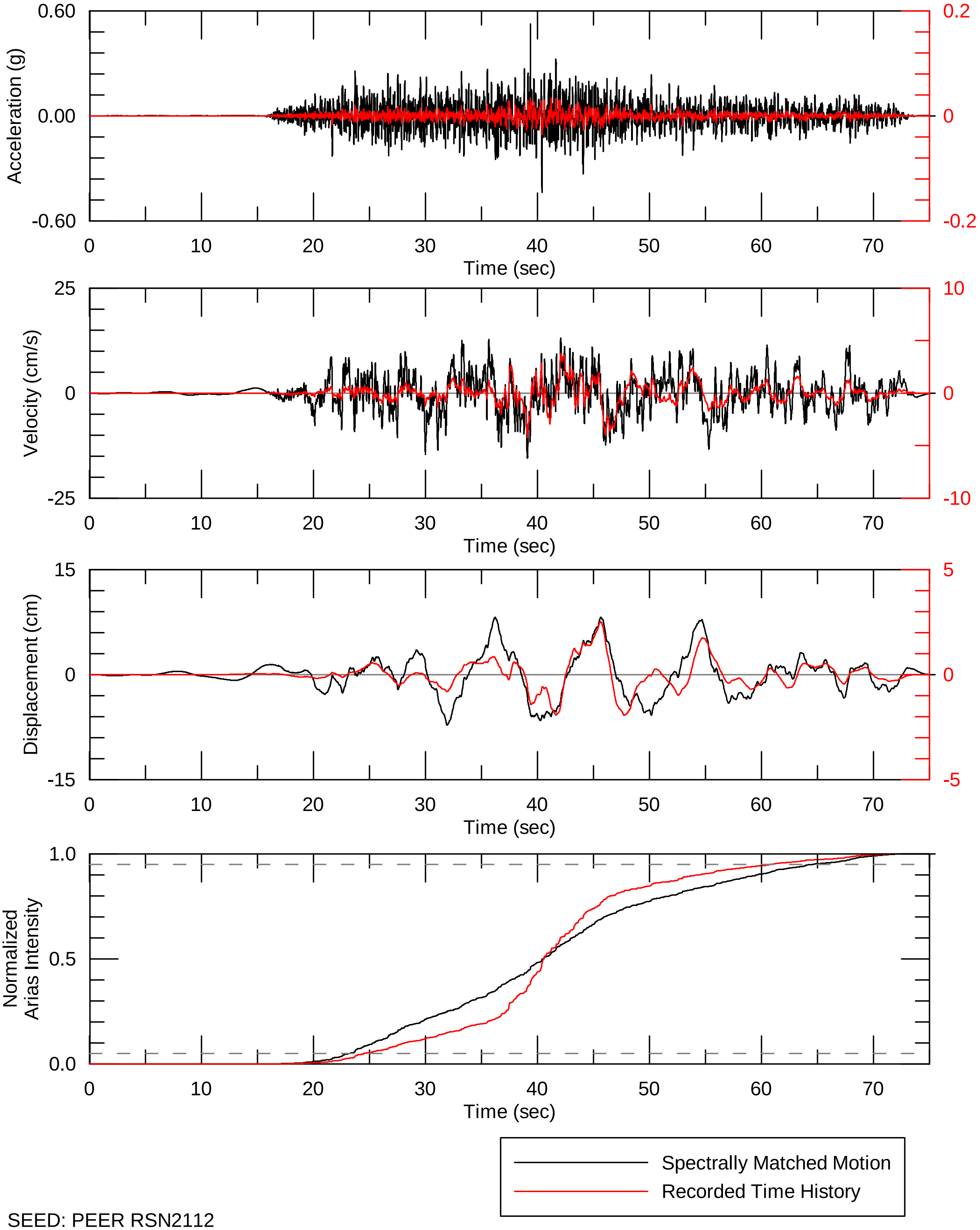
SEED: PEER RSN2112

Project No. 60442676	TIME HISTORY SPECTRALLY MATCHED TO 2,500-YEAR RETURN PERIOD UHS HORIZONTAL TARGET 2002 DENALI - TAPS PUMP STATION #8 (049) SEED	Figure 39
A.B. Brown Generating Station Vectren Corporation		



SEED: PEER RSN2112

AECOM	Project No. 60442676 A.B. Brown Generating Station Vectren Corporation	RESPONSE SPECTRUM FOR TIME HISTORY SPECTRALLY MATCHED TO 2,500-YEAR RETURN PERIOD UHS HORIZONTAL TARGET 2002 DENALI - TAPS PUMP STATION #8 (319) SEED	Figure 40



SEED: PEER RSN2112

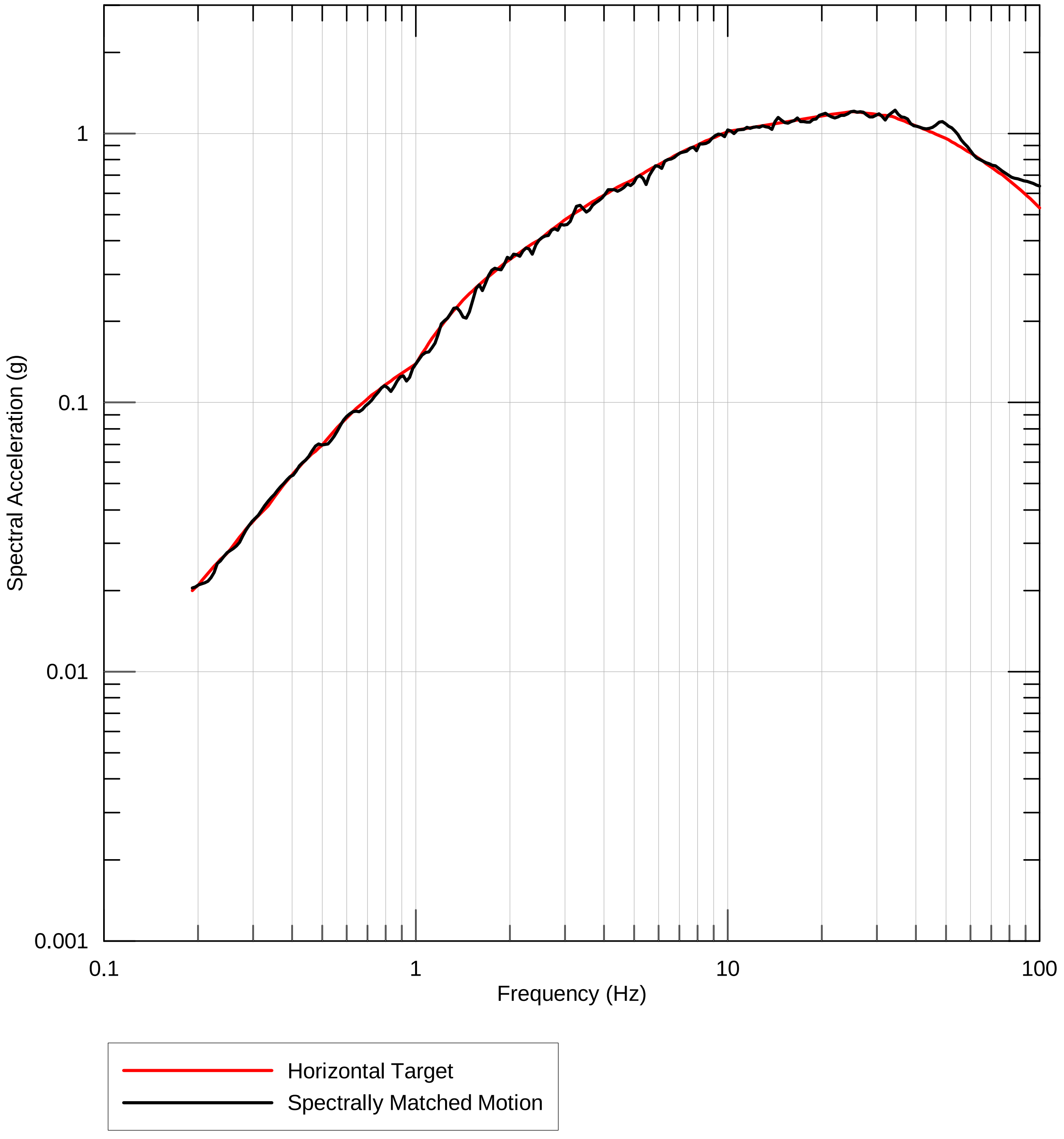
AECOM

Project No. 60442676

A.B. Brown
Generating Station
Vectren Corporation

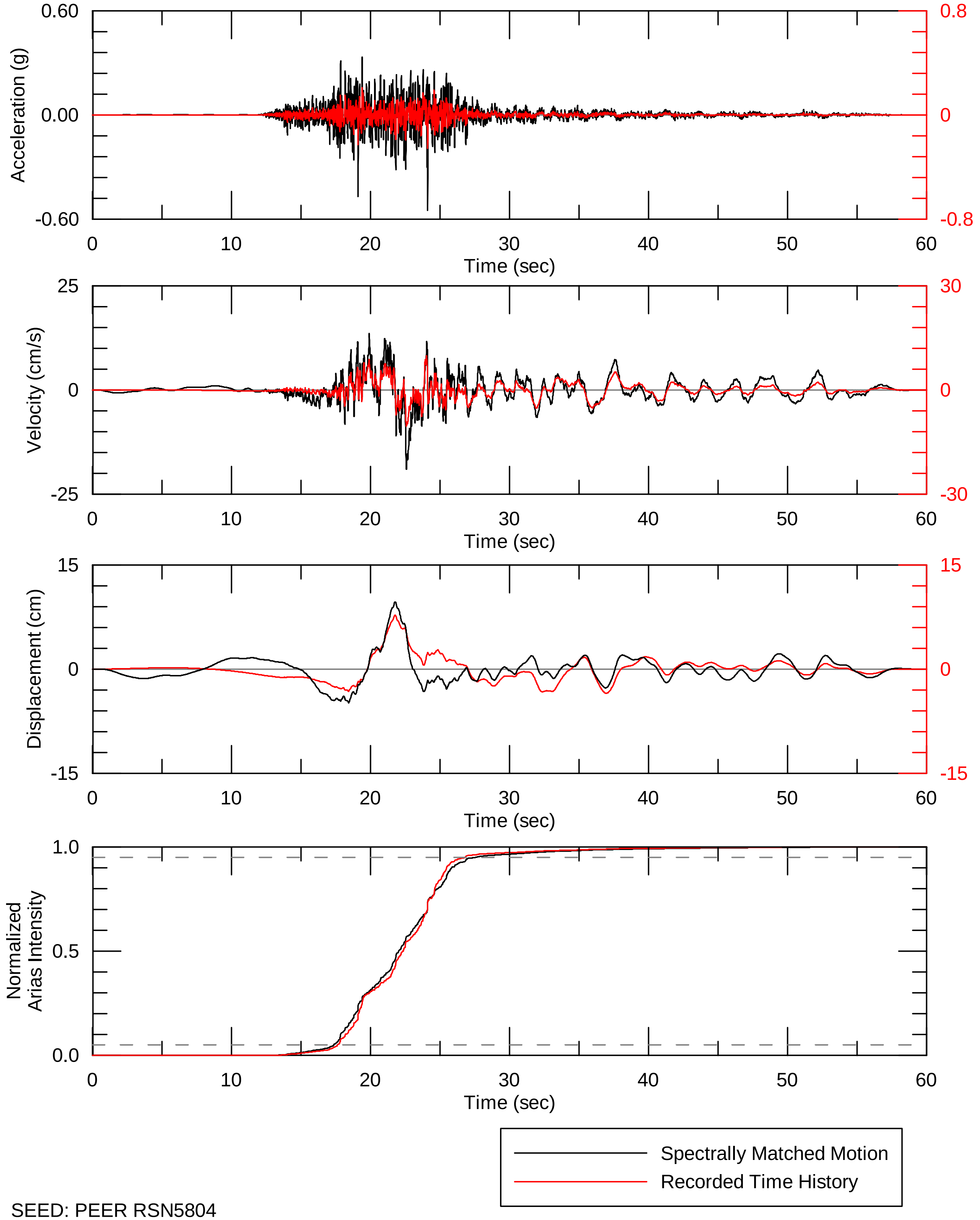
TIME HISTORY SPECTRALLY MATCHED TO
2,500-YEAR RETURN PERIOD UHS
HORIZONTAL TARGET
2002 DENALI - TAPS PUMP STATION #8 (319) SEED

Figure
41



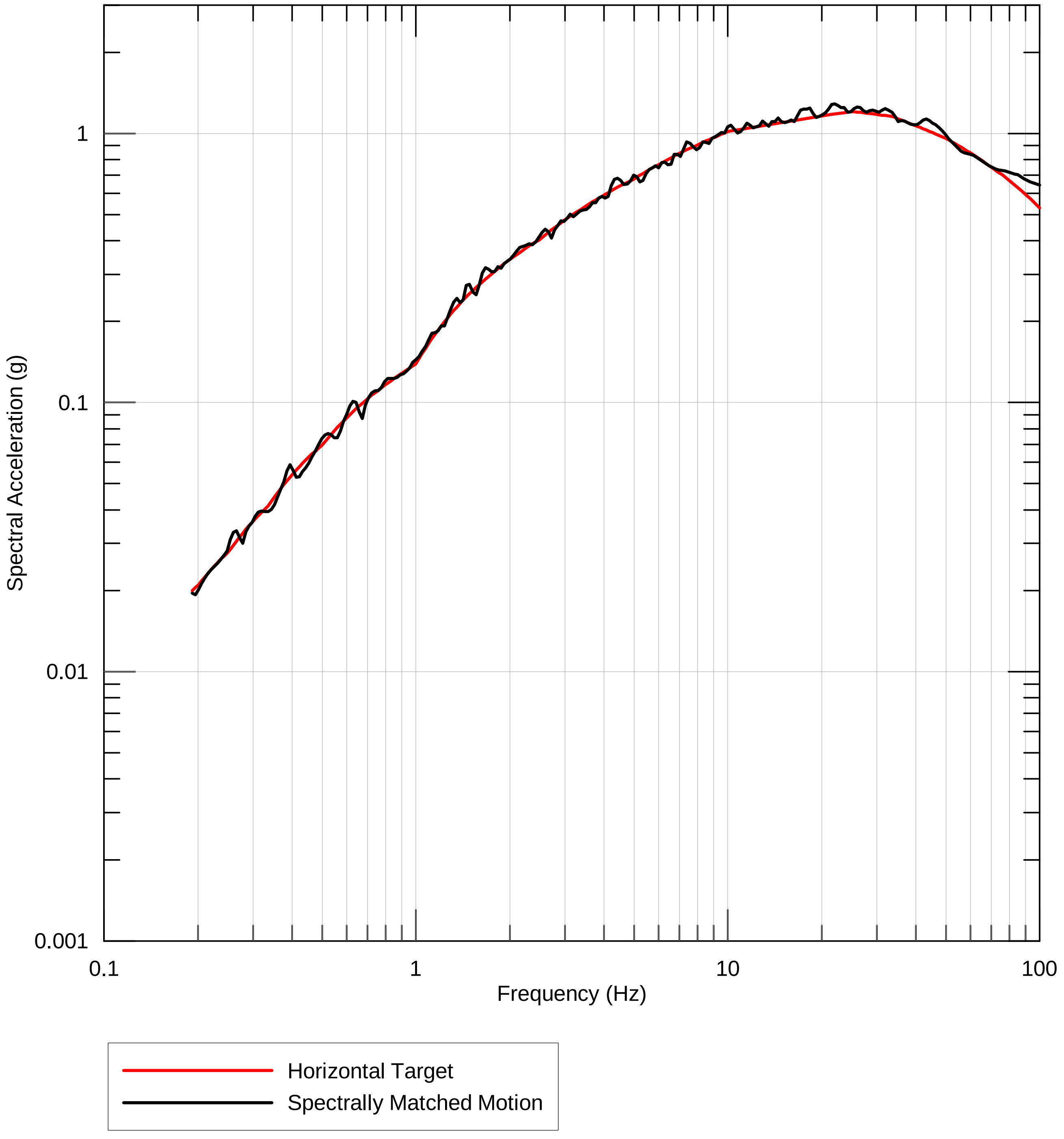
SEED: PEER RSN5804

AECOM	Project No.60442676	RESPONSE SPECTRUM FOR TIME HISTORY SPECTRALLY MATCHED TO 2,500-YEAR RETURN PERIOD UHS HORIZONTAL TARGET 2008 IWATE - YAMAUCHI TSUCHIBUCHI YOKOTE (EW) SEED	Figure 42
	A.B. Brown Generating Station Vectren Corporation		



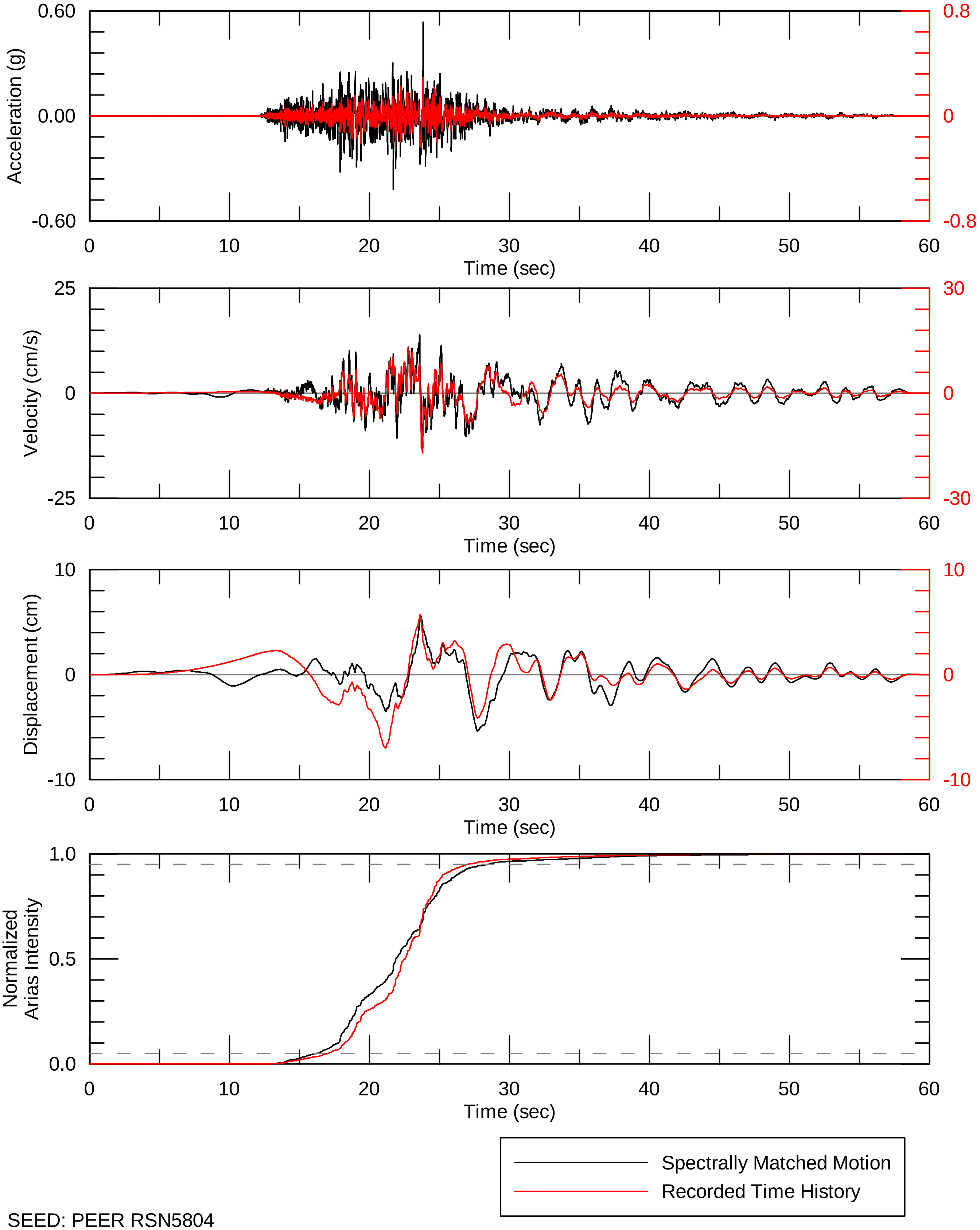
SEED: PEER RSN5804

AECOM	Project No. 60442676 A.B. Brown Generating Station Vectren Corporation	TIME HISTORY SPECTRALLY MATCHED TO 2,500-YEAR RETURN PERIOD UHS HORIZONTAL TARGET 2008 IWATE - YAMAUCHI TSUCHIBUCHI YOKOTE (EW) SEED	Figure 43



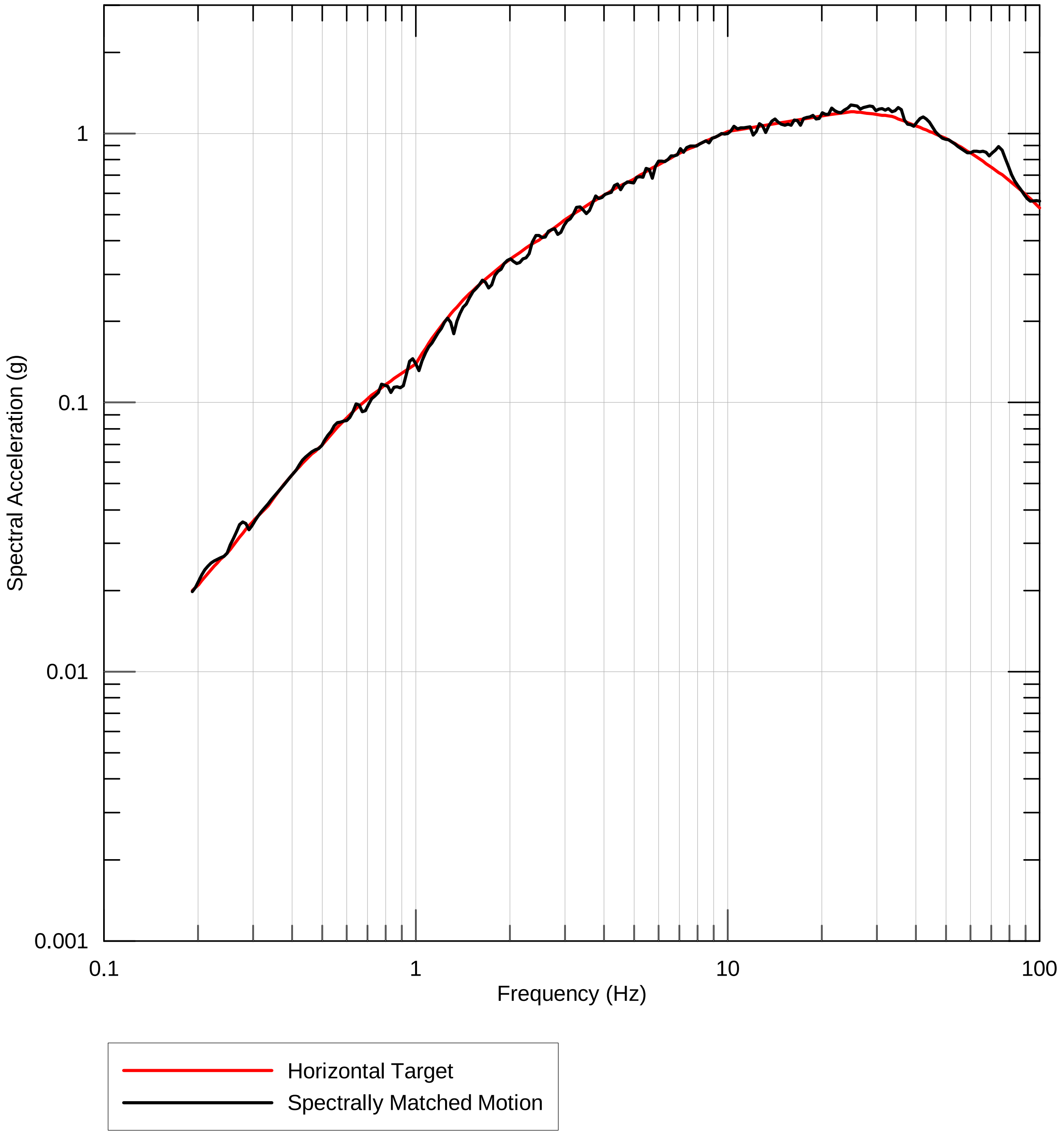
SEED: PEER RSN5804

AECOM	Project No.60442676 A.B. Brown Generating Station Vectren Corporation	RESPONSE SPECTRUM FOR TIME HISTORY SPECTRALLY MATCHED TO 2,500-YEAR RETURN PERIOD UHS HORIZONTAL TARGET 2008 IWATE - YAMAUCHI TSUCHIBUCHI YOKOTE (NS) SEED	Figure 44



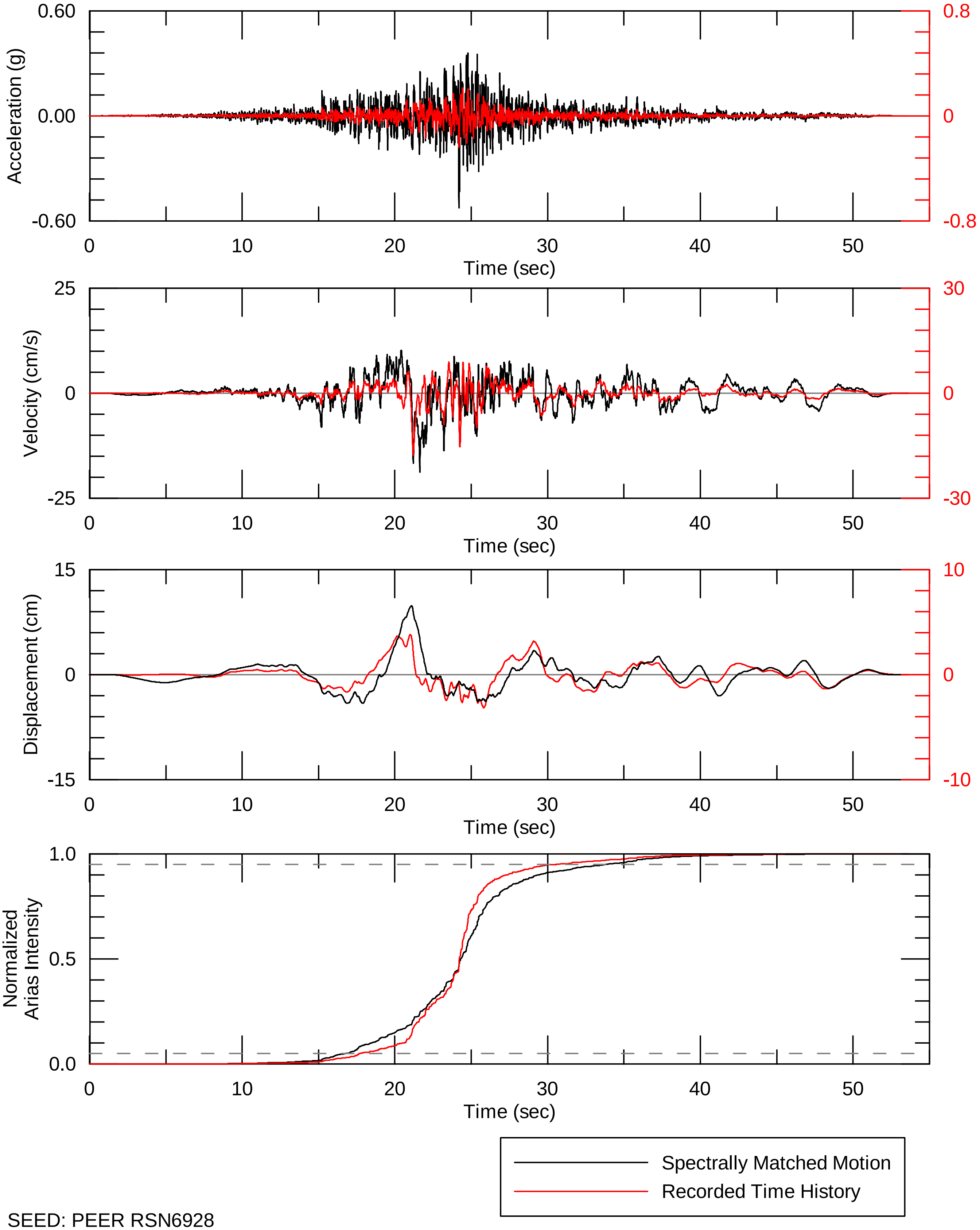
SEED: PEER RSN5804

AECOM	Project No. 60442676	TIME HISTORY SPECTRALLY MATCHED TO 2,500-YEAR RETURN PERIOD UHS HORIZONTAL TARGET 2008 IWATE - YAMAUCHI TSUCHIBUCHI YOKOTE (NS) SEED	Figure 45
	A.B. Brown Generating Station Vectren Corporation		



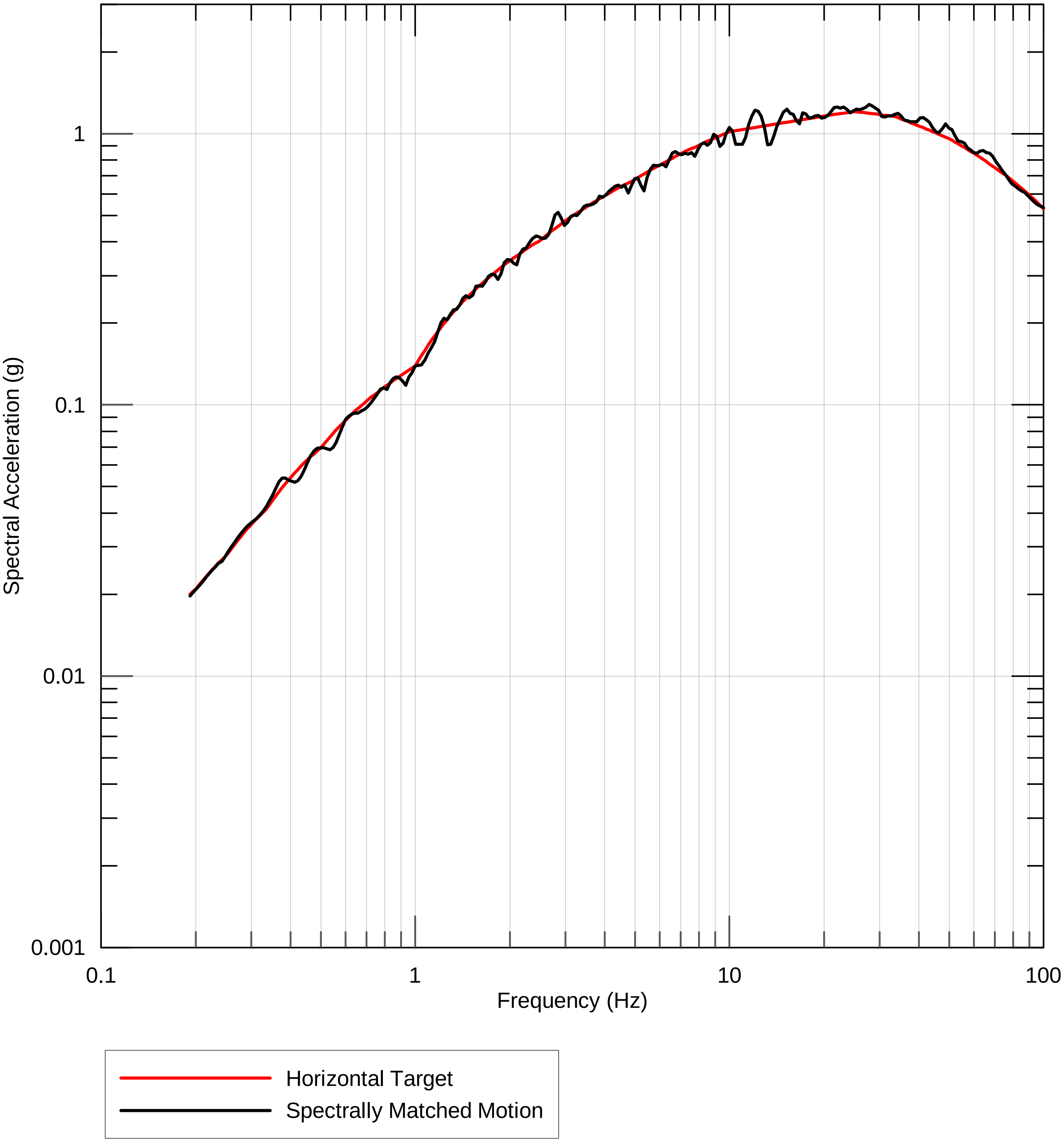
SEED: PEER RSN6928

AECOM	Project No.60442676 A.B. Brown Generating Station Vectren Corporation	RESPONSE SPECTRUM FOR TIME HISTORY SPECTRALLY MATCHED TO 2,500-YEAR RETURN PERIOD UHS HORIZONTAL TARGET 2010 DARFIELD - LPCC (080) SEED	Figure 46



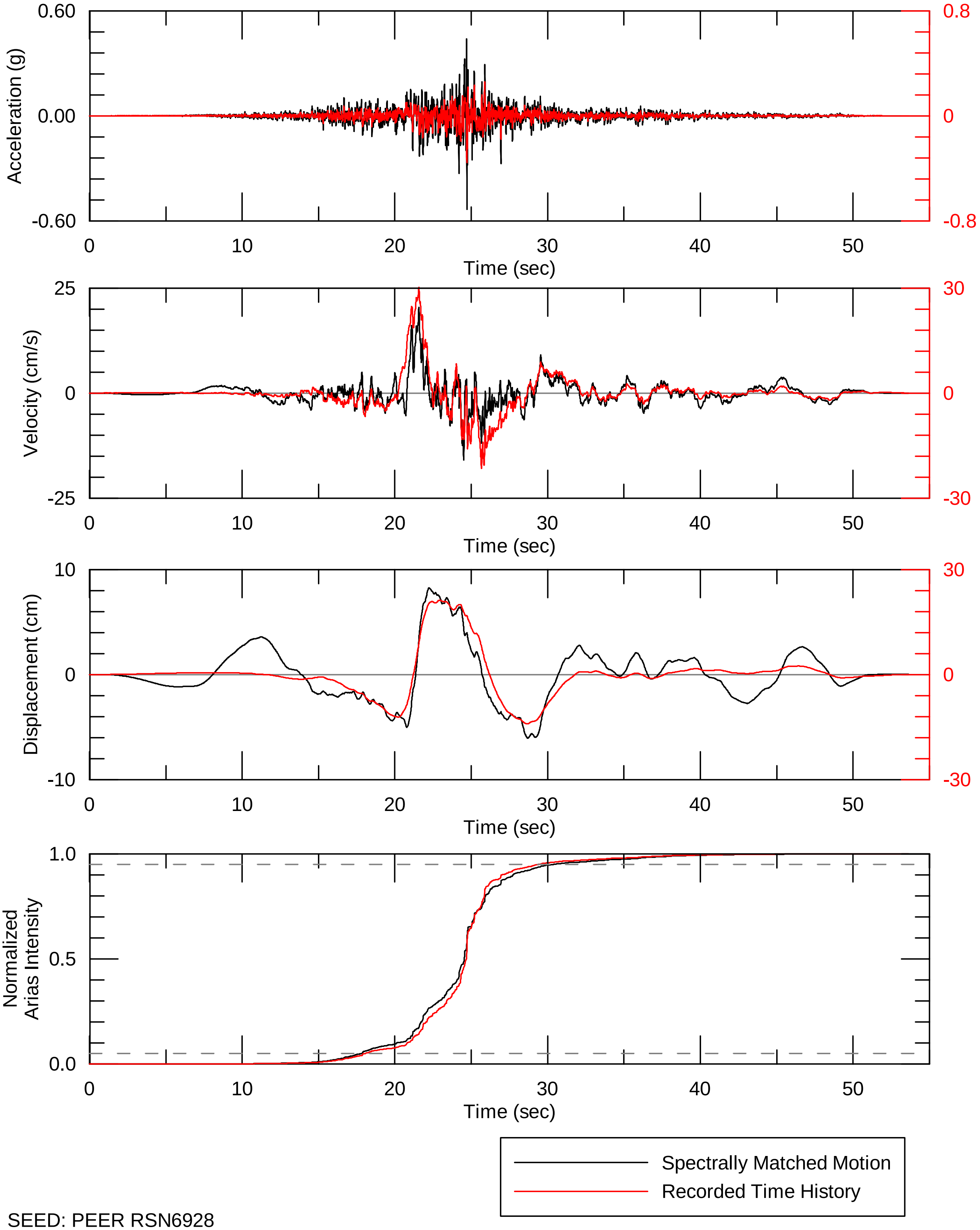
SEED: PEER RSN6928

Project No. 60442676 A.B. Brown Generating Station Vectren Corporation	TIME HISTORY SPECTRALLY MATCHED TO 2,500-YEAR RETURN PERIOD UHS HORIZONTAL TARGET 2010 DARFIELD - LPCC (080) SEED	Figure 47



SEED: PEER RSN6928

AECOM	Project No. 60442676 A.B. Brown Generating Station Vectren Corporation	RESPONSE SPECTRUM FOR TIME HISTORY SPECTRALLY MATCHED TO 2,500-YEAR RETURN PERIOD UHS HORIZONTAL TARGET 2010 DARFIELD - LPCC (170) SEED	Figure 48



SEED: PEER RSN6928

AECOM	Project No. 60442676	TIME HISTORY SPECTRALLY MATCHED TO 2,500-YEAR RETURN PERIOD UHS HORIZONTAL TARGET 2010 DARFIELD - LPCC (170) SEED	Figure 49
	A.B. Brown Generating Station Vectren Corporation		

Appendix H
Dynamic Response Analysis Calculations

Calculation Notes **AECOM** **Appendix H**

Job	**A.B. Brown Generating Station – Ash Pond System CCR Certification Report**	Project No.	**60442627**	Sheet	**1** of **8**
Description	**Appendix H**	Computed by	**VKG**	Date	**09/02/2016**
	Dynamic Response Analysis (QUAD-4)	Checked by	**ACI**	Date	**09/12/16**

This package presents the pertinent results of the Probabilistic Seismic Hazard Analysis (PSHA) performed for the Vectren A.B. Brown Generating Station site (complete PSHA report is provided in **Appendix G**) and the methodology and results of the dynamic response analysis performed for the Lower Dam. These analyses were performed to estimate ground motion parameters and the resulting cyclic shear stresses within the various strata that can be expected during the design earthquake event. The design earthquake is defined within the CCR Rule as an event that has 2% probability of exceedance in 50 years (approximately 2500-year return period). The resulting cyclic shear stresses are utilized in the liquefaction triggering analyses presented in **Appendix I**.

I. Results of Probabilistic Seismic Hazard Analysis

As presented in **Appendix G**, AECOM conducted a site-specific probabilistic seismic hazard analysis (PSHA) for the A.B. Brown Generating Station. The PSHA results are used to compute a 2,500-yr return period Uniform Hazard Spectrum (UHS) and develop horizontal acceleration time histories consistent with the hard rock 2,500-yr UHS. The site-specific acceleration time histories are then used in site response analysis to estimate seismic-induced shear stresses for use in liquefaction analysis.

A.B. Brown Generating Station is located in southwestern Indiana, within the Illinois Basin Extended Basin Zone, adjacent to the Wabash Valley Seismic Zone and about 140 km northeast of the New Madrid Seismic Zone (NMSZ). The site is in a region that has exhibited a moderate level of historical seismicity. There have been seven known earthquakes larger than moment magnitude (M) 5.0 within 200 km of the site. However, the region is capable of experiencing strong ground motions from moderate to large earthquakes (M > 6) particularly from the Wabash Seismic Zone and the New Madrid Seismic Zone to the southwest of the site. The preexisting structures formed in earlier tectonic settings are still capable of generating seismicity that can pose a hazard to the region. This seismicity has included several large historical earthquakes in the area (M > 7), e.g., the 1811 and 1812 New Madrid earthquakes. The Wabash Valley has historically been seismically active with several earthquakes of M 4.5 and larger (**Figure H-1**). Hence, the site has been strongly shaken numerous times after the 1811-1812 earthquakes.

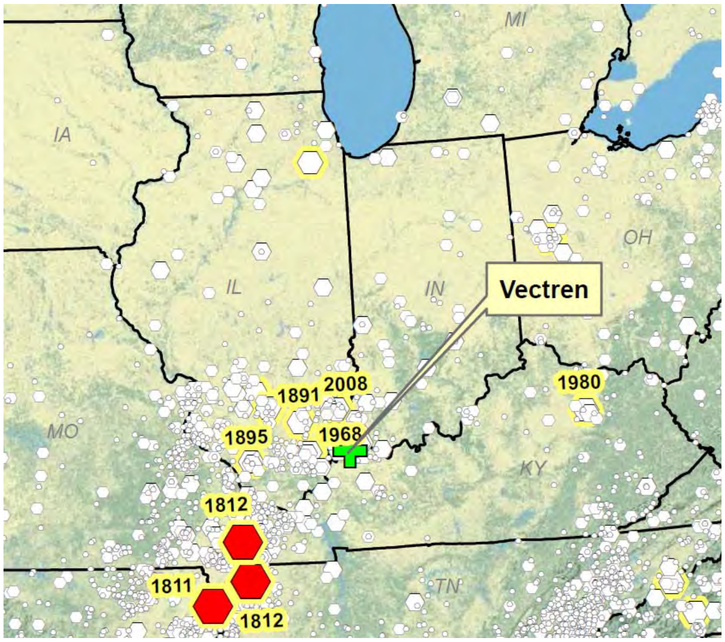


Figure H-1: Historical Seismicity Regional to the Site

The design ground motions were developed in two steps: 1) earthquake parameters; and 2) time histories. Parameters were developed including magnitude, distance, style of faulting, response spectra, and Arias Intensity for the current study. All seismically capable faults in the project region were considered. Near field and directivity effects were also considered. Response spectra were established for both hard rock (Class A rock, with shear wave velocity greater than 9,200 ft/s) and firm rock (Class B rock, with shear wave velocity between 2,500 and 9,200 ft/s). Hard rock is anticipated to be at great depth below the site. Given this, ground motions consistent with firm rock were obtained by adjusting the hard rock motions to firm rock using the generic amplification factors developed by David Boore (Frankel et al., 1996). These factors are used in the development of the National Seismic Hazard Maps (NSHMs) by the USGS.

Four sets of time histories were developed for each design spectrum. The time histories represent the site-specific ground motions associated with the controlling near-field or far-field earthquake event, and consider the magnitude, distance, and Arias Intensity. Each acceleration time history was developed from a pair of orthogonal horizontal components that was matched to the fault-normal and fault-parallel components of the design spectra. The seed motion records were selected from available strong-motion recordings obtained during previous earthquakes that have occurred in similar tectonic environments. The characteristics include earthquake magnitude, faulting mechanism, source-to-site distance, and site conditions. A time-domain approach was used to modify the natural recordings and to generate time histories compatible with the respective target response spectrum. The response spectra for the resolved acceleration time histories were developed to closely match the spectral amplitudes of the smooth target spectrum through the period range of interest. The time histories were then used as input motions for the dynamic response analyses, as discussed in Section 4.2 below.

Uniform Hazard response spectra from the PSHA are summarized in **Tables H-1 and H-2** below. An example time history (Time History 4) resulting from the analysis is provided in **Figure H-2.** The complete results of the PSHA are included in **Appendix G** of this report.

Table H-1: Uniform Hazard Response Spectrum for Hard Rock – A.B. Brown Generating Station

Period	Spectral Acceleration (g)
0.01	0.35
0.04	0.73
0.10	0.58
0.20	0.39
0.40	0.24
1.00	0.10
2.00	0.058

Table H-2: Uniform Hazard Response Spectrum for Firm Rock – A.B. Brown Generating Station

Period	Spectral Acceleration (g)
0.01	0.53
0.02	0.96
0.03	1.16
0.04	1.21
0.10	1.02
0.20	0.68
0.40	0.40
1.0	0.14
2.0	0.07
3.0	0.041
4.0	0.028

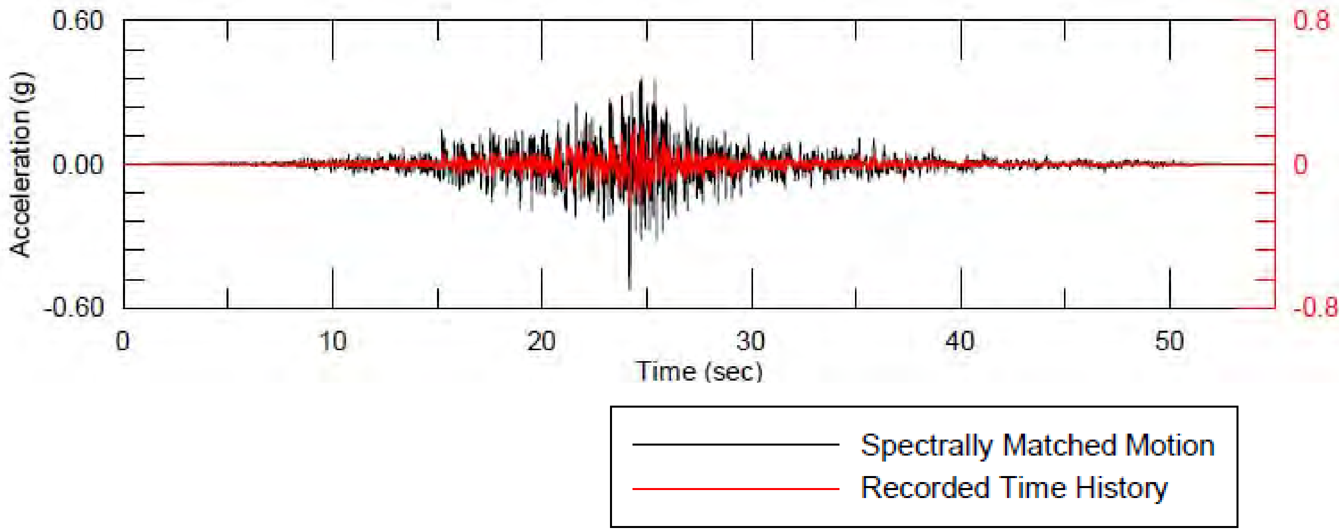


Figure H-2 – Acceleration Record of Time History 4 (With Seed Motion Superposed)

The major contributors to the hazard at the site for a return period of 2,500 years are the IBEB zone and the Wabash Valley zone. The near-site distributed seismicity corresponding to the IBEB contributes just over 70 percent of the peak ground acceleration (PGA) hazard at 2,500-year return period, and has an associated earthquake moment magnitude between M 5.0 and M 6.0. At longer periods (0.4 and 1.0 sec SA), the relative contribution of the Wabash Valley and New Madrid zones increases to up to 75 percent of the hazard at 2,500 years, with much higher associated moment magnitude (M 7.0 to M 8.25). This is illustrated in **Figures H-3 and H-4**, which portray the deaggregation of the PGA and 1.0 sec spectral acceleration hazard by magnitude and distance, respectively. **Table H-3** summarizes the modal magnitude (M*) and source distance (D*), which represent the highest contributors to the hazard for the design return period.

Job	**A.B. Brown Generating Station – Ash Pond System CCR Certification Report**	Project No.	**60442627**	Sheet	**5** of **8**
Description	**Appendix H**	Computed by	**VKG**	Date	**09/02/2016**
	Dynamic Response Analysis (QUAD-4)	Checked by	**ACI**	Date	**09/12/16**

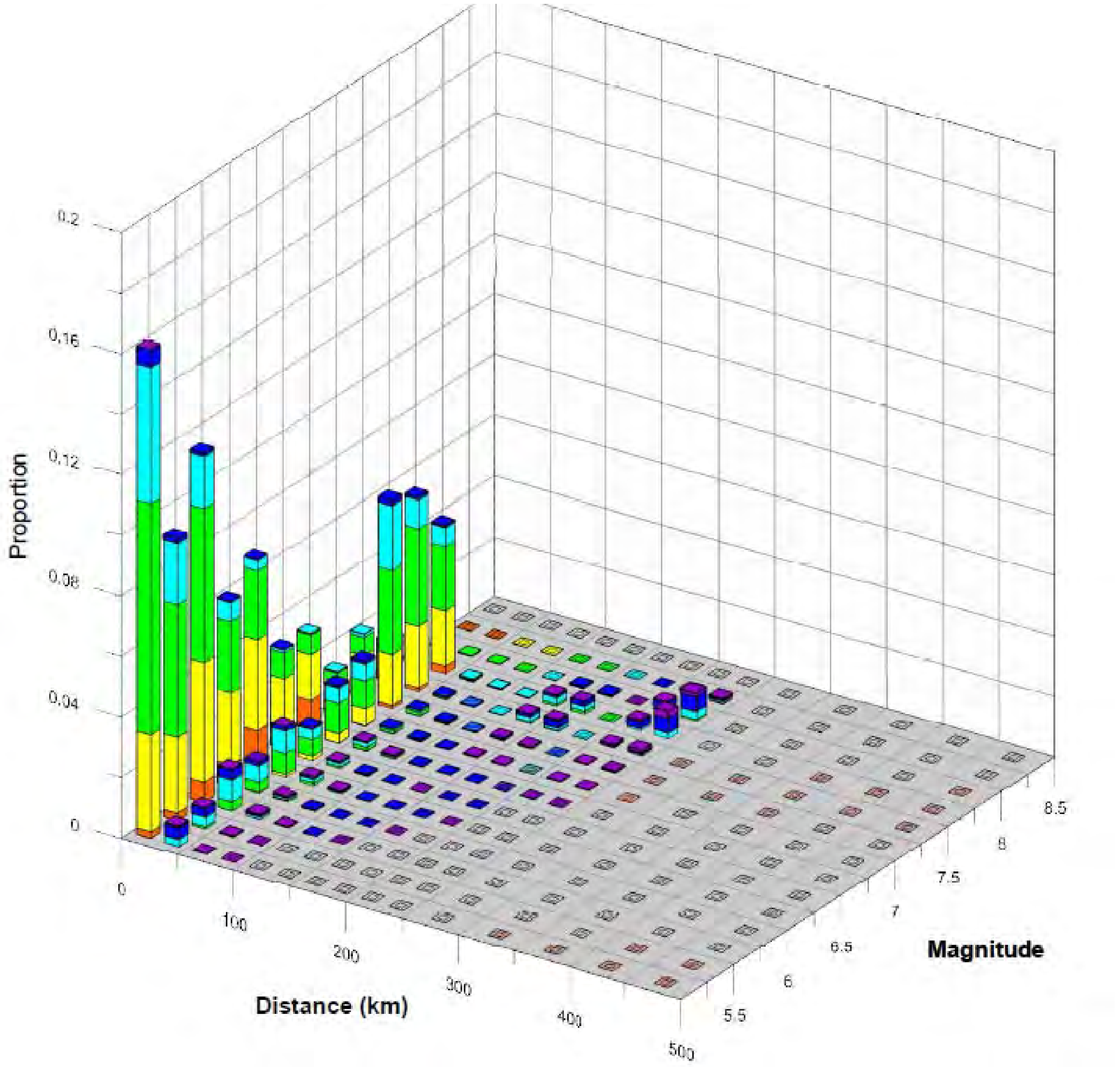


Figure H-3: Deaggregation for Peak Ground Acceleration

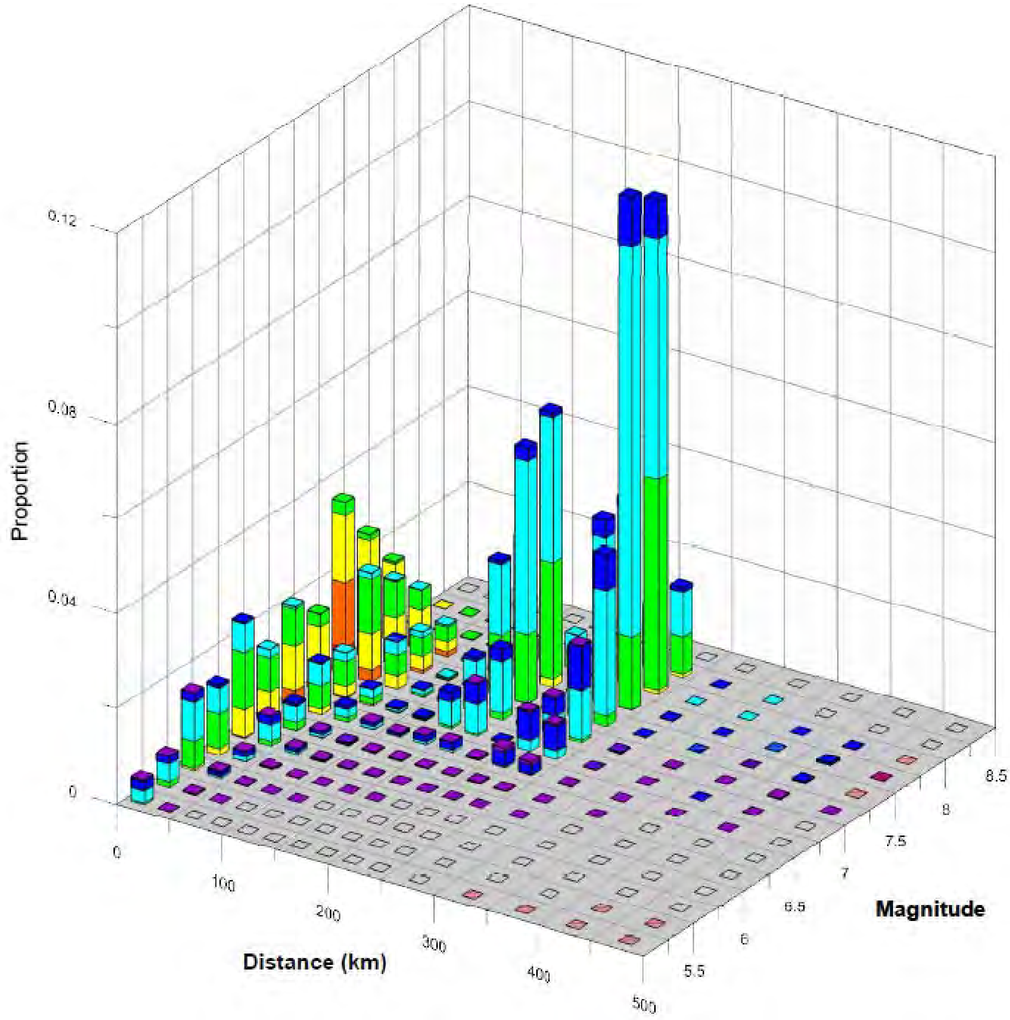


Figure H-4: – Deaggregation for 1.0-sec Spectral Acceleration

Table H-3: Modal Earthquake Magnitude and Source Distance

Period	Modal Magnitude (M*)	Modal Source Distance (D*)
PGA	5.1	12.5 km
0.4 (bimodal)	7.1 7.6	12.5 km 238 km
1.0	7.6	238 km

II. Methodology for Dynamic Response Analysis (QUAD-4 Analysis)

The dynamic response (calculation of the earthquake-induced shear stresses) of the A.B. Brown Station was evaluated by analyzing a typical cross-section through the dam using the most recent version of the finite element program QUAD4M (Hudson et al. 1994). This is a modified version of the program QUAD4, originally developed by Idriss, et al. (1973). The dynamic response analysis was useful for more precisely estimating the amplification / attenuation characteristics of the dam structure and local soils to the design rock motions and to estimate the earthquake-induced stresses within the embankment and foundation. Input to the dynamic response analyses includes the acceleration time histories developed as part of the PSHA for the A.B. Brown Station. Earthquake-induced shear stresses computed using QUAD4 were used directly in the updated SPT-based liquefaction triggering analysis.

The QUAD4M program uses a two-dimensional, dynamic finite-element formulation that utilizes equivalent-linear, strain-dependent modulus and damping properties. The program performs a time-domain analysis that allows variable damping throughout the model and uses an iterative process to approximate the nonlinear behavior of soil. Shear moduli and damping ratios are estimated initially for each element in the model, and the system is analyzed using those properties. After each iteration, values of the effective shear strain are computed and the modulus and damping values are updated to correspond to the computed strain level for each element. The analysis iterations are repeated until compatibility between moduli, damping, and strain levels is achieved in all elements.

III. Geometry

The analysis was performed for a cross-section oriented along the approximate center of the dam (north-south) – specifically, Cross-Section B (see **Appendix F** of this report). The cross section was modeled as a two-dimensional plane-strain finite element mesh with input motions applied in the transverse direction at the base of the mesh.

Separate models were created for the cross-section configuration as it existed prior to construction of the stabilizing soil buttress and the configuration after construction.

IV. Dynamic Material Properties

Dynamic response analysis of the model required characterization of the shear modulus (G), Poisson's ratio (v), and damping characteristics of embankment and foundation materials. To consider the variation in dynamic shear modulus with strain, the shear modulus is commonly represented in terms of its value at small strains (Gmax) and the variation in the ratio (G/Gmax) with shear strain, which is referred to as a modulus reduction relationship. Likewise, the variation in hysteretic damping with strain is represented by a damping relationship. For the silty clay embankment and silty clay foundation soils, the shear modulus reduction and damping relationships by Vucetic and Dobry (1991) were selected based on the index characteristics of the materials and experience. The average modulus-reduction and lower-bound damping relationships for sands by Seed and Idriss (1970) were selected to represent the silt foundation layer.

An estimate of the shear wave velocity of each soil stratum of the cross-section subsurface profile was developed using the average seismic shear wave velocity measurements obtained during the CPT testing program. Shear wave velocity measurements are summarized in **Appendix E**, and the complete CPT data report is provided in **Appendix C**. The shear wave velocities were used to evaluate the dynamic shear modulus at small strains of the embankment and foundation materials, and the corresponding values of Poisson’s ratio. The shear modulus at small strains was obtained from the measured shear wave velocity through the expression:

$$G_{max} = \rho V_s^2$$

where: Vs is the shear wave velocity and ρ is the mass density of the material.

V. Analysis Results

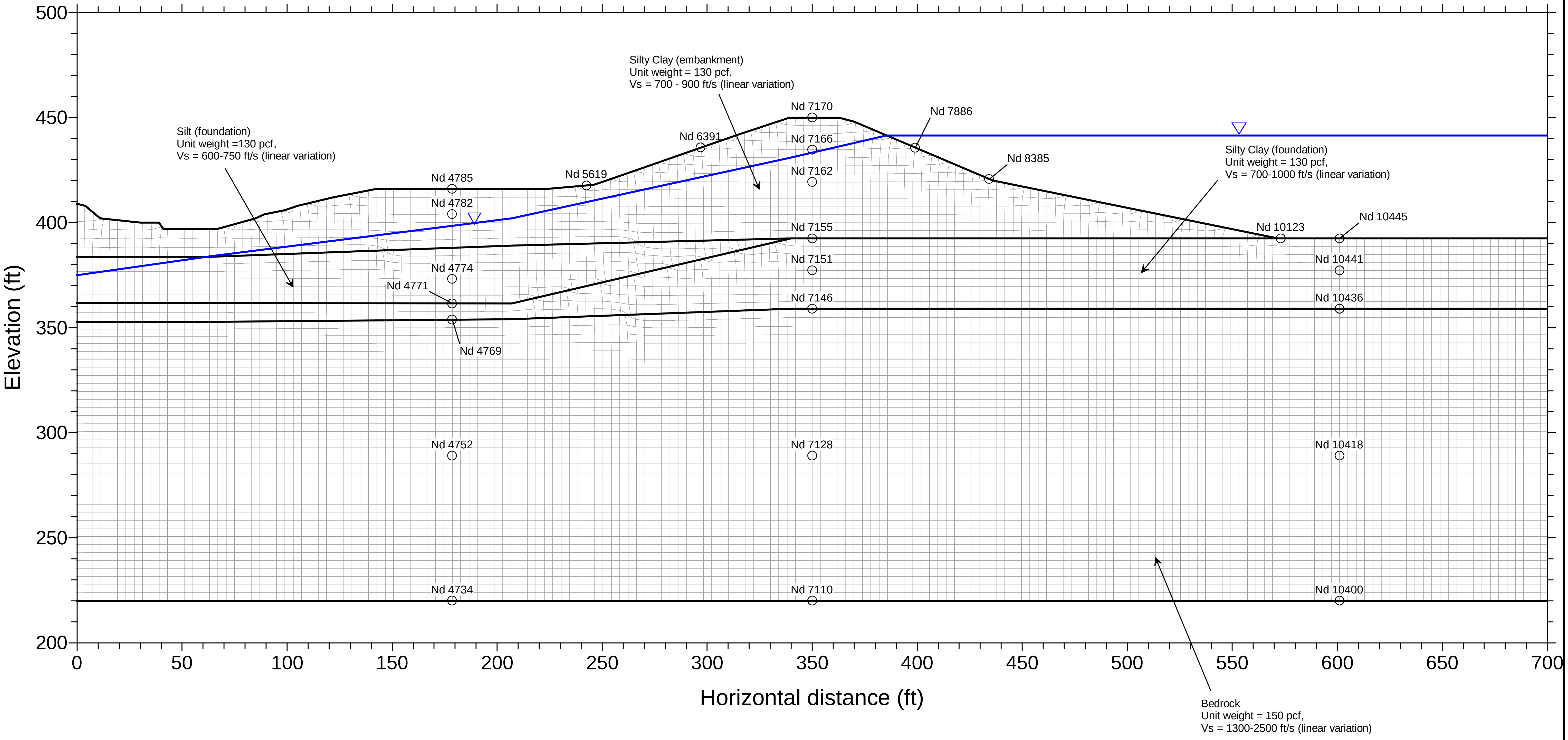
The QUAD4M model incorporates a large number of finite elements making up the meshing for the whole cross-section. Seismically induced shear stresses are calculated for each element, and 2-dimensional plots of shear stress contours within the cross-section are generated. These plots are provided for each of the four time histories analyzed in the attachment. Separate sets of plots are provided for the pre- and post-buttress configuration. In each set, estimated peak nodal accelerations are presented in Figures 2-1 to 2-4 of this attachment. Further, the peak cyclic shear stresses (in ksf) estimated for each time history are shown in Figures 3-1 to 3-4 of this attachment.

VI. References

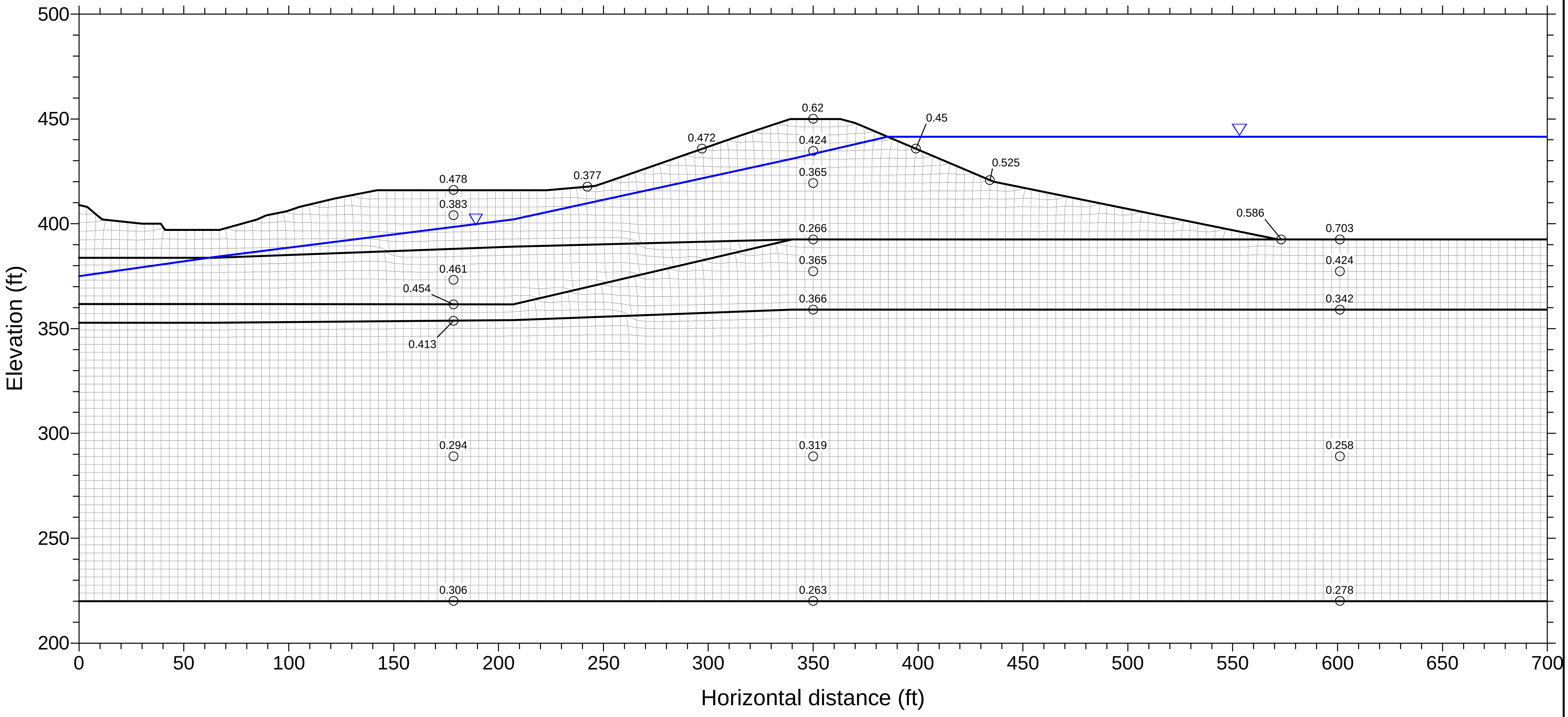
1. Frankel, A., Mueller, C., Barnhard, T., Perkins, D., Leyendecker, E.V., Dickman, N., Hanson, S.,and Hopper, M., 1996, National Seismic Hazard Maps; documentation: U.S. Geological Survey Open-File Report 96-532, 110 p.
2. Hudson, M., Idriss, I.M., and Beikae, M. (1994). User’s Manual for QUAD4M, Center for Geotechnical Modeling, Department of Civil and Environmental Engineering, University of California, Davis, California
3. Idriss, I.M., Lysmer, J., Hwang, R., and Seed, H.B., 1973, QUAD4: A computer program for evaluating the seismic response of soil structures by variable damping finite-element procedures: Earthquake Engineering Research Institute, University of California, Berkeley, Report 73-16.
4. Seed, H.B., and Idriss, I.M., (1970). "Soil Moduli and Damping Factors for Dynamic Response Analysis", Earthquake Engineering Research Center, College of Engineering, University of California, Berkley, California.

QUAD4M RESULTS

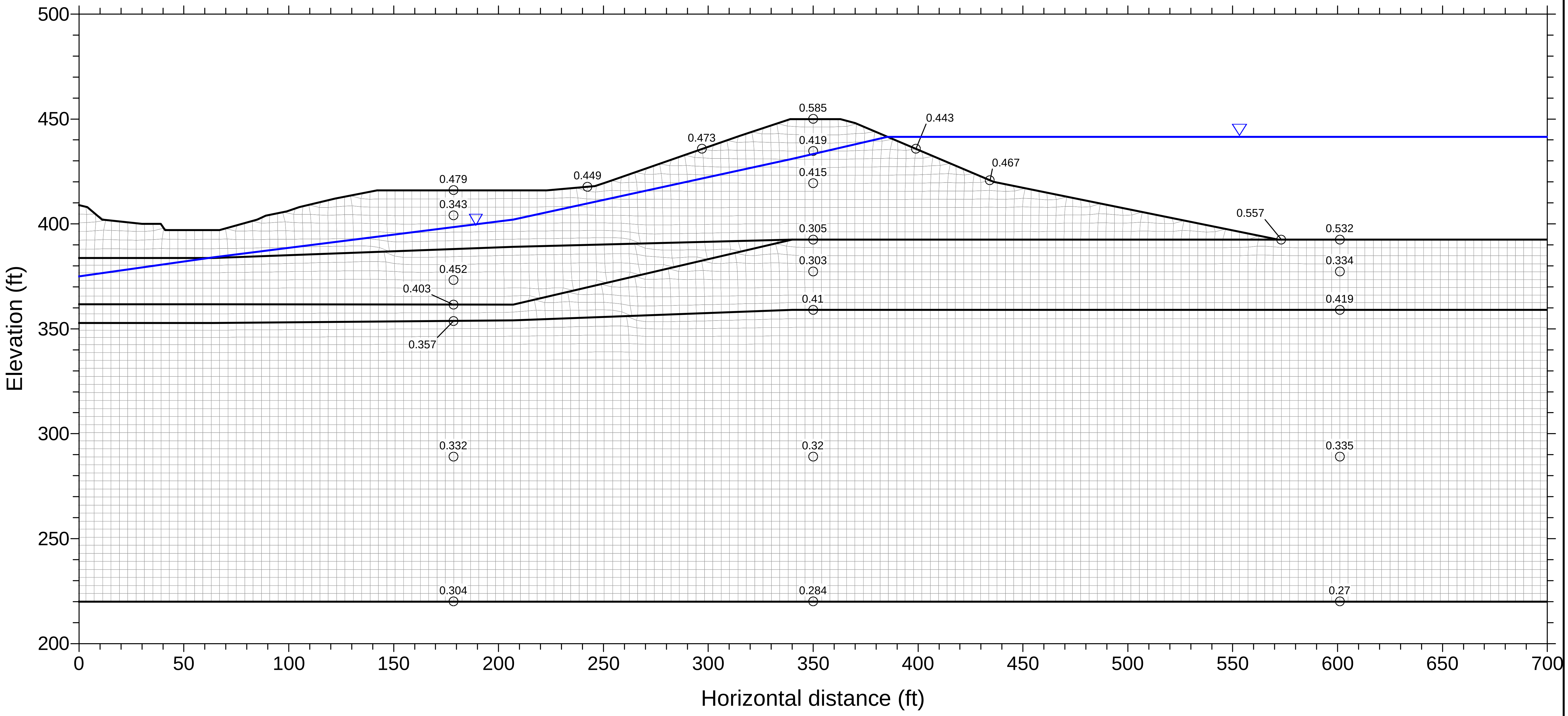
PRE-BUTTRESS CONSTRUCTION MODEL



Project No. 60442676	Vectren A.B. Brown Station CCR Study	A.B. Brown Station Section B QUAD4 Model and Output Nodes	Figure 1
	AECOM		



Project No. 60442676	Vectren A.B. Brown Station CCR Study	A.B. Brown Station Section B QUAD4 Output - Nodal Acceleration (g) Base Motion: TH1	Figure 2-1
AECOM			



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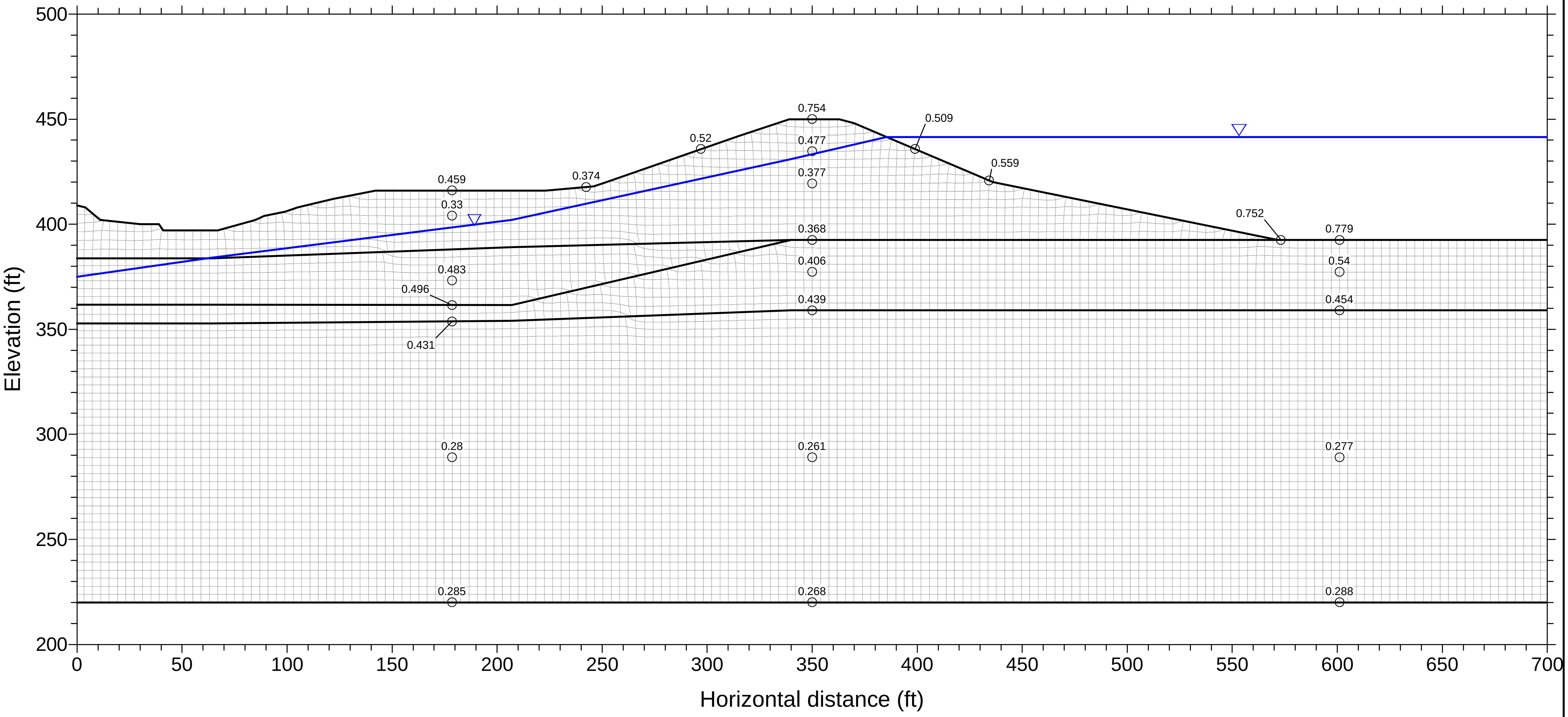
	AECOM		



Project No. 60442676	Vectren A.B. Brown Station CCR Study	A.B. Brown Station Section B QUAD4 Output - Nodal Acceleration (g) Base Motion: TH3	Figure 2-3
	AECOM		

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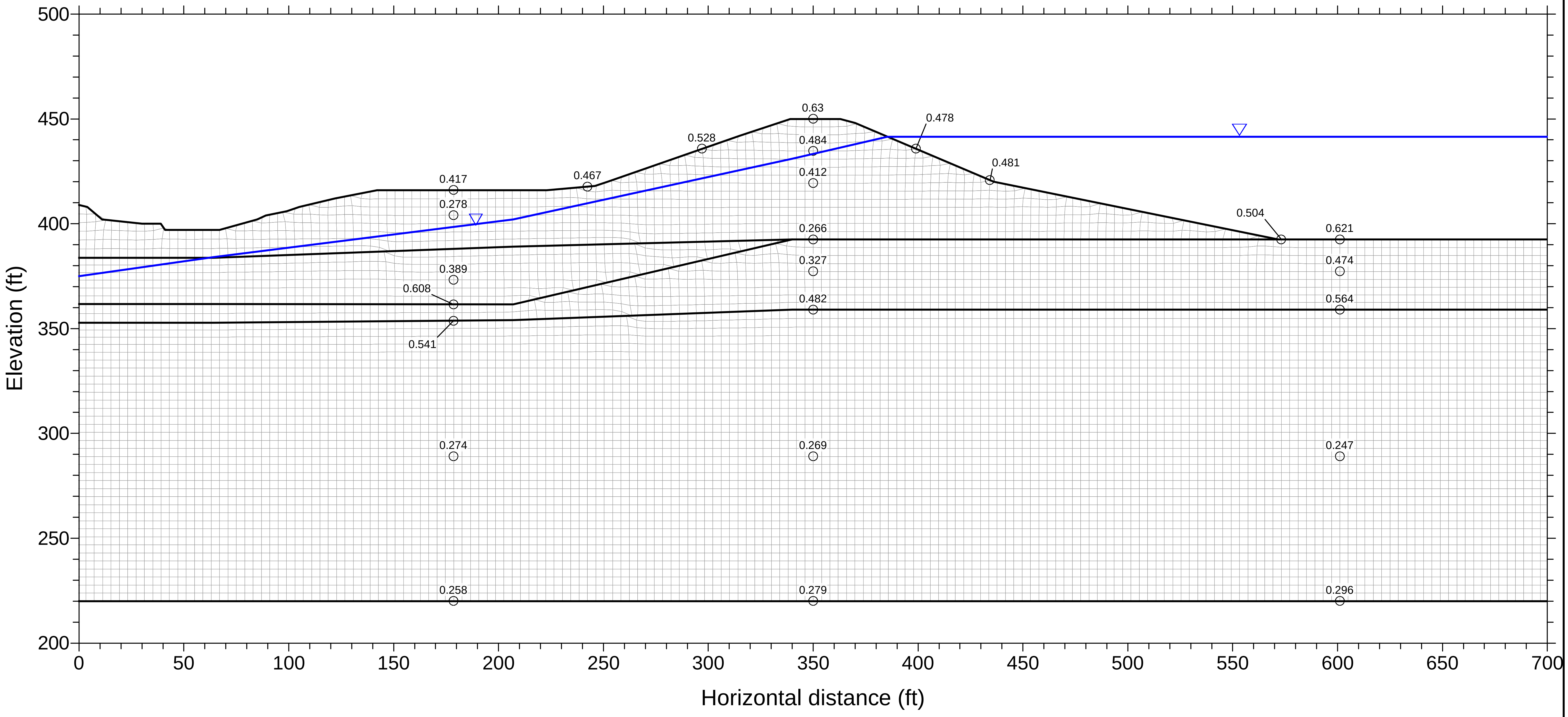

Project No. 60442676	Vectren A.B. Brown Station CCR Study	A.B. Brown Station Section B QUAD4 Output - Nodal Acceleration (g) Base Motion: TH4	Figure 2-4
AECOM			

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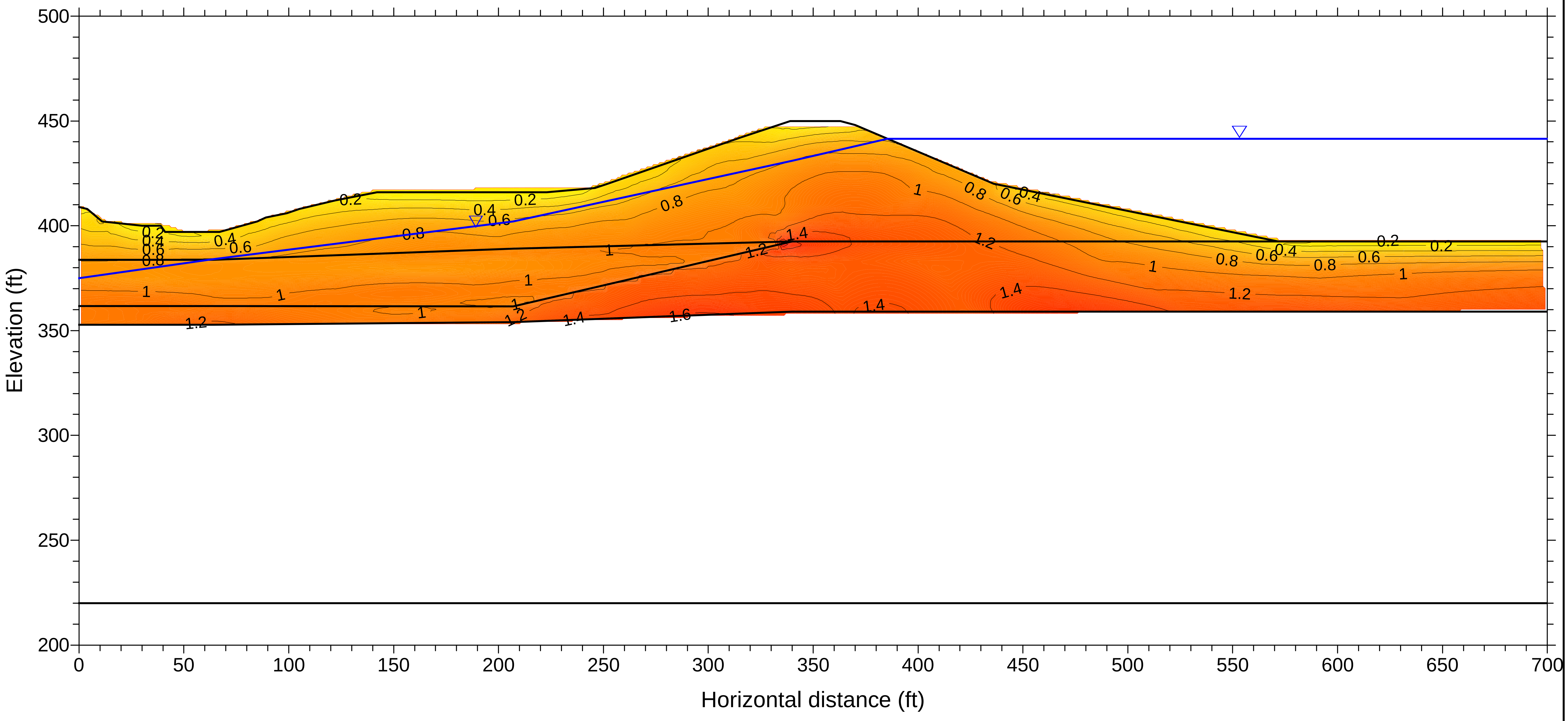

Project No.
60442676

Vectren
A.B. Brown Station
CCR Study

AECOM

A.B. Brown Station Section B
QUAD4 Output - Peak Shear Stress (ksf)
Base Motion: TH1

Figure
3-1

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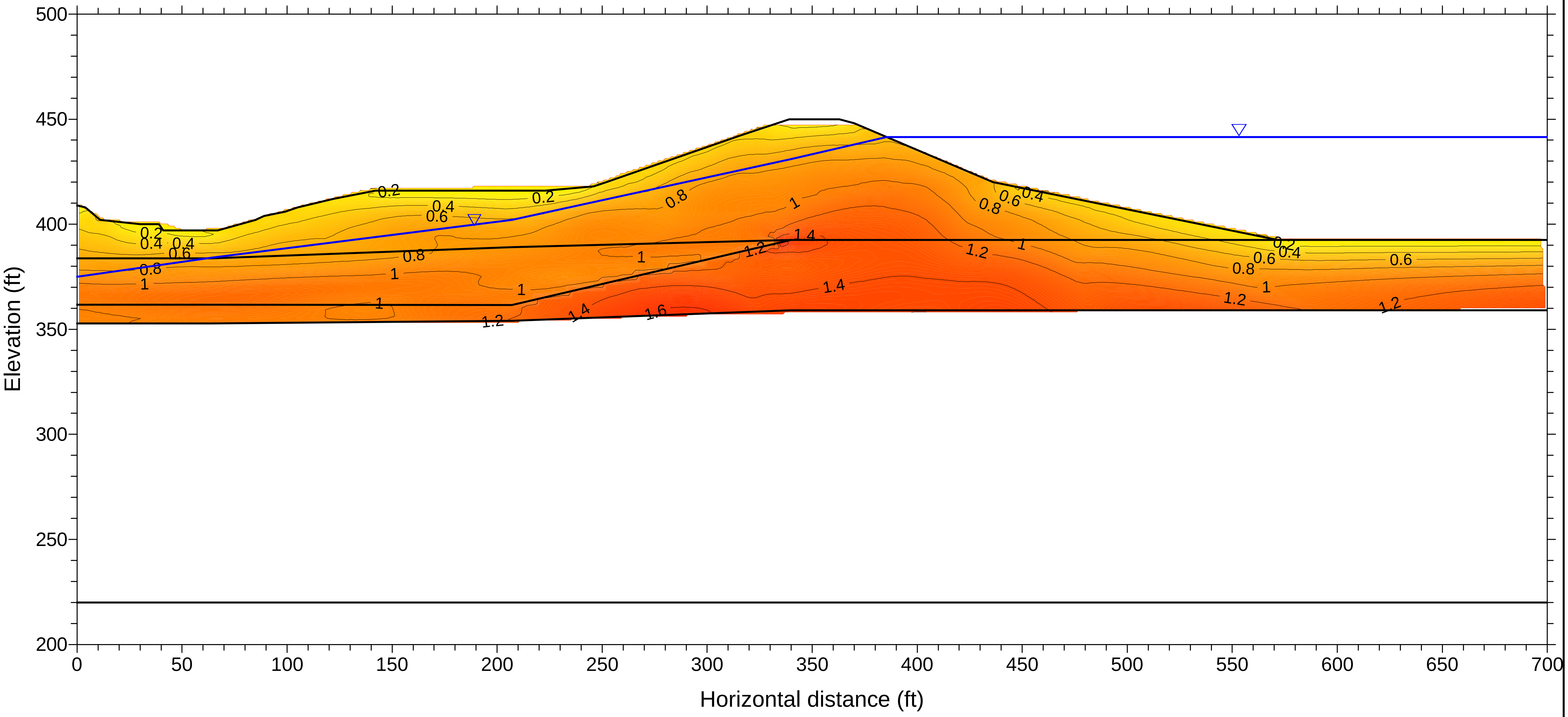
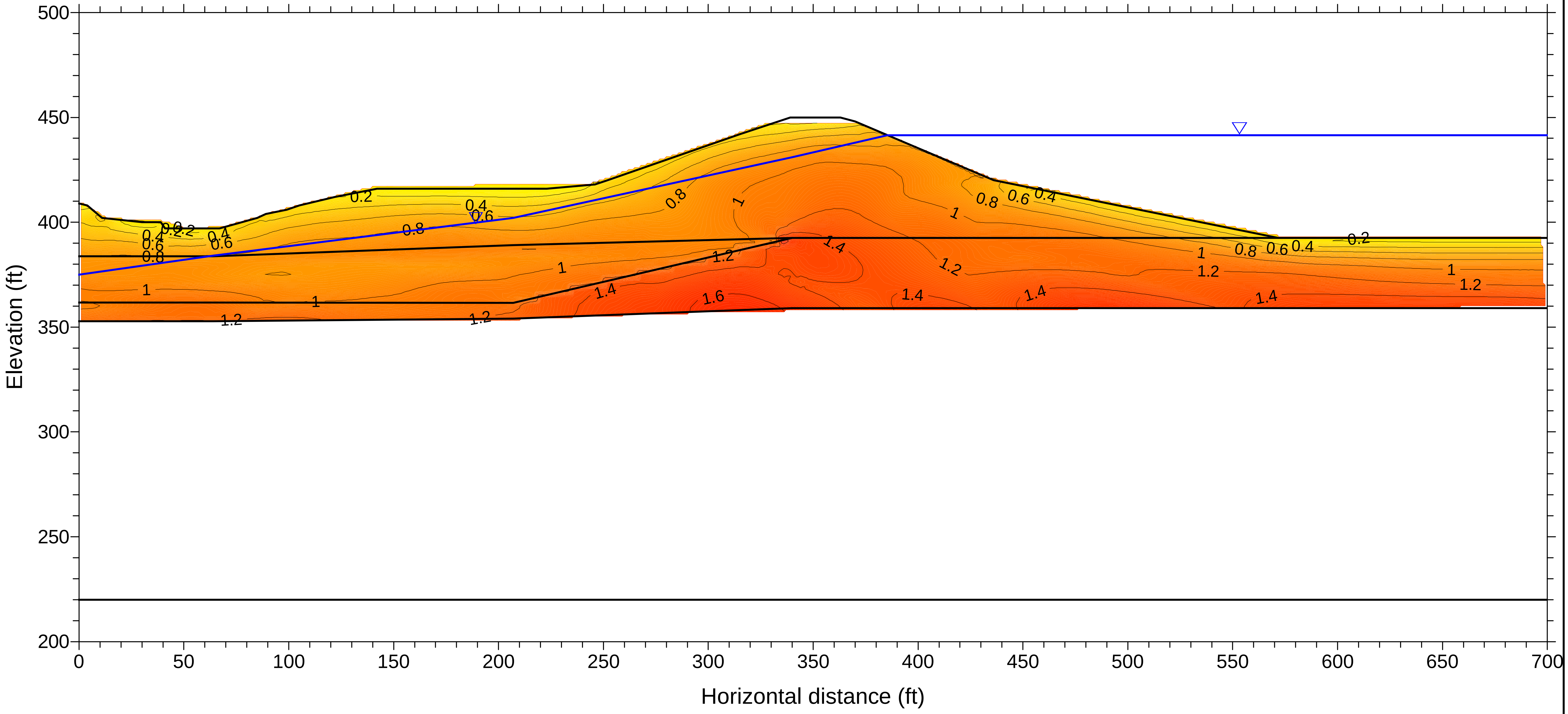


Project No. 60442676	Vectren A.B. Brown Station CCR Study	A.B. Brown Station Section B QUAD4 Output - Peak Shear Stress (ksf) Base Motion: TH2	Figure 3-2
	AECOM		



Project No.
60442676

Vectren
A.B. Brown Station
CCR Study

AECOM

A.B. Brown Station Section B
QUAD4 Output - Peak Shear Stress (ksf)
Base Motion: TH3

Figure
3-3

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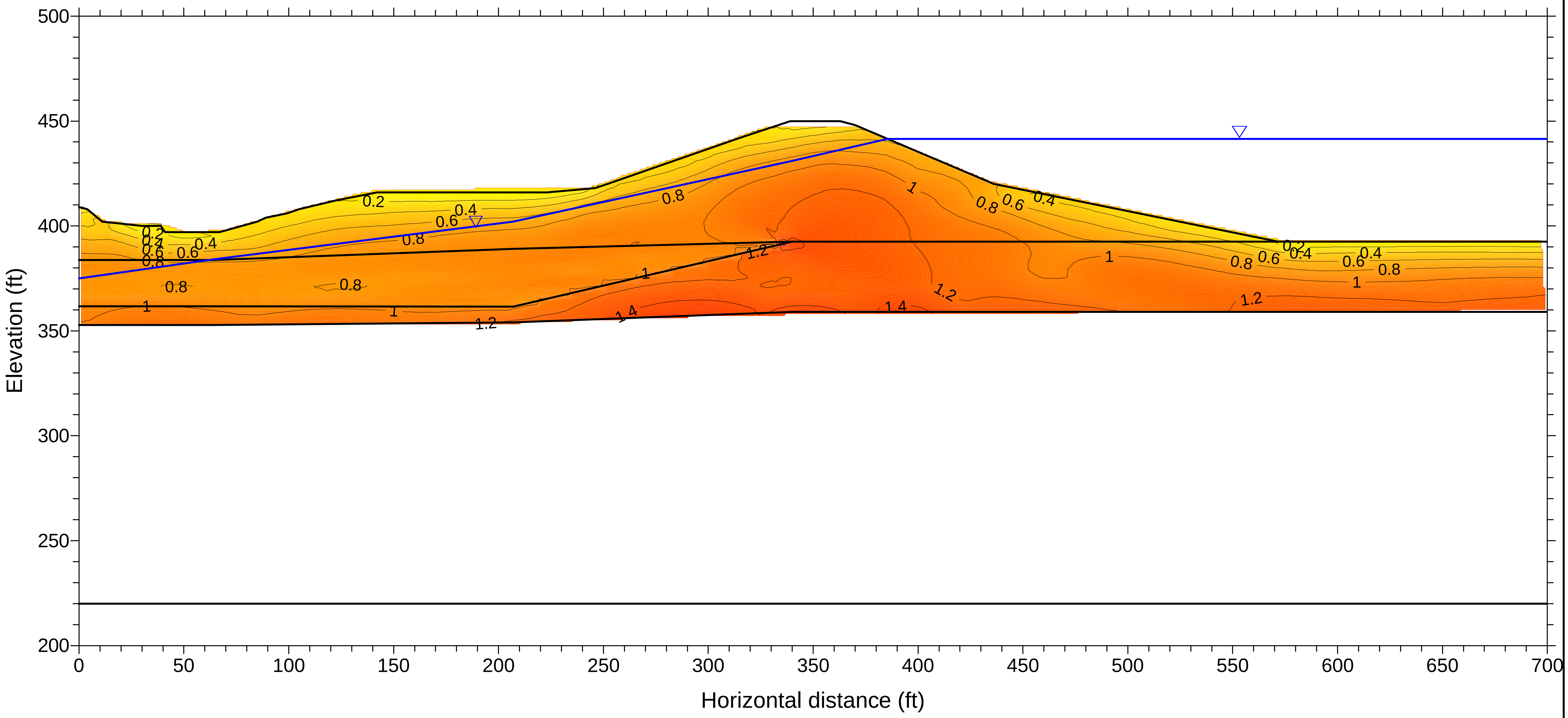

Project No. 60442676	Vectren A.B. Brown Station CCR Study	A.B. Brown Station Section B QUAD4 Output - Peak Shear Stress (ksf) Base Motion: TH4	Figure 3-4
	AECOM		

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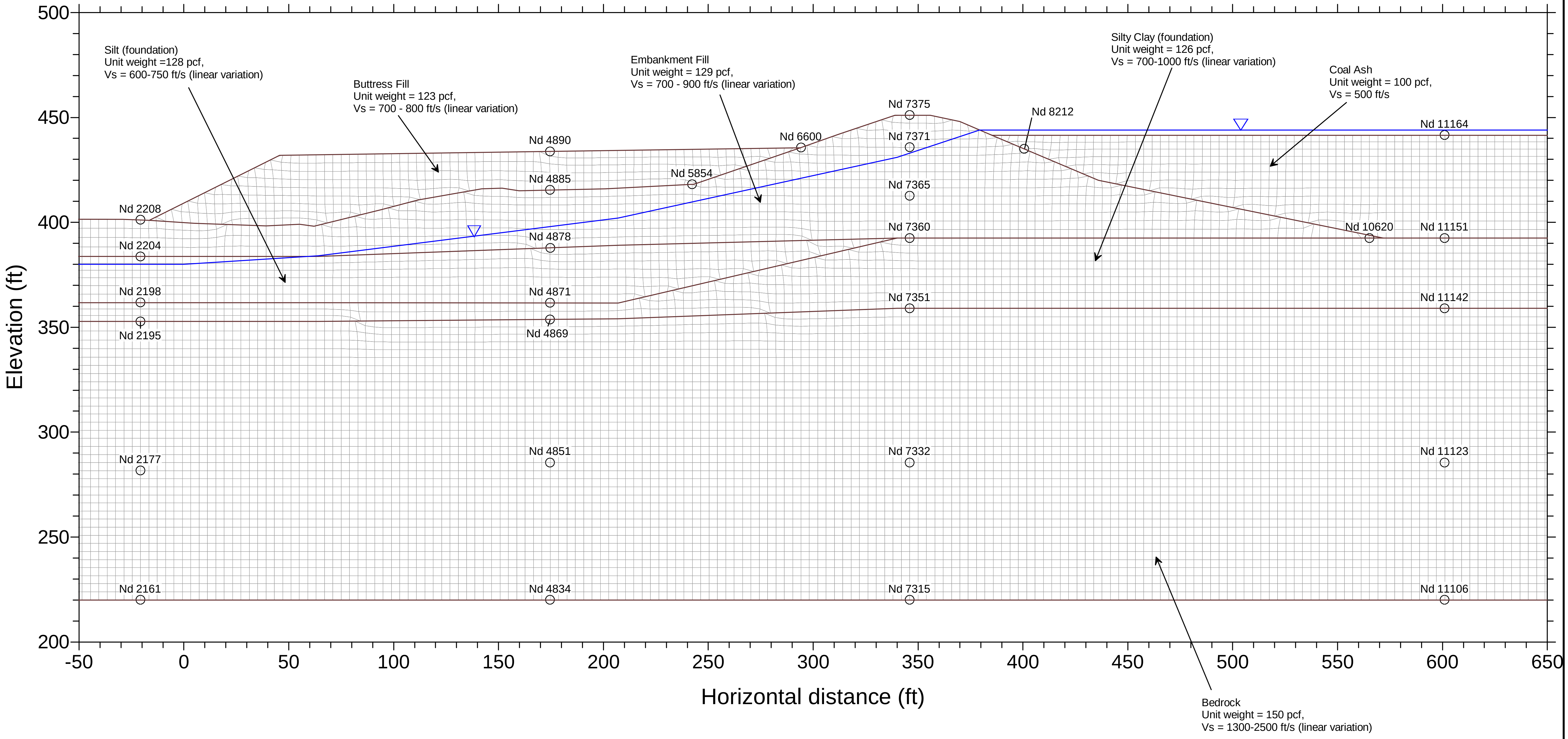
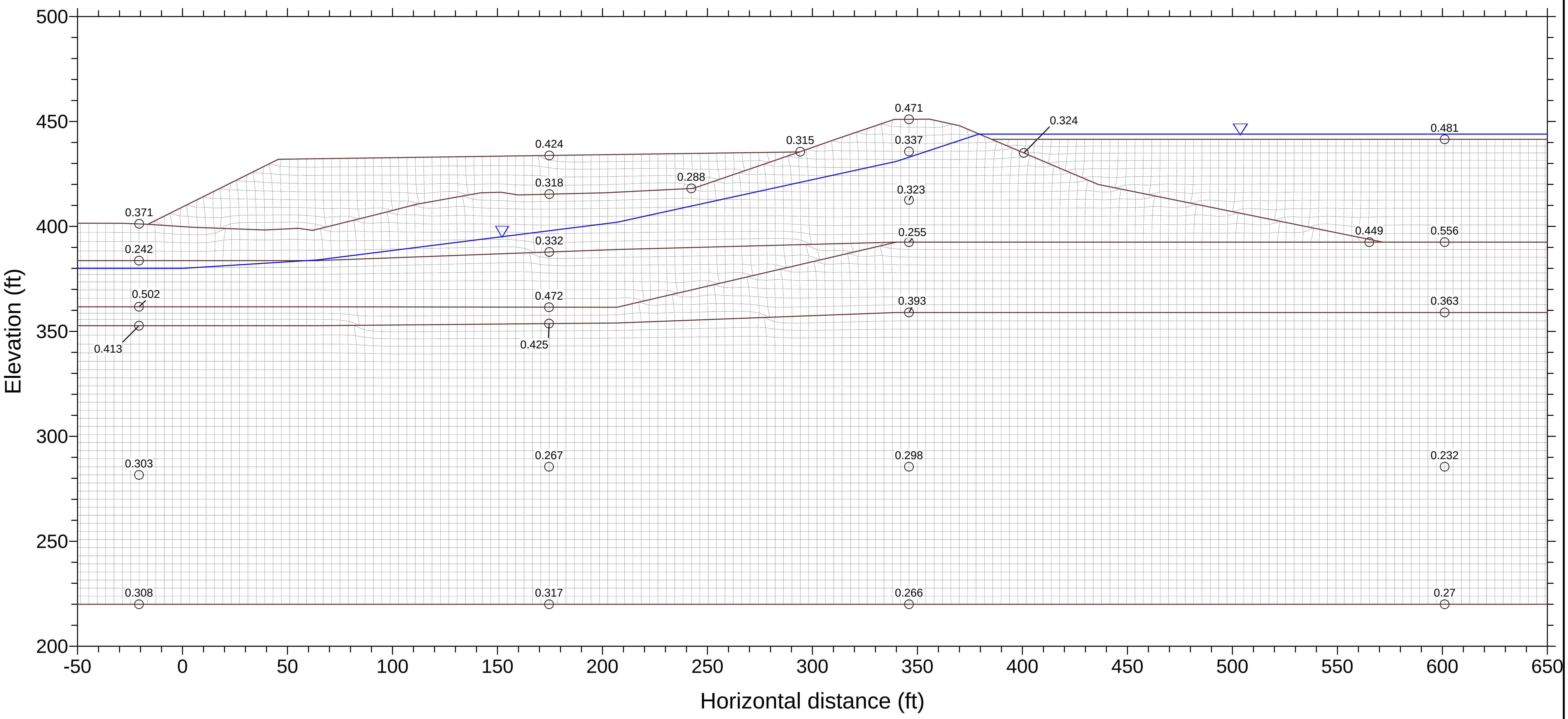
QUAD4M RESULTS

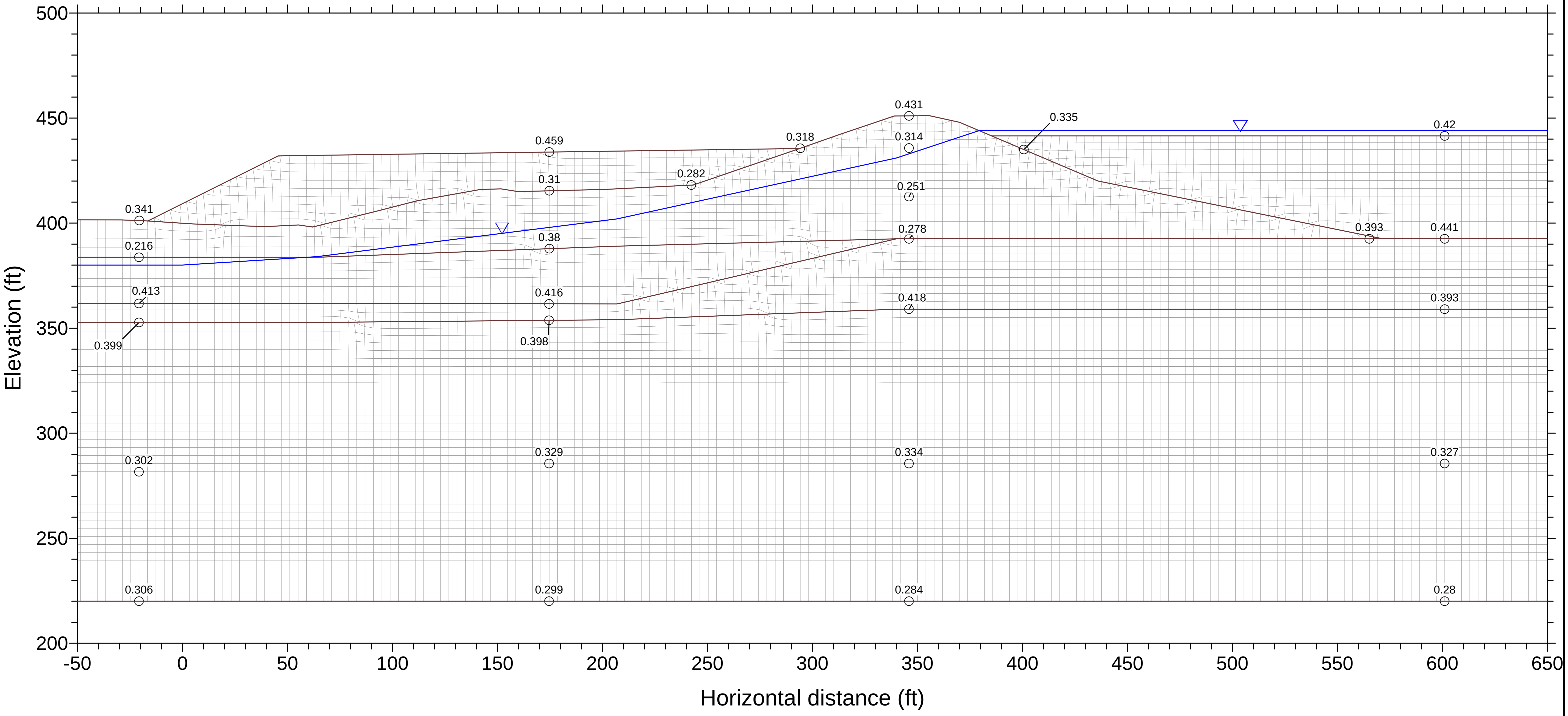
BUTTRESS MODEL



Project No. 60442676	Vectren A.B. Brown Station CCR Study	A.B. Brown Station Section B QUAD4 Model and Output Nodes	Figure 1
	AECOM		



Project No. 60442676	Vectren A.B. Brown Station CCR Study	A.B. Brown Station Section B QUAD4 Output - Nodal Acceleration (g) Base Motion: TH1	Figure 2-1
AECOM			



Project No.
60442676

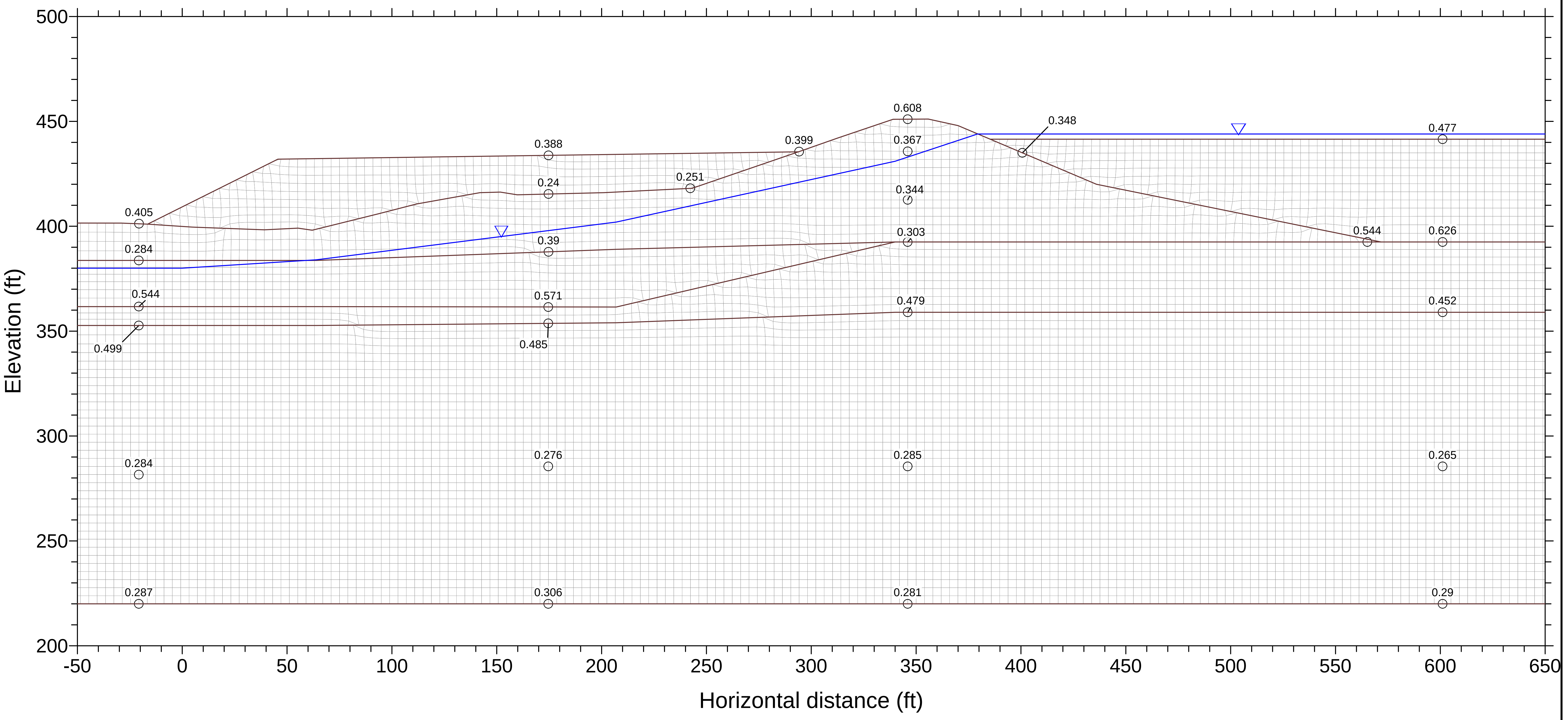
Vectren
A.B. Brown Station
CCR Study

AECOM

A.B. Brown Station Section B
QUAD4 Output - Nodal Acceleration (g)
Base Motion: TH2

Figure
2-2

\Projects\Vectren\60442676_ABBrown\400-Technical\503 Revised Quad4 Analysis\



Project No. 60442676	Vectren A.B. Brown Station CCR Study	A.B. Brown Station Section B QUAD4 Output - Nodal Acceleration (g) Base Motion: TH3	Figure 2-3
AECOM			

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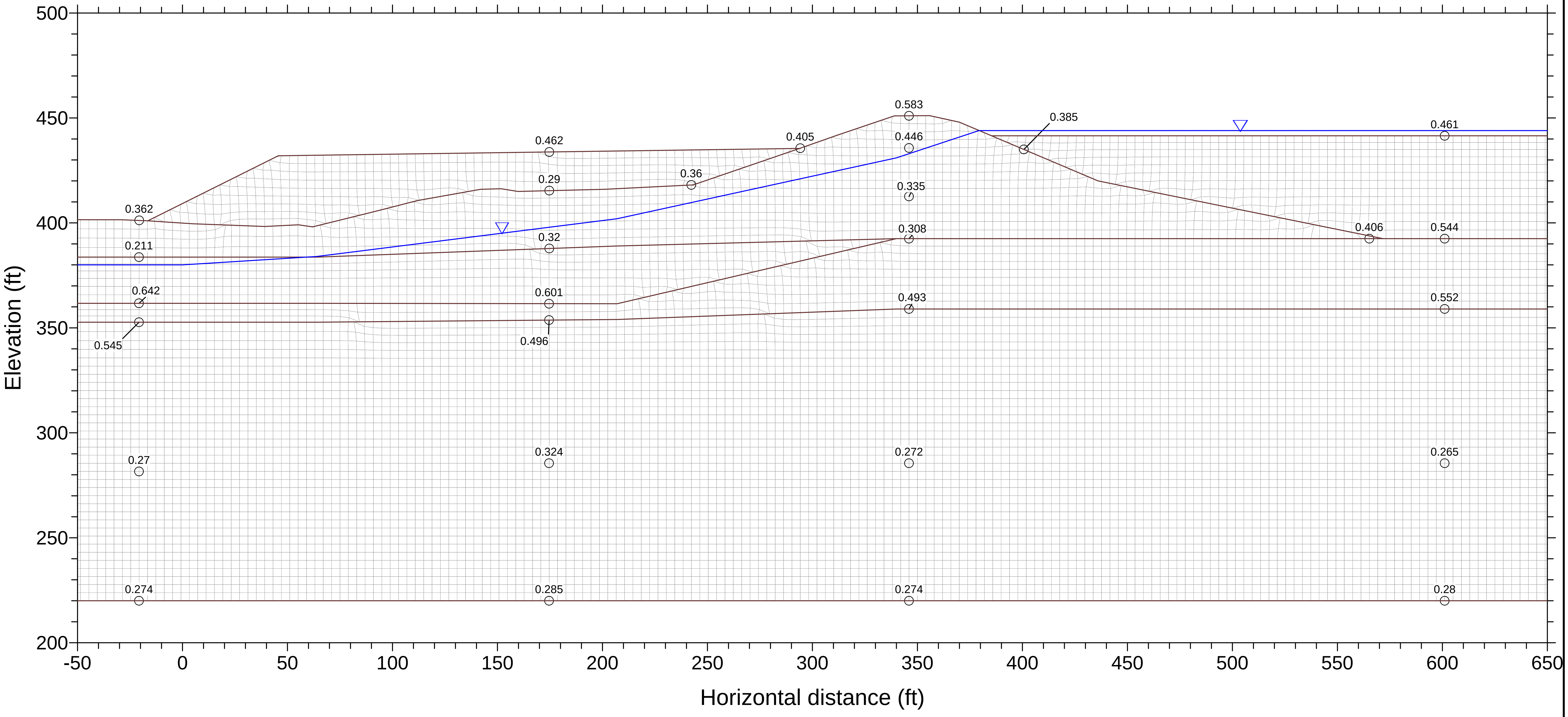


Project No.
60442676

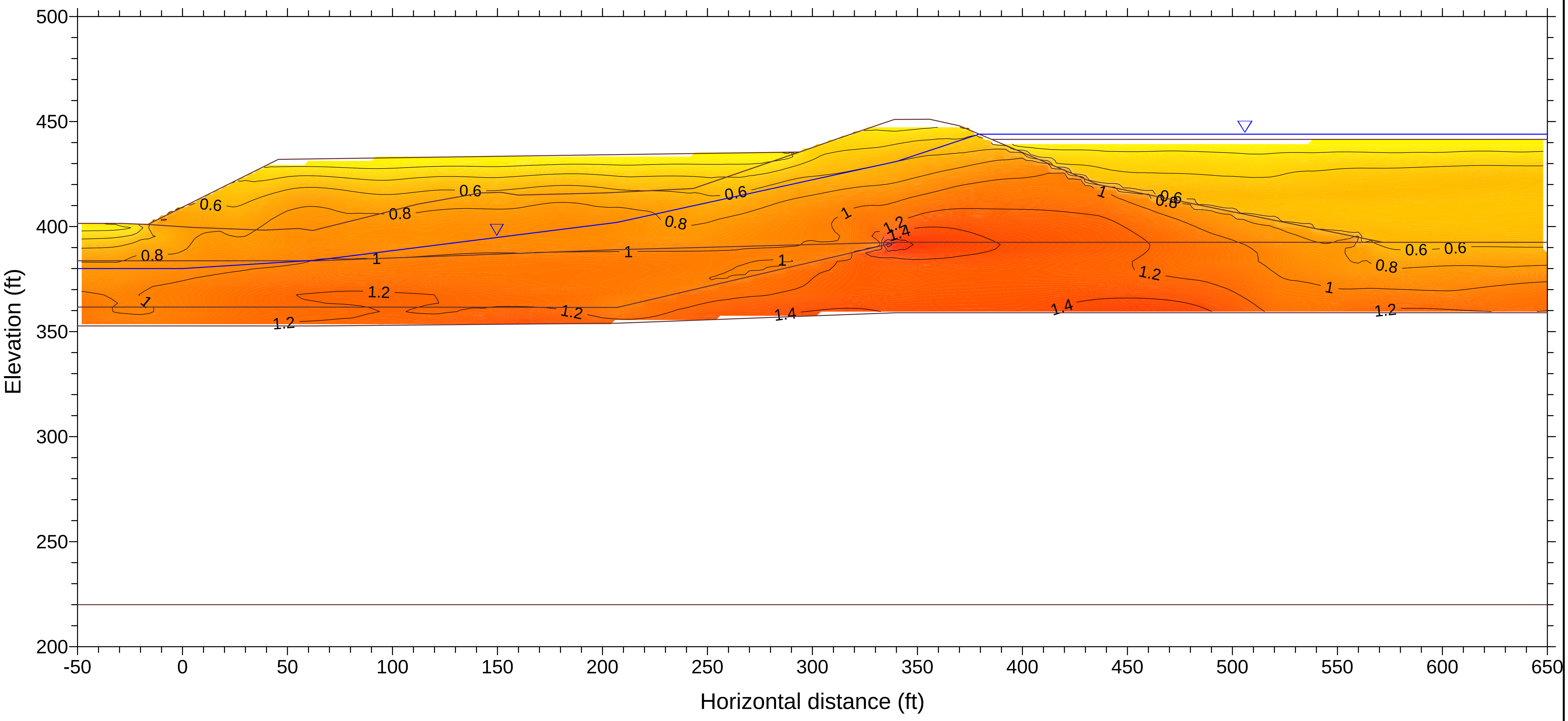
Vectren
A.B. Brown Station
CCR Study

AECOM

A.B. Brown Station Section B
QUAD4 Output - Nodal Acceleration (g)
Base Motion: TH4

Figure
2-4

\Projects\Vectren\60442676_ABBrown\400-Technical\503 Revised Quad4 Analysis\



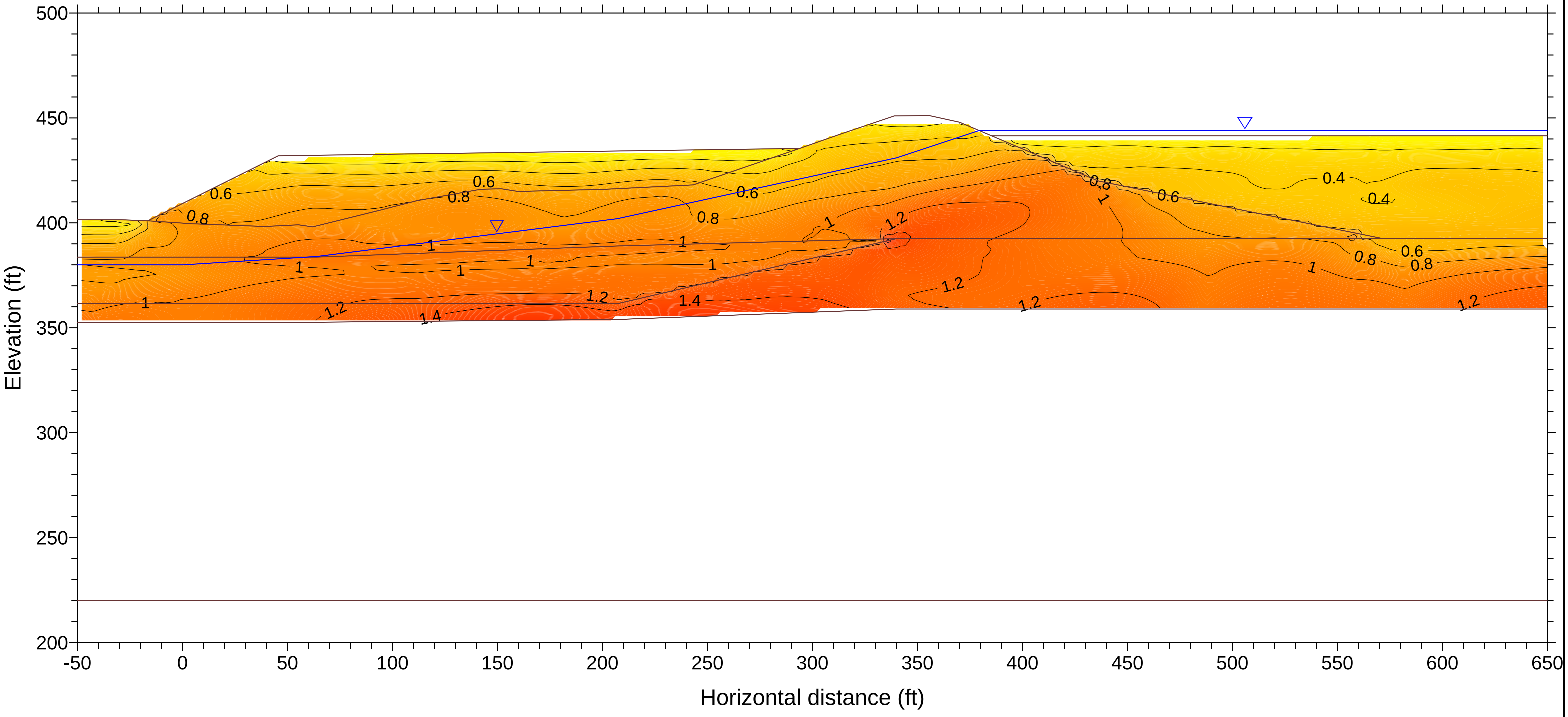
Project No.
60442676

Vectren
A.B. Brown Station
CCR Study

A.B. Brown Station Section B
QUAD4 Output - Peak Shear Stress (ksf)
Base Motion: TH1

Figure
3-1

\Projects\Vectren\60442676_ABBrown\400-Technical\503 Revised Quad4 Analysis\



Project No.
60442676

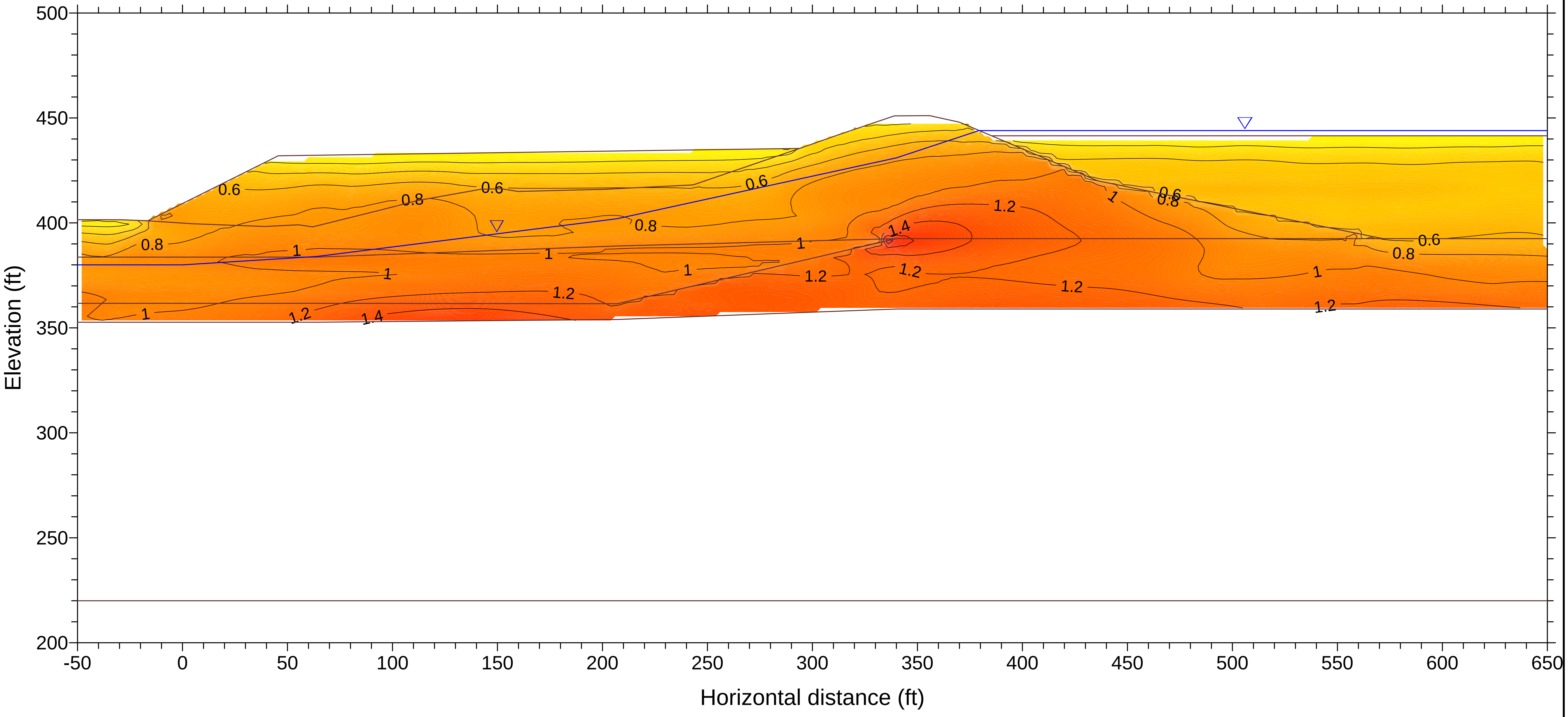
Vectren
A.B. Brown Station
CCR Study

AECOM

A.B. Brown Station Section B
QUAD4 Output - Peak Shear Stress (ksf)
Base Motion: TH2

Figure

3-2



Project No.
60442676

Vectren
A.B. Brown Station
CCR Study

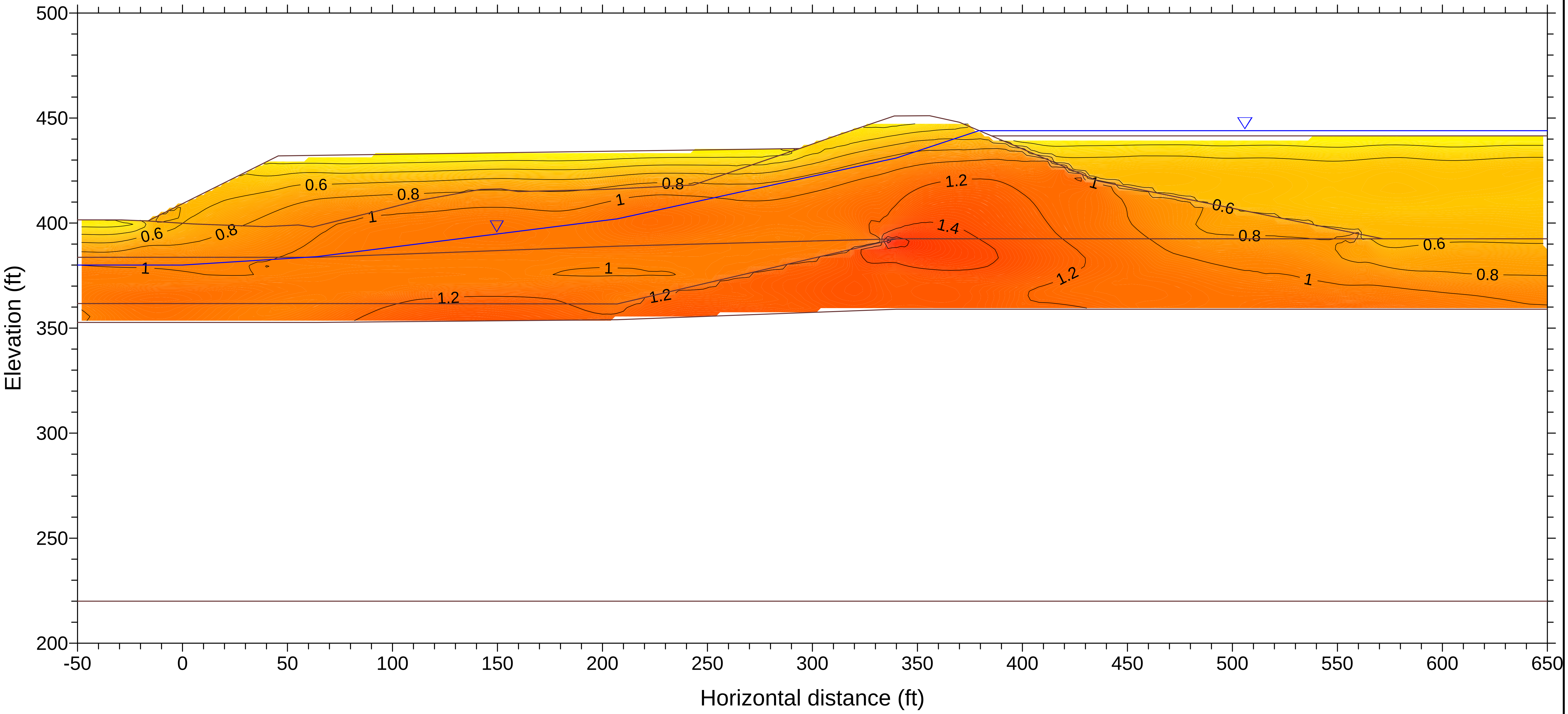
AECOM

A.B. Brown Station Section B
QUAD4 Output - Peak Shear Stress (ksf)
Base Motion: TH3

Figure

3-3

\Projects\Vectren\60442676_ABBrown\400-Technical\503 Revised Quad4 Analysis\



Project No. 60442676	Vectren A.B. Brown Station CCR Study	A.B. Brown Station Section B QUAD4 Output - Peak Shear Stress (ksf) Base Motion: TH4	Figure 3-4
AECOM			

\Projects\Vectren\60442676_ABBrown\400-Technical\503 Revised Quad4 Analysis\

Appendix I
Liquefaction Analysis Calculations

I. Purpose: This presents the liquefaction triggering evaluation performed to support stability analysis of the Lower Dam, at Vectren's A.B. Brown Generating Station.

This analysis is being performed in conjunction with dike slope stability analyses for the Lower Dam, in accordance with the requirements of Section 257.73 of the CCR Rule. Liquefaction triggering analyses of the various soil units comprising and underlying the dam are required in order to establish the shear strength of subsurface materials for use in the post-liquefaction slope stability condition. The basis for selection of these parameters is provided in Attachment F, and the post-liquefaction slope stability analyses are developed and presented in Attachment F.

II. Basis and Methodology of Liquefaction Analysis

- Based on the subsurface exploration, the materials that may have potential for liquefaction include the sluiced fly ash deposit that is impounded behind the dam, as well as the native silt deposit which underlies the dam across the majority of the site. As liquefaction of the sluiced ash poses no impact to dam stability, the liquefaction analyses presented herein focus on the native silt deposit.
- The silt deposit varied in thickness from approximately 2.0 feet to 27.5 feet as summarized in **Table I-1.** Uncorrected field SPT N-values ranged between 0 and 23 blows per foot (bpf) with an average of 7 bpf, indicating a medium stiff consistency overall. The fines content of the silt layers (as indicated by material that passes through a No. 200 sieve) was often above 95%. Atterberg limits testing indicated about half of the samples to be non-plastic, with others exhibiting very low plasticity indices, usually below 7.

Table I-1. Presence of Potentially Liquefiable Silts

Boring No.	Depth to Top of Layer (feet)	Layer Thickness (feet)
B-201	37.0	11.0
B-202	--	--
B-203	--	--
B-204	46.0	7.0
B-205	26.5	27.5
B-206	18.0	12.5
	40.0	3.0
	58.0	15.0
B-207	29.0	9.0
B-208	13.0	22.0

Table I-1. Presence of Potentially Liquefiable Silts

Boring No.	Depth to Top of Layer (feet)	Layer Thickness (feet)
B-209	45.5	7.5
B-210	53.0	12.5
B-211	64.0	≥6.0
B-212	63.0	5.0
B-213	56.0	2.0
B-214	--	--
B-215	28.0	18.0
B-216	23.5	24.5
B-217	43.0	12.5
B-218	8.5	15.0
	46.5	6.5
B-219	5.5	12.5

- The clayey fill materials that comprise the dam embankment are considered to be non-liquefiable as they have plasticity indices well above 7, and the majority of these materials lie above the phreatic surface. The materials were present in the borings as well-compacted materials with stiff to very stiff consistency and are therefore not considered to be susceptible to softening as a result of cyclic loading. Undrained strength is used to represent this deposit in the seismic and post-liquefaction stability analyses.
- The native silty clay deposit which underlies the pond consists of materials classified as lean clay (CL) and (to a lesser degree) silty clay (CL-ML). Plasticity indices in this unit were generally well above 7 (average of 13), and the materials were generally stiff to very stiff in consistency. Like the clay embankment fill, this deposit is not considered to be prone to liquefaction or softening as a result of cyclic loading. Undrained strength is used to represent this deposit in the seismic and post-liquefaction stability analyses.
- All liquefaction analyses (as well as the dynamic response analyses that are used to establish ground motions for input in these analyses) reference a design earthquake event with 2% probability of exceedance in 50 years (recurrence interval of approximately 2500 years). This event is as stipulated by the CCR Rule.
- A Probabilistic Seismic Hazard Analysis (PSHA) was performed for the A.B. Brown site and is presented in **Appendix G**. The PSHA results were used to compute a

2,500-yr return period Uniform Hazard Spectrum (UHS) and develop horizontal acceleration time histories consistent with the hard rock 2,500-yr UHS. Four sets of time histories were developed for each design spectrum. The time histories represent the site-specific ground motions associated with the controlling near-field or far-field earthquake event, and consider the magnitude, distance, and Arias Intensity.

- The site-specific acceleration time histories were then used in a dynamic response analysis to estimate seismic-induced shear stresses for use in liquefaction analysis. QUAD4M dynamic response analyses were performed for Cross-Section B-B, which is located central to the axis of the dam and is considered representative of the site. The seismic load demand (cyclic shear stresses and cyclic stress ratios) resulting in the various soil units were estimated based on the results for this section, and were broadly applied for liquefaction analyses in other locations at the dam. QUAD4 analyses were performed for both the configuration of the dam prior to construction of the stabilizing soil buttress and for the current configuration with the buttress in place. The dynamic response analyses are presented in **Appendix H**.
- Liquefaction triggering evaluations for the native silt deposit were performed using three methods:
    1. A SPT-based Procedure
    2. A comparison of the seismic load demand to cyclic resistance, established on the basis of laboratory cyclic direct simple shear testing.
- The soil buttress is designed to mitigate the potential for slope instabilities following an earthquake event, even accounting for predicted liquefaction in the silt deposit. The gravity loads applied by the buttress will consolidate and strengthen the silt deposit relative to the pre-buttress configuration, and can be expected to increase the liquefaction resistance of the silt. However, it is anticipated that any such increase in resistance would be minor.

The soil borings and CPT soundings performed at the site were advanced prior to construction of the soil buttress and therefore liquefaction resistances established on the basis of this data also represent the pre-buttress conditions. For this reason, the liquefaction potential evaluation was performed based on the configuration of the dam prior to construction of the soil buttress. This is considered to be conservative, as the liquefaction resistance of the silt soils following buttress construction would be expected to be higher, as explained above.

The presence of the buttress could influence the cyclic shear stresses generated during the design earthquake event. A comparison of the stresses in the silt deposit between the QUAD 4 models representing the pre- and post-buttress configurations was performed to address this, as described in Section III below.

- The SPT-based liquefaction triggering analyses were performed using the procedure proposed by Idriss and Boulanger (2008, 2014). The procedure considers a stress-based approach to evaluate the potential for liquefaction triggering, and compares calculated earthquake-induced cyclic stress ratios (CSRs) with the estimated cyclic resistance ratios (CRRs) of the soil to establish the factor of safety against liquefaction triggering.
- Stress-controlled Cyclic Direct Simple Shear (CDSS) tests (per ASTM D 6528) were performed on undisturbed samples of silt obtained from multiple locations from beneath the dam. A total of six silt samples were tested. The CDSS tests were performed for a range of CSRs, which covers the load demand that the silt is anticipated to experience during the design earthquake. Samples were loaded to normal stresses at or slightly above the existing overburden pressure estimated for that sample, with the intent of testing each sample in a normally consolidated condition.

III. Calculation of Seismic Load Demand

The QUAD4M model for Section B-B incorporates a large number of finite elements making up the meshing for the whole cross-section. Seismically induced shear stresses are calculated for each element, and 2-dimensional plots of shear stress contours within the cross-section are generated. These plots are provided in Appendix H, for each of the four time histories analyzed. Estimated peak nodal accelerations are presented in Figures 2-1 to 2-4 of that Appendix. Further, the peak cyclic shear stresses (in ksf) estimated for each time history are shown in Figures 3-1 to 3-4 of the Appendix.

The shear stresses vary both vertically and horizontally within the cross-section, and also vary by time history. The CSR at any location is defined as follows:

$$CSR = \frac{0.65 * \tau_{cyc}}{\sigma_{vc}{}'}$$

where:

τ_{cyc} = cyclic shear stress
$\sigma_{vc}{}'$ = effective vertical stress

As a broad interpretation of the results, the shear stresses and corresponding CSRs calculated for elements within the foundation silt layer were tallied, and ranges and averages were determined. As described above, this was done using the QUAD4M model representing the pre-buttress configuration. A summary of these values is provided in **Table I-2** below:

Table I-2: Shear Stresses and Cyclic Stress Ratios (CSR) in Silt Deposit (From QUAD4 Analysis) – Pre-Buttress Model

Time History	Range of Shear Stresses in Silt (ksf)	Average CSR in Silt	Range of CSRs in Silt
1	0.5-2.0	0.17	0.12-0.27
2	0.4-1.8	0.17	0.11-0.25
3	0.5-1.8	0.17	0.12-0.26
4	0.4-1.7	0.16	0.11-0.26

The QUAD4M results were utilized to establish the variation of CSR as a function of depth within the silt deposit for these analyses. As the majority of the borings that encountered the silt deposit were drilled at or close to the center of the mid-slope bench on the dam, the element cyclic shear stress results at the location of the centerline of the bench (the reference location) were taken from the QUAD4M results, as shown in **Figure I-1** below. These shear stresses were then transformed to CSRs for use in liquefaction analyses. **Table I-3** summarizes the average CSR (among all time histories analyzed) at the top, center, and bottom of the silt layer at the reference location. The CSRs utilized in the liquefaction screening analyses were linearly interpolated based on the values in the table.

Figure I-1: Location Used for Establishing CSRs in Silt for Liquefaction Screening

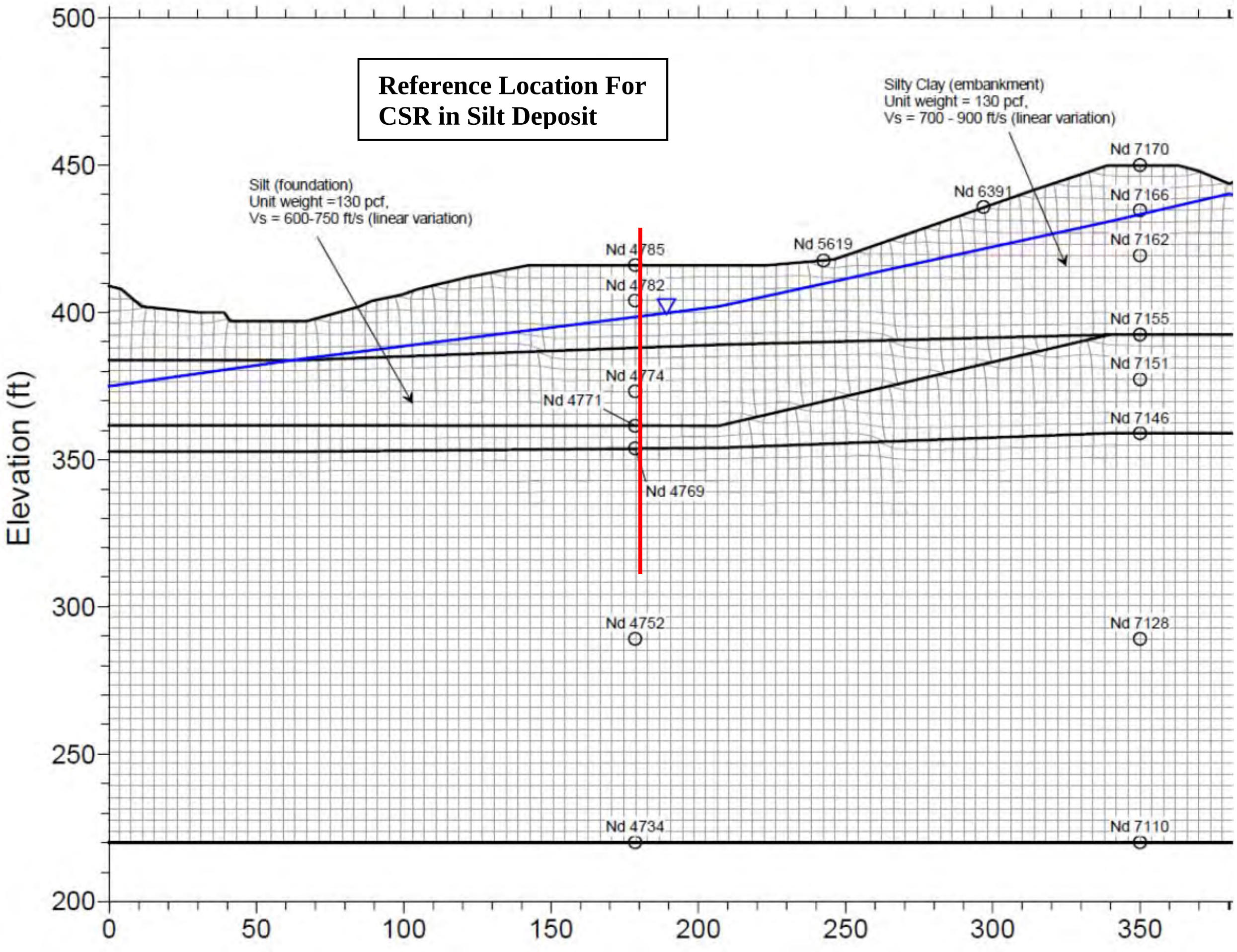


Table I-3: Shear Stresses and Cyclic Stress Ratios (CSR) in Silt Deposit (From QUAD4 Analysis) – Pre-Buttress Model

Location	Average CSR
Top of Silt Deposit	0.20
Center of Silt Deposit	0.16
Bottom of Silt Deposit	0.14

As described in Section II, the presence of the buttress may affect the cyclic shear stresses generated in the silt deposit. The above calculations of CSR were therefore repeated for the

QUAD4M model that represents the post-buttress configuration and the results were compared to the values given in **Table I-2**. The comparison is presented below in **Table I-4**:

Table I-4: Comparison of CSRs in Silt Deposit – Pre-Buttress Model vs. Post-Buttress Models

Time History	Average CSR in Silt		Range of CSRs in Silt	
	Pre-Buttress Model	Post-Buttress Model	Pre-Buttress Model	Post-Buttress Model
1	0.17	0.15	0.12-0.27	0.10-0.23
2	0.17	0.14	0.11-0.25	0.10-0.22
3	0.17	0.14	0.12-0.26	0.10-0.24
4	0.16	0.15	0.11-0.26	0.10-0.25

CSRs in the silt deposit are slightly lower in the post-buttress model than in the pre-buttress model. As stated previously, liquefaction resistance of the silt deposit in the presence of the buttress is also expected to be somewhat higher than without it. For these reasons, it is conservative to utilize the pre-buttress model results for the liquefaction potential analyses and this has been done herein.

IV. SPT-Based Liquefaction Potential Evaluation

Spreadsheets developed by AECOM utilizing the SPT-based procedures given in Idriss and Boulanger (2008, 2014) and in conjunction with SPT data from the available borings were used for the analyses.

The spreadsheets calculate a Factor of Safety against liquefaction, which is defined as the quotient of the soil's cyclic resistance ratio and the cyclic stress ratio induced by the earthquake:

$$FS_{liq} = \frac{CRR}{CSR}$$

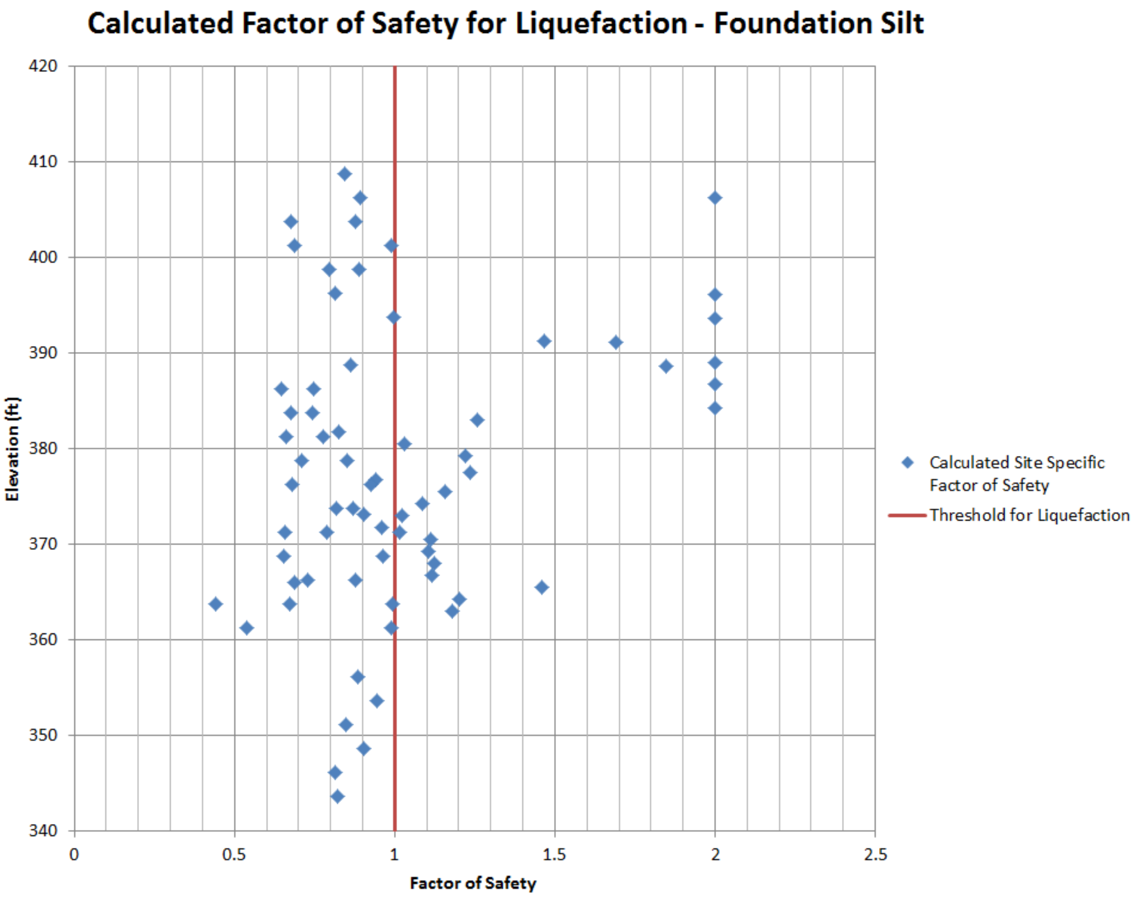
CSRs were determined as described previously. The CRR is the cyclic resistance ratio at which liquefaction occurs during an earthquake. It is obtained from case history-based semi-empirical correlations with SPT values recorded at sites with level ground conditions, and it also is normalized to $\sigma'_v \approx 1$ atm for an earthquake with $M = 7.5$. Within the SPT-based procedure, the CRR is a function of a soil's fines content (FC), relative density and effective stress, and penetration resistance (SPT). The CRR is also dependent on the duration of shaking and is adjusted to the site-specific design earthquake using a Magnitude Scaling Factor (MSF). The PSHA indicates that predicted ground motions at the site have a bimodal response, with small magnitude events dominating the short-period spectral accelerations, and large magnitude events dominating the longer period portions of the spectrum. As liquefaction is a phenomenon most commonly associated with long-duration, high-magnitude earthquakes, the magnitude assumed in the liquefaction screening analysis was 7.1, corresponding to the sources that dominate the longer-period portion of the spectrum. Regarding fines content, the foundation silt is a largely fine material. Based on the results of laboratory particle size analysis, the fines content of all silt materials was assumed to be 90% for analysis purposes.

Analyses were performed for each boring that encountered significant thickness of the silt, including B-205 to B-208, and B-215 to B-219. Analysis focused on liquefaction potential of the silt deposit. As described previously, the CSRs provided in **Table I-3** were linearly interpolated throughout the depth of the silt layer for the analysis of each SPT boring, and were manually input into the spreadsheet analysis.

In general, a factor of safety of less than 1.0 indicates that liquefaction could occur during seismic shaking. A factor of safety was calculated for each interval within the exploration (each depth at which a SPT N-value is available). The spreadsheet limits liquefaction factors of safety to 2.0, even if the computed factor of safety is higher than 2.0.

Spreadsheet analysis output files are provided in **Attachment I-1**. **Figure I-2** portrays the calculated factors of safety within the foundation silt material. Data from all borings have been combined into the figure. The majority of calculated factors of safety are below 1.0, and substantially below in many cases.

Figure I-2: Compilation of Liquefaction Factor of Safety in Foundation Silts



Based on the results of the SPT-based screening analysis, it is concluded that liquefaction can be triggered within the silt layer as a result of the design seismic event.

V. Laboratory-Based Liquefaction Potential Evaluation

While the liquefaction resistance of sand materials (especially clean sands) is well-documented within geotechnical practice, the resistance of silty soils is less well-established. In general, it is known that higher fines content in a soil increases the resistance to liquefaction, and various methodologies (including that adopted by Idriss and Boulanger (2008, 2014) and utilized in the SPT-based screening analyses presented above) have been proposed and are in use. Considering that the layer of concern consists of a high-fines silt (90% fines or greater in most samples that were tested in the laboratory), and considering that the screening analysis presented previously is a first-level, approximate evaluation, a second more rigorous laboratory-based approach was taken herein, to rule out the possibility that the silt is not prone to liquefaction during the design earthquake.

Stress-controlled CDSS testing (per ASTM D 6528) was performed on undisturbed silt samples obtained from multiple locations beneath the Lower Dam. A total of six samples were tested. As presented in **Table I-2**, the average CSR demand in the silt layer predicted from the QUAD4M dynamic response analysis, is about 0.17, and ranges from about 0.10 to about 0.25. Therefore, CDSS testing was performed at test CSRs of 0.08, 0.15, 0.20, and 0.25, to cover the expected range. Samples were loaded to normal stresses at or slightly above the existing overburden pressure estimated for that sample.

Laboratory data from the CDSS tests are presented in **Appendix D.** The test results (including excess pore pressure generated and axial strain) are presented as a function of the number of cycles that have been applied at any point in the test. Herein, failure (i.e., liquefaction) was interpreted at the cycle where the single-phase axial strain exceeded 5% (or 10% peak-to-peak) or the excess pore pressure ratio reached 85% of the applied normal stress, whichever was less.

The results of CDSS testing are summarized in **Table I-4** below.

Table I-4: Summary of CDSS Testing Results

Boring No.	Depth (feet)	Test CSR	Vertical Consolidation Stress (psf)	Number of Load Cycles To Failure	Failure Mechanism
AECOM-B1	39-41	0.25	4,275	4	Strain Criteria
AECOM-B2	56-58	0.15	4,950	17	Excess Pressure Criteria
	62-64	0.20	6,040	3	Strain Criteria
AECOM-B4	33-35	0.08	2,965	>50	Sample did not liquefy
	46-48	0.20	3,380	6	Excess Pressure Criteria
AECOM-B5	30-32	0.15	2,660	20	Excess Pressure Criteria

Figure I-3 plots the CDSS failure points as a function of the number of cycles. For an average CSR of 0.17, the expected number of cycles to failure is expected to be approximately 9. The cyclic resistance of soils in the field is likely to be less than that interpreted from laboratory results, due to the potential for multidirectional shaking. Consequently, the number of cycles to liquefaction in an earthquake setting is expected to be somewhat less than that determined from laboratory testing. Herein, the number of cycles to liquefaction in the field is assumed to be in the range of 7 to 9.

Figure I-3: CSR Vs. Number of Cycles at Failure – Laboratory CDSS Testing

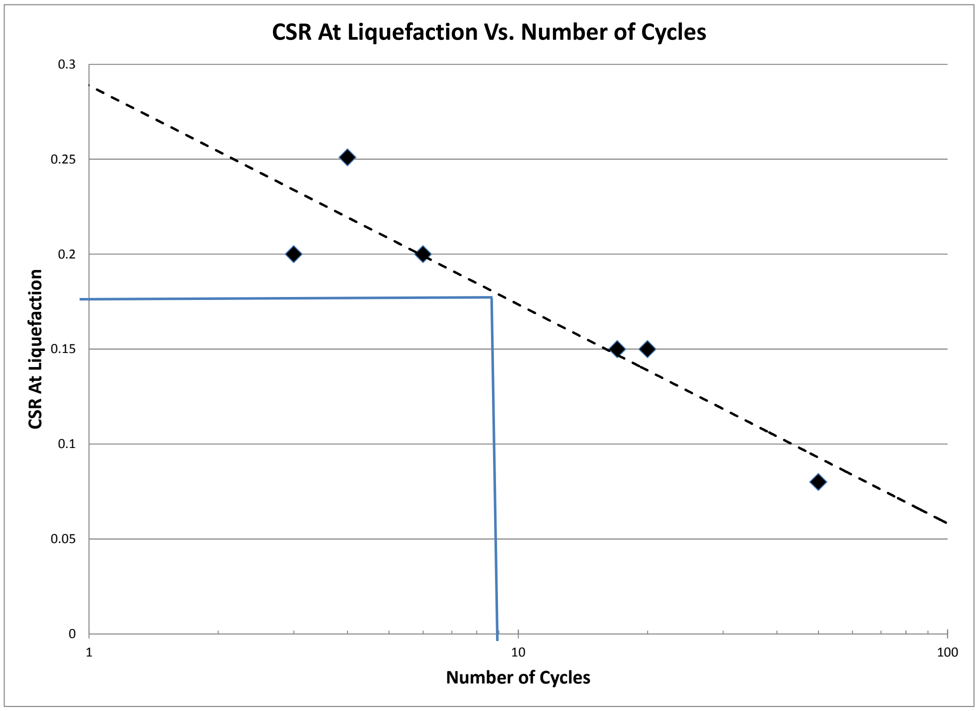
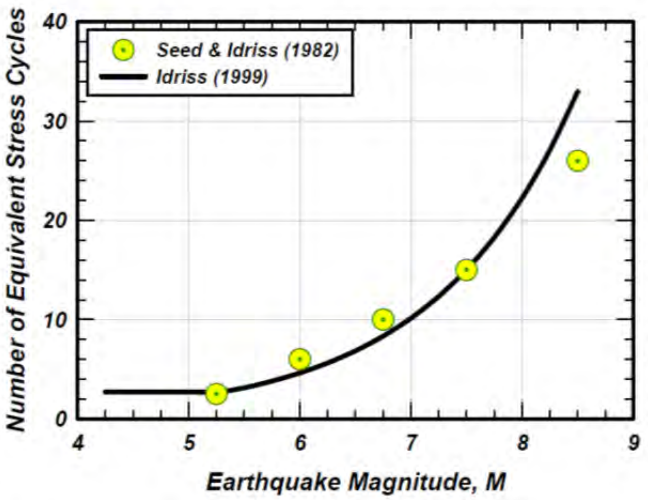


Figure I-4 (Boulanger and Idriss, 2008) shown below presents an estimate of the mean number of equivalent uniform cycles at reference stress of 65% of the peak stress (i.e., the definition of the CSR) that can be expected for a given earthquake magnitude.

Figure I-4: Mean number of equivalent uniform cycles at reference stress of 65% of the peak stress versus earthquake magnitude for sand soils (Boulanger and Idriss, 2008).



For an earthquake of magnitude 7.1, the figure indicates that approximately 12 equivalent cycles can be anticipated. As the laboratory CDSS samples reached failure in a smaller number of cycles, liquefaction of the silt is considered to be highly likely during the design earthquake.

VI. Conclusion

Based on the collective results of the laboratory-based and SPT -based triggering analyses, it is concluded that the native silt materials that underlie the dam are prone to liquefaction as a result of the design earthquake. Liquefaction and accompanying strength loss in these materials is expected to impact the factor of safety against stability in the post-liquefaction stability condition that is stipulated by the CCR Rule. As such, there is a need to establish the shear strength of the ash deposit in a liquefied state. This is presented in detail in **Appendix F**.

VII. References

1. Idriss, I.M., and Boulanger, R. W. (2008). "SPT-Based Liquefaction Triggering Procedures", Report No. UCD/CGM-10-02, Center for Geotechnical Modeling, Department of Civil and Environmental Engineering, University of California, Davis, CA.

2. Idriss, I.M. and Boulanger, R.W. (2014). "CPT and SPT Based Liquefaction Triggering Procedures", Center of Geotechnical Modeling, Department of Civil and Environmental Engineering, University of California, Davis, California.

Attachment I-1
SPT-Based Liquefaction Analysis Output

Method: Idriss and Boulanger (2008), *Soil Liquefaction during Earthquakes*, EERI MNO-12

Input Parameters:

Title:	Vectren AB Brown	Peak ground acceleration, pga (g):			
Project:	Lower Dam	Earthquake Magnitude (M):	7.1		
Project No.:	60442676	Water Table Depth at the time of drilling	9.5	ft	2.90 m
		Water Table Depth at the time of earthquake	9.5	ft	2.90 m
Date:	1/22/2016	Avg Unit Weight above GWT	130	pcf	20.4213703 kN/m^3
Boring No.	B-205	Avg Unit Weight below GWT	130	pcf	20.4213703 kN/m^3
Units	American feet, pounds, pcf	Borehole Diameter	0.5833	ft	178 mm
		Correction for Sampler Liner (N/Y)	N	ft	
		Rod stickup above ground at start of drive	5	ft	1.524 m
		Boring Total Depth	62.2	ft	18.95856 m
		Ground Surface Elevation	415.5	ft	126.6444 m

Bold values for N and Fines were directly mesured.

Data No.	Depth	Elevation	Measured N Previously corrected for gravel content (*)	Soil Type (USCS)	Flag: "Unsaturated", "Clay", "85% Sat"	Fines Content (%)	Energy Ratio (%)	N_{60}	$(N_1)_{60}$	$(N_1)_{60\text{-cs}}$ for liquefaction triggering	$(N_1)_{60\text{-cs}}$ for residual strength	CRR	CSR	Factor of Safety	Layer Thickness ΔH_i	ΔLDI_i	Vertical Reconsol. Strain, ε_v	Layer Settlement ΔS_i
	ft	ft													ft	ft		ft
1	26.5	389	**19**	ML	85% Sat	90	81	25.1	28.2	33.7	33	1.041	0.200	2.00	13.25	0.00	0.000	0.000
2	28.75	386.75	**17**	ML	85% Sat	90	81	25.1	23.7	29.2	29	0.474	0.192	2.00	14.38	0.00	0.000	0.000
3	31.25	384.25	**18**	ML	85% Sat	90	81	27.9	25.9	31.4	31	0.623	0.184	2.00	2.38	0.00	0.000	0.000
4	33.75	381.75	**5**	ML	85% Sat	90	81	7.8	6.7	12.2	12	0.145	0.176	0.82	2.50	0.93	0.033	0.083
5	36.25	379.25	**10**	ML	85% Sat	90	81	15.5	13.3	18.8	18	0.205	0.168	1.22	2.50	0.04	0.005	0.014
6	38.75	376.75	**6**	ML	85% Sat	90	81	9.3	7.6	13.1	13	0.151	0.160	0.94	2.50	0.14	0.022	0.055
7	41.25	374.25	**8**	ML	85% Sat	90	81	12.4	10.0	15.5	15	0.170	0.156	1.09	2.50	0.06	0.008	0.021
8	43.75	371.75	**6**	ML	85% Sat	90	81	9.3	7.2	12.7	12	0.146	0.152	0.96	2.50	0.12	0.020	0.049
9	46.25	369.25	**8**	ML	85% Sat	90	81	12.4	9.5	15.1	15	0.163	0.148	1.10	2.50	0.06	0.008	0.020
10	48.75	366.75	**8**	ML	85% Sat	90	81	12.4	9.3	14.8	14	0.161	0.144	1.12	2.50	0.05	0.008	0.019
11	51.25	364.25	**9**	ML	85% Sat	90	81	14.0	10.3	15.8	15	0.168	0.140	1.20	2.50	0.04	0.006	0.014

Method: Idriss and Boulanger (2008), *Soil Liquefaction during Earthquakes*, EERI MNO-12

Title: Vectren AB Brown
Project: Lower Dam
Project No.: 60442676

Date: 1/22/2016
Boring No. B-206
Units American feet, pounds, pcf

Input Parameters:

Peak ground acceleration, pga (g):				
Earthquake Magnitude (M):	7.1			
Water Table Depth at the time of drilling	8.8	ft	2.68	m
Water Table Depth at the time of earthquake	8.8	ft	2.68	m
Avg Unit Weight above GWT	130	pcf	20.4213703	kN/m^3
Avg Unit Weight below GWT	130	pcf	20.4213703	kN/m^3
Borehole Diameter	0.583	ft	178	mm
Correction for Sampler Liner (N/Y)	N	ft		
Rod stickup above ground at start of drive	5	ft	1.524	m
Boring Total Depth	80	ft	24.384	m
Ground Surface Elevation	414.8	ft	126.43104	m

Bold values for N and Fines were directly mesured.

Data No.	Depth	Elevation	Measured N Previously corrected for gravel content (*)	Soil Type (USCS)	Flag: "Unsaturated", "Clay", "85% Sat"	Fines Content (%)	Energy Ratio (%)	N_{60}	$(N_1)_{60}$	$(N_1)_{60-cs}$ for liquefaction triggering	$(N_1)_{60-cs}$ for residual strength	CRR	CSR	Factor of Safety	Layer Thickness ΔH_i	ΔLDI_i	Vertical Reconsol. Strain, ε_v	Layer Settlement ΔS_i
	ft	ft													ft	ft		ft
1	18.75	396.05	**23**	ML	85% Sat	90	81	30.4	35.9	41.4	41	2.000	0.200	2.00	9.38	0.00	0.000	0.000
2	21.25	393.55	**21**	ML	85% Sat	90	81	31.0	32.0	37.5	37	2.000	0.194	2.00	10.63	0.00	0.000	0.000
3	23.75	391.05	**13**	ML	85% Sat	90	81	19.2	19.3	24.8	24	0.319	0.189	1.69	2.50	0.01	0.001	0.003
4	26.25	388.55	**14**	ML	85% Sat	90	81	20.6	20.2	25.7	25	0.338	0.183	1.85	2.50	0.01	0.001	0.001
5	31.25	383.55	**6**	CL	Clay	90	81	9.3	na	na	na	#N/A	0.177	2.00	3.75	0.00	0.000	0.000
6	36.25	378.55	**16**	CL	Clay	90	81	24.8	na	na	na	#N/A	0.171	2.00	5.00	0.00	0.000	0.000
7	41.75	373.05	**6**	ML	85% Sat	90	81	9.3	7.6	13.1	13	0.150	0.166	0.90	5.25	0.46	0.032	0.166
8	46.25	368.55	**15**	CL	Clay	90	81	23.3	na	na	na	#N/A	0.160	2.00	5.00	0.00	0.000	0.000
9	51.25	363.55	**10**	CL	Clay	90	81	15.5	na	na	na	#N/A	0.158	2.00	4.75	0.00	0.000	0.000
10	56.25	358.55	**5**	CL	Clay	90	81	7.8	na	na	na	#N/A	0.155	2.00	5.00	0.00	0.000	0.000
11	58.75	356.05	**6**	ML	85% Sat	90	81	9.3	6.3	11.8	11	0.135	0.153	0.89	3.75	0.98	0.034	0.126
12	61.25	353.55	**7**	ML	85% Sat	90	81	10.9	7.3	12.8	12	0.142	0.150	0.95	2.50	0.14	0.022	0.055
13	63.75	351.05	**5**	ML	85% Sat	90	81	7.8	5.0	10.5	10	0.125	0.148	0.85	2.50	1.12	0.036	0.091
14	66.25	348.55	**6**	ML	85% Sat	90	81	9.3	6.0	11.5	11	0.131	0.145	0.90	2.50	0.44	0.034	0.086
15	68.75	346.05	**4**	ML	85% Sat	90	81	6.2	3.8	9.3	9	0.116	0.143	0.81	2.50	1.27	0.039	0.097
16	71.25	343.55	**4**	ML	85% Sat	90	81	6.2	3.7	9.3	9	0.115	0.140	0.82	2.50	1.28	0.039	0.098

Method: Idriss and Boulanger (2008), *Soil Liquefaction during Earthquakes*, EERI MNO-12

Title: Vectren AB Brown
Project: Lower Dam
Project No.: 60442676
Date: 1/22/2016
Boring No. B-207
Units American feet, pounds, pcf

Input Parameters:

Peak ground acceleration, pga (g):	0.47			
Earthquake Magnitude (M):	6.1			
Water Table Depth at the time of drilling	10	ft	3.05	m
Water Table Depth at the time of earthquake	10	ft	3.05	m
Avg Unit Weight above GWT	130	pcf	20.4213703	kN/m^3
Avg Unit Weight below GWT	130	pcf	20.4213703	kN/m^3
Borehole Diameter	0.583	ft	178	mm
Correction for Sampler Liner (N/Y)	N	ft		
Rod stickup above ground at start of drive	5	ft	1.524	m
Boring Total Depth	47.1	ft	14.35608	m
Ground Surface Elevation	395	ft	120.396	m

Calculated Volumetric Settlement: 0.98 ft
Calculated LDI: 10.0 ft
MSF for Sand 1.44

Bold values for N and Fines were directly mesured.

Data No.	Depth	Elevation	Measured N Previously corrected for gravel content (*)	Soil Type (USCS)	Flag: "Unsaturated", "Clay", "85% Sat"	Fines Content (%)	Energy Ratio (%)	N_{60}	$(N_1)_{60}$	$(N_1)_{60\text{-cs}}$ for liquefaction triggering	$(N_1)_{60\text{-cs}}$ for residual strength	CRR	CSR	Factor of Safety	Layer Thickness ΔH_i	ΔLDI_i	Vertical Reconsol. Strain, ε_v	Layer Settlement ΔS_i
	ft	ft													ft	ft		ft
1	29	366	**7**	ML	85% Sat	90	81	8.9	10.1	15.6	15	0.240	0.359	0.67	14.50	3.73	0.028	0.405
2	31.25	363.75	**5**	ML	85% Sat	90	81	7.8	6.9	12.4	12	0.191	0.444	0.43	15.63	5.69	0.033	0.511
3	33.75	361.25	**8**	ML	85% Sat	90	81	12.4	10.8	16.3	16	0.234	0.445	0.53	2.38	0.57	0.027	0.064

Method: Idriss and Boulanger (2008), *Soil Liquefaction during Earthquakes*, EERI MNO-12

Title: Vectren AB Brown
Project: Lower Dam
Project No.: 60442676
Date: 1/22/2016
Boring No. B-208
Units American feet, pounds, pcf

Input Parameters:				
Peak ground acceleration, pga (g):				
Earthquake Magnitude (M):	7.1			
Water Table Depth at the time of drilling	11.7	ft	3.57	m
Water Table Depth at the time of earthquake	11.7	ft	3.57	m
Avg Unit Weight above GWT	130	pcf	20.4213703	kN/m^3
Avg Unit Weight below GWT	130	pcf	20.4213703	kN/m^3
Borehole Diameter	0.583	ft	178	mm
Correction for Sampler Liner (N/Y)	N	ft		
Rod stickup above ground at start of drive	5	ft	1.524	m
Boring Total Depth	45	ft	13.716	m
Ground Surface Elevation	396.7	ft	120.91416	m

Bold values for N and Fines were directly mesured.

Data No.	Depth	Elevation	Measured N Previously corrected for gravel content (*)	Soil Type (USCS)	Flag: "Unsaturated", "Clay", "85% Sat"	Fines Content (%)	Energy Ratio (%)	N_{60}	$(N_1)_{60}$	$(N_1)_{60\text{-cs}}$ for liquefaction triggering	$(N_1)_{60\text{-cs}}$ for residual strength	CRR	CSR	Factor of Safety	Layer Thickness ΔH_i	ΔLDI_i	Vertical Reconsol. Strain, ε_v	Layer Settlement ΔS_i
	ft	ft													ft	ft		ft
1	13.75	382.95	**8**	ML	85% Sat	90	81	9.9	14.5	20.0	20	0.252	0.200	1.26	6.88	0.12	0.005	0.033
2	16.25	380.45	**7**	ML	85% Sat	90	81	10.3	11.3	16.8	16	0.196	0.190	1.03	8.13	0.25	0.010	0.084
3	19.25	377.45	**9**	ML	85% Sat	90	81	13.3	13.8	19.3	19	0.223	0.180	1.24	2.75	0.05	0.005	0.014
4	21.25	375.45	**8**	ML	85% Sat	90	81	11.8	11.8	17.3	17	0.197	0.170	1.16	2.50	0.05	0.007	0.016
5	23.75	372.95	**6**	ML	85% Sat	90	81	8.8	8.6	14.1	14	0.164	0.160	1.02	2.25	0.07	0.011	0.026
6	26.25	370.45	**7**	ML	85% Sat	90	81	10.3	9.7	15.2	15	0.172	0.155	1.11	2.50	0.05	0.008	0.019
7	28.75	367.95	**7**	ML	85% Sat	90	81	10.3	9.3	14.9	14	0.168	0.150	1.12	2.50	0.05	0.007	0.018
8	31.25	365.45	**10**	ML	85% Sat	90	81	15.5	13.8	19.3	19	0.212	0.145	1.46	2.50	0.02	0.003	0.007
9	33.75	362.95	**7**	ML	85% Sat	90	81	10.9	9.3	14.8	14	0.165	0.140	1.18	2.50	0.04	0.006	0.015

Method: Idriss and Boulanger (2008), *Soil Liquefaction during Earthquakes*, EERI MNO-12

Title: Vectren AB Brown
Project: Lower Dam
Project No.: 60442676
Date: 1/22/2016
Boring No. B-215
Units American feet, pounds, pcf

Input Parameters:

Peak ground acceleration, pga (g):				
Earthquake Magnitude (M):	7.1			
Water Table Depth at the time of drilling	9	ft	2.74	m
Water Table Depth at the time of earthquake	9	ft	2.74	m
Avg Unit Weight above GWT	130	pcf	20.4213703	kN/m^3
Avg Unit Weight below GWT	130	pcf	20.4213703	kN/m^3
Borehole Diameter	0.583	ft	178	mm
Correction for Sampler Liner (N/Y)	N	ft		
Rod stickup above ground at start of drive	5	ft	1.524	m
Boring Total Depth	60	ft	18.288	m
Ground Surface Elevation	415	ft	126.492	m

Bold values for N and Fines were directly mesured.

Data No.	Depth	Elevation	Measured N Previously corrected for gravel content (*)	Soil Type (USCS)	Flag: "Unsaturated", "Clay", "85% Sat"	Fines Content (%)	Energy Ratio (%)	N_{60}	$(N_1)_{60}$	$(N_1)_{60-cs}$ for liquefaction triggering	$(N_1)_{60-cs}$ for residual strength	CRR	CSR	Factor of Safety	Layer Thickness ΔH_i	ΔLDI_i	Vertical Reconsol. Strain, ε_v	Layer Settlement ΔS_i
	ft	ft													ft	ft		ft
1	28.75	386.25	**4**	ML	85% Sat	90	81	5.3	6.2	11.7	11	0.150	0.200	0.75	14.38	5.62	0.034	0.487
2	31.25	383.75	**3**	ML	85% Sat	90	81	4.7	4.2	9.7	9	0.126	0.187	0.68	15.63	7.66	0.038	0.595
3	33.75	381.25	**2**	ML	85% Sat	90	81	3.1	2.7	8.2	8	0.115	0.173	0.66	2.50	1.45	0.042	0.104
4	36.25	378.75	**2**	ML	85% Sat	90	81	3.1	2.6	8.1	8	0.114	0.160	0.71	2.50	1.46	0.042	0.105
5	38.75	376.25	**1**	ML	85% Sat	90	81	1.6	1.2	6.8	6	0.104	0.153	0.68	2.50	1.71	0.046	0.115
6	41.25	373.75	**3**	ML	85% Sat	90	81	4.7	3.7	9.2	9	0.120	0.147	0.82	2.50	1.30	0.039	0.098
7	43.75	371.25	**2**	ML	85% Sat	90	81	3.1	2.4	7.9	7	0.111	0.140	0.79	2.50	1.50	0.043	0.107

Method: Idriss and Boulanger (2008), *Soil Liquefaction during Earthquakes*, EERI MNO-12

Title: Vectren AB Brown
Project: Lower Dam
Project No.: 60442676
Date: 1/22/2016
Boring No. B-216
Units American feet, pounds, pcf

Input Parameters:

Peak ground acceleration, pga (g):				
Earthquake Magnitude (M):	7.1			
Water Table Depth at the time of drilling	9	ft	2.74	m
Water Table Depth at the time of earthquake	9	ft	2.74	m
Avg Unit Weight above GWT	130	pcf	20.4213703	kN/m^3
Avg Unit Weight below GWT	130	pcf	20.4213703	kN/m^3
Borehole Diameter	0.583	ft	178	mm
Correction for Sampler Liner (N/Y)	N	ft		
Rod stickup above ground at start of drive	5	ft	1.524	m
Boring Total Depth	60	ft	18.288	m
Ground Surface Elevation	415	ft	126.492	m

Bold values for N and Fines were directly mesured.

Data No.	Depth	Elevation	Measured N Previously corrected for gravel content (*)	Soil Type (USCS)	Flag: "Unsaturated", "Clay", "85% Sat"	Fines Content (%)	Energy Ratio (%)	N_{60}	$(N_1)_{60}$	$(N_1)_{60\text{-cs}}$ for liquefaction triggering	$(N_1)_{60\text{-cs}}$ for residual strength	CRR	CSR	Factor of Safety	Layer Thickness ΔH_i	ΔLDI_i	Vertical Reconsol. Strain, ε_v	Layer Settlement ΔS_i
	ft	ft													ft	ft		ft
1	23.75	391.25	**11**	ML	85% Sat	90	81	14.5	17.4	22.9	22	0.294	0.200	1.47	11.88	0.13	0.003	0.033
2	26.25	388.75	**6**	ML	85% Sat	90	81	8.8	8.6	14.1	14	0.164	0.190	0.86	13.13	1.51	0.030	0.394
3	28.75	386.25	**2**	ML	85% Sat	90	81	2.9	2.7	8.2	8	0.117	0.180	0.65	2.50	1.44	0.042	0.104
4	31.25	383.75	**3**	ML	85% Sat	90	81	4.7	4.2	9.7	9	0.126	0.170	0.74	2.50	1.23	0.038	0.095
5	33.75	381.25	**3**	ML	85% Sat	90	81	4.7	4.0	9.5	9	0.125	0.160	0.78	2.50	1.25	0.038	0.096
6	36.25	378.75	**4**	ML	85% Sat	90	81	6.2	5.2	10.7	10	0.133	0.156	0.85	2.50	1.09	0.036	0.090
7	38.75	376.25	**5**	ML	85% Sat	90	81	7.8	6.4	11.9	11	0.141	0.152	0.93	2.50	0.21	0.034	0.084
8	41.25	373.75	**4**	ML	85% Sat	90	81	6.2	4.9	10.4	10	0.129	0.148	0.87	2.50	1.13	0.036	0.091
9	43.75	371.25	**6**	ML	85% Sat	90	81	9.3	7.3	12.8	12	0.146	0.144	1.01	2.50	0.08	0.013	0.032
10	46.25	368.75	**5**	ML	85% Sat	90	81	7.8	5.9	11.4	11	0.135	0.140	0.96	2.50	0.13	0.023	0.057

Method: Idriss and Boulanger (2008), *Soil Liquefaction during Earthquakes*, EERI MNO-12

Title: Vectren AB Brown
Project: Lower Dam
Project No.: 60442676

Date: 1/22/2016
Boring No. B-217
Units American feet, pounds, pcf

Input Parameters:

Peak ground acceleration, pga (g):				
Earthquake Magnitude (M):	7.1			
Water Table Depth at the time of drilling	9	ft	2.74	m
Water Table Depth at the time of earthquake	9	ft	2.74	m
Avg Unit Weight above GWT	130	pcf	20.4213703	kN/m^3
Avg Unit Weight below GWT	130	pcf	20.4213703	kN/m^3
Borehole Diameter	0.583	ft	178	mm
Correction for Sampler Liner (N/Y)	N	ft		
Rod stickup above ground at start of drive	5	ft	1.524	m
Boring Total Depth	60	ft	18.288	m
Ground Surface Elevation	415	ft	126.492	m

Bold values for N and Fines were directly mesured.

Data No.	Depth	Elevation	Measured N Previously corrected for gravel content (*)	Soil Type (USCS)	Flag: "Unsaturated", "Clay", "85% Sat"	Fines Content (%)	Energy Ratio (%)	N_{60}	$(N_1)_{60}$	$(N_1)_{60\text{-cs}}$ for liquefaction triggering	$(N_1)_{60\text{-cs}}$ for residual strength	CRR	CSR	Factor of Safety	Layer Thickness ΔH_i	ΔLDI_i	Vertical Reconsol. Strain, ε_v	Layer Settlement ΔS_i
	ft	ft													ft	ft		ft
1	43.75	371.25	**3**	ML	85% Sat	90	81	4.4	4.5	10.0	10	0.132	0.200	0.66	21.88	10.33	0.037	0.816
2	46.25	368.75	**3**	ML	85% Sat	90	81	4.7	3.5	9.0	8	0.118	0.180	0.65	23.13	12.26	0.040	0.919
3	48.75	366.25	**3**	ML	85% Sat	90	81	4.7	3.4	8.9	8	0.116	0.160	0.73	2.50	1.34	0.040	0.100
4	51.25	363.75	**7**	ML	85% Sat	90	81	10.9	7.9	13.5	13	0.149	0.150	0.99	2.50	0.09	0.014	0.035
5	53.75	361.25	**6**	ML	85% Sat	90	81	9.3	6.6	12.1	12	0.138	0.140	0.99	2.50	0.10	0.016	0.040

Method: Idriss and Boulanger (2008), *Soil Liquefaction during Earthquakes*, EERI MNO-12

Title: Vectren AB Brown
Project: Lower Dam
Project No.: 60442676

Date: 1/22/2016
Boring No. B-218
Units American feet, pounds, pcf

Input Parameters:

Peak ground acceleration, pga (g):				
Earthquake Magnitude (M):	7.1			
Water Table Depth at the time of drilling	9	ft	2.74	m
Water Table Depth at the time of earthquake	9	ft	2.74	m
Avg Unit Weight above GWT	130	pcf	20.4213703	kN/m^3
Avg Unit Weight below GWT	130	pcf	20.4213703	kN/m^3
Borehole Diameter	0.583	ft	178	mm
Correction for Sampler Liner (N/Y)	N	ft		
Rod stickup above ground at start of drive	5	ft	1.524	m
Boring Total Depth	58.9	ft	17.95272	m
Ground Surface Elevation	415	ft	126.492	m

Bold values for N and Fines were directly mesured.

Data No.	Depth	Elevation	Measured N Previously corrected for gravel content (*)	Soil Type (USCS)	Flag: "Unsaturated", "Clay", "85% Sat"	Fines Content (%)	Energy Ratio (%)	N_{60}	$(N_1)_{60}$	$(N_1)_{60\text{-cs}}$ for liquefaction triggering	$(N_1)_{60\text{-cs}}$ for residual strength	CRR	CSR	Factor of Safety	Layer Thickness ΔH_i	ΔLDI_i	Vertical Reconsol. Strain, ε_v	Layer Settlement ΔS_i
	ft	ft													ft	ft		ft
1	8.75	406.25	**18**	ML	85% Sat	90	81	21.0	31.7	37.2	37	2.000	0.200	2.00	4.38	0.00	0.000	0.000
2	11.25	403.75	**2**	ML	85% Sat	90	81	2.6	3.5	9.1	9	0.130	0.192	0.68	5.63	2.96	0.040	0.222
3	13.75	401.25	**2**	ML	85% Sat	90	81	2.6	3.3	8.8	8	0.127	0.184	0.69	2.50	1.35	0.040	0.100
4	16.25	398.75	**3**	ML	85% Sat	90	81	4.4	5.2	10.7	10	0.140	0.176	0.80	2.50	1.10	0.036	0.090
5	18.75	396.25	**3**	ML	85% Sat	90	81	4.4	4.9	10.4	10	0.137	0.168	0.81	2.50	1.13	0.036	0.091
6	21.25	393.75	**5**	ML	85% Sat	90	81	7.4	7.7	13.3	13	0.159	0.160	1.00	2.50	0.09	0.014	0.035
7	26.25	388.75	**4**	CL	Clay	90	85	6.2	na	na	na	#N/A	0.157	2.00	3.75	0.00	0.000	0.000
8	31.25	383.75	**8**	CL	Clay	90	85	13.0	na	na	na	#N/A	0.153	2.00	5.00	0.00	0.000	0.000
9	36.25	378.75	**9**	CL	Clay	90	85	14.7	na	na	na	#N/A	0.150	2.00	5.00	0.00	0.000	0.000
10	41.25	373.75	**4**	CL	Clay	90	85	6.5	na	na	na	#N/A	0.147	2.00	5.00	0.00	0.000	0.000
11	48.75	366.25	**4**	ML	85% Sat	90	81	6.2	4.7	10.2	10	0.126	0.143	0.88	6.25	2.90	0.037	0.231
12	51.25	363.75	**0**	ML	85% Sat	90	81	0.0	0.0	5.5	5	0.094	0.140	0.67	5.00	3.98	0.050	0.252

Method: Idriss and Boulanger (2008), *Soil Liquefaction during Earthquakes*, EERI MNO-12

Title: Vectren AB Brown
Project: Lower Dam
Project No.: 60442676
Date: 1/22/2016
Boring No. B-219
Units American feet, pounds, pcf

Input Parameters:

Peak ground acceleration, pga (g):				
Earthquake Magnitude (M):	7.1			
Water Table Depth at the time of drilling	9	ft	2.74	m
Water Table Depth at the time of earthquake	9	ft	2.74	m
Avg Unit Weight above GWT	130	pcf	20.4213703	kN/m^3
Avg Unit Weight below GWT	130	pcf	20.4213703	kN/m^3
Borehole Diameter	0.583	ft	178	mm
Correction for Sampler Liner (N/Y)	N	ft		
Rod stickup above ground at start of drive	5	ft	1.524	m
Boring Total Depth	60	ft	18.288	m
Ground Surface Elevation	415	ft	126.492	m

Bold values for N and Fines were directly mesured.

Data No.	Depth	Elevation	Measured N Previously corrected for gravel content (*)	Soil Type (USCS)	Flag: "Unsaturated", "Clay", "85% Sat"	Fines Content (%)	Energy Ratio (%)	N_{60}	$(N_1)_{60}$	$(N_1)_{60\text{-cs}}$ for liquefaction triggering	$(N_1)_{60\text{-cs}}$ for residual strength	CRR	CSR	Factor of Safety	Layer Thickness ΔH_i	ΔLDI_i	Vertical Reconsol. Strain, ε_v	Layer Settlement ΔS_i
	ft	ft													ft	ft		ft
1	6.25	408.75	**4**	ML	85% Sat	90	81	4.7	7.9	13.4	13	0.175	0.200	0.88	3.13	0.37	0.031	0.097
2	8.75	406.25	**4**	ML	85% Sat	90	81	5.0	7.4	12.9	12	0.167	0.180	0.93	4.38	0.30	0.027	0.118
3	11.25	403.75	**3**	ML	85% Sat	90	81	4.0	5.3	10.8	10	0.144	0.160	0.90	2.50	1.09	0.036	0.089
4	13.75	401.25	**4**	ML	85% Sat	90	81	5.3	6.5	12.0	12	0.153	0.150	1.02	2.50	0.07	0.012	0.031
5	16.25	398.75	**2**	ML	85% Sat	90	81	2.9	3.5	9.0	8	0.127	0.140	0.90	2.50	1.32	0.040	0.099

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