Stantec

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October 3, 2012 let_006_175551015_rev_0

Mr. John C. Kammeyer, PE
Vice President
Tennessee Valley Authority
1101 Market Street, LP 5G
Chattanooga, Tennessee 37402

Re: Response to Recommendations
USEPA CCR Impoundment Assessment DRAFT Report
Johnsonville Fossil Plant (JOF)
New Johnsonville, Tennessee

Dear Mr. Kammeyer:

As requested, Stantec has reviewed the DRAFT report *Coal Combustion Residue Impoundment Dam Assessment Report, Johnsonville Fossil Plant, Tennessee Valley Authority, New Johnsonville, Tennessee, dated May 2012* prepared by Dewberry and Davis, LLC (Dewberry) for the United States Environmental Protection Agency (USEPA). The purpose of this letter is to address Dewberry's conclusions and recommendations pertaining to structural stability, hydrologic/hydraulic capacity, and technical documentation; and to provide additional supporting information relative to ongoing plant improvements, further analysis, and planned activities where applicable. Dewberry's recommendations and Stantec's corresponding responses are listed below. The recommendations and responses apply to Ash Disposal Area 2.

Dewberry Report Section 1.2.3 1): *Perform a quantitative liquefaction analysis of embankment sections overlying very loose ash.*

Stantec Response: Stantec performed a liquefaction potential assessment based on ground motion estimates for the 2,500-year earthquake scenarios, Standard Penetration Test borings, and corresponding laboratory test results. A description of the methodology and the results (ground response analysis and factor of safety against liquefaction versus elevation) are attached. Consistent with previously submitted seismic stability analyses, Section K was analyzed and the following materials are anticipated to undergo liquefaction for the 2,500-year earthquake: Ash (Saturated), Alluvial Sand and Gravel.

Dewberry Report Section 1.2.3 2): *If liquefaction is indicated by the analysis, perform a post-earthquake analysis using static slope stability analysis using reduced shear strengths.*

Stantec Response: Based on the results of the liquefaction potential assessment, residual strengths were assigned to the liquefied materials and post-earthquake static stability analysis was performed for Section K. A description of the methodology and the results (slope stability cross section, including table of material parameters) are attached. The results indicate that Section K has a factor of safety greater than or equal to the target threshold value of 1.0; thus, the slope is judged to remain stable and will not undergo significant liquefaction-induced deformations due to the 2,500-year earthquake.

Dewberry Report Section 1.2.3 3): *If it is determined that liquefaction will not occur, review/investigate the very soft to soft clayey soils in the lower part of the dike embankment and in the alluvial foundation beneath the embankment. Analyze soils deformation potential during the design earthquake (2,500-year event), and assess the impact of any such deformation on the stability of the embankment.*

Stantec Response: As noted above, Stantec's analysis indicates that liquefaction is anticipated for the 2,500-year earthquake and subsequent post-earthquake stability analysis produced acceptable results. Therefore, a deformation analysis is deemed not necessary.

Dewberry Report Section 1.2.5: *No significant problems were observed in the field assessment that would require special attention outside of routine maintenance. The minor issues observed, mostly small eroded areas or areas of poor grass growth, should be addressed by TVA's routine maintenance activities. However, it is recommended that the areas of the two small apparent seeps at either end of the gabion wall near the south end of the northeast dike be visually monitored in future inspections, to check for flowing seepage and movement of soil particles with any flowing seepage that may develop.*

Stantec Response: Erosion areas, poor grass growth, and other minor maintenance issues are addressed and will continue to be addressed by TVA's Routine Handling Operations and Maintenance (RHO&M) group. Also, with regard to the wet areas that were observed at the ends of the gabion wall along the northeast dike on the day of Dewberry's site visit, these were a result of wet ground conditions from the previous day's heavy rainfall event. Stantec and RHO&M have observed that these areas are normally dry.

Based on the above responses and additional analyses provided, it is Stantec's opinion that the final rating for Ash Disposal Area 2 can be upgraded to Satisfactory.

We appreciate the opportunity to provide these responses. If you have any questions or need additional information, please call.

Sincerely,

STANTEC CONSULTING SERVICES INC.

Stephen H. Bickel, PE
Senior Principal

Randy L. Roberts, PE
Principal

/db/cmw

Cc: Roberto L. Sanchez, PE
Michael S. Turnbow

Attachments

GENERAL METHODOLOGY
SEISMIC STABILITY ANALYSIS
TVA FOSSIL PLANTS

1. Seismic Hazards

1.1. Regional Seismic Sources

Seismicity in the TVA service area is attributed to the New Madrid fault and smaller, less concentrated crustal faults. Located in the western region, along the borders of Tennessee, Kentucky, Missouri, and Arkansas, the New Madrid source zone is capable of producing large magnitude earthquakes (M > 7). Events of this size would produce relatively long durations of strong ground shaking across the entire Tennessee River Valley. Fortunately, large magnitude New Madrid events are infrequent. Other source zones that may represent significant seismic risks for TVA facilities include those in eastern Tennessee, along the Wabash River Valley, and less significant sources throughout the region. While the maximum earthquake magnitudes associated with these other sources are smaller, compared to the New Madrid events, larger site accelerations can result from the closer proximity of TVA facilities.

These two earthquake scenarios generate significantly different seismic hazards at each locality and were considered independently in the analysis. To appropriately capture the influence of each, the assessments were completed independently for:

1. New Madrid events, and
2. events from "All Other Sources".

1.2. Site-Specific Hazards

Site-specific seismic hazards were characterized for the seismic stability assessments. AMEC Geomatrix, Inc. (Oakland, California) used the 2004 TVA "Valley-wide" seismic hazard model (Geomatrix 2004) to generate seismic inputs for each of TVA's fossil plants. Geomatrix documented their efforts in a report (AMEC Geomatrix Inc. 2011); excerpts are included herein.

The key data sets generated by Geomatrix and utilized by Stantec are:

1. Peak ground accelerations at top of hard rock (PGA_{rock}) for two different seismic sources (New Madrid Source and All Other Sources), for the 2,500-year return period, for each fossil plant location.
2. Seismic hazard deaggregation for PGA_{rock} for the 2,500-year return period. The hazards were deaggregated into appropriately sized bins of magnitude and epicentral distance.

1.3. PGA at Ground Surface

The peak horizontal accelerations obtained from the seismic hazard study represent accelerations at the top of hard bedrock (PGA_{rock}). For the assessment of liquefaction potential, the cyclic loads on natural soils and ash deposits were estimated using the simplified method described in Youd et al. (2001). This method requires estimates of the peak horizontal acceleration at the ground surface (PGA_{soil}).

Depending on the site and ground motion characteristics, peak accelerations may be amplified or attenuated (deamplified) as the energy propagates upward through the soil profile. Numerical ground response analyses can be used to model the propagation of ground motions and compute the cyclic stresses at various locations in the soil profile. One-dimensional, equivalent-linear elastic codes like ProShake can be used for this purpose if ground motion time histories are available.

To support sophisticated analyses at sites subject to higher seismic loads (i.e., large magnitudes and large accelerations), AMEC Geomatrix developed ground motion time histories for four TVA plants: Allen (ALF), Cumberland (CUF), Gallatin (GAF), and Shawnee (SHF). Relevant excerpts of the AMEC Geomatrix deliverable are provided herein. For these sites, Geocomp and Prof. Steve Kramer (University of Washington) performed ground response analyses using ProShake. These results, including profiles of acceleration and shear stress versus depth, were used for these four facilities. Compared to the more simplified method outlined below, the ProShake results allow for a more detailed representation of the ground response, particularly for facilities with extremely deep soils such as ALF and SHF.

Given the large portfolio of facilities that were considered, a simpler approach was used for the remaining facilities in this assessment. Developed for TVA by Dr. Gonzalo Castro and GEI Consultants, and implemented by Stantec in a spreadsheet, the method approximates what would be performed via one-dimensional, equivalent-linear elastic methods. For a representative soil profile, unit weights and groundwater conditions are applied to calculate total and effective stresses in the soil column. Soil stiffness (small-strain shear modulus or shear wave velocity), modulus reduction, and damping parameters are assigned based on estimated properties and published correlations. An iterative process is then used to estimate the PGA_{soil} at the top of ground, resulting from the PGA_{rock} for a given earthquake. The GEI method does not require a ground motion time history, but yields a result that appropriately considers the thickness and properties of the site-specific foundation soils. Instead of using acceleration time histories, this method utilizes response spectra for various levels of damping, which were generated by AMEC Geomatrix for use in these analyses. Relevant excerpts of the AMEC Geomatrix deliverable are provided herein. This method is more site-specific than using generic published correlations, and is judged to give reasonable results when compared to ProShake output.

2. Liquefaction Potential Assessment

2.1. Soil Loading from Earthquake Motions

The magnitude of the cyclic shear stresses induced by an earthquake is represented by the cyclic stress ratio (CSR). The simplified method proposed by Seed and Idriss (1971) and adopted by Youd et al. (2001) was used to estimate CSR. The cyclic stresses imparted to the soil were estimated from the earthquake parameters described above, representing earthquakes on the New Madrid fault and local crustal events.

2.2. Soil Resistance from Correlations with Penetration Resistance

The resistance to soil liquefaction, expressed in terms of the cyclic resistance ratio (CRR), was assessed using the empirical NCEER methodology (Youd et al. 2001). Updates to the procedure from recently published research were used where warranted. The analyses were based on the blowcount value (N) measured in the Standard Penetration Test (SPT) or the tip resistance (q_c) measured in the Cone Penetration Test (CPT).

The NCEER procedure involves a number of correction factors. Based on the site-specific conditions and soil characteristics, engineering judgment was used to select appropriate correction factors consistent with the consensus recommendations of the NCEER panel (Youd et al. 2001). To avoid inappropriately inflating the CRR, the NCEER fines content adjustment was not applied where zero blowcounts are recorded. The magnitude scaling factor (MSF) is used in the procedure to normalize the representative earthquake magnitude to a baseline 7.5M earthquake. The earthquake magnitude (M) most representative of the liquefaction risk was determined by applying the MSF to the de-aggregation data for the 2,500-year earthquakes (New Madrid and All Other Sources).

2.3. Factor of Safety Against Liquefaction

The factor of safety against liquefaction (FS_{liq}) is defined as the ratio of the liquefaction resistance (CRR) over the earthquake load (CSR). Following TVA design guidance and the precedent set by Seed and Harder (1990), FS_{liq} is interpreted as follows:

- Soil will liquefy where $FS_{liq} \leq 1.1$.
- Expect substantial soil softening where $1.1 < FS_{liq} \leq 1.4$.
- Soil does not liquefy where $FS_{liq} > 1.4$.

Using these criteria for guidance, values of FS_{liq} computed throughout a soil deposit or cross section (at specific CPT-q_c and SPT-N locations) were reviewed in aggregate. Occasional pockets of liquefied material in isolated locations are unlikely to induce a larger failure, and are typically considered tolerable. Instead, problems associated with soil liquefaction are indicated where continuous zones of significant lateral extent exhibit low values of FS_{liq}. Engineering judgment, including consideration for the likely performance in critical areas, was used in the overall assessment for each facility.

3. Post-Earthquake Slope Stability

3.1. Characterize Post-Earthquake Soil Strengths

The post-earthquake shearing resistance of each soil and coal combustion product (CCP) was estimated with consideration for the specific characteristics of that material. Specifically:

- Full static, undrained strength parameters were assigned to unsaturated soils, where significant excess pore pressures are not anticipated to develop under seismic loading.
- In saturated clays and soils with $FS_{liq} > 1.4$, 80% of the static undrained strength was assumed. These reduced strengths account for the softening effects of pore pressure buildup during an earthquake.
- In saturated, low-plasticity, granular soils with $1.1 < FS_{liq} \leq 1.4$, a reduced strength was assigned, based on the excess pore pressure ratio, r_u (Seed and Harder 1990). Typical relationships between FS_{liq} and r_u have been published by Marcuson and Hynes (1989).
- In saturated, low-plasticity, granular soils with $FS_{liq} \leq 1.1$, a residual (steady state) strength (S_r) was estimated for the liquefied soil.

Estimates of S_r can be obtained from empirical correlations published by various researchers. Typically, residual strength (or the ratio of residual strength over vertical effective stress) is correlated to corrected SPT blowcounts or corrected CPT tip resistance, based on back analysis of liquefaction case histories. For this evaluation, a new "hybrid" model developed by Kramer and Wang (in press) was used. Their hybrid model expresses mean residual strength as a function of both corrected SPT blowcounts and vertical effective stress:

$$\overline{\ln(S_r)} = -8.444 + 0.109(N_1)_{60} + 5.379(\sigma'_{vo})^{0.1}$$

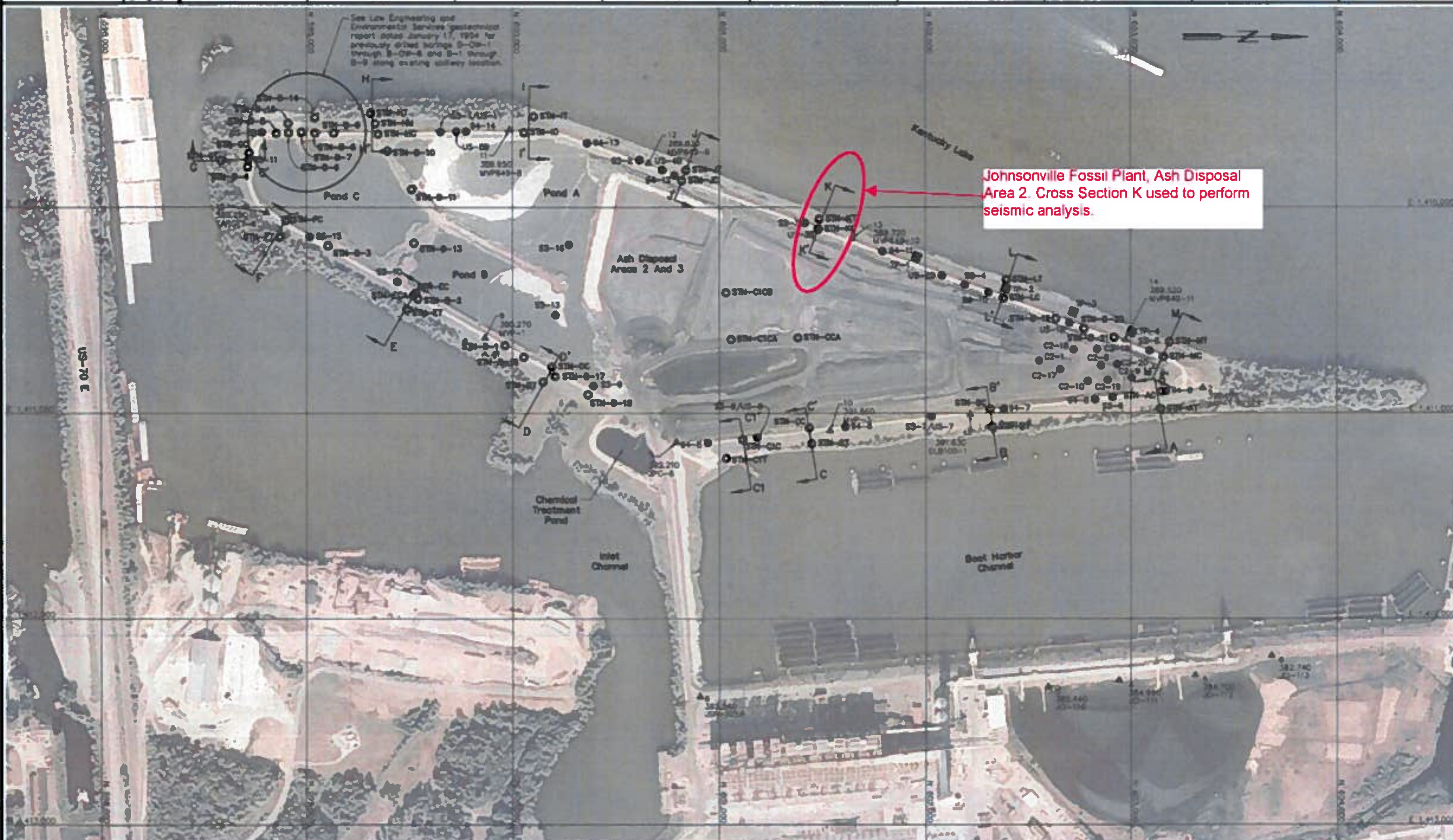
Where S_r = residual strength in atmospheres, $(N_1)_{60}$ = normalized and corrected SPT N-value, and σ_{vo}' = initial vertical effective stress in atmospheres. A representative value of $(N_1)_{60}$ was selected for each liquefiable soil layer from a detailed review of the boring logs. SPT blowcounts judged to be erroneous or nonrepresentative of the in situ conditions were discarded. For example, excessively high blowcounts resulting from the SPT sampler hitting a cobble or boulder and excessively low blowcounts associated with borehole heave were discarded. The remaining blowcounts (in terms of $(N_1)_{60}$) were then averaged to arrive at the representative value.

3.2. Analyze Slope Stability

The next step in the evaluation considered slope stability for post-earthquake conditions, including liquefied strengths where appropriate. Slope stability was evaluated using two-dimensional, limit equilibrium, slope stability methods and reduced soil strengths (from above), representing the loss of shearing resistance due to cyclic pore pressure generation during the earthquake. The analyses were accomplished using Spencer's method of analysis, as implemented in the SLOPE/W software, considering both circular and translational slip mechanisms. The analyses represent current operating conditions (geometry and phreatic levels).

If extensive liquefaction is indicated, stability was evaluated for the static conditions immediately following the cessation of the earthquake motions. Residual or steady state strengths were assigned in zones of liquefied soil, with reduced strengths that account for cyclic softening and pore pressure build up assumed in unliquefied soil. Failure (large, unacceptable displacements) is indicated if the safety factor (FS_{slope}) computed in this step is less than one. Slopes exhibiting $FS_{slope} \geq 1$ with liquefaction are assumed stable with tolerable deformations.

Within SLOPE/W, the residual strength model described previously was implemented with a cohesion (equal to S_r) that varies spatially. Based on the representative $(N_1)_{60}$ value and the initial vertical effective stress, S_r was calculated and assigned at key locations within the liquefied soil layer. The strength at any other point in the deposit was interpolated in SLOPE/W, thereby recognizing the increasing strength at higher vertical effective stress.



BORING LOCATION TABLE

BORING	NORTHING	EASTING	ELEV. (FT.)
[illegible]	[illegible]	[illegible]	[illegible]

*Borings not surveyed and elevations are approximate.

INSPECTION TEST PIT LOCATION TABLE

BORING	NORTHING	EASTING	ELEV. (FT.)
TP-1	[illegible]	[illegible]	–
TP-2	[illegible]	[illegible]	–
TP-3	[illegible]	[illegible]	–
TP-4	[illegible]	[illegible]	–

*Test Pit Locations are approximate.

FOR INFORMATION ONLY
This Record Drawing which has been previously submitted to TVA is provided for Information Only.

RECORD DRAWING

BORINGS	REFERENCE
B-1 through B-9 and B-OW-1 through B-OW-6	Report of Geotechnical Exploration, Ash Pond Dike, New Johnsonville Plant, New Johnsonville, Tennessee, Law Engineering and Environmental Services, Inc., January 17, 1994.
94-5 through 94-14	Subsurface Exploration Data, TVA Borings at Johnsonville Fossil Plant, Johnsonville, Tennessee, Law Engineering and Environmental Services, Inc., October 11, 1994
US-1B through US-6B	Johnsonville Steam Plant – Ash Pond – Soil and Foundation Exploration, J.C. McCrew, TVA Construction Services Branch, September 17, 1969.
SS-13, SS-15, SS-16	Johnsonville Groundwater Assessment, TVA Resource Group, Engineering Services, March 1993.
SS-1 through SS-11 and US-1, US-7 and US-8	Johnsonville Steam Plant – Ash Disposal Area No. 2 Dike Raising – Soil Exploration and Testing, G. Farmer, TVA Construction Services Branch, November 22, 1977.
C2-1 through C2-20	Report of Ash Pond Investigation, Johnsonville Fossil Plant, New Johnsonville, Tennessee, MACTEC Engineering and Consulting, December 4, 2003.

LEGEND

- ○ Soil Boring with Undisturbed (Shelby) Tube Samples and/or Standard Penetration Tests
- ● Soil Boring from previous explorations by Others
- ■ Excavated Inspection Test Pit
- ▲ TVA Survey Monument

NOTES:

1. Horizontal and vertical locations of 2009 borings provided by T.V.A. Location of previously drilled borings approximated from boring layouts provided in previous reports.
2. The geotechnical information and data furnished herein are not intended as representation or warranties but are furnished for information only. It shall be distinctly understood that the Owner or Engineer will not be responsible for any deduction, interpretation or conclusion drawn therefrom. The information is made available in order that the Contractor may have ready access to the same information available to the Owner and the Engineer and is not part of this contract.

GRAPHIC SCALE

For Supporting Design Calculations see

SCALE: 1"=200' EXCEPT AS NOTED

YARD
ASH DISPOSAL AREAS 2 AND 3

GEOTECHNICAL EXPLORATION
BORING LAYOUT

Stantec

JOHNSONVILLE FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY

AUTOCAD R 2000 | 30 | C | XXWXXX–01 | R 0

PLOT FACTOR:XX

STANTEC | 0
TASK COMPLETED BY | REV NO.

Section K - Ash Disposal Area 2
Johnsonville Fossil Plant
New Johnsonville, Tennessee

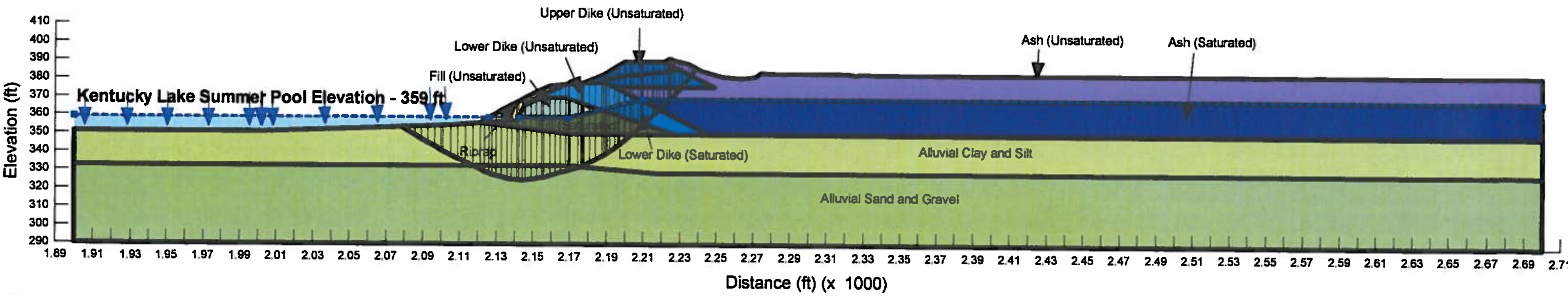
Existing Conditions - Post Earthquake

Liquefied Materials: Ash, Alluvial Sand and Gravel

Material Type	Unit Weight	Cohesion	Friction Angle
Alluvial Clay and Silt	124 pcf	571.2 psf	14.4 °
Alluvial Sand and Gravel	120 pcf	Sr=exp(-8.444+0.109N1(60)+5.379σ'^0.1), N1(60)=21	0 °
Fill (Unsaturated)	124 pcf	630 psf	17.8 °
Upper Dike (Saturated)	125 pcf	417 psf	13 °
Lower Dike (Unsaturated)	125 pcf	533 psf	20.1 °
Ash (Unsaturated)	100 pcf	0 psf	10 °
Upper Dike (Unsaturated)	125 pcf	521 psf	16.2 °
Lower Dike (Saturated)	125 pcf	426.4 psf	16.3 °
Ash (Saturated)	100 pcf	Sr=exp(-8.444+0.109N1(60)+5.379σ'^0.1), N1(60)=1	0 °
Fill (Saturated)	124 pcf	504 psf	14.4 °

Note:
The results of analysis shown here are based on available subsurface information, laboratory test results and approximate soil properties. No warranties can be made regarding the continuity of subsurface conditions between the borings.

Factor of Safety: 1.5



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Project No. 175551015

Section K - Ash Disposal Area 2
Johnsonville Fossil Plant
New Johnsonville, Tennessee

Existing Conditions - Post Earthquake

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Material Type	Unit Weight	Cohesion	Friction Angle
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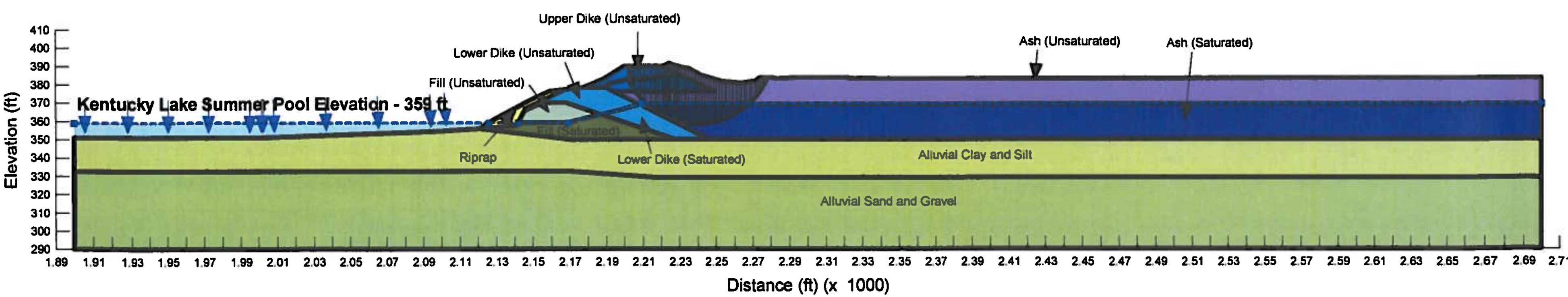
Lower Dike (Saturated)	125 pcf	426.4 psf	16.3 °
Ash (Saturated)	100 pcf	Sr=exp(-8.444+0.109N1(60)+5.379σ'^0.1), N1(60)=1	0 °
Fill (Saturated)	124 pcf	504 psf	14.4 °

Note:
The results of analysis shown here are based on available subsurface information, laboratory test results and approximate soil properties. No warranties can be made regarding the continuity of subsurface conditions between the borings.

Factor of Safety: 1.0



JOF_Section K_DS_All_Updated_20120927.gsz

Project No. 175551015

Top of Hard Rock Accelerations (from AMEC Geomatrix 2011a)

TABLE 8

HAZARD RESULTS FOR THE JOHNSONVILLE PLANT

Seismic Sources	Return Period (years)[1]	Annual Probability of Exceedance	PGA[1] (g)	$S_a(0.2)$[2] (g)	$S_a(0.4)$ (g)	$S_a(1.0)$ (g)	$S_a(2.0)$ (g)	$S_a(4.0)$ (g)
New Madrid Seismic Zone	2,□□□	□.□□□□	□2□2□	□.□□□□	□.2□□□	□.□□2□	□.□□□2	□.□□□□
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	□□□	□.□□	□	□	□	□	□	□
All Other Seismic Sources	2,□□□	□.□□□□	□.□□□□	□.□□□□	□.□□□□	□.□□□□	□.□□□□	□.□□□□
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Notes

□. Pea□ground acceleration

2. S_a□□.2□refers to the □□ damped spectral acceleration at a spectral period of □.2 seconds □spectral fre□uency of □cycles□sec□

Response Spectra Used in Ground Response Analysis (from AMEC Geomatrix 2011b)
Hazard Results for the Johnsonville Fossil Plant

Seismic Sources	Return Period (years)[1]	Annual Probability of Exceedance	PGA[1] (g)	$S_a(0.2)$[2] (g)	$S_a(0.4)$ (g)	$S_a(1.0)$ (g)	$S_a(2.0)$ (g)	$S_a(4.0)$ (g)
New Madrid Seismic Zone	2,500	0.0004	0.2324	0.6180	0.4503	0.1952	0.1211	0.0557
	1,500	0.00067	0.1734	0.4813	0.3361	0.1505	0.0838	0.0361
	1,000	0.001	0.1289	0.3631	0.2428	0.1066	0.0585	0.0221
	500	0.002	0.0522	0.1328	0.0894	0.0377	0.0156	0.0051
	250	0.004	0	0.0000	0.0000	0.0000	0.0000	0.0000
	100	0.01	0	0.0000	0.0000	0.0000	0.0000	0.0000
All Other Seismic Sources	2,500	0.0004	0.1117	0.2714	0.1758	0.0764	0.0449	0.0185
	1,500	0.00067	0.0854	0.2065	0.1313	0.0554	0.0308	0.0129
	1,000	0.001	0.068	0.1660	0.1025	0.0424	0.0228	0.0091
	500	0.002	0.0456	0.1091	0.0662	0.0254	0.0134	0.0050
	250	0.004	0.0291	0.0703	0.0409	0.0152	0.0073	0.0026
	100	0.01	0.0142	0.0344	0.0200	0.0067	0.0030	0.0010

Notes

1. Peak ground acceleration.
2. $S_a(0.2)$ refers to the 1% damped spectral acceleration at a spectral period of 0.2 seconds (spectral frequency of 5 cycles/sec).

Response Spectra Used in Ground Response Analysis (from AMEC Geomatrix 2011b)
Hazard Results for the Johnsonville Fossil Plant

Seismic Sources	Return Period (years)[1]	Annual Probability of Exceedance	PGA[1] (g)	$S_a(0.2)$[2] (g)	$S_a(0.4)$ (g)	$S_a(1.0)$ (g)	$S_a(2.0)$ (g)	$S_a(4.0)$ (g)
New Madrid Seismic Zone	2,500	0.0004	0.2324	0.4497	0.3403	0.1530	0.0976	0.0459
	1,500	0.00067	0.1734	0.3502	0.2539	0.1180	0.0676	0.0298
	1,000	0.001	0.1289	0.2642	0.1834	0.0836	0.0472	0.0183
	500	0.002	0.0522	0.0966	0.0676	0.0296	0.0126	0.0042
	250	0.004	0	0.0000	0.0000	0.0000	0.0000	0.0000
	100	0.01	0	0.0000	0.0000	0.0000	0.0000	0.0000
All Other Seismic Sources	2,500	0.0004	0.1117	0.1975	0.1328	0.0600	0.0364	0.0155
	1,500	0.00067	0.0854	0.1502	0.0992	0.0435	0.0251	0.0108
	1,000	0.001	0.068	0.1208	0.0774	0.0334	0.0186	0.0077
	500	0.002	0.0456	0.0794	0.0500	0.0200	0.0109	0.0043
	250	0.004	0.0291	0.0511	0.0309	0.0120	0.0060	0.0022
	100	0.01	0.0142	0.0250	0.0151	0.0053	0.0025	0.0009

Notes
1. Peak ground acceleration.
2. $S_a(0.2)$ refers to the 3% damped spectral acceleration at a spectral period of 0.2 seconds (spectral frequency of 5 cycles/sec).

Response Spectra Used in Ground Response Analysis (from AMEC Geomatrix 2011b)
Hazard Results for the Johnsonville Fossil Plant

Seismic Sources	Return Period (years)[1]	Annual Probability of Exceedance	PGA[1] (g)	$S_a(0.2)$[2] (g)	$S_a(0.4)$ (g)	$S_a(1.0)$ (g)	$S_a(2.0)$ (g)	$S_a(4.0)$ (g)
New Madrid Seismic Zone	2,500	0.0004	0.2324	0.3789	0.2905	0.1323	0.0852	0.0404
	1,500	0.00067	0.1734	0.2951	0.2168	0.102	0.059	0.0262
	1,000	0.001	0.1289	0.2226	0.1566	0.0723	0.0412	0.0161
	500	0.002	0.0522	0.0814	0.0577	0.0256	0.011	0.0037
	250	0.004	0	0	0	0	0	0
	100	0.01	0	0	0	0	0	0
All Other Seismic Sources	2,500	0.0004	0.1117	0.1664	0.1134	0.0519	0.0319	0.0137
	1,500	0.00067	0.0854	0.1266	0.0847	0.0377	0.022	0.0096
	1,000	0.001	0.068	0.1018	0.0661	0.0289	0.0163	0.0068
	500	0.002	0.0456	0.0669	0.0427	0.0173	0.0096	0.0038
	250	0.004	0.0291	0.0431	0.0264	0.0104	0.0053	0.002
	100	0.01	0.0142	0.0211	0.0129	0.0046	0.0022	0.0008

Notes

1. Peak ground acceleration.
2. $S_a(0.2)$ refers to the 5% damped spectral acceleration at a spectral period of 0.2 seconds (spectral frequency of 5 cycles/sec).

Response Spectra Used in Ground Response Analysis (from AMEC Geomatrix 2011b)
Hazard Results for the Johnsonville Fossil Plant

Seismic Sources	Return Period (years)[1]	Annual Probability of Exceedance	PGA[1] (g)	$S_a(0.2)$[2] (g)	$S_a(0.4)$ (g)	$S_a(1.0)$ (g)	$S_a(2.0)$ (g)	$S_a(4.0)$ (g)
New Madrid Seismic Zone	2,500	0.0004	0.2324	0.3361	0.2592	0.1188	0.0766	0.0364
	1,500	0.00067	0.1734	0.2618	0.1934	0.0916	0.0531	0.0236
	1,000	0.001	0.1289	0.1974	0.1397	0.0649	0.0371	0.0145
	500	0.002	0.0522	0.0722	0.0515	0.0230	0.0099	0.0033
	250	0.004	0	0.0000	0.0000	0.0000	0.0000	0.0000
	100	0.01	0	0.0000	0.0000	0.0000	0.0000	0.0000
All Other Seismic Sources	2,500	0.0004	0.1117	0.1476	0.1012	0.0466	0.0288	0.0124
	1,500	0.00067	0.0854	0.1123	0.0756	0.0339	0.0199	0.0087
	1,000	0.001	0.068	0.0903	0.0590	0.0260	0.0148	0.0062
	500	0.002	0.0456	0.0593	0.0381	0.0156	0.0087	0.0035
	250	0.004	0.0291	0.0382	0.0236	0.0094	0.0048	0.0018
	100	0.01	0.0142	0.0187	0.0115	0.0041	0.0020	0.0007

Notes

1. Peak ground acceleration.
2. $S_a(0.2)$ refers to the 7% damped spectral acceleration at a spectral period of 0.2 seconds (spectral frequency of 5 cycles/sec).

Response Spectra Used in Ground Response Analysis (from AMEC Geomatrix 2011b)
Hazard Results for the Johnsonville Fossil Plant

Seismic Sources	Return Period (years)[1]	Annual Probability of Exceedance	PGA[1] (g)	$S_a(0.2)$[2] (g)	$S_a(0.4)$ (g)	$S_a(1.0)$ (g)	$S_a(2.0)$ (g)	$S_a(4.0)$ (g)
New Madrid Seismic Zone	2,500	0.0004	0.2324	0.2945	0.2277	0.1048	0.0679	0.0323
	1,500	0.00067	0.1734	0.2294	0.1700	0.0808	0.0471	0.0210
	1,000	0.001	0.1289	0.1730	0.1228	0.0573	0.0329	0.0129
	500	0.002	0.0522	0.0633	0.0452	0.0203	0.0088	0.0030
	250	0.004	0	0.0000	0.0000	0.0000	0.0000	0.0000
	100	0.01	0	0.0000	0.0000	0.0000	0.0000	0.0000
All Other Seismic Sources	2,500	0.0004	0.1117	0.1293	0.0889	0.0412	0.0256	0.0111
	1,500	0.00067	0.0854	0.0984	0.0664	0.0299	0.0177	0.0078
	1,000	0.001	0.068	0.0791	0.0518	0.0230	0.0131	0.0055
	500	0.002	0.0456	0.0520	0.0335	0.0138	0.0078	0.0031
	250	0.004	0.0291	0.0335	0.0207	0.0083	0.0043	0.0017
	100	0.01	0.0142	0.0164	0.0101	0.0037	0.0018	0.0007

Notes

1. Peak ground acceleration.
2. $S_a(0.2)$ refers to the 10% damped spectral acceleration at a spectral period of 0.2 seconds (spectral frequency of 5 cycles/sec).

Seismic Risk Assessment

Plant:	Johnsonville Fossil Plant
Facility:	Ash Disposal 2
Section:	K
Seismic Zone:	All Other Zones
# of Layers	10
Total Thickness	88.1 feet

	User Input
	Drop-down selection
	Default value, user can modify
	Calculated value
	Calculated value, unoptimized

Global Inputs:

PGA_{SOIL}	0.1012	
Groundwater Elevation (Z_{GW})	360.8	feet
Additional Vert. Stress	0	psf
Pa	2116.8	psf
k	0	
Ko	0.5	
g	32.2	ft/s2
Yw	62.4	pcf
$G/G_{MAX,TOL}$	0.20%	
$G/G_{MAX,ACTUAL}$	0.18%	

(19) Composite Shear Wave Velocity	(20) Natural Period	(22) Composite Damping Ratio	Interpolated Return Period
$\bar{V}_S$	T	ξ %	(years)
622.0	0.5666	4.388	2495.9

Calculation Checks:

PGA_{SOIL} ---> ~2500 Year Return Period	OK
$G/G_{MAX,ACTUAL}$ Ratio	OK

Layer	Material	Elevations Z_{TOP} (feet)	Z_{BOTTOM} (feet)	Z_{MID} (feet)	Overburden (feet)	Specific Gravity G_S	Moist Unit Weight γ_{DRY} (pcf)	Saturated Unit Weight γ_{SAT} (pcf)	Over-consolidation Ratio OCR	Plasticity Index PI
1	Clay	378.1	366.97	372.5	5.6	2.7	125	125	1	0
2	Clay	366.97	360.8	363.9	14.2	2.7	124	124	1	0
3	Clay	360.8	354.14	357.5	20.6	2.7	124	124	1	0
4	Clay	354.14	344.14	349.1	29.0	2.7	124	124	1	0
5	Clay	344.14	334.14	339.1	39.0	2.7	124	124	1	0
6	Clay	334.14	331.75	332.9	45.2	2.7	124	124	1	0
7	Sand	331.75	321.75	326.8	51.4	2.65	120	120	1	0
8	Sand	321.75	311.75	316.8	61.4	2.65	120	120	1	0
9	Sand	311.75	300	305.9	72.2	2.65	120	120	1	0
10	Sand	300	290	295.0	83.1	2.65	120	120	1	0
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Seismic Risk Assessment

Plant:	Johnsonville Fossil Plant		User Input
Facility:	Ash Disposal 2		Drop-down selection
Section:	K		Default value, user can modify
Seismic Zone:	New Madrid		Calculated value
# of Layers	10		Calculated value, unoptimized
Total Thickness	88.1	feet	

Global Inputs:

PGA_{SOIL}	0.1977	
Groundwater Elevation (Z_{GW})	360.8	feet
Additional Vert. Stress	0	psf
Pa	2116.8	psf
k	0	
Ko	0.5	
g	32.2	ft/s2
Yw	62.4	pcf
$G/G_{MAX,TOL}$	0.20%	
$G/G_{MAX,ACTUAL}$	0.18%	

Calculation Checks:

PGA_{SOIL} —> ~2500 Year Return Period	OK
$G/G_{MAX,ACTUAL}$ Ratio	OK

(19) Composite Shear Wave Velocity	(20) Natural Period	(22) Composite Damping Ratio	Interpolated Return Period
$\bar{V}_S$	T	$\bar{\xi}$ %	(years)
536.9	0.6563	7.169	2496.8

Layer	Material	Elevations Z_{TOP} (feet)	Z_{BOTTOM} (feet)	Z_{MID} (feet)	Overburden (feet)	Specific Gravity G_S	Moist Unit Weight Y_{DRY} (pcf)	Saturated Unit Weight Y_{SAT} (pcf)	Over-consolidation Ratio OCR	Plasticity Index PI
1	Clay	378.1	366.97	372.5	5.6	2.7	125	125	1	0
2	Clay	366.97	360.8	363.9	14.2	2.7	124	124	1	0
3	Clay	360.8	354.14	357.5	20.6	2.7	124	124	1	0
4	Clay	354.14	344.14	349.1	29.0	2.7	124	124	1	0
5	Clay	344.14	334.14	339.1	39.0	2.7	124	124	1	0
6	Clay	334.14	331.75	332.9	45.2	2.7	124	124	1	0
7	Sand	331.75	321.75	326.8	51.4	2.65	120	120	1	0
8	Sand	321.75	311.75	316.8	61.4	2.65	120	120	1	0
9	Sand	311.75	300	305.9	72.2	2.65	120	120	1	0
10	Sand	300	290	295.0	83.1	2.65	120	120	1	0
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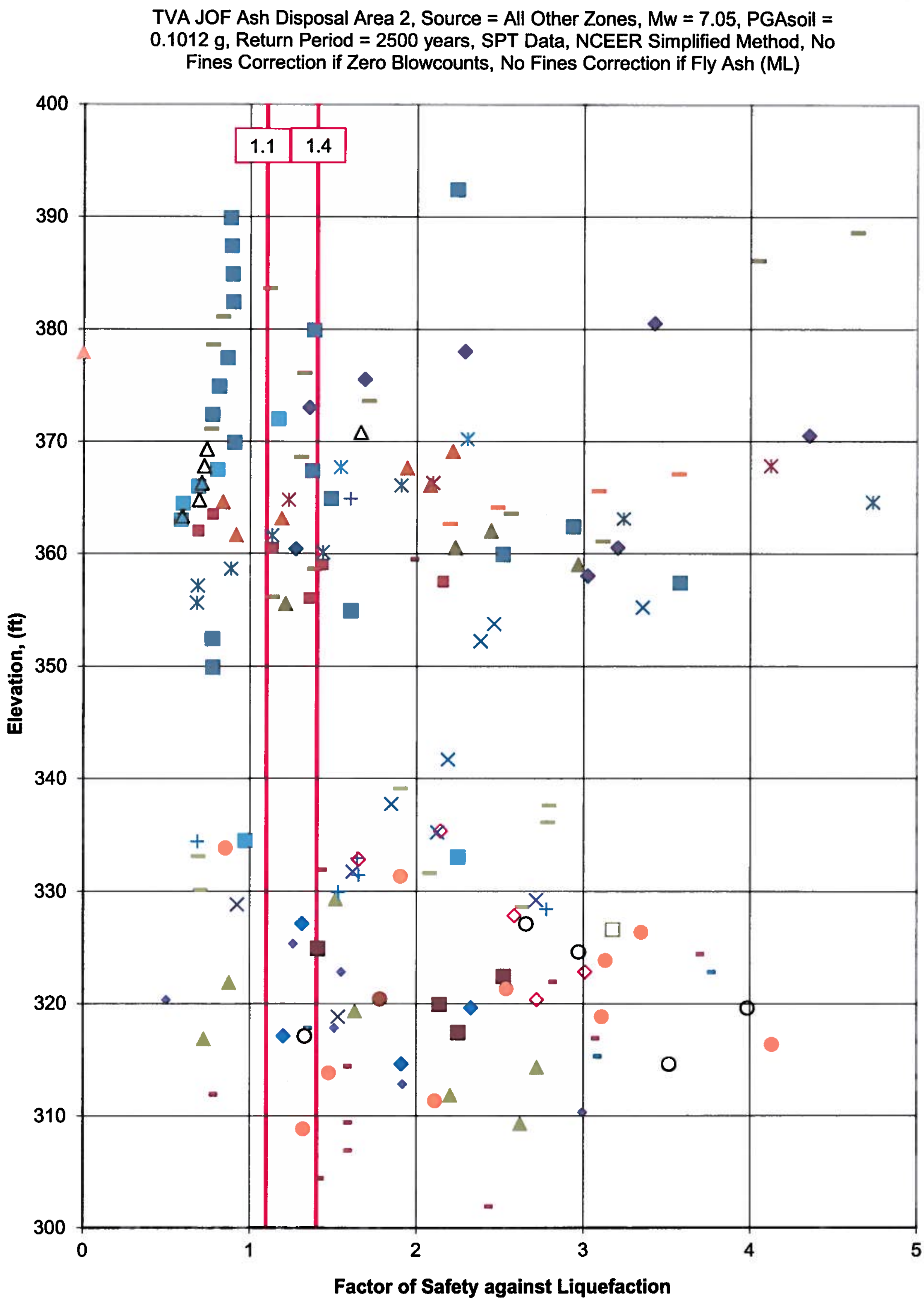
TVA JOF Ash Disposal Area 2, Source = All Other Zones, Mw = 7.05, PGAsoil = 0.1012 g, Return Period = 2500 years, SPT Data, NCEER Simplified Method, No Fines Correction if Zero Blowcounts, No Fines Correction if Fly Ash (ML)
Elevation, (ft)
Factor of Safety against Liquefaction
1.1
1.4
400
390
380
370
360
350
340
330
320
310
300
0
1
2
3
4
5

TVA JOF Ash Disposal Area 2, Source = New Madrid, Mw = 7.67, PGAsoil = 0.1977 g, Return Period = 2500 years, SPT Data, NCEER Simplified Method, No Fines Correction if Zero Blowcounts, No Fines Correction if Fly Ash (ML)