Stantec Consulting Services Inc.
10509 Timberwood Circle Suite 100
Louisville, KY 40223-5301
Tel: (502) 212-5000
Fax: (502) 212-5055

September 22, 2011 ltr_001_175551015

Mr. Michael S. Turnbow
Tennessee Valley Authority
1101 Market Street, LP 2G-C
Chattanooga, Tennessee 37402-2801

Re: Results of Seismic Slope Stability Analysis
Active CCP Disposal Facilities
Cumberland Fossil Plant

Dear Mr. Turnbow:

As requested, Stantec Consulting Services Inc. (Stantec) has conducted seismic slope stability analyses to support the U.S. Environmental Protection Agency's assessment of TVA's CCP disposal facilities. The results for Cumberland Fossil Plant (CUF) are presented in this letter.

1. Introduction

The U.S. Environmental Protection Agency is undertaking a nationwide effort to assess coal combustion product (CCP) disposal facilities. These assessments are now underway for facilities at TVA's fossil plants. To support TVA, Stantec has conducted seismic stability analyses for CUF's active disposal facilities, which include the Dry Fly Ash Stack, Gypsum Stack Complex, and the Ash Pond.

The seismic slope stability analyses results presented in this letter employ a pseudostatic approach and are representative of current conditions. For seismic assessment in upcoming closure design of these facilities, TVA will undertake a comprehensive risk/consequences-based approach, with design and mitigation decisions being based on the likelihood and consequences of failure. This approach is described in the document presented in Enclosure A. For CUF, closure of the Dry Fly Ash Stack, Gypsum Stack Complex, and Ash Pond are currently planned for 2021.

2. Seismic Stability Analysis Approach

Seismic slope stability has been performed for current conditions using pseudostatic stability methods, where the added inertial load from an earthquake is represented by a simple horizontal pseudostatic coefficient which provides an approximate representation of the dynamic loads imposed by an earthquake. Specifics related to the analyses/approach are as follows:

- Subsurface data was obtained from the following Stantec geotechnical reports:
    - Report of Geotechnical Exploration and Slope Stability Evaluation; Ash Pond; Cumberland Fossil Plant; Stewart County, Tennessee; March 29, 2010.
    - Report of Geotechnical Exploration; Dry Fly Ash Stack and Gypsum Disposal Complex; Cumberland Fossil Plant; Stewart County, Tennessee; June 11, 2010.
- SLOPE/W software (from GEO-SLOPE International, Inc.) was used to perform the calculations.
- One existing SLOPE/W cross-section model per disposal facility was selected for analysis. The selected sections are representative of the facility's lowest current static (long-term) factor of safety, with consideration given to proper representation of a release/breach. The selected SLOPE/W models were updated to reflect any significant mitigations or operational improvements that have occurred since completion of Stantec's geotechnical studies.
- Undrained shear strength parameters were used.
- Ground motion level corresponding to a return period of 500 years (or approximate exceedance probability of 10% in 50 years) was used for selection of horizontal seismic coefficients. This return period is consistent with seismic stability analysis guidance provided by Tennessee's dam safety regulations Chapter 1200-5-7, "Rules and Regulations Applied to the Safe Dams Act of 1973". The peak ground acceleration (or seismic coefficient) for a 500 year return period was selected from Table 16 of TVA's March 28, 2011 region-specific seismic hazard study performed by AMEC Geomatrix, Inc.
- A target factor of safety (FS) of 1.0 was considered for comparing results.

3. Results

The results of the pseudostatic stability analyses are presented below. Also, Enclosure B presents a summary spreadsheet, SLOPE/W cross-sections, and plan views showing cross-section locations.

Ash Pond:

The results indicate a factor of safety of 1.2 for current conditions, which exceeds the target of 1.0.

Gypsum Stack Complex and Dry Fly Ash Stack:

The minimum factors of safety for current conditions for both CUF stack facilities are 0.8 for ground motion corresponding to a 500 year return period, with resulting failure surfaces that are confined to the interior and that do not constitute a failure of the perimeter dike system. Seismic coefficients and return periods resulting in a factor of safety of 1.0 were then back-calculated for these interior failures for each stack. These resulting return periods for FS = 1.0 are 170 years for the Dry Fly Ash Stack and 225 years for the Gypsum Stack Complex, which corresponds to exceedance probabilities of approximately 25% and 20% in 50 years, respectively (or approximately 0.6% and 0.4% annually). For deeper seated failure surfaces that would result in a failure of the exterior dike systems, resulting factors of safety were found to be 1.0 for the Gypsum Stack and the Fly Ash Stack, which meets the target value.

Although the minimum FS's for the stacks under the conditions analyzed are less than the target of 1.0, it is judged that the risk of slope stability failure under seismic loading conditions is acceptable, considering 1) that the resulting minimum FS failure surfaces are upstream of the perimeter dike systems, 2) deeper seated failure surfaces that would result in a failure of the perimeter dikes meet the target of 1.0, and 3) TVA plans to close the facilities in 2021 and will further consider seismic risks during closure design as previously described.

Stantec appreciates the opportunity to provide these services. If you have questions, or if we can provide additional information, please let us know.

Sincerely,

STANTEC CONSULTING SERVICES INC.

Randy L. Roberts

Randy L. Roberts, PE
Principal

Enclosures

/cdm

Enclosure A

White Paper - Seismic Risk Assessment Closed CCP Storage Facilities

Seismic Risk Assessment
Closed CCP Storage Facilities
Tennessee Valley Authority Fossil Plants

Prepared by:

Alan F. Rauch, PhD, P.E.
Senior Associate
Stantec Consulting Services, Inc.

Wayne Quong, M.A.Sc., P.Eng.
Senior Associate
Stantec Consulting Services, Inc.

Jeffrey S. Dingrando, P.G., P.E.
Associate
Stantec Consulting Services, Inc.

Reviewed and Approved by:

Barry S. Snider, P.E.
TVA-CCP Interim Engineering Manager
Tennessee Valley Authority

William H. Walton, P.E.
Vice President, Senior Principal Engineer
AECOM

Thomas L. Cooling, P.E.
Vice President, Geotechnical Services
URS Corporation

Gonzalo Castro
Independent Consultant

Seismic Risk Assessment
Closed CCP Storage Facilities
Tennessee Valley Authority Fossil Plants

This document outlines proposed engineering analyses to estimate seismic failure risks at wet storage facilities for coal combustion products, following closure, at various TVA fossil power plants. The specific details outlined in this document are subject to future discussion and modification by the project team.

OVERVIEW

Tennessee Valley Authority (TVA) operates storage facilities for coal combustion products (CCPs) at eleven fossil power generating stations. As TVA transitions to dry systems for handling these materials, 18 to 25 wet storage facilities (CCP ponds, impoundments, dredge cells, etc.) will be closed (drained and capped). The CCP storage facilities are currently operated in accordance with state and federal regulations, but previously issued permits have not required evaluations for seismic performance. Moreover, the existing permits do not require seismic qualification for the storage facilities in their closed configurations.

TVA recognizes there is a potential for strong earthquakes to occur within the region, and there is a tangible risk for seismic failure at each closed CCP facility. These risks, including both the likelihood of failure and the consequences, must be understood to effectively manage TVA's portfolio of byproduct storage sites. This white paper summarizes the methodology that will be used to estimate these risks at the CCP storage facilities following closure.

Seismicity in the TVA service area is attributed to the New Madrid fault and smaller, less concentrated crustal faults. These two earthquake scenarios generate significantly different seismic hazards at each locality and will be considered independently within the risk assessment. At each closed byproduct facility, potential seismic failure modes will be evaluated in sequence. Instability due to soil liquefaction, slope instability due to inertial loading, and other potential failure mechanisms will be addressed. Seismic performance will be evaluated for differing earthquake return periods until a limiting (lowest return period) event that would cause failure is obtained. The probability of seismic failure will then correspond to the probability of this limiting earthquake event. The assessment of risk will also include estimates of potential consequences, as well as costs to mitigate the risks, that reflects the unique setting of the individual storage facilities after closure.

Following the same general methodology, seismic risks will be estimated in two phases. The near-term "Portfolio Seismic Assessment" will provide a rough estimate of seismic risks. The likely performance of each facility will be evaluated using simplified analyses, empirical methods, and the judgment of experienced engineers. The results will establish a ranking of the relative risks across the closure portfolio and also provide a preliminary picture of overall seismic risk. For the subsequent "Facility Seismic Assessments", seismic performance will be judged on the basis of site-specific data and detailed engineering analyses, which will be completed during the closure design process for individual facilities.

SEISMIC RISKS

This white paper provides an overview of the engineering methods proposed by Stantec for estimating seismic risks at TVA's closed byproduct storage sites. For each facility, four specific questions must be answered quantitatively:

(1) What is the approximate probability that a strong earthquake will occur?

Several seismic source zones could produce earthquakes large enough to impact these TVA sites. Very large magnitude earthquakes have occurred within the New Madrid seismic zone, which is located along the western boundaries of Tennessee and Kentucky. Because of their observed large magnitude and frequency of occurrence, New Madrid events contribute substantially to the seismic risks at all TVA sites. Ground motions from a New Madrid earthquake would attenuate with distance toward the east, such that local area sources also contribute significantly to site-specific seismic hazards.

Seismicity across the Tennessee Valley was previously characterized by AMEC/Geomatrix (2004), in a probabilistic study that focused on TVA dam sites. The same seismogenic model can be applied in evaluating earthquakes that would impact other TVA sites. Accordingly, probabilistic seismic hazards obtained from the 2004 AMEC/Geomatrix model will be used in the seismic risk assessment of the closed CCP storage facilities.

(2) Will a given earthquake cause failure in the closed facility?

Many of the TVA byproduct storage facilities are underlain by a substantial thickness of loose, saturated, alluvial soils (silts and sands). Some facilities will have layers of ash or other uncemented CCPs that remain saturated following closure. These materials, especially sluiced fly ash, are prone to liquefaction in a strong earthquake, as cyclic motions cause a build up of pore water pressure and a consequent loss of effective stress and shearing resistance. Extensive liquefaction in a foundation or CCP deposit under a storage facility would be expected, in most cases, to result in lateral spreading and massive slope movements (failure). Even without liquefaction, large slope deformations or failures may be triggered by lateral inertial loads during an earthquake. Liquefaction and dynamic loading of slopes are the most likely failure mechanisms, but other seismic failure modes, which may be unique to a particular closed storage facility, must also be evaluated.

(3) What are the potential consequences of a failure?

In addition to understanding the probability of failure, a risk assessment should consider the potential consequences. A failure is likely to have economic costs associated with clean-up and restoration of the site. Depending on the local site conditions, failure of a closed CCP facility may or may not cause significant impacts on the environment, waterways, transportation routes, buried or overhead utilities, or other infrastructure. Substantial economic costs would result if power generation is interrupted. Failure consequences may also include the potential loss of human life at some sites.

In this proposed seismic risk assessment, the definition of "failure" will be constrained to mean the displacement of stored materials to a distance beyond the permitted boundary of the facility. While smaller deformations in a closed storage facility could cause economic damages, the resulting consequences for TVA should be manageable. Hence, this risk assessment will focus on potential "failures" where stored materials could move past the permitted boundary.

(4) What are the approximate costs to mitigate the risks of a seismic failure?

With an understanding of the probability and consequences of failure, the potential risks can be quantified and understood, possibly leading to decisions to mitigate seismic risks in the closure of certain facilities. Mitigation measures might include ground improvement to reduce liquefaction potential (stone columns, deep soil mixing, jet grouting, or other appropriate technology), stabilization of slopes by flattening or buttressing, enhanced drainage features, or some other engineered solution. The potential cost of these risk mitigation strategies are needed to make appropriate management decisions.

PORTFOLIO AND FACILITY ASSESSMENTS

Seismic evaluations will be completed for each of the CCP storage facilities that TVA has slated for closure; a tentative list is given in Table 1. The assessment of seismic risks will be accomplished in two phases:

A. Portfolio Seismic Assessment

In this first phase, the seismic risk assessment will be carried out using general site information, simplified analyses, empirical methods, and the judgment of experienced engineers. A team of four to five engineers will complete this evaluation for the entire portfolio, with assistance from the engineering teams currently working on each facility. After the probabilistic seismic hazards are defined, this phase of the work can be completed in a relatively short timeframe.

Given the level of effort and the simplified engineering analyses to be employed, the seismic risk estimates from the Phase A assessment will be approximate. Rather than attempting to compute precise risk numbers, Phase A will focus on capturing the relative risks between the different closed facilities. The key to successfully meeting this objective will be the consistent application of the assessment process across the portfolio.

This effort will result in a ranked list of sites that can be used to illustrate where seismic risks are greatest within the portfolio. The results will also provide some insight for understanding and communicating the magnitude of potential risks associated with seismic loading of the closed CCP facilities.

As a secondary objective, the Phase A assessment team will also consider the potential for failure of the active storage facilities, due to an earthquake occurring prior to closure. The seismic risks associated with the operating facility will not be estimated, but the Phase A assessment process provides an opportunity to identify potential failure mechanisms that should be addressed in the short term. This information may suggest the need to re-prioritize the closure schedule. Prior to closure, many of the wet CCP storage facilities retain large pools of water and are thus more susceptible to uncontrolled releases in an earthquake. TVA has already made the decision to close these wet storage facilities to manage these risks, so the effort in Phase A will focus on identifying sites that may have unusually high seismic risks and deserve more study or higher priority in the closure program.

B. Facility Seismic Assessment

In this subsequent phase of work, more detailed engineering analyses will be carried out using site-specific geometry, subsurface conditions, material parameters, and results from static slope stability analyses. Simplified, state-of-the-practice methods of engineering analysis will be used; more complex analytical methods will be generally impractical for this risk assessment.

This phase of the work will be accomplished for individual facilities as part of the closure design, after the completion of other engineering analyses. The risks will be quantified by the design team, with assistance from the portfolio seismic assessment team. Significant, detailed effort will be required to assess each closed facility.

Compared to Phase A, the risk estimates obtained at this stage will be more reliable and better represent the actual risks for seismic failure. While it will be impossible to know how accurately the risks have been characterized at the completion of Phase B, the objective is to obtain results that are within perhaps ± 30% of the "actual" risk numbers. TVA expects to use the Phase B results to decide if the risks are acceptable, or if the closure design should be modified to mitigate risks for a seismic failure.

The engineering methodology (described below) to be followed in the Phase A and B evaluations will not characterize all of the uncertainties with respect to seismic performance. The uncertainties in the soil parameters and in the liquefaction, stability, and deformation analyses will not be quantified and carried through the risk assessment. Consequently, the estimated risk numbers will be approximate, but the results will be sufficiently accurate to support TVA decisions regarding prioritization for closure or the need for seismic mitigation. At most sites, the risks are expected to be high enough or low enough that further refinement in the risk numbers would not change these decisions. More detailed analysis beyond Phase B would be unjustified in these cases.

This assessment plan does not preclude the possibility that more detailed risk evaluations could be undertaken in subsequent phases of work. The Phase B results might reveal a subset of closed facilities with marginal risks, where a more rigorous and complete calculation of the risks would be needed to support a management decision. Hence, at the conclusion of the Phase B assessments, a "Phase C" evaluation may be needed for select sites and facilities, wherein uncertainties in the soil parameters and performance analyses would be quantified and carried through the risk assessment.

RESULTS AND APPLICATION

The results from the Phase A Portfolio Assessment will be presented in a table, like Table 1. For each facility evaluated, the estimated annual probability of failure due to a seismic event, the expected consequences (economic costs and potential loss of life), and the mitigation costs (design features to reduce risks) will be tabulated. The same parameters, but more accurate numbers, will be reported from the more in-depth Phase B assessments. A qualitative description of the data quality (based on the number of borings, test data on key soil properties, etc.) will also be included, to indicate how well the site conditions were characterized at the time of the Phase A or B assessment.

In both Phase A and B, the evaluation teams will prepare a discussion of significant issues driving the seismic risks at each site. This summary will include knowledge gaps, likely failure mechanisms, unique consequences, suggested approaches for risk mitigation, and other key information. The Phase A evaluation of a facility may point out the need for additional data to support later seismic analyses in Phase B; needed field or laboratory testing could then be accomplished and documented as part of the facility closure design effort.

In the short term, TVA will utilize the Phase A results to better plan budgets and schedules for managing the closure process over the next several years. The Phase A assessment will also be used as an opportunity to identify operating facilities with especially high seismic risks. While these risks will not be quantified for conditions prior to closure, the consideration of potential seismic failure modes may prompt additional study and reconsideration of priorities. Where justified, the priorities for closure may be changed to more quickly address sites with higher seismic risks.

More accurate risk estimates will be obtained from the Phase B assessments, which will be completed as part of the closure design process. Those results will be used, within TVA's existing decision making framework, to judge if seismic mitigation is needed. For context, the criteria in Tables 2 and 3 represent the risk-based framework TVA uses to guide enterprise-level decisions. This framework relies upon broad, qualitative scoring of consequences and risks for the organization. For managing the seismic risks at the closed CCP facilities, complete probabilistic calculations of risk are not needed; approximate estimates of seismic risk will be sufficient to support TVA decisions.

The risks computed in Phase A and B will not be compared to a prescribed threshold or design risk level. Criteria for tolerable seismic risk in these closed CCP storage facilities has not been defined in the existing permits, in TVA policy, or in TVA design guidance.

METHODOLOGY

The same general methodology, outlined in ten steps below and in Figures 1 through 4, will be used to evaluate seismic risk in both the Phase A Portfolio Assessments and the Phase B Facility Assessments. While advanced engineering analyses may be required to demonstrate acceptable seismic performance in a design situation, simplified analyses will be used here, consistent with the goal of estimating the probability of failure.

In Step 1, seismic hazard parameters will be defined for each site; the results will be used as inputs for both the Phase A and Phase B assessments. Then, the evaluation of a particular facility will begin with a review of existing site information (Step 2), followed by engineering analyses for seismic performance. As described in Steps 3 through 7 below, the engineering analyses in Phase B will be more detailed than the simplified estimates in Phase A. The analyses will commence with an initial selection of an earthquake return period and evaluation for seismic performance. Steps 3 through 7 will be repeated until the limiting (lowest) earthquake return period expected to cause failure is obtained. Flowcharts summarizing Steps 1 through 7 in the Phase A and B seismic performance assessments are given in Figures 3 and 4, respectively. The earthquake event with the lowest return period that causes failure will then be used to compute the probability of failure in Step 8. The potential consequences and mitigation costs will be estimated in Steps 9 and 10.

Step 1 – Define Seismic Input Parameters

Seismic hazards at TVA dam sites were quantified in a 2004 study by AMEC/Geomatrix. The New Madrid fault zone and several area source zones contribute to the seismicity of the region, as represented schematically in Figure 1. The New Madrid seismic zone is characterized by a large linear, combined reverse/strike-slip fault. Earthquakes in the area source zones are more diffuse (less concentrated in clusters) and tend to occur in zones of weakness of large crustal extent rather than along narrow, well-defined faults. Earthquakes occurring within the New Madrid Seismic Zone and in area sources outside of it will be considered in developing seismic input parameters for each CCP facility. However, only seismic source zones that contribute significantly to the ground motion hazard at a particular site will be used to develop seismic input parameters.

The national USGS seismic hazard model will not be used in these seismic risk assessments; instead, TVA will ask AMEC/Geomatrix to compute the site-specific seismic hazards for each closed CCP facility. The needed information can be obtained from the existing seismogenic model, but will need to separately consider the hazards associated with the New Madrid events and all other seismic sources (Figure 2), hereafter referred to in this white paper as the “earthquake scenarios”. The following parameters are needed for each earthquake scenario:

- Uniform hazard spectra for frequencies from 0.25 to 100 Hz (100 Hz value is equivalent to peak ground acceleration, PGA) at the top of rock for a range of return periods from 100 to 2,500 years.
- De-aggregation for relevant ground motion frequencies (one or more of the following: 0.5, 1.0, 2.5, 5.0, and 100 Hz) at each return period. The de-aggregation results will be used to select appropriate, representative earthquake parameters (magnitude and distance from the site), from which inputs needed for liquefaction analyses can be developed.

In the Phase A effort, the project team (including seismologists designated by TVA) will meet to consider the earthquake hazard data produced by the AMEC/Geomatrix model for each site. The team will reach consensus on the appropriate parameters (return period, earthquake magnitude, and peak ground acceleration) to be used in evaluating each facility, before proceeding with work on subsequent steps of the analysis. The seismic parameters to be tabulated (Table 4) will then be used in both the Phase A and Phase B assessments.

Ground motion time histories will be needed for the detailed Phase B calculations, and TVA will need to ask AMEC/Geomatrix to provide:

- Representative acceleration time histories (two orthogonal components), representing ground motions at the top of the rock profile for the specified earthquake return periods.

Given the results of the Phase A assessment, the Phase B analyses will focus on a narrower range of possible earthquakes. Hence, acceleration time histories will not be needed for every seismic event listed in Table 4.

Step 2 – Review Site and Facility Information

To meet the requirements for closure of TVA ash storage facilities, the closed condition may involve placement of compacted ash behind a strengthened dike, drainage of pond water to the levels of the surrounding groundwater table, and capping of the area with native soils. The collection of available site information for each facility will be reviewed from a seismic performance perspective. For the Phase B assessment, this information will be augmented with new data that becomes available during the closure design process.

The project information needed for each storage facility includes:

- Planned geometry of the closed storage facility, as needed to meet current design criteria and regulatory requirements.
- Geologic mapping and related information about the site geology.
- Historical records and other information related to site development.
- Boring logs, SPT data, CPT data, shear wave velocities, etc. from field explorations.
- Laboratory data from testing of site materials, including classification, Atterberg limits, moisture content, particle size, specific gravity, unit weight, compaction tests, and other relevant test data.
- Laboratory data on measured strength properties, for both drained and undrained conditions.
- Previously completed slope stability analyses, where available, will be modified for calculations in the risk assessments.

Step 3 - Evaluate Potential for Soil Liquefaction

The potential for soil liquefaction may be the greatest contributor to failure risk at many of the TVA storage sites. Liquefaction will thus be considered first in the assessment of seismic performance at each closed facility (Figures 3 and 4).

The Phase A assessment will utilize empirical charts and back-of-the-envelope calculations to judge if liquefaction would be likely for a given earthquake scenario. For example, Ambraseys (1988) compiled magnitude, epicentral distance, and whether or not liquefaction was observed in past earthquakes, and then suggested a threshold boundary (in terms of magnitude and epicentral distance) where liquefaction might occur in natural soil deposits. Selected, parametric calculations with the simplified procedure outlined by Youd et al (2001) will also be useful in judging what earthquakes would cause liquefaction in the Phase A Portfolio Assessments. These empirical methods may be unconservative for evaluating saturated CCPs, which are often more prone to liquefaction than a sandy soil, but the results will still provide useful guidance in the Phase A assessment.

For the Phase B liquefaction evaluations, detailed engineering analyses will be undertaken to obtain estimates of cyclic loading, soil resistance, and factor of safety as described below. Potentially liquefiable soils include saturated alluvial soils, loose granular fills, and sluiced ash. The detailed analyses will focus on critical cross sections of the closed facilities; liquefaction safety factors will not be computed for all boring locations at a site.

(a) Soil Loading from Earthquake Motions

The magnitude of the cyclic shear stresses induced by an earthquake are represented by the cyclic stress ratio (CSR). The simplified method proposed by Seed and Idriss (1971) will be used to estimate CSR in the Phase A parametric analyses (ground response analyses will not be completed in Phase A).

In Phase B, the CSR at specific locations (borings and depths where in situ penetration resistance are measured) will be computed using one-dimensional, equivalent-linear elastic methods as implemented in the ProSHAKE software. Using an acceleration time history at the top of rock (obtained from the seismic hazards study in Step 1), the computer program will model the upward propagation of the ground motions through a one-dimensional soil profile. For cases where the one-dimensional assumption is inadequate, the calculations can be accomplished using QUAKE, a two-dimensional finite element program that implements the same dynamic modulus reduction curves and damping relationships as used in ProSHAKE.

The cyclic stresses imparted to the soil will be estimated from the earthquake parameters described in Step 1, representing earthquakes on the New Madrid fault and local crustal events.

(b) Soil Resistance from Correlations with Penetration Resistance

The resistance to soil liquefaction, expressed in terms of the cyclic resistance ratio (CRR), will be assessed using the NCEER empirical methodology (Youd et al. 2001). Updates to the procedure from recently published research will be used where warranted. The analyses will be 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). In Phase A, typical or representative values will be used in parametric hand calculations; detailed data from site-specific explorations will be analyzed in Phase B.

The NCEER procedure involves a large number of correction factors. Based on the site-specific conditions and soil characteristics, engineering judgment will be 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 will not be applied where zero blowcounts (“weight of hammer” or “weight of rod”) are recorded. The magnitude scaling factor (MSF) is used in the empirical liquefaction procedure to normalize the representative earthquake magnitude to a baseline 7.5M earthquake. The earthquake magnitude (M) considered to be most representative of the liquefaction risk will be determined by applying the MSF to the de-aggregation data (from Step 1) for each selected earthquake return period.

Saturated fly ash, where it remains following closure, is likely to be more susceptible to liquefaction than indicated by these empirical methods. Values of CRR determined via the NCEER procedure are related to the observation of liquefaction in natural soils, mostly silty sands. Given the spherical particle shape and uniform, small grain size of fly ash, the NCEER procedure may give CRR values that are too high for saturated fly ash.

Lacking better methods of analysis, the lower-bound, "clean sand" base curve (Youd et al. 2001) will be assumed to apply for fly ash in the Phase A assessment. Within the liquefaction calculations, this will be accomplished for these materials by neglecting the fines content adjustment to the normalized penetration resistance. For Phase B, published and unpublished data from cyclic laboratory testing on similar materials will be sought to augment the indications of liquefaction resistance obtained from in situ penetration tests.

(c) 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 this criteria for guidance, values of FS_{liq} computed throughout a soil deposit or cross section (at specific CPT-q_c and SPT-N locations) will be 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, will be used for the overall assessment of each facility. A determination of "extensive" or "insignificant" liquefaction will then lead to the appropriate stability analyses in the next stage of the evaluation, as indicated in Figures 3 and 4.

Step 4 – Characterize Post-Earthquake Soil Strengths

The post-earthquake shearing resistance of each soil and CCP will be estimated, with consideration for the specific characteristics of that material. The full, static shear strength will be assigned to unsaturated soils. Excess pore pressures will not develop in an unsaturated soil during seismic loading, so drained strength parameters can be used. The undrained strengths of saturated soils will be decreased to account for the softening effects of pore pressure buildup during the earthquake. Specifically:

- In saturated clays and soils with $FS_{liq} > 1.4$, 80% of the static undrained strength will be assumed.
- In saturated, low-plasticity, granular soils with $1.1 < FS_{liq} \leq 1.4$, a reduced strength will be 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_{us}) will be estimated for the liquefied soil. Values of S_{us} can be obtained from the empirical correlations published by Seed and Harder (1990), Castro (1995), Olson and Stark (2002), Seed et al. (2003), and Idriss and Boulanger (2008).

Subsequent stability and deformation analyses will be accomplished using these reduced strength parameters. No attempt will be made to model the cyclic reduction in soil shear strength during an earthquake. In the deformation analyses, the fully reduced strengths will be assumed at the start of cyclic loading, which will yield conservative estimates of slope displacements.

Step 5 – Analyze Slope Stability

The next step in the performance evaluation (Figures 3 and 4) will consider slope stability, for conditions with or without significant liquefaction. Slope stability will be evaluated using two-dimensional, limit equilibrium, slope stability methods. Reduced soil strengths (from Step 4), conservatively representing the loss of shearing resistance due to cyclic pore pressure generation during the earthquake, will be used in the stability calculations. The analyses will be accomplished using Spencer's method of analysis, as implemented in the SLOPE/W software, considering both circular and translational slip mechanisms.

Input files for static stability calculations, where previously completed for a particular facility, will be updated to represent seismic conditions. These stability analyses may be not available, or the closure geometry may be undefined, for the Phase A assessment of some sites. In those cases, simplified or approximate geometries will be developed for approximate analysis in Phase A. Engineering experience will also be useful in judging likely seismic stability. For example, a complete failure is likely if liquefaction undermines the foundation of the outslope. In the absence of liquefaction, a slope that exhibits adequate safety factors under static conditions is unlikely to fail in an earthquake. Back-of-the-envelope hand calculations can be useful in assessing stability where extensive liquefaction occurs in the saturated materials within or below CCPs retained by a stable perimeter dike. Detailed slope stability calculations, which accurately represent the planned closure geometry, will be used in the Phase B facility assessments.

(a) Slope Stability if Extensive Liquefaction

If extensive liquefaction is indicated, stability will be evaluated for the static conditions immediately following the cessation of the earthquake motions. Residual or steady state strengths will be assigned in zones of liquefied soil, with reduced strengths that account for cyclic softening and pore pressure build up assumed in non-liquefied soil. In both Phase A and B, complete failure (large, unacceptable displacements) will be assumed if the safety factor (FS_{slope}) computed in this step is less than one (Figures 3 and 4).

For slopes where the post-earthquake $FS_{slope} \geq 1$, deformations will be estimated in the Phase B assessment (Step 6 and Figure 4). Slope deformations will not be estimated in the Phase A portfolio assessment, where ground motion time histories will not be available. In Phase A, slopes exhibiting $FS_{slope} \geq 1$ with liquefaction will be assumed stable with tolerable deformations; this condition may exist, for example, where liquefied ash at the base of a closed storage facility is contained within a stable perimeter dike.

Note that pseudostatic stability analyses are not useful for evaluating a factor of safety where extensive liquefaction is expected, because appropriate pseudostatic coefficients can not be defined.

(b) Slope Stability if No Significant Liquefaction

If no significant liquefaction is expected, seismic stability will be analyzed in Phase A using approximate, pseudostatic stability methods (Figure 3). The added inertial loads from the earthquake will be represented with a simple, horizontal pseudostatic coefficient (k_h), which provides an approximate representation of the dynamic loads imposed by an earthquake. The horizontal pseudostatic coefficient will be set to one-tenth of the peak ground acceleration in rock ($k_h = 0.1 \cdot PGA_{rock}$). In Phase A, tolerable deformations (less than about 5 meters) will be assumed if the pseudostatic $FS_{slope} \geq 1$, and failure will be assumed if the pseudostatic $FS_{slope} < 1$.

This approach and criteria are based on the work of Hynes-Griffin and Franklin (1984). They performed Newmark deformation analyses, integrated over 350 ground motion time histories, used an amplification factor of three to represent peak accelerations at the base of an earth embankment, and assumed a displacement of 1 meter would be tolerable for an embankment dam. For a typical CCP facility, assuming no pool is retained following closure, “failure” would imply displacements significantly greater than 1 meter. A tolerable displacement of about 5 meters will be assumed here, for the Phase A risk assessments. From the upper bound curve plotted by Hynes-Griffin and Franklin (1984), a displacement of 5 meters would correspond to a yield acceleration of about 0.03 times the peak acceleration along the slip surface. Then, assuming an amplification factor of 3 for the ground motions at the base of the embankment, this suggests $k_h = 0.1 \cdot PGA_{rock}$ can be used conservatively in the pseudostatic analysis to judge failure, as described above.

Pseudostatic factors of safety will not be computed in the Phase B assessment. Instead, where a liquefaction failure is not predicted, potential slope displacements will be computed as described in Step 6.

Step 6 – Predict Deformations

In the Phase A Portfolio Assessment, closed facilities that are expected to remain stable (pseudostatic $FS_{slope} \geq 1$ with no liquefaction, or post-earthquake $FS_{slope} \geq 1$ with liquefaction) will be assumed to have tolerable displacements. Dynamic slope deformations are difficult to estimate without detailed analysis; the available empirical or approximate methods do not represent the conditions of interest, or the level of effort is not consistent with the goals of the first phase of risk assessments. In addition, earthquake ground motion time histories will not be available for the Phase A analyses.

In the Phase B Facility Assessments, the potential deformation of stable slopes will be evaluated as indicated in Figure 4. Conventional methods of analysis will be implemented to estimate potential slope displacements that accumulate during earthquake shaking; movements are assumed to stop when the earthquake ends, consistent with a post-earthquake safety factor greater than one. The acceleration time histories obtained from the ground response analyses in Step 3a will be used as inputs for computing deformations with one of the following simplified methods:

- Newmark's (1965) method involves double integration of accelerations greater than the yield acceleration (k_y), which will be determined from a succession of pseudostatic slope stability analyses in which k_h is varied. The value of k_h where the pseudostatic FS_{slope} = 1.0 corresponds to the yield acceleration.
- The Makdisi-Seed (1978, 1979) procedure, which better accounts for the dynamic response of embankments. This procedure was developed based on parametric numerical simulations for earthen dams. The procedure is iterative, considers the fundamental periods of the embankment response, and can be completed in steps using published charts. Results from QUAKE can also be used as input in this procedure.

The slope deformations predicted in Phase B will be conservative, because the yield acceleration will be computed based on reduced, post-earthquake soil strengths. In reality, the yield acceleration declines in successive cycles of seismic loading, as pore pressures accumulate and saturated soils become weaker. The analysis outlined in Figure 4 assumes reduced strengths and, where liquefaction is predicted, residual strengths at the start of the earthquake. Detailed numerical simulations can be used to track the progressive softening and liquefaction of soil within an embankment during an earthquake; such analyses are expensive and time consuming. Rigorous analyses of this type will not be justified except in a "Phase C" analysis, or where performance in a given seismic design event must be demonstrated. Note that the logic in Figure 4 might appear to assume a slope will be stable if there is no significant liquefaction; however, the deformation analysis will indicate unlimited deformations and certain failure if $FS_{slope} < 1$ for static, post-earthquake conditions.

Step 7 – Consider Other Potential Failure Modes

For most of the closed facilities, soil liquefaction, slope instability, and slope deformations will be the most likely seismic failure modes. However, depending on the unique configuration of each CCP facility, other potential failure modes may contribute significantly to the seismic risks. For example, the loss of critical drainage structures or retaining walls could lead to a failure condition. Other potential failure modes will be identified and evaluated quantitatively in this step.

As a secondary objective of the Phase A effort, the assessment team will consider the potential for failure of the active storage facilities, due to an earthquake occurring prior to closure. Many of the wet CCP storage facilities retain large pools of water, so this assessment will need to consider additional failure modes such as seepage and embankment cracking. The objective here will be to identify operating facilities that may have unusually high seismic risks, and might deserve more study or higher priority in the closure program.

Step 8 – Estimate Annual Probability of Seismic Failure

As indicated in the flowcharts in Figures 3 and 4, the assessments of seismic performance (in both the Phase A and Phase B efforts) will consider a range of potential earthquakes with differing return periods. The analyses will be repeated until the limiting (lowest) earthquake return period (from the candidate events defined in Step 1) that predicts failure of a particular CCP storage facility is obtained. Interpolation may be used, as appropriate, to narrow the definition of the limiting earthquake.

The return period for each earthquake scenario (Table 4) represents the annual probability of exceedance for the associated ground motion parameter. Hence, for each earthquake scenario, the event with the smallest return period that causes failure represents a limiting case, where all events having longer return periods would also cause failure. The inverse of the limiting return period thus represents the annual probability of seismic failure due to that earthquake scenario.

Step 9 – Estimate Potential Consequences of Failure

The potential consequences of a failure at each closed facility will be estimated in this step. The potential consequences will be unique to each site, but may include any of the following:

- restoration of the site and storage facility,
- clean-up to address environmental impacts,
- off-site disposal of released materials,
- damages and loss of use for transportation routes, including buried or overhead utilities,
- damages to buildings and other infrastructure,
- economic losses from the possible shutdown of power generation, and
- loss of human life (expected to be unlikely at most sites following closure).

Except for the potential loss of life, the failure consequences will be expressed in terms of present day costs. Detailed cost estimates of the potential consequences of failure will not be attempted in the Phase A assessments; instead, the potential magnitude of total consequence costs will be estimated using broad categories (< $100K, < $500K, < $1M, < $5M, < $10M, < $50M, < $100M). Cost estimates that better reflect the local site conditions will be produced by the closure design teams during the Phase B assessments.

Step 10 – Estimate Possible Mitigation Costs

The final step in the process will involve estimating the costs to mitigate seismic risks, perhaps by altering the closure design to withstand stronger earthquakes. Examples of possible mitigation measures include:

- ground improvements to reduce liquefaction potential (stone columns, deep soil mixing, jet grouting, or other appropriate technology),
- altering the geometry of outslopes (setbacks, benches, or flatter slopes) to improve

stability,

- adding buttresses or other supporting structures at the toe of slopes,
- enhanced drainage features, and
- relocation of infrastructure or people away from potential impact zones.

These mitigation approaches generally involve higher construction costs, which can be quantified in terms of present dollars. As with the consequence costs, detailed estimates of mitigation costs will not be attempted in the Phase A assessments. The potential magnitude of mitigation will be estimated in categories (< $100K, < $500K, < $1M, < $5M, < $10M, < $50M, < $100M). Mitigation cost estimates that better reflect the local conditions and facility layout will be developed by the closure design teams during the Phase B assessments.

Table 1. Expected Results from the Phase A and B Seismic Risk Assessments

TVA Facility	Prob. Failure	Econ. Costs	Loss of Life	Mitigat. Costs	Data Quality
ALF East Ash Disposal					
ALF East Stilling Pond					
BRF Dry Fly Ash Disposal					
BRF Fly Ash Pond And Stilling Basin Area 2					
BRF Bottom Ash Disposal Area 1					
BRF Gypsum Disposal Area 2a					
COF Disposal Area 5					
COF Ash Pond 4					
CUF Dry Ash Stack					
CUF Ash Pond					
CUF Gypsum Storage Area					
GAF Fly Ash Pond E					
GAF Bottom Ash Pond A					
GAF Stilling Pond B, C & D					
JSF Dry Fly Ash Stack					
JSF Bottom Ash Disposal Area 2					
JOF Ash Disposal Area 2					
KIF Dike C					
PAF Scrubber Sludge Complex					
PAF Peabody Ash Pond					
PAF Slag Areas 2a & 2b					
SHF Consolidated Waste Dry Stack					
SHF Ash Pond					
WCF Ash Pond Complex					
WCF Gypsum Stack					

Prob Failure = Annual probability of failure due to earthquakes
Econ. Costs = Economic costs resulting from a failure
Loss of Life = Potential loss of life resulting from a failure
Mitigat. Costs = Costs to mitigate seismic risks in closure design
Data Quality = Qualitative indication of how well conditions in the facility are characterized

Table 2. Risk Severity Scoring (Draft) used by TVA

as of 4/22/2009

TVA Risk Event Consequence Rating Scale (Work-In-Progress)						
Strategic Objective	Success Factor	5 Worst Case	4 Severe	3 Major	2 Moderate	1 Minor
Customer	Public Image	International media attention; nearly unanimous public criticism	National media attention; federal, state officials, and customers publicly critical	Regional / local media attention; customers voice concern	Minimal media attention; letters / emails to executive leadership voicing concern	No media attention; sparse criticism
	Rate Impact	Average total retail rate increases by 15%, relative to peers	Average total retail rate increases by 10%-15%, relative to peers	Average total retail rate increases by 5%-10%, relative to peers	Average total retail rate increases by 2%-5%, relative to peers	Average total retail rate increases by 0-2%, relative to peers
People	Safety	Fatalities	Wide spread injuries	Major injuries	Significant injuries	Minor injuries
	Employee Confidence	Widespread departures of key staff with scarce skills or knowledge	Sharp, sustained drop in CHI results; departures of key staff with scarce skills or knowledge	Sharp decline in CHI results	Modest decline in CHI results	No effect on CHI results
Financial	Cash Flow Impact	>$500M	$100M - $500M	$25M - $100M	$5M - $25M	<$5M
	Credit Worthiness	Credit rating downgrade to below investment grade	Credit Rating Downgrade	TVA put on credit watch	TVA put on negative outlook	Credit rating agencies and bondholders express concern
Assets and Operations	LNS (Load not served)*	10% of System Daily Sales (48,000 MWhrs)	1% of System Daily Sales (4,800 MWhrs)	0.1% of System Daily Sales (480 MWhrs)	0.05% of System Daily Sales (240 MWhrs)	140 MWhrs
	CPI (Connection Point Interruptions)	10% of CPs are down simultaneously	5% of CPs are down simultaneously	CPI totaling 10% of current CP count (124 for FY09)	CPI totaling 7.5% of current CP count (93 for FY09)	CPI totaling 5% of current CP count (62 for FY09)
	Duration (in Hours) of Service Interruption	3,000 cumulative hours for CPs	1,000 cumulative hours for CPs	500 cumulative hours for CPs	150 cumulative hours for CPs	50 cumulative hours for CPs
	Delivered Cost of Power	Sustained increase in delivered cost of power >1 year	Increase in delivered cost of power <1 year	Increase in delivered cost of power <1 month	Increase in delivered cost of power <1 week	Delivered cost of power not effected
	Damage to environment; type and magnitude of contamination / discharge	Major coal, nuclear plant accident or dam failure	Significant hazardous waste discharged; nuclear plant accident; dam integrity failure resulting in drawdown of pool elevation	Hazardous materials / waste discharge; clean up / remediation time takes approximately two weeks	Localized environmental damage, no impact to wildlife, clean up / remediation time less than two weeks	Minimal environmental damage, no hazardous discharge; clean up time takes a few days

Table 3. Risk Likelihood Scoring used by TVA

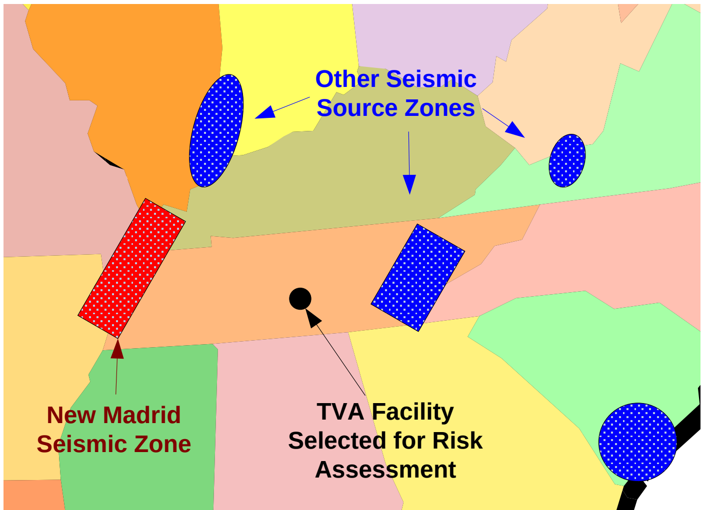
TVA Risk Event Probability Rating Scale		
Score	Rating	Description
5	Virtually Certain	95% probability that the event will occur in the next 3 years /10 years
4	Very Likely	75% probability that the event will occur in the next 3 years/10 years
3	Even Odds	50% probability that the event will occur in the next 3 years/10 years
2	Unlikely	25% probability that the event will occur in the next 3 years/10 years
1	Remote	5% probability that the event will occur in the next 3 years/10 years

- *The 3-year timeframe will be the primary focus for the business unit risk maps*
- *The 10-year risks will be collected by the ERM organization and charted separately for the enterprise*

Table 4. Seismic Hazard Input Data for Probabilistic Assessment of TVA Facilities

Seismic Sources	Return Period (years)	Annual Probability of Exceedance	Peak Ground Acceleration (g)	Earthquake Magnitude
New Madrid Seismic Zone	2,500 1,000 500 250 100	0.0004 0.001 0.002 0.004 0.01	*Values to be determined from the seismic hazard curves*	*Values to be determined from the hazard de-aggregation data**
All Other Seismic Sources	2,500 1,000 500 250 100	0.0004 0.001 0.002 0.004 0.01		

** Representative magnitude corresponding to the maximum contribution to the seismic hazard for liquefaction, as determined from the de-aggregation data weighted by the magnitude scaling factor (maximum PGA / MSF)*



Note: Schematic representation only, locations not accurately depicted, some sources omitted.

Figure 1. Schematic Representation of Seismic Source Model for TVA Facilities

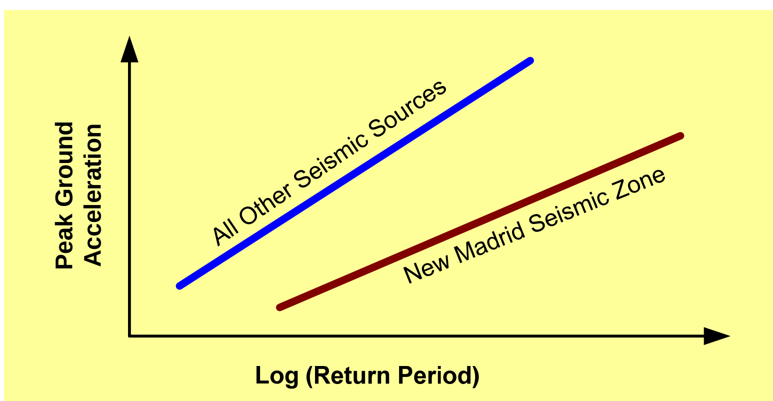


Figure 2. Typical Seismic Hazard Curves for Proposed Probabilistic Assessment of TVA Facilities

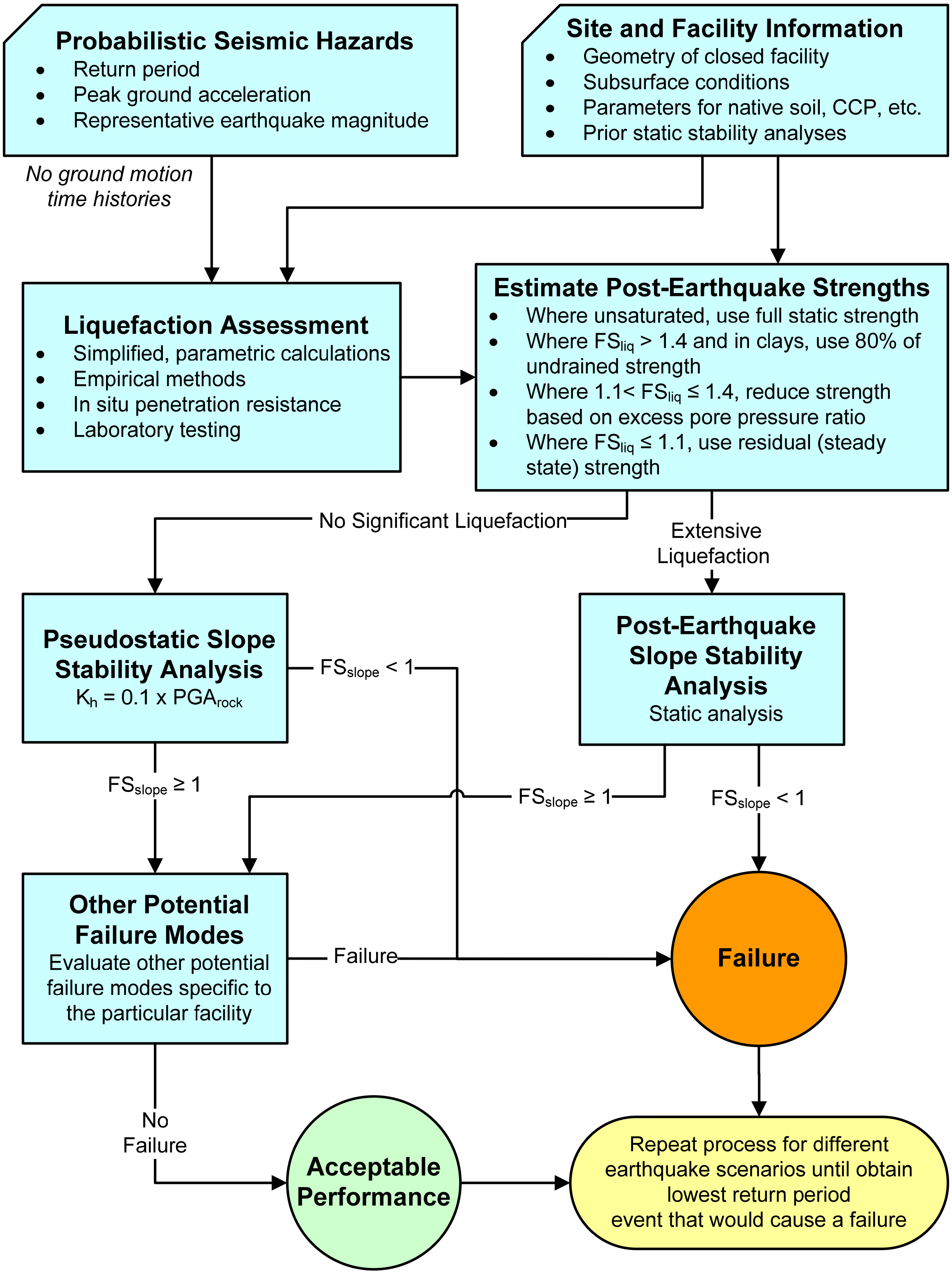


Figure 3. Simplified Flowchart for Assessing Facility Performance During a Probabilistic Seismic Event in Phase A

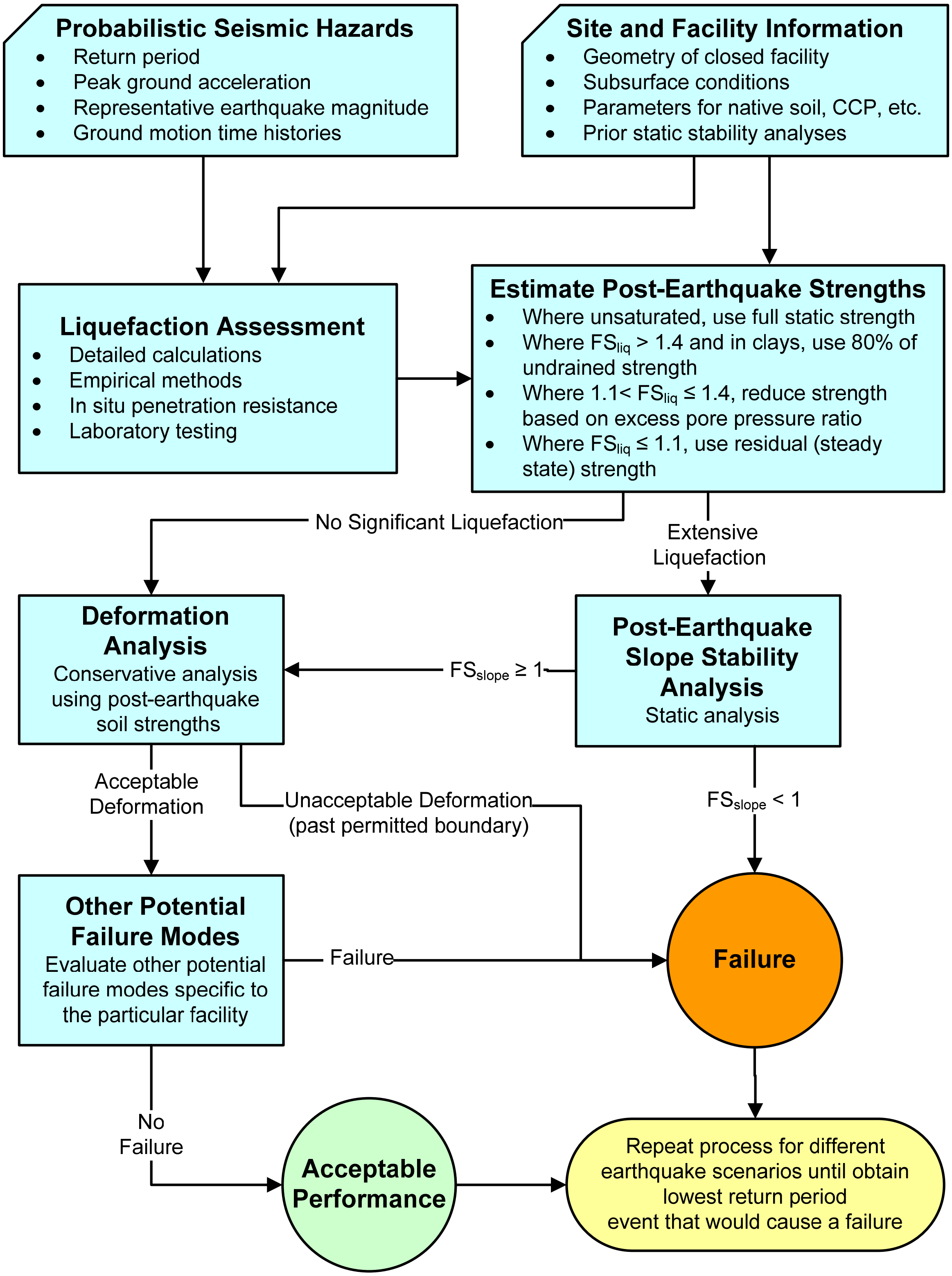


Figure 4. Simplified Flowchart for Assessing Facility Performance During a Probabilistic Seismic Event in Phase B

Enclosure B

Pseudostatic Analysis Results

Cumberland Fossil Plant - Pseudostatic Stability Analysis Summary

CCP Disposal Facility		Cross-Section Information		500 yr Return		FS = 1 Data		Mitigation and Improvement Activities Since January 2009 As-Found Conditions
Name	**Type**	**Section Analyzed**	**Section Location**	**PGA (g) for CUF**	**Factor of Safety**	**PGA (g)**	**Approx. Return Period (yrs)**	
Ash Pond	Impoundment	P	West side of dike along Wells Creek	0.083	1.2	N/A - FS ok for 500 yr Return		Mitigation activities are currently underway including rehabilitation of spillway system, addition of siphon system, addition of emergency spillway and lowering of permanent pool by six feet.
Dry Fly Ash Stack	Stack	F	Southwest corner of Stack along Wells Creek		1.0 (failure surface beneath perimeter dike)	N/A - FS ok for 500 yr Return		Slope at this section has been flattened. Currently, the stack is being regraded and surface ditches improved to enhance long term performance. Section F represents these conditions.
					0.8 (failure surface inside perimeter dike)	0.03	170	
Gypsum Stack Complex	Stack	H	Southwest corner of Stack along Wells Creek		1.0 (failure surface beneath perimeter dike)	N/A - FS ok for 500 yr Return		Toe buttress at Section H completed in December 2010. Currently, the stack is being regraded and surface ditches improved to enhance long term performance. Section H represents these conditions.
					0.8 (failure surface inside perimeter dike)	0.04	225	

Notes:

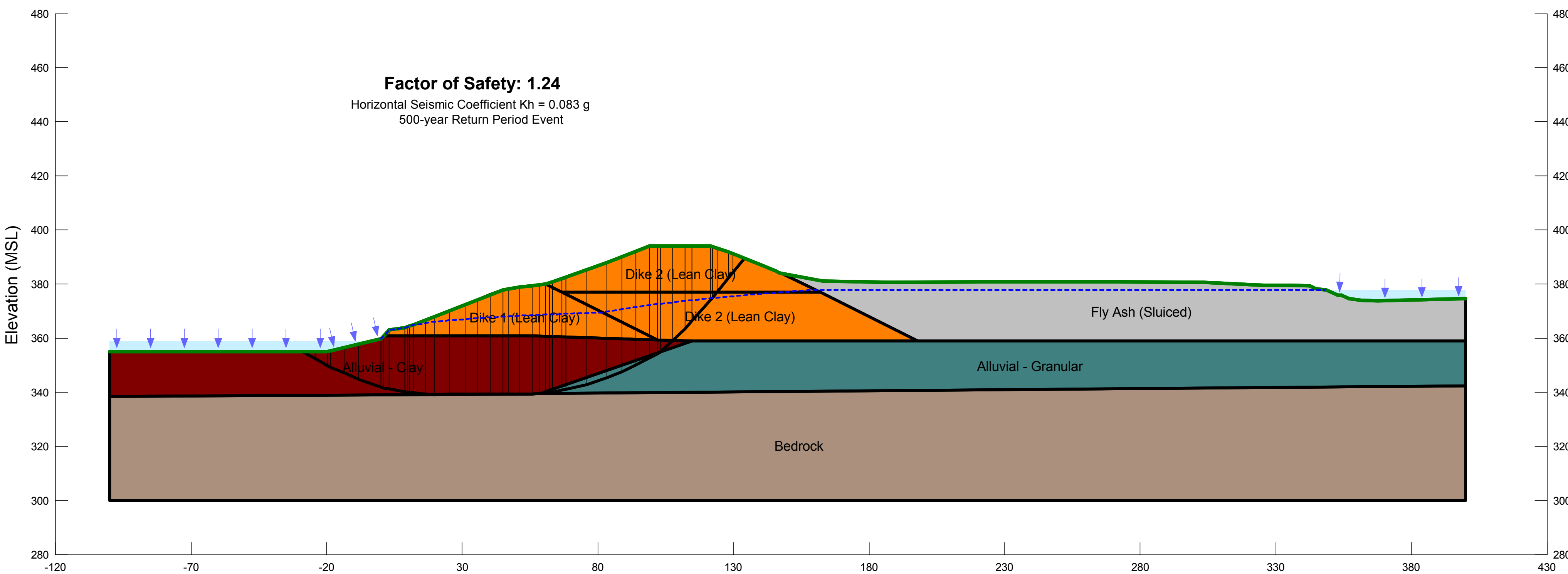
1) Accelerations are from March 28, 2011 TVA region-specific sesismic hazard study performed by AMEC Geomatrix, Inc. (total hazard).
2) Refer to layout plan for locations of cross-sections.
3) Stability models reflect current ground lines and recent improvements/mitigations using either construction drawings or as-built information, as appropriate.
4) Liquefaction was not considered in this analysis.

Pseudostatic Slope Stability Analysis
CCP Storage Facilities - Existing Conditions
Tennessee Valley Authority Fossil Plants

Section P - Ash Pond
Cumberland Fossil Plant
Cumberland City, Tennessee

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
Dike 1 (Lean Clay)	123 pcf	800 psf	20 °
Dike 2 (Lean Clay)	123 pcf	500 psf	21 °
Fly Ash (Sluiced)	100 pcf	140 psf	11 °
Alluvial - Clay	124 pcf	450 psf	20 °
Alluvial - Granular	130 pcf	100 psf	20 °
Bedrock			



Date of Assessment - 09/09/2011

Project No. 175551015

Pseudostatic Slope Stability Analysis
CCP Storage Facilities - Existing Conditions
Tennessee Valley Authority Fossil Plants

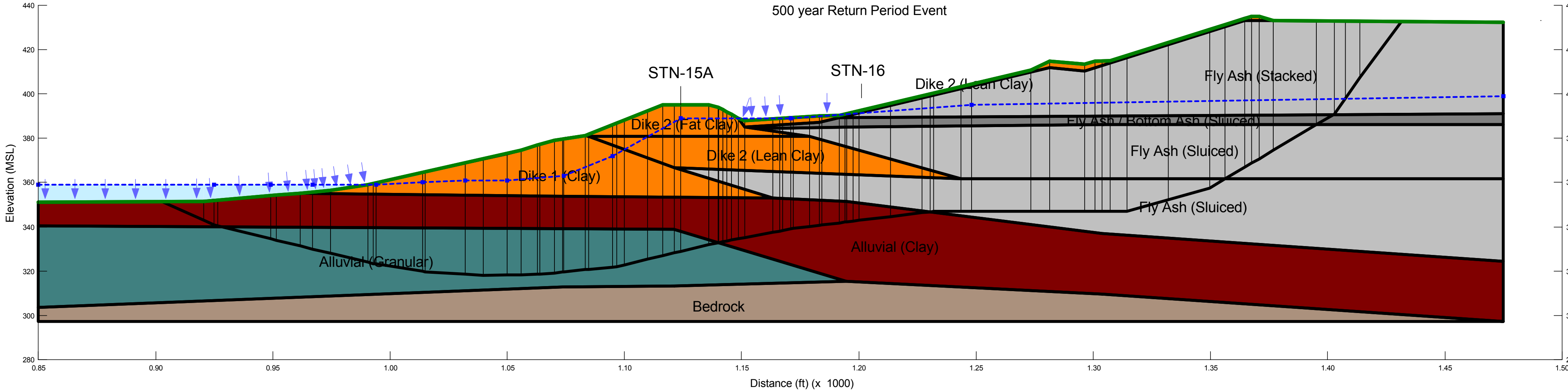
Seciton F - Dry Fly Ash Stack
Cumberland Fossil Plant
Cumberland City, Tennessee

Note:
The results of the 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
Dike 1 (Clay)	124 pcf	800 psf	20 °
Dike 2 (Lean Clay)	128 pcf	500 psf	21 °
Alluvial (Clay)	121 pcf	450 psf	20 °
Alluvial (Granular)	130 pcf	100 psf	20 °
Fly Ash (Stacked)	100 pcf	0 psf	32 °
Fly Ash (Sluiced)	100 pcf	140 psf	11 °
Fly Ash / Bottom Ash (Sluiced)	100 pcf	140 psf	11 °
Dike 2 (Fat Clay)	127 pcf	200 psf	18 °
Bedrock			

Factor of Safety: 0.99

Horizontal Seismic Coefficient Kh = 0.083 g
500 year Return Period Event



Date of Assessment - 09/09/2011

Project No. 175551015

Pseudostatic Slope Stability Analysis
CCP Storage Facilities - Existing Conditions
Tennessee Valley Authority Fossil Plants

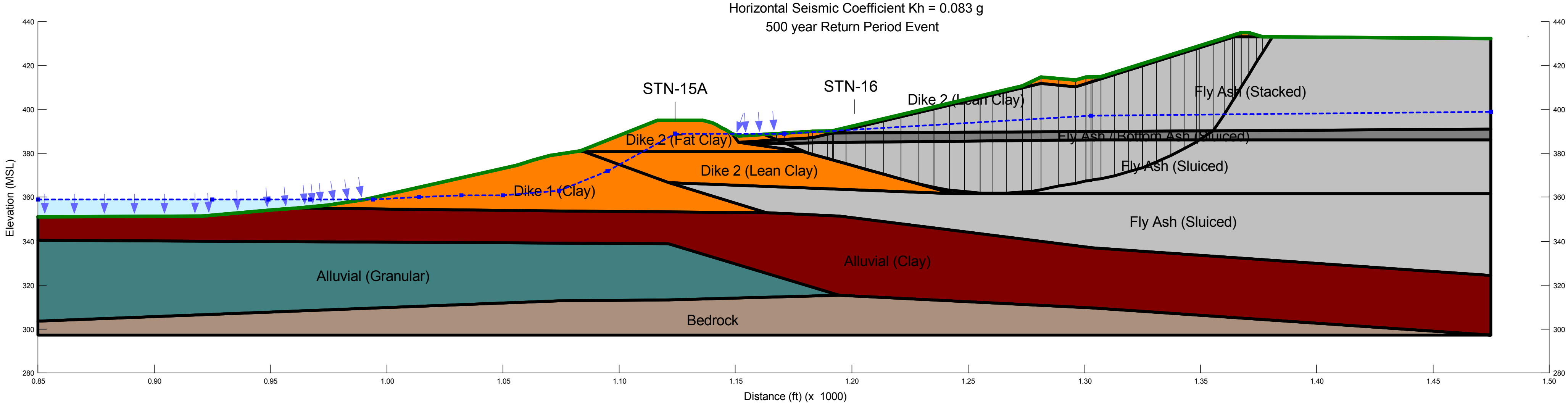
Seciton F - Dry Fly Ash Stack
Cumberland Fossil Plant
Cumberland City, Tennessee

Note:
The results of the 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
Dike 1 (Clay)	124 pcf	800 psf	20 °
Dike 2 (Lean Clay)	128 pcf	500 psf	21 °
Alluvial (Clay)	121 pcf	450 psf	20 °
Alluvial (Granular)	130 pcf	100 psf	20 °
Fly Ash (Stacked)	100 pcf	0 psf	32 °
Fly Ash (Sluiced)	100 pcf	140 psf	11 °
Fly Ash / Bottom Ash (Sluiced)	100 pcf	140 psf	11 °
Dike 2 (Fat Clay)	127 pcf	200 psf	18 °
Bedrock			

Factor of Safety: 0.81

Horizontal Seismic Coefficient Kh = 0.083 g
500 year Return Period Event



Date of Assessment - 09/09/2011

Project No. 175551015

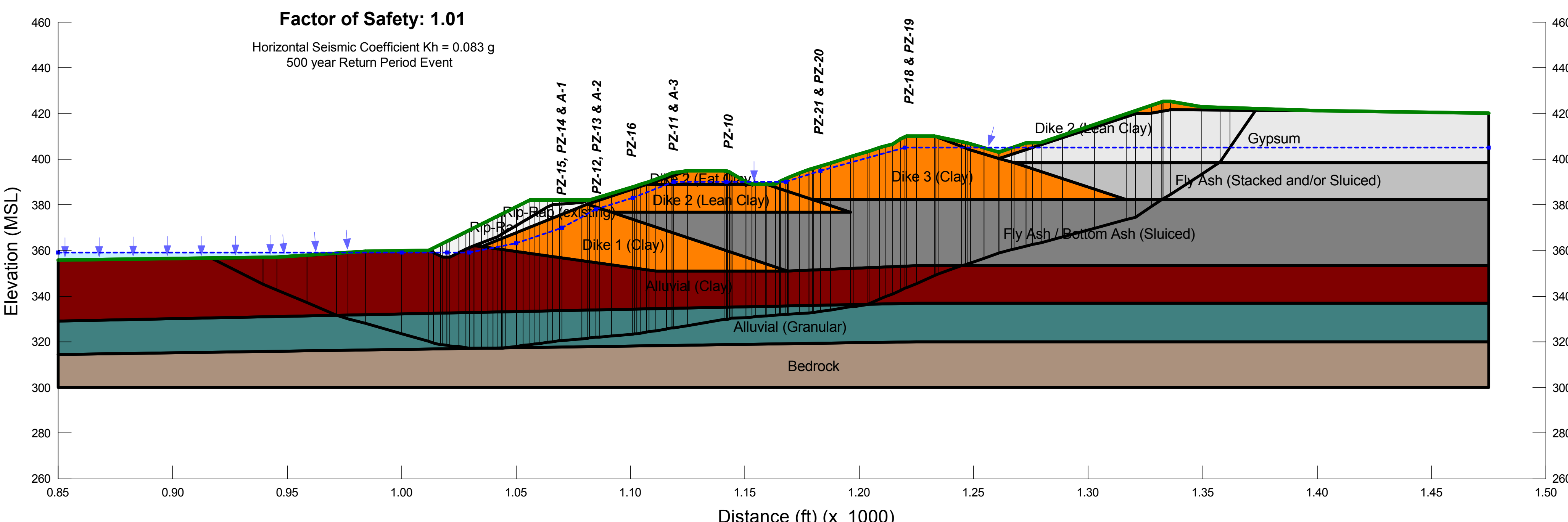
Pseudostatic Slope Stability Analysis
CCP Storage Facilities - Existing Conditions
Tennessee Valley Authority Fossil Plants

Section H - Gypsum Stack
Cumberland Fossil Plant
Cumberland City, Tennessee

Stantec

Note:
The results of the 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
Dike 1 (Clay)	124 pcf	800 psf	20 °
Dike 2 (Lean Clay)	128 pcf	500 psf	21 °
Dike 3 (Clay)	126 pcf	1000 psf	25 °
Alluvial (Clay)	121 pcf	450 psf	20 °
Alluvial (Granular)	130 pcf	100 psf	20 °
Gypsum	105 pcf	0 psf	33 °
Fly Ash (Stacked and/or Sluiced)	100 pcf	140 psf	11 °
Fly Ash / Bottom Ash (Sluiced)	100 pcf	140 psf	11 °
Dike 2 (Fat Clay)	127 pcf	200 psf	18 °
Rip-Rap	150 pcf	0 psf	38 °
Rip-Rap (existing)	150 pcf	0 psf	38 °



Date of Assessment - 09/09/2011

Project No. 175551015

Pseudostatic Slope Stability Analysis
CCP Storage Facilities - Existing Conditions
Tennessee Valley Authority Fossil Plants

Section H - Gypsum Stack
Cumberland Fossil Plant
Cumberland City, Tennessee

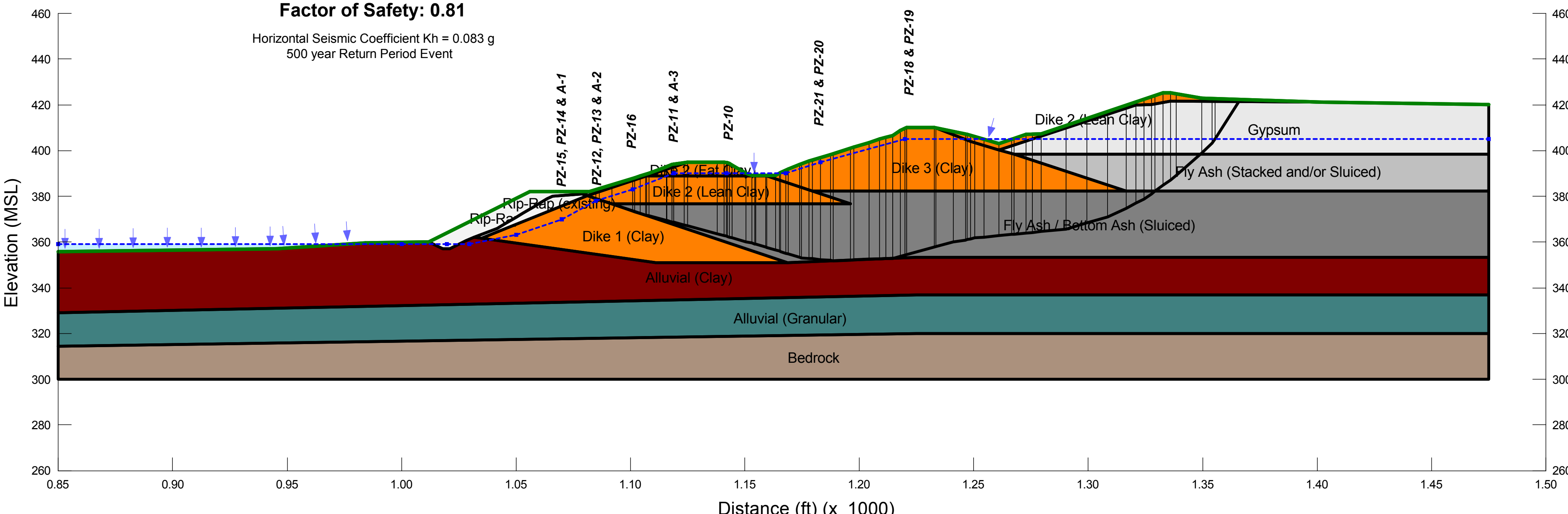
Stantec

Note:
The results of the 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
Dike 1 (Clay)	124 pcf	800 psf	20 °
Dike 2 (Lean Clay)	128 pcf	500 psf	21 °
Dike 3 (Clay)	126 pcf	1000 psf	25 °
Alluvial (Clay)	121 pcf	450 psf	20 °
Alluvial (Granular)	130 pcf	100 psf	20 °
Gypsum	105 pcf	0 psf	33 °
Fly Ash (Stacked and/or Sluiced)	100 pcf	140 psf	11 °
Fly Ash / Bottom Ash (Sluiced)	100 pcf	140 psf	11 °
Dike 2 (Fat Clay)	127 pcf	200 psf	18 °
Rip-Rap	150 pcf	0 psf	38 °
Rip-Rap (existing)	150 pcf	0 psf	38 °

Factor of Safety: 0.81

Horizontal Seismic Coefficient Kh = 0.083 g
500 year Return Period Event



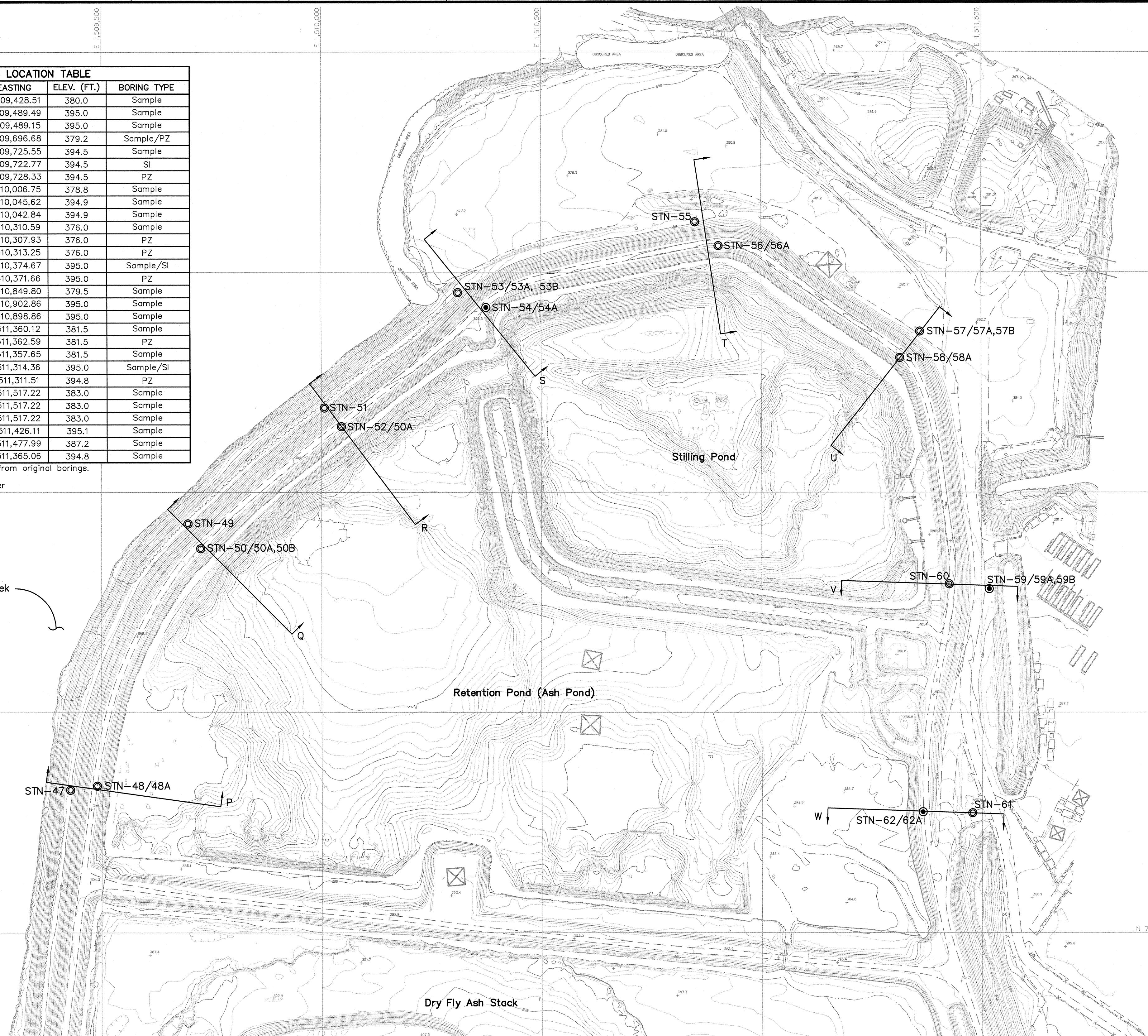
Date of Assessment - 09/09/2011

Project No. 175551015

46	C	10W544-01

BORING LOCATION TABLE				
BORING	NORTHING	EASTING	ELEV. (FT.)	BORING TYPE
STN-47	732,324.14	1,509,428.51	380.0	Sample
STN-48	732,333.24	1,509,489.49	395.0	Sample
*STN-48A	732,329.25	1,509,489.15	395.0	Sample
STN-49	732,928.84	1,509,696.68	379.2	Sample/PZ
STN-50	732,872.44	1,509,725.55	394.5	Sample
*STN-50A	732,869.56	1,509,722.77	394.5	SI
*STN-50B	732,875.32	1,509,728.33	394.5	PZ
STN-51	733,191.78	1,510,006.75	378.8	Sample
STN-52	733,149.40	1,510,045.62	394.9	Sample
*STN-52A	733,146.52	1,510,042.84	394.9	Sample
STN-53	733,453.67	1,510,310.59	376.0	Sample
*STN-53A	733,456.66	1,510,307.93	376.0	PZ
*STN-53B	733,450.68	1,510,313.25	376.0	PZ
STN-54	733,419.93	1,510,374.67	395.0	Sample/SI
*STN-54A	733,417.30	1,510,371.66	395.0	PZ
STN-55	733,614.54	1,510,849.80	379.5	Sample
STN-56	733,560.12	1,510,902.86	395.0	Sample
*STN-56A	733,560.12	1,510,898.86	395.0	Sample
STN-57	733,365.74	1,511,360.12	381.5	Sample
*STN-57A	733,368.89	1,511,362.59	381.5	PZ
*STN-57B	733,362.59	1,511,357.65	381.5	Sample
STN-58	733,305.89	1,511,314.36	395.0	Sample/SI
*STN-58A	733,308.70	1,511,311.51	394.8	PZ
STN-59	732,780.76	1,511,517.22	383.0	Sample
*STN-59A	732,784.76	1,511,517.22	383.0	Sample
STN-60	732,776.76	1,511,517.22	383.0	Sample
STN-61	732,791.74	1,511,426.11	395.1	Sample
STN-62	732,271.84	1,511,477.99	387.2	Sample
*STN-62A	732,274.04	1,511,365.06	394.8	Sample

*Estimated based on offsets from original borings.
"PZ" denotes Piezometer
"SI" denotes Slope Inclinometer



BORING LAYOUT
SCALE: 1"=100'

GRAPHIC SCALE: 1" = 100'
CONTOUR INTERVAL = 1'

LEGEND

- Soil Boring with Undisturbed (Shelby) Tube Samples and/or Standard Penetration Tests
- Soil Boring with Undisturbed (Shelby) Tube Samples and/or Standard Penetration Tests and Rock Core

NOTE:

The topographic mapping provided is based on horizontal datum NAD27 and vertical datum NGV29 using State Plane Tennessee coordinate system. The site photography was performed on 4/17/2009.

RECORD DRAWING

For Supporting Design Calculations see FPGCUFFESCDX00000020100002

STAN A. HARRIS
REGISTERED ENGINEER
AGRICULTURE
COMMERCE
No. 00113308
STATE OF TENNESSEE
3/25/10

REV. NO.	DATE	DSGN	DRWN	CHKD	SUPV	RVWD	APPD	ISSD	PROJECT ID	AS CONST	REV CD
R –	–	–	–	–	–	–	–	–	–	–	–
R 0	03/29/10	DBR	CW	DBR	SAH	SAH	SAH	TJ	–	–	–
RECORD DRAWING											

DISCIPLINE INTERFACE: 1, 2, 3, 4

SCALE: 1"=100' EXCEPT AS NOTED

YARD
RETENTION AND STILLING PONDS

GEOTECHNICAL EXPLORATION
BORING LAYOUT

DESIGNED BY:	DRAWN BY:	CHECKED BY:	SUPERVISED BY:	REVIEWED BY:	APPROVED BY:	ISSUED BY:
D. ROGERS	C. WITHERS	D. ROGERS	S. HARRIS	S. HARRIS	S. HARRIS	T. JOHNSON

Stantec
Stantec Consulting Services Inc.
11687 Lebanon Rd.
Cincinnati, Ohio
45241-2012
Tel. 513.842.8200
Fax 513.842.8250
www.stantec.com

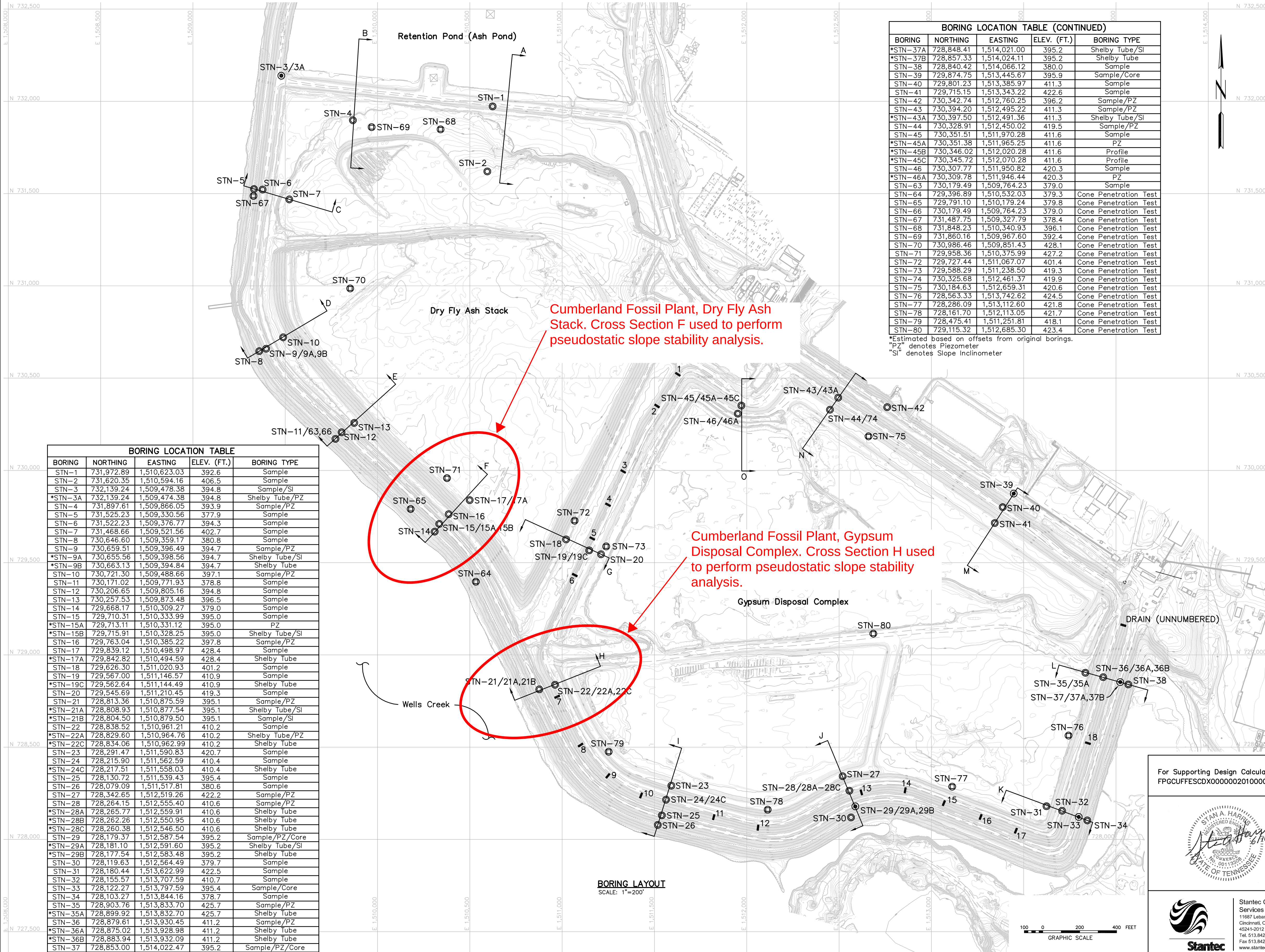
CUMBERLAND FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

AUTOCAD R 2000	DATE 03/29/10	46	C	10W544-01	R 0

STANTEC	0
TASK COMPLETED BY:	REV NO.

PLOT FACTOR:XX
W_TVA

C.A.D. DRAWING
DO NOT ALTER MANUALLY



BORING LAYOUT
SCALE: 1"=200'

GRAPHIC SCALE 100 0 200 400 FEET

BORING LOCATION TABLE

BORING	NORTHING	EASTING	ELEV. (FT.)	BORING TYPE
STN-1	731,972.89	1,510,623.03	392.6	Sample
STN-2	731,620.35	1,510,594.16	406.5	Sample
STN-3	732,139.24	1,509,478.38	394.8	Sample/SI
*STN-3A	732,139.24	1,509,474.38	394.8	Shelby Tube/PZ
STN-4	731,897.61	1,509,866.05	393.9	Sample/PZ
STN-5	731,525.23	1,509,330.56	377.9	Sample
STN-6	731,522.23	1,509,376.77	394.3	Sample
STN-7	731,468.66	1,509,521.56	402.7	Sample
STN-8	730,646.60	1,509,359.17	380.8	Sample
STN-9	730,659.51	1,509,396.49	394.7	Sample/PZ
*STN-9A	730,655.56	1,509,398.56	394.7	Shelby Tube/SI
*STN-9B	730,663.13	1,509,394.84	394.7	Shelby Tube
STN-10	730,721.30	1,509,488.66	397.1	Sample/PZ
STN-11	730,171.02	1,509,771.93	378.8	Sample
STN-12	730,206.65	1,509,805.16	394.8	Sample
STN-13	730,257.53	1,509,873.48	396.5	Sample
STN-14	729,668.17	1,510,309.27	379.0	Sample
STN-15	729,710.31	1,510,333.99	395.0	Sample
*STN-15A	729,713.11	1,510,331.12	395.0	PZ
*STN-15B	729,715.91	1,510,328.25	395.0	Shelby Tube/SI
STN-16	729,763.04	1,510,385.22	397.8	Sample/PZ
STN-17	729,839.12	1,510,498.97	428.4	Sample
*STN-17A	729,842.82	1,510,494.59	428.4	Shelby Tube
STN-18	729,626.30	1,511,020.93	401.2	Sample
STN-19	729,567.00	1,511,146.57	410.9	Sample
*STN-19C	729,562.64	1,511,144.49	410.9	Shelby Tube
STN-20	729,545.69	1,511,210.45	419.3	Sample
STN-21	728,813.36	1,510,875.59	395.1	Sample/PZ
*STN-21A	728,808.93	1,510,877.54	395.1	Shelby Tube/SI
*STN-21B	728,804.50	1,510,879.50	395.1	Sample/SI
STN-22	728,838.52	1,510,961.21	410.2	Sample
*STN-22A	728,829.60	1,510,964.76	410.2	Shelby Tube/PZ
*STN-22C	728,834.06	1,510,962.99	410.2	Shelby Tube
STN-23	728,291.47	1,511,590.83	420.7	Sample
STN-24	728,215.90	1,511,562.59	410.4	Sample
*STN-24C	728,217.51	1,511,558.03	410.4	Shelby Tube
STN-25	728,130.72	1,511,539.43	395.4	Sample
STN-26	728,079.09	1,511,517.81	380.6	Sample
STN-27	728,342.65	1,512,519.26	422.2	Sample/PZ
STN-28	728,264.15	1,512,555.40	410.6	Sample/PZ
*STN-28A	728,265.77	1,512,559.91	410.6	Shelby Tube
*STN-28B	728,262.26	1,512,550.95	410.6	Shelby Tube
*STN-28C	728,260.38	1,512,546.50	410.6	Shelby Tube
STN-29	728,179.37	1,512,587.54	395.2	Sample/PZ/Core
*STN-29A	728,181.10	1,512,591.60	395.2	Shelby Tube/SI
*STN-29B	728,177.54	1,512,583.48	395.2	Shelby Tube
STN-30	728,119.63	1,512,564.49	379.7	Sample
STN-31	728,180.44	1,513,622.99	422.5	Sample
STN-32	728,155.57	1,513,707.59	410.7	Sample
STN-33	728,122.27	1,513,797.59	395.4	Sample/Core
STN-34	728,103.27	1,513,844.16	378.7	Sample
STN-35	728,903.76	1,513,833.70	425.7	Sample/PZ
*STN-35A	728,899.92	1,513,832.70	425.7	Shelby Tube
STN-36	728,879.61	1,513,930.45	411.2	Sample/PZ
*STN-36A	728,875.02	1,513,928.98	411.2	Shelby Tube
*STN-36B	728,883.94	1,513,932.09	411.2	Shelby Tube
STN-37	728,853.00	1,514,022.47	395.2	Sample/PZ/Core

BORING LOCATION TABLE (CONTINUED)

BORING	NORTHING	EASTING	ELEV. (FT.)	BORING TYPE
*STN-37A	728,848.41	1,514,021.00	395.2	Shelby Tube/SI
*STN-37B	728,857.33	1,514,024.11	395.2	Shelby Tube
STN-38	728,840.42	1,514,066.12	380.0	Sample
STN-39	729,874.75	1,513,445.67	395.9	Sample/Core
STN-40	729,801.23	1,513,385.97	411.3	Sample
STN-41	729,715.15	1,513,343.22	422.6	Sample
STN-42	730,342.74	1,512,760.25	396.2	Sample/PZ
STN-43	730,394.20	1,512,495.22	411.3	Sample/PZ
*STN-43A	730,397.50	1,512,491.36	411.3	Shelby Tube/SI
STN-44	730,328.91	1,512,450.02	419.5	Sample/PZ
STN-45	730,351.51	1,511,970.28	411.6	Sample
*STN-45A	730,351.38	1,511,965.25	411.6	PZ
*STN-45B	730,346.02	1,512,020.28	411.6	Profile
*STN-45C	730,345.72	1,512,070.28	411.6	Profile
STN-46	730,307.77	1,511,950.82	420.3	Sample
*STN-46A	730,309.78	1,511,946.44	420.3	PZ
STN-63	730,179.49	1,509,764.23	379.0	Sample
STN-64	729,396.89	1,510,532.03	379.3	Cone Penetration Test
STN-65	729,791.10	1,510,179.24	379.8	Cone Penetration Test
STN-66	730,179.49	1,509,764.23	379.0	Cone Penetration Test
STN-67	731,487.75	1,509,327.79	378.4	Cone Penetration Test
STN-68	731,848.23	1,510,340.93	396.1	Cone Penetration Test
STN-69	731,860.16	1,509,967.60	392.4	Cone Penetration Test
STN-70	730,986.46	1,509,851.43	428.1	Cone Penetration Test
STN-71	729,958.36	1,510,375.99	427.2	Cone Penetration Test
STN-72	729,727.44	1,511,067.07	401.4	Cone Penetration Test
STN-73	729,588.29	1,511,238.50	419.3	Cone Penetration Test
STN-74	730,325.68	1,512,461.37	419.9	Cone Penetration Test
STN-75	730,184.63	1,512,659.31	420.6	Cone Penetration Test
STN-76	728,563.33	1,513,742.62	424.5	Cone Penetration Test
STN-77	728,286.09	1,513,112.60	421.8	Cone Penetration Test
STN-78	728,161.70	1,512,113.05	421.7	Cone Penetration Test
STN-79	728,475.41	1,511,251.81	418.1	Cone Penetration Test
STN-80	729,115.32	1,512,685.30	423.4	Cone Penetration Test

*Estimated based on offsets from original borings.
"PZ" denotes Piezometer
"SI" denotes Slope Inclinometer

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

NOTE:

The topographic mapping provided is based on horizontal datum NAD27 and vertical datum NGV29 using State Plane Tennessee coordinate system. The site photography was performed on 4/17/2009.

LEGEND

- Soil Boring with Undisturbed (Shelby) Tube Samples and/or Standard Penetration Tests
- Soil Boring with Undisturbed (Shelby) Tube Samples and/or Standard Penetration Tests and Rock Core
- Cone Penetration Test
- Drain, Number

RECORD DRAWING

For Supporting Design Calculations see FPGCUFFESCDX00000020100001

REV. NO.	DATE	DSGN	DRWN	CHKD	SUPV	RVWD	APPD	ISSD	PROJECT ID	AS CONST	REV CD
R –	–	–	–	–	–	–	–	–	–	–	–
R 0	06/11/10	DBR	CW	DBR	SAH	SAH	SAH	TJ	–	–	–

RECORD DRAWING

DISCIPLINE INTERFACE 1 2 3 4

SCALE: 1"=200' EXCEPT AS NOTED

YARD
DRY FLY ASH STACK AND GYPSUM DISPOSAL COMPLEX

GEOTECHNICAL EXPLORATION
BORING LAYOUT

STAN A. HARRIS REGISTERED ENGINEER AGRICULTURE COMMERCE No. 00113308 STATE OF TENNESSEE 6/11/10

DESIGNED BY:	DRAWN BY:	CHECKED BY:	SUPERVISED BY:	REVIEWED BY:	APPROVED BY:	ISSUED BY:
D. ROGERS	C. WITHERS	D. ROGERS	S. HARRIS	S. HARRIS	S. HARRIS	T. JOHNSON

Stantec Consulting Services Inc.
11687 Lebanon Rd.
Cincinnati, Ohio
45241-2012
Tel. 513.842.8200
Fax 513.842.8250
www.stantec.com

CUMBERLAND FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

AUTOCAD R 2000 | DATE 06/11/10 | 46 | C | 10W543-01 | R 0

STANTEC | 0
TASK COMPLETED BY: | REV NO.

PLOT FACTOR:XX
W_TVA

C.A.D. DRAWING DO NOT ALTER MANUALLY

PLOT DATE: 06/08/2010