EPA Comments

SUBJECT: Comments on "DRAFT REPORT - TVA Shawnee Fossil Power Plant"

DATE: September 3, 2012

COMMENTS:

1. Include the results of the seismic stability analysis in the appendix as it is the primary reason why Ash Pond 2 is given a poor condition rating.
2. On page 11, section 3.3, first two sentences, replace "inspection" with "assessment" in each sentence.
3. On page 11, section 3.3, first sentence of second paragraph, add "2" after "Ash Pond No."
4. On page 14, section 5, Conclusions, the rationale for the rating should also include the statement found on page 9, section 3.1.2: "No liquefaction potential evaluations have been performed to date for the Ash Pond 2 dikes, but may be warranted given that a portion of the upper dike is founded on sluiced ash (as described below), which is susceptible to liquefaction."

Stantec

Stantec Consulting Services Inc.
1901 Nelson Miller Parkway
Louisville KY 40223-2177
Tel: (502) 212-5000
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October 3, 2012

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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
Shawnee Fossil Plant (SHF)
Paducah, Kentucky

Dear Mr. Kammeyer:

As requested, Stantec has reviewed the report *Dam Safety Assessment of CCW Impoundments, TVA Shawnee Fossil Power Plant* dated August 17, 2012 prepared by O'Brien & Gere for the United States Environmental Protection Agency (USEPA). The purpose of this letter is to address O'Brien & Gere's conclusions and recommendations pertaining to structural stability, hydrologic/hydraulic (H&H) capacity, and technical documentation; and to provide additional supporting information relative to ongoing plant improvements, further analysis, and planned activities where applicable. O'Brien & Gere's recommendations and Stantec's corresponding responses are listed below. The recommendations and responses apply to Ash Pond No. 2.

O'Brien and Gere Report Section 6: *Based on the findings of our visual inspection and review of the available records for the Ash Pond No. 2, O'Brien & Gere recommends that the required seismic slope stability analysis be performed for critical dike slopes. The seismic loading induced by the MCE (2% probability of exceedence in 50 years) should be applied in the analysis. In addition, the seismic analysis should include an evaluation of liquefaction potential, considering that the inboard portion of the last 10 foot vertical dike raising was founded on sluiced ash material, which can be subject to liquefaction. These analyses should be completed within one year from the date of this report.*

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 N was analyzed and the results indicate that the sluiced ash and underlying sand materials are anticipated to undergo liquefaction for the 2,500-year earthquake.

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 N. A description of the methodology and the results (slope stability cross section, including table of material parameters) are attached. The results indicate that Section N 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.

Because the post-earthquake stability analysis (with liquefied/reduced strengths) produced acceptable results, no further seismic analysis is deemed necessary.

Based on the above responses and additional analyses provided, it is Stantec's opinion that the final rating for Ash Pond No. 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

Stephen H. Bickel, PE
Senior Principal

Randy L. Roberts

Randy L. Roberts, PE
Principal

/db

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:

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

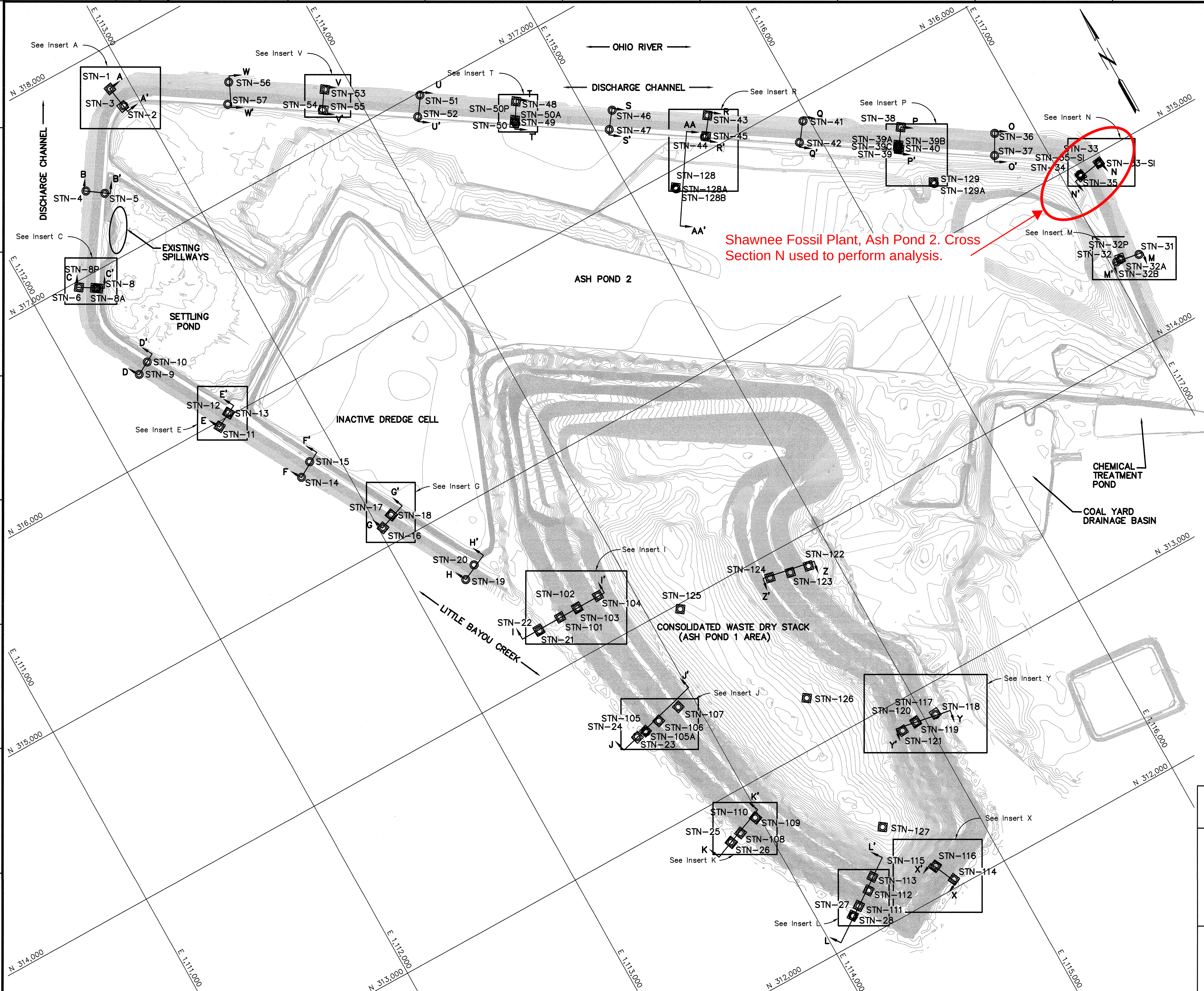
Where S_r = residual strength in atmospheres, $(N_1)_{60}$ = normalized and corrected SPT N-value, and σ_{vo}' = initial vertical effective stress in atmospheres. A representative value of $(N_1)_{60}$ was selected for each liquefiable soil layer from a detailed review of the boring logs. SPT blowcounts 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

NO.	NORTHING	EASTING	ELEV. (FT.)
STN-1	317,797.70	1,112,894.71	326.9
STN-2	317,685.89	1,112,914.65	351.1
STN-3	317,685.06	1,112,908.87	351.2
STN-4	317,392.53	1,112,524.92	328.1
STN-5	317,333.36	1,112,605.34	351.6
STN-6	316,969.98	1,112,248.35	328.6
STN-8	316,915.52	1,112,335.31	351.9
STN-8A	316,921.21	1,112,325.90	351.9
STN-8P	316,924.83	1,112,319.91	351.9
STN-9	316,418.46	1,112,301.70	328.6
STN-10	316,454.21	1,112,370.24	350.7
STN-11	315,974.74	1,112,535.84	327.3
STN-12	316,016.50	1,112,610.36	351.0
STN-13	316,012.54	1,112,611.83	350.8
STN-14	315,536.24	1,112,778.56	327.5
STN-15	315,586.82	1,112,855.70	350.5
STN-16	315,098.19	1,113,021.92	328.5
STN-17	315,137.42	1,113,091.63	350.3
STN-18	315,133.53	1,113,093.60	350.4
STN-19	314,652.04	1,113,265.95	328.5
STN-20	314,698.36	1,113,341.04	350.1
STN-21	314,231.64	1,113,471.65	349.7
STN-22	314,238.31	1,113,470.52	349.7
STN-23	313,495.53	1,113,645.65	349.0
STN-24	313,499.27	1,113,645.11	349.0
STN-25	312,778.59	1,113,804.66	349.9
STN-26	312,774.78	1,113,806.04	349.9
STN-27	312,133.86	1,114,170.06	349.4
STN-28	312,131.19	1,114,171.87	349.4
STN-31	314,432.30	1,117,155.39	327.0
STN-32	314,454.28	1,117,034.48	350.6
STN-32A	314,457.23	1,117,058.23	350.6
STN-32B	314,455.26	1,117,057.91	350.6
STN-32P	314,461.18	1,117,058.88	350.6
STN-33	314,954.00	1,117,204.67	327.6
STN-33-SI	314,949.58	1,117,204.80	327.4
STN-34	314,943.07	1,117,089.94	350.5
STN-35	314,938.34	1,117,090.74	350.7
STN-35-SI	314,947.72	1,117,089.80	350.2
STN-36	315,351.57	1,116,807.82	324.6
STN-37	315,250.60	1,116,750.94	351.3
STN-38	315,618.68	1,116,395.32	322.9
STN-39	315,524.60	1,116,324.97	350.5
STN-39A	315,541.32	1,116,337.61	350.5
STN-39B	315,538.57	1,116,334.63	350.5
STN-39C	315,533.05	1,116,333.74	350.5
STN-40	315,521.49	1,116,330.97	350.7
STN-41	315,894.30	1,115,966.79	326.6
STN-42	315,805.38	1,115,896.21	350.1
STN-43	316,162.64	1,115,545.71	327.4
STN-44	316,072.06	1,115,478.40	350.4
STN-45	316,070.97	1,115,485.44	350.8
STN-46	316,427.81	1,115,126.31	328.2
STN-47	316,346.29	1,115,063.75	350.5
STN-48	316,712.60	1,114,710.52	326.6
STN-49	316,611.94	1,114,653.63	350.5
STN-50	316,615.65	1,114,648.22	350.6
STN-50A	316,622.47	1,114,652.40	350.6
STN-50P	316,630.15	1,114,657.10	350.6
STN-51	316,984.36	1,114,289.89	325.7
STN-52	316,890.61	1,114,224.53	350.4
STN-53	317,251.66	1,113,870.22	326.0
STN-54	317,164.29	1,113,807.60	350.6
STN-55	317,161.11	1,113,811.71	350.2
STN-56	317,528.33	1,113,453.65	325.1
STN-57	317,429.25	1,113,392.39	350.4
STN-101	314,239.59	1,113,599.85	345.6
STN-102	314,242.14	1,113,701.41	375.9
STN-103	314,237.20	1,113,701.82	375.9
STN-104	314,242.09	1,113,825.68	411.3
STN-105	313,503.36	1,113,698.42	347.7

Northing, Easting and Ground Surface Elevations were provided by TVA, Power Systems Operations, Surveying and Project Services.
Horizontal Datum: NAD 27 (Kentucky Lambert South).
Vertical Datum: NGVD29.

BORING LOCATION TABLE

NO.	NORTHING	EASTING	ELEV. (FT.)
STN-105A	313,498.36	1,113,698.42	347.7
STN-106	313,516.57	1,113,784.54	375.3
STN-107	313,531.32	1,113,908.57	408.1
STN-108	312,796.11	1,113,871.67	350.3
STN-109	312,824.38	1,113,978.97	386.0
STN-110	312,829.42	1,113,977.17	385.9
STN-111	312,163.61	1,114,222.26	351.0
STN-112	312,209.67	1,114,305.60	381.8
STN-113	312,261.99	1,114,359.00	403.6
STN-114	312,043.43	1,114,723.51	349.6
STN-115	312,148.78	1,114,670.38	388.2
STN-116	312,151.06	1,114,675.00	388.2
STN-117	312,849.17	1,115,060.52	348.1
STN-118	312,843.34	1,115,059.61	348.2
STN-119	312,857.74	1,114,949.90	380.9
STN-120	312,862.75	1,114,950.57	380.8
STN-121	312,853.82	1,114,867.54	405.6
STN-122	313,845.00	1,114,860.16	360.0
STN-123	313,861.58	1,114,760.17	386.3
STN-124	313,889.25	1,114,655.68	413.9
STN-125	313,973.57	1,114,166.63	444.5
STN-126	313,245.28	1,114,516.15	441.9
STN-127	312,463.74	1,114,532.22	441.4
STN-128	315,914.33	1,115,215.99	348.5
STN-128A	315,910.33	1,115,215.99	348.5
STN-128B	315,906.33	1,115,215.99	348.5
STN-129	315,283.90	1,116,404.12	349.3
STN-129A	315,275.92	1,116,404.53	349.3

LEGEND

- Soil Boring With Standard Penetration Tests And/Or Shelby Tube Sampling And Piezometer Location
- Soil Boring With Standard Penetration Tests And/Or Shelby Tube Sampling
- Slope Inclinometer
- Piezometer Location With Limited Sampling

NOTES:

1. Horizontal and vertical locations of borings provided by T.V.A. The topographic information shown was provided by T.V.A. from a survey performed in March 2008.
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.

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

RECORD DRAWING

For Supporting Design Calculations see FPGSHFFESCDX000000020100004

R 0 07/14/10 RECORD DRAWING

SCALE: 1"=200' EXCEPT AS NOTED

YARD
ASH POND 1 & 2 AND CONSOLIDATED WASTE DRY STACK
GEOTECHNICAL EXPLORATION
BORING AND STABILITY
CROSS SECTION LAYOUT

DESIGNED BY: J. CURTSINGER | DRAWN BY: S. BRADSHAW | CHECKED BY: J. CURTSINGER | SUPERVISED BY: N. BADER | REVIEWED BY: H. APARICIO | APPROVED BY: N. BADER | ISSUED BY: T. JOHNSON

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www.stantec.com

SHAWNEE FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

AUTOCAD R 2000 | DATE 07/14/10 | 35 | C | 10W501-02 | R 0

STANTEC | 0
TASK COMPLETED BY: | REV NO.

PLOT FACTOR:XX
W_TVA

C.A.D. DRAWING DO NOT ALTER MANUALLY

Stantec

Cross Section N - Ash Pond 2
Shawnee Fossil Plant
Paducah, Kentucky

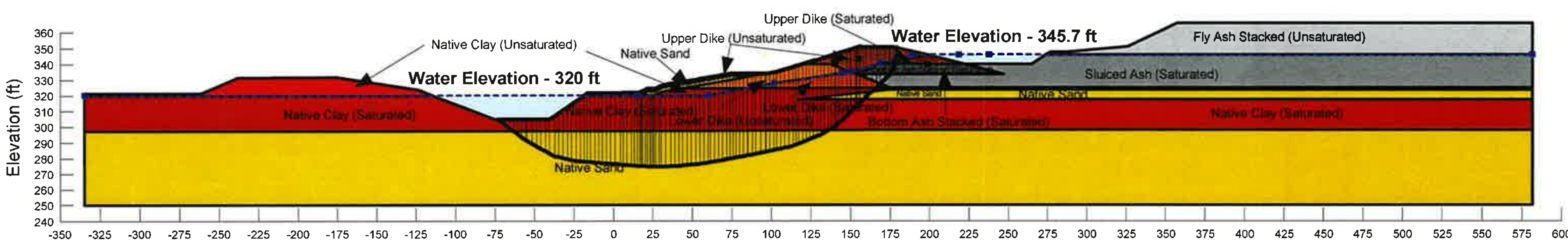
Existing Conditions - Post Earthquake

Liquefied Materials: Sluiced Ash, Native Sand

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.

Material Type	Unit Weight	Cohesion	Friction Angle
Native Clay (Unsaturated)	128 pcf	325 psf	13 °
Native Clay (Saturated)	128 pcf	260 psf	10.5 °
Native Sand	130 pcf	Sr=exp(-8.444+0.109N1(60)+5.379σ'^0.1), N1(60)=16	0 °
Upper Dike (Unsaturated)	130 pcf	800 psf	19 °
Upper Dike (Saturated)	130 pcf	640 psf	15.2 °
Lower Dike (Unsaturated)	127 pcf	460 psf	17 °
Lower Dike (Saturated)	127 pcf	368 psf	13.7 °
Bottom Ash Stacked (Saturated)	105 pcf	0 psf	26.5 °
Fly Ash Stacked (Unsaturated)	105 pcf	0 psf	32 °
Sluiced Ash (Saturated)	85 pcf	Sr=exp(-8.444+0.109N1(60)+5.379σ'^0.1), N1(60)=9	0 °

Factor of Safety: 1.0



SHF_Ash Pond_Section N_All_Final.gsz

PRELIMINARY WORK IN PROGRESS
FOR DISCUSSION PURPOSES ONLY

Distance (ft)

Project No. 175551015

Cross Section N - Ash Pond 2
Shawnee Fossil Plant
Paducah, Kentucky

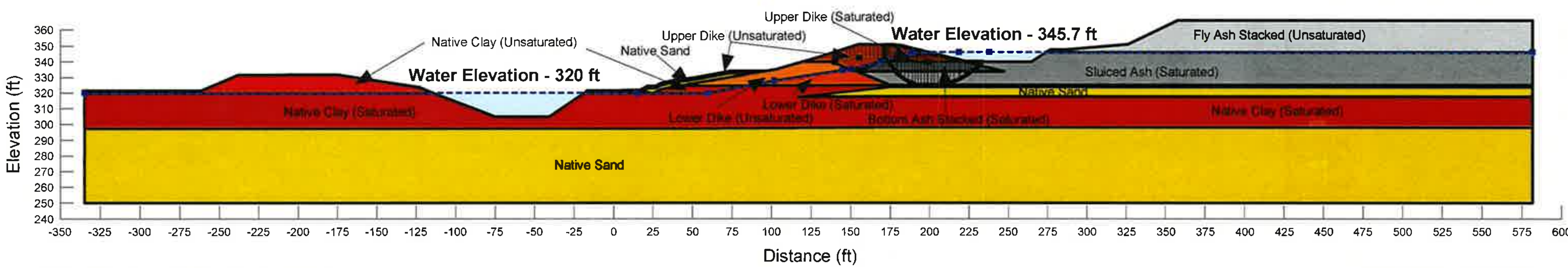
Existing Conditions - Post Earthquake

Liquefied Materials: Sluiced Ash, Native Sand

Material Type	Unit Weight	Cohesion	Friction Angle
Native Clay (Unsaturated)	128 pcf	325 psf	13 °
Native Clay (Saturated)	128 pcf	260 psf	10.5 °
Native Sand	130 pcf	Sr=exp(-8.444+0.109N1(60)+5.379σ'^0.1), N1(60)=16	0 °
Upper Dike (Unsaturated)	130 pcf	800 psf	19 °
Upper Dike (Saturated)	130 pcf	640 psf	15.2 °
Lower Dike (Unsaturated)	127 pcf	460 psf	17 °
Lower Dike (Saturated)	127 pcf	368 psf	13.7 °
Bottom Ash Stacked (Saturated)	105 pcf	0 psf	26.5 °
Fly Ash Stacked (Unsaturated)	105 pcf	0 psf	32 °
Sluiced Ash (Saturated)	85 pcf	Sr=exp(-8.444+0.109N1(60)+5.379σ'^0.1), N1(60)=9	0 °

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



SHF_Ash Pond_Section N_All_Final.gsz

Project No. 175551015

DRAFT

amec

TABLE 1

MEAN 2,500-YEAR UHRS (AT 5% DAMPING) FOR THE THREE FOSSIL PLANT SITES

(CUMBERLAND, ALLEN, SHAWNEE)

Frequency (Hz)	Spectral Acceleration (g)		
	Mean 2,500-year UHRS		
	Cumberland	Allen	Shawnee
0.1[1]	0.0100	0.0140	0.0174
0.133[1]	0.0158	0.0227	0.0286
0.167[1]	0.0223	0.0327	0.0413
0.2[1]	0.0293	0.0434	0.0552
0.25	0.0407	0.0610	0.0780
0.5	0.0832	0.1297	0.1709
1	0.1249	0.2087	0.2712
2.5	0.2673	0.4415	0.5908
5	0.3507	0.6022	0.8275
10	0.4132	0.7544	1.0565
25	0.5178	0.9491	1.3816
50	0.4544	0.8765	1.3047
100	0.2165	0.3891	0.5601

1 Extended frequencies based on ground motion spectral shapes at long periods for CEUS from NUREG/CR-6728

DRAFT

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TABLE 6

GROUND MOTION PARAMETERS FOR SPECTRALLY MATCHED TIME HISTORIES
SHAWNEE FOSSIL PLANT SITE
Tennessee Valley Authority

Comp.	Spectrally-Matched from	PGA (g)	PGV (cm/sec)	PGD (cm)	PGV/PGA (cm/sec/g)	PGA*PGD/PGV²	Duration ** (sec)
H	ABY000	0.597	25.95	15.02	43.47	13.06	16.88
H	SIL000	0.571	27.77	12.82	48.63	9.31	19.06
H	MCD090	0.568	23.32	14.83	41.06	15.20	15.12
H	FER-T1	0.594	26.32	13.77	44.31	11.58	22.68
H	ILA031-N	0.576	28.89	13.33	50.16	9.02	24.88
H	TCU025-W	0.604	33.40	17.27	55.30	9.17	14.80
H	ILA051-W	0.564	28.50	11.80	50.53	8.04	41.14

** Duration is defined as the time for cumulative energy to grow from 5% to 75% of its total value.

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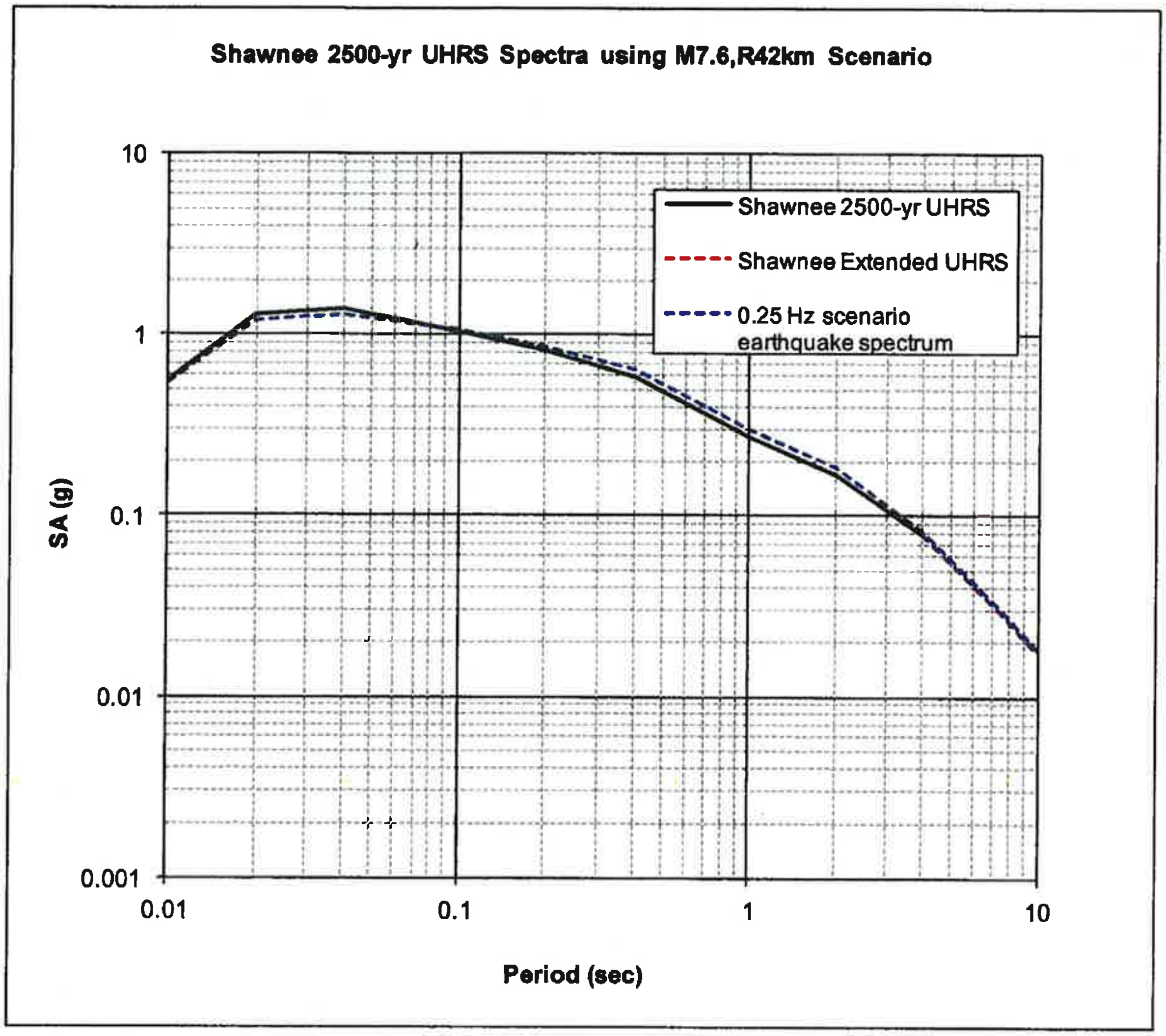
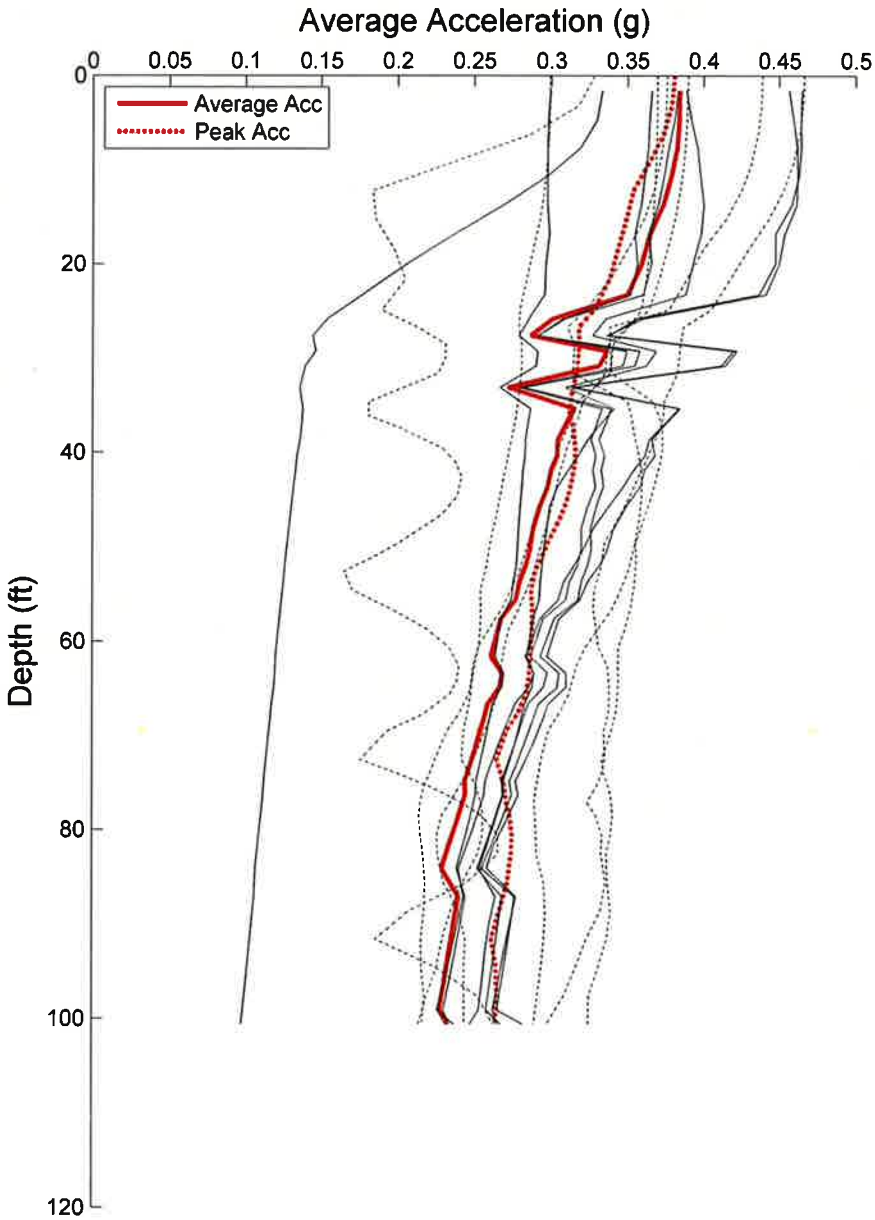
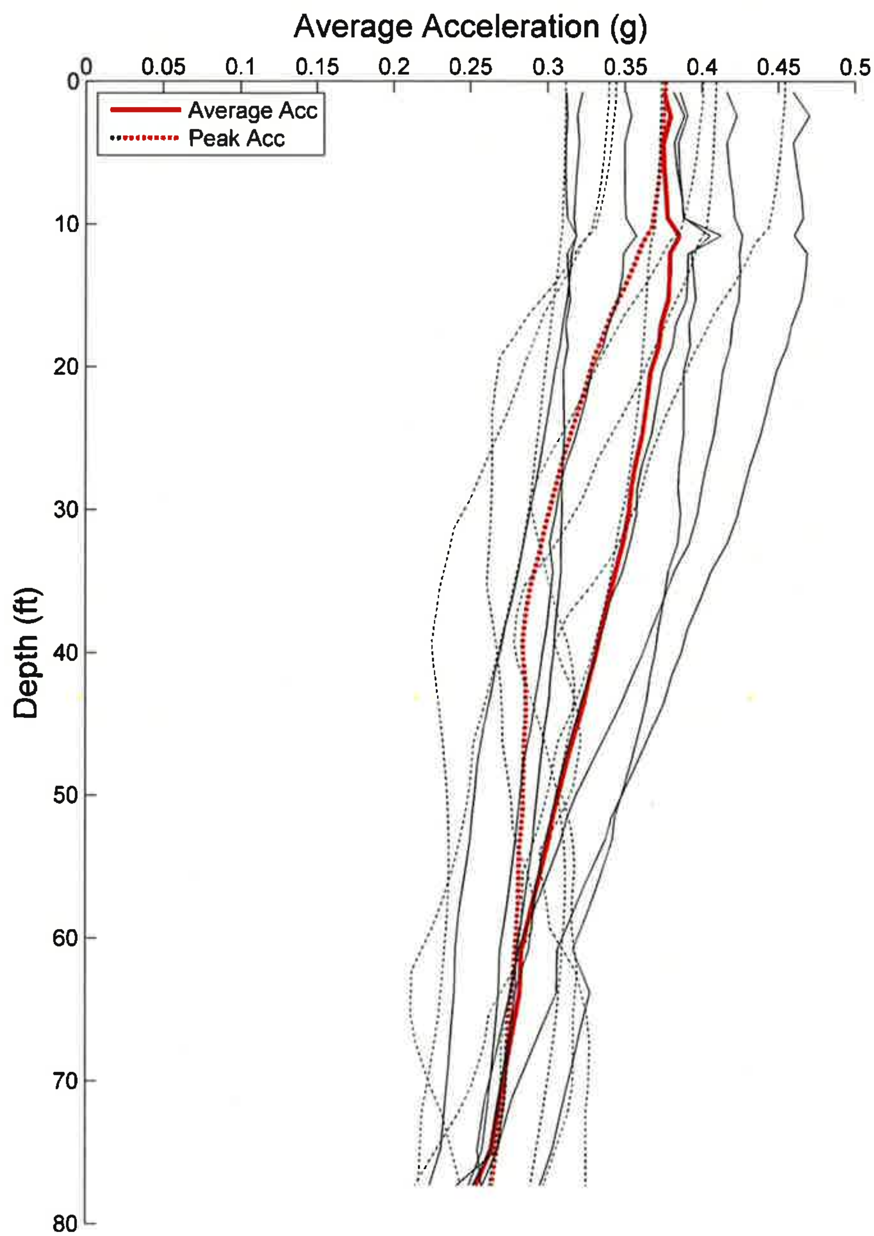


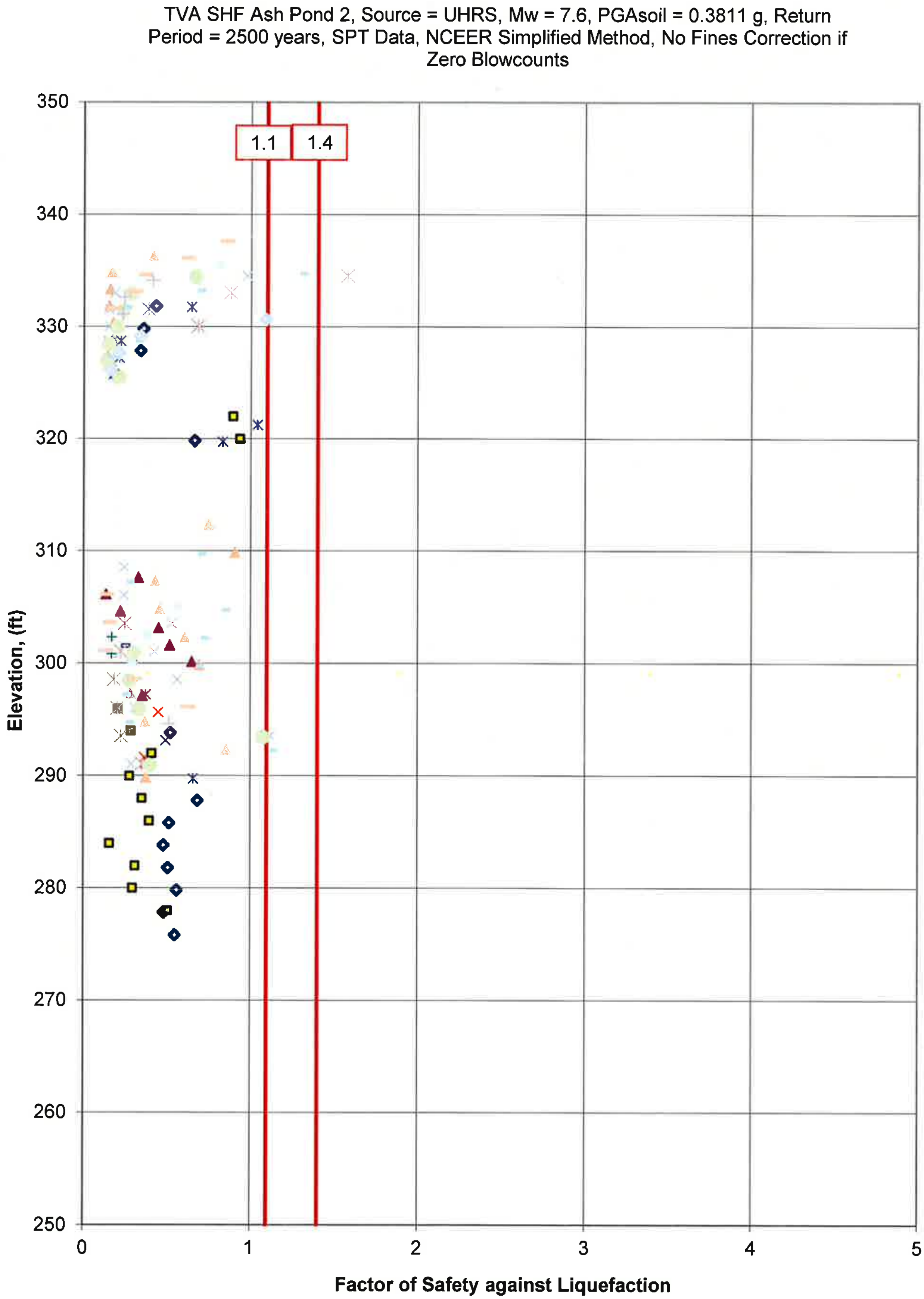
Figure 4: Horizontal Target 2500-yr UHRS (5% Damping) for the Shawnee Fossil Plant site

Acceleration versus depth profile at Boring SHF-N-2A (crest of upper dike at SHF Ash Pond 2, Section N). Results are derived from one-dimensional ground response analysis.



Acceleration versus depth profile at Boring SHF-N-2B (outboard toe of lower dike at SHF Ash Pond 2, Section N). Results are derived from one-dimensional ground response analysis.





TVA SHF Ash Pond 2, Source = UHRS, Mw = 7.6, PGAsoil = 0.3811 g, Return Period = 2500 years, SPT Data, NCEER Simplified Method, No Fines Correction if Zero Blowcounts
1.1
1.4
350
340
330
320
310
300
290
280
270
260
250
Elevation, (ft)
0
1
2
3
4
5
Factor of Safety against Liquefaction

Tennessee Valley Authority, 1101 Market Street, BR4A, Chattanooga, Tennessee 37402

October 19, 2012

Mr. Stephen Hoffman
US Environmental Protection Agency (EPA) (5304P)
1200 Pennsylvania Avenue, NW
Washington, DC 20460

TENNESSEE VALLEY AUTHORITY (TVA) – COMMENTS ON COAL ASH SITE ASSESSMENT ROUND 11 DRAFT REPORTS FOR ALLEN (ALF), BULL RUN, (BRF) COLBERT (COF), CUMBERLAND (CUF), GALLATIN (GAF), JOHN SEVIER (JSF), JOHNSONVILLE, (JOF) KINGSTON (KIF), PARADISE (PAF), SHAWNEE (SHF), WATTS BAR (WBF), AND WIDOWS CREEK (WOF) FOSSIL PLANTS

Dear Mr. Hoffman:

Tennessee Valley Authority (TVA) appreciates the opportunity to provide responses to the recommendations outlined in the Draft Coal Ash Site Assessment Round 11 Draft Reports for TVA's fossil plants. The Draft Reports were attached to EPA's September 5, 2012 email from Jana Englander to TVA's Susan Kelly. This EPA review process has provided TVA a public forum to confirm that our coal ash facilities meet current state requirements.

TVA has contracted with Stantec Consulting Services Inc., to assist in the technical review and responses to the EPA draft reports. The draft report responses are attached for your consideration in finalizing the Coal Ash Site Assessment Round 11 Reports. The following is a summary of our responses;

<u>Allen</u>: A seismic stability analysis and liquefaction analysis have been completed indicating acceptable performance under seismic loading. TVA recommends the Allen East Ash Pond be upgraded from Poor to Satisfactory.

<u>Bull Run</u>: TVA has no additional comments to EPA's analysis.

<u>Colbert</u>: TVA has no additional comments to EPA's analysis.

<u>Cumberland</u>: The operating pool level for the Ash Pond has been lowered 6.2 feet and the seepage analysis has been revised. Piping factors of safety are now satisfactory. TVA recommends the final rating for the Ash Pond be upgraded from Fair to Satisfactory.

A liquefaction potential assessment was performed for the Gypsum Disposal Area and showed the saturated ash materials are anticipated to undergo liquefaction for the 2,500-year earthquake. Therefore, a higher level of slope stability analysis was completed demonstrating that the factor of safety is satisfactory. TVA recommends the final rating for the Gypsum Disposal Area be upgraded from Poor to Satisfactory.

Additional seismic analysis and field investigation is underway for the Dry Fly Ash Stack. The results are indicating the possibility of a favorable response. However, the analysis is not complete. We anticipate its completion during EPA's review of these comments.

Gallatin: A seismic stability analysis for Ponds A and E has been completed with acceptable results. TVA recommends the final rating be upgraded from Fair to Satisfactory.

An additional stability and seepage analysis for the saddle dikes on the stilling ponds has been completed and a project to increase the hydrologic/hydraulic capacity of the ponds is underway. Based on the analysis and improvement plans underway, TVA recommends the Gallatin Stilling Ponds rating be upgraded from Poor to Fair and from Fair to Satisfactory once the project is completed.

John Sevier: The static and seismic slope stability analysis were reviewed and deemed to be appropriate for the soil materials present.

Johnsonville: A quantitative liquefaction analysis and a post-earthquake static slope stability analysis were performed. Results showed the slope to remain stable. As a result, TVA recommends that final rating for Ash Disposal Area 2 be upgraded from Fair to Satisfactory.

Kingston: TVA has no additional comments to EPA's analysis.

Paradise: A liquefaction analysis was performed and the hydrologic/hydraulic capacity was evaluated. The liquefaction analysis indicated that the materials would remain stable and not liquefy during a 2,500 year event. The H&H analysis confirmed that the ponds safely pass the 100-year 24-hour storm. However, they do not pass the Probable Maximum Flood. TVA has plans to design and construct features to correct this issue at the ponds. TVA recommends that the facilities at Paradise be upgraded from Fair to Satisfactory once the H&H issues have been addressed.

Shawnee: A liquefaction analysis and post-earthquake static stability analysis were performed with acceptable results. TVA recommends that the rating for Ash Pond No. 2 be upgraded from Poor to Satisfactory.

Watts Bar: A hydrologic/hydraulic analysis was performed for the design storm and the new spillway system currently under design and in construction. Based on the satisfactory outcome of the analysis; TVA recommends the final rating be upgraded from Fair to Satisfactory.

Widows Creek: TVA has no additional comments to EPA's analysis.

The following is a summary of the draft facility ratings and TVA's proposed final ratings.

EPA Draft Report Results				
Plant	**Facility**	**Draft Rating**	**Driver for Rating**	**Stantec Proposed Final Rating**
ALF	East Pond	**Poor**	Seismic	**Sat**
BRF	FA Pond	**Sat**		**Sat**
	BA Pond	**Fair**	Liquefaction	**Fair**
	Gyp Pond	**Fair**	Liquefaction	**Fair**
COF	Dry Stack	**Sat**		**Sat**
	BA Pond	**Fair**	Liquefaction	**Fair**
CUF	Ash Pond	**Fair**	Piping	**Sat**
	Dry Stack	**Poor**	Seismic	**Poor**
	Gyp	**Poor**	Seismic	**Sat**
GAF	Ash Ponds	**Fair**	Liquefaction	**Sat**
	Stilling Ponds	**Poor**	H&H and static	**Fair**
JSF	Dry Stack	**Sat**		**Sat**
	Ash pond	**Sat**		**Sat**
JOF	Island	**Fair**	Liquefaction	**Sat**
KIF	Ash/stilling	**Fair**	Liquefaction	**Fair**
	GDA	**Sat**		**Sat**
PAF	Scrubber sludge	**Fair**	H&H - overtopping	**Fair**
	Ash Pond	**Fair**	H&H - overtopping	**Fair**
	Slag Ponds	**Fair**	H&H - overtopping	**Fair**
SHF	Ash Pond	**Poor**	Seismic	**Sat**
WBF	Pond	**Fair**	H&H	**Sat**
WCF	Gyp stack	**Sat**		**Sat**
	Ash Pond	**Fair**	Liquefaction	**Fair**

TVA takes its environmental responsibilities very seriously and appreciates EPA's efforts to verify the quality of our impoundments. We would like to arrange a conference call once your staff has received this letter and briefly reviewed the attached reports so we can answer any immediate questions or concerns. Please contact Susan Kelly at (423)-751-2058 or sjkelly0@tva.gov to arrange this conference call.

Sincerely,

Cynthia M. Anderson
for
Brenda E. Brickhouse
Vice President
Compliance Interface and Permits

Enclosures

SJK:LMB
Enclosures
cc (electronic distribution with enclosures):
C. M. Anderson, BR 4A-C
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A. S. Cooper, OMA 1A-WDC
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J. C. Kammeyer, LP 5D-C
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