Revision 0. October 16, 2016

Geocomp
Technologies to manage risk for infrastructure

October 16, 2016

Tennessee Valley Authority
1101 Market Street
Chattanooga, TN 37402

RE: Initial Seismic Safety Factor Assessment
EPA Final CCR Rule
Cumberland Fossil Plant – Stilling Pond (Including Retention Pond)
Cumberland City, Tennessee

Dear Sir/Madam:

TVA retained Geocomp Corporation (Geocomp) to prepare a seismic and liquefaction factor of safety assessment to meet the EPA's requirements under the HAZARDOUS AND SOLID WASTE MANAGEMENT SYSTEM; DISPOSAL OF COAL COMBUSTION RESIDUALS FROM ELECTRIC UTILITIES; FINAL RULE [RIN-2050-AE81; FRL-9919-44-OSWER]. This letter provides a brief project background, summary of findings, limitations, and certification.

1.0 BACKGROUND

As required by §257.73 of the EPA Final CCR Rule, within 18 months of the published date (April 17, 2015), an initial structural integrity evaluation for seismic loading is required and must include initial assessments of seismic factor of safety and liquefaction factor of safety for each existing CCR surface impoundment that meets the conditions of paragraph (b) as follows:

1. Has a height of five feet or more and a storage volume of 20 acre-feet or more; or
2. Has a height of 20 feet or more.

The seismic and liquefaction factor of safety assessments must document whether the calculated factors of safety for the critical cross section of each existing CCR surface impoundment achieve the minimum factors of safety specified in paragraphs (e)(1)(iii) and (e)(1)(iv) of §257.73 in the EPA Final CCR Rule. In accordance with paragraph (f)(2), the owner or operator of the existing CCR surface impoundment may elect to use a previously completed assessment to serve as the initial assessment required by paragraph (e) of the EPA Final CCR Rule provided that the previously completed assessment(s) was completed no earlier than 42 months prior to October of 2016, and meets the applicable requirements of paragraph (e) of the EPA Final CCR Rule.

In support of the above assessment, Geocomp completed a subsurface and laboratory investigation, seismic stability evaluation, and liquefaction assessment for the Stilling Pond (including Retention Pond) at the Cumberland Fossil Plant in Cumberland City, Tennessee. Information gathered through the subsurface and laboratory investigation, completed in December of 2015, was used to supplement data collected by Stantec in 2010. The above information was provided in the Stantec report titled "Report of Geotechnical Exploration and Slope Stability Evaluation, Ash Pond, Cumberland Fossil Plant, Stewart County, Tennessee" dated March 29, 2010 and Stantec report titled "Report of Geotechnical Exploration, Dry Fly Ash Stack and Gypsum Disposal Complex, Cumberland Fossil Plant, Stewart County, Tennessee" dated June 11, 2010. A complete listing of documents reviewed and utilized as part of this assessment is included in the Attachment.

2.0 SUMMARY OF FINDINGS

Based on a review of the available information from documents listed in the Attachment and Geocomp's stability evaluations associated with the Stilling Pond (including Retention Pond), Cross Section R-R' was selected as the critical cross section at the Stilling Pond (including Retention Pond) of the Cumberland Fossil Plant. Table 1 below provides a summary of the factors of safety for the critical cross section.

Table 1. Summary of Factors of Safety for Critical Cross Section

Plant	Facility	Critical Cross Section	EPA Criteria	CCR Rule Reference	EPA Required FOS	Factor of Safety (FOS)
CUF	Stilling Pond (Including Retention Pond)	R-R'	Seismic Factor of Safety (Pseudo-static stability)	§257.73(e)(1)(iii)	1.00	1.10
			Liquefaction Factor of Safety (Post-earthquake stability)	§257.73(e)(1)(iv)	1.20	1.22

Based upon the information presented in Table 1, it is Geocomp's opinion that these factors of safety meet or exceed the requirements of those specified in the EPA CCR Final Rule §257.73 paragraphs (e)(1)(iii) and (e)(1)(iv). Analyses supporting these factors of safety are presented in Geocomp's report to TVA titled "Tennessee Valley Authority EPA Seismic Assessment Supplemental Site Exploration Cumberland Fossil Plant Stilling Pond (including Retention Pond) and Bottom Ash Pond Final Report" dated October 2016.

3.0 LIMITATIONS

The signature of Geocomp's authorized representative on this document represents that to the best of Geocomp's knowledge, information, and belief in the exercise of its professional judgment, Geocomp's professional opinion is that the aforementioned information is accurate as of the date of such signature. Any opinion or decisions by Geocomp are made on the basis of this information, the engineering analyses, and Geocomp's experience, qualifications, and professional judgment and are not to be construed as warranties or guaranties. In addition, opinions relating to environmental, geologic, and geotechnical conditions or other estimates are based on available data and the actual conditions may vary from those encountered at the times and locations where data were obtained, despite the use of due care.

4.0 CERTIFICATION

I, W. Allen Marr, being a Registered Professional Engineer in the State of Tennessee do hereby certify to the best of my knowledge, information, and belief that the information contained in this report is true and correct and has been prepared in accordance with accepted engineering practice.

SIGNATURE W. Allen Marr DATE 10-16-16

ADDRESS: Geocomp Corporation, 125 Nagog Park, Acton, MA 01720

TELEPHONE: 978-635-0012

ATTACHMENTS: Demonstration Document for Seismic Factor of Safety and Liquefaction Factor of Safety for TVA Cumberland Fossil Plant, Stilling Pond (including Retention Pond) and Bottom Ash Pond, Cumberland City, TN.

WILLIAM ALLEN MARR

No. 00115016

W. Allen Marr

Attachment:

Demonstration Document for Seismic Factor of Safety and Liquefaction Factor of Safety for TVA Cumberland Fossil Plant, Stilling Pond (including Retention Pond) and Bottom Ash Pond, Cumberland City, TN.

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1.0 INTRODUCTION

1.1 OBJECTIVE

On April 17, 2015 the "Final Rule: Disposal of Coal Combustion Residuals (CCR) from Electric Utilities" (Environmental Protection Agency, 2015) was published in the Federal Register. Geocomp Corporation (Geocomp) was contracted by the Tennessee Valley Authority (TVA) to analyze the Seismic Structural Integrity Criteria for the Cumberland Fossil Plant (CUF) CCR surface impoundments and to evaluate compliance with §257.73(e)(1) (iii) and (iv) of the Environmental Protection Agency (EPA) Final CCR Rule.

1.2 OUTLINE OF RULE REQUIREMENTS

As required by §257.73(e)(1) (iii) and (iv) of the EPA Final CCR Rule, an initial structural integrity evaluation is required by October 17, 2016 and must include an initial seismic and liquefaction factor of safety assessment for each existing CCR surface impoundment that meets the conditions of paragraph (b) as follows:

- Has a height of five feet or more and a storage volume of 20 acre-feet or more or
- Has a height of 20 feet or more.

§257.53 requires seismic stability assessments of CCR impoundments consider a seismic event with 2 Percent Probability of Exceedance in 50 years (i.e. probable earthquake within approximately 2,500 years) and a Horizontal Spectral Response Acceleration for 1.0-Second Period (5 Percent of Critical Damping). The safety factor assessment must document whether the calculated factors of safety for each existing CCR surface impoundment perimeter dike demonstrate the minimum seismic and liquefaction factors of safety specified in paragraphs (e)(1)(iii) and (e)(1)(iv) of the EPA Final CCR Rule for the critical cross section of the embankment.

As mandated by the EPA, TVA is required to evaluate all of its active CCR impoundment facilities for seismic factor of safety and liquefaction factor of safety. The EPA established requirements for the minimum "seismic factor of safety" and the minimum "liquefaction factor of safety". Geocomp interprets what the EPA Final CCR Rule calls "seismic factor of safety" to be what geotechnical engineers call "pseudo-static factor of safety". Geocomp interprets what the EPA's final rule calls "liquefaction factor of safety" to be what geotechnical engineers call "post-earthquake" or "post-shaking" factor of safety. The EPA Final CCR Rule requirements for seismic and liquefaction factors of safety are summarized in Table 1.1.

Table 1.1: Factor of Safety Criteria

CCR Rule Criteria	CCR Rule Factor of Safety Requirements	CCR Rule Reference
Seismic Factor of Safety	≥ 1.00	§257.73(e)(1)(iii)
Liquefaction Factor of Safety	≥ 1.20	§257.73(e)(1)(iv)

1.3 DESCRIPTION OF FACILITY

Cumberland Fossil Plant (CUF) is a coal-fired, electric-generating plant. The CUF facility is located in northwestern Tennessee along State Route 223 near the town of Cumberland City, Tennessee. It is situated on the south bank of the Cumberland River. Figure 1.1 shows a plan view of the Cumberland Fossil Plant and the location of the two CCR impoundment facilities that were analyzed as part of this evaluation. The Stilling Pond (including the Retention Pond) and Bottom Ash Pond are CCR impoundment facilities at CUF. They are located adjacent to the main power plant. Earthen dikes armored with rip rap form the perimeters of these impoundment facilities. Figure 1.1 shows that the Stilling Pond (including Retention Pond) is bordered by Wells Creek to the west and the Cumberland River to the north. The Bottom Ash Pond is bordered by the Dry Ash Stack to the west and the Gypsum Storage Area to the southeast. The western side of the impoundment facilities primarily consists of hilly, grassed, and forested areas. TVA has determined that the Stilling Pond (including the Retention Pond) and the Bottom Ash Pond are CCR surface impoundments and, therefore, are subject to the CCR rules.

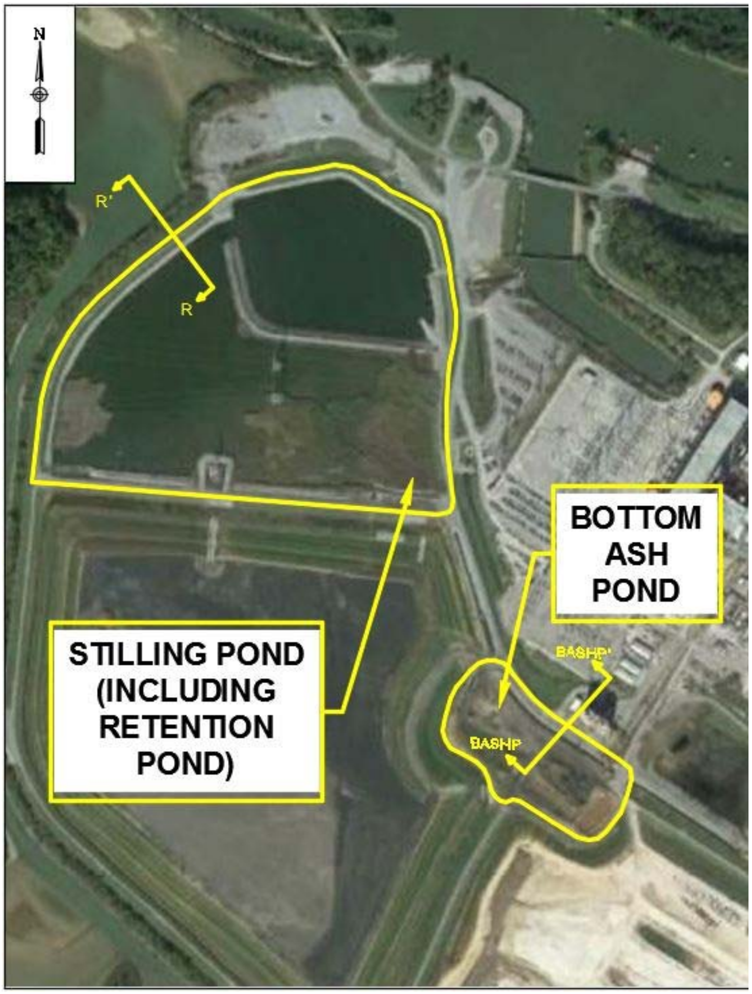


Figure 1.1: Cumberland Fossil Plant

2.0 PROJECT RECONNAISSANCE

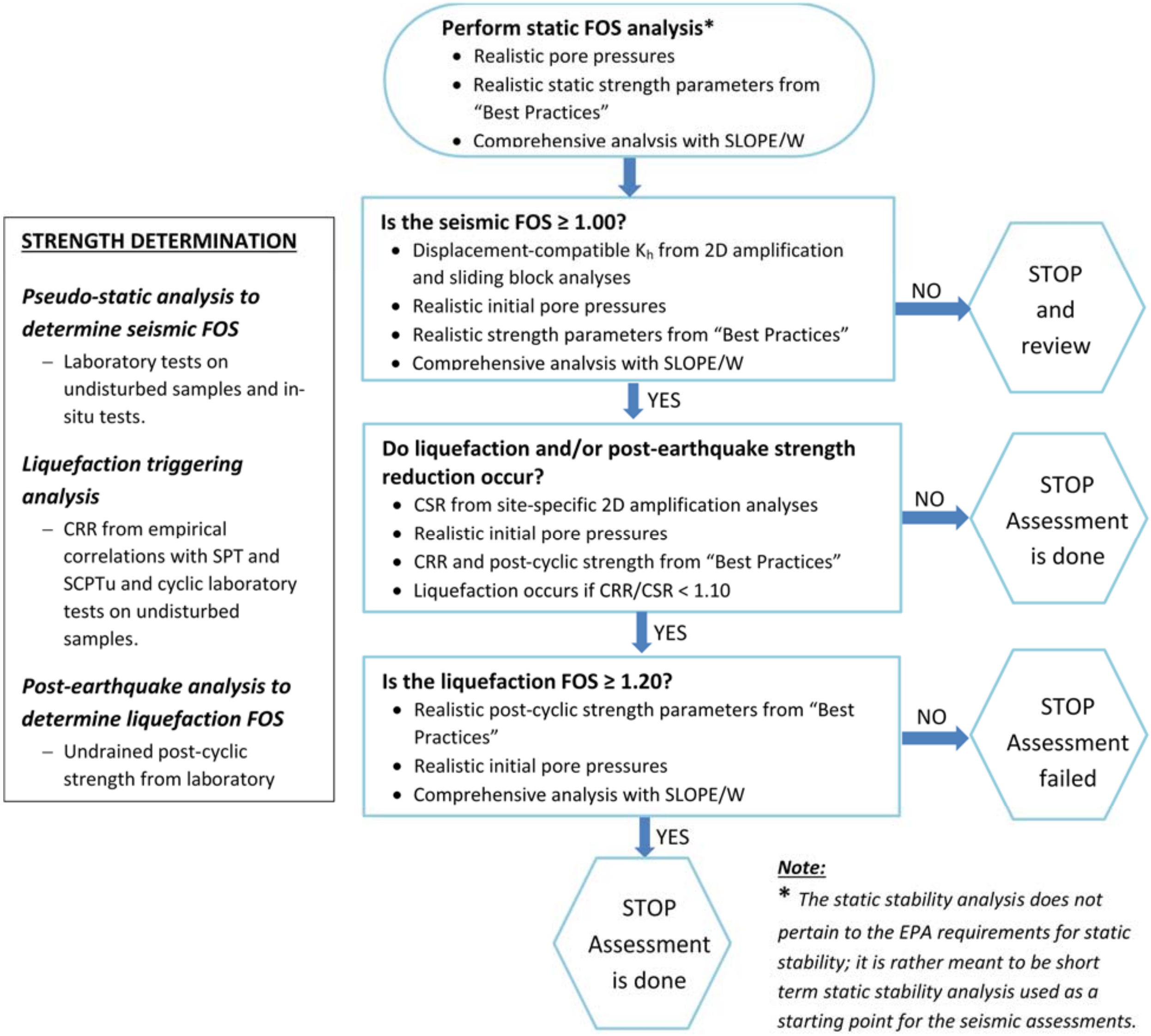
2.1 REVIEW OF EXISTING AND READILY AVAILABLE DATA

Geocomp's review of existing and readily available data included the following documents:

- Atkinson, G., & Beresnev, I. (2002). Ground Motions at Memphis and St. Louis from M7.5-8.0 Earthquakes in the New Madrid Seismic Zone. Bulletin of the Seismological Society of America, 92(3), 1015-1024.
- Cramer, C., Gomberg, J., Schweig, E., Waldron, B., & Tucker, K. (n.d.). The Memphis, Shelby County, Tennessee, Seismic Hazard Maps. United States Geological Survey Open-File Report 04-1294.
- Ford, J., Orchiston, W., & Clendening, R. (2012). The Wells Creek Meteorite Impact Site and Changing Views on Impact Cratering. *Journal of Astronomical History and Heritage*, 15 (3) 159-178.
- Hall, Blake and Associates, Inc. (October 3, 1986). Site Investigation - Cumberland Fossil Plant Soils Investigation for Ash Pond Dike and Borrow Areas.
- Hardeman, W. D. (1966). Geologic Map of Tennessee, Department of Conservation, Tennessee Division of Geology.
- Herbert A. Tiedemann, C. W. (1968). Geological Map of Cumberland City Quadrangle, Tennessee (scale 1:24000). United States Geological Survey.
- Kellberg, J. M. (December 1958). Preliminary Geologic Investigations for the Cumberland City Steam Plant Site. Knoxville, Tennessee.
- Law Engineering and Environmental Services, Inc. (January 27, 1992). Report of Subsurface Exploration and Stability Analyses, Proposed Fly Ash/Scrubber Sludge Disposal Facility, Cumberland Fossil Plant, Cumberland City, Tennessee.
- Law Engineering and Environmental Services, Inc. (March 13, 1992). Recommendations for Stability Improvement, Ash Pond Dike System, Cumberland Fossil Plant, Cumberland City, Tennessee.
- Law Engineering and Environmental Services, Inc. (July 3, 1992). Report of Hydrogeologic Evaluation, Proposed Dry Fly Ash and Gypsum Disposal Facility, TVA Cumberland Fossil Plant, Cumberland City, Tennessee.
- Lewis, R. Q., & Thaden , R. E. (1965). Geologic Map of the Cumberland City Quadrangle, Southern Kentucky. United States Geological Survey.
- Safford, J. M. (1869). Geology of Tennessee. Nashville, Tennessee.
- Stantec Consulting Services, Inc. (2010, June 11). Report of Geotechnical Exploration: Dry Fly Ash Stack and Gypsum Disposal Complex; Cumberland Fossil Plant; Stewart County, Tennessee.
- Stantec Consulting Services, Inc. (2010, March 29). Report of Geotechnical Exploration and Slope Stability Evaluation: Ash Pond; Cumberland Fossil Plant; Stewart County, Tennessee.
- Tennessee Valley Authority - Division of Engineering Design. Report 71-200 (n.d.). Cumberland Steam Plant Final Design Report. Knoxville, Tennessee.
- Tennessee Valley Authority - Office of Engineering Design & Construction. (September 1968). Supplement No. S4 to Engineering Data - TVA Steam Plants. Knoxville, Tennessee.

3.0 APPROACH TO SEISMIC ASSESSMENT

Geocomp's general approach to assess the likely performance of a CCR impoundment under seismic conditions is summarized in Figure 3.1. Please refer to Geocomp (2016) for a detailed discussion of the seismic assessment approach.



Note:

* *The static stability analysis does not pertain to the EPA requirements for static stability; it is rather meant to be short term static stability analysis used as a starting point for the seismic assessments.*

Figure 3.1: Summary of Technical Approach for Seismic Assessment

4.0 SEISMIC ASSESSMENT

4.1 SEISMICITY

CUF is located near the northern end of a thick sequence of Cretaceous period deposits in an area known as the Mississippi Embayment. These thick deposits of sediments have a significant effect on earthquake ground motions. While damaging earthquakes in northwestern Tennessee are only moderately likely, ground motions are dominated by events originating in the New Madrid Fault Zone. This zone is defined by a clustered pattern of earthquake hypocenters between 5 and 15 km deep. CUF is located approximately 103 km from the center of this seismic zone. Figure 4.1 shows the peak and spectral accelerations at a period of 1 second for a 2% probability of exceedance in 50 years, equivalent to a return period of approximately 2,500 years, for the CUF location. These were obtained from the USGS website (http://geohazards.usgs.gov/hazardtool/application.php last accessed 04/12/16). The seismic design criteria are summarized in Table 4.1. The CUF site is located to the east of the New Madrid Fault Zone, and the 2,500-yr ground motion levels are dominated by earthquakes from that source.

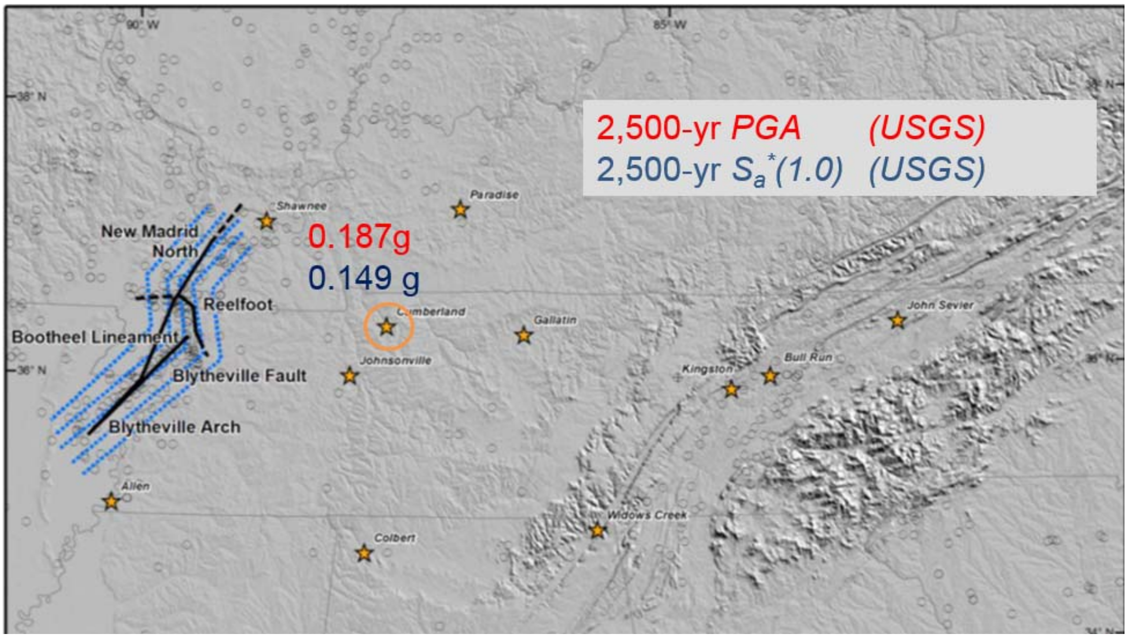


Figure 4.1: Peak Acceleration Values 2500-yr Return Periods at CUF

Table 4.1: Cumberland Fossil Site Summary of Seismic Design Parameters

Parameter	Value
Uniform Hazard Peak Ground Acceleration – Hard Rock (A)	0.187g
Peak Ground Acceleration - Hard Rock (A)	0.22g(1)
Peak Ground Acceleration - Soil Class (D)	0.34g(1)
Spectral acceleration at a period of 1.0 second	0.15g
Spectral acceleration at a period of 0.2 second	0.34g

Parameter	Value
Mean moment magnitude	M6.99
Mean distance to seismic event	103.3 km
Uniform Hazard Response Spectrum	See Figure 4.2

(1) Design PGA values from the 2015 NEHRP Provisions (FEMA 2015).

The site-specific seismic amplification analyses used seven ground motion time histories that have spectral contents that match the uniform hazard response spectrum (UHRS) are shown in Figure 4.2. The development of these input ground motions started with the selection of recorded earthquake time histories that approximate the design criteria. These motions have response spectra with shapes similar to that of the UHRS and ground motion durations within the range expected for the deaggregated mean magnitude. The recorded motions were then modified by adding and subtracting wavelets using the software SeismoMatch (SeismoSoft 2016) until the resulting response spectra closely match the UHRS.

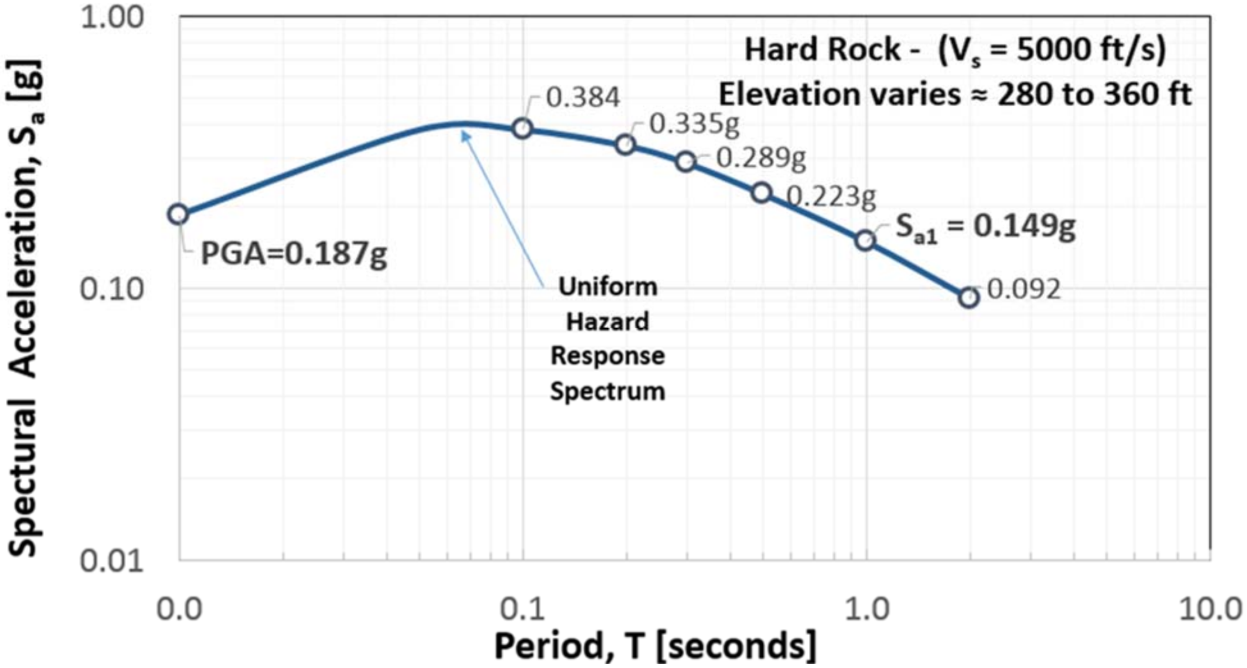


Figure 4.2: USGS Uniform Hazard Response Spectra (UHRS)

The site specific seismic study conducted for CUF is based on the design response spectra developed by USGS as required by the new EPA Final CCR Rule and presented in Figure 4.2. Table 4.2 summarizes the seven time history records selected for the site-specific amplification analysis.

Table 4.2: Time Histories Used in the Analysis

Motion	Earthquake	Arias Intensity [cm/sec]	Significant Duration [sec]	Spectral Acceleration at 1s Sa(1s) [g]
Motion 1	Synthetic Bootheel	198	41	0.148
Motion 2	Synthetic Reelfoot	35	22	0.112

Motion	Earthquake	Arias Intensity [cm/sec]	Significant Duration [sec]	Spectral Acceleration at 1s Sa(1s) [g]
Motion 3	Iwate, Japan @ Machimukai Town, UD, 2008 (ROT50)	48.8	30	0.159
Motion 4	Darfield, New Zealand @CSHS 2010 (ROT50)	32.4	24	0.142
Motion 5	Caleta de Campos, 1985	47.3	27	0.129
Motion 6	Chi Chi TCU046 ROTD50	43.2	18	0.168
Motion 7	El Mayor-Cucapah, Mexico @ Sam W. Stewart 2010 (ROT50)	9.6	31	0.066

4.2 SELECTION OF CROSS SECTIONS FOR ASSESSMENT

Geocomp's review of existing and readily available information resulted in the selection of three cross sections as potential critical cross sections for further evaluation. Two cross sections were selected at the Stilling Pond (including Retention Pond) and one cross section was selected at the Bottom Ash Pond as potential critical cross sections for further evaluation. In addition to this, two additional cross sections, located at the Stilling Pond (including Retention Pond), were selected for site subsurface investigation. These two cross sections were not considered as potential critical cross sections but were included into the site subsurface investigation program to provide supplemental information to fill potential data gaps, if any, at the two potential critical cross sections located at the Stilling Pond (including Retention Pond). The selection of the cross section at the Bottom Ash Pond, which included limited site subsurface investigations across a 300-foot length segment along the crest of the interior dike, was developed under TVA's direction. To the best of Geocomp's knowledge at that point in time, no previous site subsurface investigations had been performed at the interior dike of the Bottom Ash Pond.

In July 2009 through August 2009, Stantec performed a geotechnical exploration at the Stilling Pond (including Retention Pond) at CUF. The results of this exploration are presented in Stantec (2010). The site subsurface exploration performed by Stantec for their evaluation consisted of advancing thirty borings and installing seven piezometers at eight cross sections across the Stilling Pond (including Retention Pond). Stantec evaluated each cross section to obtain minimum factors of safety against piping and static slope stability for steady-state seepage conditions. Cross Sections P-P', Q-Q', and R-R' had the lowest factors of safety against piping which were 1.3, 2.4, and 2.6, respectively. All analyzed cross sections were found to have a static factor of safety greater than 1.5 for both the downstream and upstream slopes. Stantec's evaluation did not include seismic slope stability assessments.

Our review of readily available information on dike geometries and subsurface conditions from borings performed by Stantec in 2009, and soil profiles and static factors of safety presented in Stantec (2010) for the Stilling Pond (including Retention Pond) (Cross Sections P-P', Q-Q', R-R', S-S', T-T', U-U', V-V', and W-W') indicated the following:

- Cross Sections P-P', Q-Q', and R-R' had the highest slope heights, which were approximately 30 to

40 ft. The toe of these cross sections were located adjacent to Wells Creek indicating a more significant threat in the potential event of loss of containment.

- Cross Sections P-P’ and R-R’ appeared to have the highest phreatic surfaces.
- The static factors of safety reported by Stantec (2010) are summarized in Table 4.3.

Table 4.3: Static Factors of Safety Reported by Stantec (2010)

Cross Section	Minimum Reported Static Factor of Safety
P-P’	1.7
Q-Q’	1.9
R-R’	2.1
S-S’	2.5
T-T’	2.9
U-U’	2.1
V-V’	2.8
W-W’	2.7

- Cross Section P-P’ appeared to have the shallowest rock, which was indicative of potentially higher seismic amplification effects. Table 4.4 provides an approximate depth from ground level to rock as interpreted by Stantec (2010) for the cross sections in the Stilling Pond (including Retention Pond) adjacent to Wells Creek.

Table 4.4: Stantec (2010) Approximate Depth to Rock

Cross Section	Approximate Depth to Rock (ft)	
	At Toe	At Crest
P-P’	40	55
Q-Q’	60	90
R-R’	70	85

- Within the alluvial lean clay layer underlying the toe of Cross Sections P-P’ and Q-Q’, Stantec reported zones with SPT-N values ranging from 2 to 5 bpf, which indicated the possibility of low strength for stability. Within the alluvial lean clay layer underlying the crest and toe of Cross Section R-R’, Stantec reported zones with SPT-N values ranging from 2 to 4 bpf, which also indicated the possibility of low strength for stability.

Based on the information presented above, Geocomp selected Cross Sections P-P’ and R-R’ at the Stilling Pond (including Retention Pond) for further field exploration and testing. Cross sections at the northern perimeter of the Stilling Pond (including Retention Pond) generally had higher static factors of safety, lower slope heights, and were not located adjacent to Wells Creek. As previously mentioned, two cross sections (Cross Section S-S’ and T-T’) at the Stilling Pond (including Retention Pond) were included in the site subsurface investigation program to fill in potential data gaps and provide supplemental information for this impoundment.

The subsurface conditions at Cross Sections P-P’ and Q-Q’ were generally similar, apart from the higher phreatic surface and shallower rock at Cross Section P-P’. These differences, along with the lower static factor of safety for Cross Section P-P’, were the basis for selecting Cross Section P-P’ over Cross Section Q-Q’ for further exploration. Geocomp recommended that additional site subsurface explorations be performed at both the crest and toe of Cross Section P-P’.

Cross Section R-R’ differed from Cross Sections P-P’ and Q-Q’ in the following ways:

- Stantec reported the presence of a continuous layer of silty sand across the crest and toe of Cross Section R-R’. This continuous layer was not found at Cross Sections P-P’ and Q-Q’.
- The dike at Cross Section R-R’ was underlain by lean clay, whereas the dikes at Cross Sections P-P’ and Q-Q’ were first underlain by lean clay with considerable gravel and sand followed by the lean clay layer.

In addition to its high phreatic surface, Cross Section R-R’ was selected for further exploration in order to evaluate the seismic stability of a potentially critical subsurface profile and geometry which was different from Cross Sections P-P’ and Q-Q’. Geocomp recommended that additional site subsurface explorations be performed at both the crest and toe of Cross Section R-R’.

During execution of field work at Cross Sections P-P’ and R-R’ in November through December, 2015, the field crew encountered difficulties accessing the toe of these cross sections due to the steepness of the dike slope. Further attempts to make borings and push SCPTu at the toe were abandoned in accordance with instructions from TVA personnel who cited potential site safety concerns. Following this, Geocomp looked for alternatives of drilling at the toe of the Stilling Pond (including Retention Pond) along Wells Creek where toe access would not be an issue. However, no suitable alternative location was found where access and safety were not an issue. Geocomp recommended that additional subsurface information at the crest of Cross Sections S-S’ and T-T’ be collected to support our evaluations.

The selection of the Cross Section BAshP-BAshP’ at the Bottom Ash Pond was based on information available on bedrock depth and pore water pressure conditions.

4.3 FIELD INVESTIGATIONS

The field investigation for the current assessment included 8 borings with standard penetration testing and undisturbed sampling and 13 seismic cone penetration testing with pore pressure measurements (SCPTu). The generalized subsurface conditions encountered at the critical cross section Stilling Pond (including the Retention Pond) and are summarized in Table 4.5. The generalized subsurface conditions encountered at the critical cross section at the Bottom Ash Pond are summarized in Table 4.6. A more in-depth presentation of the field investigation is found in Geocomp (2016).

Table 4.5: Generalized Subsurface Conditions at Cross Section R-R’

CUF Cross Section R-R'		
Soil Layer	**Approximate Elevation (ft)**	**General Consistency/Density**
Clay Dike	395-355	Medium stiff to very stiff lean CLAY with varying amounts of Sand and Gravel with average PI=29
Subgrade (Fill)	355-346	Stiff lean CLAY, medium clayey GRAVEL and clayey SAND with gravel with average PI=27
Alluvial Clay	346-335	Medium stiff to stiff lean CLAY with varying proportions of sand and gravel with average PI=17
Alluvial Sand and Gravel	335-307	Medium dense clayey GRAVEL and lean CLAY with average PI=15 which transitions to loose to medium dense non-plastic silty SAND and silty GRAVEL

Table 4.6: Generalized Subsurface Conditions at Cross Section BAshP-BAshP’

CUF Cross Section BAshP-BAshP'		
Soil Layer	**Approximate Elevation (ft)**	**General Consistency/Density**
Bottom Ash_1	406-393	Dense to very dense non-plastic silty SAND (Bottom Ash)
Bottom Ash_2	393-391	Medium dense non-plastic silty SAND (Bottom Ash)
Bottom Ash_3	391-381	Loose non-plastic silty SAND (Bottom Ash)
Bottom Ash_4	381-373	Medium dense non-plastic silty SAND (Bottom Ash)
Alluvial Clay	373-360	Stiff fat CLAY with varying amounts of Sand and Gravel with average PI=54

Following the 2015 geotechnical explorations, Geocomp performed a laboratory testing program consisting of index parameters including natural moisture content, sieve and hydrometer analyses, Atterberg limits, specific gravity, and bulk density. The laboratory testing program also included constant rate of strain consolidation tests, direct shear tests, direct simple shear tests, cyclic direct simple shear with post cyclic monotonic strength measurement, modulus and damping versus strain using fixed-based resonant column, and shear wave measurements using bender element sensors. Geocomp (2016) provides detailed results of the laboratory testing program.

4.4 WATER CONDITIONS

4.4.1 Surface Water

This section discusses the surface water conditions for the critical cross sections. For the Stilling Pond (including Retention Pond) a water level of El. 378.2 feet was used for the analyses. This was based on information obtained from correspondence with TVA and confirmed by LIDAR survey data provided by TVA and dated July 30, 2015, and readings taken from water level gauge “CUF_Pond_PL” between

3/1/2015 and 12/31/2015. For Wells Creek, which runs south to north adjacent to the southern, western, and northern perimeter dikes, a surface water level of El. 359.5 feet was used for the analyses. This was based on data from river gauge data provided by TVA from water level gauge “CUF_Cumberland River_RL” located in the Cumberland River between 3/1/2015 and 12/31/2015. Table 4.7 summarizes the surface water levels used in the seismic assessment of the critical cross section at Stilling Pond (including Retention Pond).

Table 4.7: Surface Water Levels at Cross Section R-R’

Surface Water Levels	
Waterbody	**Total Head (ft)**
Stilling Pond (including Retention Pond)	378.2
Wells Creek	359.5

For the critical cross section at the Bottom Ash Pond, Cross Section BAshP-BAshP’, the surface water levels of El. 398 and 396 feet were used for the dredged cells south of the Bottom Ash Pond for the analyses. A surface water elevation of 389-feet was used for the drainage ditch located at base of the slope. These elevations were based on the LIDAR data dated July 30, 2015, provided by TVA. There were no water level data stations installed for the dredged cells at the time this report was prepared. Table 4.8 summarizes the surface water levels used in the seismic assessment of the critical cross section at the Bottom Ash Pond.

Table 4.8: Surface Water Levels at Cross Section BAshP-BAshP’

Surface Water Levels	
Waterbody	**Total Head (ft)**
West dredged cell	396.0
East dredged cell	398 .0
Drainage ditch	389.0

4.4.2 Pore Water

The pore water conditions in the vicinity of the critical cross section in the Stilling Pond (including Retention Pond), Cross Section R-R’, are hydrostatic within the Dike and Subgrade. Beneath the Subgrade, the piezometer data show a decrease in total head within the Alluvial Clay. This indicates downward flow and pore water conditions below the phreatic surface that are less than hydrostatic. Beneath the Alluvial Clay, the total head is hydrostatic through the Alluvial Sand and Gravel layers. Table 4.9 summarizes the piezometric conditions used in the analyses for the critical cross section at the Stilling Pond (including Retention Pond).

Table 4.9: Summary of Total Head Data at Cross Section R-R’

CUF Cross Section R-R’ Total Head		
Soil Layer	Total Head at Crest (ft)	Total Head at Toe (ft)
Clay Dike	376.0	359.5
Subgrade (Fill)	376.0	359.5
Alluvial Clay	370.5	359.5
Alluvial Sand and Gravel	364.3	359.5

The piezometric levels at the critical cross section at the Bottom Ash Pond, Cross Section BAshP-BAshP’, were determined based on SCPTu pore pressure dissipation tests performed during the most recent exploration. The pore water conditions in the vicinity of Cross Section BAshP-BAshP’ were hydrostatic within the dike. Beneath the Dike, SCPTu pore pressure dissipation test data show a decrease in total head within the Alluvial Clay. Table 4.10 summarizes the piezometric conditions used in the analyses for the critical cross section at the Bottom Ash Pond.

Table 4.10: Summary of Total Head Data at Cross Section BAshP-BAshP’

CUF Cross Section BAshP-BAshP’ Total Head		
Soil Layer	**Total Head at Crest (ft.)**	**Total Head at Toe (ft.)**
Bottom Ash	394.6	389.0
Alluvial Clay	389.5	389.0

4.5 MATERIAL PARAMETERS

Table 4.11 and Table 4.12 summarize the material parameters used in the site-specific amplification analyses and for the calculation of seismic factors of safety at the critical Cross Sections R-R’ and BAshP-BAshP’, respectively. Table 4.13 and Table 4.14 summarize the material parameters used for the calculations of liquefaction factors of safety at the critical Cross Sections R-R’ and BAshP-BAshP’, respectively. Please refer to Geocomp (2016) for details of the soil parameter development.

Table 4.11: Parameters used for Amplification Analyses and Calculation of Seismic Factor of Safety at Cross Section R-R’

Soil Layers	Strength Parameters			Amplification Parameters			
	Unit Weight (pcf)	Friction Angle, ϕ’ (deg)	Undrained Shear Strength (psf)	$V_s^{(1)}$ (ft/s)	$G_{max}^{(2)}$ (psf)	$G_o^{(3)}$ (psf)	$n^{(4)}$
Dike 2_Clay Upper	120	--	1000	980	3.58+06	4.08E+06	0.1
Dike 2_Clay Lower(6)	120	--	1000	980	3.58+06	3.56E+06	0.1
Subgrade (Fill)_1	120	--	1000	1200	5.37E+06	3.49E+06	1.0
Subgrade (Fill)_2	120	--	650	660	1.60E+06	3.36E+06	1.0
Subgrade (Fill)_3	120	--	300	300	3.30E+05	3.36E+06	1.0
Alluvial Clay(6)	118	--	1500	900	2.97E+06	2.29E+06	0.5

Soil Layers	Strength Parameters			Amplification Parameters			
	Unit Weight (pcf)	Friction Angle, ϕ' (deg)	Undrained Shear Strength (psf)	V_s (1) (ft/s)	G_{max} (2) (psf)	G_o(3) (psf)	n(4)
Alluvial Sand and Gravel	125	30(7)	--	950	3.51E+06	2.58E+06	0.5
Weathered Shale	130	Strong (5)		1940	1.52E+07	--	7.0
Bedrock SHALE/LIMESTONE	150	Strong (5)		5000	1.17E+08	--	--

(1) V_s = shear wave velocity values selected from field SCPTu tests

(2) G_{max} = Low strain shear modulus

(3) G_o = Reference low strain shear modulus at atmospheric pressure (1 atm)

(4) n = Exponent on normalized initial mean effective stress

(5) In stability analyses the critical failure surfaces are not expected to extend into bedrock depths. The strength parameters are assumed much stronger that the soil layers to contain the slip surface search to within the soil layers.

(6) Shear modulus degradation and damping curves obtained from resonant column test performed on an undisturbed sample of this material.

(7) Direct Shear test reported a friction angle of 30 deg and a cohesion of 455 psf. Geocomp (October 2016) provides detailed results of the laboratory testing program.

Table 4.12: Parameters used for Amplification Analyses and Calculation of Seismic Factor of Safety at Cross Section BAshP-BAshP'

Soil Layers	Strength Parameters			Amplification Parameters			
	Unit Weight (pcf)	Friction Angle, ϕ' (deg)	Undrained Shear Strength (psf)	V_s (1) (ft/s)	G_{max} (2) (psf)	G_o(3) (psf)	n(4)
Bottom Ash_1	128	40	--	870	3.01E+06	3.70E+06	0.1
Bottom Ash_2	120	35	--	670	1.67E+06	1.81E+06	0.1
Bottom Ash_3	115	35	--	520	9.67E+05	1.03E+06	0.1
Bottom Ash_4	120	37	--	720	1.93E+06	2.01E+06	0.1
Alluvial Clay	114	--	900	830	2.44E+06	2.47E+06	0.5
Weathered Shale	130	Strong(5)		2140	1.85E+07	1.72E+07	--
Bedrock SHALE	150	Strong(5)		5000	1.17E+08	--	--

Note: Please refer to Table 4.11 for an explanation of footnotes (1) - (5)

Table 4.13: Parameters used for Calculation of Liquefaction Factor of Safety at Cross Section R-R'

Soil Layers	Static Strength Parameters			Post-Earthquake Strength Parameters		
				Sand-Like Materials	Clay-Like Materials	
	Unit Weight (pcf)	Friction Angle, ϕ' (deg)	Undrained Shear Strength (psf)	Undrained Residual Shear Strength S_r/σ'_v(2)	Undrained Shear Strength (psf)	Post-Cyclic Strength Reduction(1) (%)
Dike 2_Clay Upper	120	--	1000	--	1000	--
Dike 2_Clay Lower	120	--	1000	--	1000	--
Subgrade (Fill)_1	118	--	1000	--	1000	--
Subgrade (Fill)_2	118	--	650	--	650	--
Subgrade (Fill)_3	118	--	300	--	300	--
Alluvial Clay	118	--	1500	--	970	35
Alluvial Sand and Gravel	125	30	--	0.062(4) - 0.14(3)	--	--
Weathered Shale	130	Strong (5)		Strong (5)		
Bedrock SHALE/LIMESTONE	150	Strong (5)		Strong (5)		

(1) Strength reductions (%) applied to clay-like materials were based on laboratory post-cyclic undrained shear strength test results and experience from similar materials at other TVA sites

(2) S_r/σ'_v represents the residual undrained strength assigned to sand-like materials based on laboratory post-cyclic undrained shear strength test results and SPT- and CPT-based correlations (Idriss and Boulanger, 2008)

(3) Residual strength determined based on SPT-based correlations (Idriss and Boulanger, 2008)

(4) Residual strength determined based on CPT-based correlations (Idriss and Boulanger, 2008)

(5) In stability analyses the critical failure surfaces are not expected to extend into bedrock depths. The strength parameters are assumed much stronger that the soil layers to contain the slip surface search to within the soil layers.

Table 4.14: Parameters used for Calculation of Liquefaction Factor of Safety at Cross Section BAshP-BAshP'

Soil Layers	Static Strength Parameters			Post-Earthquake Strength Parameters		
				Sand-Like Materials	Clay-Like Materials	
	Unit Weight (pcf)	Friction Angle, ϕ' (deg)	Undrained Shear Strength (psf)	Undrained Residual Shear Strength S_r/σ'_v(2)	Undrained Shear Strength (psf)	Post-Cyclic Strength Reduction(1) (%)
Bottom Ash_1	128	40	--	--	--	--
Bottom Ash_2	120	35	--	--	--	--
Bottom Ash_3	115	35	--	--	--	--
Bottom Ash_4	120	37	--	0.28-0.75(3)	--	--
Alluvial Clay	114	--	900	--	760	15
Weathered Shale	130	Strong (5)		Strong (5)		
Bedrock SHALE/LIMESTONE	150	Strong (5)		Strong (5)		

Note: Please refer to Table 4.13 for explanation of footnotes (1)-(5)

4.6 SEISMIC ANALYSES

4.6.1 Site-Specific Amplification

Site-specific two-dimensional amplification analyses were performed to model the seismic response of the soil profile at CUF. Two-dimensional equivalent-linear amplification analyses were performed at all potentially critical cross sections evaluated using the finite element program QUAD4M to help determine the most critical cross section(s). Two-dimensional non-linear amplification analyses were performed at the most critical cross sections using the finite element program OpenSees and the results were verified with the finite difference program FLAC. Dynamic models and analyses with QUAD4M and OpenSees were carried out by the University of Washington team led by Professors Arduino and Kramer. Geocomp was responsible for the verification of OpenSees results using FLAC models.

The results of these analyses were used to determine displacement-compatible accelerations used in the seismic slope stability analyses to calculate the seismic factor of safety. The results of these analyses were also used to determine cyclic shear stresses for cyclic laboratory testing to measure post-cyclic strengths for the post-earthquake slope stability analyses to calculate the liquefaction factor of safety. Geocomp (2016) provides details for the site-specific amplification analyses.

4.6.2 Seismic Factor of Safety

The seismic stability factor of safety of the dike cross sections was evaluated under seismic loading conditions using pond levels and piezometric surfaces determined by existing instrumentation and survey data provided by TVA. The pseudo-static loading conditions were determined from applied displacement-compatible accelerations. The applied displacement-compatible accelerations were selected from results of the sliding block analyses. Details of the sliding block analyses are described in detail in Geocomp (2016). These use an allowable seismic displacement of 18 inches to determine the equivalent horizontal earthquake coefficient for pseudo-static stability.

4.6.3 Liquefaction Triggering

The assessment of the factor of safety against liquefaction of discrete materials and layers within the soil profiles at each cross section was performed using a combination of the stress-based approach proposed by Idriss and Boulanger (2008) and updates to the approach proposed by Boulanger and Idriss (2014). The stress-based approach was used to calculate the Cyclic Resistance Ratio (CRR) through the soil profile. The results of the site-specific two-dimensional amplification analysis were used to obtain the Cyclic Stress ratio (CSR) through the soil profile. The factor of safety against liquefaction for discrete materials and layers was then calculated as CRR/CSR. If this value was less than 1.10, then the specific material or layer evaluated was considered potentially liquefiable for the maximum design earthquake, and its post-cyclic undrained residual strength was evaluated for use in the post-earthquake stability analysis.

4.6.4 Liquefaction Factor of Safety

The liquefaction factor of safety was calculated to evaluate the stability of the cross sections under post-earthquake conditions. For the purposes of liquefaction hazard evaluation, soils are often described as exhibiting “sand-like” or “clay-like” behavior. Sand-like soils typically have plasticity indices less than 7, and are considered susceptible to liquefaction. Clay-like soils have higher plasticity indices and are not considered susceptible to liquefaction, although they may experience some degree of strength loss from cyclic loading. Under these conditions, the sand-like materials that could potentially liquefy are modeled with undrained residual strengths. Sand-like materials that are not expected to liquefy are modeled with static strength parameters. Clay-like materials are modeled with reduced undrained shear strengths if they are expected to exhibit post-cyclic softening; otherwise these materials are modeled with undrained strength parameters. Piezometric conditions were kept the same for all seismic and post-earthquake stability analyses.

4.7 SELECTION OF CRITICAL CROSS SECTION

According to the results of seismic and post-earthquake slope stability analyses performed at CUF on the two chosen cross sections at the Stilling Pond (including Retention Pond), Cross Section R-R’ is the critical cross section for both the Stilling Pond (including Retention Pond) in terms of seismic and liquefaction factors of safety. Cross Section BAshP-BAshP’ is the critical cross section for the Bottom Ash Pond in terms of seismic and liquefaction factors of safety.

5.0 ANALYSIS RESULTS

The slope stability results were obtained with the two-dimensional limit equilibrium program SLOPE/W. The seismic and liquefaction factors of safety for the critical cross sections at CUF are summarized in Table 5.1. The minimum factors of safety reported in this table correspond to slip surfaces that could potentially result in the uncontrolled release of water and CCR materials from within the impoundment during or after the maximum design earthquake. Results of these stability analyses using the limit equilibrium slope stability method are presented in Appendix A of this document. Please refer to Geocomp (O2016) gives details of the slope stability analyses at CUF.

Table 5.1: Summary of Seismic Assessment Results at CUF critical cross sections

Plant	Facility	Cross Section	EPA Criteria	EPA Final CCR Rule Required Factor of Safety	Calculated Minimum Factor of Safety
CUF	Stilling Pond (including Retention Pond)	R-R’	Seismic Factor of Safety	≥ 1.00	1.10
			Liquefaction Factor of Safety	≥ 1.20	1.22
CUF	Bottom Ash Pond	BAshP-BAshP’	Seismic Factor of Safety	≥ 1.00	1.24
			Liquefaction Factor of Safety	≥ 1.20	1.29

6.0 CONCLUSIONS

This report documents the evaluation of seismic and liquefaction factors of safety of Cumberland Fossil Plant's Stilling Pond (including Retention Pond) and Bottom Ash Pond. The evaluation was performed in accordance with section §257.73(e) of the EPA Final CCR Rule.

The seismic assessment at Cross Section R-R', considered to be the critical cross section for the Stilling Pond (including the Retention Pond), resulted in a seismic factor of safety of 1.10 [§257.73(e)(1)(iii)] and a liquefaction factor of safety of 1.22 [§257.73(e)(1)(iv)]. These results meet or exceed the minimum required seismic factor of safety of 1.00 and liquefaction factor of safety of 1.20.

The seismic assessment at Cross Section BAshP- BAshP', considered to be the critical section for the Bottom Ash Pond, resulted in a seismic factor of safety of 1.24 [§257.73(e)(1)(iii)] and a liquefaction factor of safety of 1.29 [§257.73(e)(1)(iv)]. These results meet or exceed the minimum required seismic factor of safety of 1.00 and liquefaction factor of safety of 1.20.

7.0 REFERENCES

Note: These references were used for the described work and are cited in the Geocomp 2016 report. They are not all cited in this summary report.

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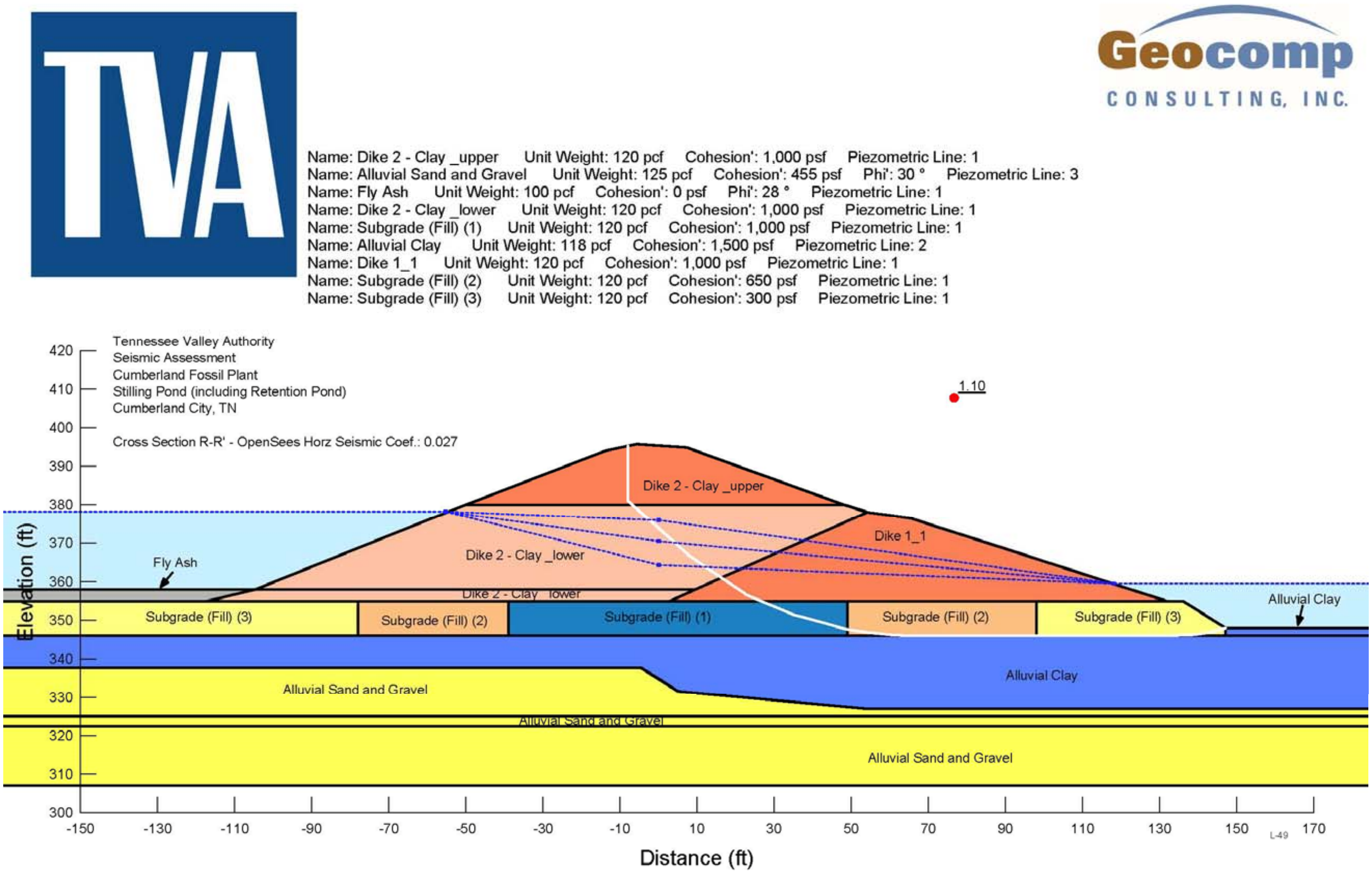
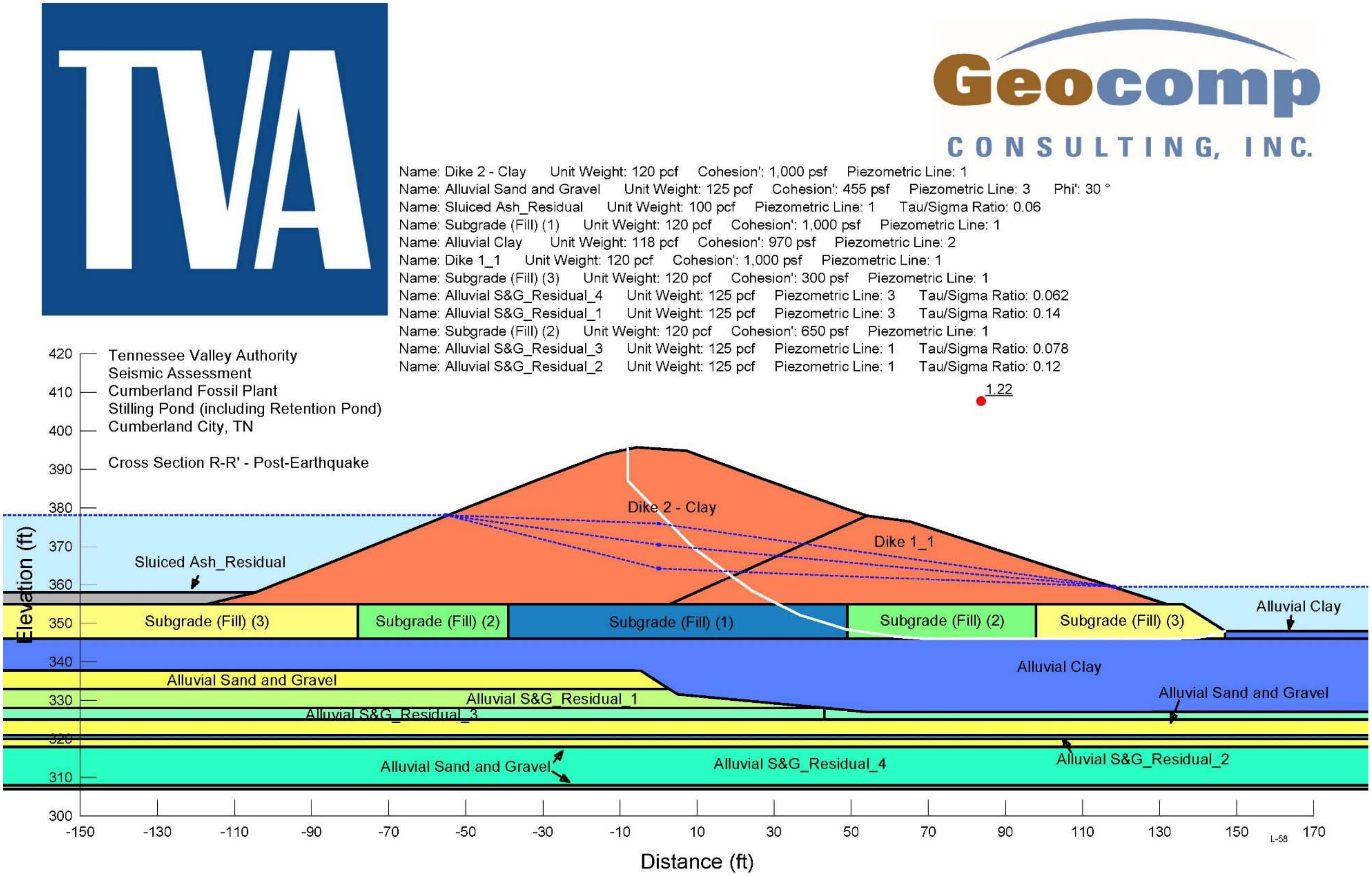
Appendix A

Slope Stability Analysis Results for Critical Cross Sections



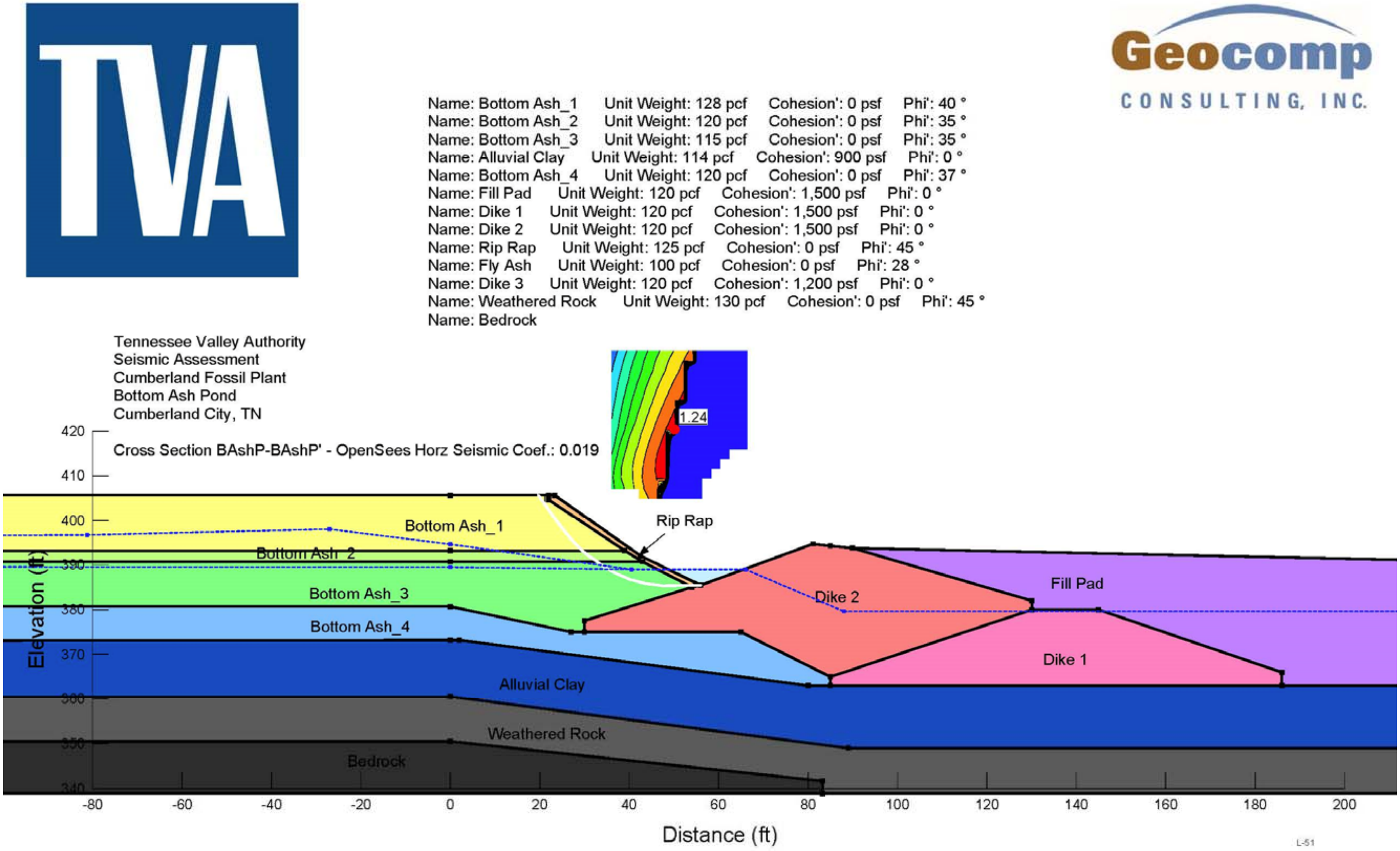
Name: Dike 2 - Clay _upper Unit Weight: 120 pcf Cohesion': 1,000 psf Piezometric Line: 1
Name: Alluvial Sand and Gravel Unit Weight: 125 pcf Cohesion': 455 psf Phi': 30 ° Piezometric Line: 3
Name: Fly Ash Unit Weight: 100 pcf Cohesion': 0 psf Phi': 28 ° Piezometric Line: 1
Name: Dike 2 - Clay _lower Unit Weight: 120 pcf Cohesion': 1,000 psf Piezometric Line: 1
Name: Subgrade (Fill) (1) Unit Weight: 120 pcf Cohesion': 1,000 psf Piezometric Line: 1
Name: Alluvial Clay Unit Weight: 118 pcf Cohesion': 1,500 psf Piezometric Line: 2
Name: Dike 1_1 Unit Weight: 120 pcf Cohesion': 1,000 psf Piezometric Line: 1
Name: Subgrade (Fill) (2) Unit Weight: 120 pcf Cohesion': 650 psf Piezometric Line: 1
Name: Subgrade (Fill) (3) Unit Weight: 120 pcf Cohesion': 300 psf Piezometric Line: 1

Figure A. 1: Seismic Factor of Safety by Seismic Slope Stability Analysis at Cross Section R-R'



Name: Dike 2 - Clay Unit Weight: 120 pcf Cohesion': 1,000 psf Piezometric Line: 1
Name: Alluvial Sand and Gravel Unit Weight: 125 pcf Cohesion': 455 psf Piezometric Line: 3 Phi': 30 °
Name: Sluiced Ash_Residual Unit Weight: 100 pcf Piezometric Line: 1 Tau/Sigma Ratio: 0.06
Name: Subgrade (Fill) (1) Unit Weight: 120 pcf Cohesion': 1,000 psf Piezometric Line: 1
Name: Alluvial Clay Unit Weight: 118 pcf Cohesion': 970 psf Piezometric Line: 2
Name: Dike 1_1 Unit Weight: 120 pcf Cohesion': 1,000 psf Piezometric Line: 1
Name: Subgrade (Fill) (3) Unit Weight: 120 pcf Cohesion': 300 psf Piezometric Line: 1
Name: Alluvial S&G_Residual_4 Unit Weight: 125 pcf Piezometric Line: 3 Tau/Sigma Ratio: 0.062
Name: Alluvial S&G_Residual_1 Unit Weight: 125 pcf Piezometric Line: 3 Tau/Sigma Ratio: 0.14
Name: Subgrade (Fill) (2) Unit Weight: 120 pcf Cohesion': 650 psf Piezometric Line: 1
Name: Alluvial S&G_Residual_3 Unit Weight: 125 pcf Piezometric Line: 1 Tau/Sigma Ratio: 0.078
Name: Alluvial S&G_Residual_2 Unit Weight: 125 pcf Piezometric Line: 1 Tau/Sigma Ratio: 0.12

Figure A. 2: Liquefaction Factor of Safety by Post-Earthquake Slope Stability Analysis at Cross Section R-R'

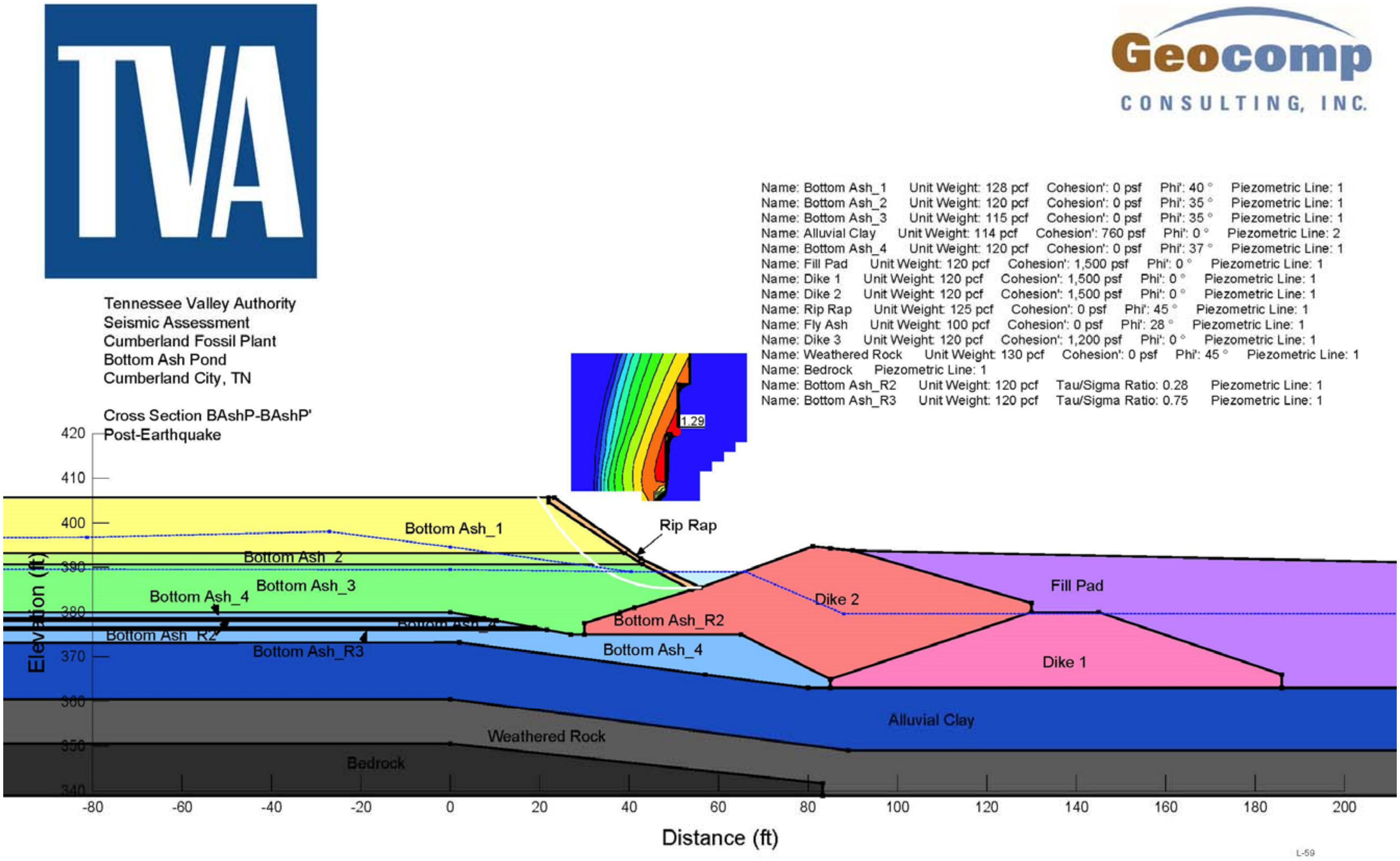


Name: Bottom Ash_1 Unit Weight: 128 pcf Cohesion': 0 psf Phi': 40 °
Name: Bottom Ash_2 Unit Weight: 120 pcf Cohesion': 0 psf Phi': 35 °
Name: Bottom Ash_3 Unit Weight: 115 pcf Cohesion': 0 psf Phi': 35 °
Name: Alluvial Clay Unit Weight: 114 pcf Cohesion': 900 psf Phi': 0 °
Name: Bottom Ash_4 Unit Weight: 120 pcf Cohesion': 0 psf Phi': 37 °
Name: Fill Pad Unit Weight: 120 pcf Cohesion': 1,500 psf Phi': 0 °
Name: Dike 1 Unit Weight: 120 pcf Cohesion': 1,500 psf Phi': 0 °
Name: Dike 2 Unit Weight: 120 pcf Cohesion': 1,500 psf Phi': 0 °
Name: Rip Rap Unit Weight: 125 pcf Cohesion': 0 psf Phi': 45 °
Name: Fly Ash Unit Weight: 100 pcf Cohesion': 0 psf Phi': 28 °
Name: Dike 3 Unit Weight: 120 pcf Cohesion': 1,200 psf Phi': 0 °
Name: Weathered Rock Unit Weight: 130 pcf Cohesion': 0 psf Phi': 45 °
Name: Bedrock

Tennessee Valley Authority
Seismic Assessment
Cumberland Fossil Plant
Bottom Ash Pond
Cumberland City, TN

Cross Section BAshP-BAshP' - OpenSees Horz Seismic Coef.: 0.019

Figure A. 3: Seismic Factor of Safety by Seismic Slope Stability Analysis at Cross Section BAshP-BAshP'



Tennessee Valley Authority
Seismic Assessment
Cumberland Fossil Plant
Bottom Ash Pond
Cumberland City, TN

Cross Section BAshP-BAshP'
Post-Earthquake

Name: Bottom Ash_1 Unit Weight: 128 pcf Cohesion': 0 psf Phi': 40 ° Piezometric Line: 1
Name: Bottom Ash_2 Unit Weight: 120 pcf Cohesion': 0 psf Phi': 35 ° Piezometric Line: 1
Name: Bottom Ash_3 Unit Weight: 115 pcf Cohesion': 0 psf Phi': 35 ° Piezometric Line: 1
Name: Alluvial Clay Unit Weight: 114 pcf Cohesion': 760 psf Phi': 0 ° Piezometric Line: 2
Name: Bottom Ash_4 Unit Weight: 120 pcf Cohesion': 0 psf Phi': 37 ° Piezometric Line: 1
Name: Fill Pad Unit Weight: 120 pcf Cohesion': 1,500 psf Phi': 0 ° Piezometric Line: 1
Name: Dike 1 Unit Weight: 120 pcf Cohesion': 1,500 psf Phi': 0 ° Piezometric Line: 1
Name: Dike 2 Unit Weight: 120 pcf Cohesion': 1,500 psf Phi': 0 ° Piezometric Line: 1
Name: Rip Rap Unit Weight: 125 pcf Cohesion': 0 psf Phi': 45 ° Piezometric Line: 1
Name: Fly Ash Unit Weight: 100 pcf Cohesion': 0 psf Phi': 28 ° Piezometric Line: 1
Name: Dike 3 Unit Weight: 120 pcf Cohesion': 1,200 psf Phi': 0 ° Piezometric Line: 1
Name: Weathered Rock Unit Weight: 130 pcf Cohesion': 0 psf Phi': 45 ° Piezometric Line: 1
Name: Bedrock Piezometric Line: 1
Name: Bottom Ash_R2 Unit Weight: 120 pcf Tau/Sigma Ratio: 0.28 Piezometric Line: 1
Name: Bottom Ash_R3 Unit Weight: 120 pcf Tau/Sigma Ratio: 0.75 Piezometric Line: 1

Figure A. 4: Liquefaction Factor of Safety by Post-Earthquake Slope Stability Analysis at Cross Section BAshP-BAshP'