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Coal Combustion Residue Impoundment

Round 11 - Dam Assessment Report

Cumberland Fossil Plant

Dry Ash Storage / Ash Pond
TVA
Cumberland City, Tennessee

Prepared for:

United States Environmental Protection Agency
Office of Resource Conservation and Recovery

Prepared by:

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INTRODUCTION, SUMMARY CONCLUSIONS AND RECOMMENDATIONS

The release of over five million cubic yards of coal combustion residue from the Tennessee Valley Authority (TVA) Kingston, Tennessee facility in December 2008, which flooded more than 300 acres of land, damaging homes and property, is a wake-up call for diligence on coal combustion residue disposal units. We must marshal our best efforts to prevent such catastrophic failure and damage. A first step toward this goal is to assess the stability and functionality of the ash impoundments and other units, then quickly take any needed corrective measures.

This assessment of the stability and functionality of the Cumberland Fossil Plant coal combustion residue (CCR) management facilities is based on a review of available documents and on the site assessment conducted by Dewberry personnel on September 7, 2011. We found the supporting technical documentation to be adequate (Section 1.1.3). As detailed in Section 1.2, there are minor recommendations based on field observations and documentation reviews that may help to maintain a safe and trouble-free operation.

The original power plant's ash pond has been modified over time, due to operational changes in the plant. The current configuration includes the Ash Pond, which is used to settle out remaining bottom ash and serves as a storm water detention basin for the storm water runoff for the Dry Fly Ash Stack and the Gypsum Disposal Area. The rest of the original pond was split into two dry storage areas - the Dry Fly Ash Stack and the Gypsum Disposal Area - and a small incised pond (Bottom Ash Pond) located at the north end of the divider dike between the Dry Fly Ash Stack and the Gypsum Disposal Area. The incised pond receives sluiced bottom ash directly from the plant and is used to capture the bulk of the bottom ash, which is excavated and processed into dry material. Since the small Bottom Ash Pond is incised, it was not separately assessed and not rated. The Ash Pond, Dry Fly Ash Stack, and the Gypsum Disposal Area were all three separately assessed and rated, since failure of their containment dikes could release significant amounts of CCR off site into Wells Creek and to the Cumberland River.

In summary, the Cumberland Ash Pond, Dry Fly Ash Stack, and Gypsum Disposal Area are each rated **SATISFACTORY.** Dewberry has upgraded the ratings for the three CCR management units based upon revised operating and seepage data for the Ash Pond (Appendix C, Doc 22), new seismic analyses of the Dry Fly Ash Stack (Appendix C, Doc 23), and liquefaction data for the Gypsum Disposal Area (Appendix C, Doc 22) . The results show that the Dry Fly Ash Stack Area has adequate safety for normal operation conditions for the design basis earthquake. Therefore, no seismic-related remedial measures are recommended. There are no other recognized existing or potential management unit safety deficiencies.

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PURPOSE AND SCOPE

The U.S. Environmental Protection Agency (EPA) is embarking on an initiative to investigate the potential for catastrophic failure of Coal Combustion Surface Impoundments (i.e., management unit) from occurring at electric utilities in an effort to protect lives and property from the consequences of a dam failure or the improper release of impounded slurry. The EPA initiative is intended to identify conditions that may adversely affect the structural stability and functionality of a management unit and its appurtenant structures (if present); to note the extent of deterioration (if present), status of maintenance and/or a need for immediate repair; to evaluate conformity with current design and construction practices; and to determine the hazard potential classification for units not currently classified by the management unit owner or by a state or federal agency. The initiative will address management units that are classified as having a Less-than-Low, Low, Significant or High Hazard Potential ranking. (For Classification, see pp. 3-8 of the 2004 Federal Guidelines for Dam Safety)

In February 2009, the EPA sent letters to coal-fired electric utilities seeking information on the safety of surface impoundments and similar facilities that receive liquid-borne material that store or dispose of coal combustion residue. This letter was issued under the authority of the Comprehensive Environmental Response, Compensation, and Liability Act (CERCLA) Section 104(e), to assist the Agency in assessing the structural stability and functionality of such management units, including which facilities should be visited to perform a safety assessment of the berms, dikes, and dams used in the construction of these impoundments.

EPA requested that utility companies identify all management units including surface impoundments or similar diked or bermed management units or management units designated as landfills that receive liquid-borne material used for the storage or disposal of residuals or by-products from the combustion of coal, including, but not limited to, fly ash, bottom ash, boiler slag, or flue gas emission control residuals. Utility companies provided information on the size, design, age and the amount of material placed in the units. The EPA used the information received from the utilities to determine preliminarily which management units had or potentially could have High Hazard Potential ranking.

The purpose of this report is **to evaluate the condition and potential of residue release from management units and to determine the hazard potential classification**. This evaluation included a site visit. Prior to conducting the site visit, a two-person team reviewed the information submitted to EPA, reviewed any relevant publicly available information from state or federal agencies regarding the unit hazard potential classification (if any) and accepted information provided via telephone communication with the management unit owner. Also, after the field visit, additional information was received by Dewberry & Davis LLC about the Cumberland CCR management units that was reviewed and used in the preparation of this report.

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Factors considered in determining the hazard potential classification of the management units(s) included the age and size of the impoundment, the quantity of coal combustion residuals or by-products that were stored or disposed of in these impoundments, its past operating history, and its geographic location relative to down gradient population centers and/or sensitive environmental systems.

This report presents the opinion of the assessment team as to the potential of catastrophic failure and reports on the condition of the management unit(s).

Note: The terms "embankment", "berm", "dike" and "dam" are used interchangeably within this report, as are the terms "pond", "basin", and "impoundment".

LIMITATIONS

The assessment of dam safety reported herein is based on field observations and review of readily available information provided by the owner/operator of the subject coal combustion residue management unit(s). Qualified Dewberry engineering personnel performed the field observations and review and made the assessment in conformance with the required scope of work and in accordance with reasonable and acceptable engineering practices. No other warranty, either written or implied, is made with regard to our assessment of dam safety.

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1.0 CONCLUSIONS AND RECOMMENDATIONS

1.1 CONCLUSIONS

Conclusions are based on visual observations from a one-day site visit on September 7, 2011, and review of technical documentation provided by the Tennessee Valley Authority (TVA).

1.1.1 Conclusions Regarding the Structural Soundness of the Management Unit(s)

The dikes containing the Dry Fly Ash Stack, Ash Pond, and Gypsum Disposal Area appear visually to be satisfactory. The dikes have documented acceptable factors of safety under static loading conditions. However, Dewberry's original evaluation considered the overall structural stability of the Ash Pond to be Fair and Gypsum Disposal Area containment dikes and outlet works to be Poor. There was an issue that the pseudostatic factor of safety was not calculated for the correct seismic return period, there was concern about a potential piping failure in the Ash Pond, and there was no liquefaction data for either management unit. In the Draft report Dewberry recommended seepage studies be performed for the Ash Pond, and liquefaction analyses for all three units, but particularly for the Gypsum Disposal Area.

At the time of the site visit, the furnished documentation of pseudostatic[1] stability analyses of the critical sections of the Dry Fly Ash Stack and the Gypsum Disposal Area containment dikes under the 500-year seismic event yielded FS = 1.0, the acceptance criterion. Thus, it appeared by inspection that for the stronger, 2,500-year seismic event required by the USEPA, a FS < 1.0 would result. Subsequent to Dewberry's Draft report being issued TVA provided additional information concerning operating practices, seepage analyses and liquefaction potential assessment (See Doc 22 and Doc 23 Appendix C). The results of the liquefaction and post-earthquake stability analyses under the new practices indicate that the Gypsum Disposal Area will remain stable and display adequate performance following the 2,500-year earthquake. TVA's consultant (Stantec) also provided additional documentation for seepage analyses and

liquefaction potential assessment for the Ash Pond (See Doc 22, Appendix C) and Dry Fly Ash Stack.[1]

The rate of filling of CCR is critical to stability and safe operation, particularly of the Dry Ash Stack, and should be controlled as recommended in Subsection 1.2.1, Paragraph 2).

Based on the additional information the containment dikes for all three management units are considered **Satisfactory**.

1.1.2 Conclusions Regarding the Hydrologic/Hydraulic Safety of the Management Unit(s)

On the basis of furnished hydrologic/hydraulic documentation, the Ash Pond (CCR Complex) currently meets accepted standards for hydrologic/hydraulic safety.

1.1.3 Conclusions Regarding the Adequacy of Supporting Technical Documentation

The documentation of hydrologic/hydraulic analyses for the Ash Pond (CCR Complex) appears overall to be adequate. Documentation of static slope stability, seepage analysis, and piping potential (where appropriate) of the CCR Complex containment dikes is adequate. The documentation of performance of the Ash Pond containment dike under seismic loading is adequate. The original documentation for the Dry Ash Stack and the Gypsum Disposal Area containment dikes under seismic loading was inadequate; however, subsequent to Dewberry's Draft report being issued TVA provided additional information that demonstrates acceptable safety factors under the design seismic event required by the USEPA.

[1] The pseudostatic method is a simplified method for determining seismic slope stability that is based on the same approach (i.e., limit equilibrium) used in analyzing static slope stability. In current practice, the pseudostatic method of analysis is used primarily as a screening tool to help assess whether an embankment dam or slope requires a more detailed seismic slope analysis. The pseudostatic method ignores cyclic loading of the earthquake, but accounts for the seismic force by applying an equivalent static force on the slope. In the limit equilibrium approach the stress-strain relationship of the soil is not considered, so the method should not be used for sensitive clays and other materials that lose shear strength during an earthquake or loose soils located below the groundwater table subject to liquefaction.

1.1.4 Conclusions Regarding the Description of the Management Unit(s)

The descriptions of the management units provided by the owner were accurate representations of what Dewberry observed in the field.

1.1.5 Conclusions Regarding the Field Observations

Dewberry staff was provided access to all areas in the vicinity of the management unit required to conduct a thorough filed observation. The visible parts of the embankment dikes and outlet structure were observed to have no signs of overstress, significant settlement, shear failure, or other signs of instability although visual observations were hampered by the presence of thick vegetation in some areas. Embankments appear structurally sound. There are no visible indications of unsafe conditions or conditions needing immediate remedial action.

1.1.6 Conclusions Regarding the Adequacy of Maintenance and Methods of Operation

The current maintenance and methods of operation appear to be adequate for the CCR management units. There was no evidence of significant unexplained embankment repairs or prior releases observed during the field assessment.

1.1.7 Conclusions Regarding the Adequacy of the Surveillance and Monitoring Program

The surveillance program appears to be adequate. The management unit dikes are instrumented with piezometers and slope inclinometers. Additional piezometers are to be installed and monitored as recommended by Stantec.

1.1.8 Classification Regarding Suitability for Continued Safe and Reliable Operation

The Ash Pond, Gypsum Disposal Area and Dry Fly Ash Stack are rated <u>SATISFACTORY</u> for continued safe and reliable operation.

No other existing or potential management unit safety deficiencies are recognized in the field assessment and review of furnished operations, maintenance, surveillance, and monitoring information. Acceptable performance is expected under applicable static loading conditions and hydrologic conditions in accordance with the applicable criteria.

1.2 RECOMMENDATIONS

1.2.1 Recommendations Regarding the Structural Stability

1) Install the planned lined ponds in the Gypsum Disposal Area as soon as possible for receiving and settling the gypsum slurry that must be sluiced to the Gypsum Disposal Area whenever the dewatering facility has an outage. Re-evaluate the piping potential factor of safety after the lined ponds have been in place for about a year, to check whether or not the elimination of sluice water in the gypsum stack reduces the seepage exit gradients sufficiently to result in acceptable factors of safety against piping. Closely monitor the seepage conditions at the critical section in the interim. If the seepage exit gradients have not sufficiently abated, develop and implement a remedial measure to lower the exit gradients and achieve an acceptable factor of safety against piping failure.

2) Install the additional piezometers around and in the Dry Ash Stack as recommended by Stantec and monitor pore-water pressures periodically as the Dry Ash Stack is filled. If or when the piezometer measurements indicate a significant increase in pore-water pressures in the underlying materials, immediately perform a slope stability analysis to verify that an acceptable factor of safety exists. If the calculated minimum factor if safety is marginal or below, cease filling operations and allow time for the pore-water pressures to dissipate to normal levels. Do not begin filling again until pore-water pressures have stabilized and an acceptable factor of safety exists.

1.2.2 Recommendations Regarding the Hydrologic/Hydraulic Safety

No recommendations for physical or operational modifications to enhance hydrologic/hydraulic capacity appear warranted at this time.

1.2.3 Recommendations Regarding the Supporting Technical Documentation

Since Dewberry issuance of the Draft Report TVA submitted additional documentation. See Docs 22, 23, 24 and 25 in Appendix C. The supporting technical documentation is adequate.

1.2.4 Recommendations Regarding the Field Observations

No significant problems were observed in the field assessment that would require special attention outside of routine maintenance. The minor issues

observed, mostly small eroded areas or areas of seepage and poor drainage, should be addressed by TVA's routine maintenance activities. These include:

1) Repair minor erosion at various locations.

2) Continue to mow/ maintain vegetation along slopes.

3) Continue to monitor and document known seepage per seepage action plan.

4) Provide positive slope to promote drainage into perimeter ditch.

1.2.5 Recommendations Regarding Continued Safe and Reliable Operation

No additional recommendations are warranted at this time, other than the recommendations given to control rate of filling of the Dry Ash Stack. See Subsection 1.2.1, Paragraph 2).

1.3 PARTICIPANTS AND ACKNOWLEDGEMENT

1.3.1 List of Participants

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Michael McLaren, Dewberry
Pamela Stanford, Dewberry

1.3.2 Acknowledgement and Signature

We acknowledge that the management unit referenced herein has been assessed on September 7, 2011.

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2.0 DESCRIPTION OF THE COAL COMBUSTION RESIDUE MANAGEMENT UNIT(S)

2.1 LOCATION AND GENERAL DESCRIPTION

The Cumberland Fossil Plant is located in western Tennessee west-southwest of Clarksville, Tennessee on the south shore of Lake Barkley. The plant is operated by the TVA. The Coal Combustion Residuals (CCR) Complex encompasses approximately 330 acres and consists of the Ash Pond (50 acres), Dry Fly Ash Stack (110 acres) (also known as Dry Ash Stack), and Gypsum Disposal Area (170 acres). A project location map is provided in Appendix A – Doc 1. An aerial photograph of the CCR Complex is provided in Appendix A – Doc 2. Initial information provided by the TVA about the CCR Complex is included in Appendix A – Doc 3. The general layout of the CCR management units is shown in Appendix A – Doc 6, Figure 3.

The entire CCR disposal area was originally constructed in 1969 as one large ash pond. The gypsum disposal area was constructed during 1995-96. It was built over the original ash pond. Additional detail is provided in Section 4.1.2.

Currently, dewatered gypsum is either conveyed directly for use in an adjacent dry wall production facility (Temple Inland Wall Board Plant) or stockpiled and later hauled by truck to the gypsum disposal area. Gypsum can be diverted into the wall board plant at a valve station operated by Synthetic Materials (SynMat). SynMat dewaters the gypsum slurry using vacuum filter presses and the filtrate is returned to the gypsum disposal area where any fines can settle. The gypsum currently is sluiced into the gypsum disposal area only during emergency events when the dewatering facility is not operational. The filtrate or sluice water, as well as surface runoff, drain to a perimeter ditch system and ultimately to the ash pond; a significant body of water is not impounded in the gypsum disposal area.

Fly ash is collected in a dry state, conditioned with moisture and then spread and compacted in the dry fly ash stack. Bottom ash is sluiced to a processing area, reclaimed, and then placed on the dry fly ash stack. The bottom ash processing area includes a small incised pond located at the north end of the divider dike that separates the dry fly ash stack area from the gypsum disposal area (see area marked “Bottom Ash” on Figure 3 in Appendix A – Doc 6.)

Water flows to the ash pond from the bottom ash processing area, which receives slurry directly from the plant. The water decanted from the bottom ash

processing area is conveyed to the 37.4-acre retention pond by a 72-inch diameter pipe spillway. Surface runoff from the gypsum disposal area and from the dry fly ash stack perimeter ditch, as well as filtrate from dewatering or any sluice water from the gypsum disposal area, is also conveyed to the ash pond via 36-inch diameter pipes at two locations through the dike between the ash pond and the dry fly ash stack. One 36-inch pipe is through the west end of the divider dike and a pair of 36-inch pipes is through the east end of the dike.

Water in the ash pond flows generally to the northwest and exits to the stilling basin portion of the ash pond through a 100-foot wide opening in the dike separating the ash retention pond from the stilling basin. A floating boom spans the opening and aids in settlement of very fine solids.

Decanted water discharges from the stilling basin through four 36-inch pipe spillways. Each spillway has a 48-inch concrete riser with a 120-inch diameter corrugated steel pipe skimmer. The spillways empty clean water into a concrete discharge channel that leads to the main plant channel and Lake Barkley.

Table 2.1: Summary of Dam Dimensions and Size	
	Dry Fly Ash Stack
Dam Height (ft)	35
Crest Width (ft)	20
Length (ft)	internal
Side Slopes (upstream) H:V	3:1
Side Slopes (downstream) H:V	3:1
Table 2.1a: Summary of Dam Dimensions and Size	
	Ash Pond
Dam Height (ft)	35
Crest Width (ft)	19 divider , 20 perimeter , 31 dry stack divider
Length (ft)	5600
Side Slopes (upstream) H:V	1.8:1 to 2.5:1
Side Slopes (downstream) H:V	2.2:1 to 2.5:1
Table 2.1b: Summary of Dam Dimensions and Size	
	Gypsum Disposal Area
Dam Height (ft)	50
Crest Width (ft)	20
Length (ft)	internal
Side Slopes (upstream) H:V	1:5 to 3:1
Side Slopes (downstream) H:V	3:1

2.2 COAL COMBUSTION RESIDUE HANDLING

Questions and answers concerning CCR generation and handling are presented in tabular form in Appendix A – Doc 12. The handling of each type of coal combustion residue is briefly described in the following subsections.

2.2.1 Fly Ash

Fly ash CCR is handled through SCR Hoppers, Precipitator Hoppers, and Surge Bins; collected in silos; then trucked to the dry fly ash stack for filling in compacted lifts after adjusting to proper moisture content for compaction.

2.2.2 Bottom Ash

Bottom ash CCR is collected in the Economizer Hoppers using Hydroveyors, and in the bottom ash hoppers using jet pumps. Bottom ash is piped (sluiced) to the bottom ash processing area, where it is reclaimed with excavators, dried, and placed in the dry fly ash stack in the same manner as the fly ash.

2.2.3 Boiler Slag

No information was provided.

2.2.4 Flue Gas Desulfurization Sludge

FGD sludge from the limestone scrubbers is piped directly to the dewatering plant; then conveyed for use in the wallboard plant or trucked for disposal in the gypsum disposal area; sluiced directly to the gypsum disposal area during dewatering plant outages.

2.3 SIZE AND HAZARD CLASSIFICATION

Based on storage capacity the classification for Ash Pond, Dry Ash Stack and gypsum storage area are “intermediate” in accordance with U.S. Army Corps of Engineers (USACE) “Recommended Guidelines for Safety Inspections of Dams” (ER 1110-2-106); the criteria are summarized in Table 2.3a. (Note: The size classification probably is overstated or even has little meaning, if the stored material is not “flowable.”)

Table 2.3a: USACE ER 1110-2-106 Size Classification		
	Impoundment	
Category	**Storage (Ac-ft)**	**Height (ft)**
Small	50 and < 1,000	25 and < 40
Intermediate	1,000 and < 50,000	40 and < 100
Large	> 50,000	> 100

The facilities are not in the National Inventory of Dams; therefore these dikes do not have hazard classifications established by the USACE. The TVA provided preliminary hazard classifications to the USEPA on July 16, 2009, and amended the hazard classifications on October 22, 2010, after a more detailed assessment was performed by their consultant, Stantec Consulting Services, Inc (Stantec). The classification was made based on the 2004 Federal Guidelines for Dam Safety classifications system (shown in Table 2.3b).

Table 2.3b: FEMA Federal Guidelines for Dam Safety Hazard Classification		
	Loss of Human Life	**Economic, Environmental, Lifeline Losses**
Low	None Expected	Low and generally limited to owner
Significant	None Expected	Yes
High	Probable. One or more expected	Yes (but not necessary for classification)

TVA's current hazard classifications for the CCR facilities at the Cumberland Fossil Plant are as follows:

Ash Pond	Significant
Dry Ash Stack	Not Rated (Not an impoundment)
Gypsum Storage Area	Significant

Loss of human life is not probable in the event of a failure of the Ash Pond dikes, but a failure of these dikes is expected to have potential for environmental damage to Lake Barkley, and economic loss to the adjacent road and bridge. Stantec's dam breach analysis of the Ash Pond dikes shows that the bridge would be overtopped before the dike failed (see Appendix A – Doc 14). Thus, the bridge would likely be closed so that there would be no traffic on the

road and bridge just before a postulated breach of the Ash Pond dikes. Also, there are no habitable dwellings within the impact area. Therefore, Dewberry concurs with TVA's current **"Significant"** hazard potential classification for the Ash Pond.

Dewberry concurs with TVA's current **"Significant"** hazard potential classification for the Gypsum Disposal Area on the basis of Stantec's dam breach analysis of the Gypsum Disposal Area dikes (see Appendix A – Doc 13). Although the Gypsum Disposal Area is currently operated primarily as a dry disposal facility, its dikes have the capability of accumulating and containing a significant body of water generated by runoff from the design storm, which was taken as the Probable Maximum Precipitation (PMP) in Stantec's analysis.

Dewberry recognizes that a significant body of water cannot be contained on the Dry Ash Stack. However, runoff from the stack is drained via perimeter ditches leading to drainage structures that discharge through the north divider dike to the Ash Pond. Design storm runoff could potentially collect along the divider dike faster than it can drain to the Ash Pond, causing the buildup of a small body of water that would be contained by relatively short segments of the Dry Ash Stack perimeter dike near the east and west ends of the divider dike. A breach through either segment would at the least have an environmental impact to waterways and drainage ways leading to Lake Barkley. Therefore, Dewberry's opinion is that the Dry Ash Stack area be rated with a **"Significant"** hazard potential classification.

2.4 AMOUNT AND TYPE OF RESIDUALS CURRENTLY CONTAINED IN THE UNIT(S) AND MAXIMUM CAPACITY

The data reviewed by Dewberry did not include the volume of the residuals stored in the ponds at the time of inspection. Volume information provided in Table 2.4, 2.4a, 2.4b was measured by TVA in 2006.

Table 2.4: Maximum Capacity of Unit	
Dry Fly Ash Stack	
Surface Area (acre)[1]	110
Current Storage Capacity (cubic yards)[1]	4,781,000
Current Storage Capacity (acre-feet)	2,963.4
Total Storage Capacity (cubic yards)[1]	12,600,00
Total Storage Capacity (acre-feet)	7,809.9
Perimeter Dike Crest Elevation (feet)	395
Normal Pond Level (feet)	N/A

Table 2.4a: Maximum Capacity of Unit	
Ash Pond	
Surface Area (acre)[1]	50
Current Storage Capacity (cubic yards)[1]	1,305,000
Current Storage Capacity (acre-feet)	808.9
Total Storage Capacity (cubic yards)[1]	2,000,000
Total Storage Capacity (acre-feet)	1,239.7
Perimeter Dike Crest Elevation (feet)	395
Normal Pond Level (feet)	384

Table 2.4b: Maximum Capacity of Unit	
Gypsum Disposal Area	
Surface Area (acre)1	**170**
Current Storage Capacity (cubic yards)1	**1,826,000**
Current Storage Capacity (acre-feet)	**1,131.8**
Total Storage Capacity (cubic yards)1	**20,000,000**
Total Storage Capacity (acre-feet)	**12,396.7**
Perimeter Dike Crest Elevation (feet)	**395**
Normal Pond Level (feet)	**N/A**

[1]2006 data provided by TVA

2.5 PRINCIPAL PROJECT STRUCTURES

2.5.1 Earth Embankment

The entire CCR disposal area was originally constructed in 1969 as one large ash pond encompassed by a perimeter dike constructed to a crest elevation of 380 feet. A divider dikes was added to separate the Ash Pond from the Dry Fly Ash Stack, and the perimeter dike was later raised to current elevation 395 feet; another dike was constructed to separate the Gypsum Disposal Area.

2.5.2 Outlet Structures

Decanted water discharges from the stilling basin through four 36-inch diameter pipe spillways. Each spillway has a 48-inch diameter concrete riser with a 120-inch diameter corrugated steel pipe skimmer. The spillways release treated water into a concrete discharge channel that leads to the main plant channel and Lake Barkley. An emergency spillway has recently been constructed; completed since the date of the site visit (see Appendix A – Doc 17). The spillway is constructed of

concrete and is 36 feet wide with a control elevation of 390 feet. A siphon spillway was under construction at the time of the site visit.

2.6 CRITICAL INFRASTRUCTURE WITHIN FIVE MILES DOWN GRADIENT

Critical infrastructure inventory data was not provided to Dewberry for review.

Based on the available area topographic maps, surface drainage in the area of the ponds are from the southeast to the northwest through the Ash Pond stilling pond to Lake Barkley. Releases from the impoundments would not impact the water level of Lake Barkley significantly; however, damage may occur to the adjacent bridge and highway to the north. If the dikes failed on the south side, releases from the impoundments would not impact the water level of Wells Creek significantly. A release on the north east side of the gypsum stack could result in damage to the adjacent dewatering facility and potentially the wall board plant.

3.0 SUMMARY OF RELEVANT REPORTS, PERMITS, AND INCIDENTS

3.1 SUMMARY OF REPORTS ON THE SAFETY OF THE MANAGEMENT UNIT

TVA provided internal inspection of CCR management units for 2011. The reports included various inspections that were performed daily, weekly, monthly, and quarterly. TVA also provided the 2011 Annual Inspection of CCP Facilities and Ponds, performed by Stantec dated July 19, 2011(see Appendix A – Docs 10, 11).

The reports concluded that the structures appeared to be performing adequately with only minor maintenance items that needed to be addressed. No conditions were observed that would affect the continued safe operation of the impoundments.

Stantec also prepared a "Seepage Action Plan (SAP)" dated June 25, 2010 that provides guidelines for controlling different levels of seepage, should they be observed in routine inspections (see Appendix A – Doc 9).

3.2 SUMMARY OF LOCAL, STATE, AND FEDERAL ENVIRONMENTAL PERMITS

Discharge from the Ash Pond is regulated by the Tennessee Department of Environmental and Conservation, Division of Water Pollution Control, and the impoundment has been issued a National Pollutant Discharge Elimination System Permit. Permit No. TN0005789 was issued November 30, 2007 (See Appendix A – Doc 4).

3.3 SUMMARY OF SPILL/RELEASE INCIDENTS

On February 2, 1997 a bypass of the Cumberland Ash Pond Discharge (outfall 001) occurred when between one-half to one million gallons of gypsum wastewater spilled into Wells Creek. Heavy rainfall contributed to the failure of the internal gypsum dike, allowing a brief surge of wastewater to pass over the exterior dike in the gypsum dewatering area and enter the creek. The bypass lasted no longer than ten minutes. The Gypsum Sluice water was diverted to another portion of the pond until repairs were made to the portion of the gypsum dike that failed. The water level was lowered two feet in the gypsum pond to provide more storage capacity for rainfall and the culverts from the gypsum pond were moved away from the area that failed.

4.0 SUMMARY OF HISTORY OF CONSTRUCTION AND OPERATION

4.1 SUMMARY OF CONSTRUCTION HISTORY

4.1.1 Original Construction

The Cumberland Fossil Plant was constructed between 1968 and 1973, with operation beginning in 1972. The entire CCR disposal area was originally constructed in 1969 as one large ash pond.

4.1.2 Significant Changes/Modifications in Design since Original Construction

In 1977, the divider dike for the ash pond to the north (interior divider dike) was constructed. In 1979, the dikes around the Ash Pond were raised to elevation 395 feet with clay. In 1986, approximately 300 feet of the west portion of the divider dike between the Ash Pond and the Dry Fly Ash Stack was constructed (exterior divider dike) to form the current configuration. In 1996, stacking within the Dry Fly Ash Stack began.

The Gypsum Disposal Area was constructed during 1995-96. It was built over the original ash pond. The Gypsum Disposal Area was constructed in several stages beginning with construction of a rock drainage blanket to collect and divert water away from the base. It is surrounded by a lower earth dike capped with bottom ash and an upper gypsum dike. Due to concerns about elevated piezometeric levels in the gypsum stack and the surrounding dikes, TVA elected to cease regular pumping of gypsum slurry to the gypsum stack in May, 2009. Dewatered gypsum is either conveyed to Temple Inland, adjacent to the plant property, for use in dry wall production or stockpiled and later hauled by truck to the gypsum disposal area.

4.1.3 Significant Repairs/Rehabilitation since Original Construction

A small landslide occurred on the facility in 2005 and temporary stabilization measures were implemented by TVA. Stantec has developed construction documents for permanent repair.

On February 2, 1997 a bypass of the Cumberland Ash Pond Discharge (outfall 001) occurred when between one-half to one million gallons of gypsum wastewater spilled into Wells Creek. Heavy rainfall contributed to the failure of the internal gypsum dike, allowing a brief surge of

wastewater to pass over the exterior dike in the gypsum dewatering area and enter the creek. The bypass lasted no longer than ten minutes. The Gypsum Sluice water was diverted to another portion of the pond until repairs were made to the portion of the gypsum dike that failed. The water level was lowered two feet in the gypsum pond to provide more storage capacity for rainfall and the culverts from the gypsum pond were moved away from the area that failed.

Since the Draft report was issued (See Docs 24 and 25 in Appendix C) several improvements have been completed to increase stability and seepage factors of safety. At the Gypsum Disposal Complex the sluicing operations to the top of the stack have been ceased. Currently SynMat is processing the gypsum slurry while new lined settling channels are being constructed along the north portion of the stack. Also, the slopes of the gypsum stack have been flattened to approximately 3H:1V and graded filters have been installed in parts. The drainage ditches around the disposal complex have also been cleaned of vegetation and regraded to promote positive drainage. Some standing water is still present in portions of the ditch, but overall, drainage conditions have been improved. In the vicinity of Section H, a rip rap buttress was also constructed along the outer dike system to repair observed sloughing and increase the stability factor of safety. At the Ash Pond spillway improvements have been completed to lower the pool level within the Ash Pond. The new pool elevation is approximately 378.2 feet and construction of the spillway improvements was completed in March 2012. As a result of the lower pool level, updated seepage analyses was performed. The results of the updated seepage analyses incorporating the recent improvements and operational changes were reviewed to identify conditions where seepage and possible piping may occur. The models predict the potential for seepage outbreaks to occur along the toe of the perimeter dikes. The calculated factor of safety against piping for the updated Section H is 3.7 which is greater than the Cedergren FS criterion of 2.5 to 3. However, the potential for seepage still exists along the toe of the dikes.

4.2 SUMMARY OF OPERATIONAL PROCEDURES

4.2.1 Original Operational Procedures

The impoundment was designed and operated for ash sedimentation and control. The pond receives plant process waste water, and coal combustion waste slurry. Treated (via sedimentation) process water is discharged through an overflow outlet structure.

4.2.2 Significant Changes in Operational Procedures and Original Startup

Sulfur dioxide scrubbers were installed for both coal fired generating units. Dry fly ash silos were constructed during the dry fly ash conversion project.

At the Gypsum Disposal Complex the sluicing operations to the top of the stack have been ceased. Currently SynMat is processing the gypsum slurry while new lined settling channels are being constructed along the north portion of the stack.

4.2.3 Current Operational Procedures

No documents were provided to indicate any operational procedures have changed.

4.2.4 Other Notable Events since Original Startup

As previously noted, TVA was constructing a siphon spillway at the time of the site visit to allow lowering of the pool elevation; a 35-foot wide concrete emergency spillway has also been constructed. The spillway improvements also include lowering the four risers by 6 feet for permanent lowering of the pool elevation. The lowering of the permanent pool and installation of the emergency spillway were done to prevent overtopping of the Ash Pond dike during the design flood.

5.0 FIELD OBSERVATIONS

5.1 PROJECT OVERVIEW AND SIGNIFICANT FINDINGS

Dewberry personnel Pamela Sanford, P.E. and Michael McLaren, P.E., performed a site visit on Wednesday, September 7, 2011 in company with the participants listed in Section 1.3.

The site visit began at 9:00 AM. The weather was cool and cloudy. Photographs were taken of conditions observed. Please refer to the Dam Inspection Checklist in Appendix B for additional information. Selected photographs are included here for ease of visual reference. All pictures were taken by Dewberry personnel during the site visit.

The overall assessment of the dam was that it was in satisfactory condition and no significant findings were noted.

5.2 DRY GYPSUM DISPOSAL

5.2.1 Crest

The dike divides the gypsum disposal area and Wells Creek. The crest had no signs of depressions, tension cracks, or other indications of settlement or shear failure, and appeared to be in satisfactory condition.

Figure 5.2.1-1 Photo showing Crest, East Dike

Figure 5.2.1-2 Photo showing Crest, South Dike

5.2.2 Upstream/Inside Slope

There were no observed scarps, sloughs, bulging, cracks, or depressions or other indications of slope instabilities or signs of erosion. Figures 5.2.2-1 and 5.2.2-2 show the general condition of the inside slope. Vegetation should be installed to help minimize erosion in bare areas and maintained to allow for inspection of the slopes.

Figure 5.2.2-1 Photo showing inside slope, East Dike

Figure 5.2.2-2 Photo showing inside slope, South Dike

5.2.3 Downstream/Outside Slope and Toe

There were no observed scarps, sloughs, bulging, cracks, or depressions or other indications of slope instabilities or signs of erosion. Figures 5.2.3-1, 5.2.3-2 and 5.2.3-3 show general conditions of the outside slope. Vegetation should be installed to help minimize erosion in bare areas and maintained to allow for inspection of the slopes.

Figure 5.2.3-1 Photo showing outside slope, South Dike

Figure 5.2.3-2 Photo showing outside slope, North Dike. Incised Bottom Ash Pond is on left side of the photo

Figure 5.2.3-3 Photo showing outside slope, East Dike

5.3 DRY FLY ASH STORAGE

5.3.1 Crest

The dike divides the ash storage area and Wells Creek. The crest had no signs of depressions, tension cracks, or other indications of settlement or shear failure, and appeared to be in satisfactory condition.

Figure 5.3.1-1 Photo showing crest, West Dike

Figure 5.3.1-2 Photo showing crest, South/West Dike

5.3.2 Upstream/Inside Slope

There were no observed scarps, sloughs, bulging, cracks, or depressions or other indications of slope instabilities or signs of erosion. The three pictures below show the general condition of the inside slope. Vegetation should be installed to help minimize erosion in bare areas and maintained to allow for inspection of the slopes.

Figure 5.3.2-1 Photo showing inside slope, North Dike

Figure 5.3.2-2 Photo showing inside slope, West Dike

Figure 5.3.2-3 Photo showing inside slope, South/West Dike

5.3.3 Downstream/Outside Slope and Toe

There were no observed scarps, sloughs, bulging, cracks, or depressions or other indications of slope instabilities or signs of erosion. Figures 5.3.3-1 and 5.3.3-2 show the general condition of outside slope. Vegetation should be installed to help minimize erosion in bare areas and maintained to allow for inspection of the slopes.

Figure 5.3.3-1 Photo showing outside slope, West Dike

Figure 5.3.3-2 Photo showing outside slope, South/West Dike

5.4 ASH POND

5.4.1 Crest

The dike divides the Ash Pond and Wells Creek on the south and Lake Barkley to the north. The crest had no signs of depressions, tension cracks, or other indications of settlement or shear failure, and appeared to be in satisfactory condition.

Figure 5.4.1-1 Photo showing crest, North Dike

Figure 5.4.1-2 Photo showing crest, West Dike

Figure 5.4.1-3 Photo showing crest, South Dike

5.4.2 Upstream/Inside Slope

There were no observed scarps, sloughs, bulging, cracks, or depressions or other indications of slope instabilities or signs of erosion. The photo below shows the general condition of the inside slope. Vegetation should be installed to help minimize erosion in bare areas and maintained to allow for inspection of the slopes.

Figure 5.4.2-1 Photo showing inside slope, North Dike

5.4.3 Outside Slope and Toe

There were no observed scarps, sloughs, bulging, cracks, or depressions or other indications of slope instabilities or signs of erosion. Figures 5.4.3-1, 5.4.3-2 and 5.4.3-3 show the general condition of outside slope. Vegetation should be installed to help minimize erosion in bare areas and maintained to allow for inspection of slopes.

Figure 5.4.3-1 Photo showing outside slope, North Dike

Figure 5.4.3-2 Photo showing outside slope, West Dike

Figure 5.4.3-3 Photo showing outside slope, South Dike

5.5 OUTLET STRUCTURES

5.5.1 Overflow Structure

The outfall of the Ash Pond is located on the north end of the pond and consists of four 48-inch RCP risers/weirs with 120- inch diameter corrugated steel pipe skimmers that discharge through four 36-inch concrete pipes that empty treated water into a concrete discharge channel that leads to the main plant channel and Lake Barkley.

The primary overflow structures were observed to be working properly, discharging flow from the ash pond. The outlet structure visually appeared to be in satisfactory condition. There were no signs of clogging.

Figure 5.5.1-1 Photo showing Outlet Structures

5.5.2 Outlet Conduit

The outlet pipes appeared to be operating normally with no signs of clogging and the water exiting the outlets was flowing clear.

Figure 5.5.2 -1 Photo showing Outlet Conduits

5.5.3 Emergency Spillway

No emergency spillway was present at the time of the site visit. TVA completed constructing an emergency spillway in March 2012 (see Appendix A – Doc 17 for letter noting completion of the emergency spillway).

5.5.4 Low Level Outlet

No low level outlet was present; TVA was installing a siphon spillway at the time of the site visit.

6.0 HYDROLOGIC/HYDRAULIC SAFETY

6.1 SUPPORTING TECHNICAL DOCUMENTATION

6.1.1 Flood of Record

No documentation has been provided about the historic maximum water surface elevations in the CCR management units. All the CCR management units (CCR Complex) are contained within a perimeter dike and do not receive off-site natural drainage. Therefore, they do not receive flood inflows from off-site. The Ash Pond within the complex serves as a storm water retention basin. The source of water into the Ash Pond, aside from sluicing water, pumped plant drainage, and pumped Coal Yard drainage, is precipitation that falls directly into the CCR Complex. The Ash Pond collects runoff from the Gypsum Disposal Area and the Dry Ash Stack, as well as rain that falls directly into it.

Historic climate data available on-line from the National Weather Service (NWS) indicate that record rainfall was experienced in middle Tennessee in the two-day period of May 1-2, 2010. A precipitation contour map developed by the NWS shows that the Cumberland Fossil Plant was on the north side of the heaviest precipitation, but the rainfall amount for the 48-hour period was still on the order of 9 inches. According to an "Average Recurrence Intervals Map for 48-Hour Duration," prepared by the Hydrometeological Design Studies Center, the plant is in a location that experienced rainfall having an average recurrence interval on the order of 200 years. At the town of Dover, approximately 10 miles northwest of the plant, the all-time record rainfall was 7.6 inches on August 31, 1982.

6.1.2 Inflow Design Flood

For a conservatively assigned "intermediate" size classification for the three disposal areas comprising the CCR Complex and "significant" hazard potential classification for the entire complex, the USACE hydrologic evaluation guidelines (ER-1110-2-106 26 Sept 1979 "Recommended Guidelines for the Safety Inspection of Dams") recommend a spillway design flood (SDF) of 1/2 Probable Maximum Flood (1/2 PMF) to PMF, where the magnitude selected most closely relates to the involved risk. For comparison, the Tennessee Dam Safety Laws and Regulations (2007) require (for existing dams) use of a

Freeboard Design Storm of 1/3 Probable Maximum Precipitation (1/3 PMP) (6-hour duration) to develop the design flood.

Stantec performed a hydrologic and hydraulic (H & H) analysis of the Ash Pond, which is the hydraulic control for the entire CCR Complex. The analysis is summarized in their memo titled "Hydrologic and Hydraulic Calculations Summary" (H & H memo) dated September 28, 2010 (see Appendix A - Doc 05 for reference). Stantec's analysis evaluated the performance of the CCR Complex for the PMP (6-hour duration), which is the design criterion adopted by the TVA. Stantec used the Hydrologic Engineering Center Hydrologic Modeling System (HEC-HMS) Version 3.4 computer program to develop the inflow hydrograph. The results of the analysis are summarized in the following Table 6.1:

Table 6.1: Summary of 6-Hour PMP Routing

	Pre-Design Conditions	**Post-Design Conditions**[1]
Drainage Area (ac)	460	460
Dam Crest El (ft)	394[2]	394[2]
Normal Pool El (ft)	384.2	378.2
Normal Freeboard (ft)	9.8	15.8
Design Storm Max Pool El (ft)	Overtops	393.9
Min Freeboard During Design Storm (ft)	None	0.1[2]

[1] *Conditions that now exist after remedial spillway improvements.*

[2] *The crest elevation according to historical information is 395 feet in which case the freeboard during the design storm would be 1.1 feet. The actual crest elevation appears to vary from 394.1 feet, to greater than 395 feet, based on furnished topographic information.*

As shown by the above results, with implementation of the spillway improvements the Ash Pond (i.e., CCR Complex) meets TVA's adopted design criterion for spillway design flood.

Since the Draft report was issued (See Docs 24 and 25 in Appendix C) several improvements have been completed. At the Ash Pond spillway improvements have been completed to lower the pool level within the Ash Pond. The new pool elevation is approximately 378.2 feet and construction of the spillway improvements was completed in March 2012.

6.1.3 Spillway Rating

Stantec's H & H memo (Appendix A-Doc 5) indicates that standard hydraulic equations were used to develop a rating curve for the existing spillways (for the pre-design analysis) and level pool routing methodology was used to route the design storm through the outlets. Although not stated in the memo, a rating curve presumably was also developed for the new emergency spillway and the modified old spillways with lowered risers (for the post-design analysis).

6.1.4 Downstream Flood Analysis

Downstream flood analysis was performed by Stantec in their Dam Breach Analysis for the Ash Pond (see Appendix A – Doc 14) and for the Gypsum Disposal Area (see Appendix A – Doc 13). These analyses were performed to determine the limit of impact in case of breach failure of the dikes and to assist in assessing hazard potential classifications. Of particular interest were potential impacts on downstream bridges in case of failure of the Ash Pond perimeter dike and potential impacts on the SynMat Dewatering Facility and Temple Inland Wall Board Plant in case of failure of the Gypsum Disposal Area perimeter dike. A dam breach analysis was not performed for the Dry Ash Stack, since such analysis does not appear to be warranted for this predominantly dry disposal area that is not capable of retaining much water in the majority of its area. A breach of one of the short segments of the perimeter dike that could potentially contain a small body of water in the northern part of the area next to the divider dike, would by inspection have much less impact than a breach of the perimeter dike around the Ash Pond.

Stantec performed the dam breach analyses with the aid of the HEC-HMS Version 3.4 computer program. The analyses examined both a "Sunny Day" breach and a "PMP Event" breach. The assumptions and details of the analyses are presented in some detail in the appended reports (see Appendix A – Doc 13 & Doc 14). The "Sunny Day" breaches were assumed to occur as a result of a piping failure during normal operations. For the Gypsum Disposal Area, wet operations (when needed) were assumed to be limited to lined ponds in a water quality cell along the northern edge of the Gypsum Disposal Area, in accordance with TVA's proposed plans. The "Sunny Day" breach was assumed through the northeast dike, which is near the dewatering facility. The "PMP Event" breach at the Ash Pond was assumed to occur as a result of overtopping of

the perimeter dike (pre-spillway improvement possibility). The "PMP Event" breach at the Gypsum Disposal Area was assumed to be initiated by a piping failure of an interior (presumably unlined) dike on the southwest side of the water quality cell during a PMP event, whose failure would release water that causes overtopping failure of the lower, southeast part of the perimeter dike, which is near the wall board plant. Stantec judged that a piping breach through the northeast dike during a PMP event "was not a concern because a failure through the liner would have an extremely low likelihood of coinciding with the peak PMP event." (Overtopping was not a concern apparently due to substantial freeboard to contain PMP runoff in the water quality cell.) It is important to note that the analyses are of "postulated" breaches, not predicted breaches, to see what the impacts would be if failures occurred as assumed in the analyses.

The results of the Dam Breach Analysis for the Ash Pond show that the most significant impact is to the Cumberland City Road Bridge over Wells Creek due to a postulated PMP Breach, which would cause the water surface elevation to rise 0.4 feet above the bridge deck elevation, although the bridge would be overtopped before a PMP Breach of the Ash Pond dike. It is indicated that the base 100-year flood elevation provided by the USACE is at the bridge deck elevation. Thus, Stantec concluded that "This small rise in the water surface elevation caused by the breach event is unlikely to result in additional risk of loss of life at the bridge." Stantec's model results also show that pier and contraction scour at the bridge could extend to a depth of 22.3 feet within the channel, which would extend 1.3 feet below the base of the pile cap supporting a bridge pier. Stantec concluded that the scour "could undermine the piers, potentially causing bridge failure." As previously mentioned, Stantec recommended that the hazard potential classification for the Ash Pond remain "High Hazard until confirmation of existing scour protection or action is taken to protect the bridge." TVA has implemented plans to place scour protection at the bridge piers and plans for Ash Pond spillway improvements that meet TVA's 6-hour PMP design requirement.

The results of the Dam Breach Analysis for the Gypsum Disposal Area show that the most significant impact is to the dewatering facility during the "Sunny Day" breach. The results indicate maximum inundation depths of approximately 1.3 feet, which Stantec indicated "based on a review of dam safety literature regarding life loss estimation would not likely present a probable threat to human life." Neither the wall board

plant nor the dewatering facility is shown to be impacted by the PMP breach through the southeast dike. Stantec recommended that the hazard potential classification be lowered from the preliminary "high" hazard to "significant" hazard.

6.2 ADEQUACY OF SUPPORTING TECHNICAL DOCUMENTATION

Although the furnished information is not detailed or complete in some respects (e.g., missing attachments and appendices), there is sufficient information to ascertain that valid hydrologic/hydraulic analysis was performed. Therefore the documentation for the Ash Pond, Dry Fly Ash Stack and Gypsum Disposal Area appears overall to be adequate.

6.3 ASSESSMENT OF HYDROLOGIC/HYDRAULIC SAFETY

Calculations provided in the hydrologic analysis report (See Appendix A-Doc 5) show under the outflow configuration at the time of the site visit that the Ash Pond, which controls flow from the entire CCR Complex, would be unable to pass the 6-hour PMP event without overtopping the embankment. The report anticipated CCR releases to the environment if overtopping occurred. As a result of the above findings, TVA re-configured the spillways and added an emergency spillway to prevent overtopping during the maximum rainfall event. The USEPA was notified by TVA in April 2012 that the spillway improvements were completed and in service in March 2012 (see Appendix A – Doc 17 for Stantec's notification letter regarding the in-service date for the spillway improvements). Therefore, on the basis of furnished hydrologic/hydraulic documentation, the Ash Pond (CCR Complex) currently meets accepted standards for hydrologic/hydraulic safety.

7.0 STRUCTURAL STABILITY

7.1 SUPPORTING TECHNICAL DOCUMENTATION

7.1.1 Stability Analyses and Load Cases Analyzed

Stantec performed extensive studies of stability of the CCR Complex dikes, including: a slope stability evaluation of the Ash Pond in March 2010; a detailed geotechnical exploration and stability evaluation of the Dry Ash and Gypsum Storage facilities in June 2010; initial seismic slope stability analysis in September 2011, using ground motions of a 500-year return period seismic event; and additional pseudostatic[2] slope stability analysis in February 2012, using ground motions of a 2,500-year return period seismic event, as requested by the USEPA. In addition, analyses of seepage, pore-water pressures, and piping potential were also performed. (See Appendix A – Docs 6, 7, 8, and 16 for selected sections of the various reports.)

The stability analyses used the computer program SLOPE/W (from GEO-SLOPE International, Inc.). The program is capable of calculating the potential failure surfaces using the Spencer's procedure. Seepage analyses used SEEP/W. Stability under undrained loading conditions, such as may be created by placing in a relatively short time frame a thick lift of CCR material over saturated ash where strength reduction may occur, was analyzed using UTEXAS4 software.

Conditions assessed were:

- Long term steady state conditions based on groundwater and pore water pressures obtained from the SEEP/W model.
- Undrained loading conditions, where appropriate.

[2] The pseudostatic method is a simplified method for determining seismic slope stability that is based on the same approach (i.e., limit equilibrium) used in analyzing static slope stability. In current practice, the pseudostatic method of analysis is used primarily as a screening tool to help assess whether an embankment dam or slope requires a more detailed seismic slope analysis. The pseudostatic method ignores cyclic loading of the earthquake, but accounts for the seismic force by applying an equivalent static force on the slope. In the limit equilibrium approach the stress-strain relationship of the soil is not considered, so the method should not be used for sensitive clays and other materials that lose shear strength during an earthquake or loose soils located below the groundwater table subject to liquefaction.

- Static analysis under rapid drawdown conditions, where appropriate.
- Potential failure due to piping (internal erosion).
- Seismic loading applied with steady state loading w/ horiz seismic coef = 0.083 (500-year return period) and horiz seismic coef = 0.096 (2,500-year return period).

7.1.2 Design Parameters and Dam Materials

The Stantec reports on Geotechnical Exploration and Slope Stability Evaluation, Seismic Stability Report, and the Geotechnical Exploration of the Dry Ash and Gypsum Storage Report provided to Dewberry for review contained sufficient information on design parameters and materials of construction (see Appendix A – Docs 6, 7, 8, and 16 and in Appendix C – Doc 22).

The dike embankment soils consist of predominantly clay for the original dikes and dike raises. The dike raise embankments were largely founded on sluiced ash. A relatively thick deposit of alluvial underlies the dikes and extends down to refusal material (presumed bedrock). The alluvial deposit typically consist of a layer of clay overlying a granular layer, although there are sections under the Ash Pond perimeter dike where only clay alluvium is present under the original dike and granular alluvium is largely present under the dike raise embankment. Based on laboratory shear strength testing and correlations with standard penetration test data from the borings, design properties and parameters were developed for use in stability analyses. The design properties and parameters used in stability analyses of the Ash Pond dikes are as shown in the following Table 7.1:

Table 7.1: Ash Pond Dikes - Design Properties and Parameters of Materials used in the Stability Analyses					
Material	Unit Wt. (pcf)	Effective Stress Parameters (Drained)		Total Stress Parameters (Undrained)	
		C´ (psf)	Ø´ (deg)	C (psf)	Ø (deg)
Clay Dike 1 – Lean Clay	123	200	22	800	20
Clay Dike 1 – Fat Clay	119	200	22	-	-
Clay Dike 2 – Lean Clay	123	200	32	500	21
Clay Dike 2 – Fat Clay	119	200	29	-	-
Fly Ash – Sluiced	100	0	22	140	11
Alluvial – Clay	124	200	33	450	20
Alluvial – Granular	130	0	32	100	20
Bedrock – Very Strong	-	-	-	-	-

Ref: Docs 6, 7, 8, and 16 in Appendix A.

Design properties and parameters used in the slope stability analyses of the dikes containing the Dry Ash Stack and Gypsum Disposal Area are as shown in the following Table 7.2:

Table 7.2: Dry Ash Stack and Gypsum Disposal Area Dikes - Design Properties and Parameters of Materials used in the Stability Analyses					
Material	Unit Wt. (pcf)	Effective Stress Parameters (Drained)		Total Stress Parameters (Undrained)	
		C´ (psf)	Ø´ (deg)	C (psf)	Ø (deg)
Clay Dike 1	124	100	25	800	20
Clay Dike 2 – Lean Clay	128	100	28	500	21
Clay Dike 2 – Fat Clay	127	200	19	200	18
Clay Dike 3	126	50	30	1000	25
Fly Ash – Stacked	100	0	32	0	32
Fly Ash – Stacked (Sat.)	100	-	-	140	11
Fly Ash – Sluiced	100	-	-	280	11
Bottom Ash - Stacked	105	0	35	-	-
Bot. Ash/Fly Ash – Sluiced	100	0	22	140	11
Bottom Ash – Regraded	105	0	32	0	32
Gypsum	105	0	38	0	33
Alluvial – Clay	121	200	30	450	20
Alluvial – Granular	130	0	32	100	20
Matrix (Gvl., Clay, Bould.)	130	0	35	0	35
Riprap	150	0	38	0	38
Bedrock – Boundary	-	-	-	-	-

Ref: Docs 6, 7, 8, and 16 in Appendix A.

Table 7.2 covers the soil parameters in most stability analyses associated with the Dry Ash Stack and the Gypsum Disposal Area. However, somewhat different strength parameters were used in the pseudostatic stability analysis of the divider dike (Section A) between the Dry Ash Stack and the Ash Pond and are presented in the following Table 7.3:

Table 7.3: Dry Ash Stack/Ash Pond Divider Dike - Design Properties and Parameters of Materials used in the Pseudostatic Stability Analysis (2,500-year Return Period Only)

Material	Unit Wt. (pcf)	Effective Stress Parameters (Drained)		Total Stress Parameters (Undrained)	
		C´ (psf)	Ø´ (deg)	C (psf)	Ø (deg)
Alluvial – Clay	121	-	-	450	21
Alluvial – Granular	130	-	-	0	32
Fly Ash – Stacked	100	-	-	0	32
Fly Ash – Sluiced	100	-	-	280	11
Fly Ash/Bot. Ash – Sluiced	100	-	-	0	25
Regraded Bottom Ash	105	-	-	0	32
Divider Dike	130	-	-	0	38
Old Wells Creek Material	130	-	-	100	20

Ref: Doc 16 in Appendix A.

7.1.3 Uplift and/or Phreatic Surface Assumptions

The Stantec reports referenced above included an embankment investigation and evaluation of the phreatic surface elevations based on piezometer data and modeling, where appropriate (Ash Pond dikes), using the SEEP/W program (see Appendix A – Docs 6, 7). The phreatic surfaces determined from the evaluation were used in the embankment slope stability analyses. The phreatic surfaces varied but were generally within the embankment sections below the embankment surface at varying depths with entry at pool or ditch water level on the interior side and exit at the waterway or drainage way along the exterior toe. However, in one area (Section H) of the perimeter dike on the southwest side of the Gypsum Disposal Area, the phreatic surface was found under the original active wet disposal (sluicing) operating assumption to be very shallow

under the outside slope of the highest dike raise (Dike 3), to crop out at ditch level behind the lower dike raise (Dike 2), and to continue at relatively shallow depth through Dike 2 and into the original dike embankment (Dike 1). In the interior dikes the phreatic surface was assumed to extend linearly from ditch water level on one side to pool or ditch water level on the other side.

7.1.4 Factors of Safety and Base Stresses

Stantec analyzed eight representative sections (Sections P, Q, R, S, T, U, V, W) of the Ash Pond perimeter dike under long term steady state (SS) loading conditions. The most critical section (P), having lowest SS factor of safety (FS), was analyzed for two earthquake events (500-year and 2,500-year return period events) using the pseudosatic method. The respective peak ground accelerations (PGA) of 0.083g and 0.217g were determined and used for the seismic coefficients (k = PGA/g). The computed factors of safety for the critical sections of the Ash Pond perimeter dike are presented in the following Table 7.4:

Table 7.4 Factors of Safety for Ash Pond

Critical Sections	Loading Condition	Computed Minimum Factor of Safety (FS)		Required FS (USACE)
		Global (Deep-Seated Pot. Failure)	Non-Global (Very Shallow Pot. Failure)	
P (Exterior Slope – W. Side of Ret. Pond)	Steady State (SS)	1.7	-	1.5
	SS w/ Seismic – 500-Yr Return (k = 0.083)	1.2	-	1.0
	SS w/ Seismic – 2,500-Yr Return (k = 0.217)	1.0	-	
U (Interior Slope – NE Side of Stilling Pond)	SS	2.1	-	1.5

Ref: Doc 6 in Appendix A

Stantec analyzed seven representative sections (Sections A, B, C, D, E, F, G) of the Dry Ash Stack dikes under long term SS loading conditions. Four of the sections (C, D, E, F) are along the perimeter dike; two of the sections (A, B) are along the divider dike between the Dry Ash Stack and the Ash Pond; one (G) is through the interior dike between the Dry Ash

Stack and the Gypsum Disposal area. The most critical section (F) was analyzed for the 500-year return period earthquake event but not for the 2,500-year event, apparently because it could be determined by inspection, based on the results for the 500-year event, that the pseudostatic analysis would yield a FS < 1.0. The divider dike (Section A) was instead analyzed for the 2,500-year event. The divider dike (Section A) was also analyzed for rapid drawdown loading conditions. The computed factors of safety for the critical sections are presented in the following Table 7.5:

Table 7.5 Factors of Safety for Dry Ash Stack

Critical Sections	Loading Condition	Computed Minimum Factor of Safety (FS)		Required FS (USACE)
		Global (Deep- Seated Pot. Failure)	Non-Global (Very Shallow Pot. Failure)	
F (Perimeter Dike – SW Corner)	Steady State (SS)	1.4	-	1.5
	SS w/ Seismic – 500-Yr Return (PGA = 0.083g)	1.0 (Failure Beneath Perim. Dike) 0.8 (Failure Inside Perim. Dike)	-	1.0
	SS w/ Seismic – 2,500-Yr Return (PGA = 0.217g)	Not Analyzed (Presumed < 1.0, based on above results)	-	
A (Divider Dike – N. Side, E. Part)	SS	2.6 (2.8 After Repair)	1.0 (1.6 After Repair)	1.5
	Rapid Draw Down Ash Pond Side	1.7	-	1.2*
	SS w/ Seismic – 2,500-Yr Return (PGA = 0.217g)	1.1 (Failure Through Dike) 1.0 (Failure Through Stack and Under Dike)	-	1.0
B (Divider Dike – N. Side, W. Part)	SS	2.8 (Est. >2.8 After Repair, based on Sect. A Analysis)	1.3 (Est. >1.6 After Repair, based on Sect. A Analysis)	1.5

*Ref: Doc 7 in Appendix A *1.5 if drawdown rate and pore pressures developed from flow nets.*

Since the issuance of Dewberry's recommendation in the Draft report TVA's consultant has conducted additional subsurface investigation and analyses, (see Doc 23, 24 and 25 in Appendix C). The results of the analyses revealed that at the Dry Fly Ash Stack liquefaction will occur. The results of the post-shaking stability analysis for Section F-F' produces a safety factor of 1.11.

Stantec analyzed eight representative sections (Sections H, I, J, K, L, M, N, O) of the Gypsum Disposal Area dikes under long term SS loading conditions. All of the sections except one (O) are along the perimeter dike; Section O is through the divider dike between the Gypsum Disposal Area and the incised Bottom Ash Pond (dredge pit). The most critical section (H) was analyzed for the 500-year return period earthquake event but not for the 2,500-year event, as noted above for Section F, apparently because it could be determined by inspection that the pseudostatic analysis would yield a FS < 1.0, and no other Gypsum Disposal Area dike section has currently been analyzed for the 2,500-year event. The computed factors of safety for the critical sections are presented in the following Table 7.6:

Table 7.6 Factors of Safety for Gypsum Disposal Area

Critical Sections	Loading Condition	Computed Minimum Factor of Safety (FS)		Required FS (USACE)
		Global (Deep-Seated Pot. Failure)	Non-Global (Very Shallow Pot. Failure)	
H (Perimeter Dike – SW. Corner)	Steady State (SS)	1.4 (1.8 After Repair)	-	1.5
	SS w/ Seismic – 500-Yr Return (PGA = 0.083g)	1.0 (Failure Beneath Perim. Dike) 0.8 (Failure Inside Perim. Dike)	-	1.0
	SS w/ Seismic – 2,500-Yr Return (PGA = 0.217g)	Not Analyzed (Presumed < 1.0, based on above results)	-	
J (Perimeter Dike – S. Side)	SS	1.7 (1.8 After Repair)	1.3(1.6 After Repair)	1.5
K (Perimeter Dike – SE Corner)	SS	2.0 (Est. >1.8 After Repair, based on Sect. J Analysis)	1.2 (Est. >1.6 After Repair, based on Sect. J Analysis)	1.5
L (Perimeter Dike – SE Side)	SS	2.0 (Est. >1.8 After Repair, based on Sect. J Analysis)	1.3 (Est. >1.6 After Repair, based on Sect. M Analysis)	1.5
M (Perimeter Dike – NE Side)	SS	2.5 (2.8 After Repair)	1.2 (1.6 After Repair)	1.5

Ref: Doc 7 in Appendix A

Undrained analyses were performed for the Dry Ash Stack, using Sections C and F, and for the Gypsum Stack, using Sections J and M, for use in determining additional loading that could be safely placed quickly ("instantaneously"). The objective was to determine the maximum lift thicknesses that could be placed quickly without inducing excessive shear strains that would produce sharp loss of shear strength in the underlying saturated sluiced ash materials. The above sections were selected because they had the lowest drained factors of safety and generally had the thickest layers of saturated ash. A sudden loss of shear strength in the underlying saturated ash originally sluiced into the areas now used for dry disposal of ash and gypsum could potentially precipitate massive failure of the stacks and the raised dike sections that are founded on sluiced ash. It appears that the maximum build-out of the stacks is planned to be around elevation 600 feet for the dry ash stack (approximately 205 feet above the existing raised perimeter dike) and around 590 feet (approximately 180 feet above the existing raised perimeter dike) for the gypsum stack. Therefore, the rate of filling is critical. The results showed for the Dry Ash Stack that the maximum lift thickness placed quickly should be limited to 12.5 feet at Section C and 20.0 feet at Section F. Based on one-dimensional consolidation calculations, Stantec calculated that if the maximum lift thickness was placed "instantaneously" in an area, no additional fill should be placed in that area for 2.5 years. For the Gypsum Disposal Area Sections J and M, "full buildout" was indicated, meaning there was no limitation on lift thickness at these locations.

For all the analysis sections for the Ash Pond perimeter dike, piping potential was evaluated. This was done by computing seepage exit gradients and comparing with the critical gradient (0.97 to 1.00, depending on location) to calculate a factor of safety against piping ($FS_{piping} = i_{crit}/i$). The minimum computed $FS_{piping} > 4.0$ for most of the analysis sections. For Sections P, Q, and R, the minimum computed $FS_{piping} = 1.3$, 2.4, and 2.6, respectively. Stantec adopted a target minimum factor of safety criterion of 4.0 against piping. This exceeds the factor of safety criterion of 2.5-3.0 proposed in 1977 by Cedergren and noted in USACE's EM 1110-2-1901. Therefore, the analysis Sections P, Q, and R, which represent the west side of the retention pond, do not meet Stantec's criterion and generally do not meet the Cedergren criterion. Stantec provided two alternatives for increasing the factor of safety against piping. One would be to construct a toe berm along the west side of the perimeter dike, along Wells Creek, but this would require 404 and 401 permitting.

The other alternative, which was recommended, would be to install a driven sheet-pile wall; the wall would not require environmental permitting. However, Stantec recommended that design of the sheet-pile wall be delayed until studies underway at the time concerning potential spillway modifications and hazard status were completed.

Only the Ash Pond in the CCR Complex represents a surface impoundment. Since the Dry Ash Stack only stores dry products, it is not considered an impoundment, since it does not retain standing water. The only water is storm water runoff that drains to the Ash Pond. Therefore, piping potential evaluation of the Dry Ash Stack dikes was not needed.

Although the Gypsum Disposal Area currently is operated as a dry disposal facility, it does on occasion receive sluiced (pumped) gypsum slurry whenever the dewatering facility is down for maintenance or repairs or power outage. In addition, infiltrated water from past sluicing operations is still slowly draining through the relatively massive gypsum stack and may be responsible for the high phreatic surface that was encountered at the most critical analysis section (Section H). In order to evaluate the effects of seepage on slope stability and piping potential at the critical Section H, Stantec performed a detailed seepage analysis to determine the water conditions for use in slope stability analysis and seepage gradients for using in computing the factor of safety against piping.

For the existing conditions (prevailing at the time of the study) and assuming active sluicing to the Gypsum Stack, the slope stability analysis at Section H incorporating the seepage results yields a computed static slope stability FS = 0.7. For the proposed repair, which included a gravel trench drain extending down below the crest of Dike 2 and riprap toe buttress into foundation soil at the toe of Dike 1 (original dike) with riprap blanket extending up slope to the crest of Dike 2, the computed FS = 1.6.

The factor of safety against piping was computed at a number of potential exit points on the outer side of the Section H perimeter dikes assuming active sluicing: for existing conditions the computed FS_{piping} = 0.68-1.05 and for the repaired conditions the computed FS_{piping} = 1.35-1.39. The factor of safety against piping was also computed assuming no active sluicing and no active stack dewatering: for existing conditions the computed FS_{piping} = 1.04-1.70 and for the repaired conditions the computed FS_{piping} = 2.26-3.59. None of these results meet the adopted FS

criterion of 4.0, although the repaired conditions with no active sluicing and no active stack dewatering comes close to meeting the Cedergren FS criterion of 2.5 to 3.0. Stantec suggested that active dewatering by pumping from wells may need to be considered and that pump tests would be needed for evaluation and design of pumping wells. Stantec recommended active sluicing be discontinued, but indicated that sluicing to small lined ponds on the Gypsum Stack would be feasible. Stantec further recommended that the proposed repairs be implemented as soon as possible.

On the basis of the results of the static stability analyses, Stantec recommended repairs in four areas of the Dry Ash Stack/ Gypsum Disposal Area, as follows:

1. Constructing toe buttress of compacted clay with surface layer of rock along the so-called "bottom ash road dike" on the perimeter of the Gypsum Disposal Area, including Sections H, J, K, L, M, and N, to improve stability to acceptable factor of safety against Non-global (shallow) potential failures.

2. Repair of "original and raised" dikes at Section H at the Gypsum Disposal Area, consisting of a gravel trench drain extending down below the crest of Dike 2 and riprap toe buttress into foundation soil at the toe of Dike 1 (original dike) with riprap blanket extending up slope to the crest of Dike 2, to improve stability to acceptable factors of safety against Global (deep-seated) potential failures and against potential piping failures.

3. Regrading of somewhat over-steepened Stacked Fly Ash slope at Section F to maximum slope of 3H to 1V, to improve Global stability to an acceptable level.

4. Regrading of over-steepened slope at Sections A and B on the Ash Pond side of divider dike to maximum slope of 3H to 1V, to improve Non-global stability to an acceptable level.

The repairs have been implemented, and Stantec has re-performed static stability analyses of as-built conditions to verify conformance with the minimum FS criterion.

7.1.5 Liquefaction Potential

There was no documentation provided to Dewberry that included an evaluation of potential liquefaction. It is understood from TVA that liquefaction potential will be addressed as part of a comprehensive risk/consequences-based evaluation of seismic failure risks being conducted in closure design. TVA's approach is described in a "Seismic Risk Assessment White Paper" provided in Stantec's report dated September 22, 2011 (see Appendix A – Doc 8). It is understood that Phase A of the seismic risk assessment study includes analysis/evaluation for liquefaction potential using Phase 2 geotechnical data. It is further understood that the results are being used to assess seismic failure risks for probable closure geometries.

Dewberry has performed an independent qualitative assessment of the Cumberland Ash Pond and determined a qualitative rating of "No Concern" about potential liquefaction under the Ash Pond dikes (see Appendix B – Doc 21).

Stantec's test borings indicate that dike embankments are generally firm and compacted. However, the upper dike raise embankments at the Dry Ash Stack and Gypsum Disposal Area are largely founded on sluiced fly ash, some of which is very loose, according to a number of Stantec's test borings. Thus, there appears to be a potential for liquefaction to occur during strong seismic shaking. Evaluation of these conditions will require a quantitative evaluation to determine the amount of potential deformation and its effects on the integrity and stability of the dike embankments, as well the stacks themselves. Based on currently available information, it is concluded that liquefaction potential at the Dry Ash Stack and Gypsum Disposal Area containment dikes under seismic loading is unknown.

7.1.6 Critical Geological Conditions

From Stantec's "Report of Geotechnical Exploration and Slope Stability Evaluation" dated March 2010 (Appendix A – Doc 6), the geologic map of the Cumberland City Quadrangle (USGS 1968, revised 1986) shows the site of the CCR Complex is predominantly underlain by bedrock belonging to the Mannie Shale, Fernvale Limestone, Hermitage, Carters, Lebanon, Ridley, Pierce, and Murfreesboro Limestone formations, in general order of descending lithology. The limestone layers may be generally described as thin to thick bedded, greenish-gray to gray, coarse

to crystalline grained, argillaceous and hard. There is a large variation in the contour of the bedrock due to an ancient meteorite impact crater below the site.

The CCR Complex is founded on alluvial soils consisting of predominantly clay and granular soils. The clay typically occurs in a layer above the granular alluvium. The clay is generally lean clay containing varying amounts of silt and sand. The consistency of the clay ranges from soft to very stiff. The granular alluvium generally consists of silty sand with gravel and has relative densities ranging from very loose to compact.

The main hazard associated with the geology of the area is the presence of some very loose granular soils and soft clays and potentially very soft clays that may behave unsatisfactorily under certain cases of loading, particularly seismic loading. Although Stantec's test borings penetrated such very loose and soft alluvial soils under the dike embankments, they appeared to occur in relatively thin layers or zones, rather than as thick deposits. The saturated fly ash that generally occurs under the dike raise embankments presents the greatest hazard.

7.2 ADEQUACY OF SUPPORTING TECHNICAL DOCUMENTATION

Structural stability and related documentation, including analyses for static slope stability, seismic (pseudostatic) slope stability, seepage, and piping potential, for the Ash Pond containment dike is adequate. Static slope stability documentation for both the Dry Ash Stack and the Gypsum Disposal Area containment dikes is adequate, as is documentation of seepage and piping potential at the critical section of the Gypsum Disposal Area containment dike. Originally the documentation of performance of the Dry Ash Stack and the Gypsum Disposal Area containment dikes under seismic loading was inadequate, because no evaluation of potential liquefaction has been provided and seismic stability analyses are incomplete or do not demonstrate acceptable safety under the design seismic event required by the USEPA. Based on Dewberry's recommendations in the Draft report, Stantec performed a liquefaction potential assessment based on groundmotion estimates for the 2,500-year earthquake scenarios. See Docs 22 and 23. The saturated ash materials are anticipated to undergo liquefaction for the 2,500-year earthquake. The results of the liquefaction and post-earth stability analyses indicate that the Gypsum disposal Area will remain stable and display adequate performance due to the 2,500-year earthquake.

7.3 ASSESSMENT OF STRUCTURAL STABILITY

The structural stability of the Ash Pond containment dike and outlet works appears to be satisfactory in most respects, based on the following:

- The containment dike crests appeared free of depressions and no significant vertical or horizontal alignment variations were observed.
- There was no indication of major scarps, sloughs or bulging along the dike.
- Boils, sinks or major uncontrolled seepage was not observed along the slopes or toes of the dike.
- The static loading slope stability factors of safety are generally well above the minimum required value.
- The seismic (pseudostatic) slope stability analyses, performed for ground motions with a return period of 2,500 years (2% probability of exceedence in 50 years), shows the dike meets the minimum factor of safety.
- There appears to be "No Concern" for liquefaction potential, based on Dewberry's qualitative evaluation.
- The outflow structures appeared to be in satisfactory condition and stable.

An issue is that, for the perimeter dike along the west side of the retention part of the Ash Pond, the factor of safety against a potential piping failure is below the acceptable minimum. Stantec has provided recommendations to improve the factor of safety against potential piping to an acceptable level. There appears to be no immediate danger of a piping failure, and TVA's surveillance program and seepage action plan appear sufficient to timely identify an emerging active piping problem and take corrective actions before an emerging condition worsens. The Draft report considered the overall structural stability of the Ash Pond dike to be fair until the recommended remedial measures are implemented. Since the Draft report was issued Stantec provided additional analyses see Doc 25 in Appendix C. The Ash Pond spillway improvements have been completed to lower the pool level within the Ash Pond. The new pool elevation is approximately 378.2 feet and construction of the spillway improvements was completed in March 2012. As a result of the lower pool level, updated seepage analyses was performed. The results of the updated seepage analyses incorporating the recent improvements and operational changes were reviewed to identify conditions where seepage and possible piping may occur. The models predict the potential for seepage outbreaks to occur along the toe of the perimeter dikes. The calculated factor of safety against piping for the

updated Section H is 3.7 which is greater than the Cedergren FS criterion of 2.5 to 3. However, the potential for seepage still exists along the toe of the dikes. Based on the review of the new analyses the structural stability of the Ash Pond is considered satisfactory.

Based on the field observations and results of static slope stability documentation and considering that repairs have been made in the four potential problem areas identified in Stantec's geotechnical investigation, the static stability of the dikes containing the Dry Fly Ash Stack appear to be satisfactory. The static stability of the dikes containing the Gypsum Disposal Area appear to be generally satisfactory, except for marginally low factors of safety against potential piping at the critical section (H), even after recommended repair and ceasing regular active sluicing of gypsum slurry into the facility. The seepage exit gradients causing the lower-than-desired piping factors of safety may eventually subside when the current infrequent sluicing, necessitated during dewatering plant outages, is to the lined ponds planned for the Gypsum Disposal Area. However, infiltration of rainfall runoff will continue to recharge the groundwater system in the gypsum stack, and the ultimate seepage gradients will depend on the long-term water balance. Stantec's suggestion of pumping from wells may need to be considered.

Based on Stantec's undrained analyses of the Dry Ash Stack and the Gypsum Stack, it is apparent that the rate of filling is critical to stability and safe operation, particularly of the Dry Ash Stack. Stantec recommended that additional piezometers be installed around and in the Dry Ash Stack for monitoring of pore-water pressures under the weight of the ash material as the stack is filled. Any significant increases detected would be cause to perform additional stability analyses to verify that acceptable factors of safety exist. Particular attention should be paid to monitoring the stack stability in this manner, as a means to control the rate of filling.

Since Dewberry's recommendations in the Draft report Stantec has provided additional analyses: seismic analyses; seepage analyses; liquefaction potential assessment; and information concerning operating practices. See Docs 22, 23, 24 and 25 in Appendix C. Since the previous geotechnical exploration of the Gypsum Disposal Complex, several improvements have been completed to increase stability and seepage factors of safety. Sluicing operations to the top of the stack have been ceased. Currently SynMat is processing the gypsum slurry while new lined settling channels are being constructed along the north portion of the stack. Also, the slopes of the gypsum stack have been flattened to approximately 3H:1V and graded filters have been installed in parts. The drainage ditches around the disposal complex have also been cleaned of vegetation and regraded to promote positive drainage.

Some standing water is still present in portions of the ditch, but overall, drainage conditions have been improved. In the vicinity of Section H, a rip rap buttress was also constructed along the outer dike system to repair observed sloughing and increase the stability factor of safety. Based on the information provided, the overall stability of the Dry Fly Ash Stack and the Gypsum Disposal Area containment dikes is considered satisfactory.

Spillway improvements have been completed to lower the pool level within the Ash Pond. The new pool elevation is approximately 378.2 feet and construction of the spillway improvements was completed in March 2012. As a result of the lower pool level, updated seepage analyses was conducted. The results of the analyses showed Factors of safety against piping in these areas are all equal to or greater than the recommended target value of 4. See Table 7.3 below for the safety factors.

Table 7.3 Summary of Computed Exit Gradients and Minimum Factors of Safety Against Piping for Updated Analyses

Cross Section	Vertical Gradient (i_y) at Critical Exit Point	Location of Critical Exit Point	Material	Critical Gradient (i_{crit})	FS_{piping}
P	0.25	Toe	Alluvial Clay	1.00	4.0
Q	0.13	Toe	Alluvial Clay	1.00	7.7
R	0.25	Toe	Dike 1 Lean Clay	1.00	4.0

FS = Factor of Safety

Based on the information provided the overall stability of the Dry Fly Ash Stack and the Gypsum Disposal Area containment dikes is considered satisfactory.

8.0 ADEQUACY OF MAINTENANCE AND METHODS OF OPERATION

8.1 OPERATING PROCEDURES

The Ash Pond is operated for settling and storage of ash deposits. The Dry Fly Ash Stack is used to store the dry fly ash produced by the plant. The Gypsum Disposal Area is used to store the dry gypsum that the drywall plant is unable to use. The Ash Pond receives effluent from the Bottom Ash pond and runoff from the perimeter ditch that surrounds the Gypsum Disposal Area. Water flows from the ash retention area of the Ash Pond to the stilling basin area through a 100-foot opening in an interior dike. The stilling basin serves as a polishing step. Treated coal combustion process waste water is discharged through four overflow risers into the main plant discharge channel and Lake Barkley.

8.2 MAINTENANCE OF THE DAM AND PROJECT FACILITIES

Plant personnel perform daily, weekly, monthly, and quarterly inspections, and hire a third party engineering firm to perform an annual inspection. All the inspections address required maintenance. It appears the maintenance procedures are adequate (see Appendix A – Docs 10, 11).

8.3 ASSESSMENT OF MAINTENANCE AND METHODS OF OPERATIONS

8.3.1 Adequacy of Operating Procedures

Based on the assessments of this report, operating procedures appear to be adequate.

8.3.2 Adequacy of Maintenance

Based on the assessments of the inspection reports and visual observations during the site visit, maintenance activities appear to be adequate.

9.0 ADEQUACY OF SURVEILLANCE AND MONITORING PROGRAM

9.1 SURVEILLANCE PROCEDURES

Daily inspections are conducted by plant personnel. Inspection reports are submitted for review by TVA management. The appropriate corrective actions are performed as required.

9.2 INSTRUMENTATION MONITORING

The Cumberland CCR management dikes have piezometers to monitor ground water levels (see Appendix A – Doc 15) and slope inclinometers to monitor the slope movement of the dikes.

9.3 ASSESSMENT OF SURVEILLANCE AND MONITORING PROGRAM

9.3.1 Adequacy of Inspection Program

Based on the data reviewed by Dewberry, including observations during the site visit, the inspection program is adequate.

9.3.2 Adequacy of Instrumentation Monitoring Program

Based on the data reviewed by Dewberry, including observations during the site visit, the instrumentation program is adequate.

APPENDIX A

Document 1

Project Location Map

APPENDIX A

Document 2

Aerial Photography

APPENDIX A

Document 3

Steam Electric Questions and Answers

Tennessee Valley Authority
400 West Summit Hill Drive
Knoxville, Tennessee 37902-1401

Anda A. Ray
Senior Vice President
Office of Environment and Research

March 25, 2009

Mr. Richard Kinch
U.S. Environmental Protection Agency
Two Potomac Yard
2733 South Crystal Drive
5th Floor: N-5783
Arlington, Virginia 22202-2733

Dear Mr. Kinch:

Enclosed is the Tennessee Valley Authority's (TVA) response to your requests for information about coal-combustion by-product management impoundments and our signed authorized certification. Your requests were received at TVA's plant sites on March 12 and March 13. Enclosed is the consolidated response from TVA for all of our fossil plants. We have also included in our response two plants (Watts Bar Fossil Plant, inactive and Cumberland Fossil Plant) for which we did not receive a request for information.

Sincerely,

Anda A. Ray

Enclosures: 2007-2008 Annual Inspection Reports of Waste Disposal Areas for all TVA fossil plants.
TVA Responses to EPA Information Request.
Ash Storage Summary.
Certification Form.

EPA believes that the information requested is essential to an evaluation of the threat of releases of pollutants or contaminants from these units. The provisions of Section 104 of CERCLA authorize EPA to pursue penalties for failure to comply with or respond adequately to an information request under Section 104(e). In addition, providing false, fictitious or fraudulent statements or representations may subject you to criminal penalties under 18 U.S.C. 1001.

Your response must include the following certification signed and dated by an authorized representative of Tennessee Valley Authority.

> I certify that the information contained in this response to EPA's request for information and the accompanying documents is true, accurate, and complete. As to the identified portions of this response for which I cannot personally verify their accuracy, I certify under penalty of law that this response and all attachments were prepared in accordance with a system designed to assure that qualified personnel properly gather and evaluate the information submitted. Based on my inquiry of the person or persons who manage the system, those persons directly responsible for gathering the information, the information submitted is, to the best of my knowledge, true, accurate, and complete. I am aware that there are significant penalties for submitting false information, including the possibility of fines and imprisonment for knowing violations.

Signature: [signature]
Name: John C. Kammeyer
Title: VP, Engineering

This request has been reviewed and approved by the Office of Management and Budget pursuant to the Paperwork Reduction Act, 44 U.S.C., 3501-3520.

Please send your reply to:

Mr. Richard Kinch
US Environmental Protection Agency (5306P)
1200 Pennsylvania Avenue, NW
Washington, DC 20460

If you are using overnight or hand delivery mail, please use the following address:

Mr. Richard Kinch
US Environmental Protection Agency
Two Potomac Yard
2733 S. Crystal Dr.
5th Floor; N-5783
Arlington, VA 22202 2733

Tennessee Valley Authority Response to Environmental Protection Agency Request for Information

1. **Relative to the National Inventory of Dams criteria for High, Significant, Low, or Less-than-Low, please provide the potential hazard rating for each management unit and indicate who established the rating, what the basis of the rating is, and what federal or state agency regulates the unit(s). If unit(s) does not have a rating, please note that fact.**

   The dam safety hazard potential rating for each management unit is identified on the attached table. The current hazard potential ratings were assigned by TVA using the National Inventory of Dams criteria as a guideline. Hazard classifications have not been assigned to dry disposal management units. The list is updated by TVA every 2 years. No other agencies, federal or state, regulate these facilities from a dam safety perspective.

   Currently, TVA has secured the services of a third party consultant to review the conditions at our coal combustion storage facilities and provide opinions relative to hazard potential. These opinions will be based on the National Inventory of Dams criteria, as well as dam safety regulations of the states in which each unit is located.

2. **What year was each management unit commissioned and expanded?**

   The year each management unit was commissioned and expanded is identified in the attached table.

3. **What materials are temporarily or permanently contained in the unit? Use the following categories to respond to the question: (1) fly ash; (2) bottom ash; (3) boiler slag; (4) flue gas emission control residuals; (5) other. If the management unit contains more than one type of material, please identify all that apply. Also, if you identify "other", please specify, the other types of materials that are temporarily or permanently contained in the unit(s)**

   The coal-combustion byproduct materials contained in each unit are identified in the attached table. Impoundments at units are also routinely used to combine and treat a variety of runoff and low volume water wastes prior to discharge.

4. **Was the management unit(s) designed by a Professional Engineer? Is or was the construction of the waste management unit(s) under the supervision of a Professional Engineer? Is inspection and monitoring of the safety of the waste management unit(s) under the supervision of a Professional Engineer?**

   Permitted solid waste landfill design documents were prepared under the supervision of a registered professional engineer, with design documents stamped by the responsible engineer. In general, for non-permitted management units, the design and construction, along with the inspection and monitoring of all management units, were performed under the supervision of professional engineers.

   TVA is currently revising our program to ensure that the supervision of all design, construction, and monitoring elements for all management units will be performed by professional engineers properly licensed in the states where the project is located and that have specific experience in dam design and operation.

5. **When did the company last assess or evaluate the safety (i.e., structural integrity) of the management unit(s)? Briefly describe the credentials of those conducting the structural integrity assessments/evaluations. Identify actions taken or planned by facility personnel as a result of these assessments or evaluations. If corrective actions were taken, briefly describe the credentials of those performing the corrective actions, whether they were company employees or contractors. If the company plans an assessment or evaluation in the future, when is it expected to occur?**

   Dates of the most recent facility inspection performed by the company or its consultant are listed in the attached table. These inspections were limited to surface observations. No intrusive sampling or testing, or engineering analyses were involved. Enclosed are the 2007-2008 inspection reports which were performed by TVA staff. All 2009 inspection reports are currently under review. These 2009 inspections were performed by TVA staff (who are experienced, degreed Civil Engineers, under the supervision of a registered professional engineer), with the exception of Cumberland, Shawnee, and Watts Bar (inactive) Fossil Plants, which were performed by Stantec.

   The most recent reviews at the Cumberland and Shawnee Fossil Plants were performed by Stantec. Stan Harris, PE, led those reviews. Mr. Harris has over 25 years experience in dam design, construction, and monitoring. In addition, Mr. Harris has experience leading dam safety training initiatives for the United States Army Corps of Engineers.

   Recommended corrective actions resulting from these evaluations are listed in the attached table. The corrective actions have been assigned to TVA staff or contractors experienced in general earth work construction and operation/construction of coal combustion disposal facilities.

TVA has retained the services of a third party consultant, Stantec, to assess each coal combustion byproducts storage facility at the eleven (11) active and one (1) inactive fossil plant. The assessments include field reconnaissance and records review for each facility. Reports will include recommendations and a priority list for additional geotechnical and engineering evaluations, if necessary. The study is on-going with results expected by the end of April 2009.

As a part of this study, TVA has initiated geotechnical explorations of the gypsum stack at our Paradise Fossil Plant, the ash pond at our Johnsonville Fossil Plant, the gypsum stack and ash dredge cell at our Widows Creek Fossil Plant, the ash disposal facility at our John Sevier Fossil Plant, and the gypsum stack and ash stack at our Cumberland Fossil Plant.

6. **When did a State or Federal regulatory official inspect or evaluate the safety (structural integrity) of the management unit(s)? If you are aware of a planned state or federal inspection or evaluation in the future, when is it expected to occur? Please identify the Federal or State regulatory agency or department which conducted or is planning the inspection or evaluation. Please provide a copy of the most recent official inspection report or evaluation.**

   TVA facilities are subject to regulation by state agencies responsible for permitting solid waste disposal and discharging of process or storm water flows. These state agencies do perform field reviews; however TVA facilities are not subject to regulation by state agencies relative to dam safety permitting and have not been subject to review or inspections by any federal regulatory agency. Copies of the most recent issued inspection report are enclosed for the 2007-2008 time period.

7. **Have assessments or evaluations, or inspections conducted by Federal regulatory officials conducted within the past year uncovered a safety issue(s) with the management unit(s), and, if so, describe the actions that have been or are being taken to deal with the issue or issues. Please provide any documentation that you have for these actions.**

   TVA facilities are subject to regulation by state agencies responsible for permitting solid waste disposal and discharging of process or storm water flows. These state agencies do perform field reviews however; TVA facilities are not subject to regulation by state or federal regulatory agencies relative to dam safety permitting and have not been subject to review or inspections. Copies of the most recent issued inspection report are enclosed for the 2007-2008 time period.

   Primarily maintenance issues were identified

during the most recent inspections. A summary of items identified are provided in the attached table. TVA is currently preparing work orders to address these items. The work will be performed by TVA staff or contractors experienced in earth work and the operation of coal combustion product disposal facilities.

8. **What is the surface area (acres) and total storage capacity of each of the management units? What is the volume of materials currently stored in each of the management unit(s)? Please provide the date that the volume measurement(s) was taken. Please provide the maximum height of the management unit(s). The basis for determining maximum height is explained later in this Enclosure.**

   The surface area, total storage capacity, volume of materials currently stored, and date of last volume measurement for each management unit are provided in the attached table. Data based on 2006 long-range plans of the projected remaining capacities ending at Fiscal Year 2008.

9. **Please provide a brief history of known spills or unpermitted releases from the unit within the last ten years, whether or not these were reported to State or federal regulatory agencies. For purposes of this question, please include only releases to surface water or to the land (do not include releases to groundwater).**

   A history of known spills or unpermitted releases from each unit within the last ten (10) years, if applicable, is listed in the attached table. All spills and unpermitted releases were reported to the appropriate state or federal agencies as required by regulation or law.

10. **Please identify all current legal owner(s) and operator(s) at the facility.**

    The United States is the owner of TVA facilities, and TVA is the operator of each facility listed in the attached table.

PLANT	FACILITY	HAZARD POTENTIAL CLASSIFICATION See footnote #1	Hazard Rating Performed By	YR MGT UNIT COMMISSIONED See footnote #2	YR MGT UNIT EXPANDED	MATERIALS CONTAINED IN UNIT	LAST TVA ASSESSMENT	NEXT SCHEDULED TVA ANNUAL INSPECTION	ACTIONS TAKEN OR PLANNED RESULTING FROM LAST ANNUAL INSPECTION	ISSUES REPORTED BY STATE OR FEDERAL ASSESSMENTS AND ACTIONS TAKEN See footnote #3	SURFACE AREA (ACRES)	TOTAL STORAGE CAPACITY (Cubic Yards)	CURRENT VOLUME OF MATERIAL (Cubic Yards)	REMAINING CAPACITY (Cubic Yards)	DATE VOLUME TAKEN	CURRENT HEIGHT (FT)	FUTURE MAX. HEIGHT (FT)	KNOWN SPILLS OR UNPERMITTED RELEASES (SURFACE WATER/LAND) See footnote #4	CURRENT LEGAL OWNER(S)) & OPERATOR(S)) AT FACILITY
Allen Fossil Plant	EAST ASH DISPOSAL	LOW	TVA	1967	1978	FLY ASH	Nov-08	2009	Maintenance concerns such as rutting, erosion, vegetation, etc., were noted; a seep was noted north of the plant - TVA has an independent consultant evaluating the seep.	NR	70	1,775,000	1,029,000	746,000	2006	20	20	NR	Owner - United States, Operator - TVA
	EAST ASH STILLING POND	Not Rated		1978	Not Expanded	Fly ash, bottom ash					23	290,000	INCLUDED IN EAST ASH DISPOSAL AREA	INCLUDED IN EAST ASH DISPOSAL AREA	2006	20	20		
Bull Run Fossil Plant	DRY FLY ASH DISPOSAL AREA	Not Rated		1982 - Phase 1	1990 - Phase II	FLY ASH	Nov-08	2009	(1) Work order written to regrade top of Bottom Ash Stack, (2) work order for regrading and placement of rip rap below drainage pipe erosion on east side of Bottom Ash Stack, (3) work orders written for numerous animal paths and burrows noted around Bottom Ash Stack and Active Fly Ash Pond Area 2, (4) work order for repair of erosion areas along the bank of Bull Run Creek on south side of Active Fly Ash Pond Area 2, (5) removal of fallen trees on west side of Area 2 Stilling Pond and north side of Gypsum Disposal Area 2A, (6) work order written to repair eroded area on south slope of Gypsum Disposal Area 2A.	NR	17 (Phase II)	4,800,000.00	3,903,000	897000	2006	60	84	NR	Owner - United States, Operator - TVA
	FLY ASH POND AND STILLING BASIN AREA 2	LOW	TVA	1967	1976 - divider dike constructed to form Stilling Pond 1981 - dike constructed to form Pond 2A	Fly ash, bottom ash					49	2,700,000	2,332,600	367,400	2006	20	20		
	BOTTOM ASH DISPOSAL AREA 1	Not Rated		1967	1980 - Dike constructed to form stacking area within former pond	BOTTOM ASH (flows to Fly Ash Pond)					32	876,500	627,000	250,000	2006	52	65		
	GYPSUM DISPOSAL AREA 2A	Not Rated		1981 (originally fly ash settlement pond) 2008 (Gypsum Disposal Area)	Not Expanded	FLUE GAS EMISSION CONTROL RESIDUALS (Flows to fly ash pond)					42	2,743,000	896,000	1,847,000	2006	45	165		
rt Fossil Plant	DISPOSAL AREA 5	LOW	TVA	1983	1990 - converted to dry ash operation	FLY ASH, potentially ammoniated.	Mar-08	2009	Disposal Area 5 - reported annual maintenance items include: cover and vegetate stack slopes semi-annually, repair erosion as needed, regrade perimeter ditch as needed. Ash Pond 4 - joint sealant applied to RCP spillway riser joints annually, semi-annual mowing of dike slopes, reportedly applied tree killer substance to sparse tress on west side of pond last year (trees not yet removed though), weekly monitoring of seepage areas.	NR	75	8,800,000	6,765,000	2,035,000	2006	120	135	NR	Owner - United States, Operator - TVA
	ASH POND 4	LOW	TVA	1972	1984	Bottom ash, fly ash (historical)					45	2,200,000	1,159,000	1,041,000	2006	40	40		
	DISPOSAL AREA 5 BASIN	Not Rated		1983	N/A	Fly Ash					12	600,000	150,000	450,000	2006	17	17		
berland Fossil Plant	DRY ASH STACK	Not Rated		1969	Dry Ash stacking began in mid- 1990s over old pond	FLY ASH/BOTTOM ASH	Feb-09	2009	Maintenance activities needed include repairs for erosion, monitoring seepage, tree removal, clearing and cleaning inner slopes and perimeter ditches, repair of animal burrows, establishing vegetation in exposed areas, and recommendations for construction of the current gypsum dikes.	NR	110	12,600,000	4,781,000	7,819,000	2006	35	200	NR	Owner - United States, Operator - TVA
	ASH POND	LOW	TVA	1969	Dikes raised in 1979	Bottom ash, fly ash (historical)					50	2,000,000	1,305,000	695,000	2006	35	35		
	GYPSUM STORAGE AREA	LOW	TVA	1969	Gypsum area constructed over old pond in mid 1990s	FLUE GAS EMISSION CONTROL RESIDUALS					170	20,000,000	1,826,000	18,174,000	2006	60	140		
in Fossil Plant	FLY ASH POND E	LOW	TVA	1970	1986 - Divider Dike Constructed Forming Ponds A and E; 2006 - Pond E Expanded	FLY ASH, bottom ash. E flows to C.	2008	2009	Annual maintenance items reported by GAF include: annual seeding of new dikes for Pond E, mow along Pond E dike slopes beneath power lines along river.	NR	157	7,100,000	4,968,000	2,132,000	2006	30	35	NR	Owner - United States, Operator - TVA
	BOTTOM ASH POND A	LOW	TVA	1970	1986 - Divider Dike Constructed Forming Ponds A and E; 1994 - Divider dike raised	BOTTOM ASH; A flows to B					269	7,083,000	4,951,409	2,131,591	2006	25	25		
	STILLING POND B, C & D	Not Rated		1970	1986 - Ponds B and C formed when divider dike constructed to form Ash Ponds A and E	FLY ASH & BOTTOM ASH and other listed in E.					55	600,000	400,000	200,000	2006	10	10		
Sevier Fossil Plant	DRY ASH STACK	Not Rated		1955 (former ash ponds)	1979 - all sluicing stopped, designated for dry ash disposal	FLY ASH	Nov-07	2009	(A) to monitor the exterior dikes slopes and toe areas of all disposal areas for surface sloughs, new seepage area, changes in existing seeps, or movements; (B) continuation of mowing program and prevention of tree growth on dikes; (C) cover exposed slopes with earth, seed, fertilize and mulch as described in the operations manual; (D) removal of sediment from Coal Yard Drainage basin; (E) reclaim animal burrows.	NR	84	3,800,000	2,098,000	1,702,000	2006	101	143	NR	Owner - United States, Operator - TVA
	BOTTOM ASH POND	LOW	TVA	1979	Not Expanded	BOTTOM ASH, FLY ASH					26 (pond area only) 41 (total area)	1,200,000	1,035,293	165,000	2006	25	25		

PLANT	FACILITY	HAZARD POTENTIAL CLASSIFICATION See footnote #1	Hazard Rating Performed By	YR MGT UNIT COMMISSIONED See footnote #2	YR MGT UNIT EXPANDED	MATERIALS CONTAINED IN UNIT	LAST TVA ASSESSMENT	NEXT SCHEDULED TVA ANNUAL INSPECTION	ACTIONS TAKEN OR PLANNED RESULTING FROM LAST ANNUAL INSPECTION	ISSUES REPORTED BY STATE OR FEDERAL ASSESSMENTS AND ACTIONS TAKEN See footnote #3	SURFACE AREA (ACRES)	TOTAL STORAGE CAPACITY (Cubic Yards)	CURRENT VOLUME OF MATERIAL (Cubic Yards)	REMAINING CAPACITY (Cubic Yards)	DATE VOLUME TAKEN	CURRENT HEIGHT (FT)	FUTURE MAX. HEIGHT (FT)	KNOWN SPILLS OR UNPERMITTED RELEASES (SURFACE WATER/LAND) See footnote #4	CURRENT LEGAL OWNER(S)) & OPERATOR(S)) AT FACILITY
Johnsonville Fossil Plant	**ASH DISPOSAL AREA 2**	LOW	TVA	1970	1978	FLY ASH & BOTTOM ASH	Nov-07	2009	Recommendations include maintenance activities: filling animal burrows, repairing erosion, filling in depressed areas, clearing heavy vegetation, and tree removal. Additionally, also monitoring seepage.	NR	87	4,360,000	4,164,000	199000	2006	30	30	Reported release of small quantity of ceneospheres on March 27, 2004 when discharge structure was disturbed during maintenance.	Owner - United States, Operator - TVA
Kingston Fossil Plant	**MAIN ASH POND**	LOW	TVA	1951	1968 - raised dike	FLY ASH & BOTTOM ASH	Oct-08	2009	Standard recommendations were to repair all erosion ditches, repair wheel ruts, remove floating ash from the pond to prevent a permit violation, remove trees from dikes and mow the dikes regularly to control the growth of vegetation. Repair broken monitoring wells along Swan Pond Road, monitor seeps and under drains.	NR	92	14,370,000	UNKNOWN	UNKNOWN	NA	50	UNKNOWN	November 7, 2003 and November 1, 2006, an ash release occurred to land from a slough in the Dredge Cell embankment. A release into the Emory River occurred on December 22, 2008 from the Dredge Cell embankment failure. No reports found of releases from the Main Ash Pond or Stilling Basin.	Owner - United States, Operator - TVA
	STILLING POND	LOW See footnote 1	TVA	1978	Not Expanded	Materials from main ash pond					29	468,000	260,000	208,000	2006	50	50		
Paradise Fossil Plant	**SCRUBBER SLUDGE COMPLEX (Gypsum Stack and Scrubber Sludge Stilling Pond)**	LOW	TVA	1986	Not Expanded	FLY ASH, FLUE GAS EMISSION CONTROL RESIDUALS	Oct-08	2009	With respect to dam safety, primarily minor concerns (rutting, erosion, vegetation, etc.) were identified in the report. The under drain ditch at the Gypsum Stack needs to be cleaned out to prevent flow over the road. Several seeps were noted at the Daniel Run Pond 3, but were not flowing. Recommended removal of fines from the Coal Yard Runoff Ponds and all of the Red Water Ponds.	NR	255	858,000	11,783,000	35,074,000	2006	62	270	NR	Owner - United States, Operator - TVA
	FLY ASH EXTENSION AREA POND (Peabody Ash and Stilling Pond and Jacob's Creek Fly Ash and Stilling Pond)	LOW	TVA	1971	1997	FLY ASH					203	6,348,000	2,956,000	3,392,000	2006	34	34		
	SLAG AREAS 2A & 2B	LOW	TVA	1967	1970	BOTTOM ASH. A portion of the flow is routed to the fly ash extension area pond.					27	968,000	752,000	216,000	2006	24	24		
Shawnee Fossil Plant	**CONSOLIDATED WASTE DRY STACK**			1984	Horizontal expansion design prepared in 2000	FLY ASH/BOTTOM ASH. Drains to ash pond	Feb-09	2009	Maintenance activities needed include repairs for erosion, monitoring seepage, tree removal, clearing and cleaning inner slopes, repair of animal burrows, establishing vegetation in exposed areas, monitoring animal paths, repairing leaking raw water line, removing sediment build-up, and recommendations for regrading intake channel dredge cell.	NR	200	33,194,000	22,811,000	10,382,000	2006	100	270	NR	Owner - United States, Operator - TVA
	ASH POND	LOW	TVA	1952	Area 2 was constructed in 1971 and the dikes were raised in 1979	FLY ASH/BOTTOM ASH					180	5,000,000	4,712,000	287,000	2006	25	25		
Widows Creek Fossil Plant	**ASH POND (Complex consists of Bottom Ash Stack, Iron Pond, Cooper Pond, Old Scrubber Sludge Pond (Dredge Cell), Asbestos Waste Disposal Area, Pump Pond, Upper and Lower Stilling Ponds)**	LOW	TVA	1950	During 2005, a dredge cell was constructed over the old scrubber sludge pond area. During 2007 dredging ceased.	FLUE GAS EMISSION CONTROL RESIDUALS, FLY ASH & BOTTOM ASH	Oct-08	2009	Review with the Constructor the Gypsum Stack operations manual and drawings to ensure the operations continue in accordance with the current stacking plan, monitor the wet area along the southern lower perimeter dike, rework a portion of the west slope next to the Stilling Pond, install sub drains on the west slope adjacent to the Gypsum Stilling Pond, uncover the slope drains on the 650/655 bench and grade per design drawings. In regards to the Wet Gypsum Stacking Stilling Pond, the planned actions are to consider and alternate skimmer design on TVA drawing 10W235-19. In regards to the Pump Pond, the planned actions are to monitor the seep in the dike between the Stilling Pond and the Pump Pond. In regards to the Active Ash Pond, the planned actions are to monitor the seepage along the south Perimeter dike next to the stilling pond.	NR	310	18,890,000	1,856,000	17,034,000	2006	50	115	Reported release of small quantity of ceneosperes from the Ash Pond which occurred on December 10, 2004 due to intense precipitation. Reported release of small quantity of ceneospheres from the Ash Pond which occurred on January 30, 2008. An abandoned decant weir in Pond 2B of the Gypsum Stack failed on January 9, 2009.	Owner - United States, Operator - TVA
	GYPSUM STACK (Wet Stacking Area)	LOW	TVA	1986	Phase I vertical expansion occurred from 1986 to 1992. Phase II horizontal expansion began in 1992.	FLUE GAS EMISSION CONTROL RESIDUALS, FLY ASH & BOTTOM ASH					110	17,683,000	7,892,000	9,791,000	2006	75	150		
Watts Bar Fossil Plant (Inactive)	**ASH POND and STILLING BASIN**	LOW	TVA	1974	1977	Previous fly ash, bottom ash	Feb-09	2009	Complete Closure Plan - currently construction is approximately 95 percent complete.	NR	14	230,000	150000	80,000	2006	30	30	NR	Owner - United States, Operator - TVA

Notes:
1. Hazard Potential listed for those facilities previously rated by TVA, all facilities are currently under evaluation. Based on hindsight at Kingston Fossil Plant, the ranking did not adequately represent the actual risk experienced on 12/22/2008.
2. Year Management Unit Commissioned approximated from available reports, drawings, or permit documents.
3. NR - None Reported
4. Does not include NPDES permit exceedences

APPENDIX A

Document 4

NPDES Permit

Document Type: NPDES Permits/Applications

STATE OF TENNESSEE
DEPARTMENT OF ENVIRONMENT AND CONSERVATION
401 CHURCH STREET
L & C ANNEX 6TH FLOOR
NASHVILLE TN 37243-1534

November 30, 2007

Mr. Gordon G Park
Manager of Environmental Affairs
Tennessee Valley Authority
5D Lookout Place
1101 Market Street
Chattanooga, TN 37402

Subject: **Modified 316(b) Requirements for NPDES Permit No. TN0005789**
TVA - Cumberland Fossil Plant
Cumberland City, Stewart County, Tennessee

Dear Mr. Park:

In accordance with the provisions of the Tennessee Water Quality Control Act, Tennessee Code Annotated, Sections 69-3-101 through 69-3-120, the Division of Water Pollution Control hereby issues the enclosed modified NPDES Permit. The continuance and/or reissuance of this NPDES Permit is contingent upon your meeting the conditions and requirements as stated therein.

This modification is subject only to those provisions as detailed in the Addendum to Rationale found in the permit. Specifically, this modification removes the reporting requirement of the January 7, 2008 Comprehesive Demonstration Study (CDS) that was previously required by the now suspended federal rule known as 316(b). However, some remaining and additional requirements and details of this modification are found in the addendum. The previously issued permit (30, November 2005) should be archived and this modified permit document placed into service effective January 1, 2008.

Please be advised that you have the right to appeal any of the provisions established in this NPDES Permit, in accordance with Tennessee Code Annotated, Section 69-3-105(i), and the General Regulations of the Tennessee Water Quality Control Board. If you elect to appeal, you should file a petition within thirty (30) days of the receipt of this permit.

If you have questions, please contact the Division of Water Pollution Control at your local Field Office at 1-888-891-TDEC; or, at this office, please contact Ms. Pamala Myers at (615) 532-0654 or by E-mail at *Pamala.Myers@state.tn.us*.

Sincerely,

Wade D. Murphy

Wade D. Murphy
Acting Manager, Permit Section
Division of Water Pollution Control

DEC 0 5 2007

WDM/prm/wdm

Enclosure

STATE OF TENNESSEE

NPDES PERMIT

No. TN0005789

MODIFICATION: November 30, 2007

Authorization to discharge under the
National Pollutant Discharge Elimination System (NPDES)

Issued By

Tennessee Department of Environment and Conservation
Division of Water Pollution Control
401 Church Street
6th Floor, L & C Annex
Nashville, Tennessee 37243-1534

Under authority of the Tennessee Water Quality Control Act of 1977 (T.C.A. 69-3-101 et seq.) and the delegation of authority from the United States Environmental Protection Agency under the Federal Water Pollution Control Act, as amended by the Clean Water Act of 1977 (33 U.S.C. 1251, et seq.)

Discharger: **TVA - Cumberland Fossil Plant**

is authorized to discharge: **ash transport water, treated chemical and nonchemical metal cleaning wastewaters, coal pile runoff, low volume wastes, and storm water runoff through Outfall 001, once through condenser cooling water, miscellaneous equipment cooling and lubricating water, and storm water through Outfall 002, intake screen backwash water through Outfall 004, and chemical and nonchemical metal cleaning wastewaters through Outfall 007**

from a facility located: **in Cumberland City, Stewart County, Tennessee**

to receiving waters named: **Cumberland River at mile 103**

in accordance with effluent limitations, monitoring requirements and other conditions set forth herein.

This permit shall become effective on: **January 1, 2008**

This permit shall expire on: **May 31, 2010**

Issuance date: **November 30, 2007**

Wade P. Murphy / for
Paul E. Davis, Director
Division of Water Pollution Control

RDAs 2352 and 2366

CN-0759

APPENDIX A

Document 5

Hydrologic & Hydraulic Analysis, Stantec, September 28, 2010

Memo

Stantec

To:	Scott Turnbow Chattanooga, TN	From:	Matthew Hoy, PE St. Louis, MO
File:	175609014	Date:	September 28, 2010

Reference: Hydrologic and Hydraulic Calculations Summary
Spillway Improvement Project
Cumberland Fossil Plant (CUF)
Main Ash Pond

The purpose of this memorandum is to summarize the hydrologic and hydraulic calculations supporting design of spillway improvements at the CUF Main Ash Pond. Detailed design calculations and descriptions will be provided with the final spillway design report and calculation package. Preliminary design drawings will be submitted to TVA on 10/4/2010.

BACKGROUND

A hydrologic and hydraulic analysis was conducted for the Cumberland Fossil Plant (CUF) Ash Pond Complex in support of the spillway improvement project. This pond complex consists of the Ash Pond and the Ash Stilling Pond. These ponds act as settling basins for the ash slurry that is discharged from the plant, as well as stormwater detention for runoff from the Gypsum Storage Area, Dry Ash Stack and pumped runoff from the Coal Yard. Four existing spillway risers are located in the Ash Stilling Pond and direct flow from the Ash Stilling Pond to the spillway discharge channel which flows to the Cumberland River.

WATERSHED & PROCESS FLOW

The area draining to the Ash Pond Complex includes direct runoff from 300 acres including the Ash Pond Complex, the Gypsum Storage Area, and the Dry Ash Stack, in addition to pumped inflow from 160 acres including the Coal Yard and plant facilities. The daily plant process flow averages roughly 23 million gallons per day (MGD).

OUTLET DESCRIPTION

Flow discharges from the ash pond through four (4) 48-inch diameter concrete riser structures connected to 36-inch diameter concrete outlet pipes through the dike. These structures are roughly 23 feet tall and are laterally unsupported. There are currently no other defined emergency spillways or overflow paths. The outlets discharge into a spillway discharge channel that flows to the Cumberland River.

Reference: Hydrologic and Hydraulic Calculations Summary
Spillway Improvement Project
Cumberland Fossil Plant (CUF)
Main Ash Pond

FREEBOARD

TVA's Master Programmatic Document requires 5 ft of operating freeboard for ash pond facilities. The perimeter dike crest elevation is 394 ft, and the Ash Pond water surface elevation is currently maintained at 384.2 ft, resulting in an operating freeboard of 9.8 ft. This facility currently meets freeboard requirements.

METHODOLOGY SUMMARY

The 6-hour PMP rainfall depth of 35.4 inches for Stuart County, Tennessee was determined and the SCS 6-hour rainfall distribution was applied to this depth. This depth was estimated from a map on page 46 of *Hydrometeorological Report No. 56; Probable Maximum and TVA Precipitation Estimates With Areal Distribution for Tennessee River Drainage Less Than 3,000 Mi^2 in Area* by US Department of Commerce. The 24-hour Type II SCS distribution was applied to 100-yr, 24-hr rainfall depth of 7.8 inches obtained from *Technical Paper No. 40; Rainfall Frequency Atlas of the United States* by US Department of Commerce. The SCS curve number method was used to convert this rainfall into runoff. Composite curve numbers ranging from 84 to 99 were assigned to subbasins within the watershed based on landuse and soil characteristics. Lag time assumptions were based on topographic mapping and ranged from 3 minutes to 13 minutes. Stage-storage relationships for the main ash pond complex were developed using one-foot contour data and a hydrographic survey provided by TVA. This data was input into a Hydrologic Engineering Center Hydrologic Modeling System (HEC-HMS) version 3.4 model of the watershed to develop an inflow hydrograph to the ash stilling pond. Standard hydraulic equations were used to develop a rating curve for the existing spillways and level-pool routing methodology was used to route the design storm through the outlets

DESIGN STORM PERFORMANCE

TVA's Master Programmatic Document requires ash pond facilities to convey the 6-hour PMP event without overtopping. Results of the H & H analysis of the ash pond complex indicate that the existing spillways are unable to convey the 6-hr PMP event without overtopping the perimeter dike. Results of storm routings are summarized in Table 1. Supporting data are provided as Attachment A and Attachment B to this memorandum and detailed supporting calculations for design conditions will be submitted with the calculation package for the design submittal.

Reference: Hydrologic and Hydraulic Calculations Summary
Spillway Improvement Project
Cumberland Fossil Plant (CUF)
Main Ash Pond

Table 1 – Freeboard and Routing Summary

	Pre-design Conditions	Post-design Conditions
Drainage Area (ac)	460	460
Crest of Dam (ft)	394	394
Normal Pool Elevation (ft)	384.2	378.2
Normal Operating Freeboard (ft)	9.8	15.8
Normal Operation Flow (MGD)	23	23
100-yr, 24-hr Storm Precipitation Depth (in)	7.83	7.83
100-yr, 24-hr max. water surface elevation (ft)	386.7	380.8
Design Storm	6-hr PMP	6-hr PMP
Design Storm max. water surface elevation (ft)	Overtopping	393.9

FUTURE MODIFICATIONS

Because the existing spillway system is not capable of passing the 6-hr PMP design storm, a spillway improvement project is currently under design to ultimately retrofit the current tall, unsupported structures and supplement with a 35 ft wide concrete emergency spillway at elevation 390 ft. Siphon spillways will be installed to lower the permanent pool elevation and allow for cleaning and inspecting of the existing primary spillways. If the structures are found to be in adequate structural condition, the top 6 feet of the riser will be sawcut and removed to allow for permanent lowering of the pool elevation as well as reduce the height of the risers to 16 ft. The risers and outlet pipes may also be lined if necessary. Analysis indicates this design will convey the 6-hour PMP storm event without overtopping the dike in addition to maintaining greater than the required operating freeboard for the facility as the proposed water surface elevation is 378 ft.

STANTEC CONSULTING SERVICES INC.

Matthew Hoy, PE
Senior Project Engineer
Matthew.Hoy@stantec.com

Attachment A: Watershed Map

Attachment B: Hydrologic and Hydraulic Model Input / Output

APPENDIX A

Document 6

Slope Stability Evaluation – Ash Pond, Stantec, March 2010

Only the cover letter, table of contents and executive summary are contained herein. The full report is contained in the Draft report.

Stantec

Report of Geotechnical Exploration and Slope Stability Evaluation

Ash Pond
Cumberland Fossil Plant
Stewart County, Tennessee

Stantec Consulting Services Inc.
One Team. Infinite Solutions
11687 Lebanon Road
Cincinnati, Ohio 45241-20026
Tel: (513) 842-4200 • Fax: (513) 842-8250
www.stantec.com

Prepared for:
Tennessee Valley Authority
Chattanooga, Tennessee

March, 2010

Stantec

Stantec Consulting Services Inc.
1409 North Forbes Road
Lexington KY 40511-2050
Tel: (859) 422-3000
Fax: (859) 422-3100

March 29, 2010
File: 175539016R01

Mr. Barry S. Snider, PE
Tennessee Valley Authority
1101 Market Street
LP2N
Chattanooga, Tennessee 37402

Re: Report of Geotechnical Exploration and Slope Stability Evaluation
Ash Pond
Cumberland Fossil Plant
Stewart County, Tennessee

Dear Mr. Snider:

Stantec Consulting Services Inc. (Stantec) has completed a geotechnical exploration of the Ash Pond at the Coal Combustion Product Disposal Complex at the Cumberland Fossil Plant. Our final report includes discussions of general site conditions, scope of work performed, subsurface conditions, and results of laboratory testing and our engineering analyses.

The report also includes a review of historical documentation provided by TVA, and our conclusions and recommendations relative to future use of the facility. These services were performed under Engineering Service Request ESR/TAO 894 in accordance with the terms and provisions established in our System-Wide Services Agreement dated December 22, 2008.

Stantec appreciates the opportunity to provide engineering services for this project. If you have any questions, or if we may be of further assistance, please contact our office.

Sincerely,

STANTEC CONSULTING SERVICES INC.

Daniel B. Rogers, PE
Project Engineer

Stan A. Harris, PE
Principal

Report of Geotechnical Exploration and Slope Stability Evaluation

Ash Pond
Cumberland Fossil Plant
Stewart County, Tennessee

Prepared for:
Tennessee Valley Authority
Chattanooga, Tennessee

March, 2010

Report of Geotechnical Exploration
Ash Pond
Cumberland Fossil Plant
Stewart County, Tennessee

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Appendix

Executive Summary

Stantec Consulting Services Inc. (Stantec) has completed a Geotechnical Exploration and Slope Stability Evaluation of the Ash Pond Complex at the Cumberland Fossil Plant. This study was performed to evaluate slope stability and seepage for existing conditions at the Ash Pond.

Background Information

The Ash Pond Complex is approximately 50 acres in area. It consists of the Retention Pond and the Stilling Basin, contiguous structures located on the north end of the larger coal combustion product (CCP) waste disposal complex. The Ash Pond receives effluent from the Bottom Ash Pond and also receives runoff from the perimeter ditch system which surrounds the Gypsum Disposal Area Complex. With a total length of approximately 4,200 feet, the dike system that surrounds the Ash Pond has a maximum height of about 36 feet above Wells Creek. The dike system was constructed with approximately 3H:1V slopes, but isolated areas are slightly steeper (2.8:1V).

Water in the Retention Pond generally flows to the northwest and exits to the Stilling Basin through a 100-foot wide opening in the dike separating the two structures. A floating boom spans the opening and aids in removal of floating solids. Decanted water discharges from the Stilling Basin through four 36-inch pipe spillways. Each spillway has a 48-inch concrete riser with a 120-inch diameter corrugated steel pipe skimmer. The spillways drain into a concrete discharge channel that leads to the main plant discharge channel and Lake Barkley.

TVA has classified the Ash Pond Complex as a "high hazard" facility due to the potential for damage to the downstream state highway and bridge should a failure of the impoundment occur. Currently, Stantec and TVA are in the early stages of performing a detailed study to more accurately determine the downstream impacts resulting from a failure. Stantec is also in the process of studying modifications and/or replacements of the existing Ash Pond spillways. One of the outcomes of these efforts may be lowering the pool elevation by several feet.

According to historical documents provided by TVA, a seep was noted in 1974 through the dike along the western side of the Retention Pond. A repair was performed consisting of placing a 40-foot wide clay seal on the interior of the dike. The area is monitored annually and no further seepage has been noted. There are no other reported cases of seepage or slope instability.

Scope of Geotechnical Exploration

This study began with a review of TVA-provided historical information along with site inspections. A geotechnical exploration program was then developed and executed. The exploration consisted of drilling 30 soil test/sample borings at 16 locations. Piezometers were installed at 7 locations and slope inclinometer casings at three locations. Drilling locations were positioned along eight cross sections around the Retention Pond and the Stilling Basin. Laboratory testing performed included moisture content, classification,

permeability and shear strength testing to establish key index properties and strength parameters.

Results of Exploration and Engineering Analyses

Eleven primary soil horizons were identified from the field and laboratory program. These primary horizons generally fall into one of three categories: 1) natural foundation soils which included alluvial clay and alluvial sands and gravels, 2) dikes constructed with natural clays and varying amounts of gravel, and 3) coal combustion byproducts including fly ash and bottom ash.

Following the drilling and laboratory testing program, seepage and slope stability analyses were performed to quantify factors of safety for current conditions. The dikes were assessed under static, long-term steady state conditions since the dikes have been in their current configuration for a long time. Analyses were performed on eight sections.

Phreatic surfaces predicted by the seepage analyses were generally in good agreement with levels measured in piezometers installed as part of this study. At three locations, Sections P, Q and R, the calculated factor of safety against piping was found to be less than the recommended acceptable minimum value of 4. Results of the slope stability analyses indicates factors of safety against long-term slope stability failure are greater than the target value of 1.5. If the pond water level or top of dike elevations are lowered in the future, the factors of safety would tend to increase.

Two alternatives have been proposed to increase the factor of safety against piping at Sections P, Q and R. The first is to construct a toe berm along the banks of Wells Creek. This method would likely require obtaining 401/404 permits. A second alternative would be installing a sheet pile cutoff wall along the interior side of the dike system. It is recommended that a final decision on the mitigation option to follow not be made until Stantec's breaching spillway studies are completed. Changes resulting from those studies, such as the operating pond level, would have an impact on the design of remedial measures.

APPENDIX A

Document 7

Dry Fly Ash Stack and Gypsum Report, Stantec, June 2010

Only the cover letter, table of contents and executive summary are contained herein. The full report is contained in the Draft report.

Stantec

Report of Geotechnical Exploration

Dry Fly Ash Stack and Gypsum Disposal Complex
Cumberland Fossil Plant
Stewart County, Tennessee

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Prepared for:
Tennessee Valley Authority
Chattanooga, Tennessee

June, 2010

Stantec

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June 11, 2010
File: 175539009R01

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

Re: Report of Geotechnical Exploration
Dry Fly Ash Stack and Gypsum Disposal Complex
Cumberland Fossil Plant
Stewart County, Tennessee

Dear Mr. Turnbow:

Stantec Consulting Services Inc. (Stantec) has completed a geotechnical exploration of the Dry Fly Ash Stack and Gypsum Disposal Complex at the Cumberland Fossil Plant. Our final report includes discussions of general site conditions, scope of work performed, subsurface conditions, and results of laboratory testing and our engineering analyses. The report also includes a review of historical documentation provided by TVA, and our conclusions and recommendations relative to future use of the facility. These services were performed under Engineering Service Request ESR/TAO 700 in accordance with the terms and provisions established in our System-Wide Services Agreement dated December 22, 2008.

Stantec appreciates the opportunity to provide engineering services for this project. If you have any questions, or if we may be of further assistance, please contact our office.

Sincerely,

STANTEC CONSULTING SERVICES INC.

Daniel B. Rogers, PE
Project Engineer

Stan A. Harris, PE
Principal

/lfb

Report of Geotechnical Exploration

Dry Fly Ash Stack and Gypsum Disposal Complex
Cumberland Fossil Plant
Stewart County, Tennessee

Prepared for:
Tennessee Valley Authority
Chattanooga, Tennessee

June, 2010

Report of Geotechnical Exploration
Dry Fly Ash Stack and Gypsum Disposal Complex
Cumberland Fossil Plant
Stewart County, Tennessee

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Appendix

Executive Summary

Stantec Consulting Services Inc. (Stantec) has completed a Geotechnical Exploration of the Dry Fly Ash Stack and Gypsum Disposal Complex at Cumberland Fossil Plant. This study was performed to evaluate slope stability and seepage for existing conditions of the disposal areas and surrounding dikes.

Background Information

The Gypsum Disposal Complex is approximately 100 acres in area. It was constructed in 1995-1996 over Area No.1, which was the original ash pond. Approximately 1,000,000 tons of gypsum is produced each year. Roughly, 75 percent of the gypsum is marketed to the adjacent wallboard company and the remaining 25 percent is sent to the Gypsum Disposal Complex. The complex is formed by a series of earth dikes around its perimeter and an upper gypsum dike. The total height of the facility is approximately 50 feet. Dike slopes generally vary from 2H:1V to 3H:1V.

TVA has classified the Gypsum Disposal Complex as a "high hazard" facility due to the consequences of failure relative to potential damage to the adjoining wallboard plant. Currently, Stantec and TVA are in the early stages of preparing a 5 to 7 year operation plan for the facility while a new dry disposal facility is being designed, permitted and constructed. Modifications being considered include constructing two small lined ponds on top of the gypsum stack and significantly reducing the amount of water which could be impounded.

A small landslide occurred on the facility in 2005 and temporary stabilization measures were implemented by TVA. Stantec has developed construction drawings for permanent repairs, which include the construction of a seepage collection system and the placement of a more substantial rock buttress. Other historical geotechnical issues on the gypsum disposal complex include seepage at various locations around the stack. Since May 2009, TVA has not been sending gypsum sluice to the stack except when the dewatering plant experiences outages.

The Dry Fly Ash stack is approximately 110 acres in size. It is also built over the original ash pond. Its current height is about 35 feet and slopes generally vary from 2.5H:1V to 3H:1V. A small dredge cell within the Dry Fly Ash Stack was filled with dredged coal fines from the Coal Yard Drainage Basin in 2007. Stantec performed an analysis of this area in early 2009 and concluded that is presence would not have a detrimental effect on the long-term stability of the stack. It was recommended that TVA excavate parallel trenches across the area and backfill the trenches with more permeable bottom ash. This work was completed on April 24, 2009.

Scope of Geotechnical Exploration

This study began with a review of TVA-provided historical information along with site inspections. A geotechnical exploration program was then developed and executed. The exploration consisted of drilling soil test/sample borings at 74 locations and advancing cone penetrometer test borings at 17 locations. Piezometers were installed at 19 locations and slope inclinometer casings at eight locations. Drilling locations were positioned along fifteen cross sections around the Dry Fly Ash Stack and the Gypsum Disposal Complex. Laboratory

testing included moisture content, classification, permeability and shear strength testing to establish key index properties and strength parameters.

Results of Exploration and Engineering Analyses

Thirteen primary soil horizons were identified from the field and laboratory program. These primary horizons generally fall into one of three categories: 1) natural foundation soils, which included alluvial clay and alluvial sands and gravels, 2) dikes constructed with natural clays and varying amounts of gravel, and 3) coal combustion byproducts including fly ash, bottom ash and gypsum.

Following the drilling and laboratory testing program, slope stability analyses were performed to quantify factors of safety for current conditions. The dikes were assessed under static, long-term steady state conditions since the dikes have been in their current configuration for a long time. Analyses were performed on fifteen sections. Factors of safety for slope stability were computed using Spencer's method of analysis, optimized curved failure surfaces, and search routines that help to identify the critical (minimum factor of safety) failure surface. The slope stability models were evaluated using phreatic surfaces based on piezometric readings and field observations. In their new Master Programmatic Document, TVA has adopted a minimum target factor of safety of 1.5 against slope failure based on U. S. Army Corps of Engineers (USACE) criteria. Factors of safety ranged from approximately 1.0 to 2.5. Lower than acceptable values were determined for eight of the fifteen sections analyzed. For the most part, the lower factors of safety correspond to sloughing shallow disturbance, not massive dike failures.

Selected cross sections were also analyzed for short-term (undrained) conditions. Acceptable factors of safety were obtained for these analyses. Furthermore, undrained analyses were performed for future increases in stack heights. These analyses were performed assuming instantaneous loading of the stack and no pore pressure dissipation. Acceptable results were obtained for the Gypsum Stack. The analyses for the Dry Ash Stack indicate that 12.5 to 20.0 feet of material can be placed quickly before FS values fall below acceptable levels.

Conclusions and Recommendations

Work Plans should be developed to improve the long-term slope stability factor of safety at Sections A, B and F on the Dry Ash Stack. Re-grading only is needed at Sections A and B. At Section F, a toe buttress and slope flattening are needed.

A Work Plan has been developed for slope repair at Section H of the Gypsum Disposal Complex. This Work Plan has been issued for construction and should be implemented as soon as practical.

A Work Plan should be developed to construct toe buttresses below the bottom ash road dike around the Gypsum Disposal Complex. This work should be coordinated with re-grading of the perimeter ditch system to promote improved surface drainage and reduce ponding of water.

Stantec recommends that full time sluicing of gypsum slurry to the Gypsum Disposal Complex not be resumed until lined ponds are constructed on top of the stack. These lined ponds will prevent the sluice water from infiltrating the stack.

A study should be performed of the Dry Ash Stack to determine if it is in full compliance with the existing permit. If not, a study should be performed to determine if it would be preferable to redesign the stack or to re-grade it so that it is in compliance.

Additional piezometers are recommended for the Dry Ash Stack to provide for better definition of phreatic levels and to monitor pore pressures during future fill placement.

Fill material should not be placed over phragmites or other vegetation. Fine ash or gypsum dipped from ponds should not be placed near the toe of slopes or concentrated at any one location. Fines should be dispersed evenly across the interior of the active stacks and not be placed near the edges.

Operations and Maintenance Manuals should be developed or updated for each facility. Elements of a maintenance program should include elimination of animal burrows, a mowing program, repair of erosion areas and a regular inspection program. A program should be established to develop record (as-built) drawings and construction records for future maintenance and construction activities.

An instrumentation program should be developed for the site. The program should include regular collection and analysis of various data including phreatic levels, rainfall and slope movements.

APPENDIX A

Document 8

Seismic Slope Stability, Stantec, September 2011

Stantec Consulting Services Inc.
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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

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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

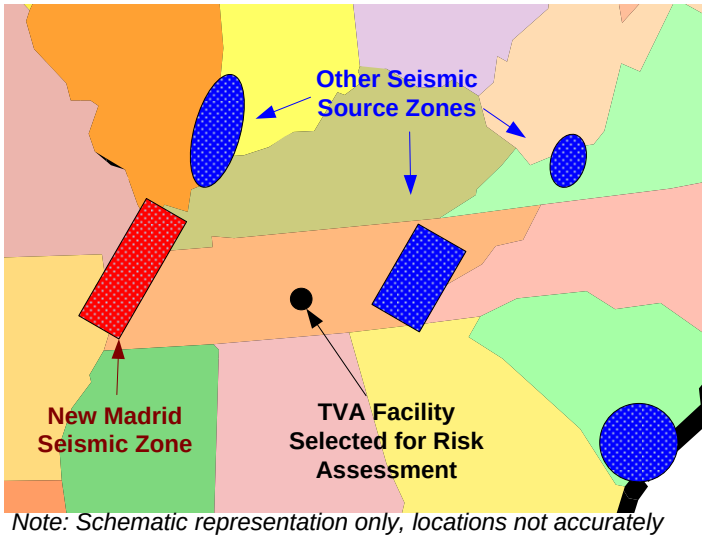
TVA Risk Event Probability Rating Scale		
Score	Rating	Description
5	Virtually Certain	95% probability that the event will occur in the next 3 years /10 years
4	Very Likely	75% probability that the event will occur in the next 3 years/10 years
3	Even Odds	50% probability that the event will occur in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

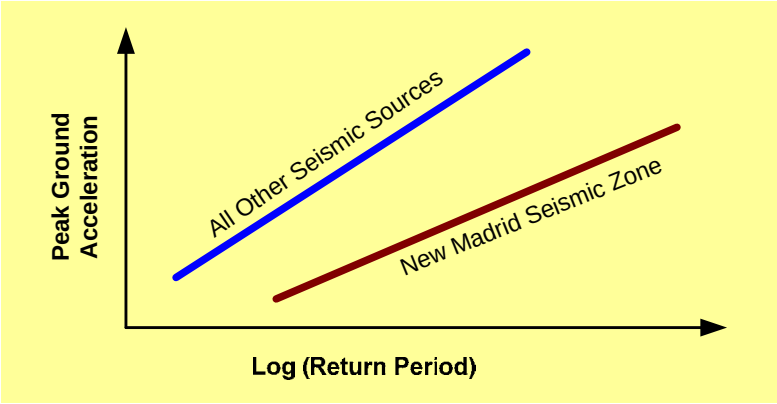


Figure 2. Typical Seismic Hazard Curves for Proposed Probabilistic Assessment of TVA Facilities

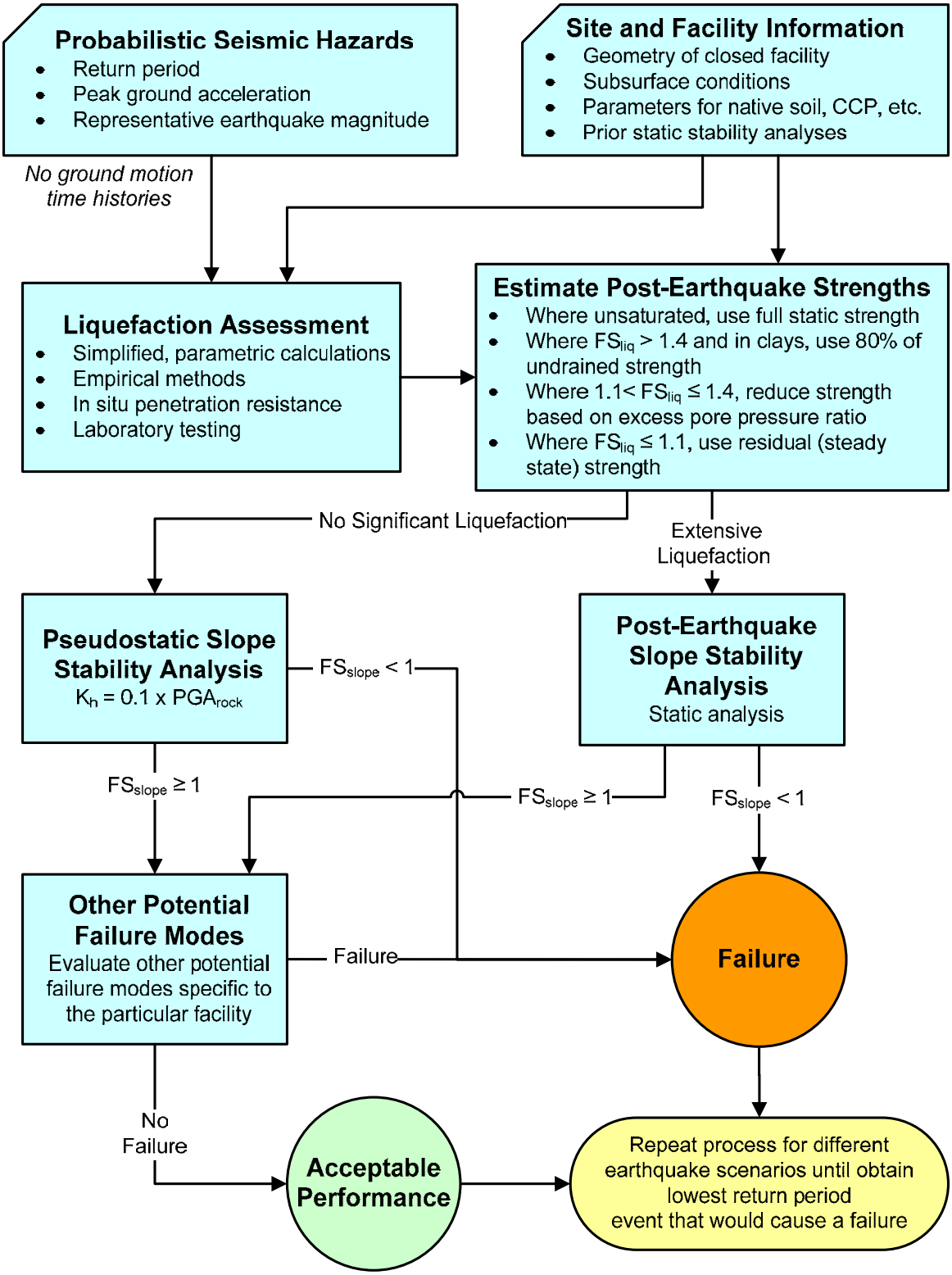


Figure 3. Simplified Flowchart for Assessing Facility Performance During a Probabilistic Seismic Event in Phase A

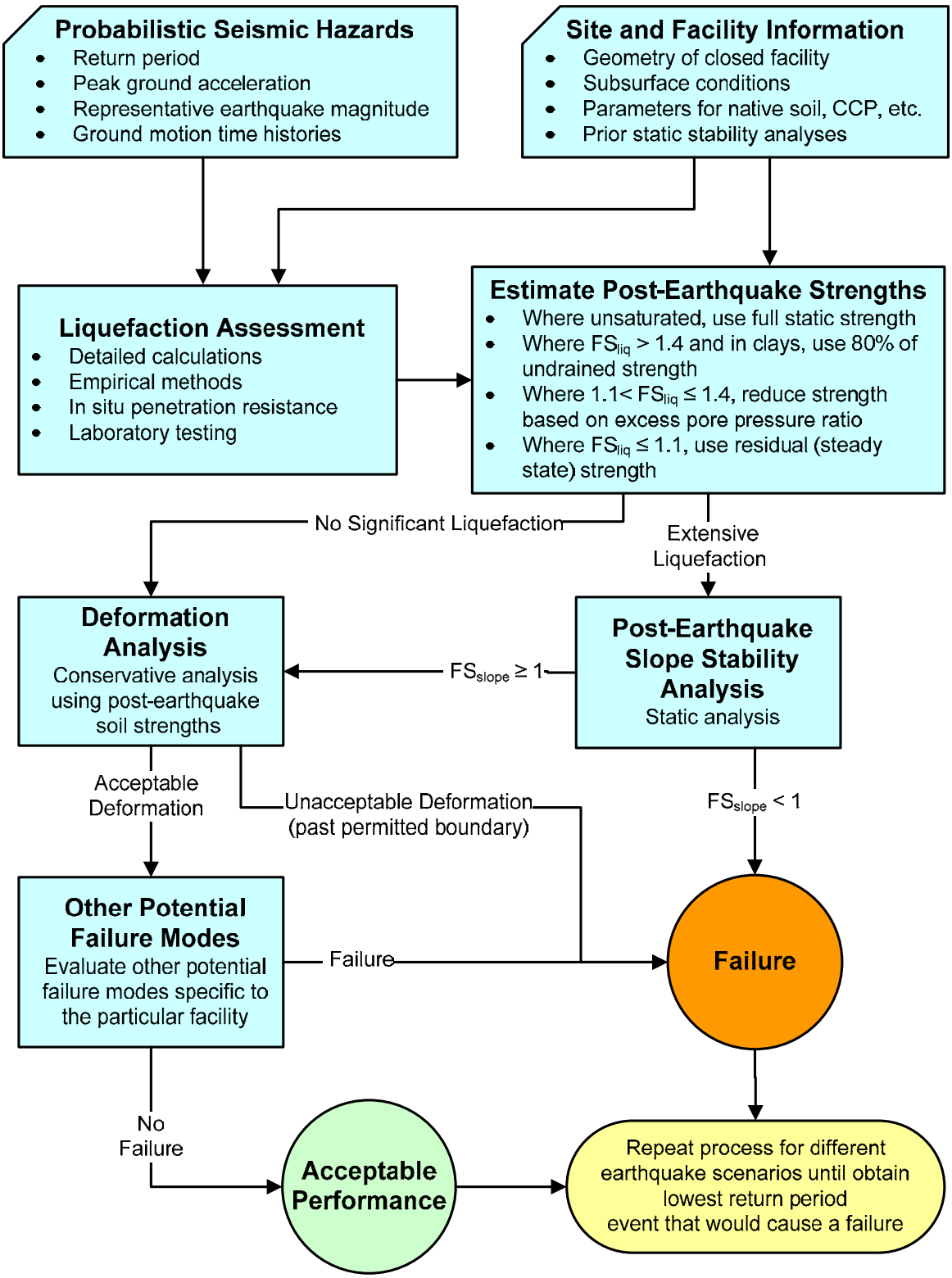


Figure 4. Simplified Flowchart for Assessing Facility Performance During a Probabilistic Seismic Event in Phase B

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.

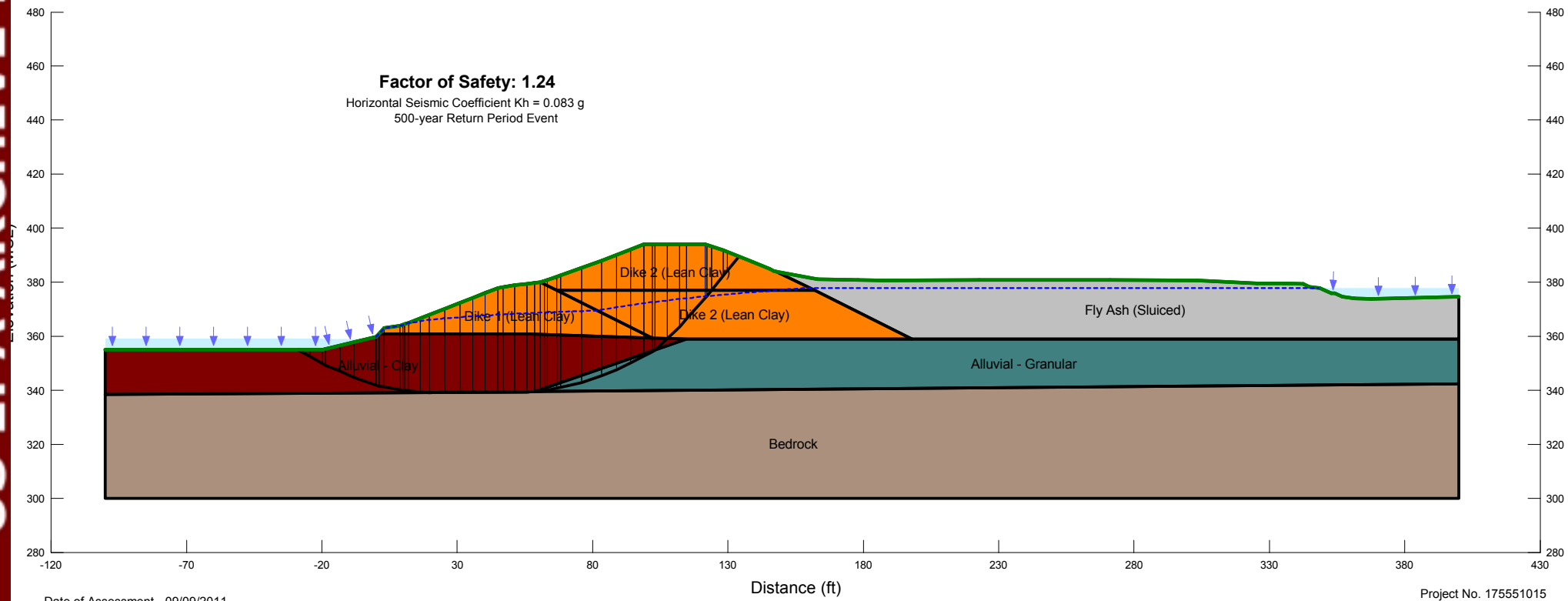
Pseudostatic Slope Stability Analysis
CCP Storage Facilities - Existing Conditions
Tennessee Valley Authority Fossil Plants

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

Stantec

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

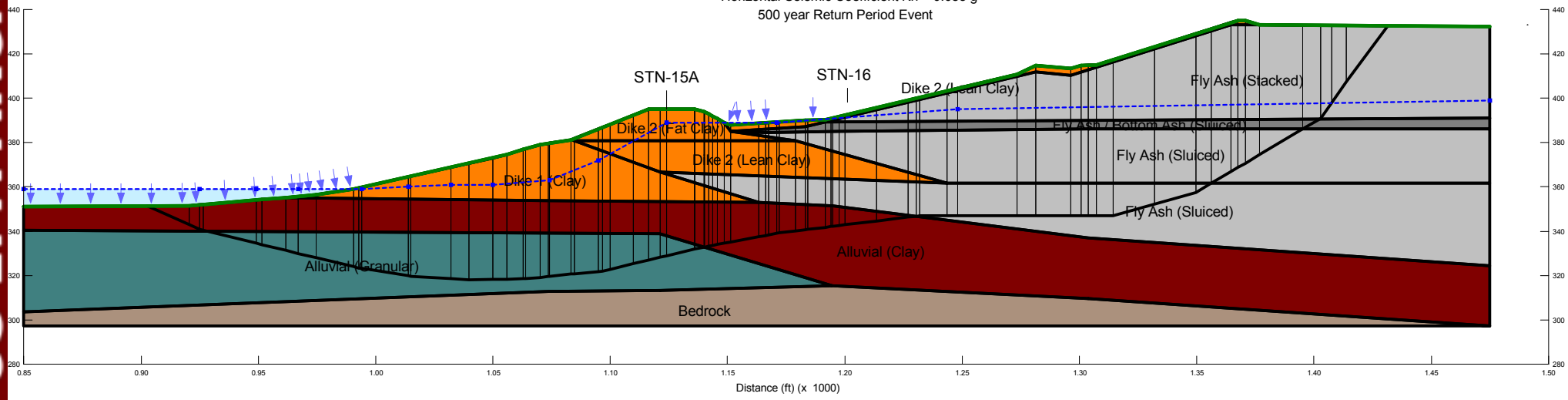
Seciton F - Dry Fly Ash Stack
Cumberland Fossil Plant
Cumberland City, Tennessee

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

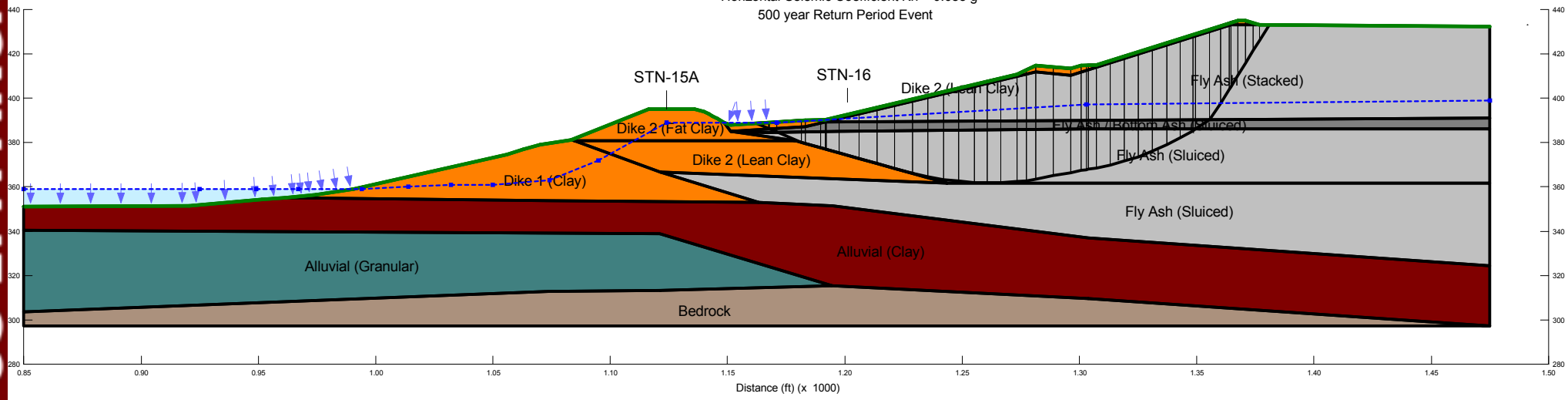
Seciton F - Dry Fly Ash Stack
Cumberland Fossil Plant
Cumberland City, Tennessee

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

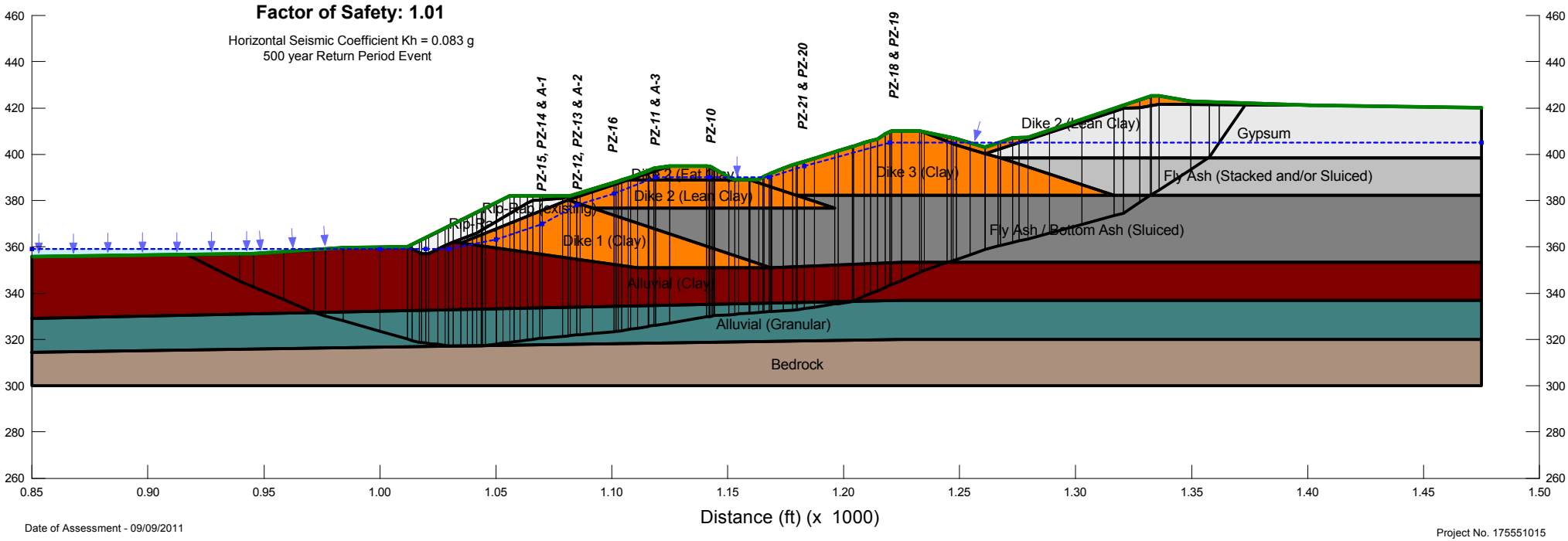
Project No. 175551015

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

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

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

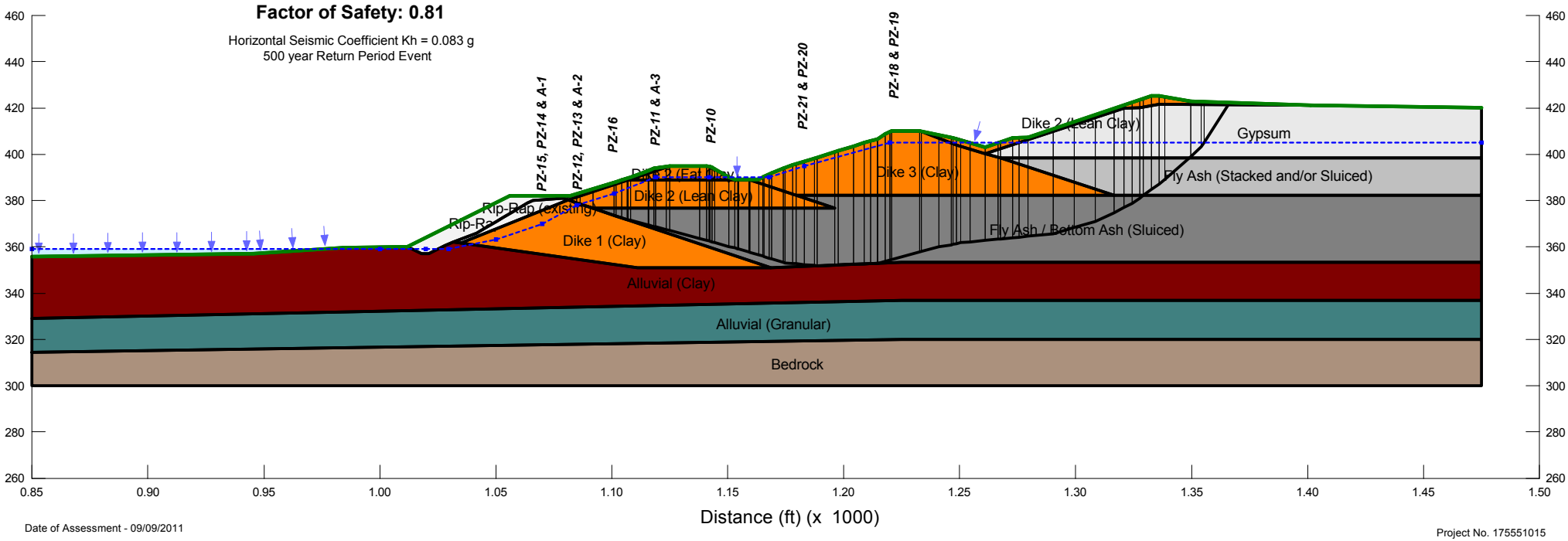
Project No. 175551015

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

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

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 °



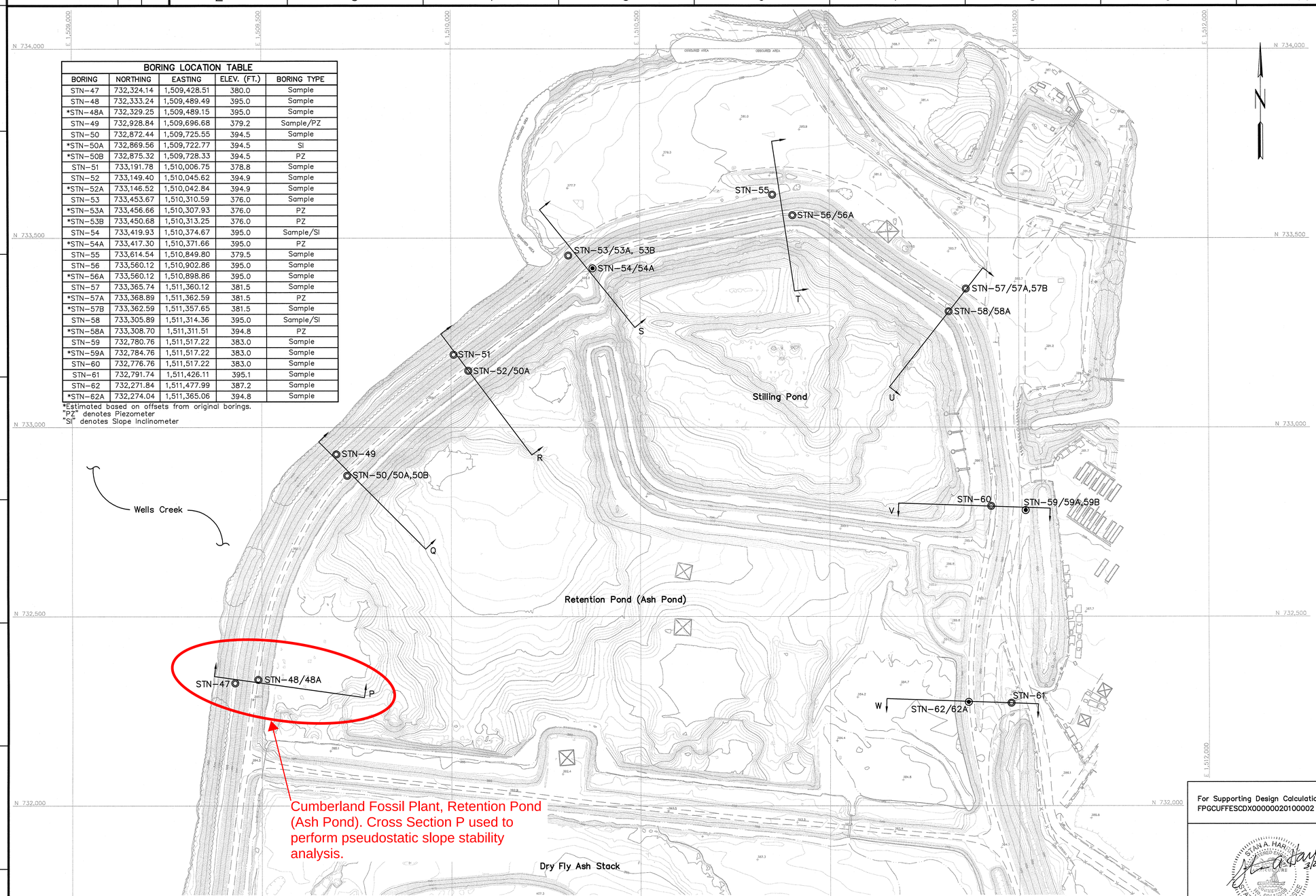
Date of Assessment - 09/09/2011

Project No. 175551015

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'

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.

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

RECORD DRAWING

PLOT DATE: 03/29/2010 USER: WITHERS, CHRISTINE
V:\1755\ACTIVE\175530501\GEOTECH\CAD\BORING\REVISE_RECORD\10W544-01-R0.DWG

For Supporting Design Calculations see FPGCUFFESCDX00000020100002

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

DISCIPLINE INTERFACE

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 Consulting Services Inc.
11687 Lebanon Rd.
Cincinnati, Ohio
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Fax 513.842.8250
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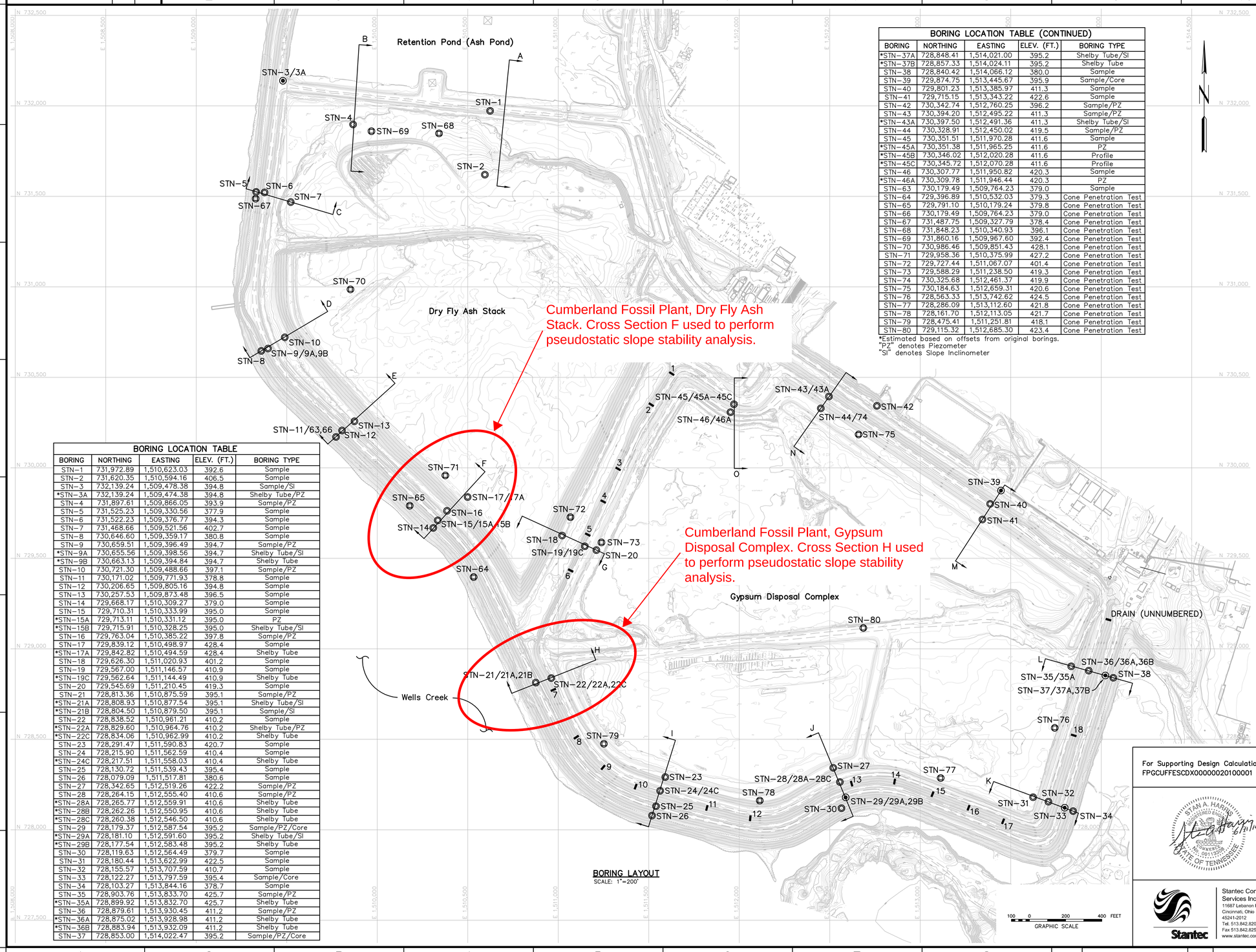
CUMBERLAND FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

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 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

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	1,512,519.28	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 LAYOUT
SCALE: 1"=200'

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

SCALE: 1"=200' EXCEPT AS NOTED

YARD
DRY FLY ASH STACK AND GYPSUM DISPOSAL COMPLEX

GEOTECHNICAL EXPLORATION
BORING LAYOUT

STAN A. HARRIS REGISTERED ENGINEER No. 0011335 STATE OF TENNESSEE 6/11/10

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

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

CUMBERLAND FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

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

PLOT FACTOR:XX
W_TVA

C.A.D. DRAWING DO NOT ALTER MANUALLY

STANTEC | 0
TASK COMPLETED BY: | REV NO.

PLOT DATE: 06/08/2010 USER: WITHERS, CHEYENNE
V:\1733\ACTIVE\173530000\GEOTECHNICAL\DRAWING\EVGL_RECORD\10W543-01-R0.DWG

APPENDIX A

Document 9

Seepage Action Plan, Stantec, June 25, 2010

Seepage Action Plan (SAP)

Cumberland Fossil Plant
Cumberland City, Tennessee

Stantec Consulting Services Inc.
One Team. Infinite Solutions
1409 North Forbes Road
Lexington, KY 40511-2050
Tel: (859) 422-3000 • Fax: (859) 422-3100
www.stantec.com

Prepared for:
Tennessee Valley Authority
Chattanooga, Tennessee

June 25, 2010

Seepage Action Plan (SAP)

Cumberland Fossil Plant
Cumberland City, Tennessee

Prepared for:
Tennessee Valley Authority
Chattanooga, Tennessee

June 25, 2010

Seepage Action Plan (SAP)

Cumberland Fossil Plant
Cumberland City, Tennessee

Table of Contents

Section **Page No.**

List of Tables

Table **Page No.**

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(Continued)

List of Figures

Figure **Page No.**

List of Appendixes

Seepage Action Plan (SAP)

Cumberland Fossil Plant
Cumberland City, Tennessee

1. Potential Seepage Areas

For readers not familiar with seepage through dams, refer to Appendix B, "Possible Seepage Problems and Recommendations" for more illustrative details. Seepage through an impoundment dam can typically be found on the lower third of the slope and extending beyond the toe approximately fifty feet. Figure 1 below displays the typical area on a cross section that should be reviewed during the seepage inspection for the Ash Pond, Dry Ash Stack and Gypsum Stack Complex. However, other seepage areas may exist, and the field inspector should be familiar with previous inspection reports and observations. Based on geotechnical analysis, plan views illustrating low factors of safety in terms of seepage have been prepared and are included in Appendix A. The areas identified, along with any other area previously identified during inspections, should be reviewed on a regular basis as identified in this document.

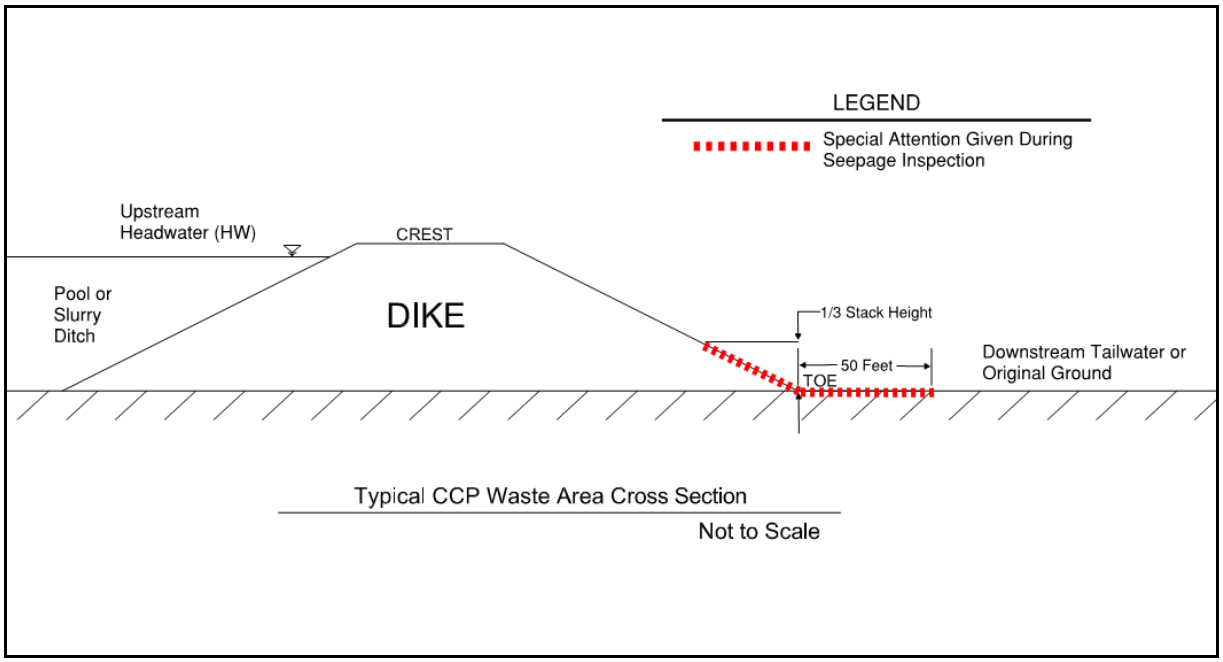

Figure 1. Seepage Inspection Location

2. Basic SAP Data

2.1. Purpose

The purpose of this SAP is to describe potential seepage action levels, and provide seepage short term management measures and actions in the event these action levels are observed.

2.2. Potential Impacted Area

Seepage related issues impact the integrity of earthen embankments. Seepage can lead to internal erosion of the embankment, known as piping, which has been the cause of many catastrophic failures in the past. Piping is a process where soil particles slowly carried out from inside the dam, eventually creating a tunnel or pipe. If the pipe forms all the way to the reservoir, the embankment will fail rapidly. Since the embankments at Cumberland Fossil Plant serve as an impoundment for ash and gypsum slurry, it is imperative to maintain the embankments and prevent any possible failure from occurring. If a failure were to occur, the ash and gypsum slurry mixture could potentially contaminate Cumberland Fossil Plant and the Cumberland River.

2.3. Primary Responsibility and Frequency of Dike Safety Inspections

1. TVA RHO&M Field Supervisor for Cumberland Fossil Plant (**Field Supervisor**)
2. TVA RHO&M West Region Construction Manager
3. TVA RHO&M Program Manager for Cumberland Fossil Plant

Documented inspections should occur at a minimum of once per month. Additionally, there are two criteria which warrant an inspection. A documented inspection should occur following a significant precipitation event (0.5 inches of rain, 4 inches of snow), as well as following a change in the operation of the stack, pond, or other CCP wet waste area (switching between east/west ditch, switching ponds, raising pool elevations, etc.). A documented inspection involves inspecting the potential seepage areas noted on the plan views in Appendix A, paying particular attention to areas of concern previously identified. The **Seepage Log** should be updated to include new descriptions and photographs of any new areas of concern or changes to previously identified areas. Random inspections can occur on a more frequent basis if deemed necessary by the **Field Supervisor**.

3. Seepage Action Level Determination

For the purpose of this plan, three seepage action levels have been identified. The levels are based on potential risk associated with progressive erosion due to seepage and resulting breach of the embankment or impoundment.

Action Level 1 – Non-Flowing

- Wet areas
- Ponded Water

Action Level 2 – Flowing Seepage – No Erosion

- Non turbid (clear water) flow

Action Level 3 – Flowing Seepage – Active Erosion

- Turbid Flow
- Deposition of Sediment from Dike or Dam
- Boils (Ground Surface/ Underwater)
- Upstream Collapse or Sinkhole

3.1. Action Level 1 – Non Flowing

Seepage occurs in all earthen dams and dikes. The key is to properly collect and control seepage in a manner that does not cause damage to the embankment. Seepage that is not flowing but is evident by damp areas or ponded water does not generally represent an imminent threat to the embankment in terms of erosion (see Figure 2). However, if left unattended this seepage can lead to slope instabilities. Therefore, this should be noted so that it can be observed for changing conditions both at the downstream observation point and immediately upstream along the interior slopes.

Figure 2. Example of Action Level 1 – Non-Flowing – Wet Area

3.2. Action Level 2 – Flowing Seepage – No Erosion

Action Level 2 involves observations of flowing seepage, but evidence of erosion is not noted. Evidence of erosion can be in the form of turbid (muddy water) flow, sediment deposition, obvious hole or soil "pipe". Evidence of erosion can be subtle and as a result, any flowing seepage should be carefully reviewed and monitored at least monthly. A picture of flowing seepage water showing no evidence of erosion is depicted in Figure 3. Note that a seep does not need to be continuously turbid for a piping situation to be forming.

Figure 3. Example of Action Level 2 – Clear Flowing – Seepage Boil

3.3. Action Level 3 – Flowing Seepage – Active Erosion

Left unmitigated seepage demonstrating active erosion can lead to progressive failure of the embankment and catastrophic loss of the impoundment. Evidence of erosion can be in the form of turbid flow, sediment deposition, boil, obvious hole or soil "pipe". Evidence of erosion can be subtle and as a result, any flowing seepage should be carefully reviewed and monitored frequently. Careful attention should be given to seepage below water such as a stilling pond, creek or river (see Figure 6). This type of seepage is difficult to observe and determine if soil erosion is occurring. In moving water, evidence of seepage boils conveying embankment soil/ash materials will likely be (partially) washed away. Examples of active erosion are shown in Figures 4 thru 5.

Figure 4. Example of Action Level 3 – Turbid Flowing – Seepage Boil

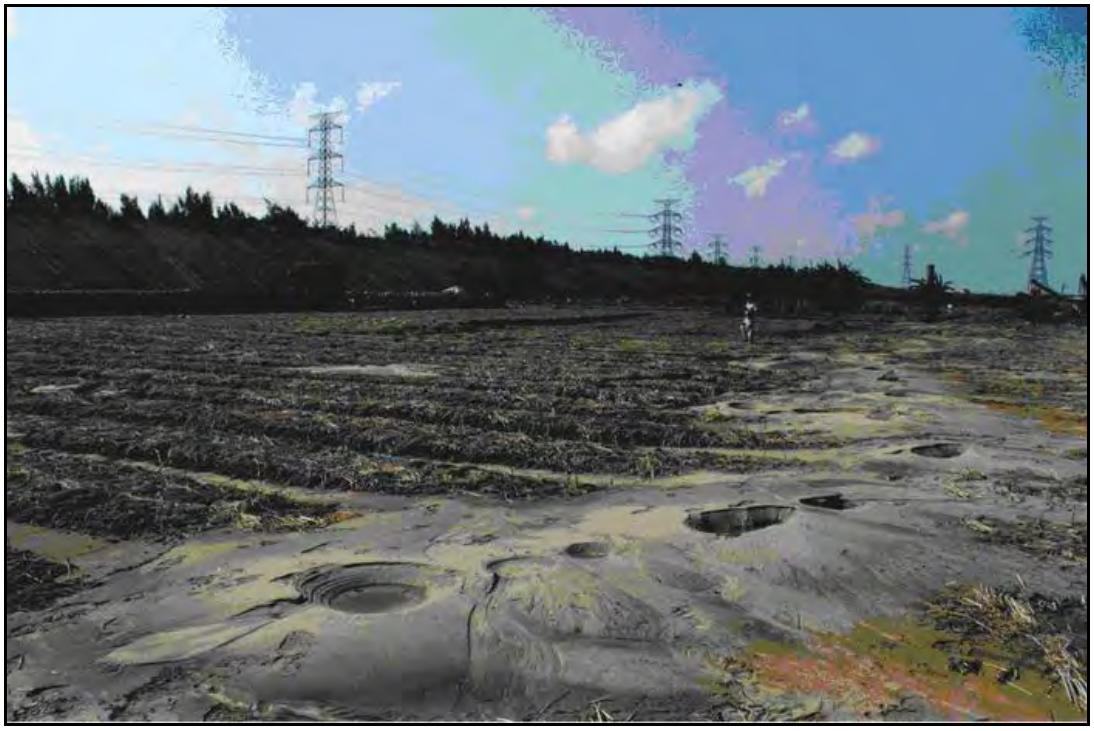

Figure 5. Example of Action Level 3 – Deposition of Sediment from Dike

Figure 6. Example of Action Level 3 – Underwater Turbid Flowing – Seepage Boil

4. Intermediate Corrective Measures

For each action level a typical corrective measure is listed below.

4.1. Action Level 1 – Non Flowing

- **Field Supervisor** should document the seepage area into the **Seepage Log** (see below).
- All observers should pay particular attention to conduits through the embankments.
- **Field Supervisor** should record the date, time, size of area, location, and photographs in the **Seepage Log**.

The **Seepage Log** should be kept at the Shift Operation Supervisor's (SOS) office such that inspectors (TVA, geotechnical consultant, or others) can document event triggers (date, time, location, pool level, etc.) and the site conditions observed for each seepage event. The **Seepage Log** shall function as a "living document" and be part of an ongoing monitoring program (to be controlled by TVA). As the monitoring program progresses, the **Seepage Log** will allow inspectors to summarize the historical conditions observed and provide a baseline of events to compare with future readings.

4.2. Action Level 2 – Flowing Seepage – No Erosion

- **Field Supervisor** should carefully inspect the area for outflow quantity, any transported material, and take photographs.

- If the seepage involves a conduit penetration associated with a spillway pipeline, storm culvert, or underdrain pipeline, the observer(s) should carefully inspect the area by probing and /or carefully shoveling to see if the cause can be determined, determine if embankment materials are being transported, evident by turbid or cloudy water, and determine quantity of flow.
- Contact team members in accordance with Figure 8.
- Send photographs to the RHO&M Regional Construction Manager and CCP Program Manager for distribution.
- Geotechnical consultant, with concurrence of the TVA Program Manager and CCP Engineering Manager, should determine a plan of action within four hours of notification
- **Field Supervisor** should record the date, time, size of area, location, and photographs in the **Seepage Log**.

4.3. Action Level 3 – Flowing Seepage – Active Erosion

- **Field Supervisor** should carefully inspect the area for outflow quantity and transported material.
- **Field Supervisor** should determine if piping has occurred and extent by observing locations of seepage exits, take photographs, and contact team members in accordance with Figure 9.
- Geotechnical consultant, TVA Program Manager, and CCP Engineering Manager should determine a plan of action within four hours of notification such as lowering the pool, constructing a reverse graded filter, or sand bagging
- A typical reverse graded filter will consist of the following:
  - One foot of Concrete Sand (TDOT Concrete Sand)
  - One foot of TDOT No. 57 Stone
  - One foot of TDOT No. 1 Stone
  - Two feet of TDOT Machine Rip-Rap Class A-1
  - Silt Fence as required by guidance provided in the Best Management Practices for Erosion Prevention and Sediment Control
- An example of sandbagging is provided in Figure 7.
- **Field Supervisor** should record the date, time, size of area, location, and photographs in the **Seepage Log**.

Figure 7. Sand Bag Treatment (Temporary)

5. Materials On-Site

In case an emergency situation is observed during the inspection of the potential seepage areas, it is necessary to have materials readily available on-site to correct the situation. Table 1 below lists the materials to be stockpiled on-site and the quantity of each material.

Table 1. Stockpile Material Quantities

Material	Tons	Cubic Yards
Concrete Sand	90	60
TDOT No. 57 Stone	90	60
TDOT No. 1 Stone	90	60
TDOT Machine Rip-Rap Class A-1	180	120
Sandbags (filled)	300 (total)	NA
30” Diameter HDPE Pipe	100 feet	NA

The amount of materials to be stockpiled is based on a production rate of 60 cubic yards per hour for a 2.5 CY long reach excavator assuming a material unit weight of 110 PCF.

The materials should be stockpiled in the corner of the Dry Ash Stack to the northwest of the West Gypsum Pond. The following earthwork equipment and qualified operator(s) should be located to place the material in case of an emergency:

- Long Reach Excavator
- Dump Truck
- Compactor, Bulldozer, Bobcat, any other nearby equipment which aids in the emergency

6. The SAP Process

6.1. Step 1 – Dike Observation or Event Detection

This step describes the detection of an unusual observation or emergency event and provides information to assist the Cumberland RHO&M **Field Supervisor** or appropriate personnel in determining the appropriate emergency level for the observation or event. These observations could be made by inspectors during routine inspections of the embankments, or by everyday personnel.

6.2. Step 2 – Emergency Level Determination

Following an unusual observation or emergency event detection, the **Field Supervisor** is responsible for classifying the event into one of the following three emergency levels:

6.2.1. Action Level 1 – Non Flowing

Observation is routine to other observations and a similar established plan of action for minor repair or continued observation will be required. If a Level 1 Emergency is identified, the following steps should be taken:

- Update maps and **Seepage Log**
- Inform CUF personnel if repairs are needed
- Determine if other work activities need to be made aware of observation.

6.2.2. Action Level 2 – Flowing – No Erosion

A change in condition or a condition that has not been previously identified and discussed with the geotechnical engineers. If a Level 2 Emergency is identified, the following steps should be taken:

- Inform individuals in accordance with the flowchart in Figure 8.
- Update map and **Seepage Log**
- Inform CUF personnel if repairs are needed
- Determine if other work activities need to be made aware of new conditions.

6.2.3. Action Level 3 – Flowing – Active Erosion

A change in condition that is drastic and could rapidly lead to failure of the embankment if not corrected. If a Level 3 Emergency is identified, the following steps should be taken:

- Inform plant SOS, who will initiate TVA plant-specific Emergency Action Plan (see Figure 9).
- Inform geotechnical consultant
- Develop safe plan of action for repair with geotechnical consultants
- Initiate repairs once plan has been approved by site safety and geotechnical consultant
- Update map and **Seepage Log**.

6.3. Step 3 – Notification and Communication

6.3.1. Notification

Following the determination of a possible seepage situation, it is necessary to notify the appropriate personnel discussed below for the required action to occur.

6.3.2. Communication

In case of an Action Level 2 emergency, the flowchart presented in Figure 8 should be followed to ensure the proper personnel are contacted. In an Action Level 3 emergency, the flowchart presented in Figure 9 should be followed.

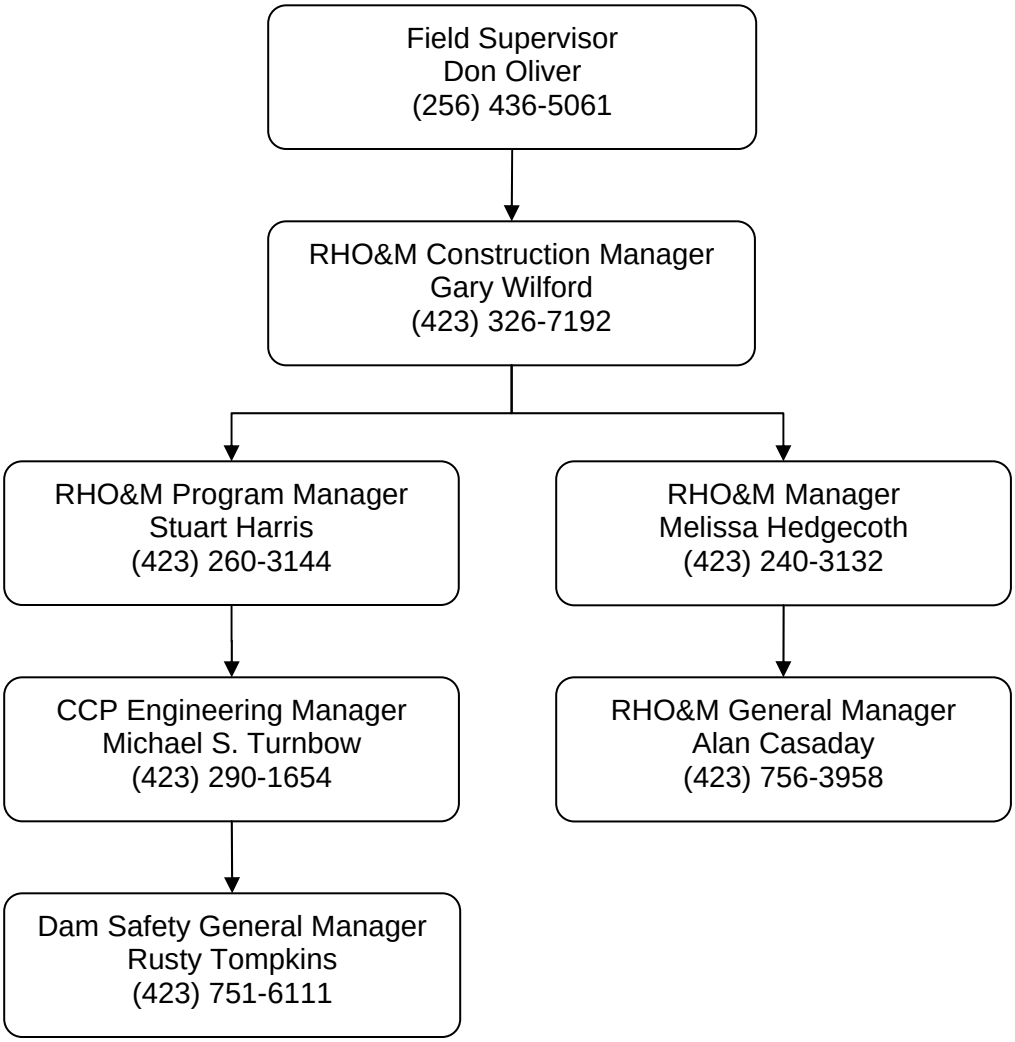

Figure 8. Level 2 Emergency Contact Flowchart

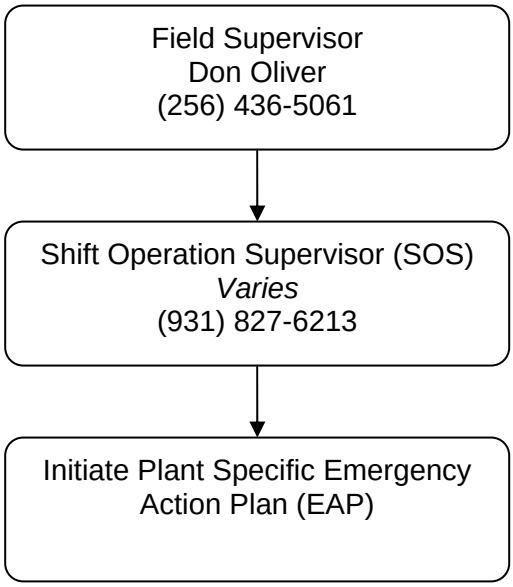

Figure 9. Level 3 Emergency Contact Flowchart

Appendix A

Ash Pond, Dry Ash Stack and Gypsum Stack Complex Site Plan

Notes:
1. The areas identified on this drawing should not be considered as the only areas where seepage might occur. Therefore, a complete inspection of the dike should be performed annually as outlined in Stantec's Dam Safety Training Presentation dated August, 2009.
2. Grid spacing is 500 feet.

LEGEND

N

Areas of Concern

Stockpile Location

Potential Seepage Zone

Feet
0 250 500

STANTEC CONSULTING SERVICES INC.
1409 N. Forbes Rd.
Lexington, Kentucky
40511-2050
859-422-3000

Stantec

SEEPAGE ACTION PLAN

Tennesee Valley Authority
Cumberland Fossil Plant
Cumberland, Houston County, Tennessee

PROJECT NO.	175560021
DATE	MAY 2010
DRAWN BY	AMG
CHECKED BY	
CHECKED BY	
SCALE	1" = 500'
REVISED	
1.	
2.	
3.	
4.	
5.	
6.	
7.	
8.	

SHEET

1 OF 1

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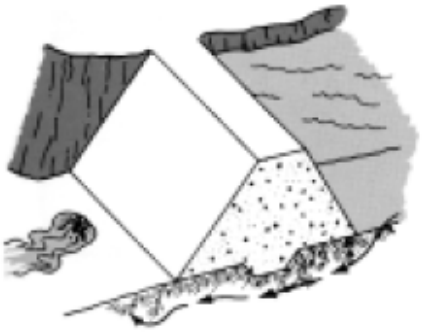
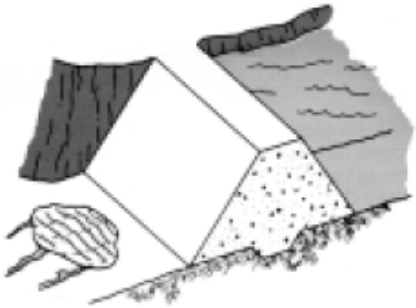
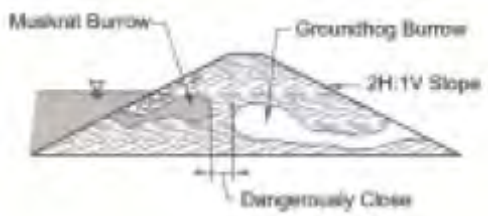
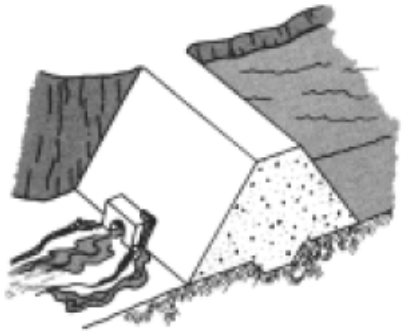
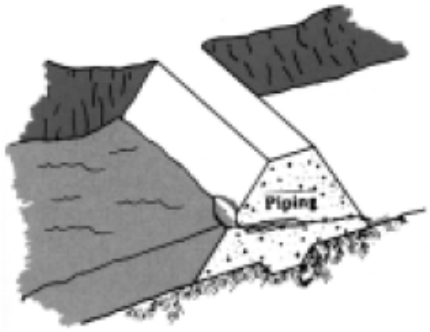
Appendix B

Possible Seepage Problems and Recommendations

Appendix B – Possible Problems and Recommendations

Seepage Problem	Recommendations
Seepage Water Exiting at Abutment Contact 	Study leakage area to determine quantity of flow and extent of saturation. Stake out the saturated area and monitor for growth or shrinkage. Inspect frequently for slides. Water level in the impoundment may be lowered to increase embankment safety. A QUALIFIED ENGINEER should inspect the conditions and recommend further actions to be taken.
Seepage Water Exiting as a Boil in the Foundation 	Examine boil for transportation of foundation materials, evidenced by discoloration. If soil particles are moving downstream, create a sand bag or earth dike around the boil. This is a temporary control measure. The pressure created by the water level within the dike may control flow velocities and prevent further erosion. If erosion continues, lower the reservoir level. A QUALIFIED ENGINEER should inspect the condition and recommend further actions to be taken.
Spongy Condition at Toe of Dam 	Carefully inspect the area for outflow quantity and any transported material. A QUALIFIED ENGINEER should inspect the condition and recommend further actions to be taken.

Appendix B – Possible Problems and Recommendations

Seepage Problem	Recommendations
Rodent Activity 	Control rodents to prevent more damage. Determine exact location of digging and extent of tunneling. Remove rodents and backfill existing holes.
Seepage Water Exiting from a Point Adjacent to the Outlet 	Investigate the area by probing and/or carefully shoveling to see if the cause can be determined. Determine if leakage water is carrying soil particles evidenced by discoloration. Determine quantity of flow. If flow increases, or is carrying embankment materials, reservoir level should be lowered until leakage stops. A QUALIFIED ENGINEER should inspect the condition and recommend further actions to be taken.
Sinkhole 	Inspect other parts of the dam for seepage or more sinkholes. Identify exact cause of sinkholes. Check seepage and leakage outflows for dirty water. A QUALIFIED ENGINEER should inspect the conditions and recommend further actions to be taken.

Appendix B – Possible Problems and Recommendations

Seepage Problem	Recommendations
Trees and Brush 	Remove all trees and shrubs on and within 25 feet of the embankment. Properly backfill void with compacted material. A QUALIFIED ENGINEER may be required.

Source: Connecticut Department of Environmental Protection, Guidelines for Inspection and Maintenance of Dams, September 2001.

Appendix C

Seepage Log

CUF Seepage Log

Cumberland Fossil Plant
Cumberland City, Tennessee
Updated June 14, 2010 Rev. 1

Area of Concern	Northing	Easting	Date Initially Observed	Time	Approximate Size (Linear Feet)	SAP Level	Description	Mitigation Status/ Future Plans
1	728264.15	1512555.40	8/1/2009	N/A	100' W x 20' H	2	Seep identified in August 2009. Area is wet and soft. There is one small area of flow, with the flow being less than 1 gpm. No movement of soil particles has been observed.	A seepage collection blanket has been designed by Stantec and will be incorporated into the perimeter ditch/slope buttress project.
2	728813.36	1510875.59	2005	N/A	200' W x 30'H	1	Slope failure along perimeter dike at the southwest corner of the complex occurred in 2005. It was reported that seepage was observed. It was addressed by the placement of rip-rap over the area.	Temporary slope repairs consisted of placing riprap on slope. A construction work plan has been prepared to install a subsurface drain and perform permanent slope repairs. Work is scheduled to be completed in the summer of 2010.
3	Survey Requested		5/26/2010	N/A	3' x 3'	1	Seep identified on the southeast toe of dike between subdrain-14 and subdrain-15	Seep should be observed for changes.

Note: Initial Seepage Log was developed based on Stantec's understanding of known issues from Phase 1 and Phase 2 assessments and the 2010 Annual Inspection. No field visit was conducted to verify current seepage areas of concern.

Area of Concern 1

8/31/09
Seepage along the southeast toe of dike.

Area of Concern 1

3/9/10
Seepage along the southeast toe of dike.

Area of Concern 2

6/9/09
Slope failure located along the perimeter of the southwest corner of the complex.

Area of Concern 2

1/19/10
Slope failure located along the perimeter of the southwest corner of the complex. Rip-rap placed over failure.

Area of Concern 3

5/26/2010
Area of erosion and seep located between subdrain-14 and subdrain-15 on the exterior slope of the perimeter ash dike on the south side of the complex.

Appendix D

COF CCP Emergency Action Plan

APPENDIX A

Document 10

2011 Annual Inspection of CCP Facilities Ponds, July 19, 2011

2011 Annual Inspection of CCP Facilities and Ponds

Cumberland Fossil Plant
Cumberland City,
Stewart County, Tennessee

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One Team. Infinite Solutions
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Cincinnati, OH 45241
Tel: (513) 842-8200 • Fax: (513) 842-8250
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Prepared for:
Tennessee Valley Authority
Chattanooga, Tennessee

July 19, 2011

Stantec Consulting Services Inc.
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Tel: (513) 842-8200
Fax: (513) 842-8250

July 19, 2011 rpt_001_175531011

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

Re: 2011 Annual Inspection of CCP Facilities and Ponds
Cumberland Fossil Plant
Cumberland City, Stewart County, Tennessee

Dear Mr. Turnbow:

Stantec Consulting Services Inc. (Stantec) has completed the 2011 annual inspections for the CCP Facilities and Ponds at the Cumberland Fossil Plant (CUF). Facilities reviewed included:

- Coal Yard Drainage Basin
- Chemical Treatment Pond
- Active Ash Pond
- Dry Fly Ash Stack
- Gypsum Disposal Complex

The field work was executed on June 20, 2011. The results of the work along with facility-specific recommendations for maintenance or other activities are included on the enclosed documents. The preparation of work plans was recommended when the deficiencies identified were considered to require some engineering evaluation, or when multiple deficiencies were observed across a wide area and did not lend themselves to recommendations for repair on a case-by-case basis. In addition, the following general plant-wide recommendations and comments are offered:

- It is recommended that vegetation maintenance continues. If lack of vegetation is observed during these operations, re-seeding should be performed as soon as possible. If vegetation establishment difficulties continue in any areas, then TVA

should consider refining existing procedures or developing site-specific specifications which address topsoil, fertilizing, seed mixtures, etc.

- It is recommended that TVA catalog, assign a responsible party and due date, and track the completion of the facility-specific recommendations provided herein.

- Note that this scope did not include a review of the current Operations and Maintenance Manual (O&M) for CUF.

- It is recommended that TVA personnel continue dike inspections/monitoring to look for changes or conditions that might affect dike integrity. The frequency and procedures for inspections should be consistent with TVA's inspection program. Particular emphasis should be placed on reviewing and monitoring the seepage areas for changed or worsened conditions, and identifying and repairing other maintenance items such as animal burrows, erosion, and lack of vegetation.

Stantec appreciates the opportunity to provide continued engineering services for the fossil plants. If you have any questions, or if we may be of further assistance, feel free to contact our office.

Sincerely,

STANTEC CONSULTING SERVICES INC.

Stan A. Harris, PE
Principal

Daniel B. Rogers, PE
Senior Project Engineer (Geotechnical)

/lfb

Enclosures

Executive Summary

Stantec Consulting Services Inc. (Stantec) in conjunction with TVA Surveying has completed an Annual Inspection of the five facilities at Cumberland Fossil Plant (CUF). These facilities include: Active Ash Pond, Chemical Treatment Pond, Coal Yard Drainage Basin (CYDB), Dry Fly Ash Stack and Gypsum Disposal Complex. This inspection was performed to evaluate the current conditions of the disposal facilities and to provide recommendations for improvement.

During the inspection and reporting process, each facility was inspected by a team of three individuals who walked the perimeter of each facility and recorded the locations of seeps, instabilities, erosional features and other inconsistencies that may affect the stability of the containment system. Once the inspection was complete, the notes, photographs and location coordinates of each item were compiled for each facility and are presented in the following report.

The forms that were completed for each facility include a list of deficiencies that were discovered and recommendations for mitigation. For reference, these deficiencies and recommendations were compiled and are presented below.

Ash Pond

- It is recommended that the rutting adjacent to the crest of the dike be repaired in accordance with the General Guidelines. Additionally, the areas of erosion observed along the exterior divider dike should be monitored and repaired in accordance with General Guidelines. (Priority 4)
- The wave action erosion along the north and west interior dike faces should be addressed by rip-rap armoring as addressed in the general guidelines (Enclosure J). However, this work should be scheduled after the completion of Work Plan 7.
- The drainage ditch on the east side of the Active Ash Pond should be cleaned of phragmites and any accumulated sedimentation. The slope of the ditch in this area should be evaluated to assess if better flow is possible to alleviate the standing water observed. (Priority 4)
- Observation of the toe of the exterior slope on the north side of the Stilling Pond should continue in order to assess if the moist slope is a result of seepage or surface runoff as a result of recent rain events. (TVA Quarterly Inspection Item 2) This should be completed in accordance with the Seepage Action Plan.
- Observation of the moist areas on the midslope and toe of the west exterior slope of the Active Ash Pond should continue to determine if these are seeps or if they are moist from recent precipitation. Any changes in the flow or color of these areas should be reported to Fossil Engineering Design Services (EDS). This should be completed in accordance with the Seepage Action Plan.
- Areas where tire rutting is observed along both the interior and exterior slopes should be repaired and reseeded as described in Enclosure H. Special care should be taken, if possible, to not perform maintenance on the slopes of the ponds during conditions that could result in rutting. (Priority 5).

The trees observed on the interior divider dike should be removed in accordance with the guidelines given in Enclosure F (Priority 4).

Chemical Treatment Pond

- The area around the concrete gutter at the outlet of the recirculation pipe on the east slope of the pond should be monitored for erosion (see Photo 1). This area should be examined during recirculation in order to determine if the gutter is capable of containing the flow. If the gutter is overtopped or if splashing occurs causing the water not to be contained in the gutter, the gutter should be widened. (Priority 5)

Coal Yard Drainage Basin

- The pond should also be sounded as needed to determine when dredging will be required. (Priority 5)

- It is recommended that the small trees that are beginning to grow along the south slope be removed in accordance with the guidelines shown in Enclosure F. (Priority 4)

- It is recommended that the depression observed in the south slope be monitored in accordance with the currently established stability monitoring program (See Photo 3). Any evidence of erosion or slope movement within this area should be reported to Stantec.

- The observed slope instability in the face of the south slope should be regarded and vegetated (see Photo 4). (Priority 3)

- The animal burrows observed on the interior slopes on the east and northeast sides of the CYDB should be repaired in accordance with the guidelines given in enclosure G. (Priority 3)

- It is recommended that a work plan be prepared to address the rutting/erosion spots observed on the west and north slopes, as well as the erosion detected on the platform for the pump station. It is expected that riprap will be needed to stabilize each slope. This work should be conducted in accordance with the general guidelines included in Enclosures H and I. (Priority 4)

- The pond was dredged in 2007 to a cell built in the Dry Fly Ash Stack area. Since that time, sedimentation has continued along the northwest portion of the pond resulting in the effectiveness of the floating boom in this area being compromised. Sedimentation accumulation was also observed in the southeast portion of the pond. Consider removing the sediment built up along the base of the pond in both locations. (Priority 5)

- The pond should also be sounded as needed to determine when dredging will be required. (Priority 5)

- The small trees that are beginning to grow along the south slope be removed in accordance with the general guidelines. (Priority 4)

- The depression observed in the south slope should be monitored in accordance with TVA's Inspection Program. Any evidence of erosion or slope movement within this area should be reported to Stantec.

- The observed slope instability in the face of the south slope should be re-graded and vegetated (see Photo 4). (Priority 3)

- The animal burrows observed on the interior slopes on the east and northeast sides of the CYDB should be repaired in accordance with the general guidelines (Priority 3)

- Erosion spots were observed on the west and north slopes, as well as on the platform for the pump station. It is expected that rip-rap will be needed to stabilize each slope. This work should be conducted in accordance with the general guidelines (Priority 4)

Dry Fly Ash Stack

- The perimeter drainage ditch should be cleaned of large vegetation and sedimentation. (Priority 4) When localized areas of standing water are encountered, it is recommended that these areas be cleared so as to not allow standing water to saturate the ditch and/or the embankment. (Priority 4) Trans Ash is currently operating in this area and should be notified when these situations occur.
- The areas of seepage noted within this report and on the Seep Log should be monitored in accordance with the Seepage Action Plan. Any changes in the flow, size or color of these seepage areas should be reported to EDS.
- The erosion observed on the outslope of the Bottom Ash Pond should be monitored and repaired in accordance with the general guidelines which are included in Enclosure I. (Priority 4)
- The area of sparse vegetation observed on the south west exterior slope of the stack should be monitored as part of TVA's Inspection Program and repaired as conditions warrant. This area should be reworked, graded, and a new soil cover be installed suitable to support vegetation. The area should then be seeded as described in the general guidelines.

Gypsum Disposal Complex

- As the interior cells drain to the southwest corner, flow drops several feet from the discharge pipes down to the rip-rap channel below. Although the rip-rap appears to be controlling erosion at the toe of the slope, continued erosion could undermine the toe. It is recommended that this area be monitored for erosion and stabilization issues during the course of TVA's inspection program and appropriate action taken if problems are noted.
- The perimeter drainage ditch should be cleaned of phragmites and sedimentation. (Priority 4) When localized areas of standing water are encountered, it is

recommended that these areas be cleared so as to not allow the standing water to saturate the ditch and/or the embankment. (Priority 4) Trans Ash is currently operating in this area and should be notified when these situations occur.

- The seep locations observed on the exterior slopes of the perimeter dike on the southwest side should continue to be monitored as described in the Seepage Action Plan. Any changes to the flow or color of these seepage areas should be reported to Stantec.
- The areas of erosion at the outlet of the subdrains along the exterior slopes of the ash dike above the perimeter ditch should be repaired in accordance with the guidelines included in Enclosure I. This work should be coordinated with the execution of Work Plans 11 and 8.
- The seeps (TVA-CUF Seep Log) along the perimeter clay dike at the southwest corner of the complex and the perimeter ash dike on the south perimeter clay dike should be monitored for signs of movement or changes in seepage that may be indicative of slope failure. Any changes should be reported to Stantec.
- The area of erosion of the access road to the north side of the complex should be monitored and repaired in accordance with the guidelines included in Enclosure I. (Priority 5)
- The observed rutting should be repaired in accordance with the general guidelines presented in Enclosure H. Additionally, alternate mowing procedures should be used for the complex to prevent surface rutting of the vegetation cover.
- It is recommended that a work plan be prepared that would address the areas of barren cover soil (see Photo 11) along the exterior slopes of the complex. This work should be conducted in accordance with the guidelines presented in Enclosure F (Priority 4)
- The inlet and outlets of all drainage pipes should be monitored as part of TVA's Inspection Program and cleared as conditions warrant.

Enclosure A

Coal Yard Drainage Basin

TVA 2011 Annual Inspection Program
Cumberland Fossil Plant (CUF)
Coal Yard Drainage Basin (CYDB)

1. General Facility Information

Facility Status:	Active	NID Identification:	Not Available
Surface Area (inside dikes):	5.4 acres	TVA Hazard Classification:	Not Ranked
Maximum Height (toe to top of dike):	20 feet	Dike Length:	2,850 feet (0.54 miles)
Plant Discharge to Facility:	2.0 MGD (Average)	Current Pool Elevation:	377.0 feet (Average)

2. Site Visit Information

Stantec Inspection Team:	Daniel B. Rogers and James R. Swindler Jr.
TVA Staff Present:	M. Jacob Horton
Field Inspection Date:	6/20/2011
Weather/Site Conditions:	Sunny and 85 degrees. The slopes were generally moist due to recent precipitation (previous day)

3. History/Current and Future Operations

History:	The Coal Yard Drainage Basin (CYDB) is located south of the powerhouse, northeast of the Gypsum Disposal Complex. The CYDB collects the storm drainage from the coal yard and many other areas of the plant.
Current Operations:	The storm drainage is temporarly stored in the CYDB. When the CYDB reaches a pre-determined water elevation, or in anticipation of a precipitation event, pumps are activated to lower the pond level. These pumps discharge into the main perimeter ditch system and the flow eventually makes it's way to the Active Ash Pond.
Future Planned Operational Changes:	None

4. Stantec Field Observations

See attached Photos and Site Plan Drawing.

4.1. Interior Slopes

Vegetation:	Heavily vegetated with phragmites at the base of all slopes. There are isolated patches on the south slope that are barren of vegetation cover.
Trees:	Small trees were observed at the base of the south slope of the basin. (See Photo 1)
Wave Wash Protection:	Heavily vegetated with phragmites at the base of all slopes. Rip-rap protection was limited to the pad supporting the pump station.
Erosion:	One area of erosion was observed during the inspection. This area was on the west slope of the basin near an apparent drainage feature (see Photo 2).
Instabilities:	Some of the slopes were not visible due to the heavy growth of vegetation around the pond. On the portions of the slopes that were visible, there were two noted stability issues. • There is a depression in the dike face that extends from the crest to below the water surface. (See Photo 3) • There is a minor slip in the south slope just to the west of the access to the pumps. (See Photo 4) This appears to be surfacial and not deep seated. • There is general rutting of the south slope from the crest to mid-slope. Vegetation is also sparse. This appears to be caused by mowing equipment. (See Photo 5)
Animal Burrows:	There were a total of 2 burrows noted during the inspection. They were located on the east slope and the northeast corner. They were probed to depths of 1-3 feet. (See Photo 6)
Freeboard:	Minimum estimated to be 4 to 6 feet.
Encroachments:	Pump station at the southeast corner of the pond (see Photo 1).
Slope:	Estimated to be on the order of 2H:1V with slightly steeper slopes observed along the south side of the pond where an ash perimeter dike acts as the boundary.

4.2. Crest

Crest Cover and Slope:	Gravel road on the south side, vegetation cover on all other sides. No slope.
Erosion:	None observed.
Alignment:	The alignment was observed to be generally straight.
Settlement/Cracking:	None observed.
Bare Spots/Rutting:	None observed.
Width:	Gravel road on south side, approximately 12 feet.

4.3. Exterior Slopes

Vegetation:	N/A
Trees:	N/A
Erosion:	N/A
Instabilities:	N/A
Uniform Appearance:	N/A
Seepage:	N/A
Benches:	N/A
Foundation Drains, and Seepage Collection Systems:	N/A
Instrumentation:	N/A
Animal Burrows:	N/A
Slope:	N/A
Height:	N/A

4.4. Spillway Weirs/Riser Inlets

Number:	N/A
Size, Type and Material:	N/A

Height of Riser Inlets:	N/A
Access:	N/A
Joints:	N/A
Mis-Alignment:	N/A
Closed/Abandoned Conduits:	N/A

4.5. Outlet Pipes

Number:	There are three plant pumps on the pump deck, located on the southeast corner of the pond, approximately 6-inch diameter outlets.
Size, Type and Material:	Steel, 6-inch diamater pipes that tie into a 24-inch diameter steel pipe that discharges up the slope and into the drainage channel on the northeast side of the Gypsum Disposal Complex.
Headwall:	N/A
Joint Separations:	N/A
Mis-Alignment:	N/A
Closed/Abandoned Conduits:	N/A

5. Repairs/Mitigation/New Construction Activities Since Last Annual Inspection

The following improvement was performed at the CYDB since the last annual inspection:

- No repairs were evident

6. Recommendations

The following recommendations are offered for the CYDB. Priority codes are included in parenthesis and described in Enclosure K.

- The pond was dredged in 2007 to a cell built in the Dry Fly Ash Stack area. Since that time, sedimentation has continued along the northwest portion of the pond resulting in the effectiveness of the floating boom in this area being compromised (see Photo 2). Sedimentation accumulation was also observed in the southeast portion of the pond (see Photo 7). Consider removing the sediment built up along

the base of the pond in both locations. (Priority 5)

- The pond should also be sounded as needed to determine when dredging will be required. (Priority 5)
- It is recommended that the small trees that are beginning to grow along the south slope be removed in accordance with the guidelines shown in Enclosure F. (Priority 4)
- It is recommended that the depression observed in the south slope be monitored in accordance with the currently established stability monitoring program (See Photo 3). Any evidence of erosion or slope movement within this area should be reported to Stantec.
- The observed slope instability in the face of the south slope should be re-graded and vegetated (see Photo 4). (Priority 3)
- The animal burrows observed on the interior slopes on the east and northeast sides of the CYDB should be repaired in accordance with the guidelines given in Enclosure G. (Priority 3)
- It is recommended that a workplan be prepared to address the rutting/erosion spots observed on the west and north slopes, as well as the erosion detected on the platform for the pump station. It is expected that rip-rap will be needed to stabilize each slope. This work should be conducted in accordance with the general guidelines included in Enclosures H and I. (Priority 4)

Photo 1

Small tree growth along the south slope of the Coal Yard Drainage Basin (CYDB).

Photo 2

Erosion at drainage feature on the west slope of the CYDB.

Photo 3

Depression in dike face. Not previously noted due to vegetation coverage.

Photo 4

Minor surface slip on the south interior dike face.

Photo 5

Rutting and sparse vegetation on the south interior slope face.

Photo 6

Animal burrow observed on the east slope.

Photo 7

Sediment accumulation at primary inlet of CYDB.

Enclosure B

Chemical Treatment Pond

TVA 2011 Annual Inspection Program
Cumberland Fossil Plant (CUF)
Chemical Treatment Pond

1. General Facility Information

Facility Status:	Active	NID Identification:	Not Available
Surface Area (inside dikes):	1.9 Acres	TVA Hazard Classification:	Not Rated
Maximum Height (toe to top of dike):	6.5 feet	Dike Length:	1300 feet (0.25 miles)
Plant Discharge to Facility:	>0.1 MGD Average	Current Pool Elevation:	380.8 feet

2. Site Visit Information

Stantec Inspection Team:	Daniel B. Rogers and James R. Swindler Jr.
TVA Staff Present:	M. Jacob Horton
Field Inspection Date:	06/20/2011
Weather/Site Conditions:	Mostly Sunny, 85 degrees. Slopes were moist from the recent precipitation

3. History/Current and Future Operations

History:	The Chemical Treatment Pond is located east of the Coal Yard Drainage Basin, northeast of the Gypsum Disposal Complex. The southern and eastern boundaries of this area are formed by a slope that was excavated into existing ground. The northern and western boundaries are formed by a dike that seperates this pond from the Coal Yard Drainage Basin.
Current Operations:	Allows metals to precipitate out of solution by recirculating the water using a pump, from the west end to the east end. The pond is only used occasionally. The effluent is pumped to the Active Ash Pond when treatment is complete.
Future Planned Operational Changes:	TVA has intentions to close this facility in the future, however no construction plans have been produced.

4. Stantec Field Observations

See attached Photos and Site Plan Drawing.

4.1. Interior Slopes

Vegetation:	The interior slopes are heavily vegetated above the rip-rap erosion control.
Trees:	None observed.
Wave Wash Protection:	The interior slopes are covered with rip-rap protection along all sides.
Erosion:	Much of the slopes were not visible due to the heavy growth of vegetation around the pond. The portions of the slope that were visible showed no signs of erosion.
Instabilities:	Much of the slopes were not visible due to the heavy growth of vegetation around the pond. The portions of the slope that were visible appear to have no stability problems.
Animal Burrows:	None observed.
Freeboard:	Approximately 6.5 feet.
Encroachments:	Pump station on the west end of the pond.
Slope:	Slopes of the pond are estimated to be on the order of 2H:1V or flatter.

4.2. Crest

Crest Cover and Slope:	Gravel road on the south side, vegetation cover on all other sides. No slope.
Erosion:	None observed.
Alignment:	Generally straight and level.
Settlement/Cracking:	None observed.
Bare Spots/Rutting:	None observed.
Width:	Gravel road on south side, approximately 12 feet.

4.3. Exterior Slopes

Vegetation:	N/A

Trees:	N/A
Erosion:	N/A
Instabilities:	N/A
Uniform Appearance:	N/A
Seepage:	N/A
Benches:	N/A
Foundation Drains, and Seepage Collection Systems:	N/A
Instrumentation:	N/A
Animal Burrows:	N/A
Slope:	N/A
Height:	N/A

4.4. Spillway Weirs/Riser Inlets

Number:	N/A
Size, Type and Material:	N/A
Height of Riser Inlets:	N/A
Access:	N/A
Joints:	N/A
Mis-Alignment:	N/A
Closed/Abandoned Conduits:	N/A

4.5. Outlet Pipes

Number:	N/A
Size, Type and Material:	N/A
Headwall:	N/A
Joint Separations:	N/A

Mis-Alignment: N/A

Closed/Abandoned Conduits: N/A

5. Repairs/Mitigation/New Construction Activities Since Last Annual Inspection

The following improvement was performed at the Chemical Treatment Pond since the last annual inspection:

- Bare areas on the slope that were noted in 2010 were not observed this year. It is presumed that these were repaired by TVA staff.

6. Recommendations

The following recommendation is offered for the Chemical Treatment Pond. Priority codes are included in parenthesis and described in Enclosure K.

- It is recommended that the area around the concrete gutter at the outlet of the recirculation pipe on the east slope of the pond be monitored for erosion (see Photo 1). This area should be examined during recirculation in order to determine if the gutter is capable of containing the flow. If the gutter is overtopped or if splashing occurs causing the water not to be contained in the gutter, the gutter should be widened. (Priority 5)

Photo 1

Recirculation pipe and concrete gutter at the east slope of the pond.

Enclosure C

Active Ash Pond

1. General Facility Information

Facility Status:	Active	NID Identification:	TN16109
Surface Area (inside dikes):	54.6 acres	TVA Hazard Classification:	High
Maximum Height (toe to top of dike):	42 feet (Approx)	Dike Length:	Approximately 4,100 feet (0.78 miles) external; approximately 2,000 feet (0.38 miles) internal. Total is approximately 6,100 feet (1.16 miles).
Plant Discharge to Facility:	21.7 MGD	Current Pool Elevation:	384.23 feet (MSL)

2. Site Visit Information

Stantec Inspection Team:	Daniel B. Rogers and James R. Swindler Jr.
TVA Staff Present:	M. Jacob Horton
Field Inspection Date:	06/20/2011
Weather/Site Conditions:	Mostly Sunny, 85 degrees

3. History/Current and Future Operations

History: The Active Ash Pond is west of the powerhouse and north of the Dry Fly Ash Stack. This disposal area was constructed in 1969. As part of this construction, Wells Creek was relocated in order to construct what was initially known as Disposal Area 1. As a result, portions of the current Active Ash Pond and Dry Stack were constructed over the original location of Wells Creek. Area 1 was located within the perimeter dikes that now include the current ash and gypsum disposal areas. In 1977, the divider dike for the stilling pool to the north (interior divider dike) was constructed of ash. In 1979, the dikes around the Active Ash Pond were raised to elevation 395 feet with clay. In 1986, approximately 300 feet of the west portion of the divider dike between the Ash Pond and the Dry Ash Stack was

constructed of ash. During 1995-1996, the current divider dike between the Ashp Pond and the Dry Stack was constructed (exterior divider dike) to form the current configuration.

Current Operations: The pond receives runoff from the adjacent Dry Fly Ash Stack to the south, the Gypsum Disposal Complex to the southeast, process water from the Bottom Ash Pond, and effluent from various other plant operations and sumps. The effluent flows northwest to the Active Ash Pond Pond and then under a floating skimmer to the Stilling Pond. Runoff from the Dry Fly Ash Stack and Gypsum Disposal Complex flows to the Active Ash Pond via perimeter ditches and piping which extends through the exterior divider dike. The discharge from the Stilling Pond flows to the river via the Condensing Cooling Water Discharge Channel. Four spillways are located along the northeast side of the Stilling Pond with outlets below the adjacent road. These outlets are 48-inch RCP riser pipe/weirs that discharge through four 36-inch RCP sections. Approximately 135,000 dry tons of bottom ash is wet sluiced to the Ash Pond annually. Dewatered bottom ash is reclaimed by pan-scrapers and hauled to construct the Dry Fly Ash Stack.

Future Planned Operational Changes: During the construction of an emergency spillway, the pool level of the ash pond will be lowered to approximately 378 feet elevation. The designs for this construction have been completed and a contract for the work has been issued. The construction is projected to be completed in 2011.

4. Stantec Field Observations

See attached Photos and Site Plan Drawing.

4.1. Interior Slopes

Vegetation: The interior slope of the east corner of the Active Ash Pond is vegetated with phragmites. Phragmite vegetation has been cut along toe of the remaining interior slopes of the Active Ash Pond, the interior divider dike, and the interior slopes of the Stilling Pond; but the base of the phragmite vegetation remains (see Photo 1).

Trees: There were a few small trees in the interior divider dike

	(See Photo 2).
Wave Wash Protection:	The interior divider dike consists of rip-rap along the slope faces. There is rip-rap along the northeast slope of the Stilling Pond at the spillway locations. One area of rip-rap was placed for wave erosion protection on the north interior slope.
Erosion:	Minor erosion was observed around the 36-inch HDPE drain pipe, located along the west end of the exterior divider dike between the Active Ash Pond and the Dry Fly Ash Stack. Several other areas of minor erosion along the divider dike were also observed. Wave action erosion was observed along the toe of the interior slopes of the Active Pond. A small area of rip-rap protection was placed, however more will be required. (TVA Item 1, See Photos 6 and 7)
Instabilities:	The interior slopes appear to be stable.
Animal Burrows:	None observed.
Freeboard:	Approximately 11 feet.
Encroachments:	None observed.
Slope:	The interior slopes of the perimeter dikes of the Active Ash Pond are on the order of 2.5H:1V. The slopes of the interior divider dike are on the order of 1.8H:1V. The interior slope of the exterior divider dike is on the order of 2.2H:1V or flatter.

4.2. Crest

Crest Cover and Slope:	Gravel road on all sides, including divider dikes. No slope of the crest was observed.
Erosion:	None observed.
Alignment:	Generally straight or unicormly curved. No unconformities observed.
Settlement/Cracking:	None observed.
Bare Spots/Rutting:	None observed.
Width:	Approximately 12 feet.

4.3. Exterior Slopes

Vegetation:	The exterior slopes have good vegetation established.
Trees:	None observed.
Erosion:	Tire rutting was observed on the exterior slope along the east side of the Stilling Pond (see Photo 3). This is presumably due to mower rutting.
Instabilities:	The exterior slopes appear to be stable.
Uniform Appearance:	All slopes for the Active Ash Pond and the Stilling Pond are uniform in appearance. The south slope of the Ash Pond does not have an exterior slope, as the Dry Fly Ash Stack serves as its boundary.
Seepage:	Seepage/wet spots were observed at the toe of the perimeter dike and at the midslope of the perimeter dike on the west side of the Active Ash Pond (see Photo 4).
Benches:	None observed.
Foundation Drains, and Seepage Collection Systems:	None observed.
Instrumentation:	As part of the geotechnical exploration conducted between April and July, 2009, Stantec installed four slope inclinometers and 8 piezometers around the crest and at midslope of the Ash Pond. During a system wide project, TVA has had many of the instruments automated.
Animal Burrows:	None observed.
Slope:	The exterior slopes of the perimeter dike are on the order of 2.7H:1V or flatter.
Height:	Approximately 42 feet from toe to crest.

4.4. Spillway Weirs/Riser Inlets

Number:	Four, located on the northeast slope of the Stilling Pond (see Photo 5).
Size, Type and Material:	48-inch riser pipe/weirs.

Height of Riser Inlets:	Unable to measure.
Access:	Walkways from the toe of the northeast interior slope of the Stilling Pond are used to access the spillways. The walkways were in good condition with some rusting observed.
Joints:	Unable to observe.
Mis-Alignment:	None observed.
Closed/Abandoned Conduits:	None observed.

4.5. Outlet Pipes

Number:	Four, located below the adjacent entrance road.
Size, Type and Material:	36-inch RCP sections.
Headwall:	None observed.
Joint Separations:	Unable to observe.
Mis-Alignment:	None observed.
Closed/Abandoned Conduits:	None observed.

5. Repairs/Mitigation/New Construction Activities Since Last Annual Inspection

The following improvements were performed at the Active Ash and Stilling Ponds since the last annual inspection:

- The animal burrow observed to the north of the floating skimmer on the perimeter dike at the entrance to the Stilling Pond has been filled.
- The roots observed on the west side of the pond have been removed.
- Rip-rap wave protection has been added on the north interior slope of the stilling pond.
- The pipe observed in the exterior slope on the west side of the pond was removed. It was not embedded more that 1 foot into the dike. The pipe was removed and the remaining depression was filled.

6. Recommendations

The following recommendations are offered for the Active Ash and Stilling Ponds. Priority codes are included in parenthesis and described in Enclosure K.

- It is recommended that a workplan be prepared to address the erosion locations observed during the inspection. These areas of erosion observed along the exterior divider dike should be monitored and repaired as conditions warrant. (Priority 4) The wave action erosion should be addressed by rip-rap armoring as addressed in the general guidelines (Enclosure J). However, this work should be scheduled after the completion of Work Plan 7.
- It is recommended that the drainage ditch on the east side of the Active Ash Pond be cleaned of phragmites and any accumulated sedimentation. The slope of the ditch in this area should be evaluated to assess if better flow is possible to alleviate the standing water observed. (Priority 4)
- Observation of the toe of the exterior slope on the north side of the Stilling Pond should continue in order to assess if the moist slope is a result of seepage or surface runoff as a result of recent rain events. (TVA Quarterly Inspection Item 2) This should be completed in accordance with the Seepage Action Plan.
- Observation of the moist areas on the midslope and toe of the west exterior slope of the Active Ash Pond should continue. Any changes in the flow or color of these seepage areas should be reported to Fossil Engineering Design Services (EDS). This should be completed in accordance with the Seepage Action Plan.
- Areas where tire rutting is observed along both the interior and exterior slopes should be repaired and reseeded as described in Enclosure H. Special care should be taken, if possible, to not perform maintenance on the slopes of the ponds during conditions that could result in rutting. (Priority 5)
- The trees observed on the interior divider dike should be removed in accordance with the guidelines given in Enclosure F (Priority 4).

Photo 1

Standing water and phragmites in exterior ditch on the east side of the Active Ash Pond.

Photo 2

View of the interior divider dike within the Active Ash Pond. Few small trees visible.

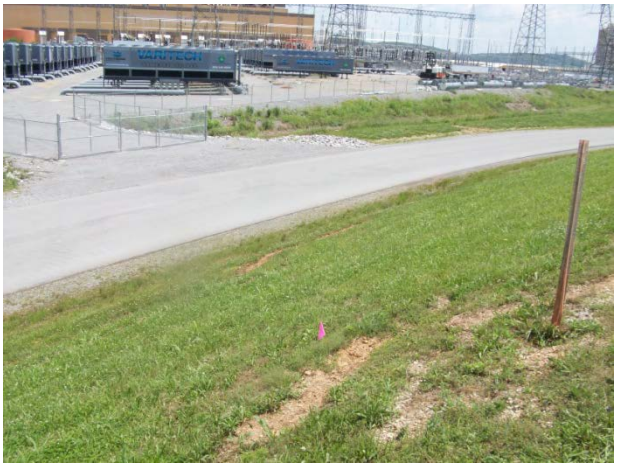


Photo 3

Rutting and absent vegetation on the exterior slope on the east side of the Stilling Pond.

Photo 4

Area at the exterior midslope and toe of the Active Ash Pond where moist/damp areas were observed.

Photo 5

Spillway location on the east side of the Stilling Pond.

Photo 6

Wave action erosion repaired using rip-rap.

Photo 7

Wave action erosion is present.

Enclosure D

Dry Ash Stack

TVA 2010 Annual Inspection Program
Cumberland Fossil Plant (CUF)
Dry Fly Ash Stack and Bottom Ash Pond

1. General Facility Information

Facility Status:	Active		
Surface Area:	113 Acres	Maximum Height (toe to top of stack):	65 feet (south and west sides) 35 feet (north side)

2. Site Visit Information

Stantec Inspection Team:	Daniel B. Rogers and James R. Swindler Jr.
TVA Staff Present:	None
Field Inspection Date:	June 20, 2011
Weather/Site Conditions:	Mostly Sunny, 85F Precipitation during the previous days

3. History/Current and Future Operations

History:

The Dry Fly Ash Stack is located south of the Active Ash Pond and west of the powerhouse. In 1972, Wells Creek was relocated in order to construct old Disposal Area 1. This area was enclosed by the existing perimeter dike and contained sluiced ash. In the 1980s, sluicing operations ceased within Area 1 and began in the current Area 2 to the north. Divider dikes were constructed to separate the current pond from the gypsum and ash stacking operations. In 1995-96, the current divider dike (exterior divider dike) between the Active Ash Pond and Dry Fly Ash Stack was constructed.

In 1996, operations within the Dry Fly Ash Stack began. The Dry Fly Ash Stack is bordered by the Active Ash Pond to the north, the Bottom Ash Pond to the East, the Gypsum Disposal Complex to the south, and perimeter ditches and the old Area 1 perimeter dike to the west.

The Bottom Ash Pond is located on the east side of the Dry Fly Ash Stack. Process water from the Bottom Ash Pond is sent to the Active Ash Pond.

Current Operations:

The current operations are intermingled with the construction activities based on the Work Plan 11. In addition to disposing of the newly generated CCP, the operations on the dry fly ash stack consist of re-grading the slopes to improve the stability factor of safety and maintain compliance with the approved facility permit. Runoff from the stack area travels through

	perimeter ditches to the Active Ash Pond. The bottom ash deposited is removed by an excavator and placed into off-road dump trucks and hauled to construct the Dry Fly Ash Stack.
Future Planned Operational Changes:	No immediate changes planned.

4. Stantec Field Observations

See attached Photos and Site Plan Drawing.

4.1. Exterior Slopes and Benches

Vegetation:	Due to the progress of Work Plan 11, the vegetation and interim soil cover is being removed from the Dry Fly Ash Stack (see Photo 1). The vegetation on the exterior slopes of the perimeter dikes appears to be adequate and maintained regularly, however one area of sparse vegetation was noted (see Photo 6).
Trees:	None observed.
Erosion:	Minor erosion was observed at various locations around the exposed CCP portion of the stack. Due to this being an active work plan, these observed rills will be excavated and graded prior to completion of the project. No significant erosion was noted across the site.
Instabilities:	None observed.
Uniform Appearance	The slopes display a predominant uniform appearance. There is a bench at the approximate midslope that is used for a construction access road (see Photo 2). Whereas the slope of the perimeter dike for the Dry Fly Ash Stack extends to Wells Creek on the west and southwest sides, the slope extends to a divider dike between the Gypsum Disposal Complex and the Dry Fly Ash Stack on the southeast side.
Benches:	There are two benches on the exterior slope on the west side of the stack that serve as access roads. There is one bench that extends along the exterior slope on the east and north sides that is used as an access road.
Slope:	The exterior slopes are on the order of 2.7H:1V or flatter and appear stable.

Height:	Approximately 65 feet on the south and west faces. Approximately 35 feet on the north face.
Other:	One seep and two wet areas were recorded during the annual inspection. The first is located just west of the recent slope repair area on the Gypsum Disposal Complex. It is wet and not flowing or transporting sediment. This area was logged as Seep # 15 on the Seep Log for CUF (see Photo 3). A possible seep was noted on the south side of the stack approximately 1000 feet southeast of the construction bridge (see Photo 4). It is not evident if this is a seep or if the area is wet from the recent precipitation. One wet area was observed on the exterior slope of the stack on the North Side (TVA Item Number 5). Each appears at the approximate midslope of the perimeter dike. The previously identified red-water seeps located at the toe of the perimeter dike near the bridge that crosses Wells Creek are still present. The seeps do not appear to be worse than in past inspections. Due to the high water level of Wells Creek, it was not possible to visually inspect the seeps from creek level. During the geotechnical exploration performed by Stantec between April and July, 2009, three slope inclinometers and six piezometers were installed along the west and southwest slopes of the stack. Three of these piezometers were abandoned during the week of June 13-17, 2011 due to conflicts with the current construction projects. The remaining instruments were automated by a separate TVA contractor.

4.2. Perimeter Drainage Ditches and Down-Drains

Vegetation:	During the course of construction of Work Plan 11, the vegetation has been removed from the perimeter ditch (See Photo 2).
Rip-Rap Channel Lining:	None observed.
Erosion:	Eroded fly ash from the stack has deposited sediment within the ditch in several areas.
Sedimentation in Ditches:	Sedimentation from construction activities was observed in the perimeter drainage ditch (see Photo 5). The side slopes of the ditch show shallow sloughs and scarps due to excavations along the ditch to clean them.
Standing Water in Ditches	Standing water was observed along the entire perimeter

or on Benches:	ditch.
Silted/Impeded Drainage Pipes:	None observed.
Other:	N/A

5. Repairs/Mitigation/New Construction Activities Since Last Annual Inspection

The following improvements and activities were performed at the Dry Fly Ash Stack since the last annual inspection:

- Work Plan 11 construction activities have been initiated. These activities include slope re-grading, ditch cleanout and re-grading, buttressing of the ditch and seepage filter construction (where necessary).
- Previously observed animal burrows have been repaired.
- Previously observed erosion areas have been repaired.
- RHO&M daily operations have been turned over to Trans Ash.

6. Recommendations

The following recommendations are offered for the Dry Fly Ash Stack. Priority codes are included in parenthesis and described in Enclosure K.

- It is recommended that the perimeter drainage ditch be cleaned of large vegetation and sedimentation. (Priority 4) When localized areas of standing water are encountered, it is recommended that these areas be cleared so as to not allow standing water to saturate the ditch and/or the embankment. (Priority 4)
- It is recommended that the areas of seepage noted within this report and on the Seep Log be monitored in accordance with the Seepage Action Plan. Any changes in the flow, size or color of these seepage areas should be reported to EDS.
- The erosion observed on the outslope of the Bottom Ash Pond should be monitored and repaired in accordance with the general guidelines which are included in Enclosure I. (Priority 4)
- The area of sparse vegetation (see Photo 6) observed on the south west exterior slope of the stack should be monitored as part of TVA's Inspection Program and repaired as conditions warrant. This area should be reworked, graded, and a new soil cover be installed suitable to support vegetation. The area should then be seeded. (Enclosure F)

Photo 1

Vegetation and soil cover being removed as part of WP 11

Photo 2

Construction access road and vegetation removal from ditch.

Photo 3

Seep #15 adjacent to the completed slope repair project.

Photo 4

Wet area noted just above the bench on the exterior slope.

Photo 5

Sedimentation and obstructions observed in perimeter ditch.

Photo 6

Sparse vegetation on the slope.

Enclosure E

Gypsum Disposal Complex

1. General Facility Information

Facility Status:	Active		
Surface Area:	153 Acres	Maximum Height (toe to top of stack):	60 feet (south side) 30 feet (north side)

2. Site Visit Information

Stantec Inspection Team:	Daniel B. Rogers and James R. Swindler Jr.
TVA Staff Present:	M. Jacob Horton
Field Inspection Date:	June 20, 2011
Weather/Site Conditions:	Mostly Sunny, 85F

3. History/Current and Future Operations

History:

The Gypsum Disposal Complex is located east of the Dry Fly Ash Stack and south of the powerhouse. It was constructed during 1995-1996 and built over Area No. 1, which was the original ash pond. Approximately 1,100,000 tons of gypsum is produced each year. According to TVA personnel, approximately 50 percent of the gypsum is conveyed directly to the adjacent wallboard manufacturing company east of the plant. Gypsum that does not go directly to the wallboard plant is stored in a shed for later use by the wallboard plant or is disposed of on the stack. If the dewatering plant goes offline, slurry is piped to the stack where it travels through a ditch to a sump at the northwest corner. A Siphon system then discharges the flow to the Bottom Ash Pond.

The complex was constructed in several stages beginning with construction of a rock drainage blanket to collect and divert water away from the base. Currently the complex is separated into north and south cells. The complex consists of an upper gypsum dike with a perimeter ash dike at lower elevations. Below the ash dike along the southwest and southeast sides, the area is encompassed by the outer clay perimeter dike at lower elevations. Along the remaining sides, the ash divider dike separates the area from the Dry Fly Ash Stack to the northwest and the perimeter ash dike separates the area from the Coal Yard Drainage Basin and the Chemical Treatment Pond to the northeast. Discharge for the complex is through a reinforced concrete pipe (RCP) riser to outlet pipes in the northwest corner of the complex into the adjacent perimeter

ditches. The perimeter ditches around the Gypsum Disposal Complex flow to the north along the neighboring Dry Fly Ash Stack and ultimately into the Active Ash Pond.

Current Operations: Beginning in May 2009, TVA diverted the flow of gypsum slurry to Synthetic Materials for dewatering prior to stacking the gypsum in the complex. Occasionally, due to power outages or maintenance, slurry must still be pumped to the top of the complex.

Future Planned Operational Changes: Currently, Stantec and TVA are in the early stages of preparing a five- to seven-year operation plan for the facility while a new dry disposal facility is being designed, permitted and constructed. Modifications being considered include constructing small lined ponds on top of the gypsum stack and significantly reducing the amount of water which could be impounded. Pond features include a 60-mil geomembrane protected by 12 inches of gypsum. A 24-inch “marker” layer of crushed rock will overlie the protective gypsum layer. Each pond will be about 11 feet deep. Sluicing would alternate between ponds to allow for settlement of solids and subsequent movement.

4. Stantec Field Observations

See attached Photos and Site Plan Drawing.

4.1. Exterior Slopes and Benches

Vegetation: Vegetation has not yet been established on all of the exterior slopes that are currently being constructed of gypsum.

Most of the perimeter ash dike slopes are heavily vegetated, including the growth of phragmites along the perimeter ditches. There are sparse locations which are rutted and barren of vegetation located on the perimeter ash dike on the south side (see Photo 1).

The perimeter clay dike slopes have good vegetation growth.

Trees: None observed.

Erosion: Seep and erosion were observed on the ash perimeter dike between subdrain-14 and subdrain-15.

Erosion was observed on the access road at the crest of the bottom ash road dike on the northeast and northwest corners

	of the complex (see Photos 2 (TVA Item Number 7) and 3) as well as above the CYDB pump station (TVA Item Number 6, see Photo 4).
Instabilities:	All exterior slopes currently appear to be stable. A slope failure along the perimeter dike at the southwest corner of the complex occurred in 2005 and was addressed by the construction of a seepage blanket and rip-rap stability berm.
Uniform Appearance	The complex consists of an upper gypsum dike with a perimeter ash dike at lower elevations. Below the perimeter ash dike along the southwest and southeast sides, the area is encompassed by the outer clay perimeter dike at lower elevations. Along the remaining sides, the ash divider dike separates the area from the Dry Fly Ash Stack to the northwest and the perimeter ash dike separates the area from the Coal Yard Drainage Basin and the Chemical Treatment Pond to the northeast.
Benches:	There are two benches that extend along the northeast, southeast, and south sides of the complex that serve as access roads. One bench extends along the northwest side that serves as an access road.
Slope:	The gypsum dikes surrounding the two cells are being constructed on approximate 3H:1V slopes or flatter. The ash divider dike along the northwest side of the gypsum stack has relatively steep slopes on the order of about 1.5H:1V. The exterior slopes of the perimeter clay dike along the southwest and southeast sides are on the order of 2.7H:1V or flatter.
Height:	Approximately 60 feet on the south side and 30 feet on the north side.
Other:	Two 24-inch corrugated metal pipes (CMP) were observed on the southwest side of the perimeter ash dike. On March 2, 2011, these pipes were explored by CCTV. The outlet of one of the pipes was damaged and it was determined that it extended only 3 to 5 feet into the dike. The other pipe was measured to a length of 114.5 feet and proceeds perpendicular into the complex from the face of the dike. No turns or wyes were noted.

During the course of the annual inspection, the seep log was reviewed and the condition of each seep was noted. Each of the previously recorded seeps around the perimeter of the gypsum stack was observed to have maintained its' size with the exception of CUF Seep #8. Seep #8 has shrunk in size by about 8 feet in width. TVA Seep #3 continues to flow but does not transport sediment.

During the geotechnical exploration perfomed by Stantec between April and July, 2009, six slope inclinometers and thirteen piezometers were installed on the south, east, and north sides of the complex. Five of these piezometers were abandoned during the week of June 13-17, 2011 for construction. The remaining instruments were automated by a separate TVA contractor.

4.2. Perimeter Drainage Ditches and Down-Drains

Vegetation:	The perimeter ditch along the southwest and southeast sides has a heavy growth of phragmites along most of its length.
Rip-Rap Channel Lining:	There is a rip-rap channel on the southwest corner of the complex, used for the drainage of the interior cells. The flow drops several feet from the pipes down to the rip-rap channel below, allowing for the possibility of erosion of the toe (Photo 5).
Erosion:	Erosion was noted below subdrains-12, 13 18 and 21 (see Photos 6, 7, 8 and 9)
Sedimentation in Ditches:	The perimeter ditch along the southwest and southeast sides has sedimentation along most of its length. The upstream slopes of the perimeter dike along the ditch consist of shallow sloughs and scarps throughout due to excavations for ditch cleaning.
Standing Water in Ditches or on Benches:	The perimeter ditch has standing water along the entire length of the complex. This is potentially due to the excess material in the ditch from the construction activities along the Dry Ash Stack.
Silted/Impeded	None observed.

Drainage Pipes:

Other: The siphon system was observed on the northwest corner of the stack to aid in the drainage of surface water. Although the running of the siphon did not occur during the inspection, Trans Ash personnel indicated that the siphon system had operated without problem.

5. Repairs/Mitigation/New Construction Activities Since Last Annual Inspection

Since the annual inspection in 2010, major construction activities have commenced.

During the course of a permit review, it was determined that the Gypsum Disposal Complex was being built outside of the permitted plan limits. To correct this, a work plan was issued (CUF-110310-WP-11) that provides for the re-grading of the facility to maintain compliance with the approved permit and to provide a base for the lined ponds to be constructed. Due to the extensive nature of the re-grading project, observations are only being listed that pertain to the perimeter clay dikes and general maintainence practices.

- A workplan has been conducted to determine the depth of the two 24-inch CMP structures located on the southwest side of the ash perimeter dike. A camera was used to determine the depth and propagation of the pipes into the stack. One pipe was damaged and penetrated only 3 to 5 feet into the dike. The second pipe was observed to be 114.5 feet in length and not bent, elbowed or wyed.
- The animal burrow observed on the perimeter ash dike on the south side of the complex appears to have been repaired.

6. Recommendations

The following recommendations are offered for the Gypsum Disposal Complex. Priority codes are included in parenthesis and described in Enclosure K.

- As the interior cells drain to the southwest corner, flow drops several feet from the discharge pipes down to the rip-rap channel below. Although the rip-rap appears to be controlling erosion at the toe of the slope, continued erosion could undermine the toe. It is recommended that this area be monitored for erosion and stabilization issues during the course of TVA's inspection program and appropriate action taken if problems are noted.
- It is recommended that the perimeter drainage ditch be cleaned of phragmites and sedimentation. (Priority 4) When localized areas of standing water are encountered, it is recommended that these areas be cleared so as to not allow the standing water to saturate the ditch and/or the embankment. (Priority 4)
- It is recommended that the seep locations observed on the exterior slopes of the

perimeter dike on the southwest side continue to be monitored as described in the Seepage Action Plan. Any changes to the flow or color of these seepage areas should be reported to Stantec.

- It is recommended that the areas of erosion at the outlet of the subdrains along the exterior slopes of the ash dike above the perimeter ditch be repaired in accordance with the guidelines included in Enclosure I. This work should be coordinated with the execution of Work Plans 11 and 8.
- It is recommended that the seeps (TVA-CUF Seep Log) along the perimeter clay dike at the southwest corner of the complex and the perimeter ash dike on the south perimeter clay dike (see Photo 10) be monitored for signs of movement or changes in seepage that may be indicative of slope failure. Any changes should be reported to Stantec.
- The area of erosion of the access road to the north side of the complex should be monitored and repaired in accordance with the guidelines included in Enclosure I. (Priority 5)
- It is recommended that the observed rutting be repaired in accordance with the general guidelines presented in Enclosure H. Additionally, alternate mowing procedures should be used for the complex to prevent surface rutting of the vegetation cover. (See Photos 9, 13 and 14)
- It is recommended that a work plan be prepared that would address the areas of barren cover soil (see Photo 11) along the exterior slopes of the complex. This work should be conducted in accordance with the guidelines presented in Enclosure F (Priority 4)
- The inlet and outlets of all drainage pipes should be monitored as part of TVA's Inspection Program and cleared as conditions warrant (see Photo 12).

Photo 1

Vegetation and rutting along bottom ash road dike crest.

Photo 2

Erosion of road at the northeast corner of the complex.

Photo 3

Erosion at north end of the diversion ditch between Dry Fly Ash Stack and Gypsum Stack Complex.

Photo 4

Erosion on bottom ash road dike above the CYDB pump station

Photo 5

Pipes and rip-rap channel used for drainage of the complex on the southwest corner.

Photo 6

Erosion noted below subdrain 12.

TVA 2010 Annual Inspection Program
Cumberland Fossil Plant (CUF)
Gypsum Disposal Complex
Photos

Photo 7

Erosion noted below subdrain 13.

Photo 8

Erosion noted below subdrain 18.

Photo 9

Ponding/Rutting below subdrain #21.

TVA 2010 Annual Inspection Program
Cumberland Fossil Plant (CUF)
Gypsum Disposal Complex
Photos

Photo 10

Area of recent seep activity located on the perimeter ash dike on the south side of the complex.

Photo 11

Area of barren soil located on the perimeter ash dike on the south side of the complex.

Photo 12

Culvert at access crossing between Gypsum Stack/Dry Ash Stack Diversion and Perimeter Dikes.

Photo 13

Rutting on exterior slope of the dike

Photo 14

Rutting on exterior slope of the dike

Enclosure F

GL – Tree Removal

General Guidelines for Tree Removal on Slopes at TVA Fossil Plants

Identification

Trees and heavy brush growth should be controlled on TVA dams and dikes. If left in place, trees can result in the creation of seepage paths within the embankment. Allowing vegetation to become overgrown restricts the level of inspection that can be performed on the structure. General guidelines for removal of trees and maintenance of vegetation are provided below. Evaluations other than those outlined below shall be made by a geotechnical engineer in consultation with facility representatives on a case-by-case basis.

Guidelines for Tree Removal and Maintenance of Vegetation

Tree Removal

At locations where it is not reasonable to remove trees by a mowing them with a bush hog or with similar mowing equipment:

- All trees shall be cut using a handsaw or chainsaw and the cut tree and branches discarded.
- Remove the remaining tree trunk, stump, and rootwad.
- Grub any remaining roots of the tree so that only 2 inches or smaller roots are left in place.
- The resulting cavity from removal of the rootwad shall be cleaned of loose soil and debris.
- The cavity shall then be backfilled with cohesive soil and compacted and the area seeded to re-establish vegetation. If the tree has been removed from along the upstream or downstream face of a slope, benches shall be cut into the slope face where the cavity is to be backfilled. This will allow for a proper bond between the existing dike and the backfill being used to reform the slope. If benches are needed, bench heights shall not exceed 4 to 5 feet in height.

Maintenance of Vegetation

- Mowing is recommended at regular intervals to allow for appropriate inspection of embankment slopes.
- If areas lacking vegetation are observed during mowing and clearing operations or subsequent inspections, the areas should be seeded to re-establish vegetation as soon as practicable.

Enclosure G

Animal Burrow Repairs

General Guidelines for Repair of Animal Burrows at TVA Fossil Plants

Identification

Animal burrows are relatively common along slopes of dams and dikes. If left untreated, these burrows can result in the creation of seepage paths through the embankment. Additionally tunnels may eventually collapse resulting in surface irregularities in the embankment. General guidelines for repair of animal burrows are provided below. However, if the burrow extends more than three (3) feet below the embankment surface or extends across a dam, the repair of these features should be evaluated by a geotechnical engineer on a case-by-case basis so that appropriate recommendations can be made.

Guidelines for Burrow Repair

It is recommended that shallow animal burrows (up to 3 feet) shall be repaired with surface treatment methods as follows:

- Animals shall be captured and removed from the area. It is recommended that a local conservation representative be consulted prior to this action.
- The animal burrow shall be excavated and cleaned of excess soil along its pathway up to a depth of 3 feet. With this type of repair, an isolated excavated area of the embankment is exposed.
- The excavated area shall be backfilled with compacted cohesive material.
- If the burrow extends more than three feet into the embankment, a geotechnical engineer shall further evaluate the burrow depth and recommend a deep burrow treatment method or other exploratory methods.
- One possible method which may be recommended to treat a deep burrow can consist of a special grout (flowable fill) pumping system with a hose inserted into the burrow.

Ultimately, these repairs will not prevent rodents from creating new burrows within dam embankments. Accordingly, continual efforts must be made to discourage rodent activity. Mowing of vegetation on the slopes / crest of the embankment and trimming of water-side vegetation at regular intervals will tend to discourage rodents from re-establishing burrows along the dike and will allow timely observation of new activity if it occurs.

Enclosure H

Rutting Repair

General Guidelines for Rutting Repair at TVA Fossil Plants

Identification

Rutting due to maintenance vehicle traffic can commonly occur along dike crests, slopes, and other areas at TVA fossil plant facilities. It is typically caused by near-surface materials which have become weak over time because of moisture infiltration. Repeated passes of equipment over weakened materials can lead to rutting. Maintenance traffic/equipment should avoid wet/rutted areas until repairs can be made. General guidelines for the repair of rutting are provided below. The following guide is intended to be applicable for minor to moderate cases of rutting, and generally consists of reworking the upper portion of the affected area, followed by re-shaping to provide positive surface drainage. Where widespread or extensively deep rutting has occurred or is recurring, case-specific engineering evaluations may be needed.

Guidelines for Rutting and Repair

- Drain any standing water and undercut affected areas to remove rutted and overly wet/soft materials. The undercut depth will be determined by TVA in the field, depending on the severity of the rutting.
- Fill undercut area with clay or bottom ash material and compact in 6 to 8 inch lifts to restore original ground line. Excavated material can be re-used if it is free of organics and can be dried to facilitate re-compaction. Otherwise, borrow material will be needed. For compaction, use hand held jumping jacks or small power equipment.
- Grade and shape repaired areas to provide positive/improved drainage. For dike crests, grade the area to drain inwardly toward the pond or perimeter ditch, as applicable. Re-grade surrounding areas and/or drainage ditches to improve drainage, if possible.
- Repaired surfaces or dike crests that are to be used as access roads should be topped with crushed stone or bottom ash. The thickness should be equal to that which was originally in place prior to the repair, or as judged by TVA to be sufficient for the expected amount of vehicle/equipment traffic.
- For other repaired areas, place seed and cover with erosion control blanket to re-establish vegetation. Materials and placement of erosion control blankets should comply with the following specifications, depending on the state in which the work is being performed.

Kentucky Plants –	KYTC Standard Specifications, Sections 212.03.03 E and 827.07
Tennessee Plants –	TDOT Standard Specifications, Section 805
Alabama Plants –	ALDOT Standard Specifications, Section 659

Enclosure I

Rill and Gully Erosion Repair

General Guidelines for Rill and Gully Erosion Repair at TVA Fossil Plants

Identification

Erosion features can commonly occur along dike slopes, dry stack slopes, or other sloped surfaces at TVA fossil plant facilities. Erosion normally appears in the form of rills (shallow channels) and gullies (larger and deeper eroded channels) and is formed by concentrated flow of storm water runoff, especially on bare slopes or where vegetation is sparse. If left untreated, the rills and gullies can progress in size and could lead to slope instability or other adverse issues. General guidelines for the repair of rills and gullies are provided below. The following guide is intended to be applicable to minor to moderate cases of rill/gully erosion. Where widespread or extensively deep gullies have formed or are recurring, case-specific engineering evaluations may be needed.

Guidelines for Rill and Gully Erosion Repair

Shallow Rills and Gullies:

For cases where shallow rills and gullies are present, repair should consist of the following:

- Dump and spread clay soil to fill, re-grade, and shape affected areas to conform to original ground line. Tracking and blading material with a dozer should be performed until the original ground line is reformed and material is reasonably compacted.

- Repaired areas should be seeded to re-establish vegetative cover. Erosion control blankets should be placed over re-graded areas following seeding. Materials and placement of erosion control blankets should comply with the following specifications, depending on the state in which the work is being performed.

Kentucky Plants –	KYTC Standard Specifications, Sections 212.03.03 E and 827.07
Tennessee Plants –	TDOT Standard Specifications, Section 805
Alabama Plants –	ALDOT Standard Specifications, Section 659

Deep Rills and Gullies:

For deep gullies that cannot be repaired as described above, the following filling procedures apply:

- Clean loose soil/debris from bottom and sides of gullies.

- Place and compact clay in 6 inch lifts using small compaction equipment or hand-held tampers. Vibratory plate compactors are not applicable for clay. Filling should start at the toe (or lowest elevation) and progress upslope.

- In some cases, over-excavation may be required to create benches to facilitate compaction on level surfaces. Benching, if required, will likely have to be performed by hand methods or using small excavation equipment.

- If several side-by-side deeper gullies are present in an area to be repaired, it may be more practical to rework the entire affected area to facilitate use of larger equipment. In this case, slight over-excavation of the slope face will be needed so that foundation benches can be cut to facilitate compaction on level surfaces. Filling should start at the lowest elevation and progress upslope.

- Final filling/shaping to reform the original ground line can be executed by tracking and blading with a dozer.

- Repaired areas should be seeded to re-establish vegetative cover. Erosion control blankets should be placed over re-graded areas following seeding. Materials and placement of erosion control blankets should comply with the following specifications, depending on the state in which the work is being performed.

Kentucky Plants –	KYTC Standard Specifications, Sections 212.03.03 E and 827.07
Tennessee Plants –	TDOT Standard Specifications, Section 805
Alabama Plants –	ALDOT Standard Specifications, Section 659

Enclosure J

Wave Wash Riprap Protection

General Guidelines for Wave Wash Erosion Repair and Riprap Protection at TVA Fossil Plants

Identification

Wave erosion should be controlled on TVA facilities to maintain the integrity of dams and dikes. When present, wave wash erosion typically occurs along interior slopes of dikes near pool level. If left unrepaired, erosion can expand, deepen, and can eventually lead to interior slope sloughing. General guidelines for repair of wave erosion using riprap are provided below.

Guidelines for Wave Wash Erosion Repair and Riprap Protection

The following describes repair of wave wash erosion using riprap protection:

- Vegetation and loose soil should be removed within the affected slope areas to be repaired. This includes undercutting the slope a minimum of 12 inches to remove vegetation and associated roots. The minimum vertical extent of the vegetation removal should extend from one-foot below the pool level upwardly to two feet above pool level.

- Place non-woven geotextile fabric along the slope where vegetation and loose soil have been removed. Use fabric meeting or exceeding the following designations, depending on the state in which the work is being performed.

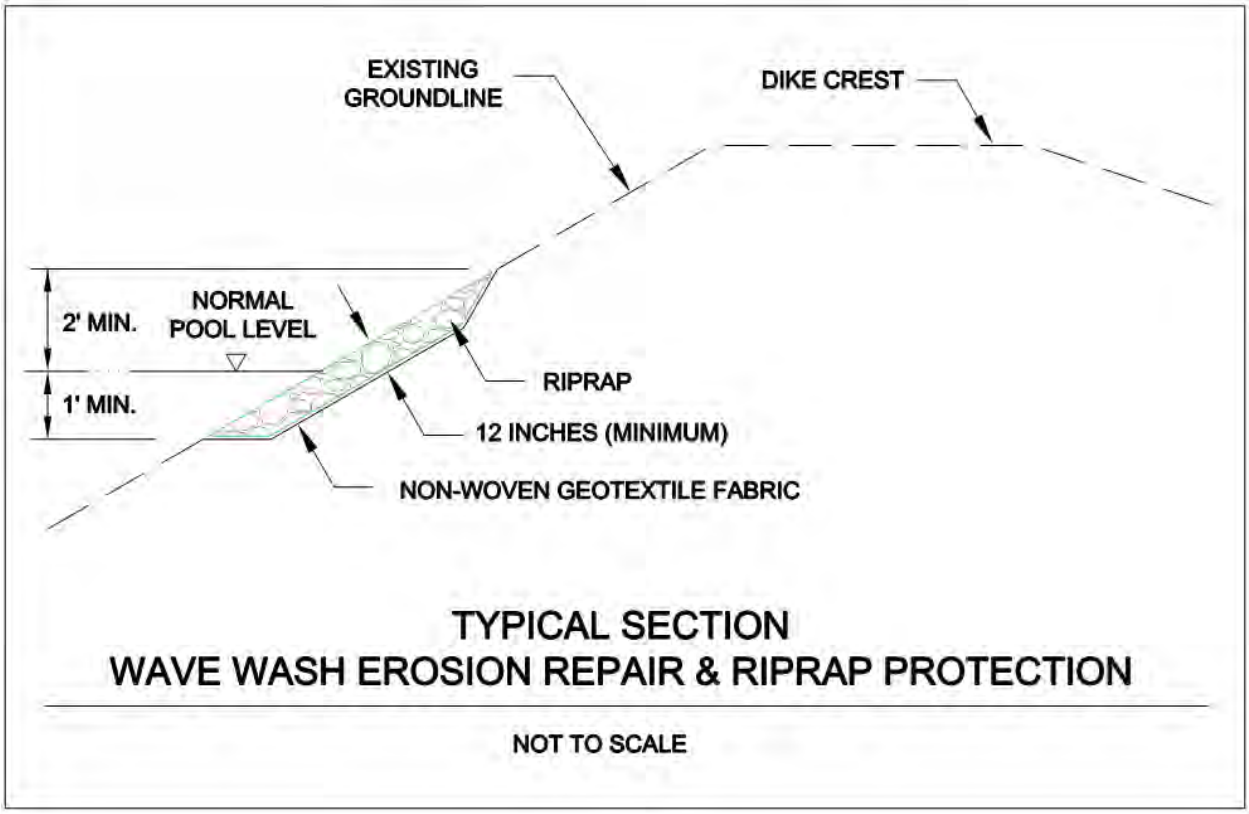
Kentucky Plants -	KYTC Type I Geotextile Fabric
Tennessee Plants -	TDOT Type III Geotextile Fabric
Alabama Plants -	Fabric conforming to Section 608 of ALDOT Standard Specifications

- Place riprap over the geotextile fabric. An excavator should be used to place the riprap in layers (starting from the bottom). Place thickness of riprap to conform to original ground line, or as necessary to create a stable slope face. Use riprap meeting the following designations, depending on the state in which the work is being performed.

Kentucky Plants -	KYTC Class II Channel Lining
Tennessee Plants -	TDOT Class A-1 Machined Riprap
Alabama Plants -	ALDOT Class 2 Riprap

- Field adjustments may be necessary as the work progresses, depending on actual conditions encountered.

A typical cross-section is presented on the following page.



Enclosure K

Maximo – Dam Safety Priorities

Dam Safety Priorities

	Description
1	Urgent - Correct Immediately
2	Complete Within 1 Week of inspection
3	Complete Within 1 Month (30 days) of Inspection
4	Complete Within 6 Months of Original Entry Date
5	Complete Within 1 Year of Original Entry Date
6	Complete Within 3 Years of Original Entry Date
7	Complete Within 5 Years of Original Entry Date
8	Work During Scheduled Outage - Blank Until Outage is Scheduled

Enclosure L

3rd Quarter Dike Inspection CUF 6.20.11

TVA Monthly /Quarterly/Special Facility Inspection Form — Form Date 6-01-10

1. Site Name: CUF 2. Facility Name: Ash Complex
3. Date and Start Time of Inspection: 6/20/11 10 am CST
4. Operator Name: Trans Ash
5. Inspection Method: X Walk ☐ Ride ☐ Both
(KNOWN KEY DEFICIENCIES MUST BE INSPECTED)
6. Inspector's Name(s): Griffin Lifsey, Robert Bagwell, Shane Harris, Shannon Bennett, Jacob Horton, Jake Booth, Bronson Reed, Mike Hulslander, Jason Toler, Brian Diaz, Jim Swindler, Mick McClung, Stuart Harris
7. Hazard Classification: ☐ High ☐ Significant ☐ Low
8. Inspection Frequency: ☐ MONTHLY X QUARTERLY (MUST BE WALKED) ☐ SPECIAL (after significant rain or earthquake event)
9. Current weather conditions Sunny 87°F Prior Conditions, if notable Rain Previous Days.

Check the appropriate box below. If not applicable, record "N/A". Provide comments when appropriate. Any other areas that should be brought to the attention of the Program Manager should also be noted in the "Comments" section. Indicate the locations of any areas identified, and photograph and attach to the form. Previous observation forms should be reviewed and any NEW observations or degradation of pervious conditions should be reported on this inspection form.
(NOTE - ONE FORM PER FACILITY)

	Yes	No		Yes	No
10. Pre-Job Safety Briefing Performed	✓		**15. DIKE TOE AREAS**		
11. Activity / Construction on/ at facility	✓		A. Seepage ○ New X Existing	✓	
12. DIKE CREST			○ Clear/Cloudy/Red/Muddy		
A. Settlement / Cracking		✓	○ Flow Increased / Decreased/Same Marked with signs		gpm
B. Rutting		✓	○ Aquatic Vegetation Growing		
C. Lateral Displacement		✓	○ Ash or Clay Deposits Below Seep Outlet		
D. Erosion		✓	B. Boils ○ New ○ Existing		✓
13. INTERIOR / EXTERIOR DIKE SLOPES			○ Clear/Cloudy/Red/Muddy		
A. Minimum Freeboard		ft.	○ Flow Increased / Decreased/Same		gpm
B. Current Freeboard		ft.	○ Growing in Size		
C. Instabilities (Sloughs or Slides)		✓	C. Sinkholes/Depressions ○ New ○ Existing		✓
D. Erosion	✓		**16. SEEPAGE COLLECTION SYSTEM**		
E. Sinkholes/Depressions ○ New ○ Existing		✓	A. Estimated Flow Measurement	N/A	gpm
F. Vegetation / Brush / Trees	✓		B. Increased Flow		
○ Heavy/(Adequate)/Sparse/Bare			C. Emitting Clear or Dirty Water		
G. Animal Burrows ○ New ○ Existing		✓	**17. SPILLWAY WEIRS & OUTLETS**		
H. Seepage ○ New X Existing			A. Decant Riser Misaligned		✓
○ Clear/Cloudy/Red/Muddy			B. Decant Pipe Joints		✓
○ Flow Increased/Decreased/Same		gpm	○ Leaking		✓
○ Ash or Clay Deposits Below Seep Outlet			○ Separated		✓
I. Seep around Drain Pipe(s)		✓	C. Headwall In Good Condition		
○ Clear/Cloudy/Red/Muddy			**18. OPERATIONS & MAINTENANCE**		
14. DEFICIENCIES			A. Routine O&M Performed	✓	
A. Prior Key Deficiencies Checked	✓		B. Weekly Observations Performed	✓	
B. New Deficiencies Identified / Flagged	✓		C. Any Changes in Operations		✓
C. Immediate Actions Taken (Note Below)		✓			
D. Photos of deficiencies attached	✓				

19. Major adverse changes in these items could cause instability and should be reported to the Program Manager as soon as possible for further evaluation. Adverse conditions noted in these items should normally be described (extent, location, etc.) in the space below and on the backside of this sheet if needed.

NOTE: Quarterly Inspection Deficiencies to be documented on spreadsheet with applicable latitude and longitude coordinates referenced. SHOW ALL QUARTERLY INSPECTION DEFICIENCIES ON AERIAL PHOTOS

Item #	Comments/New Observations/Action Taken:

20. PA(E) was Notified of New Key Deficiency: (Date / Time)

21. Who else Notified of New Key Deficiency: (Date / Time)

22. I hereby attest the above is original information (not reproduced) based on actual field observations made during the period indicated, by either myself or an appointed representative and are accurate, complete, and correct to the best of my knowledge.

Period Covered:
From: Apr 2011 To: Jun 2011 Signature: Griffin R. Lifsey Date: 6/27/11

TVA Monthly /Quarterly/Special Facility Inspection Form

Form Date 6-01-10

1. Site Name: CUF 2. Facility Name: Gypsum Complex
3. Date and Start Time of Inspection: 6/20/11 10 am CST
4. Operator Name: Trans Ash
5. Inspection Method: X Walk ☐ Ride ☐ Both
(KNOWN KEY DEFICIENCIES MUST BE INSPECTED)
6. Inspector's Name(s): Griffin Lifsey, Robert Bagwell, Shane Harris, Shannon Bennett, Jacob Horton, Jake Booth, Bronson Reed, Mike Hulslander, Jason Toler, Brian Diaz, Jim Swindler, Mick McClung, Stuart Harris
7. Hazard Classification: ☐ High ☐ Significant ☐ Low
8. Inspection Frequency: ☐ MONTHLY X QUARTERLY (MUST BE WALKED) ☐ SPECIAL (after significant rain or earthquake event)
9. Current weather conditions Sunny 87°F Prior Conditions, if notable Rain Previous days

Check the appropriate box below. If not applicable, record "N/A". Provide comments when appropriate. Any other areas that should be brought to the attention of the Program Manager should also be noted in the "Comments" section. Indicate the locations of any areas identified, and photograph and attach to the form. Previous observation forms should be reviewed and any NEW observations or degradation of pervious conditions should be reported on this inspection form.
(NOTE - ONE FORM PER FACILITY)

	Yes	No
10. Pre-Job Safety Briefing Performed	✓	
11. Activity / Construction on/ at facility	✓	
12. DIKE CREST		
A. Settlement / Cracking		✓
B. Rutting		✓
C. Lateral Displacement		✓
D. Erosion		✓
13. INTERIOR / EXTERIOR DIKE SLOPES		
A. Minimum Freeboard	ft.	
B. Current Freeboard	>4' ft.	
C. Instabilities (Sloughs or Slides)		✓
D. Erosion	✓	
E. Sinkholes/Depressions ○ New ○ Existing		✓
F. Vegetation / Brush / Trees	✓	
○ Heavy/(Adequate)/Sparse/Bare		
G. Animal Burrows X New ○ Existing	✓	
H. Seepage X New X Existing	✓	
○ Clear/Cloudy/Red/Muddy		
○ Flow Increased/Decreased/Same	gpm	
○ Ash or Clay Deposits Below Seep Outlet		
I. Seep around Drain Pipe(s)		✓
○ Clear/Cloudy/Red/Muddy		
14. DEFICIENCIES		
A. Prior Key Deficiencies Checked	✓	
B. New Deficiencies Identified / Flagged	✓	
C. Immediate Actions Taken (Note Below)		✓
D. Photos of deficiencies attached	✓	

	Yes	No
15. DIKE TOE AREAS		
A. Seepage X New ○ Existing	✓	
○ Clear/Cloudy/Red/Muddy		
○ Flow Increased / Decreased/Same	gpm	
○ Aquatic Vegetation Growing		
○ Ash or Clay Deposits Below Seep Outlet		
B. Boils ○ New ○ Existing		✓
○ Clear/Cloudy/Red/Muddy		
○ Flow Increased / Decreased/Same	gpm	
○ Growing in Size		
C. Sinkholes/Depressions ○ New ○ Existing		✓
16. SEEPAGE COLLECTION SYSTEM		
A. Estimated Flow Measurement	N/A gpm	
B. Increased Flow		
C. Emitting Clear or Dirty Water		
17. SPILLWAY WEIRS & OUTLETS		
A. Decant Riser Misaligned		✓
B. Decant Pipe Joints		✓
○ Leaking		✓
○ Separated		✓
C. Headwall In Good Condition		
18. OPERATIONS & MAINTENANCE		
A. Routine O&M Performed	✓	
B. Weekly Observations Performed	✓	
C. Any Changes in Operations		✓

19. Major adverse changes in these items could cause instability and should be reported to the Program Manager as soon as possible for further evaluation. Adverse conditions noted in these items should normally be described (extent, location, etc.) in the space below and on the backside of this sheet if needed.

NOTE: Quarterly Inspection Deficiencies to be documented on spreadsheet with applicable latitude and longitude coordinates referenced. SHOW ALL QUARTERLY INSPECTION DEFICIENCIES ON AERIAL PHOTOS

Item #	Comments/New Observations/Action Taken:

20. PA(E) was Notified of New Key Deficiency: (Date / Time)

21. Who else Notified of New Key Deficiency: (Date / Time)

22. I hereby attest the above is original information (not reproduced) based on actual field observations made during the period indicated, by either myself or an appointed representative and are accurate, complete, and correct to the best of my knowledge.

Period Covered:
From: Apr 2011 To: Jun 2011 Signature: Griffin R. Lifsey Date: 6/27/11

LOCATION: Cumberland Fossil Plant - 3rd Quarter FY2011 Dike Inspection
WEATHER: 87 degrees F, Sunny
INSPECTION BY: Robert Bagwell, Shane Harris, Griffin Lifsey, Jacob Horton, Jake Booth, Bronson Reed, Mike Hulslander, Jim Swindler, Nick McClung, Stuart Harris, Shannon Bennett, Jason toler, Brian Diaz
DATE: 06/20/2011

ITEM NO.	DESCRIPTION	PICTURE NO.	POINT NO.	NORTHING	EASTING	COMMENTS
1	Wave erosion	1179	1000	733431.71	1511088.16	To be addressed as part of WP-7
2	Possible seeapge area (5' x 15')	1085	2000	732097.84	1511453.16	Monitor to determine if seep. If so notify Engineering and add to Seepage Log
3	Several small trees on slope	1086	2001	730414.07	1512726.13	Repair in accordance to the General Guidelines
4	(2) Animal burrows	1087	2002	730244.73	1512923.23	Repair in accordance to the General Guidelines
5	Possible seeapge area (50' L)	5707	5000	731152.02	1511747.64	Monitor to determine if seep. If so notify Engineering and add to Seepage Log
6	Erosion (typical of top of slope)	5708	5001	730352.60	1512581.31	Repair in accordance to the General Guidelines
7	Erosion/rill (15' L X 1' D)	5709	5002	729203.14	1514021.32	Previously Identified (FY11 - 2nd Quarter PT# 1013) Repair in accordance to the General Guidelines
8	Erosion under Drain #18 (3' dia)	5710	5003	728035.03	1513450.84	Repair in accordance to the General Guidelines
9	Erosion under Drain #11 (10' L rill)	5711	5004	728049.57	1512059.18	Repair in accordance to the General Guidelines
10	Erosion under Drain #12 (10' L)	5712	5005	728101.93	1511815.72	Repair in accordance to the General Guidelines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Enclosure M

Annual Report Survey Coordinates and Plan View

LOCATION: Cumberland Fossil Plant - 3rd Quarter FY2011 Dike Inspection
WEATHER: 87 degrees F, Sunny
INSPECTION BY: Robert Bagwell, Shane Harris, Griffin Lifsey, Jacob Horton, Jake Booth, Bronson Reed, Mike Hulslander, Jim Swindler, Nick McClung, Stuart Harris, Shannon Bennett, Jason toler, Brian Diaz
DATE: 6/20/2011

ITEM NO.	DESCRIPTION	PICTURE NO.	POINT NO.	NORTHING	EASTING	Recommendations
3rd Quarter - Ash Pond						
1	Wave erosion	1179	1000	733431.71	1511088.16	To be addressed as part of WP-7
2	Possible seeapge area (5' x 15')	1085	2000	732097.84	1511453.16	Monitor to determine if seep. If so notify Engineering and add to Seepage Log
3rd Quarter - Coal Yard Drainage Basin						
3	Several small trees on slope	1086	2001	730414.07	1512726.13	Repair in accordance to the General Guidelines
3rd Quarter - Chemical Treatment Pond						
4	(2) Animal burrows	1087	2002	730244.73	1512923.23	Repair in accordance to the General Guidelines
3rd Quarter - Dry Fly Ash Stack and Bottom Ash Pond						
5	Possible seeapge area (50' L)	5707	5000	731152.02	1511747.64	Monitor to determine if seep. If so notify Engineering and add to Seepage Log
3rd Quarter - Gypsum Disposal Complex						
6	Erosion (typical of top of slope)	5708	5001	730352.60	1512581.31	Repair in accordance to the General Guidelines
7	Erosion/rill (15' L X 1' D)	5709	5002	729203.14	1514021.32	Previously Identified (FY11 - 2nd Quarter PT# 1013) Repair in accordance to the General Guidelines
8	Erosion under Drain #18 (3' dia)	5710	5003	728035.03	1513450.84	Repair in accordance to the General Guidelines
9	Erosion under Drain #11 (10' L rill)	5711	5004	728049.57	1512059.18	Repair in accordance to the General Guidelines
10	Erosion under Drain #12 (10' L)	5712	5005	728101.93	1511815.72	Repair in accordance to the General Guidelines

OCATION: Cumberland Fossil Plant - 2011 Annual Dike Inspection
VEATHER: 93 degrees F, Sunny
NSPECTION BY: Daniel Rogers, Jim Swindler, Jacob Horton
)ATE: 6/20/2011

ITEM NO.	DESCRIPTION	PICTURE NO.	POINT NO.	NORTHING	EASTING	Recommendations
Annual Inspection - Coal Yard Drainage Basin						
11	Erosion at Drain to the CYDB	2	100	730787.815	1512408.175	Repair in accordance to the General Guidelines
12	DEPRESSION in Dike Face	3	101	730434.356	1512635.655	Monitor for evidence of erosion or slope movement. If observed, notify Engineering.
13	SMALL SLIP	4	102	730406.812	1512736.619	Repair by re-grading and establishing vegetation
14	Animal Burrows	6	103	730460.571	1512992.755	Repair in accordance to the General Guidelines
15	Animal Burrow		104	730297.324	1513341.827	Repair in accordance to the General Guidelines
Annual Inspection - Gypsum Stack Complex						
16	Rutting	13	105	727948.786	1513474.615	Repair in accordance to the General Guidelines
17	Rutting	14	106	728131.271	1512547.519	Repair in accordance to the General Guidelines
18	PONDING		107	728256.175	1511163.533	Repair in accordance to the General Guidelines
19	POSSIBLE SEEP		108	730482.167	1512429.948	Monitor to determine if seep. If so notify Engineering and add to Seepage Log
20	PONDING Below Drain #21	9	109	729567.314	1513763.112	Repair in accordance to the General Guidelines
Annual Inspection - Dry Fly Ash Stack						
21	POSSIBLE SEEP	4	110	729943.216	1510036.437	Monitor to determine if seep. If so notify Engineering and add to Seepage Log
Annual Inspection - Ash Pond						
22	POSSIBLE SEEP	4	111	733384.285	1510260.948	Monitor to determine if seep. If so notify Engineering and add to Seepage Log
23	POSSIBLE SEEP	4	112	733183.536	1510007.549	Monitor to determine if seep. If so notify Engineering and add to Seepage Log

STANTEC
CONSULTING
SERVICES INC.
11687 Lebanon Rd.
Cincinnati, Ohio
45241-2012
513-842-8200

Stantec

Plan View - CUF 2011 Annual Inspection

Tennesee Valley Authority
Cumberland Fossil Plant
Cumberland, Stewart County, Tennessee

PROJECT NO.	175531011
DATE	JUNE 2011
DRAWN BY	ANP
CHECKED BY	DBR
CHECKED BY	SAH
SCALE	AS SHOWN
REVISED	
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APPENDIX A

Document 11

TVA Monthly / Quarterly Safety Inspections, June 1, 2010

TVA Monthly /Quarterly/Special Facility Inspection Form — Form Date 6-01-10

1. Site Name: CUF 2. Facility Name: Gypsum Complex 3. Date and Start Time of Inspection: 9:01 Am CST

4. Operator Name: TVA RHO&M 5. Inspection Method: ☑ Walk ☐ Ride ☐ Both (KNOWN KEY DEFICIENCIES MUST BE INSPECTED)

6. Inspector's Name(s): Stuart Harris 7. Hazard Classification: ☑ High ☐ Significant ☐ Low

8. Inspection Frequency: ☐ MONTHLY ☑ QUARTERLY (MUST BE WALKED) ☐ SPECIAL (after significant rain or earthquake event)

9. Current weather conditions 22°F, SUNNY Prior Conditions, if notable

Check the appropriate box below. If not applicable, record "N/A". Provide comments when appropriate. Any other areas that should be brought to the attention of the Program Manager should also be noted in the "Comments" section. Indicate the locations of any areas identified, and photograph and attach to the form. Previous observation forms should be reviewed and any NEW observations or degradation of pervious conditions should be reported on this inspection form. (NOTE - ONE FORM PER FACILITY)

	Yes	No		Yes	No
10. Pre-Job Safety Briefing Performed	✓		**15. DIKE TOE AREAS**		
11. Activity / Construction on/ at facility	✓		A. Seepage ○ New ○ Existing	✓	
12. DIKE CREST			○ Clear/Cloudy/Red/Muddy		
A. Settlement / Cracking		✓	○Flow Increased / Decreased/Same	SAME	gpm
B. Rutting		✓	○ Aquatic Vegetation Growing	✓	
C. Lateral Displacement		✓	○ Ash or Clay Deposits Below Seep Outlet		✓
D. Erosion	✓		B. Boils ○ New ○ Existing		✓
13. INTERIOR / EXTERIOR DIKE SLOPES			○ Clear/Cloudy/Red/Muddy		
A. Minimum Freeboard	NA	ft.	○ Flow Increased / Decreased/Same		gpm
B. Current Freeboard	NA	ft.	○ Growing in Size		
C. Instabilities (Sloughs or Slides)		✓	C. Sinkholes/Depressions ○ New ○ Existing	NA	
D. Erosion	✓		**16. SEEPAGE COLLECTION SYSTEM**	NA	
E. Sinkholes/Depressions ○ New ○ Existing	✓		A. Estimated Flow Measurement		gpm
F. Vegetation / Brush / Trees			B. Increased Flow		
○ Heavy/Adequate/Sparse/Bare	IN PLACES		C. Emitting Clear or Dirty Water		
G. Animal Burrows ○ New ○ Existing	✓		**17. SPILLWAY WEIRS & OUTLETS**	NA	
H. Seepage ○ New ○ Existing		✓	A. Decant Riser Misaligned		
○ Clear/Cloudy/Red/Muddy			B. Decant Pipe Joints		
○ Flow Increased/Decreased/Same		gpm	○ Leaking		
○ Ash or Clay Deposits Below Seep Outlet			○ Separated		
I. Seep around Drain Pipe(s)		✓	C. Headwall In Good Condition		
○ Clear/Cloudy/Red/Muddy			**18. OPERATIONS & MAINTENANCE**		
14. DEFICIENCIES			A. Major Changes in Operations		✓
A. Prior Key Deficiencies Checked	✓				
B. New Deficiencies Identified / Flagged	✓				
C. Immediate Actions Taken (Note Below)		✓			
D. Photos of deficiencies attached	✓				

19. Major adverse changes in these items could cause instability and should be reported to the Program Manager as soon as possible for further evaluation. Adverse conditions noted in these items should normally be described (extent, location, etc.) in the space below and on the backside of this sheet if needed.

NOTE: Quarterly Inspection Deficiencies to be documented on spreadsheet with applicable latitude and longitude coordinates referenced. SHOW ALL QUARTERLY INSPECTION DEFICIENCIES ON AERIAL PHOTOS

20. Item #	Comments/New Observations/Action Taken:

21. PA(E) was Notified of New Key Deficiency: (Date / Time)

22. Who else Notified of New Key Deficiency: (Date / Time)

23. I hereby attest the above is original information (not reproduced) based on actual field observations made during the period indicated, by either myself or an appointed representative and are accurate, complete, and correct to the best of my knowledge.

Period Covered:

From: 9/10 To: 12/10 Signature: Stuart Harris Date: 12/15/10

TVA Monthly /Quarterly/Special Facility Inspection Form — Form Date 6-01-10

1. Site Name: CUF 2. Facility Name: Ash Complex 3. Date and Start Time of Inspection: 9:01 Am CST

4. Operator Name: TVA RHO&M 5. Inspection Method: ☑ Walk ☐ Ride ☐ Both (KNOWN KEY DEFICIENCIES MUST BE INSPECTED)

6. Inspector's Name(s): Stuart Harris 7. Hazard Classification: ☐ High ☐ Significant ☐ Low

8. Inspection Frequency: ☐ MONTHLY ☑ QUARTERLY (MUST BE WALKED) ☐ SPECIAL (after significant rain or earthquake event)

9. Current weather conditions 22°F Sunny Prior Conditions, if notable

Check the appropriate box below. If not applicable, record "N/A". Provide comments when appropriate. Any other areas that should be brought to the attention of the Program Manager should also be noted in the "Comments" section. Indicate the locations of any areas identified, and photograph and attach to the form. Previous observation forms should be reviewed and any NEW observations or degradation of pervious conditions should be reported on this inspection form. (NOTE - ONE FORM PER FACILITY)

	Yes	No		Yes	No
10. Pre-Job Safety Briefing Performed	✓		**15. DIKE TOE AREAS**		
11. Activity / Construction on/ at facility	✓		A. Seepage ○ New ○ Existing	✓	
12. DIKE CREST			○ Clear/Cloudy/Red/Muddy		
A. Settlement / Cracking		✓	○Flow Increased / Decreased/Same		gpm
B. Rutting		✓	○ Aquatic Vegetation Growing		✓
C. Lateral Displacement		✓	○ Ash or Clay Deposits Below Seep Outlet		✓
D. Erosion		✓	B. Boils ○ New ○ Existing		✓
13. INTERIOR / EXTERIOR DIKE SLOPES			○ Clear/Cloudy/Red/Muddy		
A. Minimum Freeboard	> 3'	ft.	○ Flow Increased / Decreased/Same		gpm
B. Current Freeboard	> 3'	ft.	○ Growing in Size		
C. Instabilities (Sloughs or Slides)		✓	C. Sinkholes/Depressions ○ New ○ Existing		✓
D. Erosion	✓		**16. SEEPAGE COLLECTION SYSTEM**	NA	
E. Sinkholes/Depressions ○ New ○ Existing		✓	A. Estimated Flow Measurement		gpm
F. Vegetation / Brush / Trees			B. Increased Flow		
○ Heavy/Adequate/Sparse/Bare			C. Emitting Clear or Dirty Water		
G. Animal Burrows ○ New ○ Existing	✓		**17. SPILLWAY WEIRS & OUTLETS**	NA	
H. Seepage ○ New ○ Existing	✓		A. Decant Riser Misaligned		
○ Clear/Cloudy/Red/Muddy			B. Decant Pipe Joints		
○ Flow Increased/Decreased/Same	SAME	BEST CAN TELL gpm	○ Leaking		
○ Ash or Clay Deposits Below Seep Outlet		✓	○ Separated		
I. Seep around Drain Pipe(s)		✓	C. Headwall In Good Condition		
○ Clear/Cloudy/Red/Muddy			**18. OPERATIONS & MAINTENANCE**		
14. DEFICIENCIES			A. Major Changes in Operations		✓
A. Prior Key Deficiencies Checked	✓				
B. New Deficiencies Identified / Flagged	✓				
C. Immediate Actions Taken (Note Below)		✓			
D. Photos of deficiencies attached	✓				

19. Major adverse changes in these items could cause instability and should be reported to the Program Manager as soon as possible for further evaluation. Adverse conditions noted in these items should normally be described (extent, location, etc.) in the space below and on the backside of this sheet if needed.

NOTE: Quarterly Inspection Deficiencies to be documented on spreadsheet with applicable latitude and longitude coordinates referenced. SHOW ALL QUARTERLY INSPECTION DEFICIENCIES ON AERIAL PHOTOS

20. Item #	Comments/New Observations/Action Taken:

21. PA(E) was Notified of New Key Deficiency: (Date / Time)

22. Who else Notified of New Key Deficiency: (Date / Time)

23. I hereby attest the above is original information (not reproduced) based on actual field observations made during the period indicated, by either myself or an appointed representative and are accurate, complete, and correct to the best of my knowledge.

Period Covered:

From: 9/10 To: 12/10 Signature: Stuart Harris Date: 12/15/10

LOCATION: Cumberland Fossil Plant - 4th Quarter FY2010 Dike Inspection
WEATHER: 22 degrees F, Sunny
INSPECTION BY: Stuart Harris,Jason Hill, Jacob Horton, Jake Booth, Mike Hulslander, Robert Bagwell, Bronson Reed, Danny Stephens, Dan Rogers (Stantec), Carrie McCarty, Jessica Tnin
DATE: 12/09/2010

ITEM NO.	DESCRIPTION	PICTURE NO.	POINT NO.	NORTHING	EASTING	COMMENTS
1	Wave wash erosion	662	1000	733273.00	1511317.62	Repair in accordance to the General Guidelines
2	Animal burrow	663	1001	733586.42	1510796.96	Repair in accordance to the General Guidelines
3	Animal burrow	-	1002	733571.09	1510736.47	Repair in accordance to the General Guidelines
4	Red water seep	664	1003	730964.45	1509203.98	Previously Identified. Continue to monitor seep. Appears unchanged.
5	Red water seep (end)	664	1004	731060.66	1509206.96	Previously Identified. Continue to monitor seep. Appears unchanged.
6	Red water seep (beginning)	665	1005	731151.04	1509213.77	Previously Identified. Continue to monitor seep. Appears unchanged.
7	Red water seep near old bridge	665	1006	730901.20	1509200.97	Previously Identified. Continue to monitor seep. Appears unchanged.
8	Red water seep near old bridge	666	1007	730861.69	1509216.60	Previously Identified. Continue to monitor seep. Appears unchanged.
9	Erosion/rill (40' Long x 2'wide x 1'deep)	-	2000	731960.59	1511371.80	Repair in accordance to the General Guidelines
10	Erosion/rill (20' Long x 4'wide x 3'deep)	661	2005	729908.40	1510214.83	Repair in accordance to the General Guidelines
11	No vegetation	1679	3000	731436.09	1511616.37	Reseed in accordance with the TVA T-1 Section 580 Grassing Specifications
12	Animal burrow (2' deep)	1681	3002	730185.23	1513039.87	Repair in accordance to the General Guidelines
13	Animal burrow (1' deep)	1682	3003	728624.99	1513896.65	Repair in accordance to the General Guidelines
14	Metal pipe	1683	3004	728079.89	1513337.41	Field Supervisor to investigate and determine if it's just old trash and remove.
15	Seep	1684	3005	728185.72	1513057.30	Previously Identified. Continue to monitor seep. Appears unchanged.
16	Seep	1685	3006	728216.69	1512974.53	Previously Identified. Continue to monitor seep. Appears unchanged.
17	Seep	1686	3007	728257.94	1512751.79	Previously Identified. Continue to monitor seep. Appears unchanged.
18	Seep	1687	3008	728232.07	1512572.11	Previously Identified. Continue to monitor seep. Appears unchanged.
19	Seep	1687	3009	728189.12	1512471.07	Previously Identified. Continue to monitor seep. Appears unchanged.
20	Seep	1688	3010	728139.77	1512409.93	Previously Identified. Continue to monitor seep. Appears unchanged.
21	Seep	1689	3011	728033.75	1512155.44	Previously Identified. Continue to monitor seep. Appears unchanged.
22	Seep	1690	3012	728048.98	1512079.71	Previously Identified. Continue to monitor seep. Appears unchanged.
23	Seep	1691	3013	728067.05	1511972.07	Previously Identified. Continue to monitor seep. Appears unchanged.
24	Seep	1692	3014	728074.85	1511917.78	Previously Identified. Continue to monitor seep. Appears unchanged.
25	Seep	1693	3015	728566.28	1511049.66	Previously Identified. Continue to monitor seep. Appears unchanged.
26	Seep	1694	3016	728651.91	1511006.69	Previously Identified. Continue to monitor seep. Appears unchanged.
27	Hole (1' deep)	1695	3017	728680.91	1510979.88	Repair in accordance to the General Guidelines
28	24" Dia. Corrugated Metal Pipe (2)	1696-1697	3018	728845.75	1510896.82	**Engineering is investigating using CCTV to inspect - Previously Identified (4th quarter, PT# 2005)**
29	Erosion area by rock (6'x20')	1152	5000	731257.09	1511684.28	Repair in accordance to the General Guidelines
30	Skid Steer ruts from mowing (typical of area)	1153	5001	730302.71	1512703.52	Field Supervisor to talk with mowing crew to prevent/repair in the future.
31	Erosion/rill	1155	5003	730086.32	1512977.08	Repair in accordance to the General Guidelines
32	Erosion/rill	1156	5004	729972.34	1513160.58	Repair in accordance to the General Guidelines
33	Seep #1 (beginning)	1157	5007	728159.58	1512656.14	Previously Identified. Continue to monitor seep. Appears unchanged.
34	Seep #1 (end)	1158	5008	728073.17	1512470.31	Previously Identified. Continue to monitor seep. Appears unchanged.
35	Erosion gullies by outlet pipes	1159-1161	5009	731230.12	1511603.56	Repair in accordance to the General Guidelines and divert runoff to prevent recurrence of erosion.


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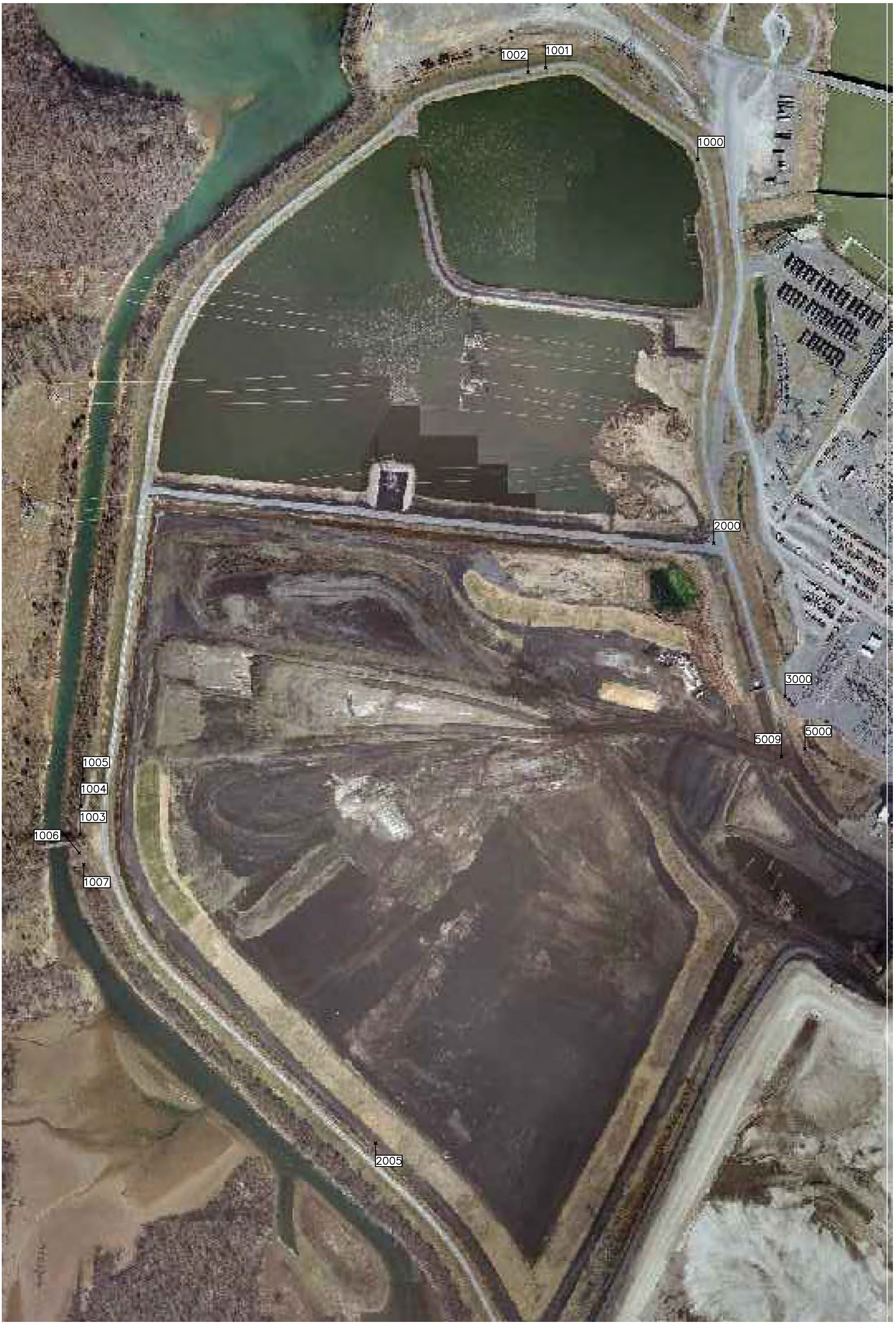
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APPENDIX A

Document 12

CCR Generation and Handling Questions

CCR Generation and Handling Questions:	Cumberland
1. Does the utility have drawings showing the CCR generation/handling/storage train for:	
a. Fly Ash	Yes
b. Bottom Ash	Yes
c. Boiler Slag	Yes
d. FGD wastes	Yes
2. What specific equipment is used to collect, handle, and store CCR material? For:	
a. Fly Ash	SCR Hoppers, Precipitator Hoppers, Surge Bins, piping, silos, ash pond
b. Bottom Ash	Economizer Hoppers, Hydroveyor, Air separator tank, bottom ash hoppers, jet pumps, piping, bottom ash reclaim pit
c. Boiler Slag	N/A
d. FGD wastes	Limestone preparation facilities, absorbers, recycle pumps, piping, FGD pond
3. Is there design information on the handling and transport equipment?	Yes
a. Example:size and length of pipe for sluicing the CCR	Yes
b. Is equipment within a secondary containment or just sitting on the ground?	Precip Hoppers, Economizer Hoppers and bottom ash hoppers are inside a building. Limestone preparation is done inside a building, the absorbers and recycle pumps are inside a building. Some piping is inside the building. The remainder is outside going to the ponds or wallboard plant.
c. Volume of storage silo	32000 Tons
4. What equipment is outside versus enclosed?	Precip Hoppers, Economizer Hoppers and bottom ash hoppers are inside a building. Limestone preparation is done inside a building, the absorbers and recycle pumps are inside a building. Some piping is inside the building. The remainder is outside going to the ponds or wallboard plant.
5. Has there ever been a release of CCR to the environment from the collection/handling/disposal system?	Yes, release of gypsum wastewater into Wells Creek
6. How much CCR per hour are they handling in each system, actual and design?	

APPENDIX A

Document 13

Dam Breach Analysis – Gypsum, Stantec, September 2010

Stantec

Dam Breach Analysis

Cumberland Gypsum Disposal Complex
Cumberland Fossil Plant
Stewart County, Tennessee

September, 2010

Stantec Consulting Services Inc.
11687 Lebanon Road
Cincinnati OH 45241-2012
Tel: (513) 842-8200
Fax: (513) 842-8250

Stantec

September 10, 2010
File: 175639026

Daniel G. Stephens, PE
Program Manager - CUF
Tennessee Valley Authority – CCP Engineering
1101 Market Street, LP5E-C
Chattanooga, Tennessee 37402

Reference: Cumberland Fossil Plant Gypsum Stack Breach Inundation Analysis

Dear Mr. Stephens:

Enclosed is our report of the breach inundation analysis for the Cumberland Fossil Plant Gypsum Stack. This report summarizes Stantec Consulting Services Inc.'s analysis, methodologies, modeling results and hazard classification recommendation. We appreciate the opportunity to assist Tennessee Valley Authority on this project. If you have any questions, please call our office.

Sincerely,

STANTEC CONSULTING SERVICES INC.

John Menninger, PE
Senior Project Engineer
Tel: (513) 842-8200
Fax: (513) 842-8250
john.menninger@stantec.com

Stan A. Harris, PE
Principal
Tel: (513) 842-8200
Fax: (513) 842-8250
stan.harris@stantec.com

/lfb

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Appendices

1.0 Study Description

The Cumberland Fossil Plant is located at the confluence of Wells Creek and the Cumberland River in Stewart County, Tennessee. The Gypsum Disposal Complex has a total footprint of approximately 153 acres with a proposed dike crest elevation of the wet operation cell at approximately 432 feet.

Stantec had previously performed breach analyses of the existing Gypsum Disposal Complex at the Cumberland Fossil Plant using approximate methods. The results of this study were included in the summary titled, "Preliminary Dam Breach Approximate Limits of Impact – Methodology" and submitted to the TVA on July 24, 2009 (Reference 1). Since that time, plans have been made to modify the facility and limit wet operations to a series of lined water quality cells. Construction of the lined cells is not expected to be completed until 2011. The current layout and operation of the gypsum facility, as of September 2010, limits wet operations to emergency events when the dewatering facility is nonoperational. Based on the current dike configuration and site grading at the gypsum facility, a storm event would cause water to pool at the west side of the facility with minor pooling (less than 3 feet) along the northeast dike. A failure of the existing gypsum facility would most likely occur along the west dike with impacts limited to property owned and operated by the TVA. Based on this review of the existing facility, no additional breach analyses were performed for existing conditions.

The remainder of this report summarizes Stantec 's review of impacts of the proposed gypsum disposal complex changes on the risk classification including the methodology utilized to model possible breach scenarios and calculate the impact to downstream areas. Specifically, the impact of these breach events on the adjacent Synthetic Materials (SynMat) Dewatering Facility and the Temple Inland Wall Board Plant was studied.

2.0 Breach Hydrograph Development

2.1 FAILURE SCENARIOS

Stantec performed breach analyses of the Gypsum Disposal Complex using the U.S. Army Corps of Engineers (USACE) HEC-HMS computer modeling software, Version 3.4. Breach analyses were performed for two failure scenarios: (1) A "Sunny Day" breach to the northeast of the Gypsum Disposal Complex which consists of a piping failure that is assumed to occur during normal operational conditions. (2) A PMP Event breach to the southeast of the Gypsum Stack which consists of a piping failure of an internal dike causing an overtopping failure of the external dike during a PMP event. Specific assumptions for the two scenarios are outlined below. It was decided that a PMP breach to the northeast was not a concern because a failure through the liner would have an extremely low likelihood of coinciding with the peak PMP event.

For a piping failure, HEC-HMS simulates a rectangular breach that begins at the bottom elevation of the breach and has a gradually increasing breach orifice height and width, until the dam crest is reached and then expands as a trapezoid. Likewise, for a PMP failure, HEC-HMS simulates a trapezoid failure that begins at the top of the embankment and has gradually increasing breach width and decreasing weir elevation until the bottom elevation is reached.

2.1.1 "Sunny Day" Northeast Breach Scenario

The proposed design for the Cumberland Gypsum Disposal Complex limits all wet operations to the water quality cell along the northern edge of the complex with a dike crest elevation of 432.0 feet. The water quality cell is divided into three long rectangular lined settlings ponds and one lined water quality pond. These individual ponds are divided by internal dikes within the water quality cell with crest elevations at 429.0 feet such that only one pond will fail during the "Sunny Day" scenario. A breach to the northeastern tip of the water quality cell draining Settling Pond 3 is the most likely scenario to cause damage to the SynMat Dewatering Facility.

Settling Pond 3 has a normal operating water surface elevation of 426.8 feet. Inflow to the water quality cell was neglected. A piping failure was assumed to occur along the northeastern edge of the water quality cell as shown in Figure A1 of Appendix A. The impounded water within the pond was assumed to be lost down to the bottom of the pond at elevation 418.0 feet. Figure A2 in Appendix A is a schematic cross section through the Gypsum Disposal Complex showing the "Sunny Day" failure configuration.

2.1.2 PMP Southeast Breach Scenario

The PMP Southeast Breach scenario involves the failure of two dikes in series. The first breach would occur on the southern edge of the water quality cell dike. The impounded water within the area breached was assumed to be lost down to the bottom of the pond at elevation 426.0 feet. The water would then discharge to South Cell A which has a dike crest elevation of 415.0

feet. When the water surface elevation in South Cell A reaches the dike crest elevation, the east dike was assumed to fail by overtopping. The impounded water is assumed to be lost down to an elevation of 411.1 feet. Figure A3 in Appendix A is a schematic cross section through the Gypsum Disposal Complex showing the PMP Southeast Breach failure configuration. The water surface elevation of the receiving waterway was assumed at the 100-year flood event, an elevation of 381.6 feet at the breach location.

Inflow to the pond consisted of the 6-hour PMP event precipitation (35.4 inches) obtained from the National Oceanic and Atmospheric Administration (NOAA) Hydrometeorological Report No.56 (HMR-56) (Reference 2). The SCS Type-B 6-hour hyetograph is a standard shape currently being used for spillway design at various TVA fossil plants (Reference 3).

2.2 ESTIMATION OF DAM BREACH PARAMETERS

Empirical equations have been developed from case studies to predict average breach width and breach development time based on the height of the dam, depth of the water, volume impounded, and/or type of breach. The equations developed by Von Thun and Gillette (Reference 4) were selected since they include factors to account for easily erodible material such as the material making up the Gypsum Disposal Complex. Stantec used these equations and based final breach parameters on the range of the estimates obtained and engineering judgment.

Estimates for breach development time and breach parameters for the “Sunny Day” Northeast Breach scenario are summarized below. The predicted average breach width (B_{av}) was 41.9 feet and breach development time (t_f) ranged from 0.06 hours to 0.16 hours with the average of 0.1 hours used. These estimates are based on the assumed failure conditions, height of the breach (14.0 feet), and impoundment water volume of 11.9 acre-feet in Settling Pond 3 above the breach elevation. Piping initiates at the elevation of 418.0 feet which is the bottom elevation of settling pond 3 and the top of the is the crest of the dike at 432.0 feet. A piping coefficient of value of 0.8 was used in modeling the breach.

Estimates for breach development time and breach parameters for the PMP Southeast Breach scenario are summarized below. This scenario includes the series of two breaches, first the breach of the water quality cell to the south into South Cell A, then the failure of South Cell A to the southeast into an Unnamed Stream which drains to Wells Creek. The predicted average breach width (B_{av}) of the first breach was 26.5 feet and breach development time (t_f) ranged from 0.03 hours to 0.12 hours with the average of 0.1 hours used. These estimates are based on the assumed failure conditions, height of the breach (6.0 feet), and impoundment water volume of 15.6 acre-feet in the Water Quality Cell above the breach elevation. Piping initiates at the elevation of 426.0 feet which is the bottom elevation of the water quality cell at the location of the breach and the top of the breach is the crest of the dike at 432.0 feet. A piping coefficient of value of 0.8 was used in modeling the breach. The predicted average breach width (B_{av}) of the second breach was 31.0 feet and breach development time (t_f) ranged from 0.02 hours to 0.14 hours with the average of 0.1 hours used. These estimates are based on the assumed

failure conditions, height of the breach (4.4 feet), and impoundment water volume of 44.3 acre-feet in South Cell A above the breach elevation. The impoundment water volume at the time of the breach of South Cell A includes the runoff volume into the impoundment (35.4 acre-feet) as well as the volume of water entering the impoundment through the breach in the Water Quality Cell. Overtopping initiates at the elevation of 415.5 feet which is the top of the dike, proceeding linearly down to the elevation of 411.1 feet.

The empirical calculations that served as the basis for the dam breach parameters estimation are included in Figures A4, A5, and A6 of Appendix A, for the "Sunny Day" Northeast Breach and the two PMP Southeast Breaches, respectively.

2.3 "SUNNY DAY" SCENARIO HYDROLOGIC MODELING

A dam breach outflow hydrograph for the "Sunny Day" Northeast Breach scenario was estimated using the dam break capabilities of HEC-HMS version 3.4. The data required for the model included (1) an elevation-storage relationship for Settling Pond 3 impoundment, (2) starting water surface elevation, and (3) dam breach parameters. Hydrologic inputs are described as follows:

(1) The stage-storage curve, shown in Figure A7 of Appendix A, was developed using the contours from plans for the proposed configuration of the Gypsum Disposal Complex.

(2) The starting water surface was set to the normal operating water surface elevation of 426.8 feet.

(3) The dam breach parameters described in Section 2.2 were applied to the model.

The computed outflow hydrograph for the "Sunny Day" Northeast Breach scenario had a peak outflow of 1,279 cfs which occurred 4 minutes after the start of the breach. The hydrograph is included as Figure A8 of Appendix A.

2.4 PRE-FAILURE HYDROLOGIC MODEL DEVELOPMENT FOR PMP EVENT

A hydrologic model of the proposed Gypsum Disposal Complex modifications was developed during the design of the facility and summarized in the Gypsum Disposal Complex Modifications Stormwater Management Report developed by Stantec (Reference 5). Inflow hydrographs and peak water surface elevations were obtained from the previous study and utilized in the development of the breach modeling.

The computed peak PMP water surface elevation was computed as 428.6 feet in the Water Quality Cell and as 414.6 feet in South Cell A. For the Southeast Breach scenario, the peak water surface elevations in the Water Quality Cell and South Cell A were assumed to occur simultaneously.

2.5 PMP SOUTHEAST BREACH SCENARIO HYDROLOGIC MODELING

A dam breach outflow hydrograph for the PMP Southeast Breach scenario was estimated using the dam break capabilities of HEC-HMS version 3.4. The data required for the model included (1) an elevation-storage relationship for the water quality cell impoundment draining to the South Cell A during the breach and South Cell A, (2) starting water surface elevations, and (3) dam breach parameters. Hydrologic inputs are described as follows:

(1) The stage-storage curves, shown in Figure A9 of Appendix A, were developed using the contours from plans for the proposed configuration of the Gypsum Disposal Complex .

(2) The starting water surface was set to the maximum PMP water surface elevation occurring in the water quality cell impoundment draining to the South Cell A during the breach of 428.6 feet. The maximum water surface elevation occurring in South Cell A during the PMP event is 414.6 feet before failure.

(3) The dam breach parameters described in Section 2.2 were applied to the model.

(4) The inflow hydrograph for the remaining PMP event after the water surface elevation in the water quality cell reaches an elevation of 428.6 feet was obtained from the pre-failure hydrologic model for a PMP event described in Section 2.4. The inflow hydrograph for the remaining PMP event after the water surface elevation in South Cell A reaches an elevation of 414.6 feet was obtained from the pre-failure hydrologic model for a PMP event described in Section 2.4. These remaining PMP event hydrographs were included as inflows during the breach.

The computed outflow hydrograph for the PMP Southeast Breach scenario had a peak outflow of 849 cfs which occurred 34 minutes after the start of the breach in the Water Quality Cell. The hydrograph is included as Figure A10 of Appendix A.

3.0 Hydraulic Analysis

For the "Sunny Day" Northeast Breach scenario, uniform flow calculations were performed to estimate the maximum water surface elevation at the SynMat Dewatering Facility. An unsteady flow hydraulic model was developed in USACE HEC-RAS Version 4.1 software to calculate maximum water surface elevations for the postulated PMP Southeast Breach scenario near the Wallboard Plant.

3.1 "SUNNY DAY" NORMAL DEPTH

The area to the northeast of the Gypsum Disposal Complex is flat with numerous buildings, ponds and ditches. The area is not suitable for the application of a 1-dimensional hydraulic model such as HEC-RAS. Instead, the estimated water surface elevations were calculated assuming a wide rectangular ditch and uniform flow.

To determine the width of flow to be used in the normal depth calculations, it was assumed that the width of flow would increase at the rate of 1 foot outward on each side of the flood wave for every 3 feet traveled in the direction of the wave. The flood wave was assumed to flow to the northeast perpendicular to the corner of the lined settling pond closest to the SynMat Dewatering Facility. Similar assumptions are routinely made when determining areas of effective flow in open channel flow calculations. The distance measured from the midpoint of the breach to the toe of the dike is approximately 220 feet and the distance measured from the toe of the dike to the SynMat Dewatering Facility is approximately 100 feet. At the assumed spreading rate, flow is assumed to be approximately 180 feet wide at the toe of the dike and 250 feet wide at the SynMat Dewatering Facility. After the plant, flow will gradually flow to the northwest via the system of plant ditches and ponds and overland flow. This flow was not considered in this analysis.

The downstream face of the Gypsum Disposal Complex is assumed to have a slope of 3H:1V and a Manning's coefficient of 0.03. Using Manning's Equation, the depth, velocity and momentum of flow at the toe of the dike were estimated for the peak discharge from the breach hydrograph.

$$(Average\ Velocity) = \frac{1.49}{(Manning\ Coefficient)} \times (Hydraulic\ Radius)^{2/3} \times (Slope)^{1/2}$$

At the toe of the dike, the flow is expected to undergo a hydraulic jump from supercritical to subcritical flow as a result of the abrupt change from a steep slope to a shallow slope. This jump results in an abrupt increase in the depth of flow corresponding to reduced velocity in which momentum is conserved.

$$\frac{(Flow\ Depth\ After\ Jump)}{(Flow\ Depth\ Before\ Jump)} = \frac{1}{2} \times (\sqrt{1 + 8 \times (Froude\ Number\ Before\ Jump)^2} - 1)$$

Finally, the resulting flow after the jump is assumed to continue spreading while conserving energy until it reaches the structure of interest. The resultant depth of flow was then added to the elevation of the ground to determine the water surface elevation for comparison to the elevation of the structure of interest. Figure B1 in Appendix B summarizes normal depth calculations for the “Sunny Day” Northeast Breach.

3.2 PMP SOUTHEAST BREACH HEC-RAS UNSTEADY HYDRAULIC MODELING

3.2.1 Model Geometry

The HEC-RAS model previously developed for the approximate study (Reference 1) was used as the basis for the updated depth calculations. The HEC-RAS model was developed using cross sections with an average spacing of less than 1,000 feet for Wells Creek and the Unnamed Tributary. Cross section overbank geometry was developed from 1-foot contour interval aerial mapping provided by TVA dated March 2010 (Reference 6) where available. In areas where aerial mapping was not available, cross section information was developed from USGS 10-Meter Digital Elevation Map data (Reference 7).

3.2.2 HEC-RAS Unsteady Hydraulic Modeling

The PMP Southeast Breach was assumed to occur during a 100-year flood of Wells Creek and the Cumberland River. Detailed flood information was not available for Wells Creek. The approximate 100-year peak discharge for Wells Creek of 13,600 cfs was obtained from “Tennessee Streamstats” (Reference 8) and applied as an inflow to Wells Creek in the HEC-RAS model. Because this approximate geometry does not contain channel information below the normal water surface elevation, flows for the Unnamed Stream draining into Wells Creek were set to 100 cfs for model stability purposes. The breach hydrograph was applied as a lateral inflow to the Unnamed Stream upstream to the southeast of the Gypsum Disposal Complex Figure A1 of Appendix A. The simulation had a 24-hour duration time and a computation interval of 30-seconds.

4.0 Results and Inundation Mapping

The primary areas of concern were the SynMat Dewatering Facility and the Temple Inland Wall Board Plant to the east of the Gypsum Disposal Complex. The peak water surface elevations at each of the structures are provided in Table 4. The impact elevation of the Wallboard Plant was based on 2-foot contours provided by TVA (Reference 6) while the impact elevation of the SynMat Dewatering Facility was provided by TVA based on a letter provided by the TVA to the President of the SynMat Dewatering Facility in 2006. The SynMat Dewatering Facility was identified within the potential impact zone of the "Sunny Day" Northeast Breach with inundation depths of approximately 1.3 feet. Neither structure was identified within the potential impact zone of the PMP Southeast Breach scenario. Figure B2 in Appendix B shows the approximate inundation limits of the "Sunny Day" Northeast Breach scenario.

Table 4. Dam Breach Modeling Impact Summary

Facility	Structure Elevation (feet)	Max. Post-Breach WSE (feet)	
		"Sunny Day" Northeast Breach	PMP Southeast Breach
SynMat Dewatering Facility	395.2	396.5	381.7
Wallboard Plant	386.0	n/a	381.7

5.0 Hazard Classification

The SynMat Dewatering Facility was identified within the dam breach impact zone where maximum computed water surface elevations exceed the defined impact elevation. However, the inundation depth is expected to reach a maximum of 1.3 feet, which, based on a review of dam safety literature regarding life loss estimation (Reference 10), would not likely present a probable threat to human life. Based on existing and proposed conditions for the operation of the Gypsum Disposal Complex, it is recommended that the hazard classification be lowered from High Hazard to Significant Hazard. If the proposed Gypsum Disposal Complex is constructed differently than shown on the proposed drawings (i.e. berm crest elevation raised) or development occurs within the impact zone, the hazard classification should be re-evaluated.

6.0 References

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4. Von Thun, Lawrence J. and D. R. Gillette (1990). *Guidance on Breach Parameters*, unpublished internal document, USBR, Denver, Colorado, 17. (Referenced in Wahl, 1998).

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8. USGS (2007). "StreamStats for Tennessee" USGS, <http://water.usgs.gov/osw/streamstats/tennessee.html> (Feb. 10, 2010).

9. Federal Emergency Management Agency (FEMA). (1991). *Flood Insurance Rate Map: Marshall County, Kentucky (Unincorporated Areas)*. FEMA, 210252 0025 B.

10. McClelland, Duane M. and David S. Bowles (2002). *Estimating Life Loss for Dam Safety Risk Assessment: A Review and New Approach*. Institute for Dam Safety Risk Management, Utah State University, Logan, Utah.

Appendix A

Breach Analysis Calculations

Geographic Information Systems

Cartographic Design By: Daniel Hoffman

FIGURE A.1

Assumed Breach Location
Cumberland Fossil Plant Dam Breach Analysis

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FIGURE A2 - "SUNNY DAY" NORTHEAST FAILURE CONFIGURATION SCHEMATIC

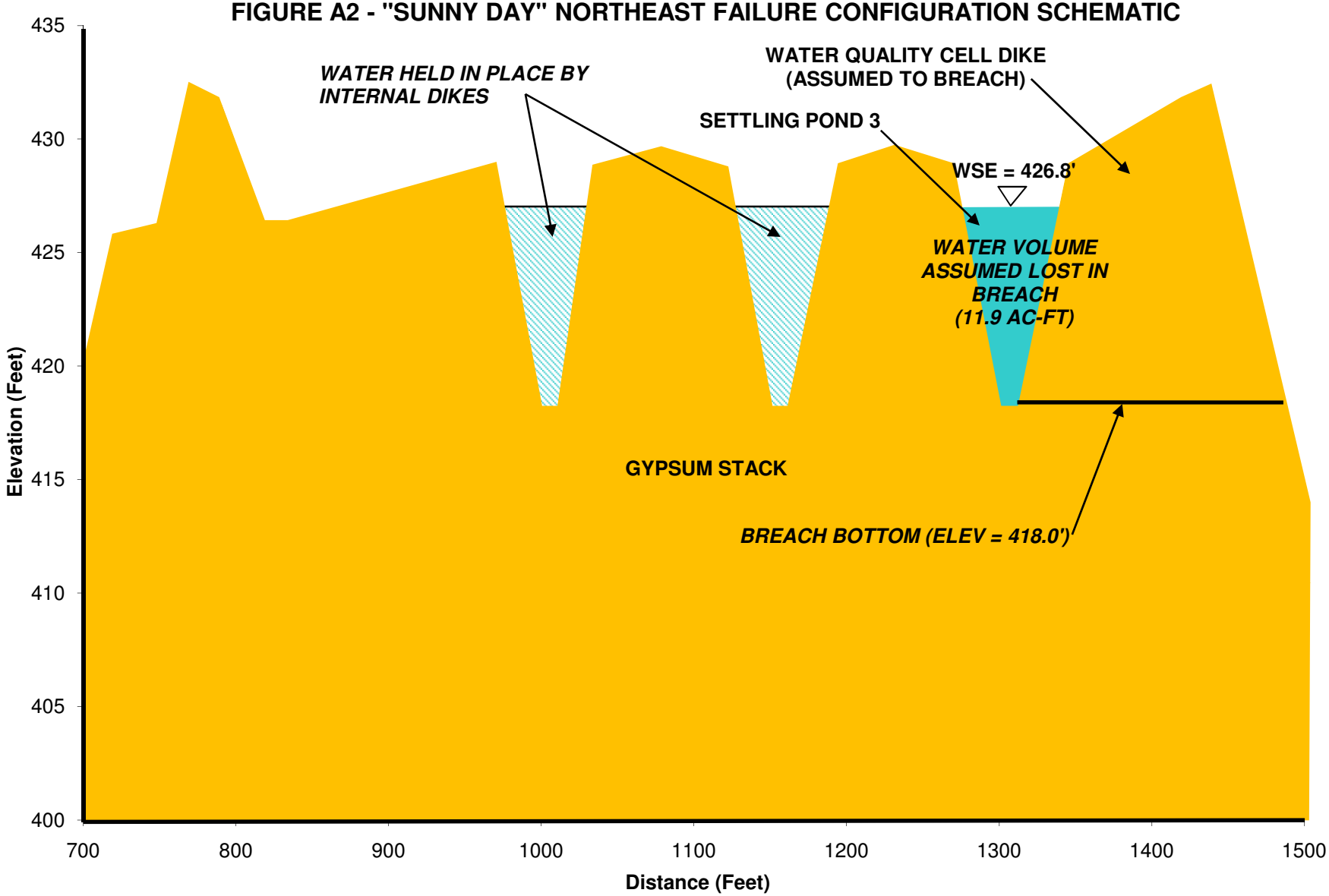


FIGURE A3 - PMP SOUTHEAST FAILURE CONFIGURATION SCHEMATIC

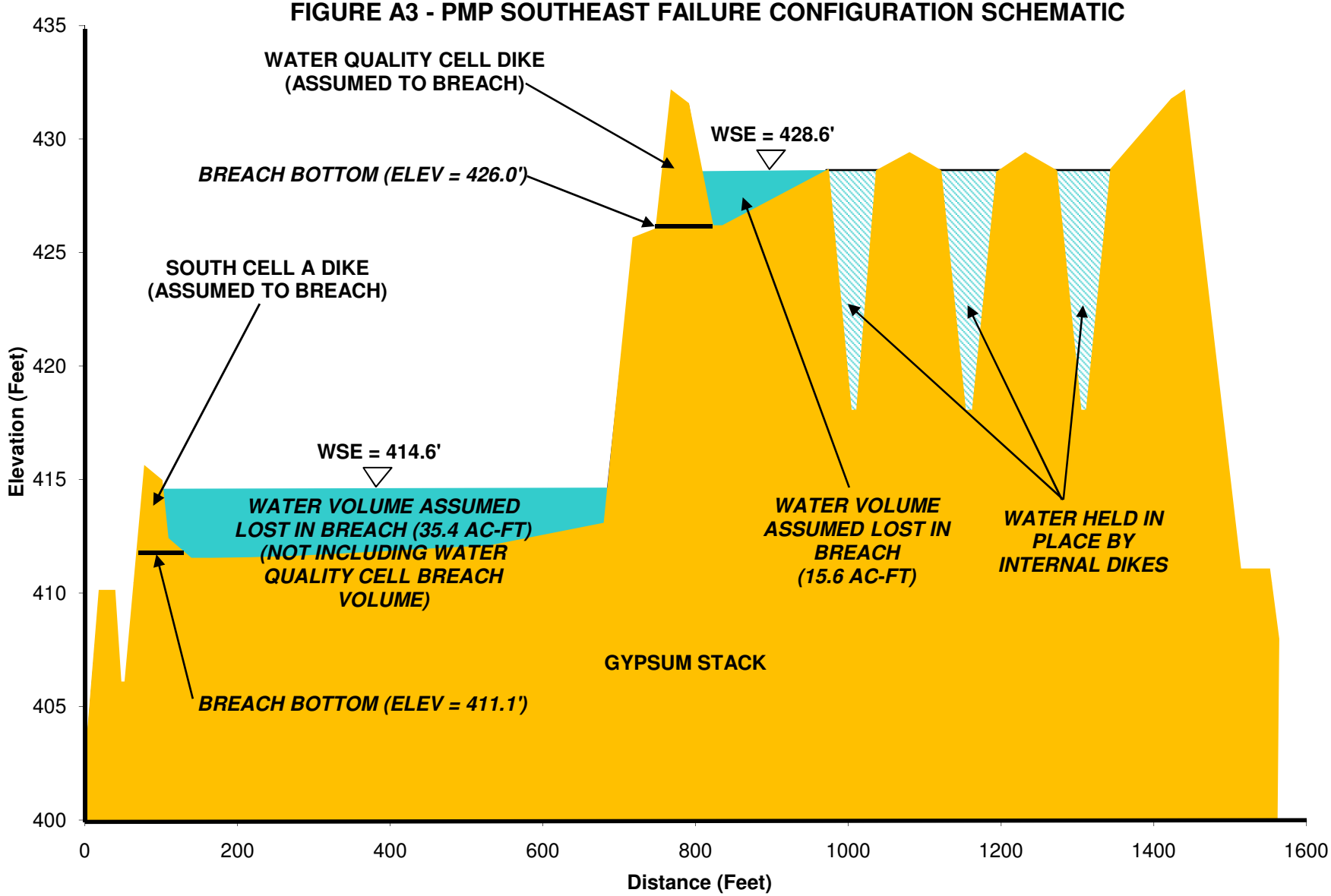


FIGURE A4 - "SUNNY DAY" NORTHEAST BREACH SCENARIO PARAMETERS

Dam Name: **Water Quality Cell - Settling Pond 3**

HEC-HMS Dam Breach Geometry and Development Time	
Breach Geometry	
Elevation of Water/Emergency Spillway Elevation (ft)	426.77
Top Elevation (ft)	432
Breach Bottom Elevation (ft)	418.0
Left Slope (xH:1V)	1
Right Slope (xH:1V)	1
Average Predicted Bottom Width (ft)	33.2
Average Predicted Width (ft)	41.9
Average Predicted Development Time (HR)	0.1

Breach Width and Time to Failure Parameters			
Overtopping correction factor - 1.0 for piping failure	K_0	1	
Height of dam (feet)	H_d	14.0	
Hydraulic depth of water at dam failure, above breach bottom (feet)	H_w	8.8	
Constant in Von Thun and Gillette breach width relation	C_b	20	
Volume of water above breach invert elevation at time of breach (acre-feet)	V_w	11.9	
	Breach Width	Time to Failure	
Froehlich (1987)(1)	21.7	0.176	$B_{av}=0.47*K_0*(V_w H_w)^{0.25}$ $t_f=0.59*V_w^{0.47}/H_d^{0.9}$
Froehlich (1995)(2)	15.4	0.111	$B_{av}=15*K_0*(*V_w)^{0.32}(H_w)^{0.19}$ $t_f=3.84*V_w^{0.53}/H_d^{0.9}$
USBR (1988)(3)	26.3	0.088	$B_{av}=3*H_w$ $t_f=B_{av}*0.011$
Von Thun and Gillette (1990)(4)	41.9		$B_{av}=2.5*H_w+C_b$
(based on t_f vs h_w)		0.064	*easily erodible;* $t_f=0.015*(H_w)$
(based on t_f vs h_w)		0.335	*erosion resistant;* $t_f=0.020*(H_d)+0.25$
(based on lateral erosion rates)		0.164	*easily erodible;* $t_f=B_{av}/\{4*(H_d/3.28)+61\}$
(based on lateral erosion rates)		0.749	*erosion resistant;* $t_f=B_{av}/\{4*(H_d/3.28)\}$

Chosen Values		
Bav=	41.9	feet
td=	0.1	hours
Bottom width=	33.13	feet
Width at WS=	50.67	feet
Width at Dam Top=	55.9	feet

FIGURE A5 - PMP SOUTHEAST BREACH SCENARIO PARAMETERS

Dam Name: **Water Quality Cell - Drainage Basin**

HEC-HMS Dam Breach Geometry and Development Time	
Breach Geometry	
Elevation of Water/Emergency Spillway Elevation (ft)	428.6
Top Elevation (ft)	432
Breach Bottom Elevation (ft)	426.0
Left Slope (xH:1V)	1
Right Slope (xH:1V)	1
Average Predicted Bottom Width (ft)	23.9
Average Predicted Width (ft)	26.5
Average Predicted Development Time (HR)	0.1

Breach Width and Time to Failure Parameters			
Overtopping correction factor - 1.0 for piping failure	K_0	1	
Height of dam (feet)	H_d	6.0	
Hydraulic depth of water at dam failure, above breach bottom (feet)	H_w	2.6	
Constant in Von Thun and Gillette breach width relation	C_b	20	
Volume of water above breach invert elevation at time of breach (acre-feet)	V_w	15.6	
	Breach Width	Time to Failure	
Froehlich (1987)(1)	17.1	0.428	$B_{av}=0.47*K_0*(V_w H_w)^{0.25}$ $t_f=0.59*V_w^{0.47}/H_d^{0.9}$
Froehlich (1995)(2)	13.3	0.275	$B_{av}=15*K_0*(*V_w)^{0.32}(H_w)^{0.19}$ $t_f=3.84*V_w^{0.53}/H_d^{0.9}$
USBR (1988)(3)	7.8	0.026	$B_{av}=3*H_w$ $t_f=B_{av}*0.011$
Von Thun and Gillette (1990)(4)	26.5		$B_{av}=2.5*H_w+C_b$
(based on t_f vs h_w)		0.027	*easily erodible;* $t_f=0.015*(H_w)$
(based on t_f vs h_w)		0.287	*erosion resistant;* $t_f=0.020*(H_d)+0.25$
(based on lateral erosion rates)		0.118	*easily erodible;* $t_f=B_{av}/\{4*(H_d/3.28)+61\}$
(based on lateral erosion rates)		1.104	*erosion resistant;* $t_f=B_{av}/\{4*(H_d/3.28)\}$

Chosen Values		
Bav=	26.5	feet
td=	0.1	hours
Bottom width=	23.9	feet
Width at WS=	29.1	feet
Width at Dam Top=	32.5	feet

FIGURE A6 - PMP SOUTHEAST BREACH SCENARIO PARAMETERS

Dam Name: **South Cell A**

HEC-HMS Dam Breach Geometry and Development Time	
Breach Geometry	
Elevation of Water/Emergency Spillway Elevation (ft)	415.5
Top Elevation (ft)	415.5
Breach Bottom Elevation (ft)	411.1
Left Slope (xH:1V)	1
Right Slope (xH:1V)	1
Average Predicted Bottom Width (ft)	26.6
Average Predicted Width (ft)	31.0
Average Predicted Development Time (HR)	0.1

Breach Width and Time to Failure Parameters			
Overtopping correction factor - 1.0 for piping failure	K_0	1.4	
Height of dam (feet)	H_d	4.4	
Hydraulic depth of water at dam failure, above breach bottom (feet)	H_w	4.4	
Constant in Von Thun and Gillette breach width relation	C_b	20	
Volume of water above breach invert elevation at time of breach (acre-feet)	V_w	44.3	
	Breach Width	Time to Failure	
Froehlich (1987)(1)	35.5	0.924	$B_{av}=0.47*K_0*(V_w H_w)^{0.25}$ $t_f=0.59*V_w^{0.47}/H_d^{0.9}$
Froehlich (1995)(2)	28.7	0.632	$B_{av}=15*K_0*(*V_w)^{0.32}(H_w)^{0.19}$ $t_f=3.84*V_w^{0.53}/H_d^{0.9}$
USBR (1988)(3)	13.2	0.044	$B_{av}=3*H_w$ $t_f=B_{av}*0.011$
Von Thun and Gillette (1990)(4)	31.0		$B_{av}=2.5*H_w+C_b$
(based on t_f vs h_w)		0.020	*easily erodible;* $t_f=0.015*(H_w)$
(based on t_f vs h_w)		0.277	*erosion resistant;* $t_f=0.020*(H_d)+0.25$
(based on lateral erosion rates)		0.142	*easily erodible;* $t_f=B_{av}/\{4*(H_d/3.28)+61\}$
(based on lateral erosion rates)		1.761	*erosion resistant;* $t_f=B_{av}/\{4*(H_d/3.28)\}$

Chosen Values		
Bav=	27.5	feet
td=	0.1	hours
Bottom width=	23.1	feet
Width at WS=	31.9	feet
Width at Dam Top=	31.9	feet

FIGURE A7 - SETTLING POND 3 STAGE STORAGE CURVE

Facility Name: Cumberland Gypsum Stack

Elevation (ft)	Cumulative Volume (acre-ft)
418	0.0
420	1.3
422	3.4
424	6.3
426	10.1
428	14.8
430	47.7
432	114.1

NOTE: This stage-storage curve was used for the modeling of the dam breach scenario in HEC-HMS for the "Sunny Day" Northeast scenario only.

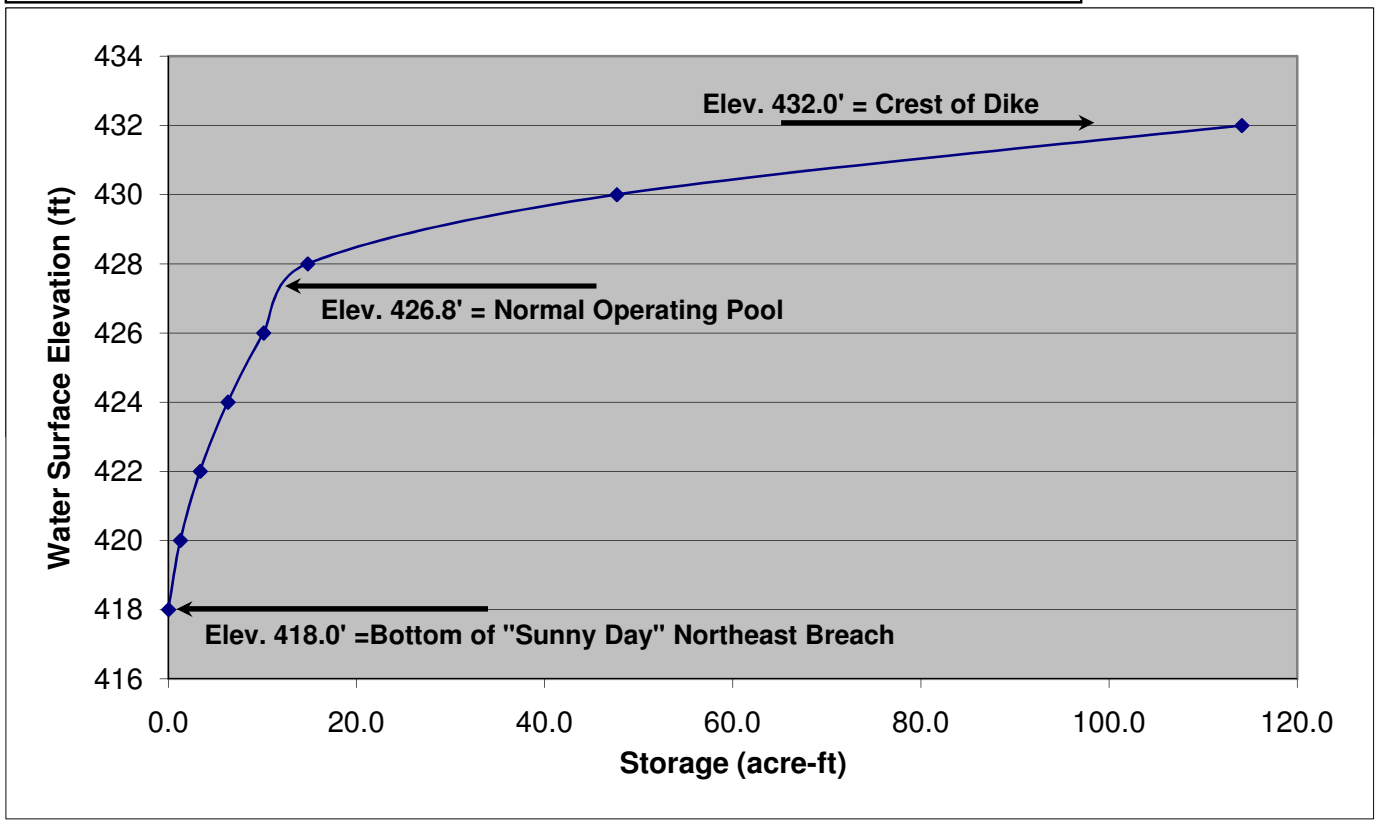


FIGURE A8 -"SUNNY DAY" NORTHEAST BREACH SCENARIO OUTFLOW HYDROGRAPH

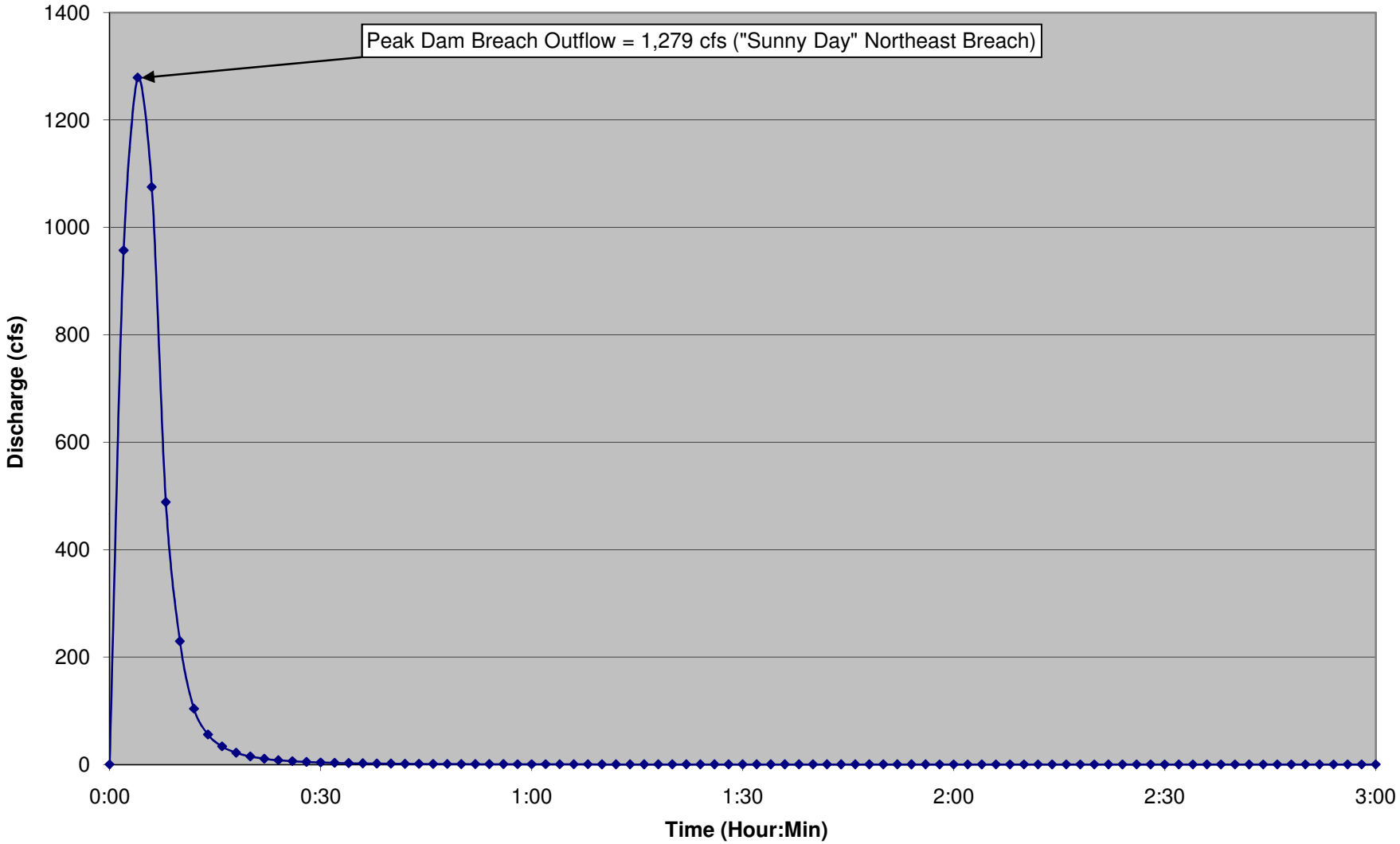


FIGURE A9 - WATER QUALITY CELL AND SOUTH CELL A STORAGE CURVES (FOR PMP SOUTHEAST BREACH HYDROLOGIC ROUTING)

Facility Name: Water Quality Cell and South Cell A

Water Quality Cell

Elevation (ft)	Cumulative Volume (acre-ft)
424	0.0
426	1.2
428	6.9
430	41.9
432	108.4

South Cell A

Elevation (ft)	Cumulative Volume (acre-ft)
410	0.0
410.5	1.1
411	2.2
411.5	5.0
412	7.8
412.5	12.0
413	16.3
414	28.4
415	40.5
415.5	47.1

NOTE: This stage-storage curve was used for the modeling of the dam breach scenario in HEC-HMS for the PMP Southeast Breach scenario only.

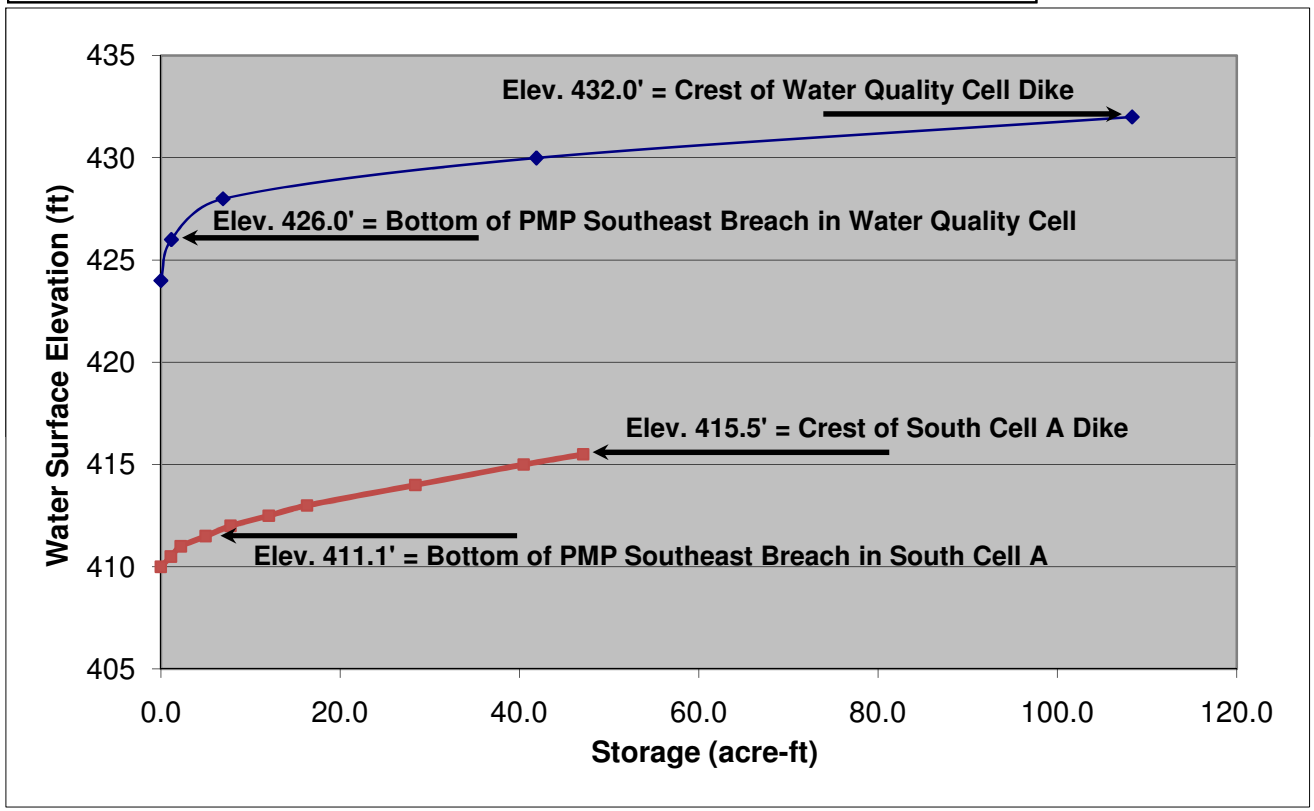
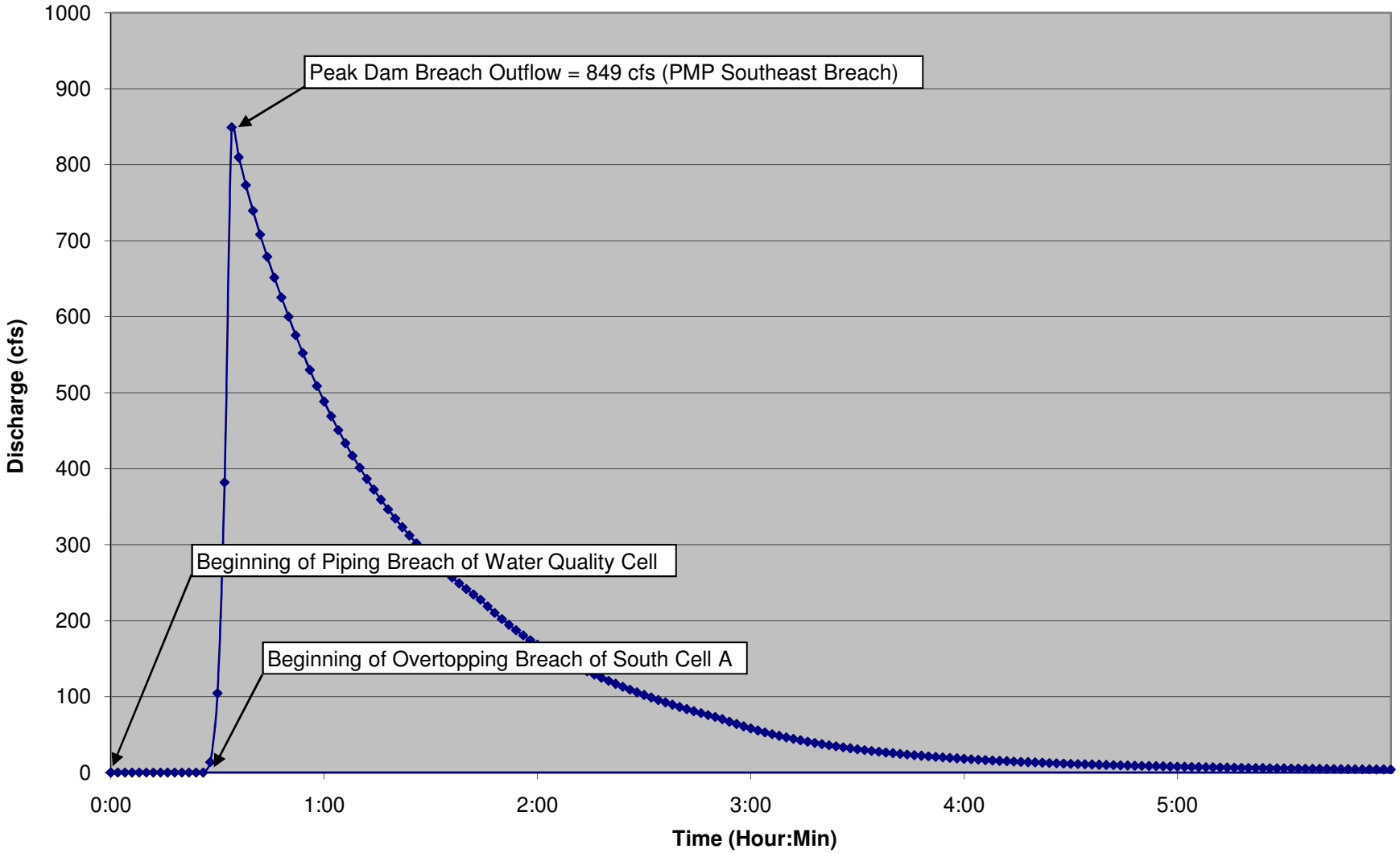


FIGURE A10 - PMP SOUTHEAST BREACH SCENARIO OUTFLOW HYDROGRAPH

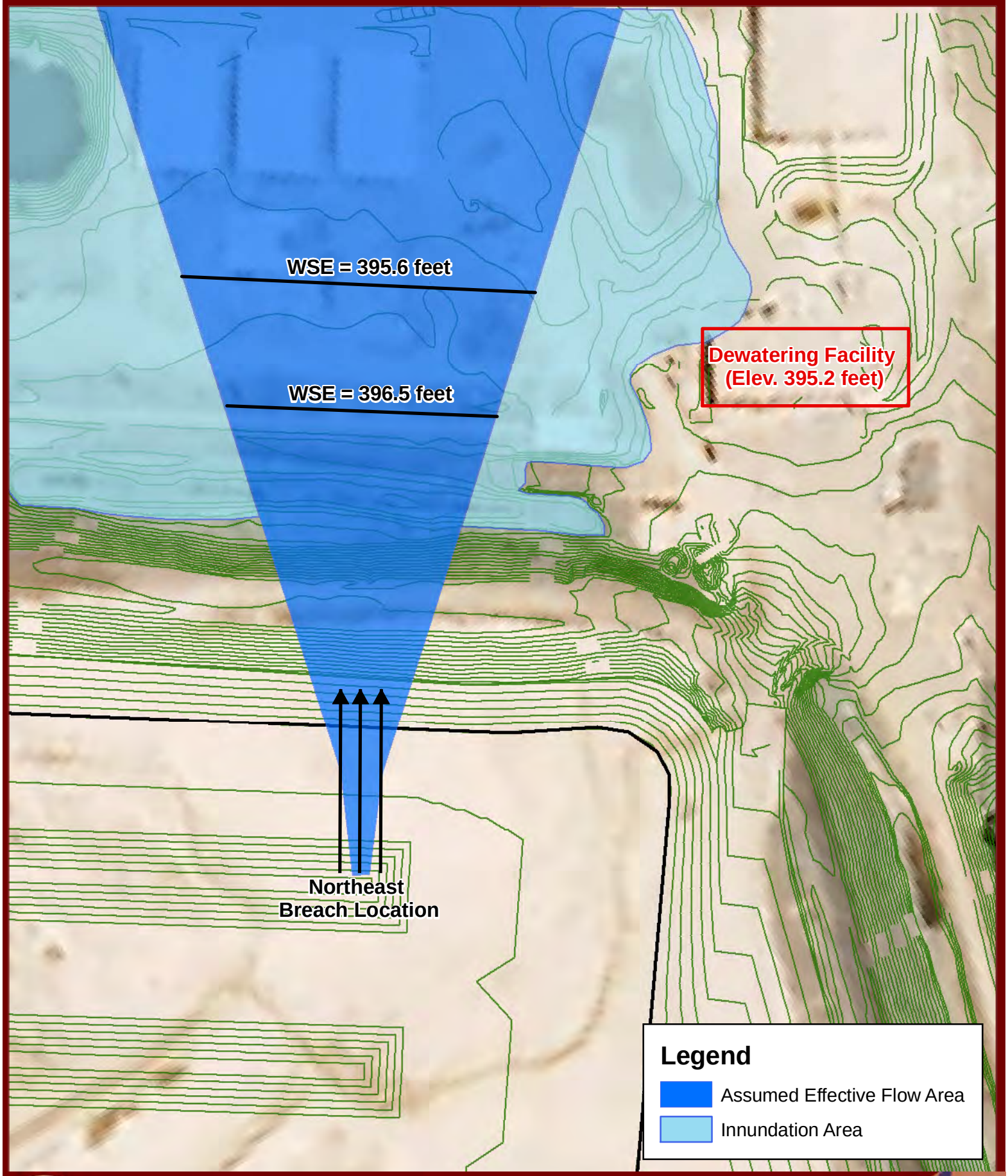


Appendix B

Breach Routing Calculations

FIGURE B1 - "SUNNY DAY" NORTHEAST BREACH HYDRAULIC ANALYSIS

Known Information		
Breach Bottom Width (ft)	33.2	
Breach Peak Flow Rate (cfs)	1279	
Mannings Roughness	0.03	
Outer Dike Approximate Slope (ft/ft)	0.33	
Width of Flow at Toe of Dike (ft)	180	
Width of Flow at Dewatering Plant (ft)	250	
Application of Manning's Equation		
Normal Depth Before Hydraulic Jump (ft)	0.4	Determined using solver
Velocity Before Hydraulic Jump (ft/s)	16.3	
Froude Number Before Hydraulic Jump	4.4	
Flow Properties After Hydraulic Jump		
Ratio of Depth After Jump to Depth Before Jump	5.7	
Normal Depth After Hydraulic Jump (ft)	2.47	
Velocity After Hydraulic Jump (ft/s)	2.9	
Specific Energy (ft)	2.6	
Flow Properties at Dewatering Plant		
Specific Energy (ft)	2.6	
Normal Depth After Hydraulic Jump (ft)	2.5	Determined using solver
Velocity After Hydraulic Jump (ft/s)	2.0	



Geographic Information Systems

Cartographic Design By: Daniel Hoffman

FIGURE B.2

"Sunny Day" Northeast Breach
Cumberland Fossil Plant Dam Breach Analysis

Stantec 11687 Lebanon Rd., Cincinnati, OH 45241-2012 Phone 513.842.8200 Fax 513.842.8250 www.stantec.com

APPENDIX A

Document 14

Dam Breach Analysis – Ash Pond, Stantec, September 2010

Stantec

Dam Breach Analysis and Inundation Mapping

Cumberland Fly Ash Pond
Cumberland Fossil Plant
Stewart County, Tennessee

September, 2010

Stantec Consulting Services Inc.
11687 Lebanon Road
Cincinnati OH 45241-2012
Tel: (513) 842-8200
Fax: (513) 842-8250

Stantec

September 2, 2010
File: 175639026

Daniel G. Stephens, PE
Program Manager - CUF
Tennessee Valley Authority – CCP Engineering
1101 Market Street, LP5E-C
Chattanooga, Tennessee 37402

Reference: Cumberland Fossil Plant Ash Pond Breach Inundation Analysis

Dear Mr. Stephens:

Enclosed is our report of the breach inundation analysis for the Cumberland Fossil Plant Ash Pond. This report summarizes Stantec Consulting Services Inc.'s analysis, methodologies, modeling results and hazard classification recommendation. We appreciate the opportunity to assist Tennessee Valley Authority on this project. If you have any questions, please call our office.

Sincerely,

STANTEC CONSULTING SERVICES INC.

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/lfb

Table of Contents

1.0 Study Description

The Cumberland Fossil Plant is located at the confluence of Wells Creek and the Cumberland River in Stewart County, Tennessee. The fly ash pond has a footprint of approximately 50 acres with a dike crest elevation of approximately 394 feet.

Stantec had previously performed breach analyses of the fly ash pond at the Cumberland Fossil Plant using approximate methods. The results of this study were included in the summary titled, “Preliminary Dam Breach Approximate Limits of Impact – Methodology” and submitted to the TVA on July 24, 2009 (Reference 1).

Stantec has been requested to perform a detailed analysis using recently developed topographic data to determine the limit of impact caused by a breach of the ash pond dike. The following report summarizes the additional study of the breach impacts using HEC-HMS, a hydrologic routing software, and HEC-RAS, hydraulic modeling software capable of performing unsteady flow routing.

2.0 Breach Hydrograph Development

2.1 FAILURE SCENARIOS

Stantec developed breach hydrographs for the ash pond using the U.S. Army Corps of Engineers (USACE) HEC-HMS computer modeling software, Version 3.4. Breach analyses were performed for two failure scenarios: (1) A "Sunny Day" breach which consists of a piping failure that is assumed to occur during normal operational inflows. The impoundment water surface elevation is normally assumed to be at the top of the lowest non-clogging spillway. (2) A Probable Maximum Precipitation (PMP) event which consists of an overtopping failure during a PMP event. Specific assumptions for the two scenarios are outlined below:

For a piping failure, HEC-HMS simulates a trapezoidal breach that begins at the bottom elevation of the breach and has a gradually increasing breach orifice height and width, until the dam crest is reached. Likewise, for a PMP failure, HEC-HMS simulates a trapezoid failure that begins at the top of the embankment and has gradually increasing breach width and decreasing weir elevation until the bottom elevation is reached.

2.1.1 "Sunny Day" Scenario

Since the Cumberland Ash Pond does not have an emergency spillway, the water surface elevation at the time of the breach was assumed to be equal to the perimeter dike crest elevation of 394.0 feet. Inflow to the ash pond was neglected and the water surface elevation of the Cumberland River assumed to be 359.0 feet which is the summer normal pool of Lake Barkley (Reference 2). The resulting water surface elevation at likely breach locations along Wells Creek was estimated at 359.3 feet. Piping failures were assumed to occur along Wells Creek or the Cumberland Fossil Plant Discharge Channel as shown in Figure A1 of Appendix A. The impounded water and fly ash within the pond was assumed to be lost down to elevation 359.3 feet since the surrounding water would act as tailwater and limit outflow. Conservatively, all sluiced ash above elevation 359.3 feet was assumed to mobilize and be lost through the breach. Figure A2 in Appendix A is a schematic cross section through the ash pond showing the "Sunny Day" failure configuration.

2.1.2 PMP Scenario

The water surface in the ash pond at the beginning of the PMP event was assumed at normal pool elevation, 384.3 feet. Overtopping failures were to occur along Wells Creek or the Cumberland Fossil Plant Discharge Channel as shown in Figure A1 of Appendix A. The water surface elevation on Wells Creek and the Cumberland River was assumed at the level of the 100-year flood event, an elevation of 381.0 feet at the breach location. This assumption is reasonable since some level of flooding of the surrounding waterways would be expected during a PMP event of the ash pond but the water surface elevations of the surrounding waterways would be expected to be less than the PMP elevations since Wells Creek and the Cumberland

River have much larger drainage areas and lag times. The overtopping failure was assumed to begin when the ash pond water surface reached the crest of the dike, elevation 394.0 feet. Figure A3 in Appendix A is a schematic cross section through the ash pond showing the PMP failure configuration.

The inflow consisted of the 6-hour PMP event precipitation (35.4 inches) obtained from the National Oceanic and Atmospheric Administration (NOAA) Hydrometeorological Report No.56 (HMR-56) (Reference 3). Three different hyetograph shapes were evaluated: (1) SCS Type-B 6-hour hyetograph, (2) “Early Peak” 6-hour hyetograph, and (3) “Late Peak” 6-hour hyetograph. The SCS Type-B 6-hour hyetograph is a standard shape currently being used for spillway design at various TVA fossil plants (Reference 4). The "Early Peak" and "Late Peak" hyetographs were developed using a procedure outlined in HMR-56. The 1-, 2-, 3-, 4-, 5-, and 6-hr PMP depths were taken from Figure 16 in HMR-56 and arranged sequentially. Incremental depths were determined for each hour and then rearranged to develop the two hyetographs according to rules presented in HMR-56.

2.2 ESTIMATION OF DAM BREACH PARAMETERS

Many empirical equations have been developed from case studies to predict average breach width and breach development time based on the height of the dam, depth of the water, volume impounded, and type of breach. Since there is great uncertainty in predicting dam breach parameters, Stantec used different empirical equations and based final breach parameters on the range of the estimates obtained and engineering judgment.

Estimates for breach development time and average breach parameters for the “Sunny Day” scenario are summarized in Table 1. The predicted average breach width (B_{av}) ranged from 85.9 feet to 146.8 feet and breach development time (t_f) ranged from 0.2 hours to 1.1 hours. These estimates are based on the assumed failure conditions, height of the breach (35 feet), and impoundment water volume of 1141 acre-feet in the ash pond above the breach elevation. While the total volume of both water and sluiced ash above the breach elevation is 1762 acre-feet, for use in determination of the dam breach parameters, ash volume is excluded.

Table 1. Estimate of Dam Breach Parameters Based on “Sunny Day” Scenario

Equation Name	B_{av} (feet)	t_f (hours)
Froehlich (1987)[5]	95.8	0.7
Froehlich (1995)[6]	85.9	0.6
USBR (1988)[7]	104.1	0.3
Von Thun and Gillette (1990)[8]	146.8	0.2-1.1
Average	**108.1**	**0.5**

The selected parameters for the "Sunny Day" Scenario are summarized below:

(1) The average breach width along Wells Creek is 108.1 feet, which is the average of the breach widths from the equations referenced. For the breach along the Discharge Channel, an average breach width of 65.7 feet was selected because the lined spillway would restrict the bottom width of the breach to just 36.0 feet at this location.

(2) The breach development time is 0.5 hours.

(3) The piping initiates at the Wells Creek normal pool elevation of 359.3 feet at the location of the breach, which is also the bottom elevation of the breach, and progresses linearly. This elevation was used for both breach analyses as a conservative assumption.

(4) The top of breach is the dike crest elevation of 394.0 feet.

(5) The piping coefficient is 0.8, a common orifice coefficient value.

Estimates for breach development time and average breach parameters for the PMP event scenario are summarized in Table 2. For the overtopping failure of the PMP event scenario, only the Froehlich equations were utilized because of their incorporation of a correction factor specifically for overtopping failures. The predicted average breach width (B_{av}) ranged from 81.2 feet to 89.3 feet and breach development time (t_f) ranged from 0.1 hours to 1.2 hours. These estimates are based on the assumed failure conditions, height of the breach (13.0 feet), and impoundment volume of water of 598 acre-feet in the ash pond above the breach elevation. While the total volume of both water and sluiced ash above the breach elevation is 631 acre-feet, for use in determination of the dam breach parameters, only the water volume is used.

Table 2. Estimate of Dam Breach Parameters Based on PMP Scenario

Equation Name	B_{av} (feet)	t_f (hours)
Froehlich (1987)[5]	89.3	1.2
Froehlich (1995)[6]	81.2	0.9
USBR (1988)[7]	Not Considered	0.1
Von Thun and Gillette (1990)[8]	Not Considered	0.1-1.0
Average	**85.2**	**0.6**

The selected parameters for the PMP Scenario are summarized below:

(1) The average breach width is 85.2 feet, which is the average of the average breach widths from the equations referenced.

(2) The breach development time is 0.6 hours.

(3) The bottom elevation of the breach is at the Wells Creek 100-year flood elevation of 381.0 feet.

(4) The overtopping failure initiates at the dike crest elevation, 394.0 feet and progresses linearly.

The empirical calculations that served as the basis for the dam breach parameters estimation are included in Figures A4 and A5 of Appendix A, for the "Sunny Day" and PMP Scenarios, respectively.

2.3 "SUNNY DAY" SCENARIO HYDROLOGIC MODELING

A dam breach outflow hydrograph for the "Sunny Day" scenario was estimated using the dam break capabilities of HEC-HMS Version 3.4. The data required for the model included (1) an elevation-storage relationship for the ash pond impoundment, (2) starting water surface elevation, and (3) dam breach parameters. Hydrologic inputs are described as follows:

(1) The stage storage curve, shown in Figure A6 of Appendix A, was developed using three sources provided by TVA:

a. Hydrographic survey dated September, 2008 (Reference 9)
b. Aerial survey dated April, 2009 (Reference 10)
c. Design drawings dated January,1969 (Reference11)

(2) The starting water surface was set to the dike crest elevation of 394.0 feet.

(3) The dam breach parameters described in Section 2.2 were applied to the model.

The computed outflow hydrograph for the "Sunny Day" scenario resulted in a peak outflow of 44,363 cfs which occurred 22 minutes after the start of the breach when the failure occurred along Wells Creek. The computed outflow hydrograph for the "Sunny Day" scenario resulted in a peak outflow of 22,522 cfs which occurred 24 minutes after the start of the breach when the failure occurred along the Discharge Channel. The hydrographs are included as Figure A7 of Appendix A.

2.4 PRE-FAILURE HYDROLOGIC MODEL DEVELOPMENT FOR PMP EVENT

The purpose of the pre-failure hydrologic model was to establish the time during the PMP event that the ash pond water surface would reach the top of embankment elevation and overtopping would begin to occur. Stantec used available data to develop a hydrologic model of the ash pond in HEC-HMS Version 3.4. Hydrologic information and outlet geometry information was taken from "Cumberland Fossil Plant – Design Support Calculations for Spillway Replacement Project" by Stantec dated 2010 (Reference 12). The data required for the model included (1) an elevation-storage relationship for the impoundment, (2) a starting water surface elevation, (3) an

outflow rating curve, (4) watershed parameters, and (5) an inflow hydrograph. Hydrologic inputs are described as follows:

(1) The stage-storage curve, shown in Figure A8 of Appendix A, was developed for the PMP pre-failure modeling from a hydrographic survey provided by TVA and dated September, 2008 (Reference 9).

(2) The starting water surface was set to the normal pool elevation of 384.3 feet. The normal operating pool elevation was selected because it would be unlikely for the pool elevation to be at the crest elevation at the start of the PMP event.

(3) The ash pond outlet consists of four circular riser structures. A rating curve for these structures was developed based on a construction detail drawing (Reference 13) assuming inlet control for the PMP breach scenario and is included in Figure A9 of Appendix A.

(4) Watershed parameters input to the model included:

a. Composite Curve Number = 89
b. Lag Time = 9.2 min
c. Watershed Area = 467 acres

(5) The inflow hydrograph was computed in HEC-HMS based on the watershed parameters and the 6-hour SCS Type-B PMP event, 6-hour “Early Peak” PMP event or 6-hour “Late Peak” PMP event. An additional inflow of 57 cfs, which is the maximum expected plant flow pumped to the pond, was also applied as a constant baseflow.

The model showed that overtopping would be expected to begin 2 hours and 40 minutes after the start of the 6-hour SCS Type-B PMP event, 1 hour and 20 minutes after the start of the 6-hour “Early Peak” PMP event and 5 hours and 26 minutes after the start of the 6-hour “Late Peak” PMP event. The computed PMP hydrographs are included in Figure A10 of Appendix A.

2.5 PMP SCENARIO HYDROLOGIC MODELING

A dam breach outflow hydrograph for the PMP scenario was calculated using the dam break capabilities of HEC-HMS Version 3.4. The simulation was run from the time of overtopping, until 24-hours after the start of the PMP. The data required for the model included (1) an elevation-storage relationship for the ash pond impoundment, (2) starting water surface elevation, (3) dam breach parameters, and (4) an inflow hydrograph for PMP event. These inputs are described below:

(1) The stage storage curve, shown in Figure A6 of Appendix A, was developed using three sources provided by TVA:

a. Hydrographic survey dated September, 2008 (Reference 9)
b. Aerial survey dated April, 2009 (Reference 10)
c. Design drawings dated January, 1969 (Reference 11)

(2) The starting water surface was set to the top of embankment elevation of 394 feet.

(3) The dam breach parameters described in Section 2.2 were applied to the model.

(4) The inflow hydrograph for the remaining PMP event after the water surface elevation in the fly ash pond reaches an elevation of 394.0 feet was obtained from the pre-failure hydrologic model for a PMP event described in Section 2.4. This remaining PMP event hydrograph was included as an inflow during the breach.

The peak outflow computed for the three PMP scenarios are summarized in Table 3. The hydrographs are included as Figure A11 of Appendix A.

Table 3. Summary of Peak Outflow for PMP Breach Scenarios

PMP Event Description	Time of Peak (Hour:Min After Start of PMP Event)	Peak Outflow (cfs)
6-Hour SCS Type-B Hyetograph	3:16	12,055
6-Hour “Early Peak” Hyetograph	1:56	10,521
6-Hour “Late Peak” Hyetograph	6:02	14,381

3.0 Hydraulic Model Development

An unsteady flow hydraulic model was developed in USACE HEC-RAS Version 4.1.0 software to calculate maximum water surface elevations for the postulated breach scenarios.

3.1 MODEL GEOMETRY

The HEC-RAS model was developed using cross sections with an average spacing of less than 1000 feet for Wells Creek and an average spacing of less than 5000 feet for the Cumberland River in the vicinity of the Cumberland Fossil Plant. Cross section overbank geometry was developed from 1-foot contour interval aerial mapping provided by TVA and dated March 2010 (Reference 14) where available. Wells Creek and the Cumberland River channel geometry for the underwater portion of the cross sections was developed from a hydrographic survey of Wells Creek and the Cumberland River Channel performed by TVA in January 2010 (Reference 15). Channel geometry for the Cumberland Fossil Plant Discharge Channel underwater portion of the cross sections and structures was developed from channel design drawings provided by TVA (Reference 16)

In areas where aerial mapping along the Cumberland River was not available, cross section information was developed from USGS 10-Meter Digital Elevation Map data (Reference 17).

The Cumberland City Road bridge over Wells Creek was added to the hydraulic model based on field survey performed by TVA in January 2010 (Reference 15) and design drawings provided by the Tennessee Department of Transportation (Reference 18). The Cumberland City Road bridge over the Discharge Channel was based on design drawings provided by TVA (Reference 16)

3.2 HEC-RAS UNSTEADY HYDRAULIC MODELING

The “Sunny Day” breach was assumed to occur during a non-flood condition. The approximate baseflow for Wells Creek of 14 cfs was obtained from “Tennessee Streamstats” (Reference 19) and applied as an inflow to Wells Creek in the HEC-RAS model. Baseflow in the Discharge Channel was set at approximately 59 cfs (Reference 12). Baseflow in the Cumberland River was approximated as 24,520 cfs based on average annual flow rates recorded at USGS Gage 03437000 near Dover, Tennessee from 1938 to 1965 (Reference 20). A downstream boundary condition was applied to the Cumberland River reach such that the initial water surface elevation was 359.0 feet, which is the Cumberland River summer normal pool elevation (Reference 2). The downstream boundary condition represents a backwater effect from Lake Barkley. The appropriate “Sunny Day” breach hydrograph was applied as a lateral inflow to Wells Creek upstream of the Cumberland City Road bridge and as an inflow to the Discharge Channel at the locations shown on Figure A1 of Appendix A during separate simulations. The simulations used a 24-hour duration time and a computation interval of 20-seconds.

The PMP breach was assumed to occur during a 100-year flood of Wells Creek and the Cumberland River. Detailed flood information was not available for Wells Creek. The approximate 100-year peak discharge for Wells Creek of 13,600 cfs was obtained from "Tennessee Streamstats" (Reference 19) and applied as an inflow to Wells Creek in the HEC-RAS model. Baseflow in the Discharge Channel was approximately 59 cfs because the majority of the contributing drainage area is regulated through the ash ponds to be breached (Reference 12).The approximate 100-year peak discharge of the Cumberland River, 300,000 cfs, was obtained from a USGS gage on the Cumberland River near Dover, Tennessee, approximately 15 miles downstream from the Cumberland Fossil Plant at river mile 88 (Reference 21). A downstream boundary condition was applied to the Cumberland River reach such that the 100-year water surface elevation was 380.1 feet based on data developed by the USACE from the Cumberland River gage at Cumberland Fossil Plant at river mile 104 (Reference 22). Each of the PMP breach hydrographs was applied as a lateral inflow to Wells Creek upstream of the Cumberland City Road bridge as shown in Figure A1 of Appendix A. The simulation used a 24-hour duration time and a computation interval of 20-seconds.

3.3 BRIDGE SCOUR ANALYSIS

During the "Sunny Day" breach simulation, flow velocities beneath the Cumberland City Road bridge over Wells Creek reach a peak of 13.2 feet per second. During the "PMP" breach simulation flow velocities beneath the bridge reach a peak of 9.7 feet per second. While these flows occur only briefly, their magnitude makes failure of the bridge by scour a concern.

To estimate the depth of potential scour in the vicinity of the bridge foundation, the hydraulic design functions of the HEC-RAS software were utilized. HEC-RAS performs scour analysis based on the methodology outlined in HEC-18 (Reference 23). Bridge scour calculations were performed for only the "Sunny Day" simulation due to the higher velocities. Figure B1 and B2 of Appendix B summarize the HEC-RAS bridge scour input parameters as well as support calculations.

According to construction drawings provided by TDOT, the existing bridge abutments, extending 40 feet upstream and downstream, are protected by a riprap blanket (Reference 18). At the time of this study, the presence of scour protection along the Wells Creek channel bottom was not confirmed. As such, the channel bottom was assumed to be free of rip rap and scour projection.

For purposes of the scour analysis, the riprap was assumed to have a D50 of 150.0 mm and D95 of 300.0 mm based on field observations. For the bed material within the channel, Stantec assumed the material properties of fine sediments. The entered properties were based on lab analysis of soil boring B-49 (Reference 24) located approximately 5000 feet upstream of the bridge and taken at the estimated depth of the original stream bed. This sample was selected to provide what is thought to be a conservative estimate of the channel properties in the absence of specific data.

4.0 Results and Inundation Mapping

The inundation limits for each scenario were mapped to determine which structures/roadways would be impacted. The primary areas of concern were the Cumberland City Road bridges at the mouth of Wells Creek and crossing the Discharge Channel. The bridge impact elevations, defined as the top of deck elevation, were determined based on field survey data. The peak water surface elevations at each of the bridges are provided in Table 4. Based on the base flood elevations provided by USACE and routing model results, the Cumberland City Road bridge over Wells Creek overtops during the 100-year event. The model results indicate that the PMP breach outflows result in an increase of water surface elevation of 0.4 feet. No additional structures or bridges were identified within the potential impact zone of the "Sunny Day" or PMP scenarios. For each scenario, the breach was applied upstream of the Cumberland City Road bridges. These locations correspond to the locations at which a breach produces the most severe rise in water surface elevation at each of the bridges.

The individual PMP events modeled produced slightly different breach hydrographs. The model indicated that the 6-hour "Late Peak" PMP event produces the greatest water surface elevations along both Wells Creek and the Cumberland River. The PMP event impact elevations are based on values produced by the 6-hour "Late Peak" PMP event.

Inundation mapping was developed for each of the breach scenarios and is included as Figures B3 and B4 of Appendix B for the "Sunny Day" and PMP scenarios, respectively. The inundation limits were delineated using the hydraulic model outputs and the imagery and topographic data described in Section 3.2.

Table 4. Dam Breach Modeling Impact Summary

Facility	Base Sunny Day WS (feet)	Base 100-Year WS (feet)	Impact Elevation (feet)	Max. Post-Breach WS (feet)	
				Sunny Day Breach	PMP Breach
Cumberland City Road Bridge at Mouth of Wells Creek	359.3	380.9	380.9	363.0	381.3
Cumberland City Road Bridge Over Discharge Channel	359.3	380.6	381.2	359.6	380.7

According to model results, the combination of pier and contraction scour, at the Cumberland City Road bridge over Wells Creek, could cause scour to a depth of 322.3 feet within the channel for the grain sizes taken from the boring sample. In the areas covered by the riprap blanket, no significant scour occurred. Table 5 summarizes the maximum scour depth within the channel based on the assumed sediment properties. According to design drawings provided by the Tennessee Department of Transportation (Reference 18) the top of the pile cap

within the channel is at an elevation of 326.6 feet and extends down to an elevation of 323.6 feet. The scour depth calculations assume sustained flow conditions and thus provide a conservative value for this simulation because peak flow velocities are sustained for only minutes. The maximum scour depth is below the base of the pile cap and could undermine the piers, potentially causing bridge failure.

Table 5. Dam Breach Bridge Scour Summary

	Grain Size
Channel D50 (mm)	0.009
Channel D95 (mm)	1.00
Starting Channel Elevation (ft)	344.6
Max Scour Depth (ft)	22.3
Max Scour Elevation (ft)	322.3
Impact Elevation (ft)	323.6
Difference Between Scour and Impact Elevations (ft)	-1.3

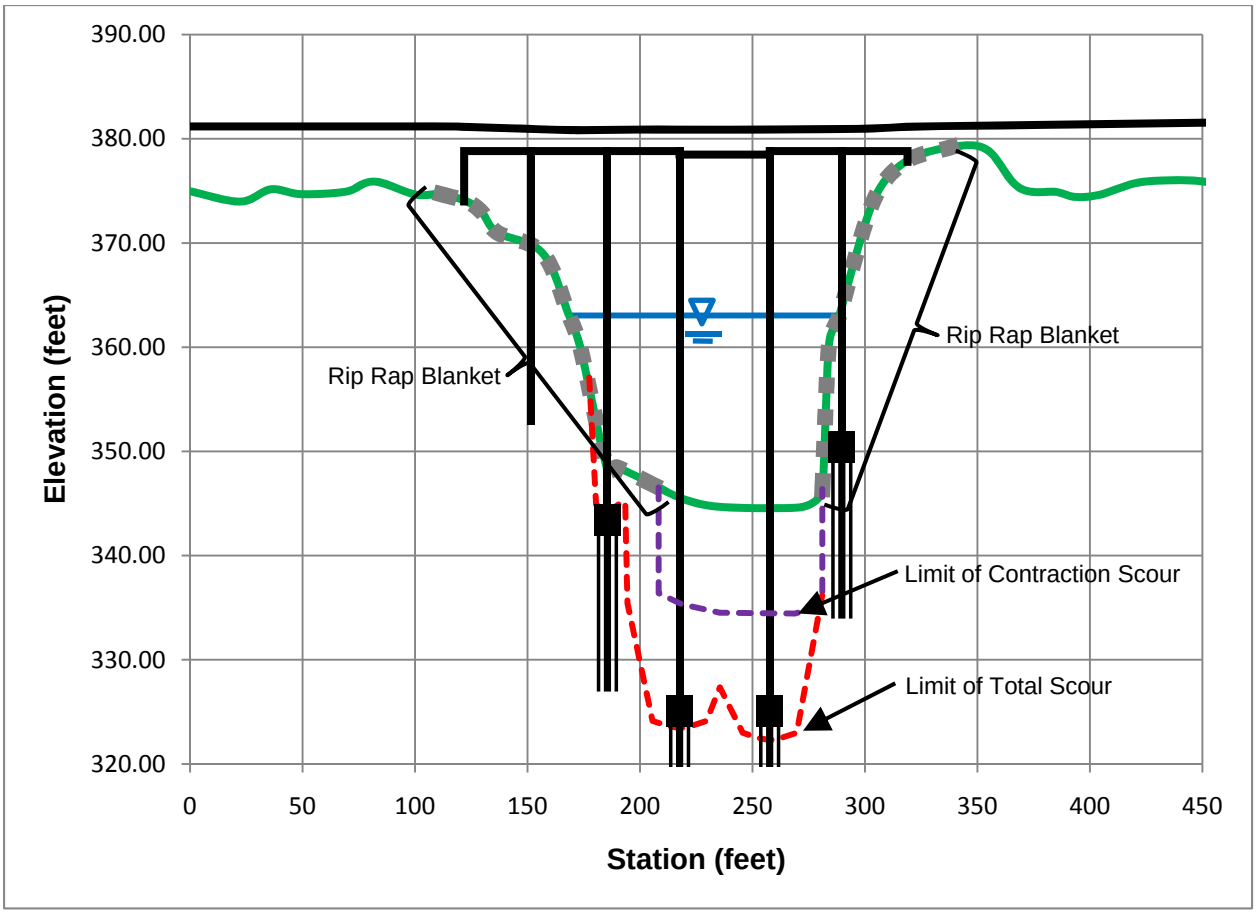

Figure 1. Bridge Scour Depth Results During "Sunny Day" Simulation

5.0 Hazard Classification

The Cumberland City Road bridge at the mouth of Wells Creek was identified within the dam breach impact zone where maximum computed water surface elevations exceed the defined impact elevation during the PMP event; however, the model indicates that the bridge overtops prior to the breach event and the subsequent rise in water surface elevations at the bridge during the PMP event is less than 0.5 feet. This small rise in water surface elevations caused by the breach event is unlikely to result in additional risk of loss of life at the bridge.

Additionally, the analysis indicates that scour is a potential risk to the Cumberland City Road bridge over Wells Creek during a dam breach event. It is recommended that the hazard classification remain at High Hazard until confirmation of existing scour protection or action is taken to protect the bridge. The confirmation of the existing presence or the placement of a riprap blanket through the bridge cross section would reduce the risk that scour poses to the Cumberland City Road bridge during a dam breach event and allow the hazard classification of the ash pond to be reduced from High Hazard to Significant Hazard.

If additional scour protection is required, the design and construction of the scour protection should be in accordance with requirements and specifications of the Tennessee Department of Transportation.

If the ash pond is modified (i.e. berm crest elevation raised) or development occurs within the impact zone, the hazard classification should be re-evaluated. Additionally, if the Cumberland City Road bridges across Wells Creek or the discharge channel are significantly modified, the hazard classification could be affected, since the maximum water surface elevations upstream of the bridge could increase.

6.0 References

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APPENDIX A

Document 15

Stantec Cumberland Piezometer Summary Report

Stantec

Cumberland Fossil Plant
815 Cumberland City Rd
Cumberland City,TN
175539009

	6/13/2009				7/16/2009				8/19/2009			
Location	Surface Elevation (ft)	Stickup (ft)	Depth Measurement (ft)	Water Elevation (ft)	Surface Elevation (ft)	Stickup (ft)	Depth Measurement (ft)	Water Elevation (ft)	Surface Elevation (ft)	Stickup (ft)	Depth Measurement (ft)	Water Elevation (ft)
B-3	0.0	0.0	0.0	0.0	394.8	2.5	13.3	384.0	394.8	0.0	11.1	383.6
B-4	0.0	3.0	7.8	-4.8	393.9	3.0	11.2	385.7	393.9	2.6	11.2	385.4
B-9	394.7	0.0	17.2	377.5	394.7	0.0	17.5	377.2	394.7	0.0	17.5	377.2
B-10	0.0	0.0	0.0	0.0	397.1	3.0	21.0	379.1	397.1	2.8	20.8	379.1
B-15A	395.0	0.0	7.8	387.2	395.0	0.0	8.8	386.3	395.0	0.0	8.4	386.7
B-16	397.8	2.3	39.1	361.0	397.8	2.3	39.0	361.2	397.8	2.3	40.2	359.9
B-21	395.1	0.0	4.6	390.5	395.1	0.0	4.9	390.3	395.1	0.0	4.6	390.6
B-22	410.2	3.8	19.9	394.1	410.2	3.8	24.1	389.9	410.2	2.8	23.4	389.6
B-27	0.0	0.0	0.0	0.0	422.2	0.0	0.0	422.2	422.2	2.3	27.5	397.0
B-28	410.6	0.8	28.2	383.2	410.6	0.8	30.5	380.9	410.6	2.5	32.2	380.9
B-29	395.2	0.0	20.0	375.2	395.2	0.0	20.7	374.5	395.2	0.0	19.9	375.3
B-35	425.7	2.2	29.6	398.2	425.7	2.2	33.9	393.9	425.7	2.6	45.4	382.9
B-36	411.2	0.0	25.1	386.1	411.2	0.0	25.7	385.4	411.2	2.4	27.8	385.7
B-37	395.2	1.8	17.1	380.0	395.2	0.0	20.1	375.1	395.2	0.0	18.1	377.1
B-42	0.0	0.0	0.0	0.0	396.2	0.0	18.0	378.3	396.2	0.0	16.7	379.5
B-43	0.0	0.0	0.0	0.0	411.3	0.8	19.8	392.4	411.3	2.2	21.2	392.3
B-44	0.0	0.0	0.0	0.0	419.5	2.0	27.5	394.0	419.5	1.5	27.5	393.5
B-45	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	411.6	2.5	21.1	393.0
B-46	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	420.3	3.2	23.3	400.2

Change in elevation
Significant Change

Stantec

Cumberland Fossil Plant
815 Cumberland City Rd
Cumberland City,TN
175539009

	9/15/2009				10/20/2009				11/5/2009			
Location	**Surface Elevation (ft)**	**Stickup (ft)**	**Depth Measurement (ft)**	**Water Elevation (ft)**	**Surface Elevation (ft)**	**Stickup (ft)**	**Depth Measurement (ft)**	**Water Elevation (ft)**	**Surface Elevation (ft)**	**Stickup (ft)**	**Depth Measurement (ft)**	**Water Elevation (ft)**
B-3	394.8	0.0	11.0	383.8	394.8	0.0	10.9	383.9	394.8	0.0	0.0	394.8
B-4	393.9	2.6	11.3	385.2	393.9	2.6	10.9	385.6	393.9	2.6	0.0	396.5
B-9	394.7	0.0	17.8	376.9	394.7	0.0	17.3	377.4	394.7	0.0	0.0	394.7
B-10	397.1	2.8	21.3	378.6	397.1	2.8	21.1	378.8	397.1	2.8	0.0	399.9
B-15A	395.0	0.0	8.7	386.3	395.0	0.0	8.4	386.7	395.0	0.0	0.0	395.0
B-16	397.8	2.3	42.1	358.1	397.8	2.3	40.6	359.5	397.8	2.3	37.5	362.7
B-21	395.1	0.0	0.0	395.1	395.1	0.0	0.0	395.1	395.1	0.0	4.4	390.7
B-22	410.2	2.8	24.0	389.0	410.2	2.8	23.5	389.5	410.2	2.8	0.0	413.0
B-27	422.2	2.3	27.7	396.7	422.2	2.3	27.5	397.0	422.2	2.3	0.0	424.5
B-28	410.6	2.5	32.4	380.7	410.6	2.5	31.8	381.3	410.6	2.5	0.0	413.1
B-29	395.2	0.0	20.1	375.1	395.2	0.0	19.5	375.7	395.2	0.0	0.0	395.2
B-35	425.7	2.6	35.6	392.6	425.7	2.6	35.5	392.8	425.7	2.6	0.0	428.3
B-36	411.2	2.4	27.8	385.7	411.2	2.4	27.6	386.0	411.2	2.4	0.0	413.5
B-37	395.2	0.0	18.1	377.1	395.2	0.0	17.7	377.5	395.2	0.0	0.0	395.2
B-42	396.2	0.0	0.0	396.2	396.2	0.0	0.0	396.2	396.2	0.0	17.1	379.1
B-43	411.3	2.2	21.6	391.9	411.3	2.2	21.2	392.3	411.3	2.2	0.0	413.5
B-44	419.5	1.5	28.0	393.0	419.5	1.5	27.7	393.2	419.5	1.5	0.0	421.0
B-45	411.6	2.5	21.3	392.8	411.6	2.5	21.3	392.8	411.6	2.5	0.0	414.1
B-46	420.3	3.2	24.3	399.2	420.3	3.2	23.4	400.1	420.3	3.2	0.0	423.5

Change in elevation
Significant Change

Stantec

Cumberland Fossil Plant
815 Cumberland City Rd
Cumberland City,TN
175539009

	11/17/2009				12/7/2009				1/18/2010			
Location	**Surface Elevation (ft)**	**Stickup (ft)**	**Depth Measurement (ft)**	**Water Elevation (ft)**	**Surface Elevation (ft)**	**Stickup (ft)**	**Depth Measurement (ft)**	**Water Elevation (ft)**	**Surface Elevation (ft)**	**Stickup (ft)**	**Depth Measurement(ft)**	**Water Elevation (ft)**
B-3	394.8	0.0	10.8	384.0	394.8	0.0	11.0	383.8	394.8	0.0	11.0	383.8
B-4	393.9	2.7	10.9	385.7	393.9	2.7	11.2	385.5	393.9	2.7	11.3	385.3
B-9	394.7	0.0	16.9	377.8	394.7	0.0	17.2	377.4	394.7	0.0	17.7	377.0
B-10	397.1	2.8	20.8	379.1	397.1	2.8	21.2	378.7	397.1	2.8	21.4	378.5
B-15A	395.0	0.0	8.4	386.6	395.0	0.0	8.9	386.1	395.0	0.0	9.3	385.8
B-16	397.8	2.3	41.4	358.6	397.8	2.3	41.8	358.2	397.8	2.3	41.8	358.3
B-21	395.1	0.0	4.6	390.5	395.1	0.0	5.2	389.9	395.1	0.0	5.7	389.5
B-22	410.2	2.9	23.6	389.5	410.2	2.9	24.1	389.0	410.2	2.9	24.4	388.7
B-27	422.2	3.0	27.5	397.7	422.2	3.0	27.9	397.3	422.2	3.0	28.1	397.0
B-28	410.6	2.7	31.8	381.5	410.6	2.7	32.3	381.0	410.6	2.7	32.5	380.7
B-29	395.2	0.0	19.2	376.0	395.2	0.0	19.6	375.6	395.2	0.0	19.7	375.4
B-35	425.7	2.5	35.2	392.9	425.7	2.5	35.5	392.6	425.7	2.5	36.1	392.1
B-36	411.2	2.4	27.5	386.0	411.2	2.4	27.8	385.8	411.2	2.4	27.8	385.8
B-37	395.2	0.0	17.6	377.6	395.2	0.0	18.0	377.3	395.2	0.0	17.9	377.3
B-42	396.2	0.0	17.4	378.8	396.2	0.0	17.4	378.8	396.2	0.0	18.1	378.1
B-43	411.3	2.3	21.3	392.2	411.3	2.3	21.9	391.6	411.3	2.3	22.5	391.1
B-44	419.5	1.6	27.8	393.3	419.5	1.6	28.5	392.5	419.5	1.6	29.1	392.0
B-45	411.6	2.6	21.3	392.9	411.6	2.6	21.3	392.9	411.6	2.6	21.1	393.0
B-46	420.3	3.3	23.4	400.1	420.3	3.3	23.4	400.1	420.3	3.3	23.3	400.2

Change in elevation
Significant Change

Stantec

Cumberland Fossil Plant
815 Cumberland City Rd
Cumberland City,TN
175539009

	2/11/2010				3/16/2010				4/22/2010			
Location	Surface Elevation (ft)	Stickup (ft)	Depth Measurement (ft)	Water Elevation (ft)	Surface Elevation (ft)	Stickup (ft)	Depth Measurement (ft)	Water Elevation (ft)	Surface Elevation (ft)	Stickup (ft)	Depth Measurement(ft)	Water Elevation (ft)
B-3	394.8	0.0	10.9	383.8	394.8	0.0	11.3	383.5	394.8	0.0	11.1	383.7
B-4	393.9	2.7	10.6	386.0	393.9	2.7	11.2	385.4	393.9	2.7	11.4	385.2
B-9	394.7	0.0	16.7	378.0	394.7	0.0	17.7	377.0	394.7	0.0	17.6	377.1
B-10	397.1	2.8	20.4	379.5	397.1	2.8	21.8	378.1	397.1	2.8	21.3	378.6
B-15A	395.0	0.0	8.4	386.6	395.0	0.0	9.6	385.4	395.0	0.0	9.3	385.7
B-16	397.8	2.3	36.5	363.6	397.8	2.3	42.6	357.5	397.8	2.3	39.7	360.4
B-21	395.1	0.0	5.3	389.8	395.1	0.0	5.9	389.2	395.1	0.0	6.0	389.2
B-22	410.2	2.9	24.0	389.1	410.2	2.9	24.8	388.3	410.2	2.9	24.8	388.3
B-27	422.2	3.0	27.8	397.3	422.2	3.0	28.1	397.1	422.2	3.0	28.2	397.0
B-28	410.6	2.7	31.4	381.8	410.6	2.7	32.7	380.6	410.6	2.7	32.6	380.7
B-29	395.2	0.0	18.7	376.5	395.2	0.0	19.9	375.3	395.2	0.0	19.6	375.6
B-35	425.7	2.5	36.2	392.0	425.7	2.5	36.5	391.6	425.7	2.5	36.7	391.4
B-36	411.2	2.4	27.8	385.8	411.2	2.4	28.2	385.4	411.2	2.4	28.4	385.2
B-37	395.2	0.0	17.8	377.5	395.2	0.0	18.3	376.9	395.2	0.0	18.7	376.5
B-42	396.2	0.0	17.4	378.8	396.2	0.0	18.2	378.0	396.2	0.0	17.7	378.5
B-43	411.3	2.3	22.2	391.3	411.3	2.3	22.8	390.7	411.3	2.3	22.9	390.6
B-44	419.5	1.6	28.8	392.3	419.5	1.6	29.6	391.5	419.5	1.6	29.6	391.5
B-45	411.6	2.6	21.0	393.1	411.6	2.6	21.0[a]	393.1	411.6	2.6	21.0[a]	393.1
B-46	420.3	3.3	21.3	402.3	420.3	3.3	21.3[a]	402.3	420.3	3.3	21.3[a]	402.3

Change in elevation [a] Dry
Significant Change

Stantec

Cumberland Fossil Plant
815 Cumberland City Rd
Cumberland City,TN
175539009

	5/18/2010				6/14/2010				7/14/2010			
Location	Surface Elevation (ft)	Stickup (ft)	Depth Measurement(ft)	Water Elevation (ft)	Surface Elevation (ft)	Stickup (ft)	Depth Measurement(ft)	Water Elevation (ft)	Surface Elevation (ft)	Stickup (ft)	Depth Measurement(ft)	Water Elevation (ft)
B-3	394.8	0.0	10.9	383.9	394.8	0.0	11.0	383.8	394.8	0.0	11.0	383.8
B-4	393.9	2.7	10.4	386.2	393.9	2.7	11.0	385.6	393.9	2.7	11.0	385.6
B-9	394.7	0.0	5.8	388.9	394.7	0.0	17.7	377.0	394.7	0.0	17.9	376.8
B-10	397.1	2.8	19.6	380.3	397.1	2.8	21.2	378.7	397.1	2.8	21.4	378.5
B-15A	395.0	0.0	8.3	386.7	395.0	0.0	8.4	386.6	395.0	0.0	8.8	386.3
B-16	397.8	2.3	35.8	364.3	397.8	2.3	38.5	361.6	397.8	2.3	39.0	361.0
B-21	395.1	0.0	5.0	390.2	395.1	0.0	5.0	390.2	395.1	0.0	5.2	389.9
B-22	410.2	2.9	23.7	389.4	410.2	2.9	23.8	389.3	410.2	2.9	24.1	389.0
B-27	422.2	3.0	27.5	397.7	422.2	3.0	27.3	397.9	422.2	3.0	27.6	397.6
B-28	410.6	2.7	31.1	382.2	410.6	2.7	32.4	380.9	410.6	2.7	32.2	381.0
B-29	395.2	0.0	19.2	376.0	395.2	0.0	19.9	375.2	395.2	0.0	19.8	375.3
B-35	425.7	2.5	35.2	393.0	425.7	2.5	34.6	393.5	425.7	2.5	35.0	393.2
B-36	411.2	2.4	21.2	392.4	411.2	2.4	27.1	386.5	411.2	2.4	27.4	386.2
B-37	395.2	0.0	17.6	377.6	395.2	0.0	17.8	377.4	395.2	0.0	18.1	377.1
B-42	396.2	0.0	17.6	378.6	396.2	0.0	19.0	377.2	396.2	0.0	16.7	379.5
B-43	411.3	2.3	21.3	392.3	411.3	2.3	21.2	392.4	411.3	2.3	21.9	391.6
B-44	419.5	1.6	27.7	393.4	419.5	1.6	27.7	393.4	419.5	1.6	28.4	392.7
B-45	411.6	2.6	21.0	393.1	411.6	2.6	21.0	393.1	411.6	2.6	21.0	393.1
B-46	420.3	3.3	23.3	400.3	420.3	3.3	23.3	400.3	420.3	3.3	23.3	400.3

Change in elevation
Significant Change

Stantec

Cumberland Fossil Plant
815 Cumberland City Rd
Cumberland City,TN
175539009

	8/10/2010				9/24/2010				10/27/2010			
Location	**Surface Elevation (ft)**	**Stickup (ft)**	**Depth Measurement(ft)**	**Water Elevation (ft)**	**Surface Elevation (ft)**	**Stickup (ft)**	**Depth Measurement(ft)**	**Water Elevation (ft)**	**Surface Elevation (ft)**	**Stickup (ft)**	**Depth Measurement(ft)**	**Water Elevation (ft)**
B-3	394.8	0.0	11.0	383.7	394.8	0.0	11.2	383.6	394.8	0.0	11.1	383.6
B-4	393.9	2.7	11.2	385.4	393.9	2.7	11.9	384.8	393.9	2.7	12.0	384.6
B-9	394.7	0.0	18.3	376.4	394.7	0.0	18.8	375.8	394.7	0.0	18.8	375.9
B-10	397.1	2.8	21.7	378.2	397.1	2.8	22.3	377.6	397.1	2.8	22.5	377.4
B-15A	395.0	0.0	9.1	385.9	395.0	0.0	9.6	385.4	395.0	0.0	9.7	385.3
B-16	397.8	2.3	40.3	359.7	397.8	2.3	42.4	357.6	397.8	2.3	42.8	357.3
B-21	395.1	0.0	5.4	389.8	395.1	0.0	5.8	389.4	395.1	0.0	5.6	389.5
B-22	410.2	2.9	24.2	388.9	410.2	2.9	24.6	388.5	410.2	2.9	24.6	388.5
B-27	422.2	3.0	27.4	397.8	422.2	3.0	28.1	397.0	422.2	3.0	28.4	396.8
B-28	410.6	2.7	32.6	380.7	410.6	2.7	33.2	380.1	410.6	2.7	33.6	379.7
B-29	395.2	0.0	19.9	375.2	395.2	0.0	20.5	374.7	395.2	0.0	20.8	374.4
B-35	425.7	2.5	35.2	393.0	425.7	2.5	36.1	392.1	425.7	2.5	36.5	391.6
B-36	411.2	2.4	27.6	385.9	411.2	2.4	28.2	385.4	411.2	2.4	28.3	385.3
B-37	395.2	0.0	18.4	376.8	395.2	0.0	18.5	376.7	395.2	0.0	18.8	376.4
B-42	396.2	0.0	17.2	379.0	396.2	0.0	17.3	378.9	396.2	0.0	17.6	378.6
B-43	411.3	2.3	22.3	391.3	411.3	2.3	22.8	390.8	411.3	2.3	23.1	390.4
B-44	419.5	1.6	29.2	391.9	419.5	1.6	29.4	391.7	419.5	1.6	29.9	391.1
B-45	411.6	2.6	21.0	393.1	411.6	2.6	21.3	392.9a	411.6	2.6	21.9	392.3[a]
B-46	420.3	3.3	23.3	400.3	420.3	3.3	23.3	400.3a	420.3	3.3	23.2	400.3[a]

Change in elevation
Significant Change

[a] Dry

Stantec

Cumberland Fossil Plant
815 Cumberland City Rd
Cumberland City,TN
175539009

	11/19/2010				12/7/2010				1/5/2011			
Location	**Surface Elevation (ft)**	**Stickup (ft)**	**Depth Measurement(ft)**	**Water Elevation (ft)**	**Surface Elevation (ft)**	**Stickup (ft)**	**Depth Measurement**	**Water Elevation (ft)**	**Surface Elevation (ft)**	**Stickup (ft)**	**Depth Measurement(ft)**	**Water Elevation (ft)**
B-3	394.8	0.0	11.2	383.6	394.8	0.0	11.1	383.7	394.8	0.0	11.0	383.8
B-4	393.9	2.7	12.1	384.6	393.9	2.7	11.6	385.0	393.9	2.7	11.6	385.1
B-9	394.7	0.0	18.9	375.7	394.7	0.0	18.4	376.2	394.7	0.0	18.3	376.4
B-10	397.1	2.8	22.5	377.4	397.1	2.8	22.0	377.9	397.1	2.8	21.9	378.0
B-15A	395.0	0.0	9.9	385.2	395.0	0.0	9.4	385.6	395.0	0.0	9.2	385.8
B-16	397.8	2.3	42.5	357.5	397.8	2.3	41.6	358.5	397.8	2.3	40.8	359.2
B-21	395.1	0.0	5.6	389.6	395.1	0.0	5.5	389.6	395.1	0.0	5.3	389.9
B-22	410.2	2.9	24.5	388.6	410.2	2.9	24.5	388.6	410.2	2.9	24.2	388.9
B-27	422.2	3.0	28.5	396.7	422.2	3.0	28.4	396.7	422.2	3.0	28.1	397.0
B-28	410.6	2.7	33.6	379.6	410.6	2.7	33.0	380.3	410.6	2.7	33.0	380.3
B-29	395.2	0.0	20.8	374.4	395.2	0.0	20.0	375.2	395.2	0.0	19.9	375.3
B-35	425.7	2.5	36.7	391.4	425.7	2.5	36.6	391.6	425.7	2.5	36.4	391.8
B-36	411.2	2.4	28.3	385.3	411.2	2.4	28.1	385.4	411.2	2.4	28.4	385.2
B-37	395.2	0.0	18.8	376.5	395.2	0.0	18.3	377.0	395.2	0.0	18.1	377.1
B-42	396.2	0.0	17.5	378.7	396.2	0.0	17.3	378.9	396.2	0.0	17.4	378.8
B-43	411.3	2.3	23.0	390.6	411.3	2.3	22.7	390.8	411.3	2.3	22.5	391.0
B-44	419.5	1.6	29.8	391.3	419.5	1.6	29.4	391.7	419.5	1.6	29.0	392.1
B-45	411.6	2.6	21.1	393.1	411.6	2.6	21.1	393.1[a]	411.6	2.6	21.1	393.1[a]
B-46	420.3	3.3	23.3	400.3[a]	420.3	3.3	23.3	400.3[a]	420.3	3.3	23.2	400.3[a]

Change in elevation
Significant Change

[a] Dry

Stantec

Cumberland Fossil Plant
815 Cumberland City Rd
Cumberland City,TN
175539009

	2/4/2011				3/10/2011				4/7/2011			
Location	Surface Elevation (ft)	Stickup (ft)	Depth Measurement(ft)	Water Elevation (ft)	Surface Elevation (ft)	Stickup (ft)	Depth Measurement(ft)	Water Elevation (ft)	Surface Elevation (ft)	Stickup (ft)	Depth Measurement(ft)	Water Elevation (ft)
B-3	394.8	0.0	11.1	383.7	394.8	0.0	11.1	383.7	394.8	0.0	11.0	383.8
B-4	393.9	2.7	11.6	385.0	393.9	2.7	10.5	386.1	393.9	2.7	10.6	386.1
B-9	394.7	0.0	18.2	376.5	394.7	0.0	17.0	377.7	394.7	0.0	17.2	377.5
B-10	397.1	2.8	21.9	378.0	397.1	2.8	20.8	379.1	397.1	2.8	20.9	379.0
B-15A	395.0	0.0	9.2	385.8	395.0	0.0	8.7	386.3	395.0	0.0	8.7	386.3
B-16	397.8	2.3	40.4	359.6	397.8	2.3	35.3	364.8	397.8	2.3	37.7	362.3
B-21	395.1	0.0	5.6	389.6	395.1	0.0	5.2	389.9	395.1	0.0	5.3	389.8
B-22	410.2	2.9	24.5	388.6	410.2	2.9	24.0	389.1	410.2	2.9	24.2	388.9
B-27	422.2	3.0	28.1	397.0	422.2	3.0	27.4	397.7	422.2	3.0	27.5	397.6
B-28	410.6	2.7	32.9	380.4	410.6	2.7	31.9	381.4	410.6	2.7	32.3	380.9
B-29	395.2	0.0	19.4	375.8	395.2	0.0	19.5	375.7	395.2	0.0	19.8	375.4
B-35	425.7	2.5	36.3	391.9	425.7	2.5	35.4	392.8	425.7	2.5	35.0	393.2
B-36	411.2	2.4	27.9	385.6	411.2	2.4	27.3	386.3	411.2	2.4	27.3	386.3
B-37	395.2	0.0	18.1	377.1	395.2	0.0	17.6	377.6	395.2	0.0	17.8	377.4
B-42	396.2	0.0	18.2	378.1	396.2	0.0	17.5	378.7	396.2	0.0	17.1	379.1
B-43	411.3	2.3	23.0	390.5	411.3	2.3	22.0	391.5	411.3	2.3	26.7	386.9
B-44	419.5	1.6	29.5	391.6	419.5	1.6	28.7	392.4	419.5	1.6	28.2	392.8
B-45	411.6	2.6	NA	NA[a]	411.6	2.6	21.1	393.1[a]	411.6	2.6	[a]	[a]
B-46	420.3	3.3	NA	NA[a]	420.3	3.3	23.2	400.3[a]	420.3	3.3	[a]	[a]

Change in elevation [a] Dry
Significant Change

Stantec

Cumberland Fossil Plant
815 Cumberland City Rd
Cumberland City,TN
175539009

	5/4/2011				6/9/2011				7/13/2011			
Location	**Surface Elevation (ft)**	**Stickup (ft)**	**Depth Measurement(ft)**	**Water Elevation (ft)**	**Surface Elevation (ft)**	**Stickup (ft)**	**Depth Measurement(ft)**	**Water Elevation (ft)**	**Surface Elevation (ft)**	**Stickup (ft)**	**Depth Measurement(ft)**	**Water Elevation (ft)**
B-3	394.8	0.0	10.8	384.0	394.8	0.0	10.9	383.9	394.8	0.0	11.1	383.7
B-4	393.9	2.7	9.6	387.0	393.9	2.7	10.3	386.3	393.9	2.7	Abandoned	
B-9	394.7	0.0	14.6	380.1	394.7	0.0	17.1	377.5	394.7	0.0	17.1	377.6
B-10	397.1	2.8	18.6	381.3	397.1	2.8	20.6	379.3	397.1	2.8	Abandoned	
B-15A	395.0	0.0	7.0	388.0	395.0	0.0	8.0	387.0	395.0	0.0	8.5	386.5
B-16	397.8	2.3	25.6	374.5	397.8	2.3	38.0	362.0	397.8	2.3	Abandoned	
B-21	395.1	0.0	4.7	390.5	395.1	0.0	4.7	390.4	395.1	0.0	5.3	389.9
B-22	410.2	2.9	23.2	389.9	410.2	2.9	23.6	389.5	410.2	2.9	24.2	388.9
B-27	422.2	3.0	26.8	398.4	422.2	3.0	27.3	397.8	422.2	3.0	Abandoned	
B-28	410.6	2.7	28.6	384.6	410.6	2.7	32.5	380.7	410.6	2.7	33.0	380.3
B-29	395.2	0.0	14.8	380.4	395.2	0.0	20.5	374.7	395.2	0.0	20.5	374.7
B-35	425.7	2.5	34.1	394.0	425.7	2.5	34.6	393.5	425.7	2.5	Abandoned	
B-36	411.2	2.4	26.2	387.3	411.2	2.4	27.3	386.3	411.2	2.4	27.3	386.3
B-37	395.2	0.0	15.9	379.4	395.2	0.0	18.2	377.0	395.2	0.0	18.1	377.1
B-42	396.2	0.0	17.2	379.0	396.2	0.0	17.6	378.6	396.2	0.0	16.9	379.3
B-43	411.3	2.3	21.1	392.4	411.3	2.3	21.1	392.4	411.3	2.3	21.3	392.2
B-44	419.5	1.6	27.7	393.4	419.5	1.6	27.5	393.6	419.5	1.6	Abandoned	
B-45	411.6	2.6	[a]	[a]	411.6	2.6	[b]	[b]	411.6	2.6	Abandoned	
B-46	420.3	3.3	[a]	[a]	420.3	3.3	[a]	[a]	420.3	3.3	Abandoned	

Change in elevation
Significant Change

[a] Dry
[b] Destroyed

APPENDIX A

Document 16

Stantec, Results of Pseudostatic Slope Analysis, February 15, 2012

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

February 15, 2012 ltr_001_175551015

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

Re: Results of Pseudostatic Slope Stability Analysis
Active CCP Disposal Facilities
Cumberland Fossil Plant (CUF)

Dear Mr. Turnbow:

As requested, Stantec Consulting Services Inc. (Stantec) has conducted pseudostatic slope stability analyses for ground motion levels corresponding to a return period of 2,500 years to support the U.S. Environmental Protection Agency's assessment of TVA's CCP disposal facilities. The results for Cumberland's Ash Pond and Dry Fly Ash Stack are provided in this letter.

Approach

The analyses were performed for current conditions using pseudostatic stability methods, where the added inertial load from an earthquake is assumed to be represented by a simple horizontal pseudostatic coefficient. Specifics related to the analyses/approach are as follows:

- Subsurface data was obtained from the Stantec's recent geotechnical studies performed in 2009 and 2010 time frame.
- 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 from the previous studies for analysis. For the Ash Pond, the selected section represents the facility's lowest current static (long-term) factor of safety. The section selected for the Dry Fly Ash Stack is located along the north side where a failure may impact the adjacent Ash Pond. The SLOPE/W models were updated to reflect current conditions.
- Undrained shear strength parameters were used.
- A ground motion level corresponding to a return period of 2,500 years (or approximate exceedance probability of 2% in 50 years) was used for selection of a horizontal seismic coefficient. For simplicity, the horizontal seismic coefficient was selected to equal the total hazard peak ground acceleration (rock) for 2,500 year return periods as shown in 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.

Results

The results of the pseudostatic stability analyses are enclosed (summary spreadsheet, SLOPE/W cross-sections, and plan views showing cross-section locations). The results indicate factors of safety greater than or equal to the target of 1.0.

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

Pseudostatic Stability Analysis Summary - TVA Active CCP Disposal Facilities

Cumberland Fossil Plant

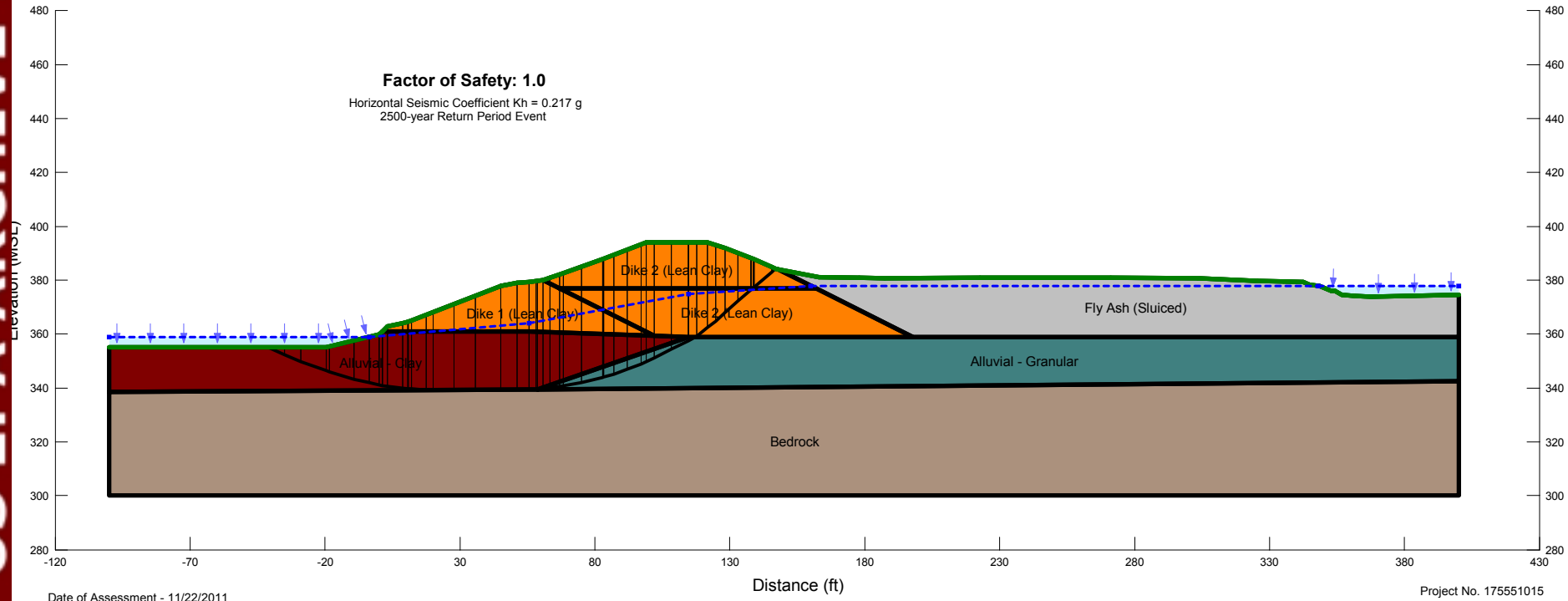
Plant	CCP Disposal Facility		Cross-Section	2,500 yr Return	
	Name	Type		PGA (g)	Factor of Safety
CUF	Ash Pond	Wet Stack	P	0.217	1.0
	Dry Fly Ash Stack	Stack	A		1.1 for shallower surface through divider dike; 1.0 for deeper surface beneath divider dike

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			



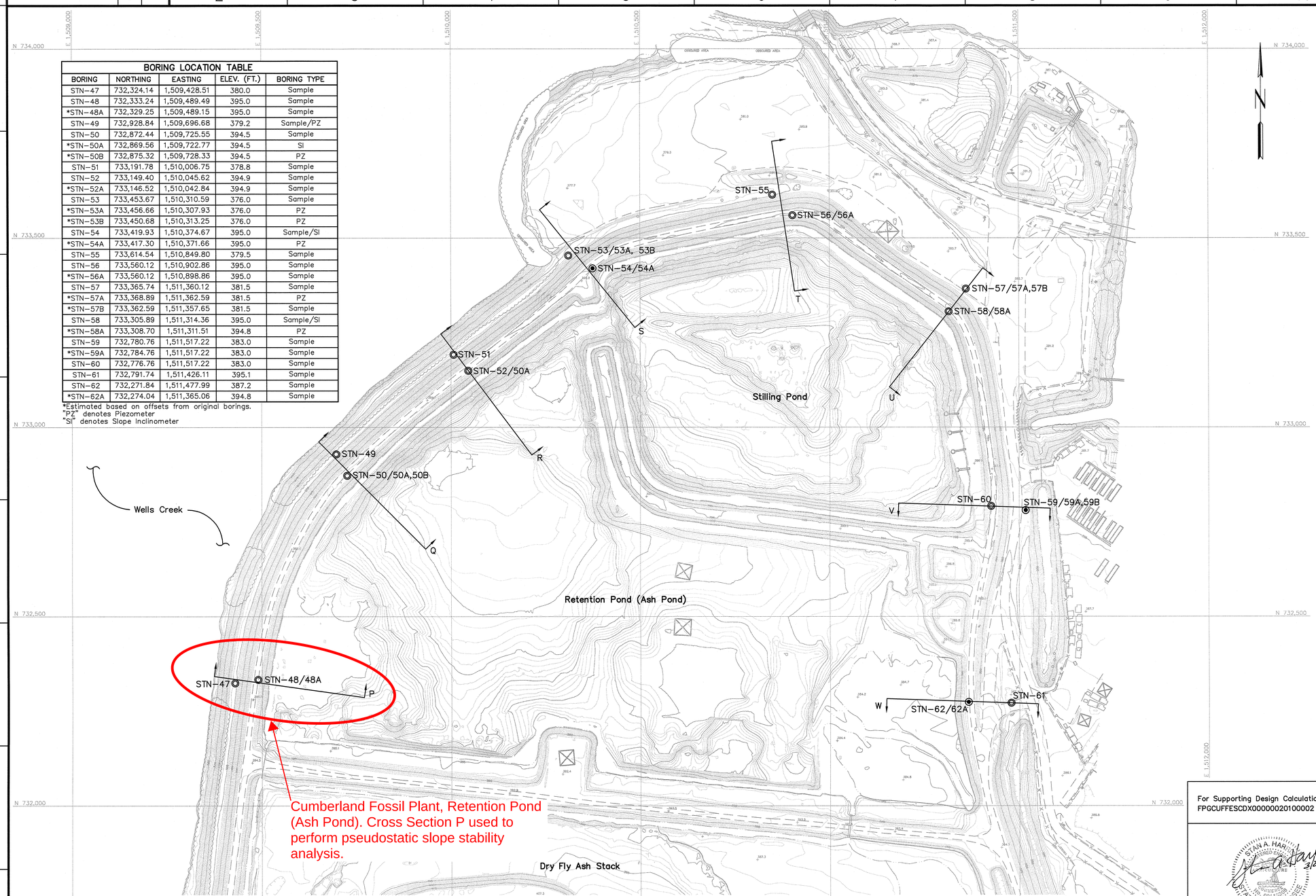
Date of Assessment - 11/22/2011

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'

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.

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

RECORD DRAWING

PLOT DATE: 03/29/2010 USER: WITHERS, CHRISTINE
V:\1755\ACTIVE\175530501\GEOTECH\CAD\BORING\REVISE_RECORD\10W544-01-R0.DWG

For Supporting Design Calculations see FPGCUFFESCDX00000020100002

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

DISCIPLINE INTERFACE

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 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

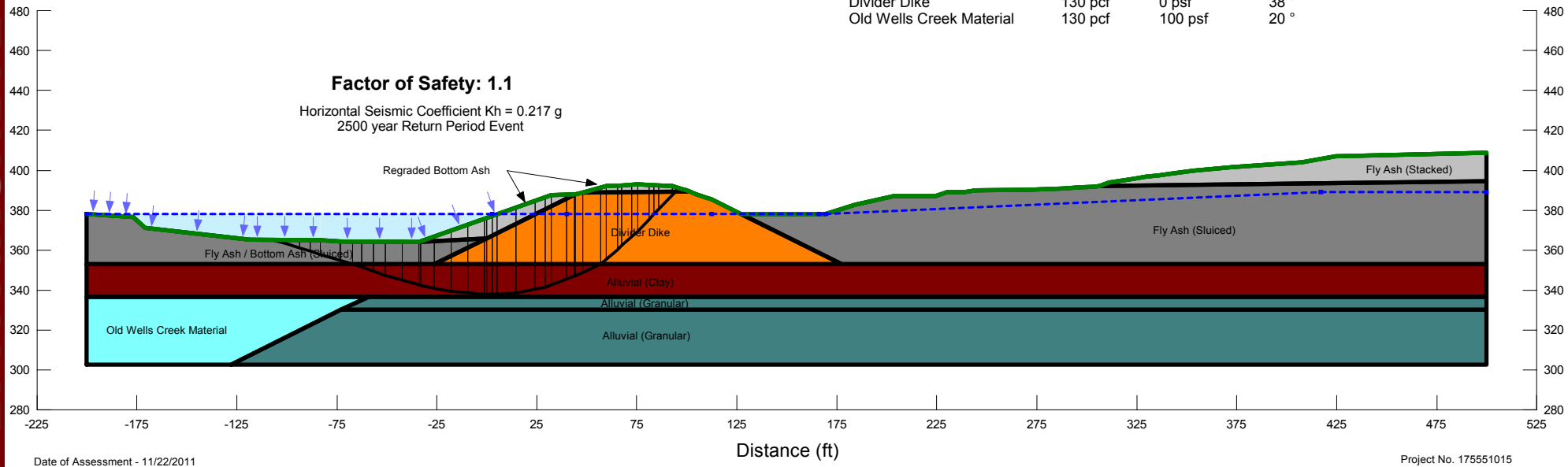
Psuedostatic Slope Stability Analysis
CCP Storage Facilities - Existing Conditions
Tennessee Valley Authority Fossil Plants

Section A - Dry Fly Ash 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
Alluvial (Clay)	121 pcf	450 psf	21 °
Alluvial (Granular)	130 pcf	0 psf	32 °
Fly Ash (Stacked)	100 pcf	0 psf	32 °
Fly Ash (Sluiced)	100 pcf	280 psf	11 °
Fly Ash / Bottom Ash (Sluiced)	100 pcf	0 psf	25 °
Regraded Bottom Ash	105 pcf	0 psf	32 °
Divider Dike	130 pcf	0 psf	38 °
Old Wells Creek Material	130 pcf	100 psf	20 °



Date of Assessment - 11/22/2011

Project No. 175551015

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

Section A - Dry Fly Ash 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
Alluvial (Clay)	121 pcf	450 psf	21 °
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Fly Ash (Stacked)	100 pcf	0 psf	32 °
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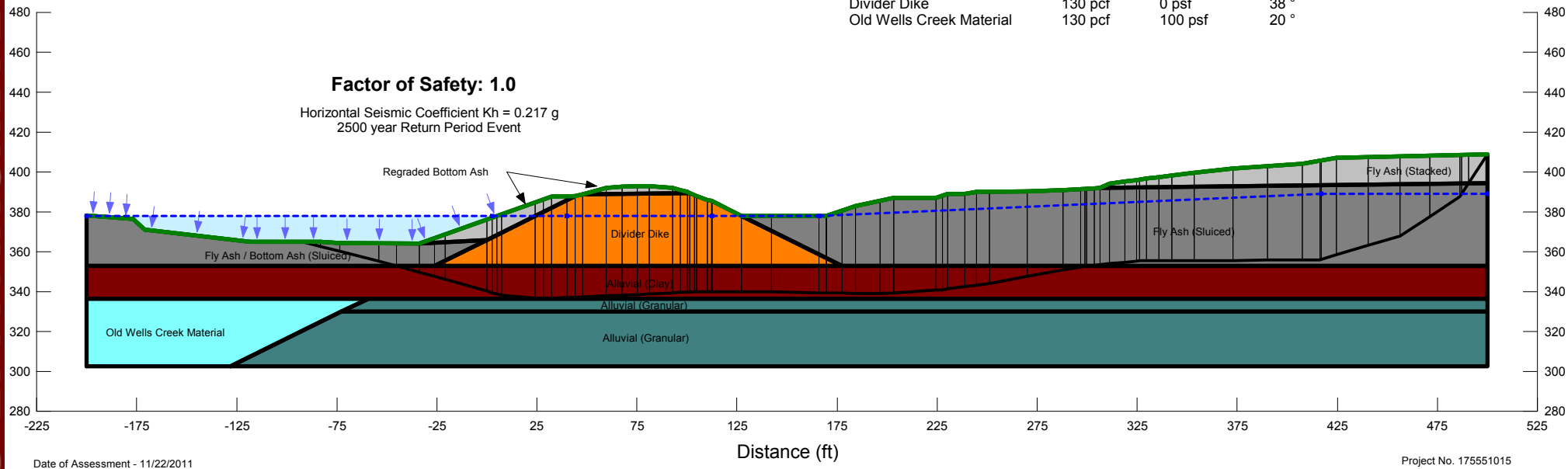
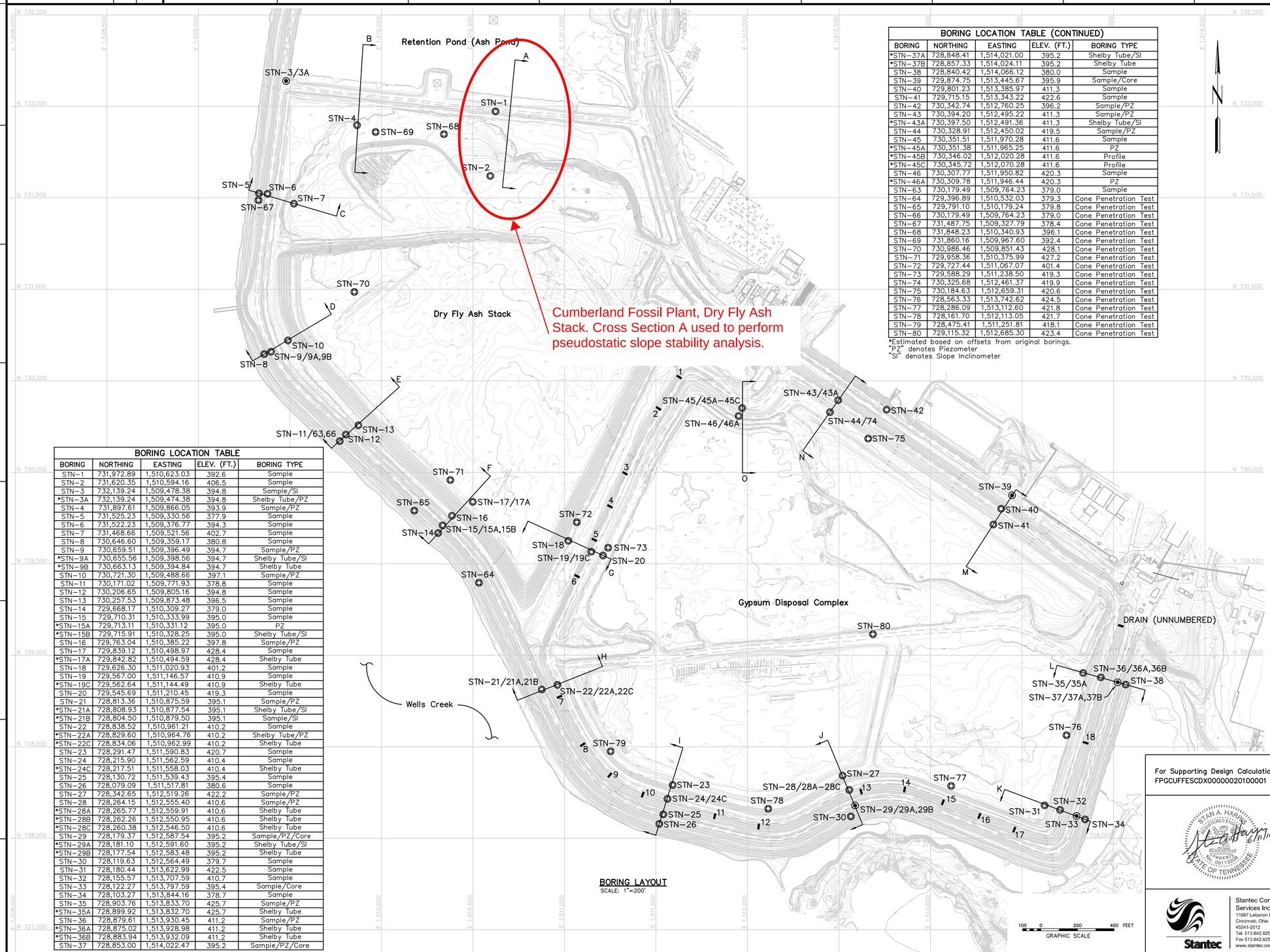
Fly Ash / Bottom Ash (Sluiced)	100 pcf	0 psf	25 °
Regraded Bottom Ash	105 pcf	0 psf	32 °
Divider Dike	130 pcf	0 psf	38 °
Old Wells Creek Material	130 pcf	100 psf	20 °



Date of Assessment - 11/22/2011

Project No. 175551015



BORING LAYOUT
SCALE: 1"=200'

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

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

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

STAN A. HARRIS REGISTERED ENGINEER NO. 0011305 STATE OF TENNESSEE 6/11/10

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

SCALE: 1"=200' EXCEPT AS NOTED

YARD
DRY FLY ASH STACK AND GYPSUM DISPOSAL COMPLEX

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 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

PLOT DATE: 06/08/2010 USER: WITHERS, CHEYENNE
K:\TVA\ACTIVE\TV35350\05 GEOTECHNICAL\DRAWING\10W543-01-R0.DWG

DISCIPLINE INTERFACE
1
2
3
4

STANTEC | 0
TASK COMPLETED BY: | REV NO.

PLOT FACTOR:XX
W_TVA

C.A.D. DRAWING
DO NOT ALTER MANUALLY

APPENDIX A

Document 17

CUF Spillway Improvement Project Letter, March 29, 2012

Stantec Consulting Services Inc.
1859 Bowles Avenue Suite 250
St. Louis MO 63026-1944
Tel: (636) 343-3880
Fax: (636) 343-3554

March 29, 2012
File: 175609014

1101 Market Street
EB 4H-C
Chattanooga, TN 37402-2801

Dear Mr. Skelton:

Reference: CUF Spillway Improvement Project
TVA Project No. 203579

A project in-service date of March 29, 2012 was taken for the Cumberland Fossil Plant Spillway Improvement Project. The retrofitted spillways are in-service and the emergency spillway has been installed. A final walk-down will be conducted on April 10, 2012 and a punch-list of any open items will be developed and included in the final closure package.

Stantec appreciates the opportunity to assist TVA on this project. If there are any questions please feel free to contact me.

Respectfully,

STANTEC CONSULTING SERVICES INC.

Matthew Hoy, PE
Senior Project Engineer
Tel: (636) 343-3880
Fax: (636) 343-3554
Matthew.Hoy@stantec.com

MAH/ncb

APPENDIX B

Document 18

Dam Inspection Check List Form – Ash Pond

Coal Combustion Dam Assessment Checklist Form

Site Name:	**Cumberland Fossil**	Date:	**September 7, 2011**
Unit Name:	**Ash Pond**	Operator's Name:	**Tennessee Valley Authority**
Unit I.D.:		Hazard Potential Classification:	**High** ☐ **Significant** ☒ **Low** ☐
Inspector's Name:		Stanford/McLaren	

Check the appropriate box below. Provide comments when appropriate. If not applicable or not available, record "N/A". Any unusual conditions or construction practices that should be noted in the comments section. For large diked embankments, separate checklists may be used for different embankment areas. If separate forms are used, identify approximate area that the form applies to in comments.

	Yes	No		Yes	No
1. Frequency of Company's Dam Inspections?	x		18. Sloughing or bulging on slopes?		x
2. Pool elevation (operator records)?	x		19. Major erosion or slope deterioration?		x
3. Decant inlet elevation (operator records)?		x	20. Decant Pipes:		
4. Open channel spillway elevation (operator records)?	x		Is water entering inlet, but not exiting outlet?		x
5. Lowest dam crest elevation (operator records)?	x		Is water exiting outlet, but not entering inlet?		x
6. If instrumentation is present, are readings recorded (operator records)?	x		Is water exiting outlet flowing clear?	x	
7. Is the embankment currently under construction?		x	21. Seepage (specify location, if seepage carries fines, and approximate seepage rate below):		
8. Foundation preparation (remove vegetation, stumps, topsoil in area where embankment fill will be placed)?	x		From underdrain?		x
9. Trees growing on embankment? (If so, indicate largest diameter below)		x	At isolated points on embankment slopes?	x	
10. Cracks or scarps on crest?		x	At natural hillside in the embankment area?	x	
11. Is there significant settlement along the crest?		x	Over widespread areas?		x
12. Are decant trashracks clear and in place?	x		From downstream foundation area?	x	
13. Depressions or sinkholes in tailings surface or whirlpool in the pool area?		x	"Boils" beneath stream or ponded water?		x
14. Clogged spillways, groin or diversion ditches?		x	Around the outside of the decant pipe?		x
15. Are spillway or ditch linings deteriorated?		x	22. Surface movements in valley bottom or on hillside?		x
16. Are outlets of decant or underdrains blocked?		x	23. Water against downstream toe?	x	
17. Cracks or scarps on slopes?		x	24. Were Photos taken during the dam inspection?	x	

Major adverse changes in these items could cause instability and should be reported for further evaluation. Adverse conditions noted in these items should normally be described (extent, location, volume, etc.) in the space below and on the back of this sheet.

Issue #	Comments

Coal Combustion Residuals (CCR)

Impoundment Assessment

Impoundment NPDES Permit TN0005789 **INSPECTOR**

Date November 30, 2007 / Expires 5-31-2010 (TVA has reapplied for permit)
Impoundment Name Ash Pond

Impoundment Company TVA-Cumberland Fossil Plant
EPA Region 4

State Agency Tennessee Department of Environment and Conservation (DEC)
(Field Office) Address 61 Forsyth Street, SW Atlanta GA 30303-1754
Name of Impoundment Ash Pond (Outfall 001)

(Report each impoundment on a separate form under the same Impoundment NPDES Permit number)

New ☐ **Update** ☒

	Yes	No
Is impoundment currently under construction? Pond is currently actively used for settling of bottom ash and storm water management for the entire CCR Complex.	☐	☒
Is water or ccw currently being pumped into the impoundment?	☒	☐

IMPOUNDMENT FUNCTION: Settling Pond

Nearest Downstream Town Name: Cumberland City

Distance from the impoundment: 1.7 Miles

Location:

Latitude	36	Degrees	23	Minutes	30.18	Seconds	**N**
Longitude	-87	Degrees	39	Minutes	48.96	Seconds	**W**

State Tennessee **County** Stewart

	Yes	No
Does a state agency regulate this impoundment?	☐	☒

If So Which State Agency? No dam safety regulatory agency, but Tenn. DEC Div. of Water Pollution Control regulates discharge.

HAZARD POTENTIAL *(In the event the impoundment should fail, the following would occur):*

☐ **LESS THAN LOW HAZARD POTENTIAL:** Failure or misoperation of the dam results in no probable loss of human life or economic or environmental losses.

☐ **LOW HAZARD POTENTIAL:** Dams assigned the low hazard potential classification are those where failure or misoperation results in no probable loss of human life and low economic and/or environmental losses. Losses are principally limited to the owner's property.

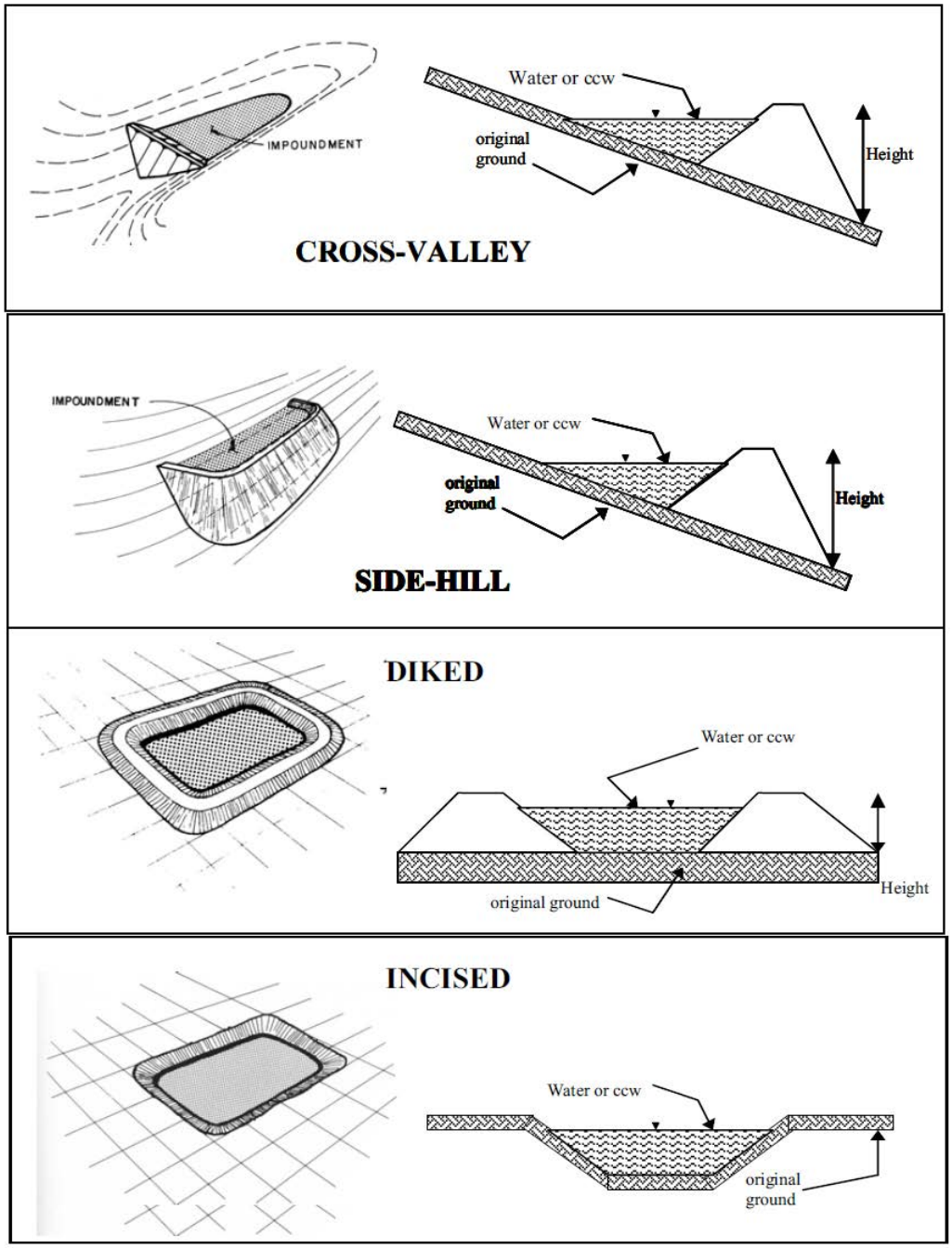
☒ **SIGNIFICANT HAZARD POTENTIAL:** Dams assigned the significant hazard potential classification are those dams where failure or misoperation results in no probable loss of human life but can cause economic loss, environmental damage, disruption of lifeline facilities, or can impact other concerns. Significant hazard potential classification dams are often located in predominantly rural or agricultural areas but could be located in areas with population and significant infrastructure.

☐ **HIGH HAZARD POTENTIAL:** Dams assigned the high hazard potential classification are those where failure or misoperation will probably cause loss of human life.

DESCRIBE REASONING FOR HAZARD RATING CHOSEN:

The Pond is considered significant hazard due to the potential for damage to the downstream state highway and bridge and due to off-site environmental damage should a failure of the impoundment occur.

CONFIGURATION:



☐ Cross-Valley ☐ Side-Hill ☒ Diked

☐ Incised (form completion optional) ☐ Combination Incised/Diked

Embankment Height (ft)	35	**Embankment Material**	Clay
Pool Area (ac)	50	**Liner**	No

Current Freeboard (ft) 10’ estimated **Liner Permeability** N/A

TYPE OF OUTLET (Mark all that apply)

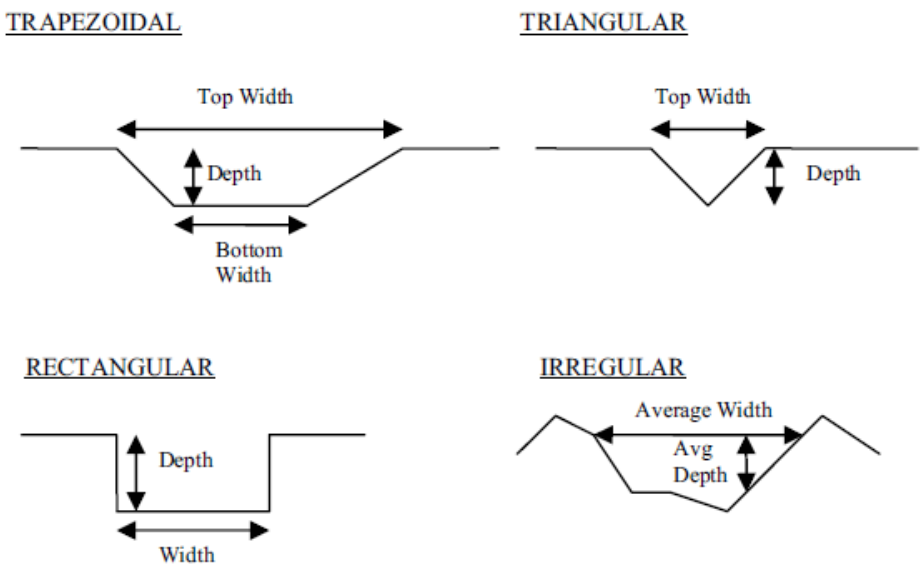
☐ **Open Channel Spillway**

- ☐ Trapezoidal
- ☐ Triangular
- ☐ Rectangular
- ☐ Irregular

depth (ft)

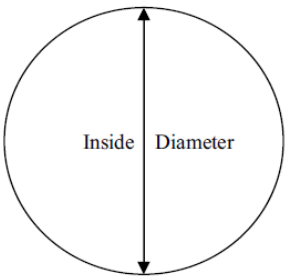
average bottom width (ft)

top width (ft)



☒ **Outlet**

4 36–inch dia. RCPs (Outfall 001)



Material

- ☐ corrugated metal
- ☐ welded steel
- ☒ concrete
- ☐ plastic (hdpe, pvc, etc.)
- ☐ other (specify):

	Yes	No
Is water flowing through the outlet?	☒	☐

☐ **No Outlet**

☐ **Other Type of Outlet** (specify):

The Impoundment was Designed By Not known at this time.

	Yes	No
Has there ever been a failure at this site?	☐	☒

If So When?

If So Please Describe :

No failure of dike at Ash Pond has occurred, but on 2/2/1997 bypass of the Cumberland Ash Pond Discharge Structure (Outfall 001) occurred when between one-half to one million gallons of gypsum wastewater spilled from the hydraulically connected Gypsum Disposal Area into Wells Creek. Heavy rainfall contributed to the failure of the internal gypsum dike, allowing a brief surge of wastewater to pass over the exterior dike in the Gypsum Disposal Area and enter the creek. The bypass lasted no longer than ten minutes.

	Yes	No
Has there ever been significant seepages at this site?	☒	☐
If So When?		

If So Please Describe :

In 1974 a seep was reported through the dike along the western side of the retention pond. A repair was performed consisting of placing a 40 foot wide clay seal on the interior of the dike. The area is monitored annually and no further seepage has been noted.

	Yes	No
Has there ever been any measures undertaken to monitor/lower Phreatic water table levels based on past seepages or breaches at this site?	☐	☒

If so, which method (e.g., piezometers, gw pumping,...)?

If So Please Describe :

ADDITIONAL INSPECTION QUESTIONS

Concerning the embankment foundation, was the embankment construction built over wet ash, slag, or other unsuitable materials*? **If there is no information just note that.*** *No construction documents are available at the time of the site visit. Current borings show that the dike raise embankments were constructed over sluiced fly ash.*

Did the dam assessor meet with, or have documentation from, the design Engineer-of-Record concerning the foundation preparation? *NO*

From the site visit or from photographic documentation, was there evidence of prior releases, failures, or patchwork on the dikes? *NO*

APPENDIX B

Document 19

Dam Inspection Check List Form – Dry Gypsum Storage

Site Name:	**Cumberland Fossil**	Date:	**September 7, 2011**
Unit Name:	**Gypsum Disposal Area**	Operator's Name:	**Tennessee Valley Authority**
Unit I.D.:		Hazard Potential Classification:	**High** ☐ **Significant** ☒ **Low** ☐
Inspector's Name:		Stanford/McLaren	

Check the appropriate box below. Provide comments when appropriate. If not applicable or not available, record "N/A". Any unusual conditions or construction practices that should be noted in the comments section. For large diked embankments, separate checklists may be used for different embankment areas. If separate forms are used, identify approximate area that the form applies to in comments.

	Yes	No		Yes	No
1. Frequency of Company's Dam Inspections?	x		18. Sloughing or bulging on slopes?		x
2. Pool elevation (operator records)?	N/A		19. Major erosion or slope deterioration?		x
3. Decant inlet elevation (operator records)?	N/A		20. Decant Pipes:		
4. Open channel spillway elevation (operator records)?	N/A		Is water entering inlet, but not exiting outlet?	N/A	
5. Lowest dam crest elevation (operator records)?	x		Is water exiting outlet, but not entering inlet?	N/A	
6. If instrumentation is present, are readings recorded (operator records)?	x		Is water exiting outlet flowing clear?	N/A	
7. Is the embankment currently under construction?		x	21. Seepage (specify location, if seepage carries fines, and approximate seepage rate below):		
8. Foundation preparation (remove vegetation, stumps, topsoil in area where embankment fill will be placed)?	x		From underdrain?		x
9. Trees growing on embankment? (If so, indicate largest diameter below)		x	At isolated points on embankment slopes?	x	
10. Cracks or scarps on crest?		x	At natural hillside in the embankment area?	x	
11. Is there significant settlement along the crest?		x	Over widespread areas?		x
12. Are decant trashracks clear and in place?	N/A		From downstream foundation area?	x	
13. Depressions or sinkholes in tailings surface or whirlpool in the pool area?	N/A		"Boils" beneath stream or ponded water?		x
14. Clogged spillways, groin or diversion ditches?		x	Around the outside of the decant pipe?	N/A	
15. Are spillway or ditch linings deteriorated?		x	22. Surface movements in valley bottom or on hillside?		x
16. Are outlets of decant or underdrains blocked?		x	23. Water against downstream toe?	x	
17. Cracks or scarps on slopes?		x	24. Were Photos taken during the dam inspection?	x	

Major adverse changes in these items could cause instability and should be reported for further evaluation. Adverse conditions noted in these items should normally be described (extent, location, volume, etc.) in the space below and on the back of this sheet.

Issue #	Comments

Coal Combustion Residuals (CCR)

Impoundment Assessment

Impoundment NPDES Permit TN0005789 **INSPECTOR**

Date November 30, 2007 / Expires 5-31-2010 (TVA has reapplied for permit)
Impoundment Name Gypsum Disposal Area

Impoundment Company TVA-Cumberland Fossil Plant
EPA Region 4

State Agency Tennessee Department of Environment and Conservation (DEC)
(Field Office) Address 61 Forsyth Street, SW Atlanta GA 30303-1754
Name of Impoundment Gypsum Disposal Area (Hydraulically connected to Outfall 001)

(Report each impoundment on a separate form under the same Impoundment NPDES Permit number)

New ☐ **Update** ☒

	Yes	No
Is impoundment currently under construction? Area is currently actively used for dry filling of gypsum stacks and infrequent wet-sluicing of gypsum slurry only as needed.	☐	☒
Is water or ccw currently being pumped into the impoundment?	☐	☒

IMPOUNDMENT FUNCTION: Dry Gypsum Storage

Nearest Downstream Town Name: Cumberland City

Distance from the impoundment: 1.7 Miles

Location:

Latitude	36	Degrees	23	Minutes	1.80	Seconds	**N**
Longitude	-87	Degrees	39	Minutes	22.08	Seconds	**W**

State Tennessee **County** Stewart

	Yes	No
Does a state agency regulate this impoundment?	☐	☒

If So Which State Agency? No dam safety regulatory agency, but Tenn. DEC Div. of Water Pollution Control regulates discharge at Ash Pond (Outfall

001).

HAZARD POTENTIAL *(In the event the impoundment should fail, the following would occur):*

☐ **LESS THAN LOW HAZARD POTENTIAL:** Failure or misoperation of the dam results in no probable loss of human life or economic or environmental losses.

☐ **LOW HAZARD POTENTIAL:** Dams assigned the low hazard potential classification are those where failure or misoperation results in no probable loss of human life and low economic and/or environmental losses. Losses are principally limited to the owner's property.

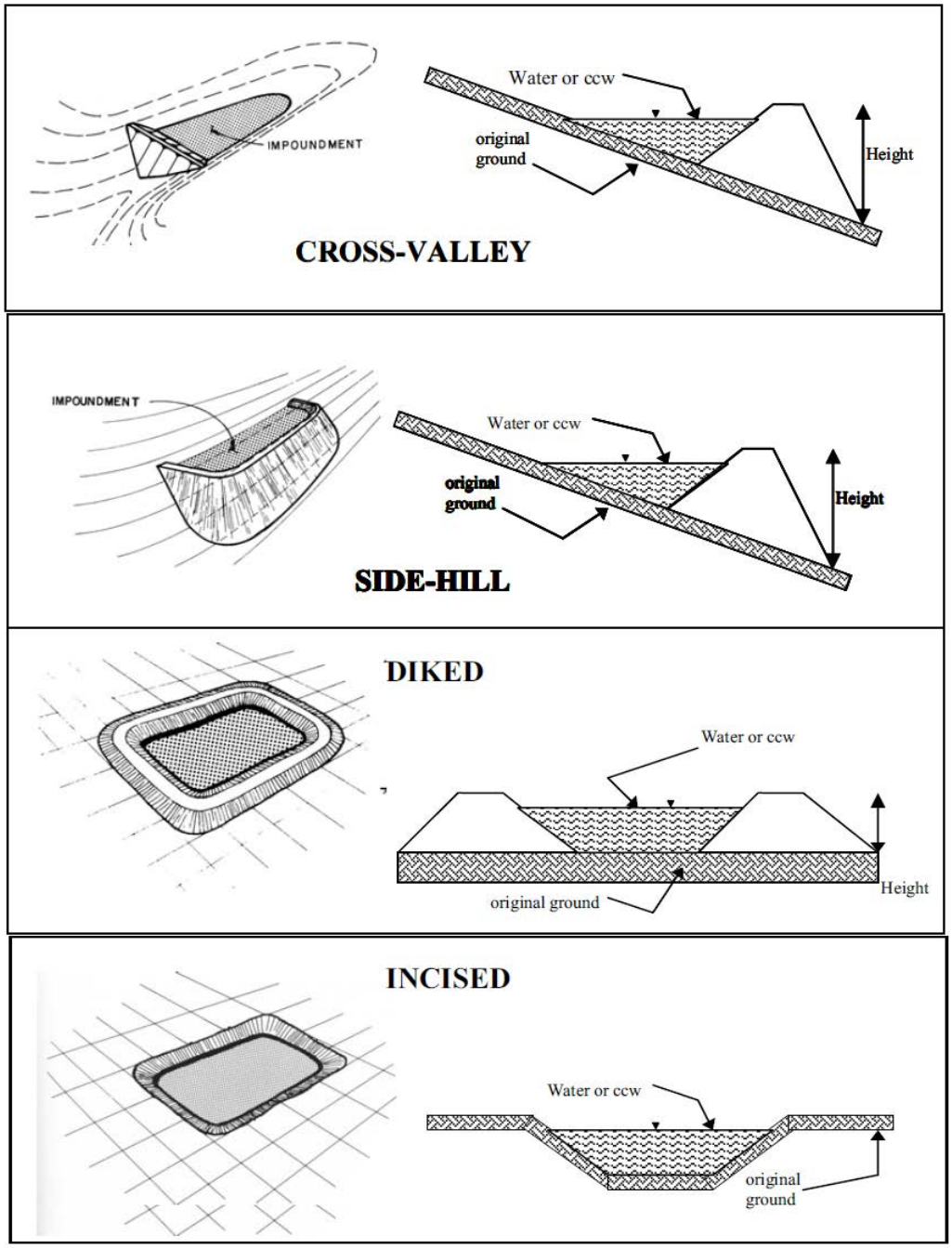
☒ **SIGNIFICANT HAZARD POTENTIAL:** Dams assigned the significant hazard potential classification are those dams where failure or misoperation results in no probable loss of human life but can cause economic loss, environmental damage, disruption of lifeline facilities, or can impact other concerns. Significant hazard potential classification dams are often located in predominantly rural or agricultural areas but could be located in areas with population and significant infrastructure.

☐ **HIGH HAZARD POTENTIAL:** Dams assigned the high hazard potential classification are those where failure or misoperation will probably cause loss of human life.

DESCRIBE REASONING FOR HAZARD RATING CHOSEN:

The Gypsum Disposal Area is considered significant hazard due to the potential for off-site environmental damage or damage to on-site structures should a failure of the containment dike occur.

CONFIGURATION:



☐ Cross-Valley ☐ Side-Hill ☒ Diked

☐ Incised (form completion optional) ☐ Combination Incised/Diked

Embankment Height (ft) 60 estimated **Embankment Material** Clay (Starter Dike Clayey Gravel)

Pool Area (ac)	170	**Liner**	No
Current Freeboard (ft)	N/A	**Liner Permeability**	N/A

TYPE OF OUTLET (Mark all that apply)

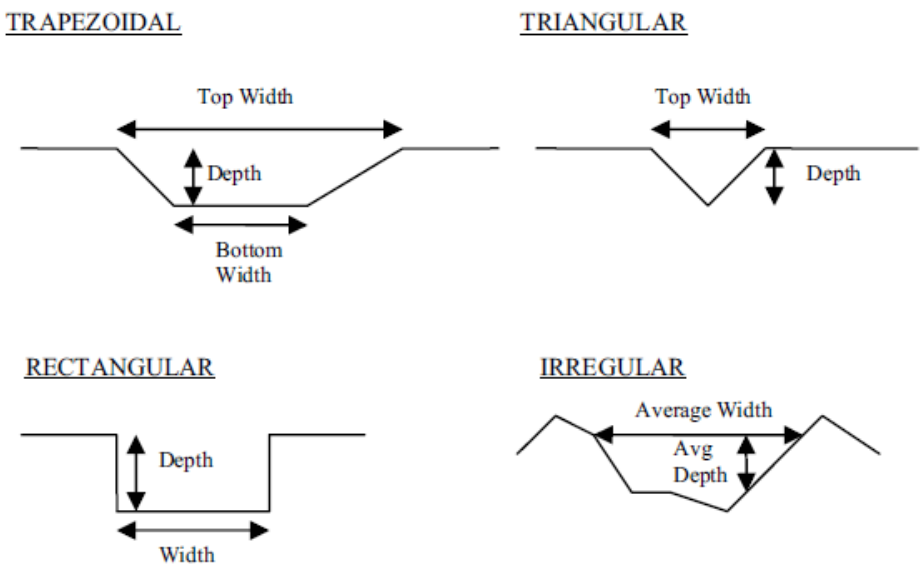
☐ **Open Channel Spillway**

- ☐ Trapezoidal
- ☐ Triangular
- ☐ Rectangular
- ☐ Irregular

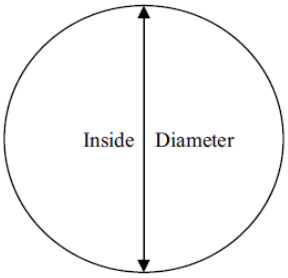
depth (ft)

average bottom width (ft)

top width (ft)



☐ **Outlet**



Material

- ☐ corrugated metal
- ☐ welded steel
- ☐ concrete
- ☐ plastic (hdpe, pvc, etc.)
- ☐ other (specify):

	Yes	No
Is water flowing through the outlet?	☐	☐

☒ **No Outlet (to Exterior)** — Hydraulically connected to Ash Pond via perimeter ditches and drainage pipes

☐ **Other Type of Outlet** (specify):

The Impoundment was Designed By Not Known at this time.

	Yes	No
Has there ever been a failure at this site?	☒	☐

If So When? 2/2/1997

If So Please Describe :

No structural failure of dike at Gypsum Disposal Area has occurred, but on 2/2/1997 bypass of the hydraulically connected Cumberland Ash Pond Discharge Structure (Outfall 001) occurred when between one-half to one million gallons of gypsum wastewater spilled from the Gypsum Disposal Area into Wells Creek. Heavy rainfall contributed to the failure of the internal gypsum dike, allowing a brief surge of wastewater to pass over the exterior dike in the Gypsum Disposal Area and enter the creek. The bypass lasted no longer than ten minutes.

	Yes	No
Has there ever been significant seepages at this site?	☒	☐

If So When? On-going

If So Please Describe : Seepage on the exterior side of the containment dike on the southwest side of the Gypsum Disposal Area is monitored in accordance with TVA's Seepage Action Plan

	Yes	No
Has there ever been any measures undertaken to monitor/lower Phreatic water table levels based on past seepages or breaches at this site?	☐	☒

If so, which method (e.g., piezometers, gw pumping,...)?

If So Please Describe :

ADDITIONAL INSPECTION QUESTIONS

Concerning the embankment foundation, was the embankment construction built over wet ash, slag, or other unsuitable materials? If there is no information just note that. *No construction documents are available at the time of the site visit. Current borings do not show that the embankments were constructed of wet ash, slag, or unsuitable materials.*

Did the dam assessor meet with, or have documentation from, the design Engineer-of-Record concerning the foundation preparation? *NO*

From the site visit or from photographic documentation, was there evidence of prior releases, failures, or patchwork on the dikes? *NO*

APPENDIX B

Document 20

Dam Inspection Check List Form – Dry Ash Storage

Site Name:	**Cumberland Fossil**	Date:	**September 7, 2011**
Unit Name:	**Dry Ash Stack**	Operator's Name:	**Tennessee Valley Authority**
Unit I.D.:		Hazard Potential Classification:	**High** ☐ **Significant** ☒ **Low** ☐
Inspector's Name:		Stanford/McLaren	

Check the appropriate box below. Provide comments when appropriate. If not applicable or not available, record "N/A". Any unusual conditions or construction practices that should be noted in the comments section. For large diked embankments, separate checklists may be used for different embankment areas. If separate forms are used, identify approximate area that the form applies to in comments.

	Yes	No		Yes	No
1. Frequency of Company's Dam Inspections?	x		18. Sloughing or bulging on slopes?		x
2. Pool elevation (operator records)?	N/A		19. Major erosion or slope deterioration?		x
3. Decant inlet elevation (operator records)?	N/A		20. Decant Pipes:		
4. Open channel spillway elevation (operator records)?	N/A		Is water entering inlet, but not exiting outlet?	N/A	
5. Lowest dam crest elevation (operator records)?	x		Is water exiting outlet, but not entering inlet?	N/A	
6. If instrumentation is present, are readings recorded (operator records)?	x		Is water exiting outlet flowing clear?	N/A	
7. Is the embankment currently under construction?		x	21. Seepage (specify location, if seepage carries fines, and approximate seepage rate below):		
8. Foundation preparation (remove vegetation, stumps, topsoil in area where embankment fill will be placed)?	x		From underdrain?		x
9. Trees growing on embankment? (If so, indicate largest diameter below)		x	At isolated points on embankment slopes?	x	
10. Cracks or scarps on crest?		x	At natural hillside in the embankment area?	x	
11. Is there significant settlement along the crest?		x	Over widespread areas?		x
12. Are decant trashracks clear and in place?	N/A		From downstream foundation area?	x	
13. Depressions or sinkholes in tailings surface or whirlpool in the pool area?	N/A		"Boils" beneath stream or ponded water?		x
14. Clogged spillways, groin or diversion ditches?		x	Around the outside of the decant pipe?	N/A	
15. Are spillway or ditch linings deteriorated?		x	22. Surface movements in valley bottom or on hillside?		x
16. Are outlets of decant or underdrains blocked?		x	23. Water against downstream toe?	x	
17. Cracks or scarps on slopes?		x	24. Were Photos taken during the dam inspection?	x	

Major adverse changes in these items could cause instability and should be reported for further evaluation. Adverse conditions noted in these items should normally be described (extent, location, volume, etc.) in the space below and on the back of this sheet.

Issue #	Comments

Coal Combustion Residuals (CCR)

Impoundment Assessment

Impoundment NPDES Permit TN0005789 **INSPECTOR**

Date November 30, 2007 / Expires 5-31-2010 (TVA has reapplied for permit)
Impoundment Name Dry Ash Stack

Impoundment Company TVA-Cumberland Fossil Plant
EPA Region 4

State Agency Tennessee Department of Environment and Conservation (DEC)
(Field Office) Address 61 Forsyth Street, SW Atlanta GA 30303-1754
Name of Impoundment Dry Ash Stack (Hydraulically connected to Outfall 001)

(Report each impoundment on a separate form under the same Impoundment NPDES Permit number)

New ☐ **Update** ☒

	Yes	No
Is impoundment currently under construction? Area is currently actively used for dry filling of ash in stacks.	☐	☒
Is water or ccw currently being pumped into the impoundment?	☐	☒

IMPOUNDMENT FUNCTION: Dry Ash Storage

Nearest Downstream Town Name: Cumberland City

Distance from the impoundment: 1.7 Miles

Location:

Latitude	36	Degrees	23	Minutes	15.00	Seconds	**N**
Longitude	-87	Degrees	39	Minutes	43.08	Seconds	**W**

State Tennessee **County** Stewart

	Yes	No
Does a state agency regulate this impoundment?	☐	☒

If So Which State Agency? No dam safety regulatory agency, but Tenn. DEC Div. of Water Pollution Control regulates discharge at Ash Pond (Outfall 001).

HAZARD POTENTIAL *(In the event the impoundment should fail, the following would occur):*

☐ **LESS THAN LOW HAZARD POTENTIAL:** Failure or misoperation of the dam results in no probable loss of human life or economic or environmental losses.

☐ **LOW HAZARD POTENTIAL:** Dams assigned the low hazard potential classification are those where failure or misoperation results in no probable loss of human life and low economic and/or environmental losses. Losses are principally limited to the owner's property.

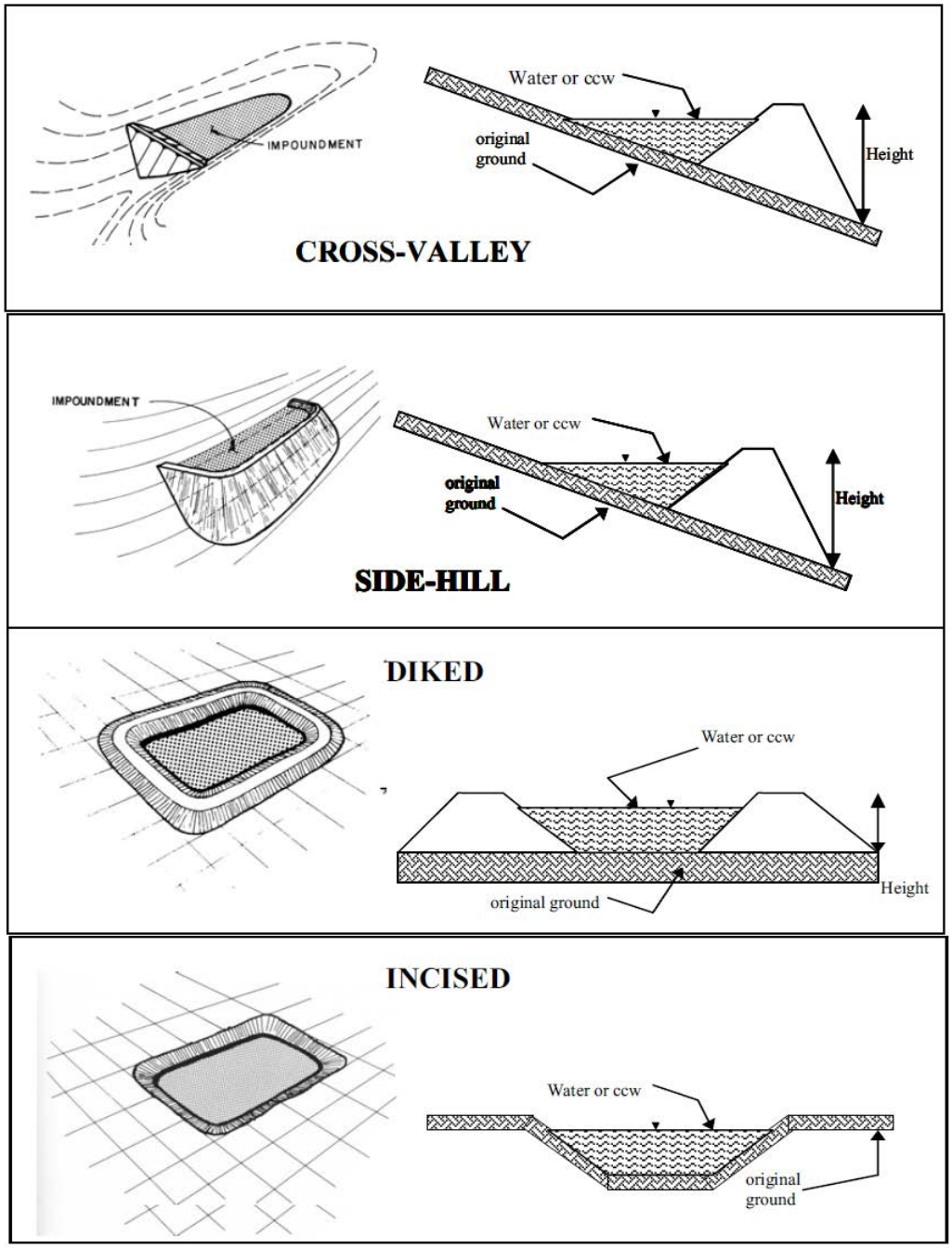
☒ **SIGNIFICANT HAZARD POTENTIAL:** Dams assigned the significant hazard potential classification are those dams where failure or misoperation results in no probable loss of human life but can cause economic loss, environmental damage, disruption of lifeline facilities, or can impact other concerns. Significant hazard potential classification dams are often located in predominantly rural or agricultural areas but could be located in areas with population and significant infrastructure.

☐ **HIGH HAZARD POTENTIAL:** Dams assigned the high hazard potential classification are those where failure or misoperation will probably cause loss of human life.

DESCRIBE REASONING FOR HAZARD RATING CHOSEN:

The Dry Ash Stack is considered significant hazard due to the potential for off-site environmental damage should a failure of the containment dike occur.

CONFIGURATION:



☐ Cross-Valley ☐ Side-Hill ☒ Diked

☐ Incised (form completion optional) ☐ Combination Incised/Diked

Embankment Height (ft)	35	**Embankment Material**	Clay
Pool Area (ac)	110	**Liner**	No

Current Freeboard (ft) N/A **Liner Permeability** N/A

TYPE OF OUTLET (Mark all that apply)

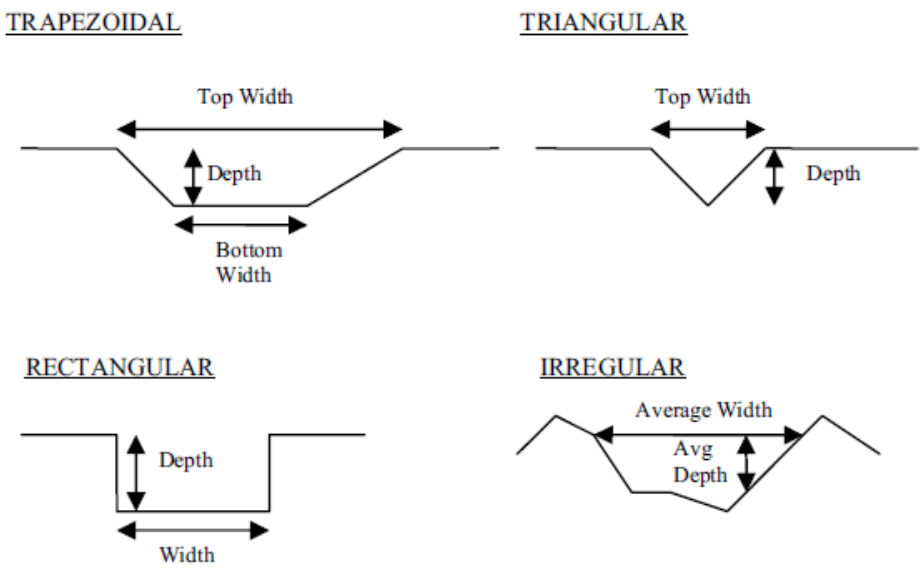
☐ **Open Channel Spillway**

- ☐ Trapezoidal
- ☐ Triangular
- ☐ Rectangular
- ☐ Irregular

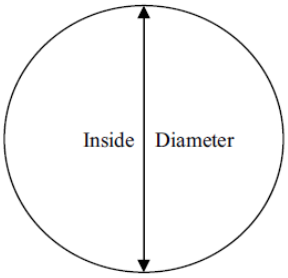
depth (ft)

average bottom width (ft)

top width (ft)



☐ **Outlet**



Material

- ☐ corrugated metal
- ☐ welded steel
- ☐ concrete
- ☐ plastic (hdpe, pvc, etc.)
- ☐ other (specify):

	Yes	No
Is water flowing through the outlet?	☐	☐

☒ **No Outlet (to Exterior)** — Hydraulically connected to Ash Pond via perimeter ditches and culverts

☐ **Other Type of Outlet** (specify):

The Impoundment was Designed By Not Known at this time.

	Yes	No
Has there ever been a failure at this site?	☐	☒

If So When?

If So Please Describe :

No failure of dike at Dry Ash Stack has occurred, but on 2/2/1997 bypass of the Cumberland Ash Pond Discharge Structure (Outfall 001) occurred when between one-half to one million gallons of gypsum wastewater spilled from the hydraulically connected Gypsum Disposal Area into Wells Creek. Heavy rainfall contributed to the failure of the internal gypsum dike, allowing a brief surge of wastewater to pass over the exterior dike in the Gypsum Disposal Area and enter the creek. The bypass lasted no longer than ten minutes.

	Yes	No
Has there ever been significant seepages at this site?	☒	☐
If So When?	On-going	

If So Please Describe :

Seepage on the exterior side of the containment dike on the southwest side of the Dry Ash Stack is monitored in accordance with TVA's Seepage Action Plan

	Yes	No
Has there ever been any measures undertaken to monitor/lower Phreatic water table levels based on past seepages or breaches at this site?	☐	☒

If so, which method (e.g., piezometers, gw pumping,...)?

If So Please Describe :

ADDITIONAL INSPECTION QUESTIONS

Concerning the embankment foundation, was the embankment construction built over wet ash, slag, or other unsuitable materials? If there is no information just note that. *No construction documents are available at the time of the site visit. Current borings show that the dike raise embankment was constructed over sluiced fly ash.*

Did the dam assessor meet with, or have documentation from, the design Engineer-of-Record concerning the foundation preparation? *NO*

From the site visit or from photographic documentation, was there evidence of prior releases, failures, or patchwork on the dikes? *NO*

APPENDIX B

Document 21

Dewberry Memorandum dated May 25, 2012, Regarding Qualitative Assessment

Memorandum

To: Stephen Hoffman, USEPA
Through: Jerry Strauss

From: Joe Klein

Date: May 25, 2012

Re: Qualitative Assessment
Liquefaction Potential
TVA Fossil Plant CCR Impoundments
Dewberry Project No, 50047151

This memorandum provides the results of a qualitative assessment of CCR impoundment embankment susceptibility to liquefaction at eight of the TVA fossil fuel plants assessed by Dewberry. The plants are: Bull Run; Colbert; Cumberland; Gallatin; John Sevier; Johnsonville; Kingston, and Widows Creek. We have not included Watts Bar (small pond, inactive for 30 years, minimal potential ash release), and Allen (TVA continuing deformation analyses, awaiting data and report)

TVA has indicated that a formal assessment of liquefaction susceptibility is underway; a completion date has not been provided. In prior rounds of the EPA CCR program, Dewberry has provided a preliminary indication of the presence of soils susceptible to liquefaction based on the geotechnical data provided with the slope stability analysis. The purpose of this assessment is to include similar information as a component of our reports to EPA, and to provide a uniform approach to the remaining plant sites.

Generally the geotechnical review looks at the soil stratification beneath both the embankments and impoundments to identify soil types considered susceptible to liquefaction; i.e., fine to medium grain sands, and some silts with Standard Penetration Resistance, or N-Values of less than 15 blows per foot[1]. That criterion, is an accepted industry standard for first level reviews.

Because several of the embankments had been constructed to their current configuration in stages, and because the raised sections were typically constructed by extending embankments in the *upstream* direction, most of TVA raised dikes are supported in part on stored bottom ash and/or fly ash. As bottom ash and fly ash are both known to be somewhat susceptible to liquefaction, an assessment of the potential impact on loss of subgrade support to the raised dike sections is a key consideration in the assessments.

For most of the other management units I have visited, the impoundments were expanded by *building out* on the downstream side of the dikes, eliminating the situation of building on the existing ash layer. The one site that did expand inward conducted a liquefaction analysis which indicated a potential for liquefaction in the ash at certain groundwater elevations. In that case the utility combined a groundwater monitoring system and construction schedule in an effort to prevent groundwater elevation

[1]Winterkorn, H.F., and Fang, H., *Foundation Engineering Handbook, Van Nostrand Reinhold, Ltd., New York, NY, 1975, pg. 268*

increases. If the approach proved to be unsuccessful, the utility had a drainage system design ready to be installed to stabilize the embankment against a potential liquefaction failure.

Because the assessments are qualitative rather than quantitative, I elected not to consider the results as indicative of either SATISFACTORY or UNSATISFACTORY. The assessed liquefaction condition at each impoundment is presented as either NO CONCERN or CONCERN. Each impoundment is assessed based on the natural foundation soils at the site, and the supporting material of raised dike sections. A composite rating is provided as described below.

The evaluations are based on the embankment cross-sections used in the recent (February 2012 and April 2012) pseudo static slope stability analyses conducted by Stantec Consulting Services for TVA.

Foundation Rating

Foundation soils are rated not only on the presence of liquefaction susceptible soils, but also the depth and thickness of the stratum, the slope of the base of the stratum, and whether the stratum extends beneath the base dike, or is restricted to the impoundment area. A CONCERN rating indicates the presence of soils susceptible to liquefaction at a relatively shallow depth below the embankment, and sufficiently thick to result in substantial deformations to the embankment in the event liquefaction occurs.

Dike Rating

Dikes were rated based on the presence of bottom ash, fly ash or other CCR material underlying raised dike sections. If the CCR material supported 50 percent or more of the raised dike, the dike received a CONCERN rating.

Composite Ratings

Composite ratings are based on a judgment of deformations that may occur to the embankments in the event of liquefaction of materials supporting the initial and/or raised dikes, The rating reflects the potential volume of material released in the event of an embankment failure, and the nature of the adjoining area expected to receive the outflow. In most cases, the controlling parameter for each perimeter dike is the potential failure of raised dikes supported in part by CCR material. Conversely, the controlling factor for interior dikes is the foundation rating.

Results

Table 1 presents a summary of the results of this assessment.

Plant	Impoundment	Liquefaction Stability Rating		
		Foundation	Dikes	Composite
Bull Run	Disposal Area 2A	NO CONCERN	CONCERN	**CONCERN**
	Disposal Area 2	NO CONCERN	NO CONCERN	NO CONCERN
	Bottom Ash Disposal Area 1	NO CONCERN	CONCERN	**CONCERN**
Colbert	Ash Pond 4	CONCERN	CONCERN	**CONCERN**
	Ash Pond 5	NO CONCERN	NO CONCERN	NO CONCERN
Cumberland	Ash Pond	NO CONCERN	NO CONCERN	NO CONCERN
Gallatin	Ash Pond A	NO CONCERN	CONCERN	NO CONCERN
	Ash Pond E	NO CONCERN	CONCERN	NO CONCERN
John Sevier	Bottom Ash Pond	NO CONCERN	NO CONCERN	NO CONCERN
	Ash Disposal Area J	NO CONCERN	NO CONCERN	NO CONCERN
Johnsonville	Ash Disposal Area 2	NO CONCERN	CONCERN	**CONCERN**
Kingston	Ash Pond Dike C	CONCERN	CONCERN	**CONCERN**
	Gypsum Stack	NO CONCERN	NO CONCERN	NO CONCERN
Widows Creek	Main Ash Pond Complex	NO CONCERN	CONCERN	**CONCERN**
	Gypsum Stack	NO CONCERN	NO CONCERN	NO CONCERN

The embankment composite ratings at Gallatin Fossil Plant are the exception to the general case of the dike rating being the controlling factor. Gallatin Ash Pond A embankment is an interior dike separating Ash Pond A and Stilling Pond B. Failure of the embankment due to liquefaction of the supporting ash would result in an intermingling of ash and decant water within the impoundment, a release from the impoundment would not be expected to occur.

Gallatin Ash Pond E is supported on an underlying layer of ash that extends beyond the toe of the embankment to a natural slope, expected to be the excavation limits for the original impoundment area. Failure of the Ash Pond E due to liquefaction of the underlying material is not expected to result in a significant release beyond the boundaries of the current impoundment,

Conclusions

Based on the results of this review, the stability of six impoundments is rated as CONCERN relative to potential liquefaction during a seismic event.

As previously discussed, the embankment stability ratings are based on a qualitative review of the current geotechnical data. More rigorous analytical assessments may arrive at different results. Such analyses should evaluate both the likelihood of liquefaction occurring from susceptible soils in the event of the design earthquake, and the effects of liquefaction on the embankments. The second phase of analyses is important to assess the risk posed by potential liquefaction of (or beneath) the CCR impoundment embankments.

Limitations

Our assessment of the stability of CCR impoundment embankments includes evaluation of many variables, including liquefaction potential. Most of the other variables have data developed with significantly more technical rigor than this qualitative assessment. *Therefore, I caution against using the results of this assessment as a primary determinant on the overall rating of a CCR impoundment.* Although reasonable judgment was used throughout the evaluation, uncertainties were evaluated using the most conservation assumptions.

Further, it is likely that the geotechnical data provided by TVA is "inconsistent" with the data (i.e., procedure) used in the Foundation Engineering Handbook (Footnote 1) to develop correlations with liquefaction susceptibility and N-values. That is, information in the TVA geotechnical reports indicate that the Standard Penetration Tests were conducted using an automatic hammer to drive the sampler. Research has shown that automatic hammers impart a significantly higher percentage of the theoretical maximum hammer to the drive anvil energy than achieved by traditional manual methods using a rope and cathead to raise and release the hammer. The result is that TVA's recorded N-values can be expected to be lower than those achieved by manual hammers in use at the time the industry-practice (i.e., Handbook) liquefaction correlations were developed.

Further, the sand strata encountered at TVA sites were below the ground water level. The boring logs indicated borings were advanced using a hollow stem auger. Hollow stem augers are a standard method for advancing soil borings, and comply with ASTM requirements. However, it is difficult to maintain the required hydrostatic head inside the augers while inserting and removing the sampler. If the hydrostatic head is not maintained, an upward gradient can develop at the tip of the auger which also reduces the N-value below the theoretical value.

It is for these reasons that the results of this assessment should not be used as the primary determinate of the overall rating for an embankment.

APPENDIX C

Document 22

Stantec, Response to Recommendations, October 16, 2012

Stantec Consulting Services Inc.
1901 Nelson Miller Parkway
Louisville KY 40223-2177
Tel: (502) 212-5000
Fax: (502) 212-5055

October 16, 2012 let_010_175551015_rev_0

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

Re: Response to Recommendations
USEPA CCR Impoundment Assessment DRAFT Report
Cumberland Fossil Plant (CUF)
Cumberland City, Tennessee

Dear Mr. Kammeyer:

As requested, Stantec has reviewed the report *Coal Combustion Residue Impoundment Dam Assessment Report, Cumberland Fossil Plant, Tennessee Valley Authority, Cumberland City, Tennessee,* dated August 2012 prepared by Dewberry and Davis, LLC (Dewberry) for the United States Environmental Protection Agency (USEPA). The purpose of this letter is to address Dewberry's conclusions and recommendations pertaining to structural stability, hydrologic/hydraulic capacity, and technical documentation; and to provide additional supporting information relative to ongoing plant improvements, further analysis, and planned activities where applicable. Dewberry's recommendations and Stantec's corresponding responses are listed below. Please note that additional seismic analysis is being conducted for the Dry Fly Ash Stack; therefore, the corresponding Dewberry recommendations for that facility will be addressed under a future submittal.

Dewberry Report Section 1.2.1 1) – Ash Pond: *Install Stantec's recommended remedial measures for increasing the factor of safety against piping failure to the acceptable margin. If the driven sheet-pile wall is selected as the remedial measure, close attention should be paid to sheet-wall alignment location and depth to achieve maximum benefit in lengthening the seepage path to reduce exit gradients; the sheet-wall alignment should generally be at or upstream of the centerline of the dike crest.*

Stantec Response: Since the time of Dewberry's assessment, the operating pool level for this pond has been lowered from El. 384.2 to El. 378.0 (6.2 feet). Stantec has revised the seepage analysis for Sections P, Q and R based on the lowered pool level and recent piezometer data. Piping factors of safety (FS) have increased to 3.5, 3.5 and 4.0 for Sections P, Q and R, respectively (see attached results). Piping FS's for these sections are now satisfactory because they are greater than the Cedergren FS criterion of 2.5 to 3 (that is

referenced in USCACE's EM 1110-2-1901, *Engineering and Design-Seepage Analysis and Control for Dams*). No remedial measures are deemed necessary.

Dewberry Report Section 1.2.1 2) – Gypsum Disposal Area: *Install the planned lined ponds in the Gypsum Disposal Area as soon as possible for receiving and settling the gypsum slurry that must be sluiced to the Gypsum Disposal Area whenever the dewatering facility has an outage. Reevaluate the piping potential factor of safety after the lined ponds have been in place for about a year, to check whether or not the elimination of sluice water in the gypsum stack reduces the seepage exit gradients sufficiently to result in acceptable factors of safety against piping. Closely monitor the seepage conditions at the critical section in the interim. If the seepage exit gradients have not sufficiently abated, develop and implement a remedial measure to lower the exit gradients and achieve acceptable factor of safety against piping failure.*

Stantec Response: The FML-Lined Gypsum Settling Channels project is currently under construction and is scheduled for completion in April 2013. Sluicing to the stack has been discontinued and has not occurred for about 3 years. Since sluicing has stopped, piezometer levels in the gypsum have lowered. Stantec has revised the seepage analysis for Section H and the current piping FS is 3.1 (see attached results) which is satisfactory. TVA will continue to collect piezometer data and ensure that an acceptable factor of safety against piping is maintained.

Dewberry Report Section 1.2.3 1) – Gypsum Disposal Area: *Perform a quantitative liquefaction analysis of embankment sections overlying very loose/ loose saturated fly ash at the Dry Fly Ash Stack and the Gypsum Disposal Area; evaluate the impact of liquefaction on the containment dikes, if liquefaction is indicated; and evaluate the consequences of liquefaction failure of the containment dikes.*

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 H was analyzed for the Gypsum Disposal Area. The saturated ash materials are anticipated to undergo liquefaction for the 2,500-year earthquake.

Dewberry Report Section 1.2.3 2) – Gypsum Disposal Area: *If it is determined that liquefaction will not occur, review/investigate any soft or very soft clays in the lower part of the dike embankments and in the alluvial foundation beneath the embankments. If significant soft/very soft clay deposits are indicated (e.g., 10 feet or more in thickness and continuous for 100 feet or more), analyze their deformation potential during the design earthquake, and assess the impact of any such deformations on the stability of the embankments.*

Stantec Response: As noted in the previous response, Stantec's analysis indicates that liquefaction will occur for saturated ash materials under the 2,500-year earthquake; therefore the deformation analysis described by Dewberry in the above recommendation is not necessary.

Dewberry Report Section 1.2.3 3) – Gypsum Disposal Area: *Review the basis and reasoning for the "design" seismic coefficient used in the pseudostatic slope stability analysis, rerun the analysis if a modification appears appropriate, or perform a higher level of analysis that uses more sophisticated methods. (Note: If a deformation analysis is done, there may be no need for the pseudostatic analysis. However, a post-earthquake static slope stability analysis using reduced shear strengths would be appropriate.)*

Stantec Response: A higher level of analysis that uses more sophisticated methods was performed for Gypsum Disposal Area Section H. A description of the methodology and the results (slope stability cross sections, including table of material parameters) are attached. The results indicate that Section H has a factor of safety of 1.1 for post-earthquake stability using reduced shear strengths, which is satisfactory.

Dewberry Report Section 1.2.1 3) – Gypsum Disposal Area: *Depending on the results of additional seismic stability analyses and of liquefaction potential analyses recommended in Subsection 1.2.3, develop and implement measures to ensure adequate performance of the Dry Fly Ash Stack and the Gypsum Disposal Area containment dikes under the 2,500-year seismic event.*

Stantec Response: The results of the liquefaction and post-earthquake stability analyses indicate that the Gypsum Disposal Area will remain stable and display adequate performance due to the 2,500-year earthquake. Therefore, no seismic-related remedial measures are required.

Dewberry Report Section 1.2.4 – Maintenance Items: No significant problems were observed in the field assessment that would require special attention outside of routine maintenance. The minor issues observed, mostly small eroded areas or areas of seepage and poor drainage, should be addressed by TVA's routine maintenance activities. These include: 1) Repair minor erosion at various locations, 2) Continue to mow/ maintain vegetation along slopes, 3) Continue to monitor and document known seepage per seepage action plan, 4) Provide positive slope to promote drainage into perimeter ditch.

Stantec Response:

1) TVA repaired the erosion as part of Work Plan 11 (see attached photos).
2) TVA's Routine Handling Operations and Maintenance (RHO&M) group will continue vegetation maintenance along slopes.
3) TVA will continue to monitor areas of known seepage in accordance with the Seepage Action Plan.
4) TVA re-graded the perimeter drainage ditch as part of Work Plan 11 (see attached photos).

Summary:

Based on the results of Dewberry's report, and on the responses/additional analyses provided herein, it is Stantec's opinion that the final rating for the CUF Ash Pond and Gypsum Disposal Area should be upgraded to Satisfactory.

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

Sincerely,

STANTEC CONSULTING SERVICES INC.

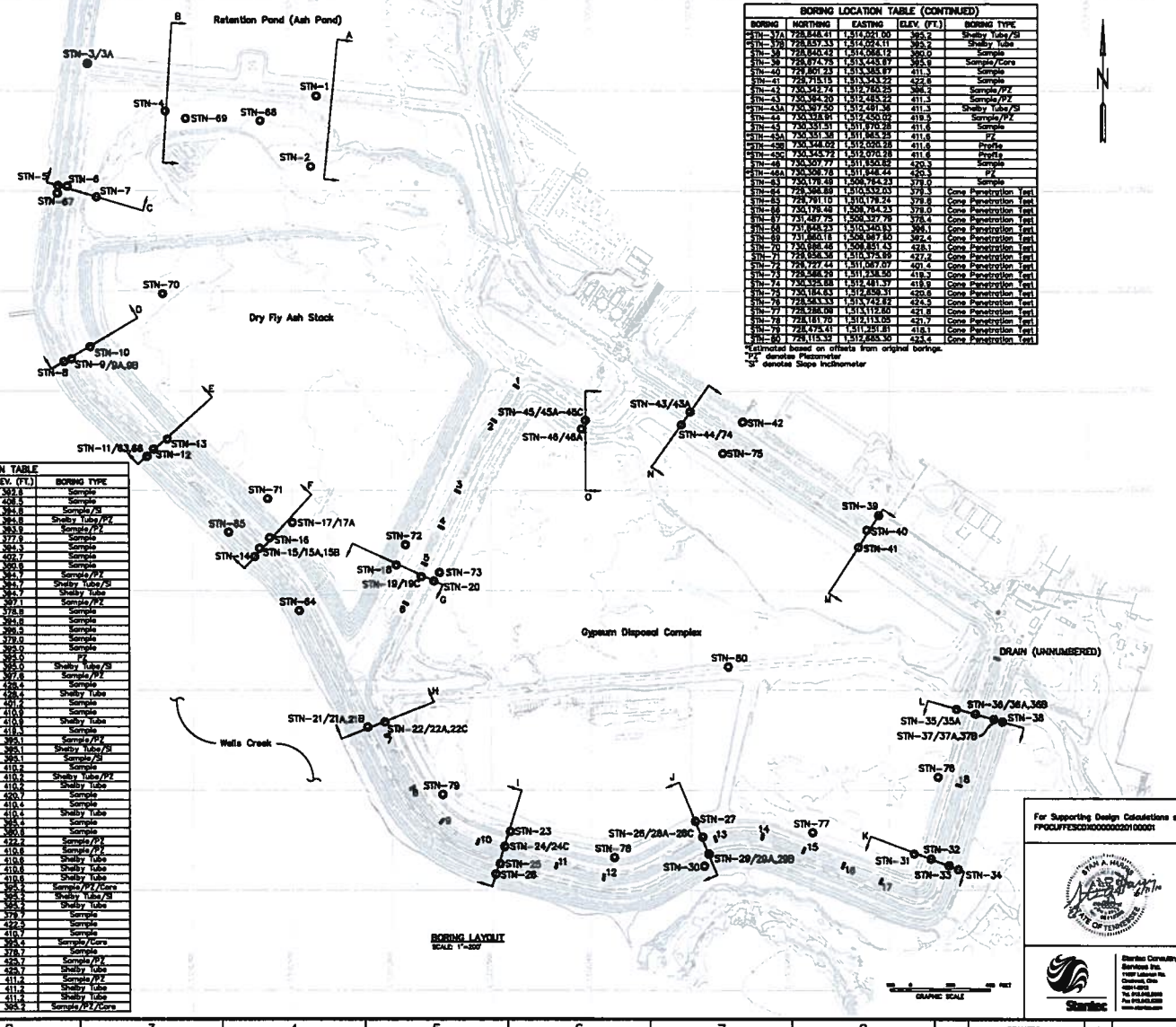
Stephen H. Bickel, PE
Senior Principal

Randy L. Roberts, PE
Principal

/db

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

Attachments



BORING LAYOUT
SCALE: 1"=200'

BORING LOCATION TABLE (CONTINUED)				
BORING	NORTHING	EASTING	ELEV. (FT.)	BORING TYPE
*STN-37A	728,846.41	1,514,521.00	385.2	Shelby Tube/SI
*STN-37B	728,837.33	1,514,524.11	385.2	Shelby Tube
STN-38	728,840.42	1,514,566.12	380.0	Sample
STN-39	728,874.75	1,513,443.87	385.6	Sample/Core
STN-40	729,801.23	1,513,383.97	411.3	Sample
STN-41	728,715.15	1,513,343.22	422.6	Sample
STN-42	730,342.74	1,512,760.25	386.2	Sample/PZ
STN-43	730,384.20	1,512,483.22	411.3	Sample/PZ
*STN-43A	730,387.30	1,512,481.34	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	1,511,963.25	411.6	PZ
*STN-45B	730,348.02	1,512,006.26	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	730,308.78	1,511,948.44	420.3	PZ
STN-83	730,178.49	1,508,784.23	378.0	Sample
STN-84	728,384.89	1,510,532.03	378.3	Cone Penetration Test
STN-85	728,791.10	1,510,178.24	378.8	Cone Penetration Test
STN-86	730,178.49	1,508,784.33	378.0	Cone Penetration Test
STN-87	731,487.75	1,508,327.75	378.4	Cone Penetration Test
STN-88	731,846.23	1,510,340.63	396.1	Cone Penetration Test
STN-89	731,800.18	1,508,087.80	382.4	Cone Penetration Test
STN-70	730,986.48	1,508,851.43	428.1	Cone Penetration Test
STN-71	728,958.38	1,510,375.88	427.2	Cone Penetration Test
STN-72	728,727.44	1,511,087.07	401.4	Cone Penetration Test
STN-73	728,588.29	1,511,238.50	418.3	Cone Penetration Test
STN-74	730,325.88	1,512,481.37	418.0	Cone Penetration Test
STN-75	730,184.63	1,512,858.31	420.8	Cone Penetration Test
STN-76	728,583.33	1,513,742.82	424.5	Cone Penetration Test
STN-77	728,586.08	1,513,117.80	421.8	Cone Penetration Test
STN-78	728,181.70	1,513,113.05	421.7	Cone Penetration Test
STN-79	728,475.41	1,511,251.81	418.1	Cone Penetration Test
STN-80	728,115.32	1,512,805.30	423.4	Cone Penetration Test

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

BORING LOCATION TABLE				
BORING	NORTHING	EASTING	ELEV. (FT.)	BORING TYPE
STN-1	731,972.88	1,510,823.03	382.8	Sample
STN-2	731,820.35	1,510,594.18	408.5	Sample
STN-3	732,138.74	1,509,478.38	384.8	Sample/SI
*STN-3A	732,138.74	1,509,474.38	384.8	Shelby Tube/PZ
STN-4	731,897.81	1,509,866.05	383.9	Sample/PZ
STN-5	731,555.33	1,509,330.58	377.9	Sample
STN-6	731,522.23	1,509,376.77	384.3	Sample
STN-7	731,488.68	1,509,531.56	402.7	Sample
STN-8	730,648.80	1,509,358.17	380.6	Sample
STN-9	730,658.51	1,509,384.40	381.1	Sample/PZ
*STN-9A	730,653.98	1,509,388.58	384.7	Shelby Tube/SI
*STN-9B	730,653.13	1,509,394.84	384.7	Shelby Tube
STN-10	730,721.30	1,509,488.68	387.1	Sample/PZ
STN-11	730,171.02	1,509,771.83	378.1	Sample
STN-12	730,008.83	1,509,805.18	384.6	Sample
STN-13	730,257.33	1,509,873.48	386.5	Sample
STN-14	729,888.17	1,510,308.27	379.0	Sample
STN-15	729,710.31	1,510,333.98	393.0	Sample
*STN-15A	729,713.11	1,510,331.12	393.0	PZ
*STN-15B	729,715.81	1,510,328.25	393.0	Shelby Tube/SI
STN-16	729,743.14	1,510,383.37	397.6	Sample/PZ
STN-17	729,438.12	1,510,488.97	428.4	Sample
*STN-17A	729,842.82	1,510,494.58	428.4	Shelby Tube
STN-18	729,829.30	1,511,020.83	401.2	Sample
STN-19	729,587.37	1,511,148.57	410.0	Sample
*STN-19C	729,582.84	1,511,144.48	410.0	Shelby Tube
STN-20	729,545.69	1,511,210.45	418.3	Sample
STN-21	728,813.38	1,510,815.58	395.1	Sample/PZ
*STN-21A	728,808.83	1,510,817.54	395.1	Shelby Tube/SI
*STN-21B	728,804.50	1,510,819.50	395.1	Sample/SI
STN-22	728,836.53	1,510,981.31	410.2	Sample
*STN-22A	728,833.60	1,510,984.76	410.2	Shelby Tube/PZ
*STN-22C	728,834.08	1,510,982.89	410.2	Shelby Tube
STN-23	728,391.47	1,511,590.83	420.7	Sample
STN-24	728,315.80	1,511,582.58	410.4	Sample
*STN-24C	728,317.31	1,511,584.03	410.4	Shelby Tube
STN-25	728,130.72	1,511,538.43	395.4	Sample
STN-26	728,078.09	1,511,517.81	380.8	Sample
STN-27	728,342.15	1,512,318.26	422.2	Sample/PZ
STN-28	728,264.15	1,512,593.40	410.6	Sample/PZ
*STN-28A	728,265.77	1,512,594.91	410.6	Shelby Tube
*STN-28B	728,262.28	1,512,590.95	410.6	Shelby Tube
*STN-28C	728,260.33	1,512,588.50	410.6	Shelby Tube
STN-29	728,179.37	1,512,587.54	395.2	Sample/PZ/Core
*STN-29A	728,181.10	1,512,591.80	395.2	Shelby Tube/SI
*STN-29B	728,177.54	1,512,583.48	395.2	Shelby Tube
STN-30	728,118.83	1,512,584.48	378.7	Sample
STN-31	728,180.44	1,513,622.98	422.5	Sample
STN-32	728,155.57	1,513,707.38	410.7	Sample
STN-33	728,122.37	1,513,797.38	395.4	Sample/Core
STN-34	728,103.27	1,513,844.18	378.7	Sample
STN-35	728,905.76	1,513,833.70	423.7	Sample/PZ
*STN-35A	728,898.93	1,513,832.70	423.7	Shelby Tube
STN-36	728,878.81	1,513,830.45	411.2	Sample/PZ
*STN-36A	728,875.02	1,513,828.98	411.2	Shelby Tube
*STN-36B	728,863.84	1,513,832.08	411.2	Shelby Tube
STN-37	728,853.00	1,514,022.47	385.2	Sample/PZ/Core

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 NGVD29 using State Plane Tennessee coordinate system. The site photography was performed on 4/17/2008.

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 FPOCUFFSCDX00000020100001

STAN A. HUNTER
STATE OF TENNESSEE

Stantec

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www.stantec.com

REV	DATE	DRAWN	CHK	DSGN	SAH	SAH	SAH	TJ
0	06/11/10							

RECORD DRAWING

SCALE: 1"=200' EXCEPT AS NOTED

YARD
DRY FLY ASH STACK AND GYPSUM DISPOSAL COMPLEX

GEOTECHNICAL EXPLORATION
BORING LAYOUT

DESIGNED BY	DRAWN BY	REVIEWED BY	CONTROLLED BY	MANAGER BY	APPROVED BY	DATE
G. BORDEN	G. BORDEN	G. BORDEN	S. HARDIN	S. HARDIN	S. HARDIN	S. JOHNSON

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

SLOPE STABILITY ANALYSIS
Cumberland Fossil Plant - Fly Ash Stack
Tennessee Valley Authority (TVA)

File Name: Section P.gsz

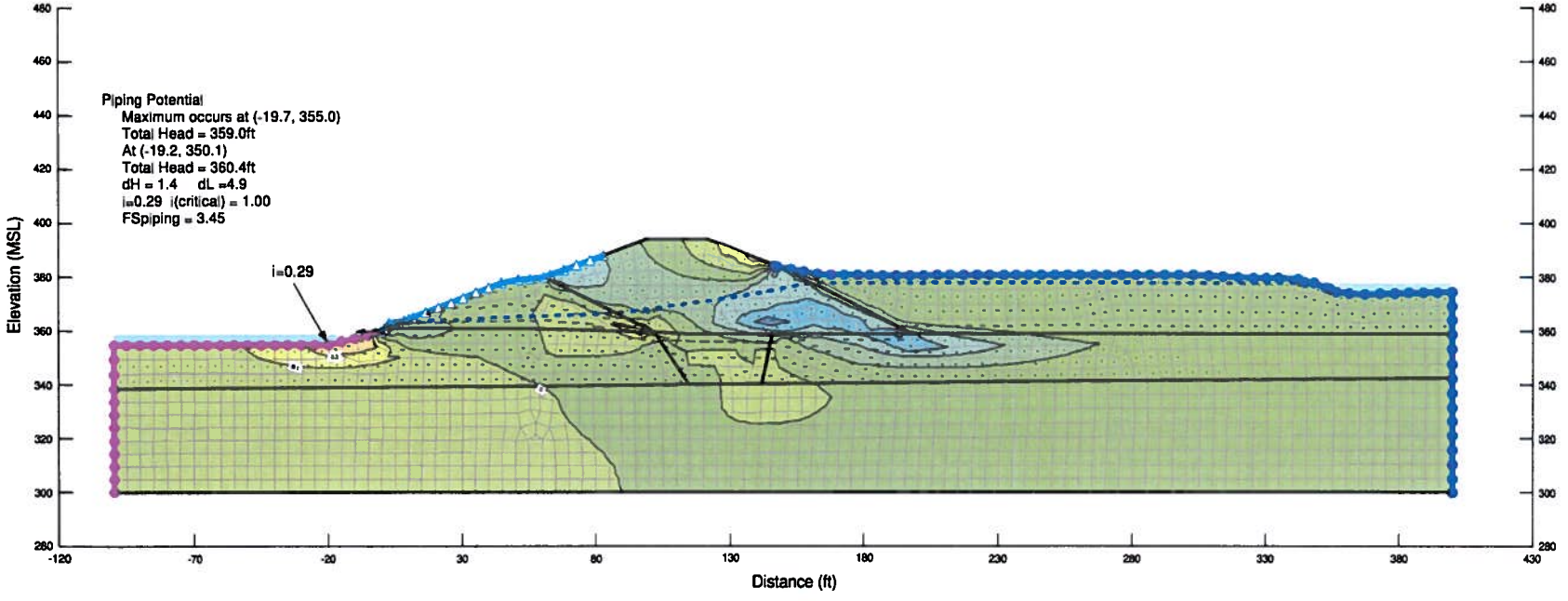
Analysis Name: Steady-State Seepage

Date Saved: 10/12/2012

Last Solved on 10/12/2012 at 1:03:58 PM

Stantec

Y-Gradient



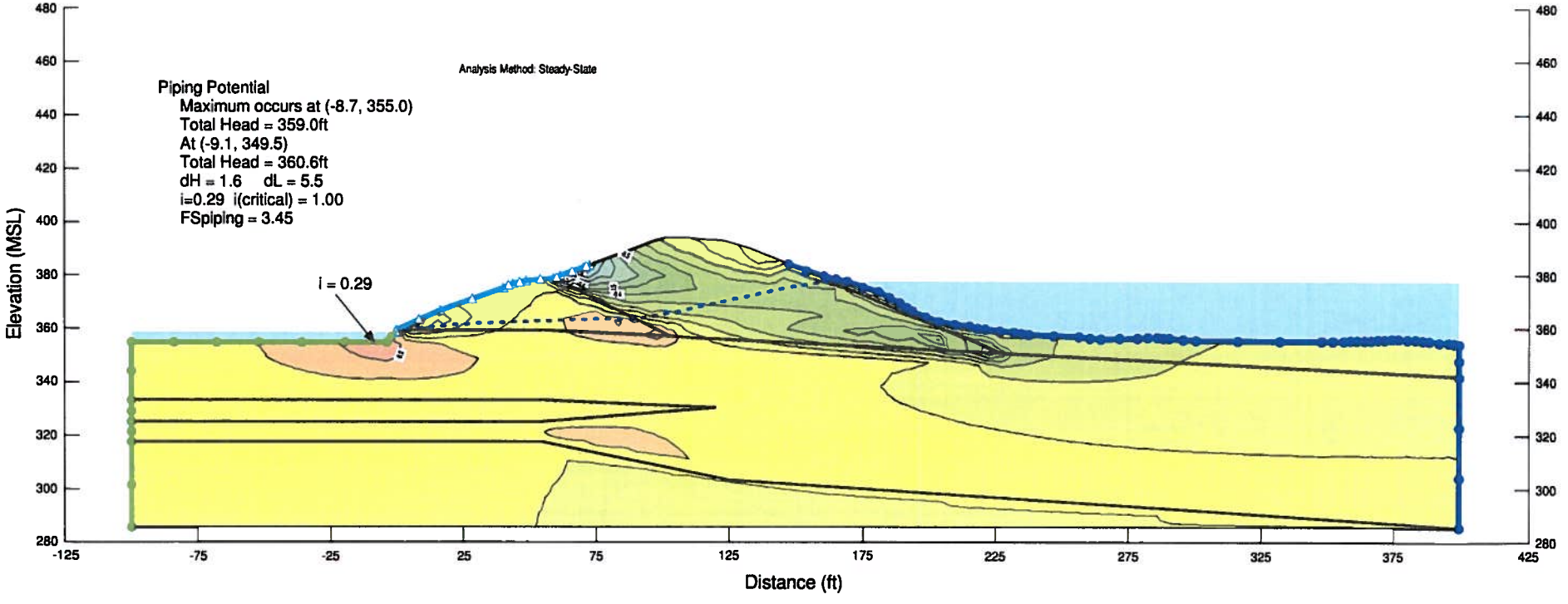
SLOPE STABILITY ANALYSIS
Cumberland Fossil Plant - Fly Ash Stack
Tennessee Valley Authority (TVA)

File Name: Section Q.gsz

Analysis Name: Steady-State Seepage
Last Solved on 9/20/2012 at 9:33:02 AM
Date Saved: 9/20/2012

Stantec

Y Gradient



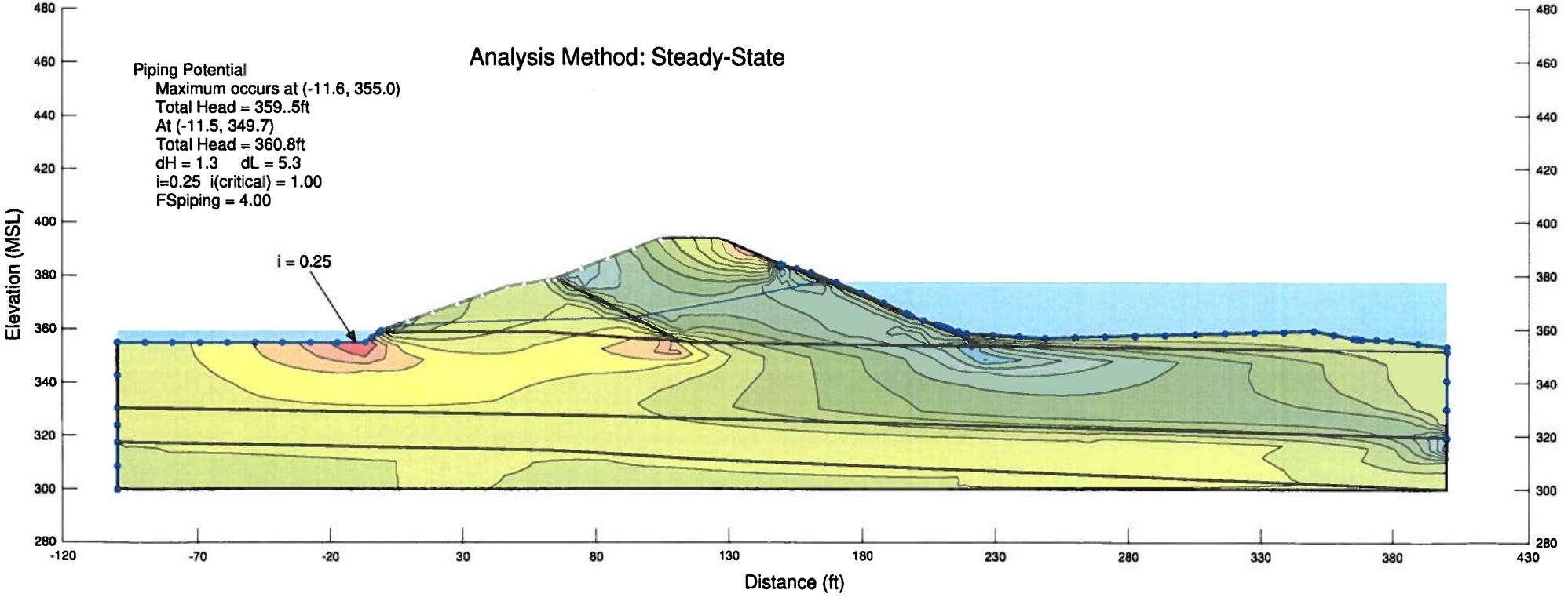
SLOPE STABILITY ANALYSIS
Cumberland Fossil Plant - Fly Ash Stack
Tennessee Valley Authority (TVA)

File Name: Section R.gsz

Analysis Name: Steady-State Seepage
Last Solved on 9/20/2012 at 9:49:06 AM
Date Saved: 9/20/2012

Stantec

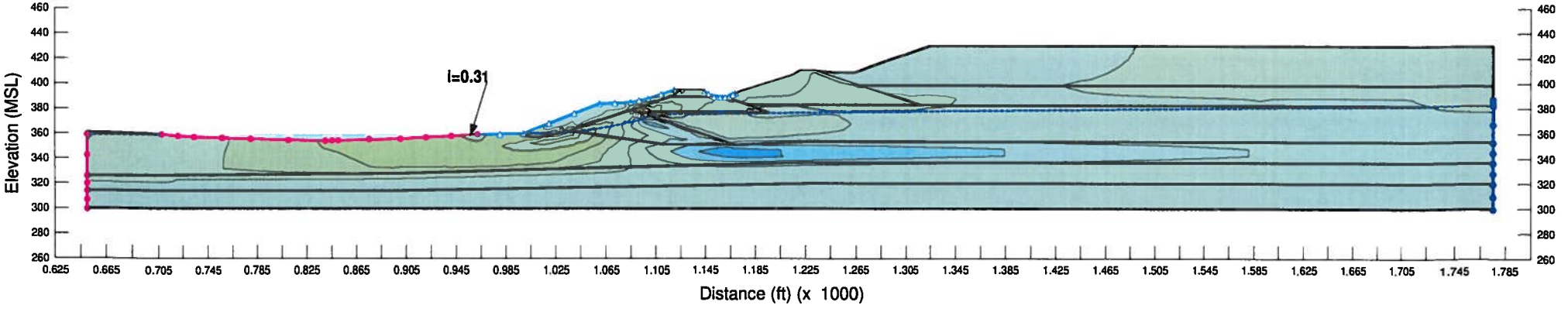
Y-Gradient



SLOPE STABILITY ANALYSIS
Cumberland Fossil Plant - Gypsum Stack Complex
Tennessee Valley Authority (TVA)

File Name: Section H (StabRepDgn)revised.gsz
Analysis Name: Steady-State Seepage
Date Saved: 10/12/2012
Last Solved on 10/12/2012 at 11:06:02 AM
Analysis Method: Steady-State

Piping Potential
Maximum occurs at (956.7, 358.7)
Total Head = 359.0ft
At (956.6, 353.9)
Total Head = 360.5ft
dH = 1.50 dL = 4.80
i=0.31 i(critical) = 0.97
FSpiping = 3.13



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.

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TABLE 1

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

	Spectral Acceleration (g)		
	Mean 2,500-year UHRS		
Frequency (Hz)	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

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TABLE 4

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

Comp.	Spectrally -Matched from	PGA (g)	PGV (cm/sec)	PGD (cm)	PGV/PGA (cm/sec/g)	PGA*PGD/PGV^2	Duration (sec)**
H	SER000	0.220	11.09	6.51	50.41	11.42	29.51
H	GRN180	0.221	13.89	8.59	62.85	9.65	24.55
H	SUL320	0.224	14.42	9.66	64.38	10.21	24.23
H	KSH-T1	0.222	13.24	8.68	59.64	10.78	15.25
H	TAP067-W	0.216	12.44	8.69	57.59	11.90	38.80
H	TAP075-W	0.219	11.01	7.92	50.27	14.04	23.74
H	TAP078-N	0.220	15.41	7.96	70.05	7.23	25.74

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

DRAFT

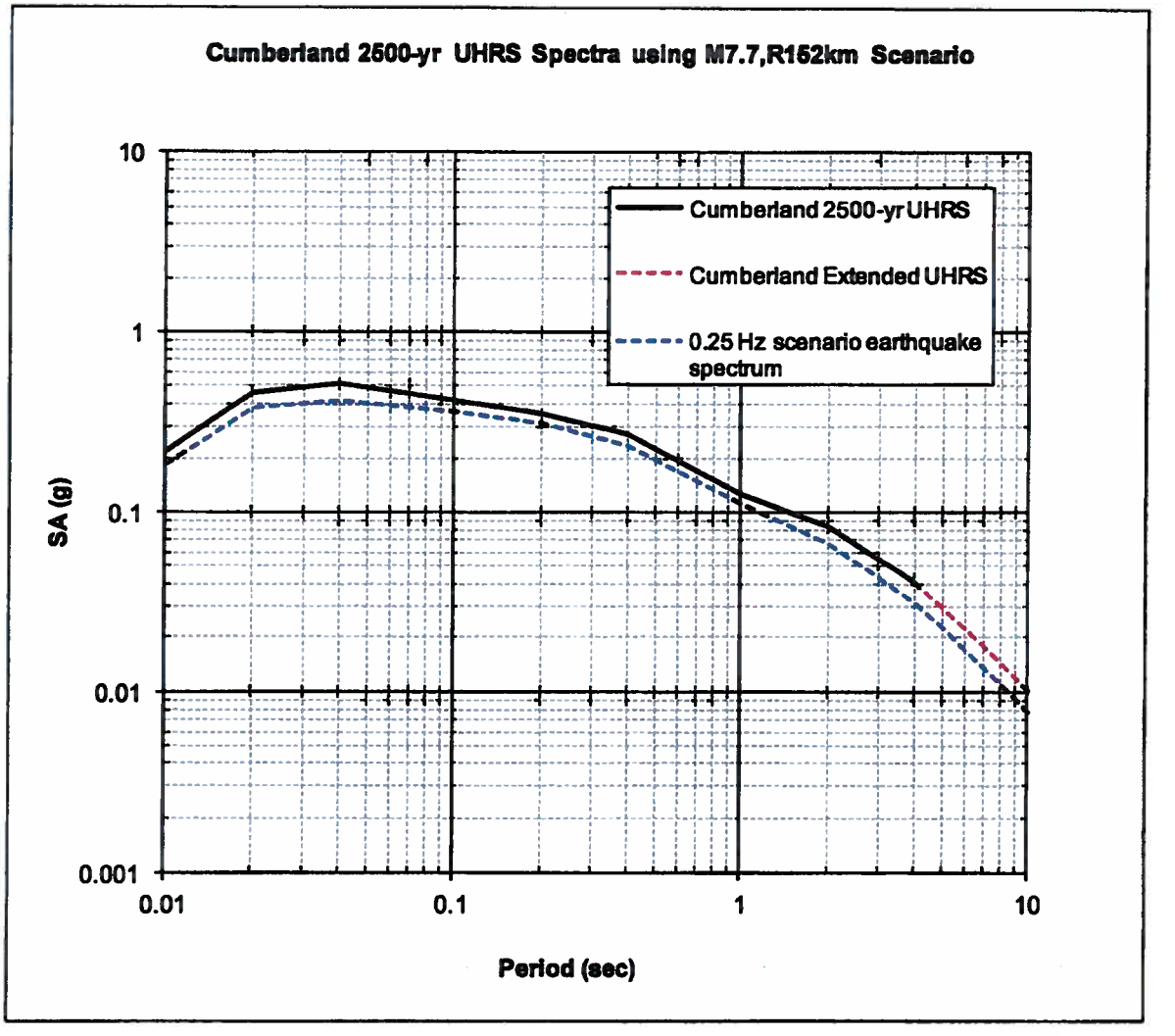
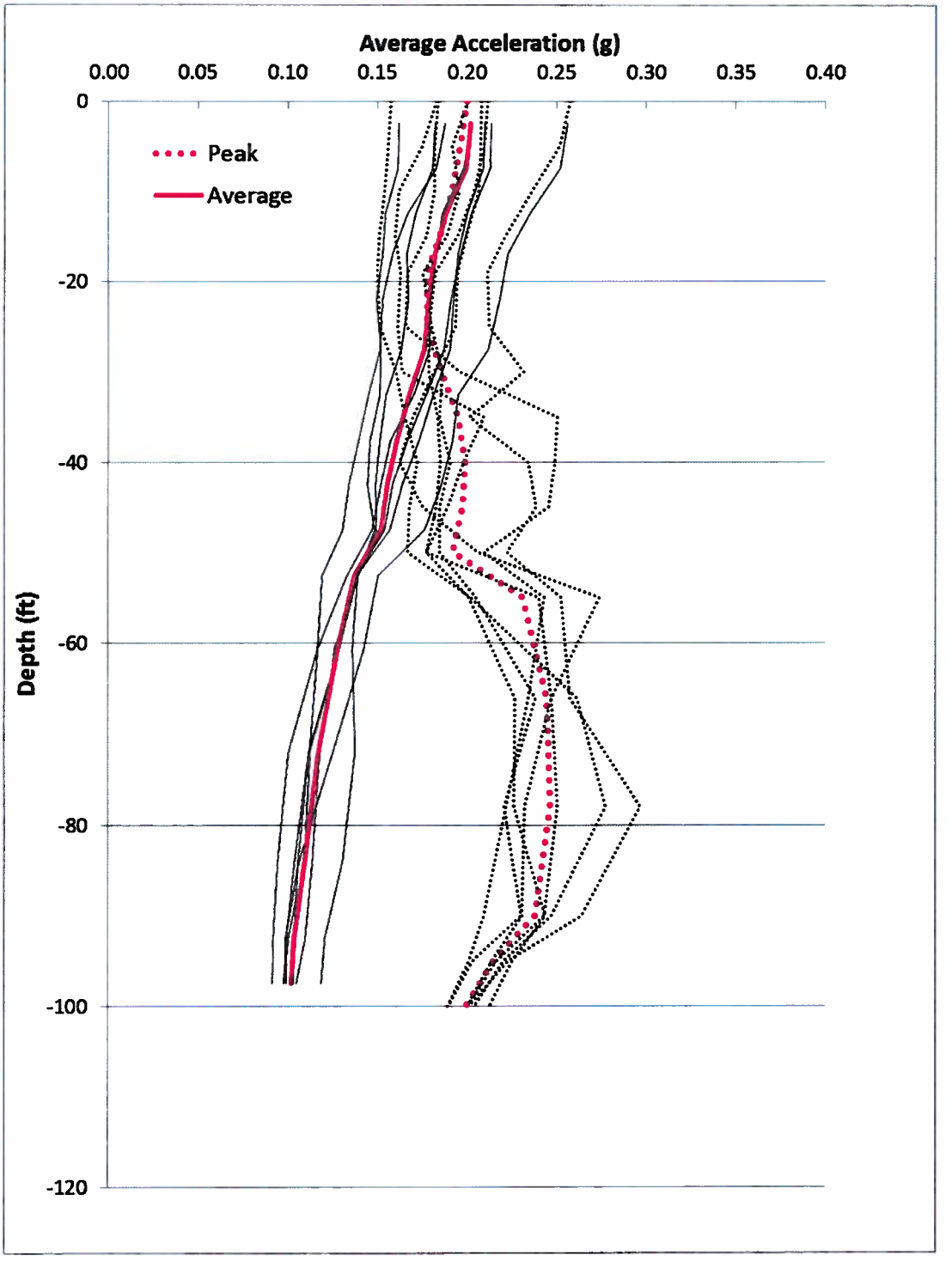
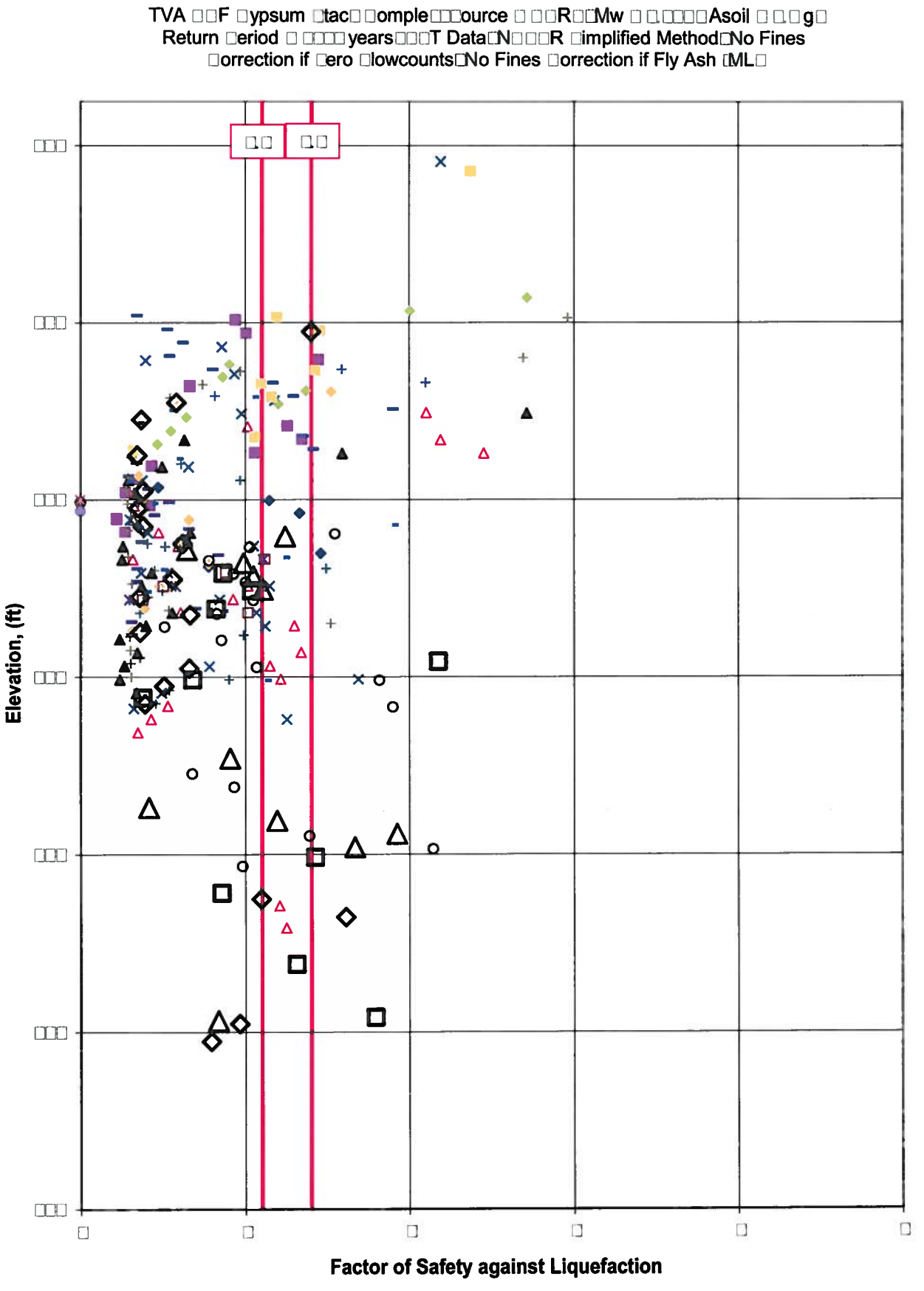


Figure 2: Horizontal Target 2500-yr UHRS (5% Damping) for the Cumberland Fossil Plant site

Acceleration versus depth profile at Boring CUF-H-2B (CUF Gypsum Stack, Section H). Results are derived from one-dimensional ground response analysis.




TVA □□F □ypsum □tac□ □omple□□□ource □ □□R□□Mw □ □.□□□□Asoil □ □.□g□
Return □eriod □ □□□□years□□□T Data□N□□□R □implified Method□No Fines
□orrection if □ero □lowcounts□No Fines □orrection if Fly Ash □ML□
□.□
□.□
Elevation, (ft)
Factor of Safety against Liquefaction

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

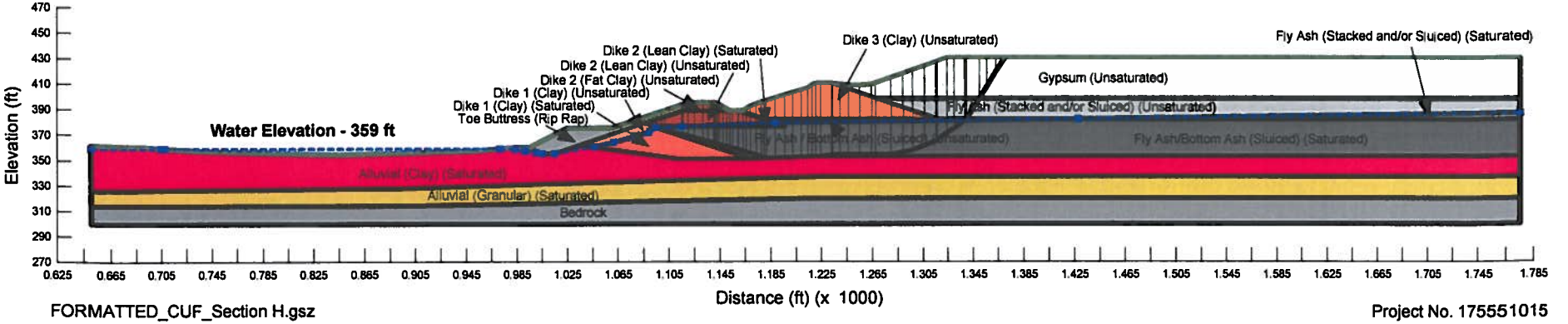
Existing Conditions - Post Earthquake

Liquefied Materials: Fly Ash/Bottom Ash (Sluiced)

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
Alluvial (Clay) (Saturated)	121 pcf	360 psf	16.2 °
Alluvial (Granular) (Saturated)	130 pcf	80 psf	16.2 °
Dike 3 (Clay) (Unsaturated)	126 pcf	1000 psf	25 °
Dike 2 (Fat Clay) (Unsaturated)	127 pcf	200 psf	18 °
Dike 2 (Lean Clay) (Unsaturated)	128 pcf	500 psf	21 °
Dike 2 (Lean Clay) (Saturated)	128 pcf	400 psf	17.1 °
Dike 1 (Clay) (Unsaturated)	124 pcf	800 psf	20 °
Dike 1 (Clay) (Saturated)	124 pcf	640 psf	16.2 °
Fly Ash / Bottom Ash (Sluiced) (Unsaturated)	100 pcf	140 psf	11 °
Fly Ash/Bottom Ash (Sluiced) (Saturated)	100 pcf	Sr=exp(-8.444+0.109N1(60)+5.379σ'^0.1), N1(60)=12	0 °
Fly Ash (Stacked and/or Sluiced) (Unsaturated)	100 pcf	0 psf	32 °
Fly Ash (Stacked and/or Sluiced) (Saturated)	100 pcf	0 psf	26.6 °
Gypsum (Unsaturated)	105 pcf	0 psf	33 °
Toe Buttress (Rip Rap)	140 pcf	0 psf	38 °

Factor of Safety: 1.1



FORMATTED_CUF_Section H.gsz

Project No. 175551015

Erosion Repair and Perimeter Ditch Improvements from CUF Work Plan 11

Perimeter Ditch Improvements from CUF Work Plan 11

Project added through Newport News Comp. Terrorism Plan proposal 1-22-2009 RLEWIS

2/9/2009 --from Steve Shapiro:
These activities were performed as a subconsultant to Iteris on VDOT's Statewide Systems Operations/ITS On-Call Services Project (#103-CS)
Dewberry Project No: 50015108
The project is "ongoing"
The Subcontract value to date is approximately $50,000.

APPENDIX C

Document 23

TVA – Comments on Dry Fly Ash Stack, December 20, 2012 (Geocomp Report November 20, 2012)

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Louisville KY 40223-2177
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October 16, 2012 let_010_175551015_rev_0

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

Re: Response to Recommendations
USEPA CCR Impoundment Assessment DRAFT Report
Cumberland Fossil Plant (CUF)
Cumberland City, Tennessee

Dear Mr. Kammeyer:

As requested, Stantec has reviewed the report *Coal Combustion Residue Impoundment Dam Assessment Report, Cumberland Fossil Plant, Tennessee Valley Authority, Cumberland City, Tennessee,* dated August 2012 prepared by Dewberry and Davis, LLC (Dewberry) for the United States Environmental Protection Agency (USEPA). The purpose of this letter is to address Dewberry's conclusions and recommendations pertaining to structural stability, hydrologic/hydraulic capacity, and technical documentation; and to provide additional supporting information relative to ongoing plant improvements, further analysis, and planned activities where applicable. Dewberry's recommendations and Stantec's corresponding responses are listed below. Please note that additional seismic analysis is being conducted for the Dry Fly Ash Stack; therefore, the corresponding Dewberry recommendations for that facility will be addressed under a future submittal.

Dewberry Report Section 1.2.1 1) – Ash Pond: *Install Stantec's recommended remedial measures for increasing the factor of safety against piping failure to the acceptable margin. If the driven sheet-pile wall is selected as the remedial measure, close attention should be paid to sheet-wall alignment location and depth to achieve maximum benefit in lengthening the seepage path to reduce exit gradients; the sheet-wall alignment should generally be at or upstream of the centerline of the dike crest.*

Stantec Response: Since the time of Dewberry's assessment, the operating pool level for this pond has been lowered from El. 384.2 to El. 378.0 (6.2 feet). Stantec has revised the seepage analysis for Sections P, Q and R based on the lowered pool level and recent piezometer data. Piping factors of safety (FS) have increased to 3.5, 3.5 and 4.0 for Sections P, Q and R, respectively (see attached results). Piping FS's for these sections are now satisfactory because they are greater than the Cedergren FS criterion of 2.5 to 3 (that is

referenced in USCACE's EM 1110-2-1901, *Engineering and Design-Seepage Analysis and Control for Dams*). No remedial measures are deemed necessary.

Dewberry Report Section 1.2.1 2) – Gypsum Disposal Area: *Install the planned lined ponds in the Gypsum Disposal Area as soon as possible for receiving and settling the gypsum slurry that must be sluiced to the Gypsum Disposal Area whenever the dewatering facility has an outage. Reevaluate the piping potential factor of safety after the lined ponds have been in place for about a year, to check whether or not the elimination of sluice water in the gypsum stack reduces the seepage exit gradients sufficiently to result in acceptable factors of safety against piping. Closely monitor the seepage conditions at the critical section in the interim. If the seepage exit gradients have not sufficiently abated, develop and implement a remedial measure to lower the exit gradients and achieve acceptable factor of safety against piping failure.*

Stantec Response: The FML-Lined Gypsum Settling Channels project is currently under construction and is scheduled for completion in April 2013. Sluicing to the stack has been discontinued and has not occurred for about 3 years. Since sluicing has stopped, piezometer levels in the gypsum have lowered. Stantec has revised the seepage analysis for Section H and the current piping FS is 3.1 (see attached results) which is satisfactory. TVA will continue to collect piezometer data and ensure that an acceptable factor of safety against piping is maintained.

Dewberry Report Section 1.2.3 1) – Gypsum Disposal Area: *Perform a quantitative liquefaction analysis of embankment sections overlying very loose/ loose saturated fly ash at the Dry Fly Ash Stack and the Gypsum Disposal Area; evaluate the impact of liquefaction on the containment dikes, if liquefaction is indicated; and evaluate the consequences of liquefaction failure of the containment dikes.*

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 H was analyzed for the Gypsum Disposal Area. The saturated ash materials are anticipated to undergo liquefaction for the 2,500-year earthquake.

Dewberry Report Section 1.2.3 2) – Gypsum Disposal Area: *If it is determined that liquefaction will not occur, review/investigate any soft or very soft clays in the lower part of the dike embankments and in the alluvial foundation beneath the embankments. If significant soft/very soft clay deposits are indicated (e.g., 10 feet or more in thickness and continuous for 100 feet or more), analyze their deformation potential during the design earthquake, and assess the impact of any such deformations on the stability of the embankments.*

Stantec Response: As noted in the previous response, Stantec's analysis indicates that liquefaction will occur for saturated ash materials under the 2,500-year earthquake; therefore the deformation analysis described by Dewberry in the above recommendation is not necessary.

Dewberry Report Section 1.2.3 3) – Gypsum Disposal Area: *Review the basis and reasoning for the "design" seismic coefficient used in the pseudostatic slope stability analysis, rerun the analysis if a modification appears appropriate, or perform a higher level of analysis that uses more sophisticated methods. (Note: If a deformation analysis is done, there may be no need for the pseudostatic analysis. However, a post-earthquake static slope stability analysis using reduced shear strengths would be appropriate.)*

Stantec Response: A higher level of analysis that uses more sophisticated methods was performed for Gypsum Disposal Area Section H. A description of the methodology and the results (slope stability cross sections, including table of material parameters) are attached. The results indicate that Section H has a factor of safety of 1.1 for post-earthquake stability using reduced shear strengths, which is satisfactory.

Dewberry Report Section 1.2.1 3) – Gypsum Disposal Area: *Depending on the results of additional seismic stability analyses and of liquefaction potential analyses recommended in Subsection 1.2.3, develop and implement measures to ensure adequate performance of the Dry Fly Ash Stack and the Gypsum Disposal Area containment dikes under the 2,500-year seismic event.*

Stantec Response: The results of the liquefaction and post-earthquake stability analyses indicate that the Gypsum Disposal Area will remain stable and display adequate performance due to the 2,500-year earthquake. Therefore, no seismic-related remedial measures are required.

Dewberry Report Section 1.2.4 – Maintenance Items: No significant problems were observed in the field assessment that would require special attention outside of routine maintenance. The minor issues observed, mostly small eroded areas or areas of seepage and poor drainage, should be addressed by TVA's routine maintenance activities. These include: 1) Repair minor erosion at various locations, 2) Continue to mow/ maintain vegetation along slopes, 3) Continue to monitor and document known seepage per seepage action plan, 4) Provide positive slope to promote drainage into perimeter ditch.

Stantec Response:

1) TVA repaired the erosion as part of Work Plan 11 (see attached photos).
2) TVA's Routine Handling Operations and Maintenance (RHO&M) group will continue vegetation maintenance along slopes.
3) TVA will continue to monitor areas of known seepage in accordance with the Seepage Action Plan.
4) TVA re-graded the perimeter drainage ditch as part of Work Plan 11 (see attached photos).

Summary:

Based on the results of Dewberry's report, and on the responses/additional analyses provided herein, it is Stantec's opinion that the final rating for the CUF Ash Pond and Gypsum Disposal Area should be upgraded to Satisfactory.

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

Sincerely,

STANTEC CONSULTING SERVICES INC.

Stephen H. Bickel, PE
Senior Principal

Randy L. Roberts, PE
Principal

/db

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

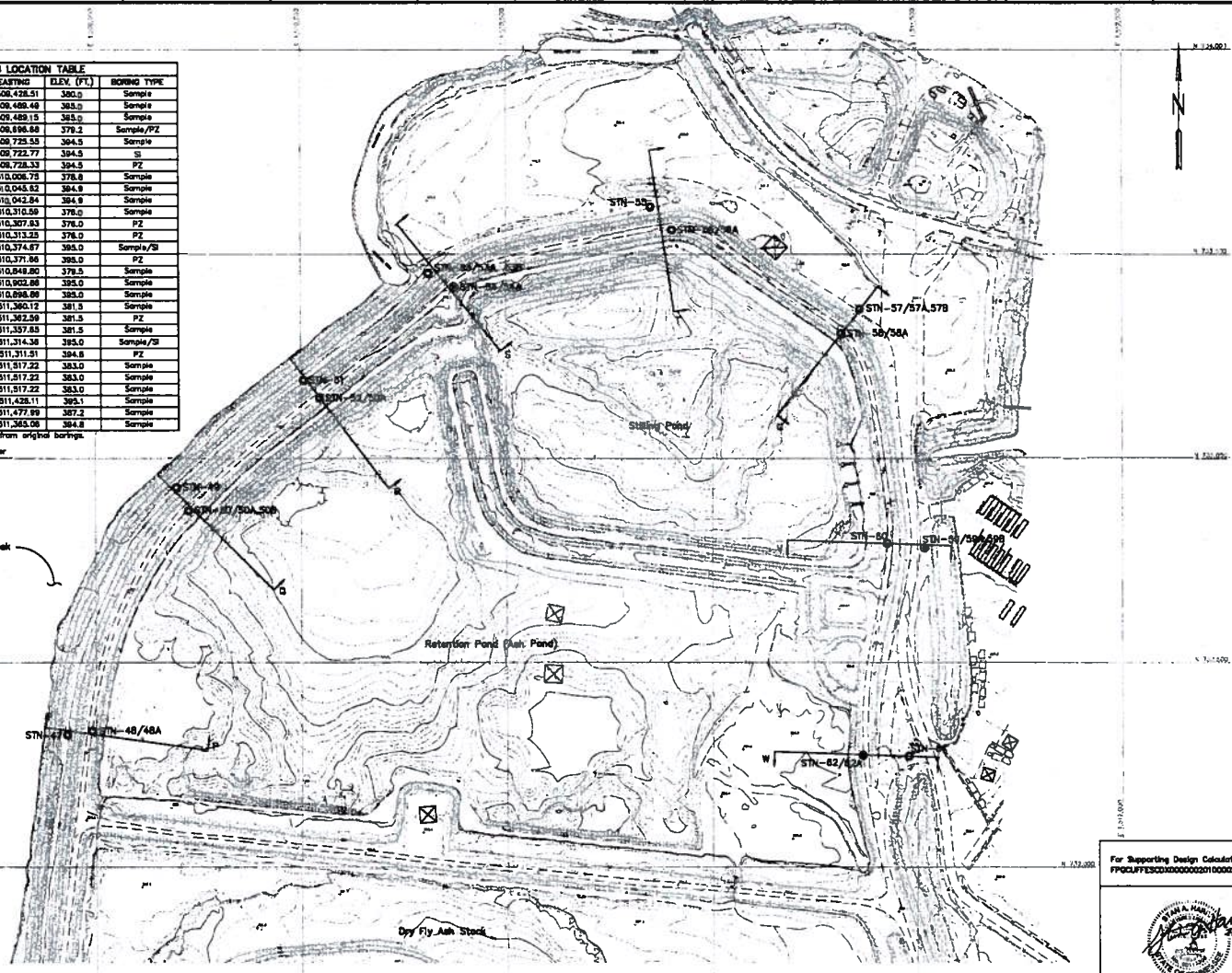
Attachments

46 | C | 10W544-01

BORING LOCATION TABLE

BORING	NORTHING	EASTING	ELEV. (FT.)	BORING TYPE
STN-47	732,334.14	1,508,428.51	380.0	Sample
STN-48	732,333.24	1,508,489.46	385.0	Sample
*STN-48A	732,329.25	1,508,489.15	385.0	Sample
STN-49	732,928.64	1,508,698.68	379.2	Sample/PZ
STN-50	732,872.44	1,508,725.55	394.5	Sample
*STN-50A	732,869.58	1,508,722.77	394.5	SI
*STN-50B	732,875.32	1,508,728.33	394.5	PZ
STN-51	733,191.78	1,510,006.75	378.8	Sample
STN-52	733,149.40	1,510,045.82	384.9	Sample
*STN-52A	733,148.52	1,510,042.84	384.9	Sample
STN-53	733,453.67	1,510,310.59	378.0	Sample
*STN-53A	733,456.66	1,510,307.53	378.0	PZ
*STN-53B	733,450.68	1,510,313.25	378.0	PZ
STN-54	733,419.93	1,510,374.67	395.0	Sample/SI
*STN-54A	733,417.30	1,510,371.86	395.0	PZ
STN-55	733,614.54	1,510,848.80	379.5	Sample
STN-56	733,560.12	1,510,902.88	395.0	Sample
*STN-56A	733,560.12	1,510,898.88	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	733,362.56	1,511,357.85	381.5	Sample
STN-58	733,305.89	1,511,314.38	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	1,511,383.08	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 NGVD29 using State Plane Tennessee coordinate system. The site photography was performed on 4/17/2009.

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 FPGCUFFESCOXX00000020100002

REV.	DATE	DSN	CK	DRN	SAH	SAH	SAH	TJ
R 0	02/24/10	DSR	CW	DRM	SAH	SAH	SAH	TJ

RECORD DRAWING

SCALE: 1"=100' EXCEPT AS NOTED

YARD
RETENTION AND STILLING PONDS

GEOTECHNICAL EXPLORATION
BORING LAYOUT

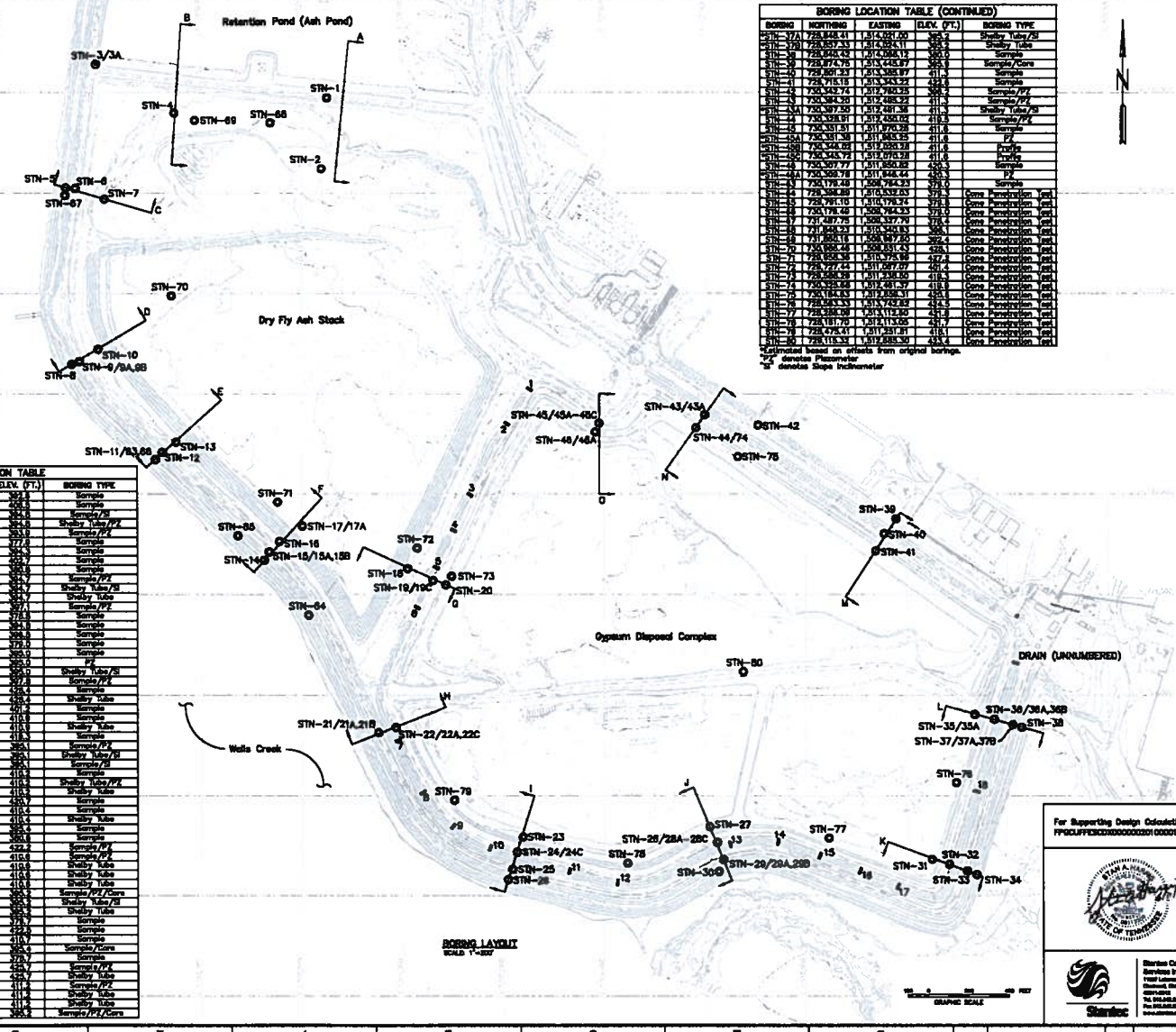
CUMBERLAND FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

AUTOCAD R 2000 | 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 LOCATION TABLE (CONTINUED)				
BORING	NORTHING	EASTING	ELEV. (FT.)	BORING TYPE
STN-37A	729,838.41	1,514,621.00	395.2	Shelby Tube/S
STN-37B	729,837.53	1,514,624.11	395.2	Shelby Tube
STN-38	729,840.43	1,514,588.12	395.0	Sample
STN-39	729,874.75	1,513,445.87	395.5	Sample/Core
STN-40	729,901.23	1,513,385.97	411.3	Sample
STN-41	729,715.15	1,513,343.22	424.6	Sample
STN-42	730,342.74	1,512,782.25	396.2	Sample/PZ
STN-43	730,384.20	1,512,485.22	411.3	Sample/PZ
STN-43A	730,397.50	1,512,481.38	411.3	Shelby Tube/S
STN-44	730,328.91	1,512,430.02	419.3	Sample/PZ
STN-45	730,331.51	1,511,970.28	411.6	Sample
STN-45A	730,331.38	1,511,963.95	411.6	PZ
STN-45B	730,344.02	1,511,958.58	411.6	Piezo
STN-45C	730,345.72	1,511,970.28	411.6	Piezo
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-47	730,179.49	1,508,784.23	378.0	Sample
STN-48	729,384.89	1,510,532.03	378.3	Cone Penetration test
STN-53	729,791.10	1,510,179.34	378.8	Cone Penetration test
STN-54	730,179.49	1,508,784.23	378.0	Cone Penetration test
STN-57	731,487.75	1,508,327.70	378.4	Cone Penetration test
STN-58	731,458.23	1,510,520.83	395.1	Cone Penetration test
STN-59	731,300.16	1,508,897.20	392.4	Cone Penetration test
STN-70	730,985.46	1,509,931.43	393.1	Cone Penetration test
STN-71	729,958.38	1,510,375.98	397.8	Cone Penetration test
STN-72	729,727.44	1,511,097.97	401.4	Cone Penetration test
STN-73	729,636.39	1,511,230.10	410.3	Cone Penetration test
STN-74	730,326.64	1,512,481.37	418.1	Cone Penetration test
STN-75	730,184.83	1,512,836.31	424.1	Cone Penetration test
STN-76	729,543.33	1,513,732.62	434.5	Cone Penetration test
STN-77	729,535.08	1,513,112.85	419.8	Cone Penetration test
STN-78	729,181.70	1,512,113.05	421.7	Cone Penetration test
STN-79	729,475.41	1,511,251.91	418.1	Cone Penetration test
STN-80	729,115.33	1,512,885.30	434.4	Cone Penetration test

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

BORING LOCATION TABLE				
BORING	NORTHING	EASTING	ELEV. (FT.)	BORING TYPE
STN-1	731,872.88	1,510,824.03	392.3	Sample
STN-2	731,825.35	1,510,984.18	408.5	Sample
STN-3	731,930.74	1,508,978.38	384.8	Sample/PZ
STN-3A	731,930.74	1,508,974.38	384.8	Shelby Tube/PZ
STN-4	731,897.21	1,509,884.05	393.0	Sample/PZ
STN-5	731,928.34	1,508,330.98	377.8	Sample
STN-6	731,922.33	1,508,376.77	384.3	Sample
STN-7	731,888.84	1,508,431.58	384.7	Sample
STN-8	730,844.60	1,508,136.17	380.8	Sample
STN-9	730,834.91	1,508,188.48	384.7	Sample/PZ
STN-9A	730,835.58	1,508,188.58	384.7	Shelby Tube/S
STN-9B	730,833.13	1,508,184.84	384.7	Shelby Tube
STN-10	730,721.30	1,508,480.68	397.1	Sample/PZ
STN-11	730,174.02	1,508,771.83	373.8	Sample
STN-12	730,204.83	1,509,033.16	384.3	Sample
STN-13	730,237.83	1,509,173.48	384.3	Sample
STN-14	730,008.17	1,510,108.37	378.5	Sample
STN-15	729,723.31	1,510,333.89	365.0	Sample
STN-15A	729,713.11	1,510,331.13	365.0	PZ
STN-15B	729,713.31	1,510,338.25	365.0	Shelby Tube/S
STN-16	729,753.04	1,510,386.22	367.8	Sample/PZ
STN-17	729,833.13	1,510,484.97	439.4	Sample
STN-17A	729,842.83	1,510,484.58	439.4	Shelby Tube
STN-18	729,928.30	1,511,026.83	401.2	Sample
STN-19	729,567.20	1,511,144.57	410.8	Sample
STN-19C	729,563.24	1,511,144.48	410.8	Shelby Tube
STN-20	729,543.68	1,511,210.48	410.8	Sample
STN-21	729,611.38	1,510,675.38	365.1	Sample/PZ
STN-21A	729,608.63	1,510,677.34	365.1	Shelby Tube/S
STN-21B	729,604.50	1,510,676.50	365.1	Sample/PZ
STN-22	729,534.53	1,510,881.31	410.3	Sample
STN-22A	729,534.83	1,510,884.76	410.3	Shelby Tube/PZ
STN-22C	729,533.08	1,510,883.00	410.3	Shelby Tube
STN-23	729,391.47	1,511,500.83	430.7	Sample
STN-24	729,315.90	1,511,582.58	410.4	Sample
STN-24C	729,317.51	1,511,586.03	410.4	Shelby Tube
STN-25	729,130.72	1,511,832.43	395.4	Sample
STN-26	729,079.09	1,511,917.81	395.8	Sample
STN-27	729,143.83	1,512,816.28	422.2	Sample/PZ
STN-28	729,362.15	1,512,555.43	410.6	Sample/PZ
STN-28A	729,363.77	1,512,558.91	410.6	Shelby Tube
STN-28B	729,362.35	1,512,550.95	410.6	Shelby Tube
STN-28C	729,360.38	1,512,548.90	410.6	Shelby Tube
STN-29	729,179.37	1,512,347.14	395.5	Sample/PZ/Core
STN-29A	729,181.10	1,512,347.30	395.5	Shelby Tube/S
STN-29B	729,177.54	1,512,353.48	395.5	Shelby Tube
STN-30	729,118.93	1,512,534.48	378.7	Sample
STN-31	729,180.44	1,513,022.06	422.2	Sample
STN-32	729,159.37	1,513,107.58	410.7	Sample
STN-33	729,122.47	1,513,197.00	395.4	Sample/Core
STN-34	729,103.57	1,513,324.18	378.7	Sample
STN-35	729,353.76	1,513,333.70	425.7	Sample/PZ
STN-35A	729,355.62	1,513,332.70	425.7	Shelby Tube
STN-36	729,374.81	1,513,330.45	411.8	Sample/PZ
STN-36A	729,374.55	1,513,328.88	411.8	Shelby Tube
STN-36B	729,374.55	1,513,332.08	411.8	Shelby Tube
STN-37	729,835.00	1,514,622.47	395.2	Sample/PZ/Core

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 NGVD29 using State Plane Tennessee coordinate system. The site photography was performed on 4/17/2008.

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 FP0CUFFERCDX0000000201000001

STANTEC | 0
TASK COMPLETED BY: | REV NO.

REV.	DATE	DRWN	CHK	DSGN	SUPV	SUPV	SUPV	TA
0	05/17/10							

RECORD DRAWING

REV. NO.	DATE	DRAWN	CHECKED	DESIGNED	SUPERVISED	SUPERVISED	SUPERVISED	REVIEWED	APPROVED	REVIEWED
0										

SCALE: 1"=200' EXCEPT AS NOTED

YARD
DRY FLY ASH STACK AND GYPSUM DISPOSAL COMPLEX

GEOTECHNICAL EXPLORATION
BORING LAYOUT

Stantec Consulting Services Inc.
1100 Lakeview Rd.
Clearwood, Ohio
38312-6202
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www.stantec.com

CUMBERLAND FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

AUTOCAD R 2000 | 05/17/10 | 46 | C | 10W543-01 | R 0

PLOT FACTOR:XX
W_TVA

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46 C 10W543-01

SLOPE STABILITY ANALYSIS
Cumberland Fossil Plant - Fly Ash Stack
Tennessee Valley Authority (TVA)

File Name: Section P.gsz

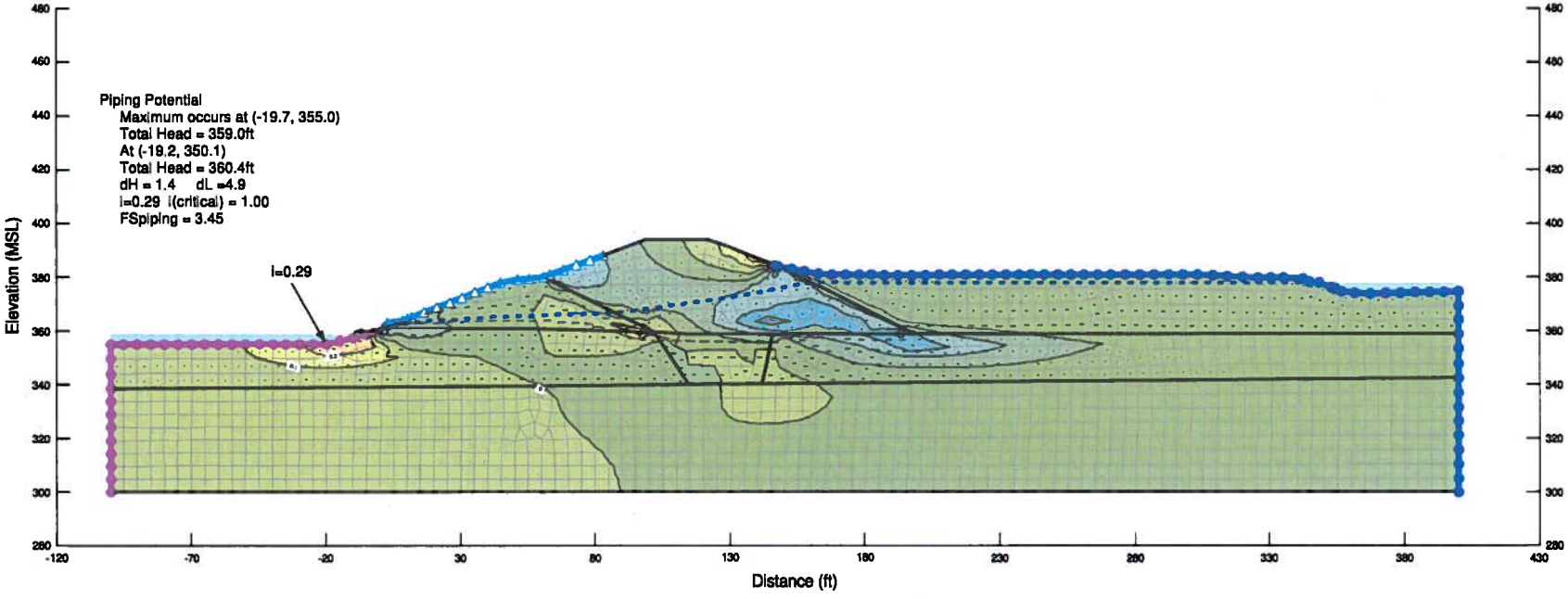
Analysis Name: Steady-State Seepage

Date Saved: 10/12/2012

Last Solved on 10/12/2012 at 1:03:58 PM

Stantec

Y-Gradient



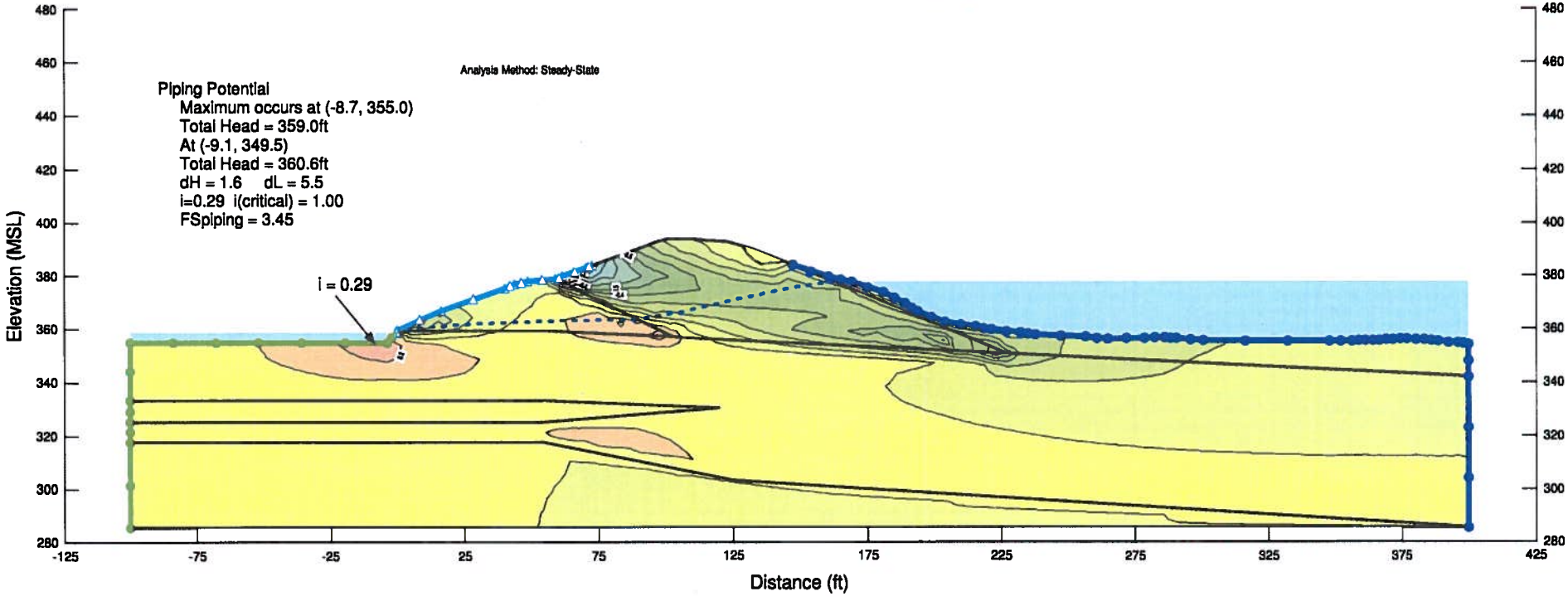
SLOPE STABILITY ANALYSIS
Cumberland Fossil Plant - Fly Ash Stack
Tennessee Valley Authority (TVA)

File Name: Section Q.gsz

Analysis Name: Steady-State Seepage
Last Solved on 9/20/2012 at 9:33:02 AM
Date Saved: 9/20/2012

Stantec

Y Gradient



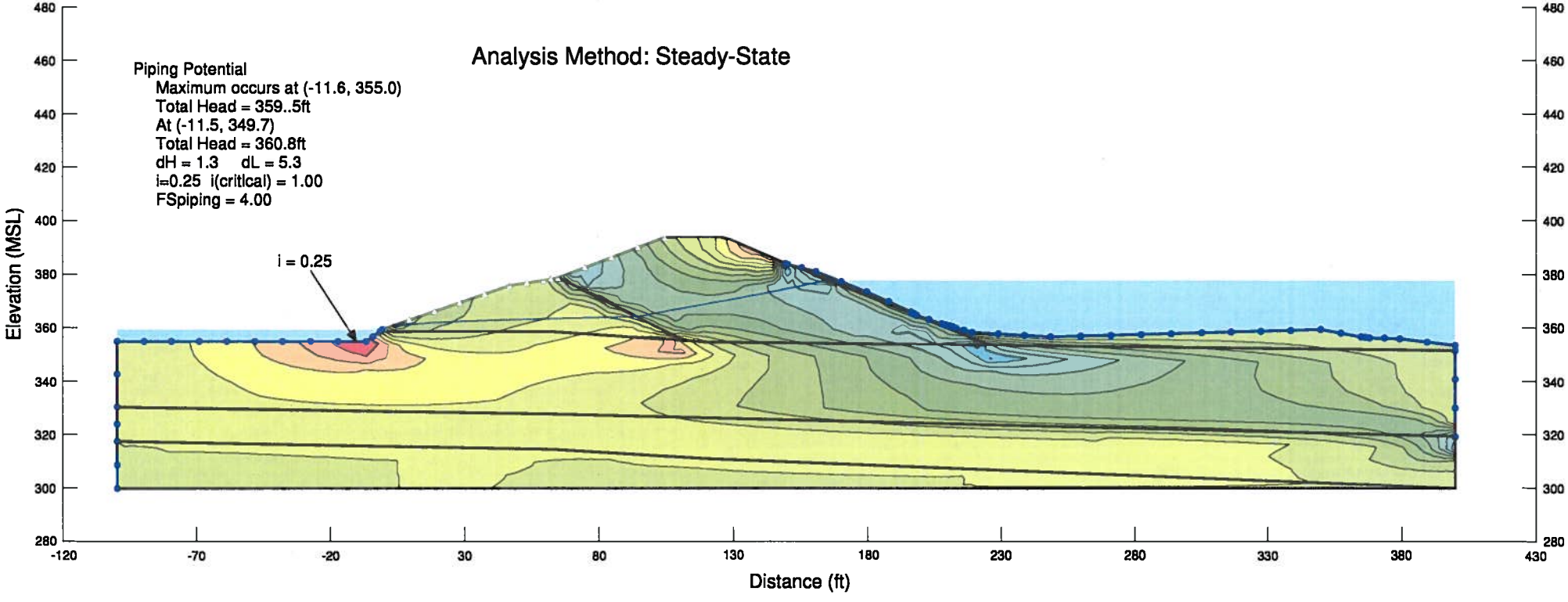
SLOPE STABILITY ANALYSIS
Cumberland Fossil Plant - Fly Ash Stack
Tennessee Valley Authority (TVA)

File Name: Section R.gsz

Analysis Name: Steady-State Seepage
Last Solved on 9/20/2012 at 9:49:06 AM
Date Saved: 9/20/2012

Stantec

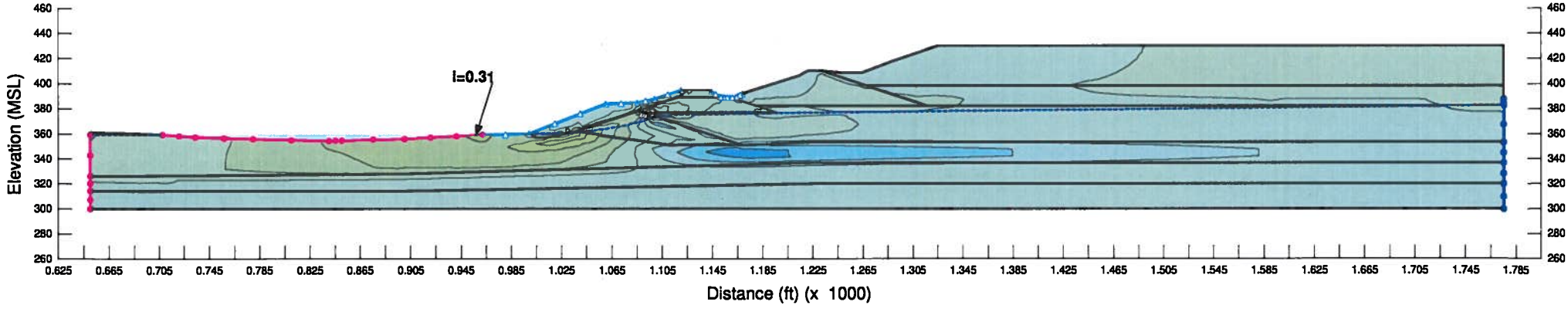
Y-Gradient



SLOPE STABILITY ANALYSIS
Cumberland Fossil Plant - Gypsum Stack Complex
Tennessee Valley Authority (TVA)

File Name: Section H (StabRepDgn)revised.gsz
Analysis Name: Steady-State Seepage
Date Saved: 10/12/2012
Last Solved on 10/12/2012 at 11:06:02 AM
Analysis Method: Steady-State

Piping Potential
Maximum occurs at (956.7, 358.7)
Total Head = 359.0ft
At (956.6, 353.9)
Total Head = 360.5ft
dH = 1.50 dL = 4.80
i=0.31 i(critical) = 0.97
FSpiping = 3.13



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.

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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

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TABLE 4

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

Comp.	Spectrally -Matched from	PGA (g)	PGV (cm/sec)	PGD (cm)	PGV/PGA (cm/sec/g)	PGA*PGD/PGV^2	Duration (sec)**
H	SER000	0.220	11.09	6.51	50.41	11.42	29.51
H	GRN180	0.221	13.89	8.59	62.85	9.65	24.55
H	SUL320	0.224	14.42	9.66	64.38	10.21	24.23
H	KSH-T1	0.222	13.24	8.68	59.64	10.78	15.25
H	TAP067-W	0.216	12.44	8.69	57.59	11.90	38.80
H	TAP075-W	0.219	11.01	7.92	50.27	14.04	23.74
H	TAP078-N	0.220	15.41	7.96	70.05	7.23	25.74

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

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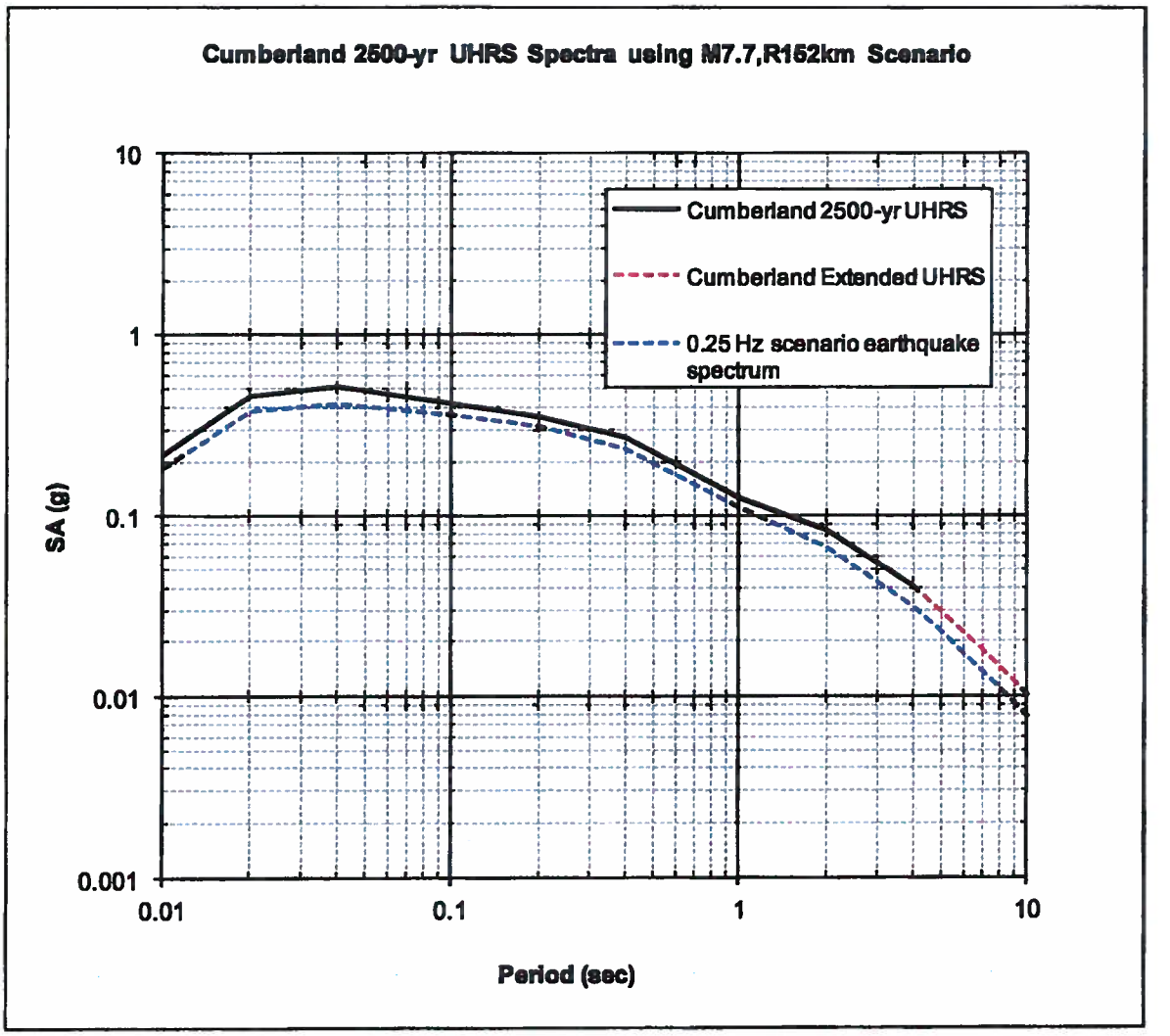
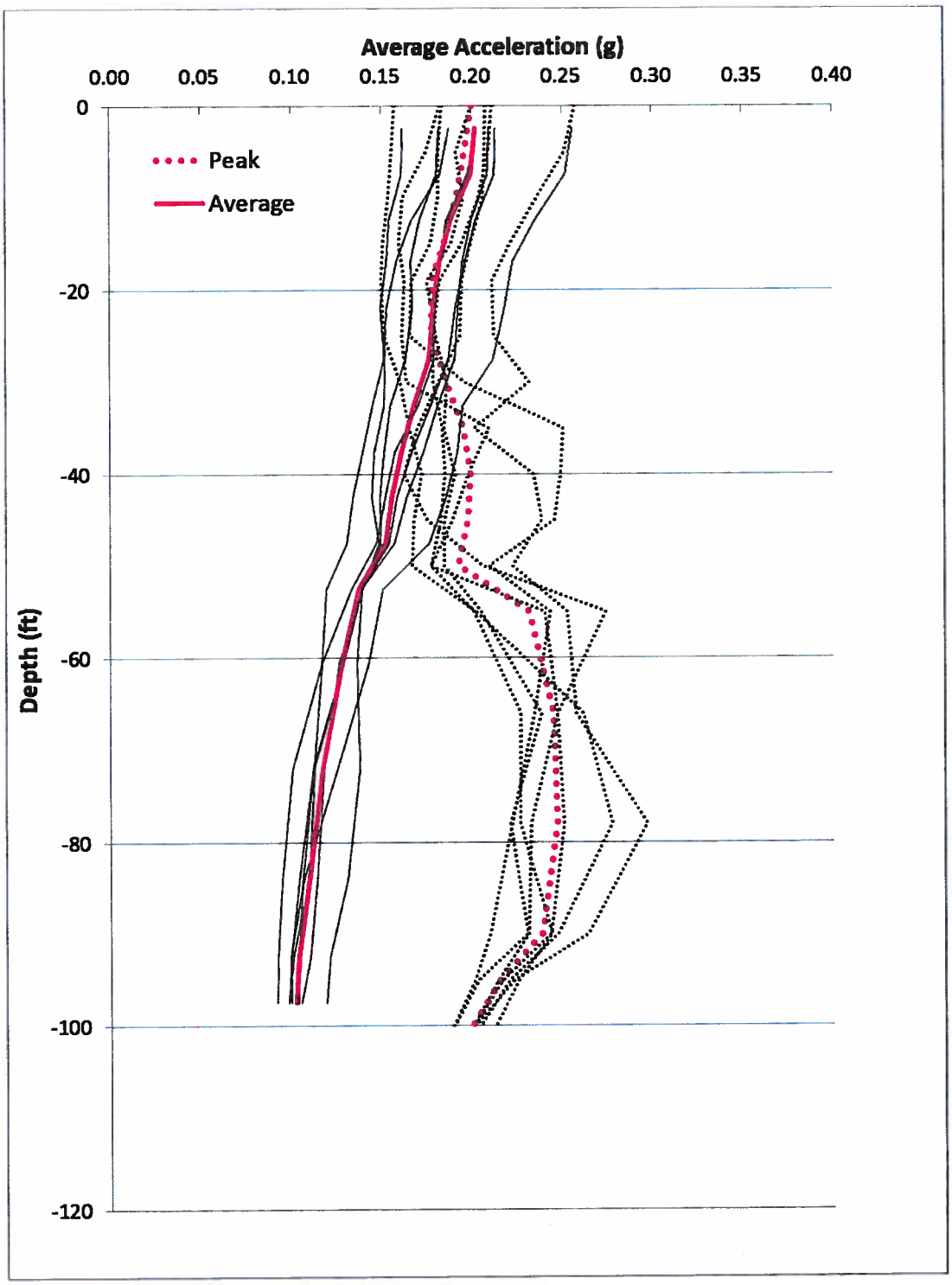
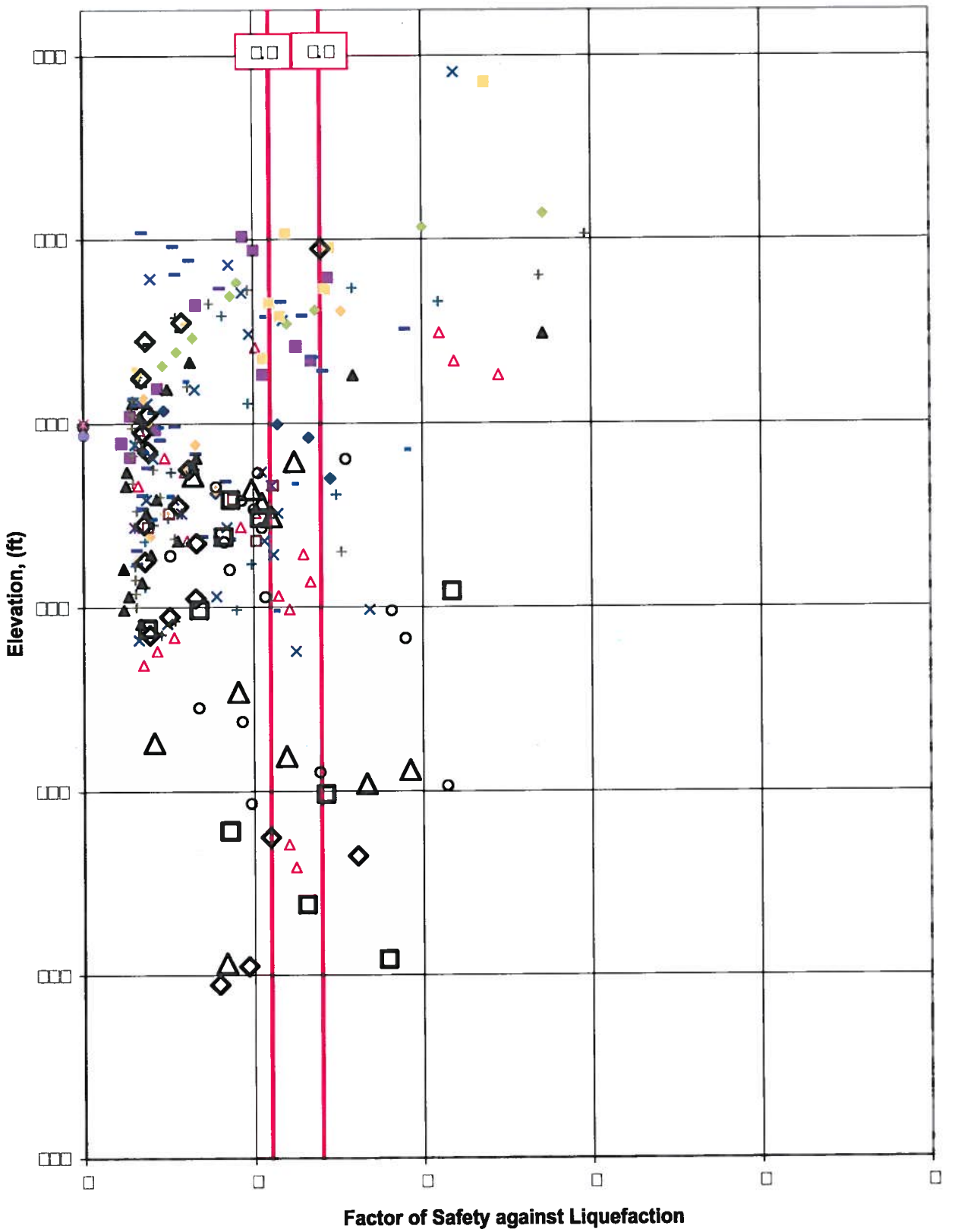


Figure 2: Horizontal Target 2500-yr UHRS (5% Damping) for the Cumberland Fossil Plant site

Acceleration versus depth profile at Boring CUF-H-2B (CUF Gypsum Stack, Section H). Results are derived from one-dimensional ground response analysis.



TVA □□F □ypsum □tac□ □omple□□□ource □ □□R□□Mw □ □.□□□□Asoil □ □.□g□
Return □eriod □ □□□□ years□□□T Data□N□□□R □implified Method□No Fines
□orrection if □ero □lowcounts□No Fines □orrection if Fly Ash □ML□



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

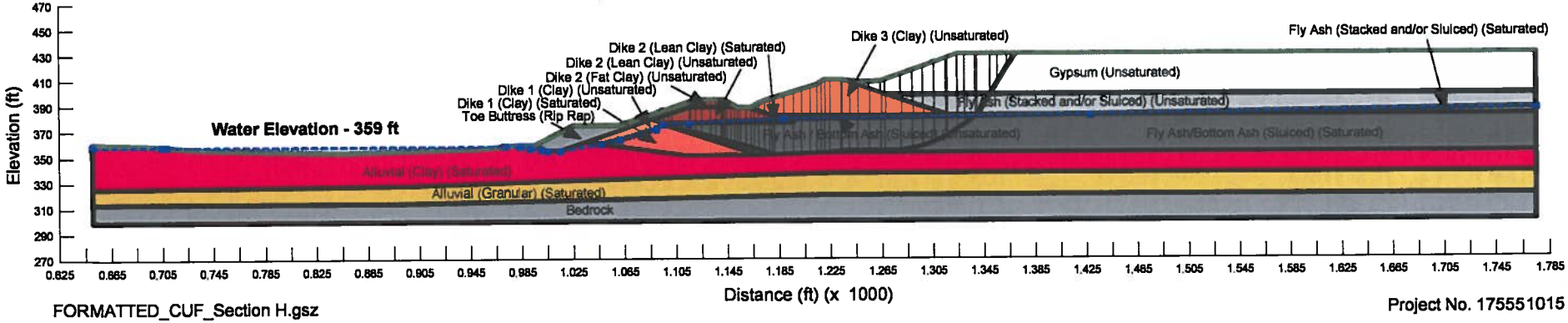
Existing Conditions - Post Earthquake

Liquefied Materials: Fly Ash/Bottom Ash (Sluiced)

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
Alluvial (Clay) (Saturated)	121 pcf	360 psf	16.2 °
Alluvial (Granular) (Saturated)	130 pcf	80 psf	16.2 °
Dike 3 (Clay) (Unsaturated)	126 pcf	1000 psf	25 °
Dike 2 (Fat Clay) (Unsaturated)	127 pcf	200 psf	18 °
Dike 2 (Lean Clay) (Unsaturated)	128 pcf	500 psf	21 °
Dike 2 (Lean Clay) (Saturated)	128 pcf	400 psf	17.1 °
Dike 1 (Clay) (Unsaturated)	124 pcf	800 psf	20 °
Dike 1 (Clay) (Saturated)	124 pcf	640 psf	16.2 °
Fly Ash / Bottom Ash (Sluiced) (Unsaturated)	100 pcf	140 psf	11 °
Fly Ash/Bottom Ash (Sluiced) (Saturated)	100 pcf	Sr=exp(-8.444+0.109N1(60)+5.379σ'^0.1), N1(60)=12	0 °
Fly Ash (Stacked and/or Sluiced) (Unsaturated)	100 pcf	0 psf	32 °
Fly Ash (Stacked and/or Sluiced) (Saturated)	100 pcf	0 psf	26.6 °
Gypsum (Unsaturated)	105 pcf	0 psf	33 °
Toe Buttress (Rip Rap)	140 pcf	0 psf	38 °

Factor of Safety: 1.1



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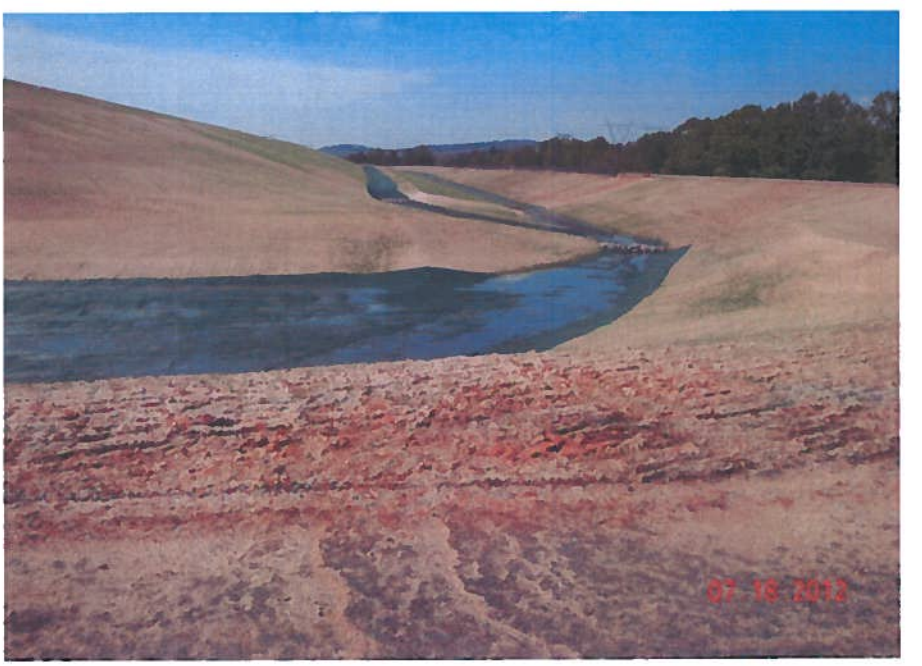
Project No. 175551015

Erosion Repair and Perimeter Ditch Improvements from CUF Work Plan 11



Perimeter Ditch Improvements from CUF Work Plan 11

APPENDIX C

Document 24

Draft Updated Seepage Analyses Gypsum Disposal Complex November 29, 2012

Stantec

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Instrument Installation and Updated Seepage Analyses

Ash Pond
Cumberland Fossil Plant
Cumberland City, Tennessee

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Prepared for:
Tennessee Valley Authority
Chattanooga, Tennessee

November 29, 2012

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November 29, 2012 rpt_001_175552005

Ms. Shannon Bennett
Tennessee Valley Authority
1101 Market Street
Chattanooga, Tennessee 37402

Re: Instrument Installation and Updated Seepage Analyses
Ash Pond
Cumberland Fossil Plant
Cumberland City, Tennessee

Dear Ms. Bennett:

As requested, Stantec Consulting Services Inc. (Stantec) has completed instrumentation installation and updating seepage analyses for the Ash Pond at the Cumberland Fossil Plant. The additional services are a result of the recommendations provided within the geotechnical report submitted on March 29, 2010 and included evaluation of the phreatic conditions within the dikes after completion of the improvements to the Ash Pond spillway that lowered the pond level. This report documents the additional instrumentation installed, the results of the piezometer monitoring, the results of updated seepage analyses, and additional recommendations for the facility. If you have any questions, or if we may be of further assistance, feel free to contact our office.

Sincerely,

STANTEC CONSULTING SERVICES INC.

Jason R. Curtsinger, PE
Project Engineer

Nathan A. Bader, PE
Senior Associate

/cdm

Enclosures

Instrument Installation and Updated Seepage Analyses
Ash Pond
Cumberland Fossil Plant
Cumberland City, Tennessee

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Appendix

Instrument Installation and Updated Seepage Analyses
Ash Pond
Cumberland Fossil Plant
Cumberland City, Tennessee

1. Introduction

Stantec Consulting Services Inc. (Stantec) performed a geotechnical exploration of the Ash Pond at the Cumberland Fossil Plant (CUF) in 2009 and the report was submitted on March 29, 2010. Geotechnical analyses performed as part of this study identified several areas with deficient factors of safety for piping including Ash Pond Sections P, Q, and R. Factors of safety from the previous analyses were less than 3 for each of these sections. The geotechnical report contained a number of recommendations for improving the identified deficiencies including construction of a toe berm along Wells Creek or construction of a sheet pile seepage cutoff wall. Since that time, spillway improvements have been completed to lower the pool level within the Ash Pond. The new pool elevation is approximately 378.2 feet and construction of the spillway improvements was completed in March 2012. As a result of the lower pool level, updated seepage analyses are required to determine if the lower pool has improved seepage conditions sufficiently such that further construction projects to improve piping factors of safety may not be required. The scope of this project includes updating the seepage analysis based on completed construction projects and recent instrumentation data.

Refer to the original Report of Geotechnical Exploration and Slope Stability Evaluation submitted by Stantec dated March 29, 2010 for additional background information, site geology, historical information, and results of drilling and laboratory testing previously performed.

2. Scope of Additional Exploration

The scope included further piezometer installation, updating seepage analyses after various improvements were completed around the Ash Pond, and the preparation of an addendum to the previous geotechnical report with updated recommendations. The following paragraphs summarize the scope of services for these tasks.

- Stantec installed two (2) additional piezometers (STN-100 and STN-101) and advanced one sample boring (STN-102) adjacent to Section P along the perimeter dike of the Ash Pond. The purpose of the additional piezometers was to establish more accurate phreatic levels following lowering of the Ash Pond pool level. Piezometer construction consisted of one-inch diameter Schedule 40 PVC, well screen (ten feet) and riser pipe. The annular backfill consisted of a sand filter pack to some distance above the screened interval followed by a bentonite seal. After allowing the bentonite to hydrate, the remaining annulus was backfilled with bentonite grout and auger cuttings. Stick-up protective covers and bollards were installed to protect the piezometers. During the advancement of all the borings (STN-100 to

STN-102), soil samples were obtained at selected intervals to confirm subsurface stratigraphy in the area of Section P.

- Site visits were made by Stantec personnel to review the perimeter dike for signs of seepage and measure stabilized water levels within the new piezometers.

- The results of the instrumentation monitoring and field inspection services were then used to revise cross-sections P, Q, and R seepage analyses with the lower Ash Pond pool level. At the conclusion of updating the analyses, this addendum to the original geotechnical report was prepared to provide more accurate seepage models and to provide updated conclusions and recommendations.

3. Results of Piezometer Installation and Subsurface Exploration

3.1. Summary of Supplemental Piezometers

Supplemental piezometers were installed adjacent to Ash Pond Section P to measure pore water pressures to provide a more accurate phreatic surface for the updated seepage analyses. Piezometer readings were taken after installation to determine the stabilized water level within the new piezometers. Refer to Appendix C for the supplemental piezometer installation details and readings (up to most recent set of readings). Supplemental piezometer locations and tip elevations are summarized in Table 1 below (all measurements are expressed in feet).

Table 1. Summary of Supplemental Piezometers

Boring No.	Ground Surface Elevation	Northing	Easting	Piezometer Tip Elevation
STN-100	395.0	732,555.67	1,509,526.03	316.5
STN-101	395.1	732,353.39	1,509,479.25	341.9

Refer to the original Report of Geotechnical Exploration and Slope Stability Evaluation submitted by Stantec dated March 29, 2010 for information regarding previously installed piezometers. For piezometer readings within the previously installed piezometers, refer to the monthly instrumentation letters submitted for CUF.

3.2. Supplemental Subsurface Investigation

During the advancement of STN-100 through 102, soil samples were obtained to confirm the stratigraphy near Section P. After the borings were advanced, the soil stratigraphy from the new borings was compared to the stratigraphy determined from the previous exploration to confirm subsurface features at Section P. The original section displayed an alluvial granular material that began beneath Dike 1 and extended east below the Ash Pond. This material layer extended from approximate elevation 359.0 feet to bedrock and was described as sand with gravel to cobbles with sand and gravel. The new borings advanced encountered alluvial clay visually classified as Lean Clay to Fat Clay within the same elevation range as the alluvial granular material from the original cross-section. The subsurface conditions observed correlate well with historical documentation that Wells Creek's channel previously

meandered along the perimeter dike of the Ash Pond. The creek channel was diverted to accommodate construction of the perimeter dike system of the Ash Pond. Therefore, the alluvial granular material that was encountered is likely the previous Wells Creek channel confined by clay material and not a continuous stratum that extends beneath the Ash Pond as depicted in the original cross-section.

During drilling activities, reviews of the perimeter dikes were also performed and signs of seepage were not observed.

The locations of the new borings and piezometers are depicted on the revised layout drawing in Appendix A. The boring logs are located in Appendix B.

4. Updated Engineering Analyses

4.1. General

The geotechnical engineering analyses performed during the original evaluation included evaluations of strength and permeability parameters, seepage analyses, and slope stability analyses. These previous analyses included several cross-sections and analyses were performed for static, long-term loading conditions. Sufficient stability factors of safety were obtained for each cross-section analyzed, however, the seepage analyses determined that three sections (Sections P, Q, and R) had piping factors of safety that were deficient. For this supplemental evaluation, only the seepage analyses were evaluated and re-analyzed. The new phreatic surface was approximated using the piezometer readings and observations in the field. The pool level within the Ash Pond was approximated at elevation 378.0 feet based on the new spillway design. For the updated analyses, the Ash Pond water level was assumed to exhibit steady state seepage conditions through the dike in the seepage models. Typically, the soil horizons and seepage parameters previously established were used in these updated analyses. Any differences between the original cross-sections and parameters are explained in the following paragraphs.

Results of the updated analyses are summarized in the following paragraphs and are shown on computer output provided in Appendix D.

4.2. Updated Seepage Analyses

Seepage analyses were updated to evaluate the Ash Pond dikes with the lowered pool level. The following paragraphs summarize the assumptions, parameters, etc. used in these updated seepage models.

4.2.1. SEEP/W Model

An analysis of long-term, steady state seepage through the Ash Pond perimeter dikes was performed to estimate the magnitude of seepage gradients for the evaluation of potential piping. The numerical seepage models were developed using SEEP/W 2007 (Version 7.19), a finite element code tailored for modeling groundwater seepage in soil and rock. SEEP/W is distributed by GEO-SLOPE International, Ltd., of Calgary, Alberta, Canada (www.geo-slope.com).

Three dike cross-sections through the Ash Pond were updated with SEEP/W for the long-term, steady state loading condition with the lower pool level. For the numerical analysis,

each cross-section was subdivided into a mesh of elements, consisting of first-order quadrilateral and triangular finite elements. For seepage problems, where the primary unknown (hydraulic head) is a scalar quantity, first-order elements provide for efficient, effective modeling. Given appropriate hydraulic conductivity properties and applied boundary conditions, the finite element method (as implemented in the SEEP/W code) was then used to simulate steady seepage across the mesh. The total hydraulic head is computed at each nodal location, from which pore water pressures and seepage gradients can be determined.

Steady-state seepage was assumed for the updated long-term loading analysis, with the static pool level placed at approximate elevation 378.0 feet. The upstream boundary condition was updated to model the lowered Ash Pond pool level because of the spillway improvement project being completed. The other boundary conditions for the SEEP/W analysis were not changed from the original models. Refer to the original Report of Geotechnical Exploration and Slope Stability Evaluation for further information regarding the boundary conditions used.

The cross-section geometry and soil horizons of Ash Pond Section Q and Section R were not changed from the original Report of Geotechnical Exploration and Slope Stability Evaluation. However, soil stratigraphy encountered in the supplemental borings adjacent to Section P resulted in changes to the soil horizons within the seepage model. The original Section P displayed an alluvial granular material that began beneath Dike 1 and extended east below the Ash Pond. This granular layer extended from approximate elevation 359.0 feet to bedrock. The new borings advanced encountered alluvial clay within the same elevation range as the alluvial granular material from the original cross-section. The subsurface conditions observed correlate well with historical documentation that Wells Creek's channel previously meandered along the perimeter dike of the Ash Pond in this area. The alluvial granular material that was encountered is likely the previous Wells Creek channel confined by clay material and not a continuous stratum that stretches beneath the Ash Pond as depicted in the original cross-section. Therefore, the soil horizons were modified to reflect this change.

4.2.2. Updated Seepage Properties

For each modeled cross-section, the previous subsurface profile was used and the material seepage properties used were updated to reflect the actual phreatic surfaces within the dikes based on piezometer readings. Updated material properties used in the seepage analysis are summarized in Table 2.

Table 2. Updated Material Properties for SEEP/W Analysis

Soil Horizon	Saturated k_v (cm/s)	Ratio k_h / k_v	Specific Gravity G_s	Void Ratio e	Volumetric Water Content		Basis
					Residual	Saturated	
Clay Dike 1 - Lean Clay	6.50E-07	1 to 10	2.67	0.704	0.06	0.413	Available Laboratory Data and Correlation w/ Typical Values
Clay Dike 2 - Lean Clay	4.27E-08	1 to 10	2.71	0.540	0.08	0.351	Available Laboratory Data and Correlation w/ Typical Values
Fly Ash - Sluiced	8.41E-05	50	2.50	0.550	0.015	0.3548	Available Laboratory Data and Correlation w/ Typical Values
Alluvial – Clay	7.41E-08	20	2.67	0.667	0.07	0.401	Available Laboratory Data and Correlation w/ Typical Values
Alluvial – Granular	1.00E-04	20	2.68	0.370	0.02	0.27	Correlation w/ Typical Values
Bedrock	3.05E-11	10	N/A	N/A	0	0.05	Correlation w/ Typical Values

Note: SEEP/W requires input parameters k_h and ratio of k_v/k_h

For the updated long-term loading analyses, an iterative process of parametric calibration using actual phreatic levels measured was used to arrive at final estimates of the seepage properties. After the initial seepage parameters were estimated, results from the SEEP/W model were compared to observations and measured water levels. Nodes were placed in the model at the screened piezometer intervals, so that the average head across these nodes could be compared to the corresponding piezometer reading. The material properties in each modeled cross-section were then varied until a reasonable match was obtained between the seepage predictions, actual conditions, and observations. Specifically, the kh/kv ratios were adjusted (while still maintaining the parameters within expected ranges) to give model predictions as consistent as possible with the water level measurements and observations made. The comparison between the field piezometer measurements and final SEEP/W predictions show the predicted groundwater table through the dikes mostly within the range of about four feet below to two foot above the readings obtained in the piezometers installed within the dikes.

4.2.3. Results of Updated Seepage Analyses

Plots from the updated SEEP/W analyses of the three cross-sections through the Ash Pond dikes are presented in Appendix D. The plots show the finite element mesh, material zones, and boundary conditions used in each analysis. The results are depicted in contour plots of total head, pore water pressure, and seepage gradients. The seepage gradients were assessed for maximum exit gradients and the potential for soil piping.

On each modeled cross-section, examination of the output (predicted phreatic surface and vertical gradients) can be made to look for areas where the potential for excessive vertical gradients might exist that could possibly initiate the erosion or piping of material. In general, areas of potential concern are where water seeps laterally out onto a sloping ground surface, or where vertical, upward seepage occurs at the ground surface. First, contour plots of vertical gradient were examined to determine the general location of the maximum vertical exit gradient. On the modeled cross-sections, the maximum upward gradient occurs near the toe of the dikes at each of the sections analyzed. For the factor of safety calculations, vertical gradients from these locations were then used along with the critical gradients determined from the soil properties.

The calculated factors of safety against piping are summarized in Table 3. They range from 4.0 to 7.7. Stantec recommends a target factor of safety against piping of four, based on information contained in United States Army Corps of Engineers (USACE) manual EM 1110-2-1901. Hence, all of the three cross sections modeled for the updated analyses meet or exceed the recommended design criteria for piping at the critical seepage exit points located at the dike toes.

Table 3. Summary of Computed Exit Gradients and Minimum Factors of Safety Against Piping for Updated Analyses

Cross Section*	Vertical Gradient (i_v) at Critical Exit Point	Location of Critical Exit Point	Material	Critical Gradient (i_{crit})	FS_{piping}
P	0.25	Toe	Alluvial Clay	1.00	4.0
Q	0.13	Toe	Alluvial Clay	1.00	7.7
R	0.25	Toe	Dike 1 Lean Clay	1.00	4.0

*Refer to Appendix A for locations of cross-sections.

5. Conclusions and Recommendations

The supplemental conclusions and recommendations that follow are based on the results of the supplemental analyses and monitoring performed. In addition, information evaluated during the original report regarding the history of the Ash Pond and data obtained during the original geotechnical exploration were also used as a basis for these supplemental recommendations.

5.1. The results of the updated seepage analyses following lowering of the pool level were reviewed to identify conditions where seepage and possible piping may occur. The models

predict the potential for seepage outbreaks to occur along the toe of the dikes. Factors of safety against piping in these areas are all equal to or greater than the recommended target value of 4. Although factors of safety against piping are all sufficient at the critical locations, the potential for seepage still exists along the toe of the dikes in the areas adjacent to Sections P, Q, and R.

Based on the higher factors of safety against piping now that the pool within the Ash Pond is lowered, the mitigation design previously recommended including construction of a toe berm along Wells Creek or construction of a sheet pile seepage cutoff wall are no longer judged to be required. However, continued monitoring of the dikes for seepage should occur until the Ash Pond is taken out of service and closed in accordance with the Seepage Action Plan for CUF.

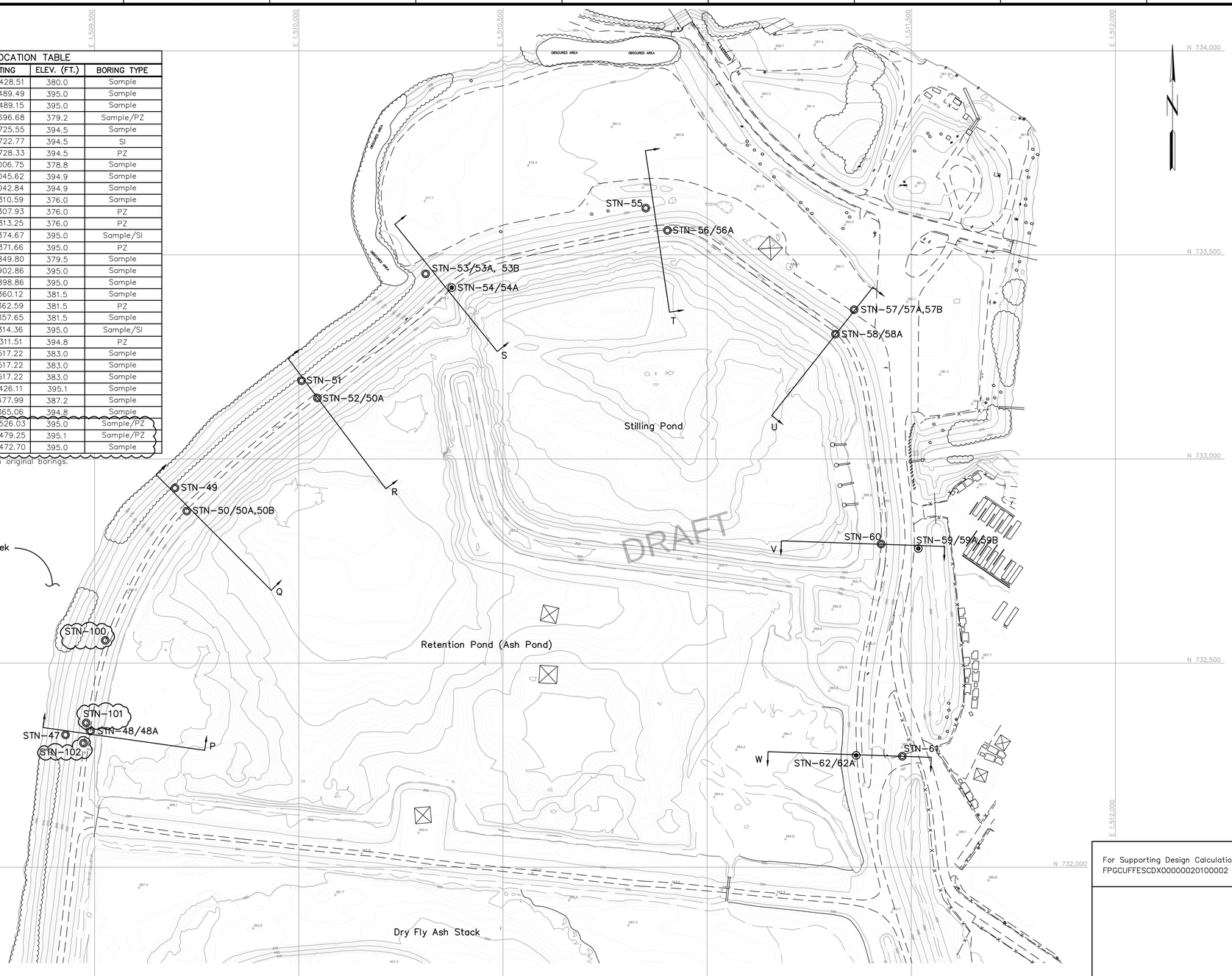
5.2. All other conclusions, recommendations, references, and limitations of study provided in the original Report of Geotechnical Exploration and Slope Stability Evaluation submitted by Stantec and dated March 29, 2010 remain applicable. Refer to this original report for additional information.

Appendix A

Boring Layout

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
STN-100	732,555.67	1,509,526.03	395.0	Sample/PZ
STN-101	732,353.39	1,509,479.25	395.1	Sample/PZ
*STN-102	732,303.82	1,509,472.70	395.0	Sample

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



BORING LAYOUT
SCALE: 1"=100'

50 0 100 200 FEET
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:

1. The topographic mapping provided is based on horizontal datum NAD27 and vertical datum NGVD29 using State Plane Tennessee coordinate system. The site photography was performed on 4/17/2009.
2. Clouded areas represent additional borings performed and piezometers installed to obtain water level and stratigraphy data to supplement the previous Phase 2 Geotechnical Exploration Seepage Analyses for the Ash Pond.

RECORD DRAWING

For Supporting Design Calculations see FPGCUFFESCDX00000020100002

REV. NO.	DATE	DSGN	DRWN	CHKD	SUPV	RVWD	APPD	ISSD	PROJECT ID	AS CONST	REV. CD
R 1	11/XX/12	JRC	RRP	JRC	NAB	SEB	MST	JCK	–	–	–
ADDITIONAL PIEZOMETERS											
R 0	03/29/10	DBR	CW	DBR	SAH	SAH	SAH	TJ	–	–	–
RECORD DRAWING											

DISCIPLINE INTERFACE

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:
J.R. CURTSINGER	R.R. PETTY	J.R. CURTSINGER	N.A. BADER	S.E. BENNETT	M.S. TURNBOW	J.C. KAMMEYER

Stantec Consulting Services Inc.
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Tel. 502.212.5000
Fax 502.212.5055
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 1

PLOT DATE: 11/05/2012 USER: PETTY, RICHARD
V:\1755\Active\175552005\GEOTECHNICAL\DRAWING\SHEET_FILES\10W544-01-RI.DWG

STANTEC | 1
TASK COMPLETED BY: | REV NO.

PLOT FACTOR:100
W_TVA

C.A.D. DRAWING
DO NOT ALTER MANUALLY

Appendix B

Supplemental Boring Logs

SUBSURFACE LOG

Project Number	175552005		Location	N=732555.67 ft, E=1509526.03 ft	
Project Name	CUF PZ Installation		Boring No.	**STN-100**	Total Depth 78.6 ft
County	Stewart, TN		Surface Elevation	395.0 ft	
Project Type			Date Started	10/8/12	Completed 10/8/12
Supervisor	D. Rogers	Driller M. Wethington	Depth to Water	18.0 ft	Date/Time 10/8/12
Logged By	M. Jones		Depth to Water	N/A	Date/Time N/A

Lithology Elevation	Lithology Depth	Description	Overburden / Rock Core	Sample # / RQD	Depth / Run	Rec. Ft. / Rec. Ft.	Blows / Rec. %	Mois.Cont. % / Run Depth	Remarks
395.0	0.0	Top of Hole							
		Lean Clay With Gravel, reddish brown, moist, very stiff		SPT-1	5.0 - 6.5	0.8	7-18-10	--	
				SPT-2	10.0 - 11.5	0.5	6-6-12	--	
380.0	15.0								
		Lean Clay, brown to grayish brown, moist, stiff		SPT-3	15.0 - 16.5	0.7	3-5-8	--	
377.0	18.0								
		Gravel With Sand, gray, wet, medium dense, trace clay		SPT-4	20.0 - 21.5	0.1	7-7-8	--	free water encountered during drilling at 18' Cobbles 18' to 23'
370.0	25.0								
		Lean Clay With Gravel, yellowish brown, wet, medium stiff, trace gravel		SPT-5	25.0 - 26.5	1.0	3-3-5	--	Cobbles 27' to 29.5
365.5	29.5								
		Lean Clay, brown to grayish brown, moist to wet, stiff		SPT-6	30.0 - 31.5	1.2	9-6-7	--	
360.0	35.0								
				SPT-7	35.0 - 36.5	1.2	4-4-4	--	

SUBSURFACE LOG

Project Number	175552005	Location	N=732555.67 ft, E=1509526.03 ft
Project Name	CUF PZ Installation	Boring No.	**STN-100** Total Depth 78.6 ft

Lithology Elevation	Lithology Depth	Description	Overburden / Rock Core	Sample # / RQD	Depth / Run	Rec. Ft. / Rec. Ft.	Blows / Rec. %	Mois.Cont. % / Run Depth	Remarks
355.0	40.0	Lean Clay, gray with yellowish brown, wet, medium stiff, trace organics *(Continued)*							
		Lean Clay, yellowish brown mottled gray, moist, very stiff		SPT-8	40.0 - 41.5	1.5	12-14-18	--	
350.0	45.0								
		Lean Clay With Sand, reddish brown, wet, stiff, trace gravel		SPT-9	45.0 - 46.5	0.3	10-6-6	--	
				SPT-10	50.0 - 51.5	1.5	10-17-24	--	
				SPT-11	55.0 - 56.5	1.5	8-14-14	--	
334.5	60.5								
		Fat Clay, reddish brown to greenish gray, moist, stiff to very stiff, sand lenses at 76.4'		SPT-12	60.0 - 61.5	1.5	8-14-17	--	
				SPT-13	65.0 - 66.5	1.5	7-10-15	--	
				SPT-14	70.0 - 71.5	1.0	7-6-5	--	
				SPT-15	75.0 - 76.5	1.5	3-4-6	--	

SUBSURFACE LOG

Project Number	175552005	Location	N=732555.67 ft, E=1509526.03 ft
Project Name	CUF PZ Installation	Boring No.	**STN-100** Total Depth 78.6 ft

Lithology			Overburden	Sample #	Depth	Rec. Ft.	Blows	Mois.Cont. %	
Elevation	Depth	Description	Rock Core	RQD	Run	Rec. Ft.	Rec. %	Run Depth	Remarks
316.4	78.6								

Auger Refusal /
Bottom of Hole

Installed piezometer in borehole
Top of Rock = 78.6
Elevation (316.4)

SUBSURFACE LOG

Project Number	175552005	Location	N=732353.39 ft, E=1509479.25 ft
Project Name	CUF PZ Installation	Boring No.	**STN-101** Total Depth 53.3 ft
County	Stewart, TN	Surface Elevation	395.1 ft
Project Type		Date Started	10/9/12 Completed 10/9/12
Supervisor	D. Rogers Driller M. Wethington	Depth to Water	22.0 ft Date/Time 10/9/12
Logged By	M. Jones	Depth to Water	N/A Date/Time N/A

Lithology Elevation	Lithology Depth	Description	Overburden / Rock Core	Sample # / RQD	Depth / Run	Rec. Ft. / Rec. Ft.	Blows / Rec. %	Mois.Cont. % / Run Depth	Remarks
395.1	0.0	Top of Hole							
		Fat Clay, reddish brown to yellowish brown, moist, very stiff, trace fine gravel							
				SPT-1	5.0 - 6.5	1.0	9-11-10	--	
				SPT-2	10.0 - 11.5	1.5	7-11-13	--	
				SPT-3	15.0 - 16.5	1.2	5-7-9	--	
				SPT-4	20.0 - 21.5	1.0	9-16-13	--	
									Free water encountered during drilling at 22'
369.6	25.5								
		Lean Clay, grayish brown to dark yellowish brown, wet, very stiff, trace sand and gravel		SPT-5	25.0 - 26.5	0.2	13-7-10	--	Cobbles 25.5' to 30'
				SPT-6	30.0 - 31.5	0.5	5-22-9	--	
				SPT-7	35.0 - 36.5	0.0	6-6-5	--	gravel blockage

SUBSURFACE LOG

Project Number	175552005	Location	N=732353.39 ft, E=1509479.25 ft
Project Name	CUF PZ Installation	Boring No.	**STN-101** Total Depth 53.3 ft

Lithology Elevation	Lithology Depth	Description	Overburden / Rock Core	Sample # / RQD	Depth / Run	Rec. Ft. / Rec. Ft.	Blows / Rec. %	Mois.Cont. % / Run Depth	Remarks
355.1	40.0								
		Fat Clay, yellowish brown with gray, moist, very stiff		SPT-8	40.0 - 41.5	1.5	12-11-13	--	
				SPT-9	45.0 - 46.5	0.0	12-11-12	--	gravel blockage
				SPT-10	50.0 - 51.5	1.1	10-15-23	--	
341.8	53.3								
		Auger Refusal / Bottom of Hole Installed piezometer in borehole Top of Rock = 53.3 Elevation (341.8)							

SUBSURFACE LOG

Project Number	175552005		Location	N=732303.82 ft E=1509472.90 ft	
Project Name	CUF PZ Installation		Boring No.	**STN-102**	Total Depth 46.5 ft
County	Stewart, TN		Surface Elevation	395.0 ft	
Project Type			Date Started	10/10/12	Completed 10/10/12
Supervisor	D. Rogers	Driller M. Wethington	Depth to Water	20.0 ft	Date/Time 10/10/12
Logged By	M. Jones		Depth to Water	N/A	Date/Time N/A

Lithology Elevation	Lithology Depth	Description	Overburden / Rock Core	Sample # / RQD	Depth / Run	Rec. Ft. / Rec. Ft.	Blows / Rec. %	Mois.Cont. % / Run Depth	Remarks
395.0	0.0	Top of Hole							
		Fat Clay, reddish brown to grayish brown, moist, stiff, trace fine gravel							
				SPT-1	5.0 - 6.5	1.1	8-6-7	--	
				SPT-2	10.0 - 11.5	1.2	8-12-12	--	
				SPT-3	15.0 - 16.5	1.5	5-7-7	--	
375.0	20.0								
		Fat Clay With Gravel, brown to reddish brown, wet, very stiff		SPT-4	20.0 - 21.5	0.9	5-16-18	--	
370.0	25.0								
		Lean Clay With Sand, reddish brown, wet, stiff, little gravel		SPT-5	25.0 - 26.5	1.2	6-9-6	--	
364.5	30.5								
		Lean Clay, brown and gray, wet, medium stiff		SPT-6	30.0 - 31.5	0.8	11-3-3	--	
360.0	35.0								
				SPT-7	35.0 - 36.5	0.9	5-7-8	--	

SUBSURFACE LOG

Project Number	175552005	Location	N=732303.82 ft E=1509472.90 ft
Project Name	CUF PZ Installation	Boring No.	**STN-102** Total Depth 46.5 ft

Lithology Elevation	Lithology Depth	Description	Overburden / Rock Core	Sample # / RQD	Depth / Run	Rec. Ft. / Rec. Ft.	Blows / Rec. %	Mois.Cont. % / Run Depth	Remarks
355.0	40.0	Lean Clay, grayish green, moist, stiff, silty, trace organics *(Continued)*							
350.0	45.0	Fat Clay, orangish brown mottled with gray, moist, very stiff		SPT-8	40.0 - 41.5	1.5	7-9-10	--	
348.5	46.5	Fat Clay With Sand, brown, moist, very stiff		SPT-9	45.0 - 46.5	1.0	10-11-15	--	

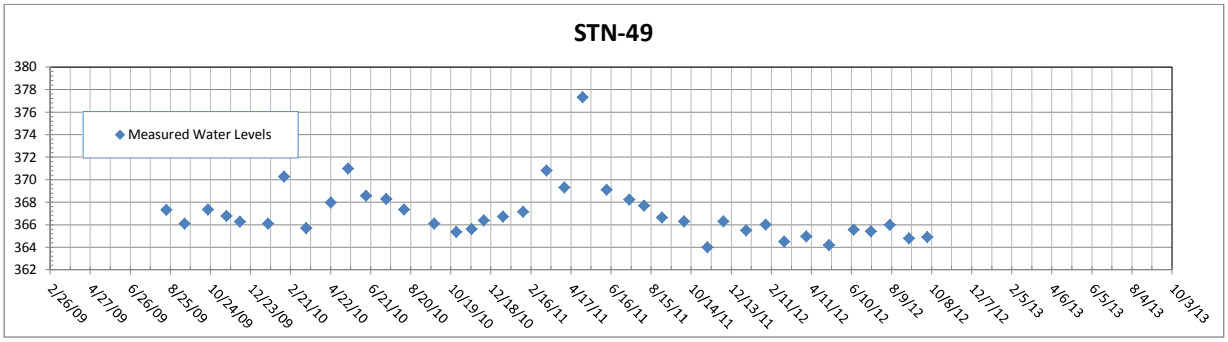
Auger Refusal / Bottom of Hole

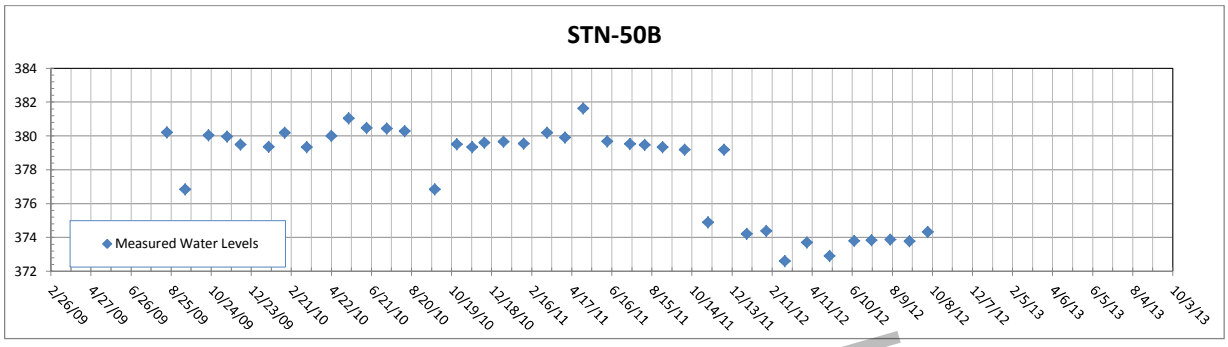
Top of Rock = 46.5
Elevation (348.5)

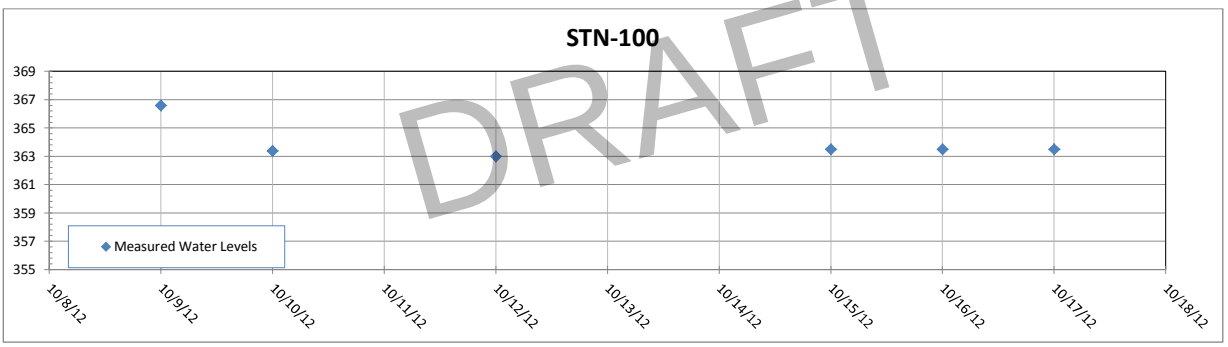
Appendix C

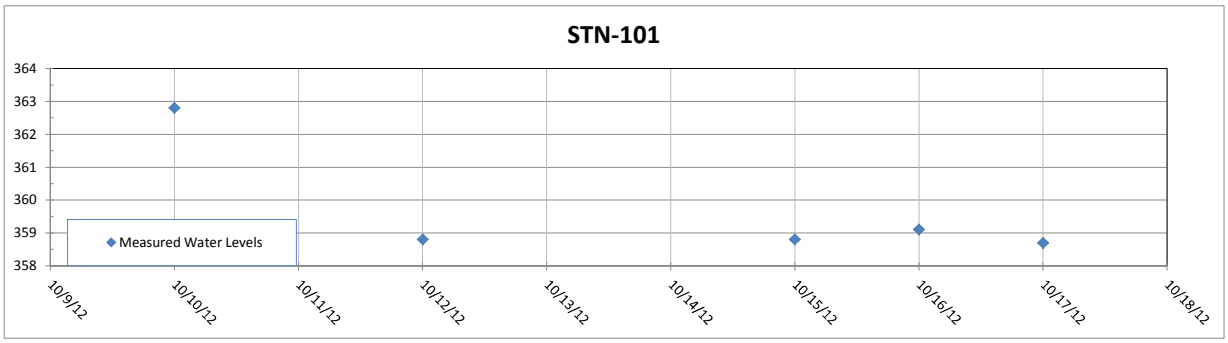
Supplemental Piezometer Installation Details and Readings

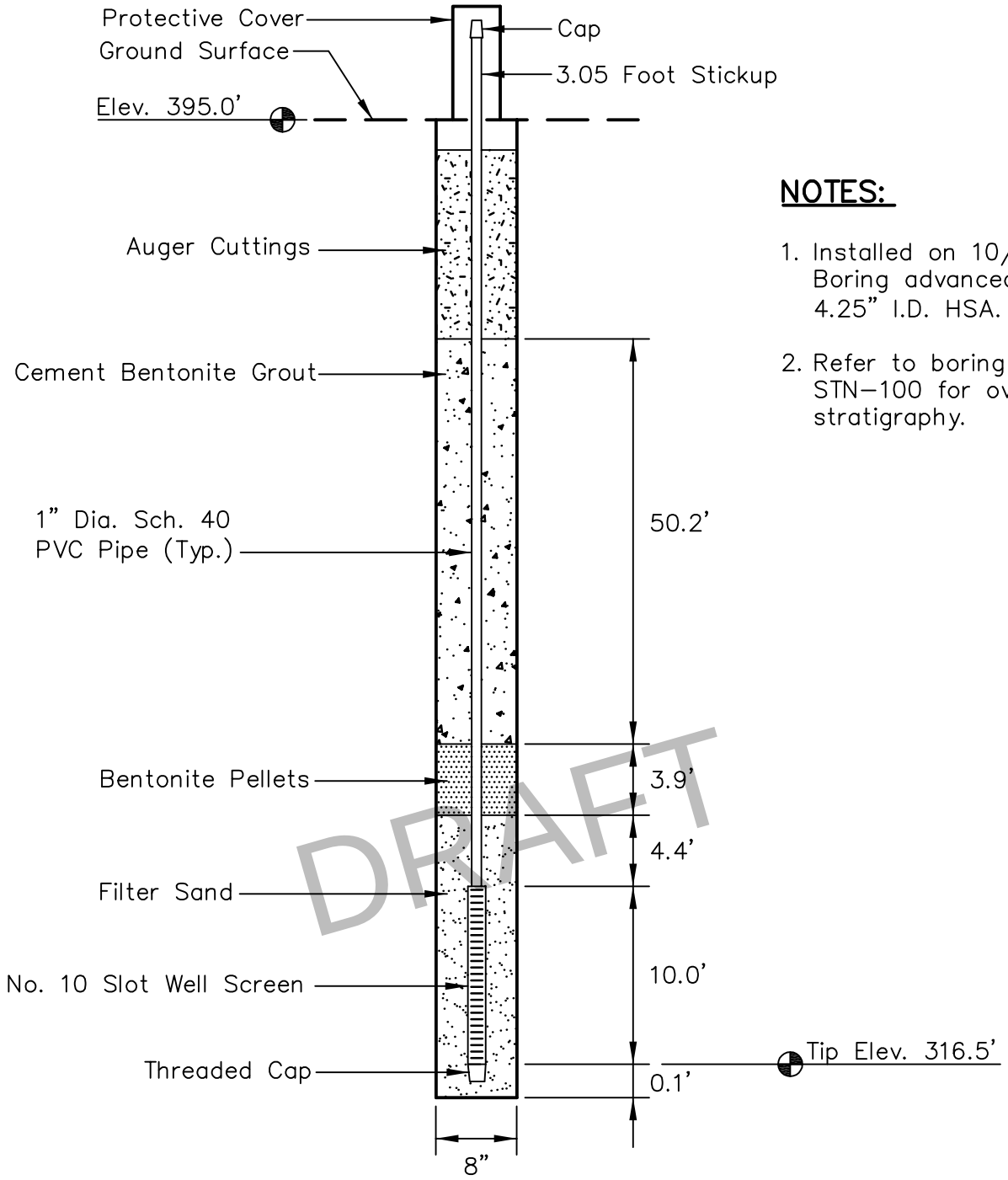
CUF Piezometer Readings - Ash Pond











NOTES:

1. Installed on 10/08/12. Boring advanced with 4.25" I.D. HSA.
2. Refer to boring log STN-100 for overburden stratigraphy.

LOCATION

Northing: 732,555.67
Easting: 1,509,526.03
Ground Elevation: 395.0'

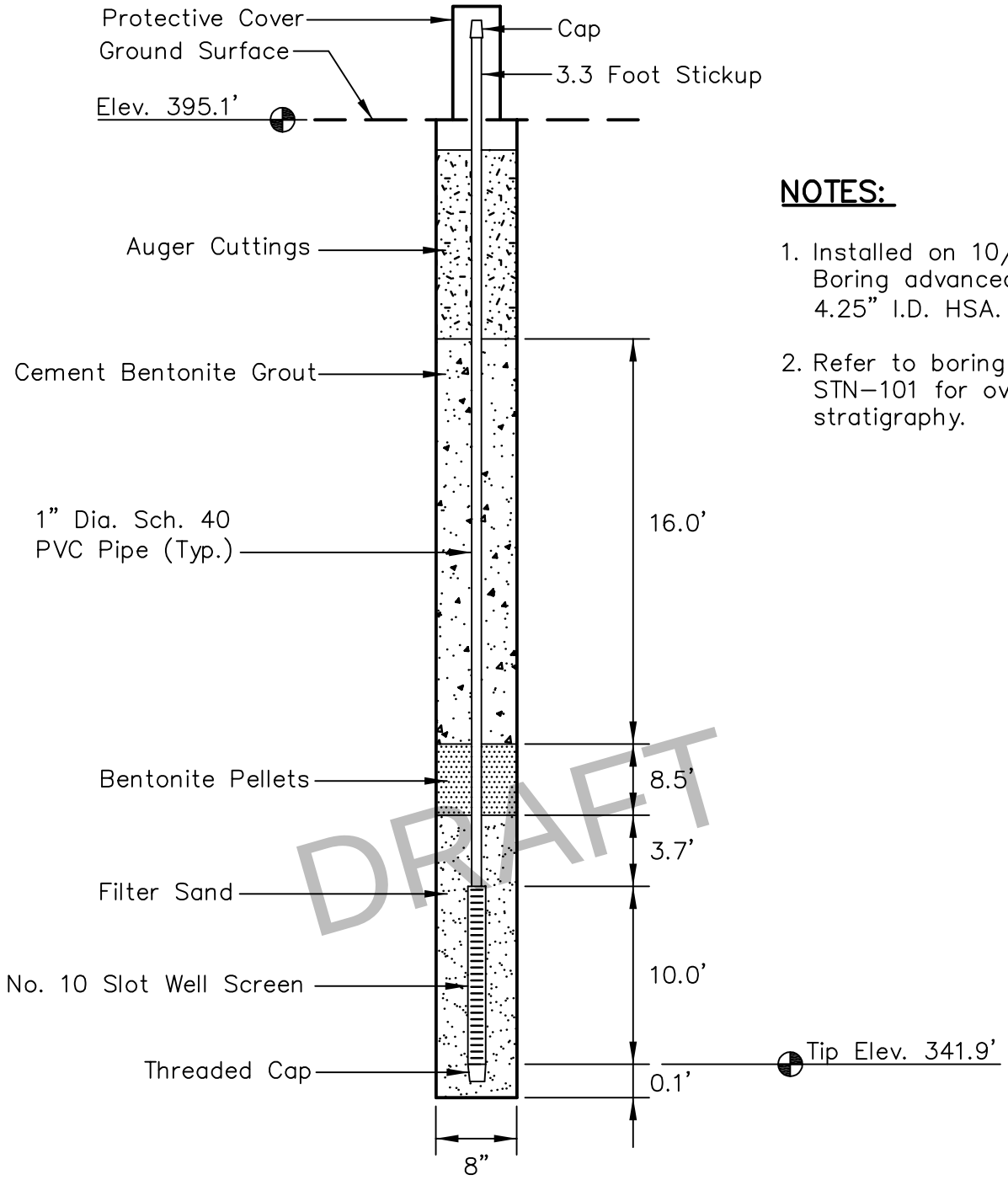
Locations provided by TVA, Power Systems Operations, Surveying and Project Services.
Horizontal Datum: NAD 27
Vertical Datum: NGVD29

STN-100
PIEZOMETER INSTALLATION DETAIL
CUMBERLAND FOSSIL PLANT
ASH POND

Stantec Consulting Services Inc.
1901 Nelson Miller Parkway
Louisville, Kentucky
40223-2177
502-212-5000
www.stantec.com

DRAWN BY	RRP	DATE	OCT. 2012	REVISED		SHEET
CHECKED BY	JRC	PROJ. NO.	175552005	1.	3.	1 OF 2
CHECKED BY	NAB	SCALE	NTS	2.	4.	

PLOT DATE: 11/05/2012 USER: PETTY, RICHARD
V:\1755\ACTIVE\175552005\GEOTECHNICAL\DRAWING\SHEET_FILES\59035PZ-STN-100.DWG



NOTES:

1. Installed on 10/09/12. Boring advanced with 4.25" I.D. HSA.
2. Refer to boring log STN-101 for overburden stratigraphy.

LOCATION

Northing: 732,353.39
Easting: 1,509,479.25
Ground Elevation: 395.1'

Locations provided by TVA, Power Systems Operations, Surveying and Project Services.
Horizontal Datum: NAD 27
Vertical Datum: NGVD29

STN-101
PIEZOMETER INSTALLATION DETAIL
CUMBERLAND FOSSIL PLANT
ASH POND

Stantec

Stantec Consulting Services Inc.
1901 Nelson Miller Parkway
Louisville, Kentucky
40223-2177
502-212-5000
www.stantec.com

DRAWN BY	RRP	DATE	OCT. 2012	REVISED		SHEET
CHECKED BY	JRC	PROJ. NO.	175552005	1.	3.	2 OF 2
CHECKED BY	NAB	SCALE	NTS	2.	4.	

PLOT DATE: 11/05/2012 USER: PETTY, RICHARD
V:\1755\ACTIVE\175552005\GEOTECHNICAL\DRAWING\SHEET_FILES\59035PZ-STN-101.DWG

Appendix D

Seepage Analyses Results

Seepage Analysis
Section P
Ash Pond

Cumberland Fossil Plant
Tennessee Valley Authority (TVA)

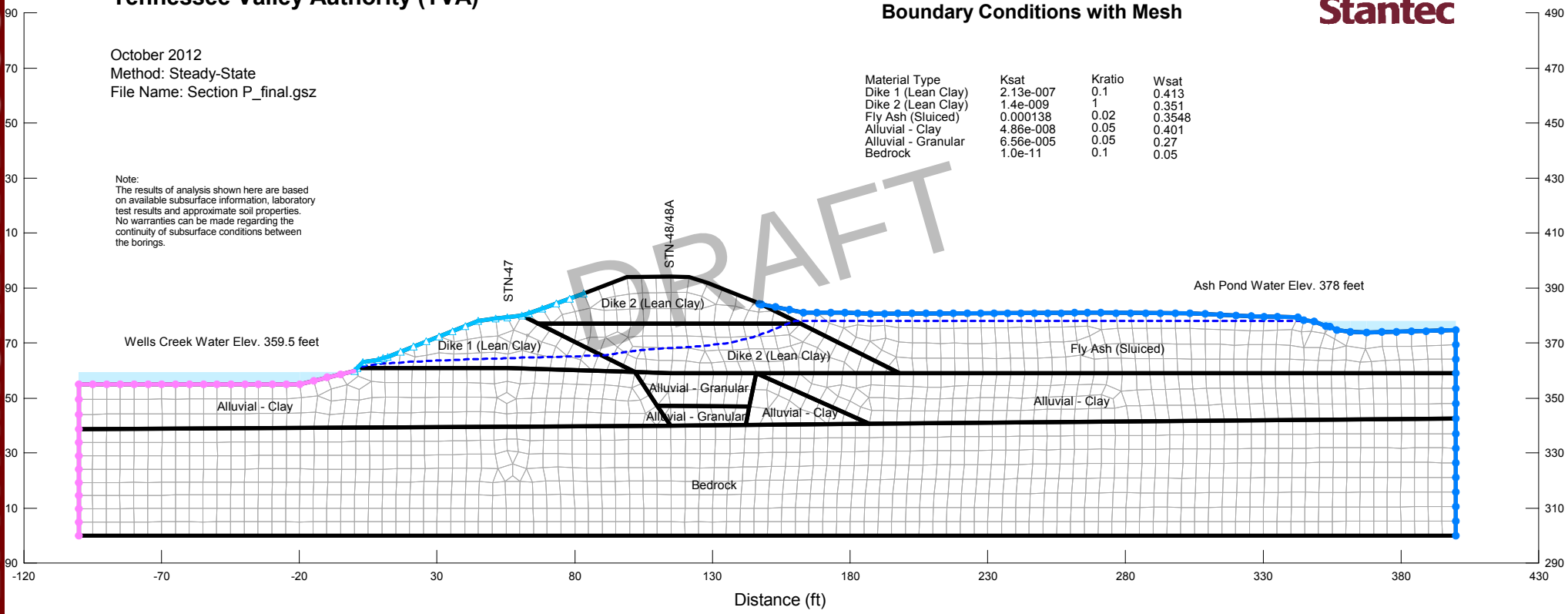
Stantec

October 2012
Method: Steady-State
File Name: Section P_final.gsz

Boundary Conditions with Mesh

Material Type	Ksat	Kratio	Wsat
Dike 1 (Lean Clay)	2.13e-007	0.1	0.413
Dike 2 (Lean Clay)	1.4e-009	1	0.351
Fly Ash (Sluiced)	0.000138	0.02	0.3548
Alluvial - Clay	4.86e-008	0.05	0.401
Alluvial - Granular	6.56e-005	0.05	0.27
Bedrock	1.0e-11	0.1	0.05

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.



Seepage Analysis
Section P
Ash Pond

Cumberland Fossil Plant
Tennessee Valley Authority (TVA)

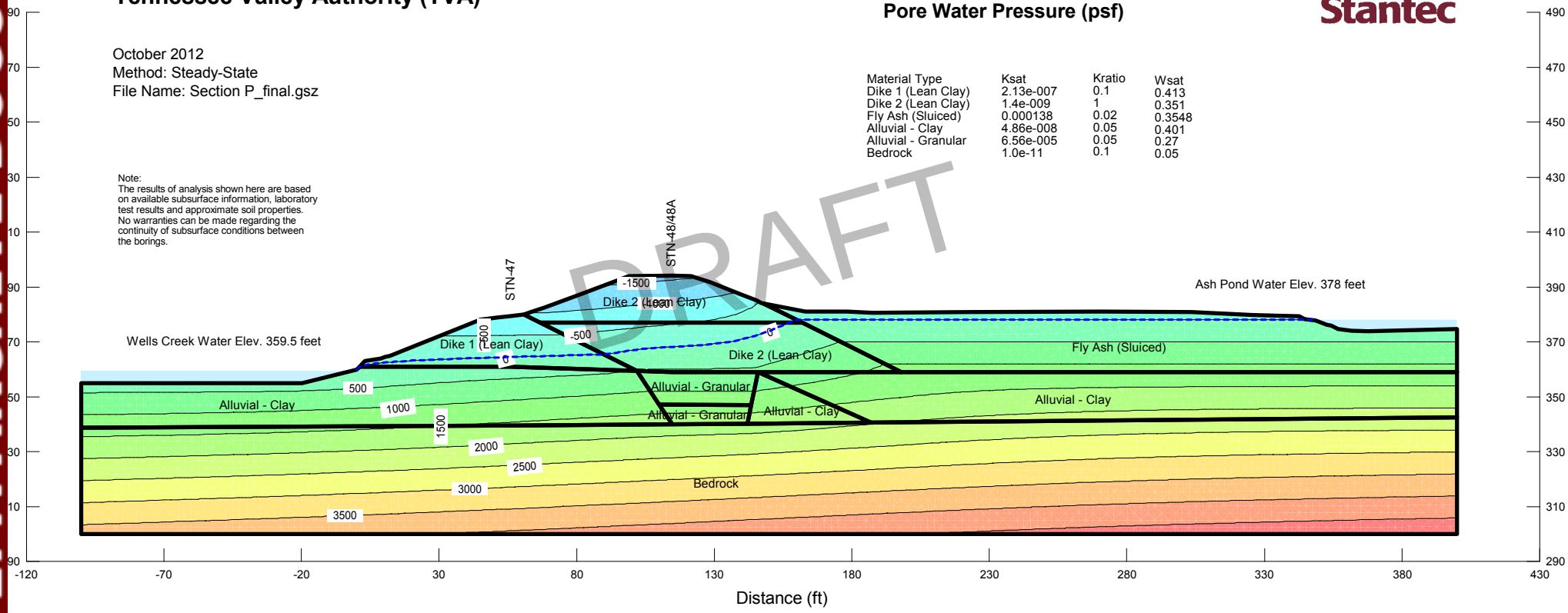
Stantec

October 2012
Method: Steady-State
File Name: Section P_final.gsz

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.

Pore Water Pressure (psf)

Material Type	Ksat	Kratio	Wsat
Dike 1 (Lean Clay)	2.13e-007	0.1	0.413
Dike 2 (Lean Clay)	1.4e-009	1	0.351
Fly Ash (Sluiced)	0.000138	0.02	0.3548
Alluvial - Clay	4.86e-008	0.05	0.401
Alluvial - Granular	6.56e-005	0.05	0.27
Bedrock	1.0e-11	0.1	0.05



Seepage Analysis
Section P
Ash Pond

Cumberland Fossil Plant
Tennessee Valley Authority (TVA)

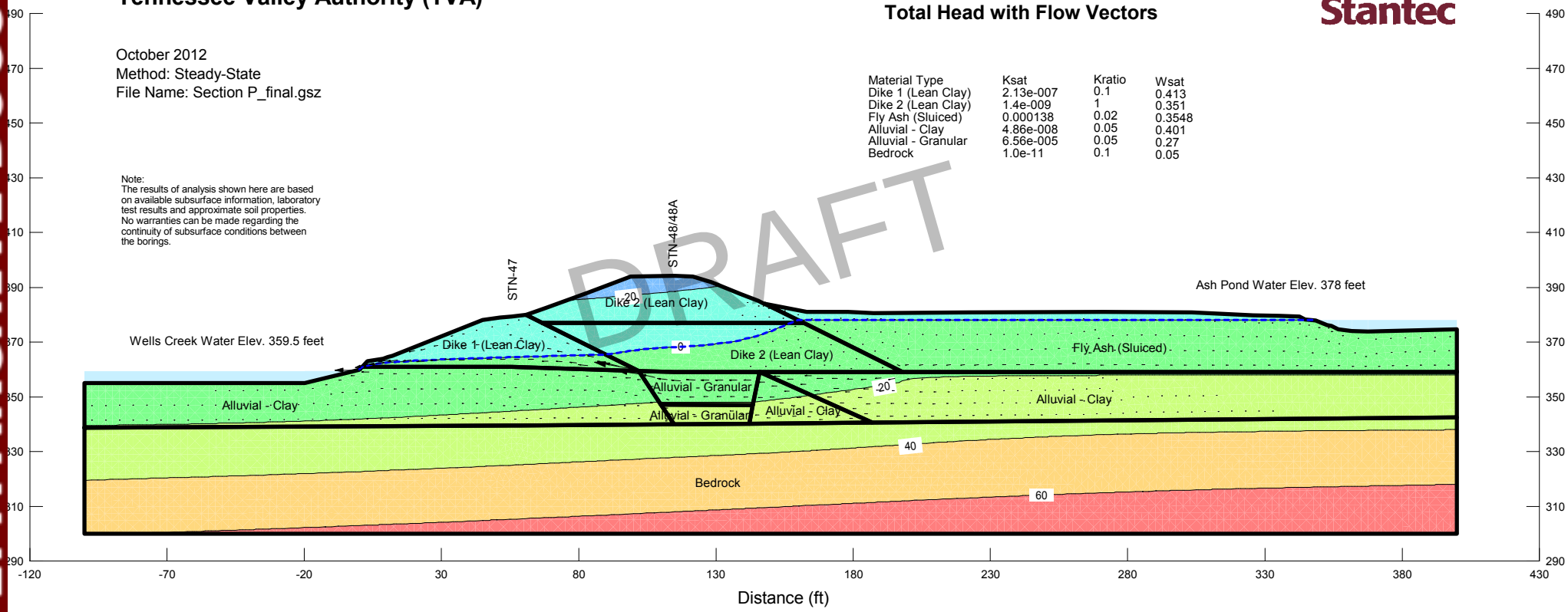
Stantec

October 2012
Method: Steady-State
File Name: Section P_final.gsz

Total Head with Flow Vectors

Material Type	Ksat	Kratio	Wsat
Dike 1 (Lean Clay)	2.13e-007	0.1	0.413
Dike 2 (Lean Clay)	1.4e-009	1	0.351
Fly Ash (Sluiced)	0.000138	0.02	0.3548
Alluvial - Clay	4.86e-008	0.05	0.401
Alluvial - Granular	6.56e-005	0.05	0.27
Bedrock	1.0e-11	0.1	0.05

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.



Seepage Analysis
Section P
Ash Pond

Cumberland Fossil Plant
Tennessee Valley Authority (TVA)

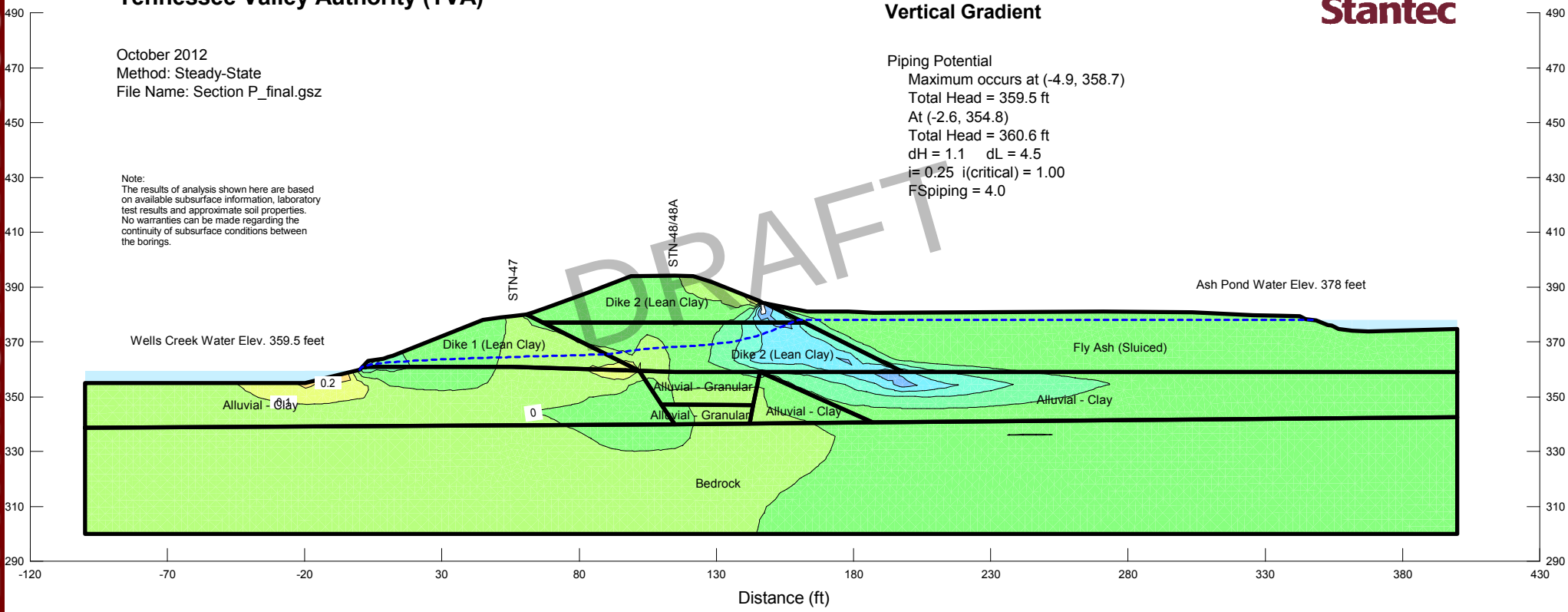
Stantec

October 2012
Method: Steady-State
File Name: Section P_final.gsz

Vertical Gradient

Piping Potential
Maximum occurs at (-4.9, 358.7)
Total Head = 359.5 ft
At (-2.6, 354.8)
Total Head = 360.6 ft
dH = 1.1 dL = 4.5
i= 0.25 i(critical) = 1.00
FSpiping = 4.0

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.



Seepage Analysis
Section Q
Ash Pond

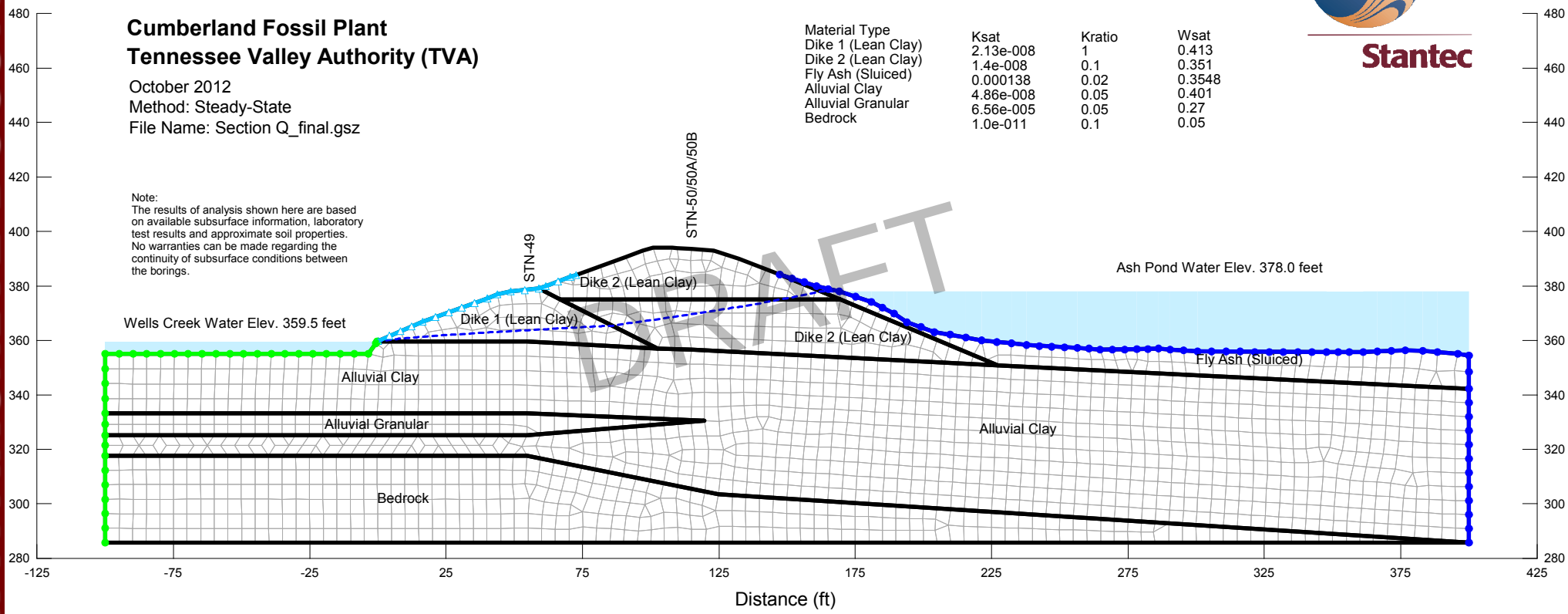
Boundary Conditions with Mesh

Cumberland Fossil Plant
Tennessee Valley Authority (TVA)

October 2012
Method: Steady-State
File Name: Section Q_final.gsz

Material Type	Ksat	Kratio	Wsat
Dike 1 (Lean Clay)	2.13e-008	1	0.413
Dike 2 (Lean Clay)	1.4e-008	0.1	0.351
Fly Ash (Sluiced)	0.000138	0.02	0.3548
Alluvial Clay	4.86e-008	0.05	0.401
Alluvial Granular	6.56e-005	0.05	0.27
Bedrock	1.0e-011	0.1	0.05

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.



Seepage Analysis
Section Q
Ash Pond

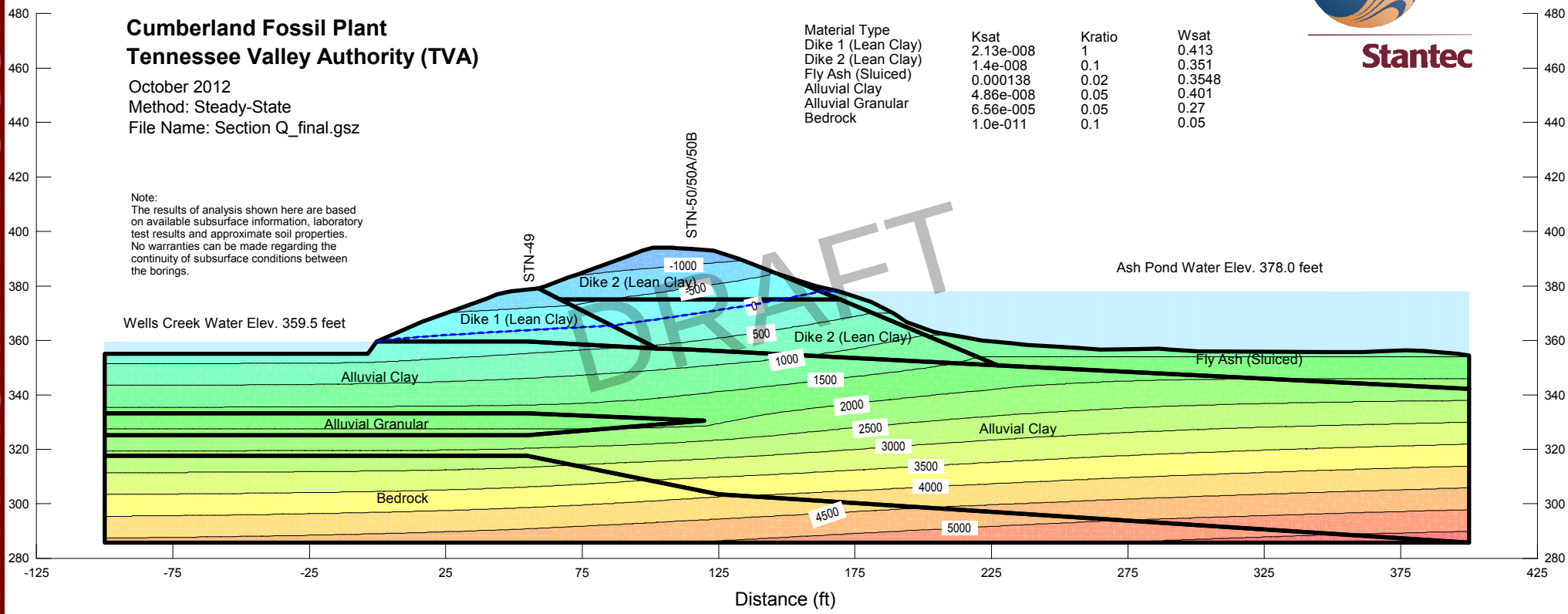
Cumberland Fossil Plant
Tennessee Valley Authority (TVA)

October 2012
Method: Steady-State
File Name: Section Q_final.gsz

Pore Water Pressure (psf)

Material Type	Ksat	Kratio	Wsat
Dike 1 (Lean Clay)	2.13e-008	1	0.413
Dike 2 (Lean Clay)	1.4e-008	0.1	0.351
Fly Ash (Sluiced)	0.000138	0.02	0.3548
Alluvial Clay	4.86e-008	0.05	0.401
Alluvial Granular	6.56e-005	0.05	0.27
Bedrock	1.0e-011	0.1	0.05

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.



Seepage Analysis
Section Q
Ash Pond

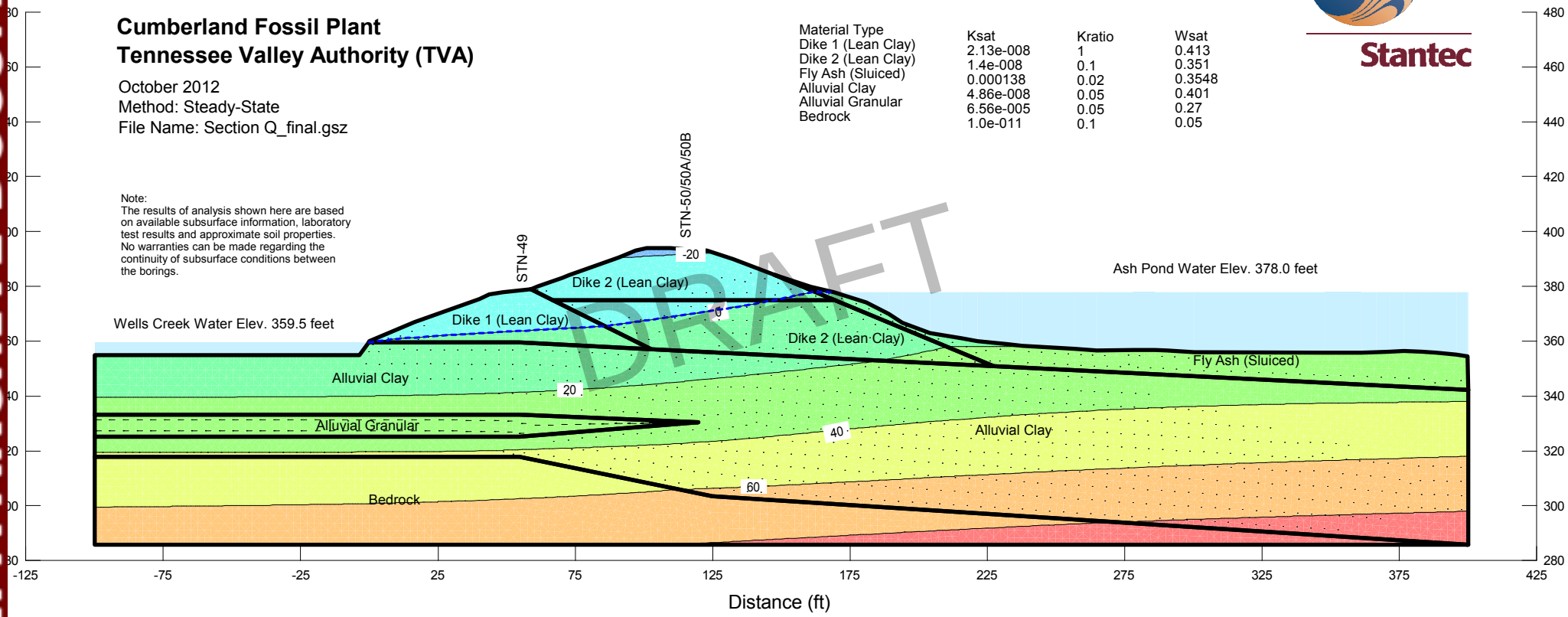
Cumberland Fossil Plant
Tennessee Valley Authority (TVA)

October 2012
Method: Steady-State
File Name: Section Q_final.gsz

Total Head with Flow Vectors

Material Type	Ksat	Kratio	Wsat
Dike 1 (Lean Clay)	2.13e-008	1	0.413
Dike 2 (Lean Clay)	1.4e-008	0.1	0.351
Fly Ash (Sluiced)	0.000138	0.02	0.3548
Alluvial Clay	4.86e-008	0.05	0.401
Alluvial Granular	6.56e-005	0.05	0.27
Bedrock	1.0e-011	0.1	0.05

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.



Seepage Analysis
Section Q
Ash Pond

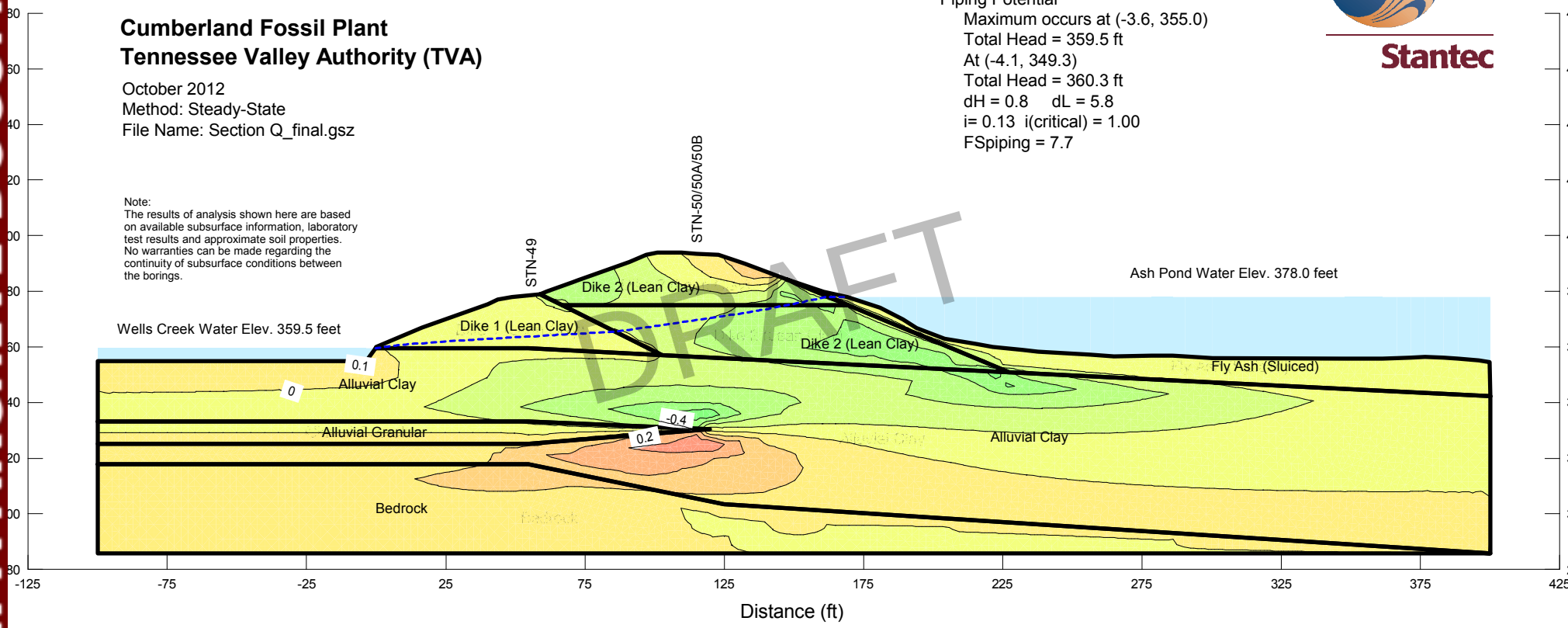
Cumberland Fossil Plant
Tennessee Valley Authority (TVA)

October 2012
Method: Steady-State
File Name: Section Q_final.gsz

Vertical Gradient

Piping Potential
Maximum occurs at (-3.6, 355.0)
Total Head = 359.5 ft
At (-4.1, 349.3)
Total Head = 360.3 ft
dH = 0.8 dL = 5.8
i= 0.13 i(critical) = 1.00
FSpiping = 7.7

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.



Seepage Analysis
Section R
Ash Pond

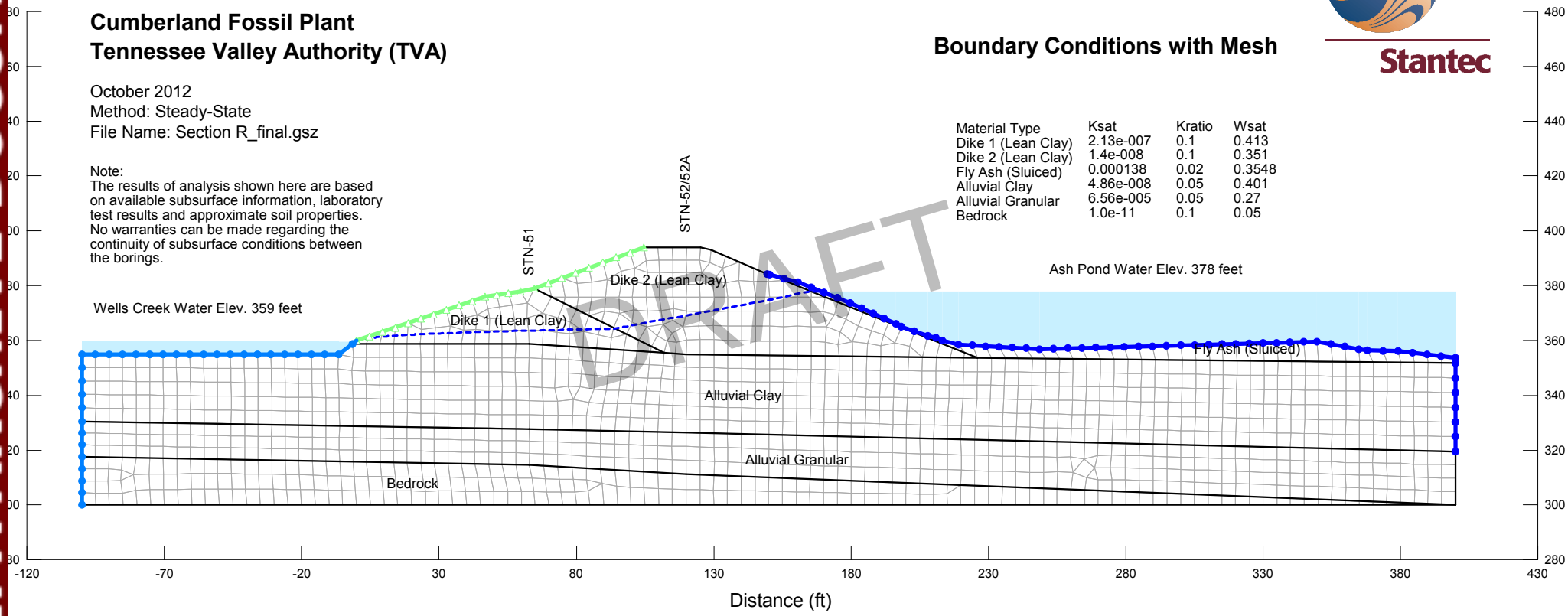
Cumberland Fossil Plant
Tennessee Valley Authority (TVA)

October 2012
Method: Steady-State
File Name: Section R_final.gsz

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.

Boundary Conditions with Mesh

Material Type	Ksat	Kratio	Wsat
Dike 1 (Lean Clay)	2.13e-007	0.1	0.413
Dike 2 (Lean Clay)	1.4e-008	0.1	0.351
Fly Ash (Sluiced)	0.000138	0.02	0.3548
Alluvial Clay	4.86e-008	0.05	0.401
Alluvial Granular	6.56e-005	0.05	0.27
Bedrock	1.0e-11	0.1	0.05



Seepage Analysis
Section R
Ash Pond

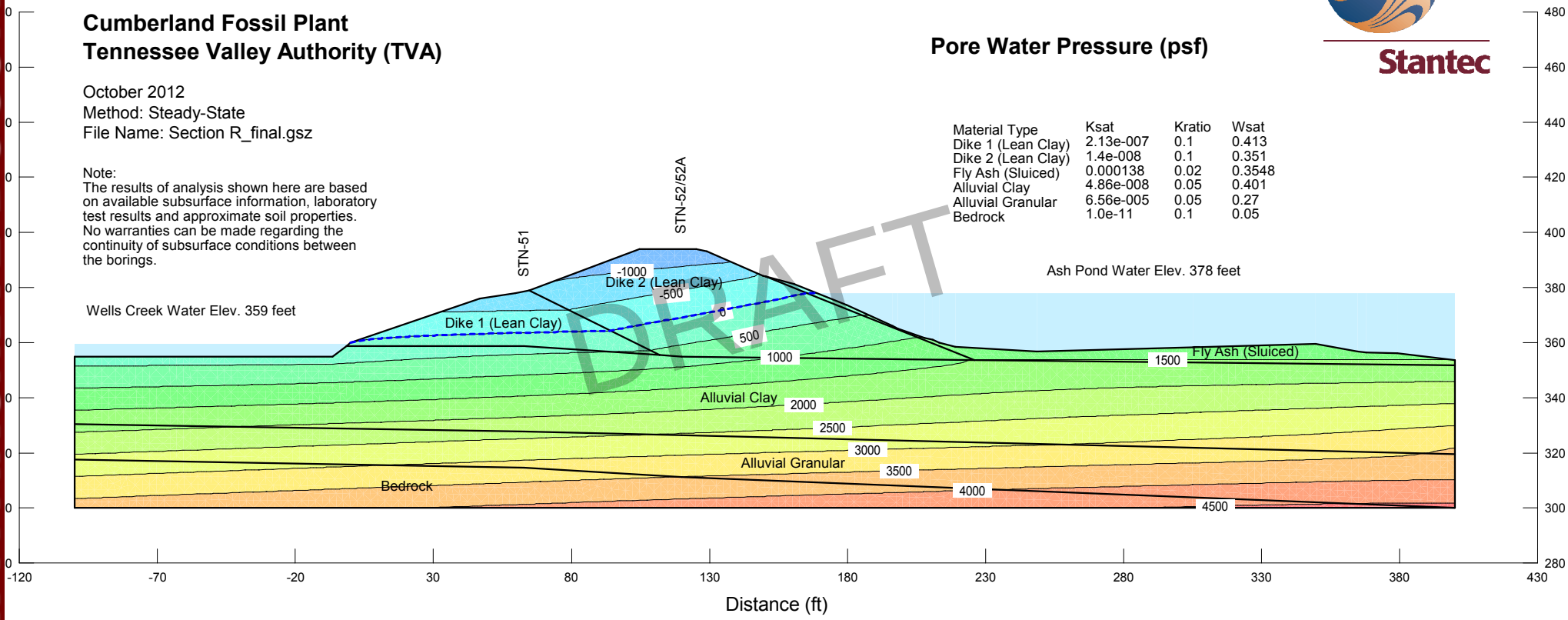
Cumberland Fossil Plant
Tennessee Valley Authority (TVA)

October 2012
Method: Steady-State
File Name: Section R_final.gsz

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.

Pore Water Pressure (psf)

Material Type	Ksat	Kratio	Wsat
Dike 1 (Lean Clay)	2.13e-007	0.1	0.413
Dike 2 (Lean Clay)	1.4e-008	0.1	0.351
Fly Ash (Sluiced)	0.000138	0.02	0.3548
Alluvial Clay	4.86e-008	0.05	0.401
Alluvial Granular	6.56e-005	0.05	0.27
Bedrock	1.0e-11	0.1	0.05



Seepage Analysis
Section R
Ash Pond

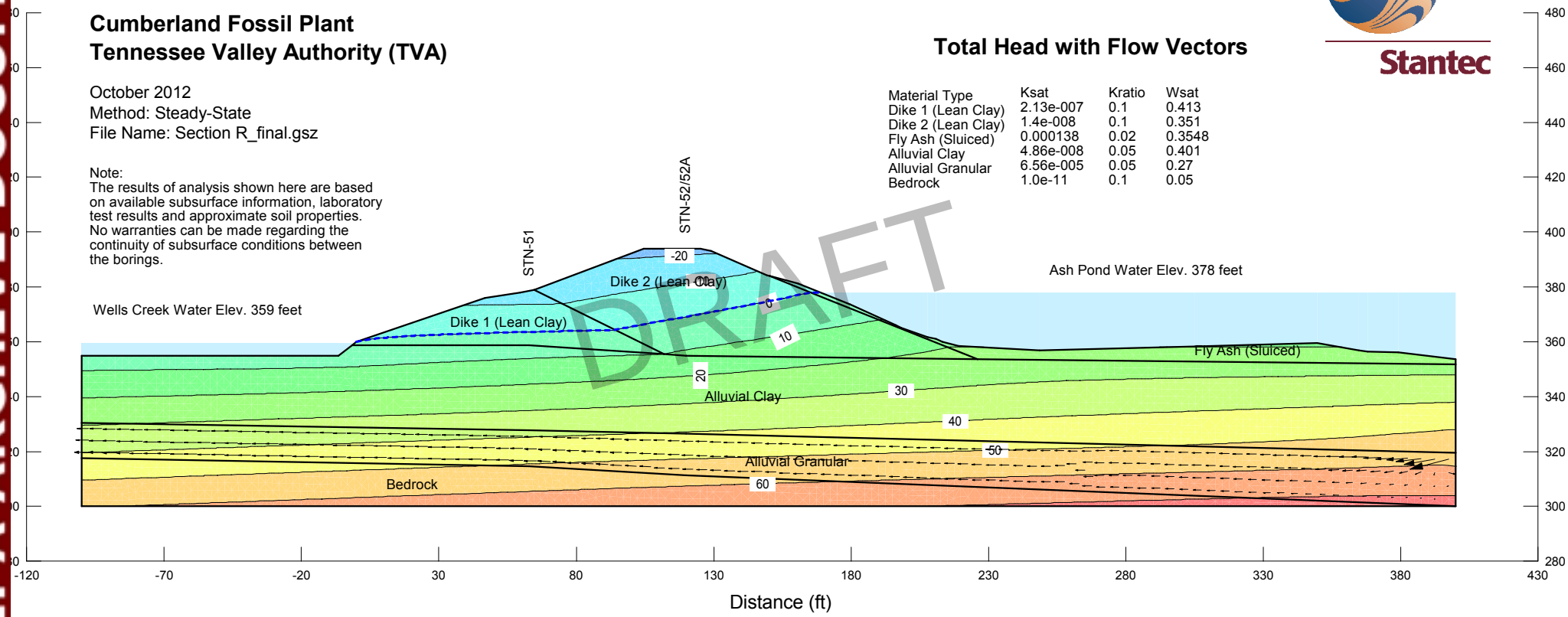
Cumberland Fossil Plant
Tennessee Valley Authority (TVA)

October 2012
Method: Steady-State
File Name: Section R_final.gsz

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.

Total Head with Flow Vectors

Material Type	Ksat	Kratio	Wsat
Dike 1 (Lean Clay)	2.13e-007	0.1	0.413
Dike 2 (Lean Clay)	1.4e-008	0.1	0.351
Fly Ash (Sluiced)	0.000138	0.02	0.3548
Alluvial Clay	4.86e-008	0.05	0.401
Alluvial Granular	6.56e-005	0.05	0.27
Bedrock	1.0e-11	0.1	0.05



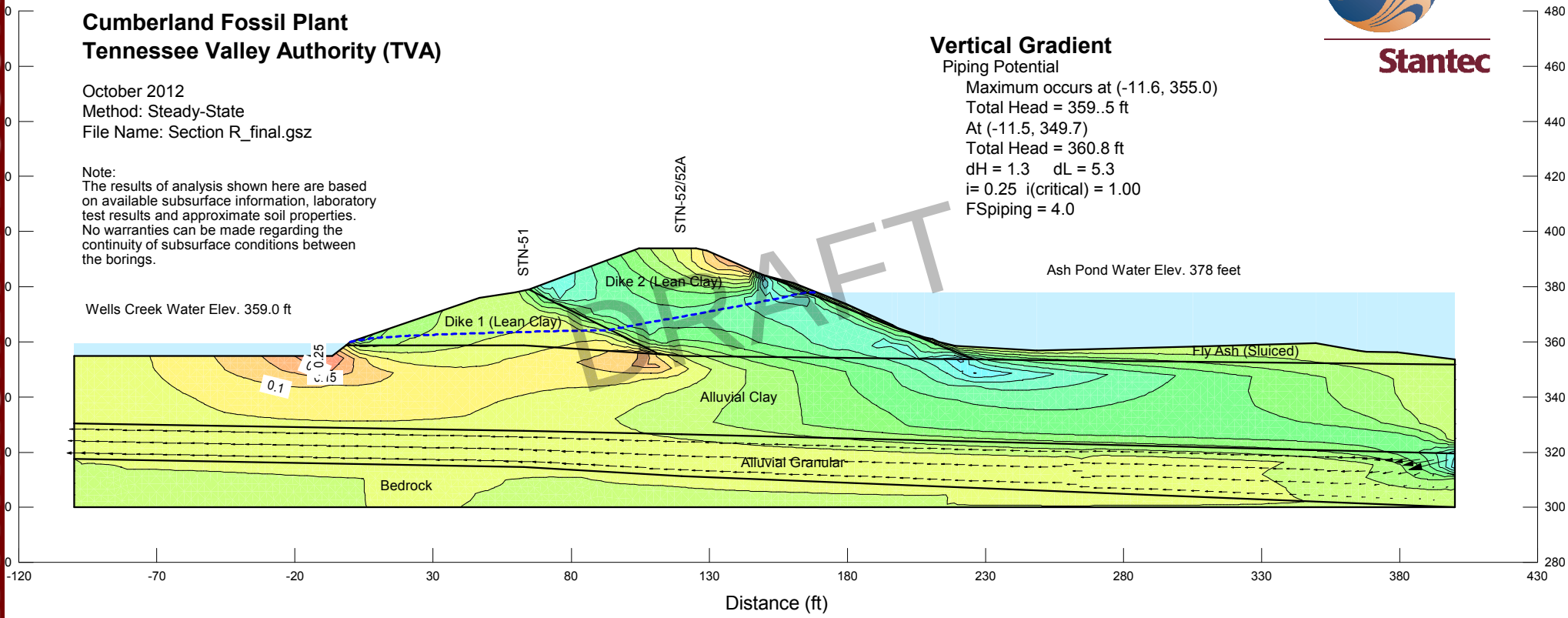
Seepage Analysis
Section R
Ash Pond

Cumberland Fossil Plant
Tennessee Valley Authority (TVA)

October 2012
Method: Steady-State
File Name: Section R_final.gsz

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.

Vertical Gradient
Piping Potential
Maximum occurs at (-11.6, 355.0)
Total Head = 359..5 ft
At (-11.5, 349.7)
Total Head = 360.8 ft
dH = 1.3 dL = 5.3
i= 0.25 i(critical) = 1.00
FSpiping = 4.0



APPENDIX C

Document 25

Draft Updated Seepage Analyses Ash Pond
November 29, 2012

Stantec

DRAFT

Updated Seepage Analyses

Gypsum Disposal Complex
Cumberland Fossil Plant
Cumberland City, Tennessee

Stantec Consulting Services Inc.
One Team. Infinite Solutions.
10509 Timberwood Circle Suite 100
Louisville, Kentucky 40223-5301
Tel: (502) 212-5000 • Fax: (502) 212-5055
www.stantec.com

Prepared for:
Tennessee Valley Authority
Chattanooga, Tennessee

November 29, 2012

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

November 29, 2012 rpt_002_175552005

Ms. Shannon Bennett
Tennessee Valley Authority
1101 Market Street
Chattanooga, Tennessee 37402

Re: Updated Seepage Analyses
Gypsum Disposal Complex
Cumberland Fossil Plant
Cumberland City, Tennessee

Dear Ms. Bennett:

As requested, Stantec Consulting Services Inc. (Stantec) has completed updating seepage analyses for the Gypsum Disposal Complex at the Cumberland Fossil Plant. The additional services are a result of the recommendations provided within the geotechnical report submitted on June 11, 2010 and included evaluation of the phreatic conditions within the dikes after completion of various improvements around the Gypsum Disposal Complex. This report documents the various improvements, results of updated seepage analyses models, and updated recommendations for the facility. Stantec appreciates the opportunity to provide engineering services for this project. If you have any questions, or if we may be of further assistance, feel free to contact our office.

Sincerely,

STANTEC CONSULTING SERVICES INC.

Jason R. Curtsinger, PE
Project Engineer

Nathan A. Bader, PE
Senior Associate

/cdm

Enclosures

Updated Seepage Analyses
Gypsum Disposal Complex
Cumberland Fossil Plant
Cumberland City, Tennessee

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Appendix

Updated Seepage Analyses
Gypsum Disposal Complex
Cumberland Fossil Plant
Cumberland City, Tennessee

1. Introduction

Stantec Consulting Services Inc. (Stantec) performed a geotechnical exploration of the Gypsum Disposal Complex at the Cumberland Fossil Plant (CUF) in 2009. The original Report of Geotechnical Exploration for the Dry Fly Ash Stack and Gypsum Disposal Complex was submitted on January 29, 2010. The report was revised and resubmitted on June 11, 2010. As part of this report, a seepage analysis was performed at Section H to show that sluicing gypsum to the top of the stack resulted in piping factors of safety less than the target value. Factors of safety from these analyses were less than 1 when sluice water was assumed to be present on top of the stack. The geotechnical report contained a number of recommendations for improving the deficient factor of safety against piping at Section H. Recommended improvements included grading the drainage ditches to promote positive drainage, discontinuing sluicing to the top of the gypsum stack and installation of graded filters in areas with observed seepage.

Since the previous geotechnical exploration of the Gypsum Disposal Complex, several improvements have been completed to increase stability and seepage factors of safety. Sluicing operations to the top of the stack have been ceased. Currently SynMat is processing the gypsum slurry while new lined settling channels are being constructed along the north portion of the stack. Also, the slopes of the gypsum stack have been flattened to approximately 3H:1V and graded filters have been installed in parts. The drainage ditches around the disposal complex have also been cleaned of vegetation and regraded to promote positive drainage. Some standing water is still present in portions of the ditch, but overall, drainage conditions have been improved. In the vicinity of Section H, a rip rap buttress was also constructed along the outer dike system to repair observed sloughing and increase the stability factor of safety.

Refer to the revised Report of Geotechnical Exploration submitted by Stantec dated June 11, 2010 for additional background information, site geology, historical information, and results of drilling and laboratory testing previously performed.

2. Scope of Additional Exploration

The scope included updating seepage analyses following the various improvements being completed around the Gypsum Disposal Complex. The original cross-section geometry for Section H was reviewed and updated to display the geometry changes from the completed improvement projects. The previous seepage analyses were also updated using the results of the on-going instrumentation monitoring program to reflect the changes in phreatic surface since sluicing to the stack was ceased. At the conclusion of updating the analyses, this addendum to the original geotechnical report was prepared to provide more accurate seepage models and to provide updated conclusions and recommendations.

3. Updated Engineering Analyses

3.1. General

The geotechnical engineering analyses performed during the original evaluation included evaluations of strength and permeability parameters, seepage analyses, and slope stability analyses. These previous stability analyses included several cross-sections and analyses were performed for various loading conditions. The seepage analysis was only performed on Section H to model the effects of sluicing to the top of the gypsum stack. Deficient factors of safety for piping were obtained. For this supplemental evaluation, only the seepage analyses were re-evaluated. The new phreatic surface was approximated using the piezometer readings and observations in the field. The water level within the Gypsum Disposal Complex was approximated at elevation 390.0 feet. For the updated analyses, the water level was assumed to exhibit steady state seepage conditions through the dike in the seepage models. Typically, the soil horizons and seepage parameters previously established were used in these updated analyses. Any differences between the original cross-sections and parameters are explained in the following paragraphs.

A layout showing the location of Section H on the Gypsum Disposal Complex is presented in Appendix A. Measured water levels at piezometers located at Section H (STN-21 and STN-22A) are displayed in Appendix B. Results of the updated analyses are summarized in the following paragraphs and are shown on computer output provided in Appendix C.

3.2. Updated Seepage Analyses

Seepage analyses were updated to evaluate the Gypsum Disposal Complex dikes with various improvements completed. The following paragraphs summarize the assumptions, parameters, etc. used in these updated seepage models.

3.2.1. SEEP/W Model

An analysis of long-term, steady state seepage through the Gypsum Disposal Complex perimeter dikes was performed to estimate the magnitude of seepage gradients for the evaluation of potential piping. The numerical seepage models were developed using SEEP/W 2007 (Version 7.19), a finite element code tailored for modeling groundwater seepage in soil and rock. SEEP/W is distributed by GEO-SLOPE International, Ltd., of Calgary, Alberta, Canada (www.geo-slope.com).

Cross-section H was updated with SEEP/W for the long-term, steady state loading condition with the various improvements. For the numerical analysis, the cross-section was subdivided into a mesh of elements, consisting of first-order quadrilateral and triangular finite elements. For seepage problems, where the primary unknown (hydraulic head) is a scalar quantity, first-order elements provide for efficient, effective modeling. Given appropriate hydraulic conductivity properties and applied boundary conditions, the finite element method (as implemented in the SEEP/W code) was then used to simulate steady seepage across the mesh. The total hydraulic head is computed at each nodal location, from which pore water pressures and seepage gradients can be determined.

Steady-state seepage was assumed for the updated analysis, with the static water level in the Gypsum Disposal Complex placed at approximate elevation 390.0 feet. The upstream boundary condition was updated to model the lack of sluice water on the crest of the stack

because regular sluicing activities have ceased. The other boundary conditions for the SEEP/W analysis were not changed from the original models. Refer to the original Report of Geotechnical Exploration for further information regarding the boundary conditions used.

Various construction projects have been completed or are on-going at the Gypsum Disposal Complex. Improvements that resulted in changes to the original cross-section geometry of Section H include:

- A rock buttress was constructed along the downstream face of the perimeter dike system in the vicinity of Section H to repair observed sloughing. The slopes of the rock buttress range from approximately 2.5H:1V to 3.5H:1V with a bench at approximate elevation 384.0 feet.

- The slope of the gypsum stack was graded to an approximate 3H:1V slope to remain within compliance with the existing permit.

- The drainage ditch located between Dike 2 and Dike 3 was regraded to promote positive drainage.

3.2.2. Updated Seepage Properties

For the modeled cross-section, the previous subsurface profile was updated with constructed improvements and the material seepage properties used were updated to reflect the actual phreatic surfaces within the dikes based on piezometer readings. Updated material properties used in the seepage analysis are summarized in Table 1.

Table 1. Updated Material Properties for SEEP/W Analysis

Soil Horizon	Saturated k_v (cm/s)	Ratio k_h / k_v	Specific Gravity G_s	Void Ratio e	Volumetric Water Content		Basis
					Residual	Saturated	
Dike 1 (Clay)	2.83E-07	1	2.71	0.670	0.060	0.399	Available Laboratory Data and Correlation w/ Typical Values
Dike 2 (Lean Clay)	2.83E-07	5	2.67	0.555	0.109	0.355	Available Laboratory Data and Correlation w/ Typical Values
Dike 2 (Fat Clay)	2.83E-07	10	2.73	0.830	0.090	0.444	Available Laboratory Data and Correlation w/ Typical Values
Dike 3 (Clay)	4.17E-06	10	2.65	0.628	0.109	0.384	Available Laboratory Data and Correlation w/ Typical Values
Alluvial (Clay)	4.30E-08	20	2.74	0.794	0.056	0.400	Available Laboratory Data and Correlation w/ Typical Values

Table 1. Updated Material Properties for SEEP/W Analysis

Soil Horizon	Saturated k_v (cm/s)	Ratio k_h / k_v	Specific Gravity G_s	Void Ratio e	Volumetric Water Content		Basis
					Residual	Saturated	
Alluvial (Granular)	3.60E-03	20	2.66	0.368	0.041	0.270	Available Laboratory Data and Correlation w/ Typical Values
Gypsum (Stacked)	2.83E-06	50	2.58	1.084	0.041	0.516	Available Laboratory Data and Correlation w/ Typical Values
Fly Ash (Stacked and/or Sluiced)	1.85E-06	50	2.55	1.260	0.015	0.543	Available Laboratory Data and Correlation w/ Typical Values
Fly Ash/Bottom Ash (Sluiced)	1.85E-06	50	2.59	0.605	0.027	0.355	Available Laboratory Data and Correlation w/ Typical Values
Toe Buttress (Rip Rap)	5.00E+01	1	2.60	0.663	0.020	0.400	Correlation w/ Typical Values
Bedrock (saturated only)	3.05E-12	10	N/A	N/A	0.050	0.050	Correlation w/ Typical Values

Note: SEEP/W requires input parameters k_h and ratio of k_v/k_h

For the updated analyses, an iterative process of parametric calibration using actual phreatic levels measured was used to arrive at final estimates of the seepage properties. After the initial seepage parameters were estimated, results from the SEEP/W model were compared to observations and measured water levels. Nodes were placed in the model at the screened piezometer intervals, so that the average head across these nodes could be compared to the corresponding piezometer reading. The material properties in the modeled cross-section were then varied until a reasonable match was obtained between the seepage predictions, actual conditions, and observations. Specifically, the kh/kv ratios were adjusted (while still maintaining the parameters within expected ranges) to give model predictions as consistent as possible with the water level measurements and observations made. The comparison between the field piezometer measurements and final SEEP/W predictions show the predicted groundwater table through the dikes within about four feet of the readings obtained in the piezometers installed within the dikes. The recorded water level readings are presented in Appendix B.

3.2.3. Results of Updated Seepage Analyses

Plots from the updated SEEP/W analyses of Section H through the Gypsum Disposal Complex are presented in Appendix C. The plots show the finite element mesh, material zones, and boundary conditions used in each analysis. The results are depicted in contour

plots of total head, pore water pressure, and seepage gradients. The seepage gradients were assessed for maximum exit gradients and the potential for soil piping.

On the modeled cross-section, examination of the output (predicted phreatic surface and vertical gradients) can be made to look for areas where the potential for excessive vertical gradients might exist that could possibly initiate the erosion or piping of material. In general, areas of potential concern are where water seeps laterally out onto a sloping ground surface, or where vertical, upward seepage occurs at the ground surface. First, contour plots of vertical gradient were examined to determine the general location of the maximum vertical exit gradient. On the modeled cross-section, the maximum upward gradient occurs near the toe of the perimeter dike. For the factor of safety calculations, vertical gradients from these locations were then used along with the critical gradients determined from the soil properties.

The calculated factor of safety against piping for the updated Section H is 3.7. This is greater than the Cedergren FS criterion of 2.5 to 3 that is referenced in United States Army Corps of Engineers (USACE) manual EM 1110-2-1901. In addition, regular sluicing to the top of the stack has ceased resulting in lower phreatic levels. New lined channels are also currently being constructed that will minimize infiltration of future sluice water into the stack. As a result, the water table within the gypsum stack will only be recharged by passing rain events and is expected to continue to lower over time increasing the piping factor of safety. Also, the location of the highest vertical gradient is located at the toe of the perimeter dikes beneath the rip rap buttress. The weight of the buttress was neglected in the piping factor of safety calculation. Finally, during the field activities, no seepage was observed exiting from beneath the rip rap buttress or along the dike slopes.

4. Conclusions and Recommendations

The supplemental conclusions and recommendations that follow are based on the results of the supplemental analyses and monitoring performed. In addition, information evaluated during the original report regarding the history of the Gypsum Disposal Complex and data obtained during the original geotechnical exploration were also used as a basis for these supplemental recommendations.

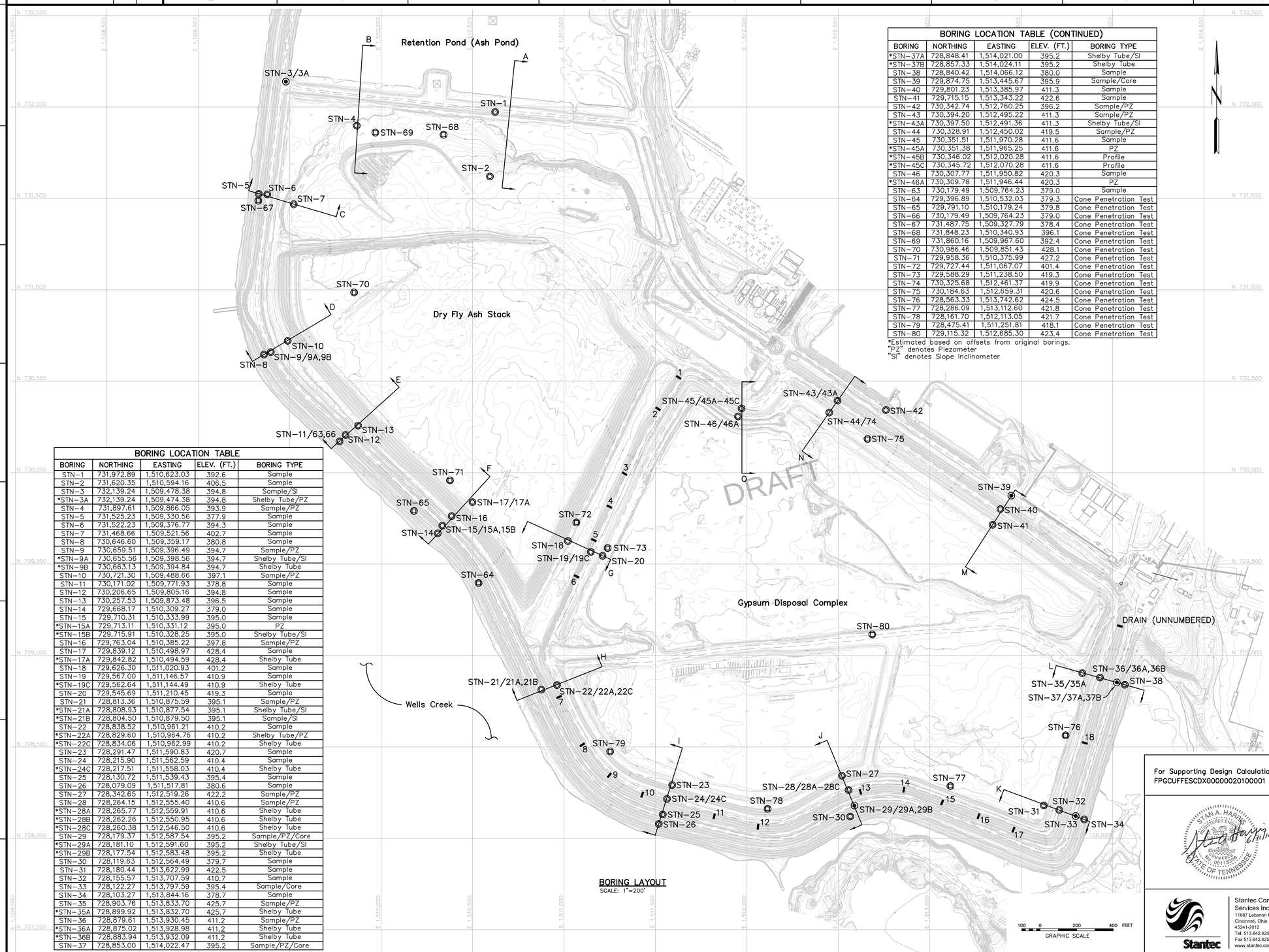
4.1. The results of the updated seepage analyses incorporating the recent improvements and operational changes were reviewed to identify conditions where seepage and possible piping may occur. The models predict the potential for seepage outbreaks to occur along the toe of the perimeter dikes. The calculated factor of safety against piping for the updated Section H is 3.7 which is greater than the Cedergren FS criterion of 2.5 to 3. However, the potential for seepage still exists along the toe of the dikes.

Based on the higher factors of safety against piping now that regular sluicing to the stack has ceased and slope improvements have been completed, additional mitigation to improve piping factors of safety are no longer judged to be required. However, continued monitoring of the dikes for seepage should occur until the Gypsum Disposal Complex is taken out of service and closed in accordance with the Seepage Action Plan for CUF.

4.2. All other conclusions, recommendations, references, and limitations of study provided in the original Report of Geotechnical Exploration submitted by Stantec dated June 11, 2010 remain applicable. Refer to this original report for additional information.

Appendix A

Boring Layout



BORING LOCATION
SCALE: 1"=200'

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

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

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

STAN A. HARRIS REGISTERED ENGINEER NO. 0011305 STATE OF TENNESSEE 6/11/10

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

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

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Cincinnati, Ohio
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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

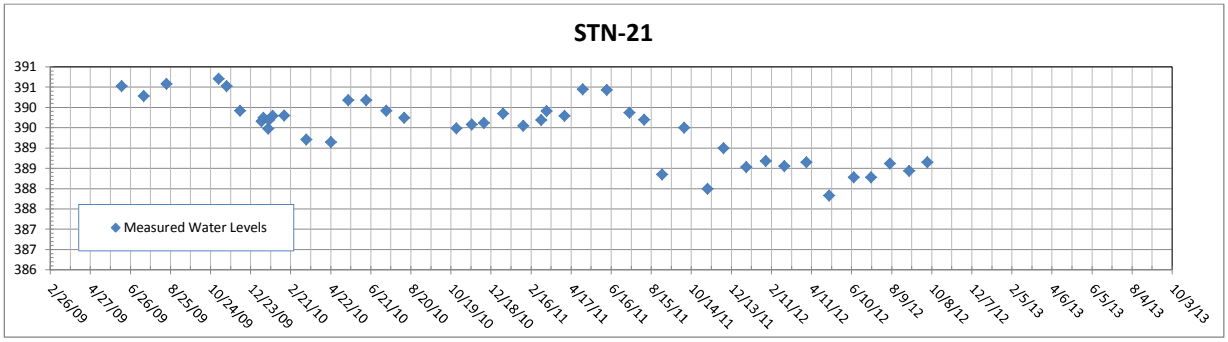
C.A.D. DRAWING
DO NOT ALTER MANUALLY

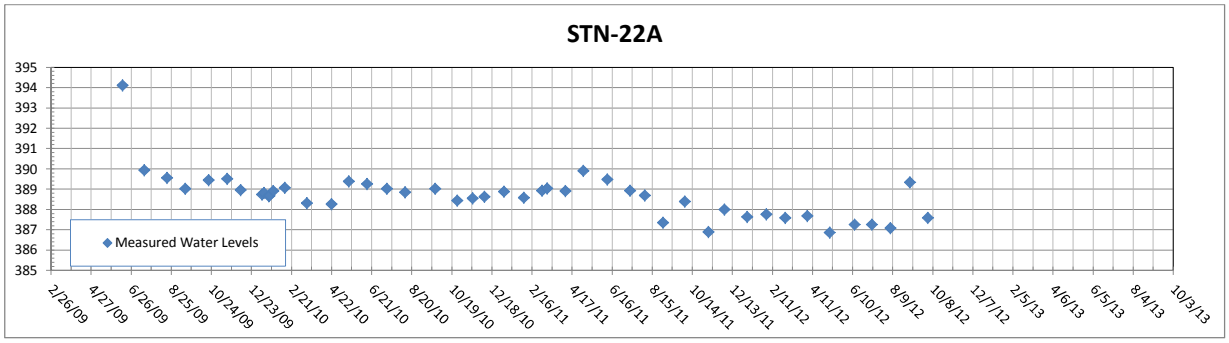
PLOT DATE: 06/08/2010 USER: WITHERS, CHEYENNE
K:\TVA\ACTIVE\T2533500\GEOTECHNICAL\DRAWING\EVO_RECORD\10W543-01-R0.DWG

Appendix B

Piezometer Readings

CUF Piezometer Readings - Gypsum Stack





Appendix C

Seepage Analyses Results

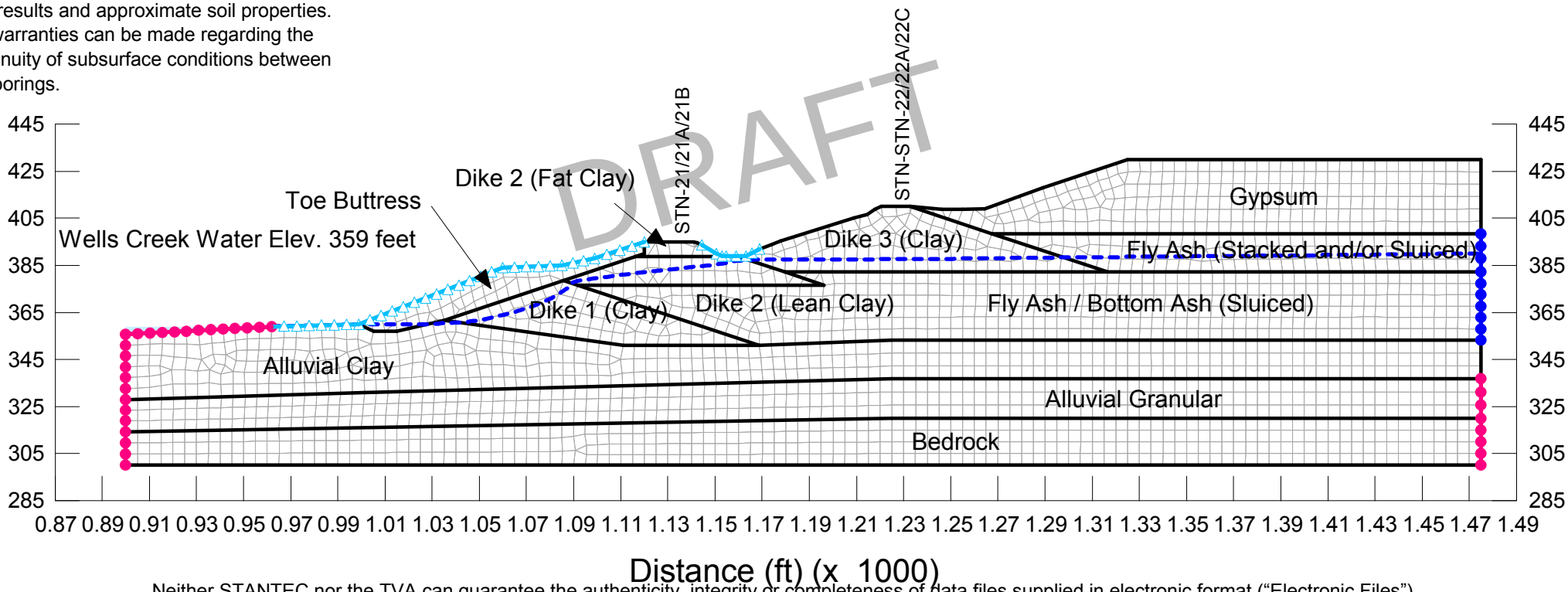
epage Analysis
ction H
ypsum Disposal Complex

mberland Fossil Plant
nnessee Valley Authority (TVA)

tober 2012
thod: Steady-State
e Name: Section H_final.gsz
e:
e results of analysis shown here are based available subsurface information, laboratory t results and approximate soil properties. warranties can be made regarding the tinuity of subsurface conditions between borings.

Boundary Conditions with Mesh

Material Type	Ksat	Kratio	Wsat
Dike 1 (Clay)	9.28e-009	1	0.399
Dike 2 (Lean Clay)	4.64e-008	0.2	0.355
Dike 2 (Fat Clay)	9.28e-008	0.1	0.444
Dike 3 (Clay)	1.37e-006	0.1	0.384
Gypsum	4.64e-006	0.02	0.516
Fly Ash (Stacked and/or Sluiced)	3.03e-006	0.02	0.543
Fly Ash / Bottom Ash (Sluiced)	3.03e-006	0.02	0.355
Alluvial (Clay)	2.82e-008	0.05	0.4
Alluvial (Granular)	0.00236	0.05	0.27
Toe Buttress (Rip Rap)	1.64	1	0.4
Bedrock	1.0e-12	0.1	0.05



epage Analysis
ction H
psum Disposal Complex

mberland Fossil Plant
nnessee Valley Authority (TVA)

tober 2012

thod: Steady-State

e Name: Section H_final.gsz

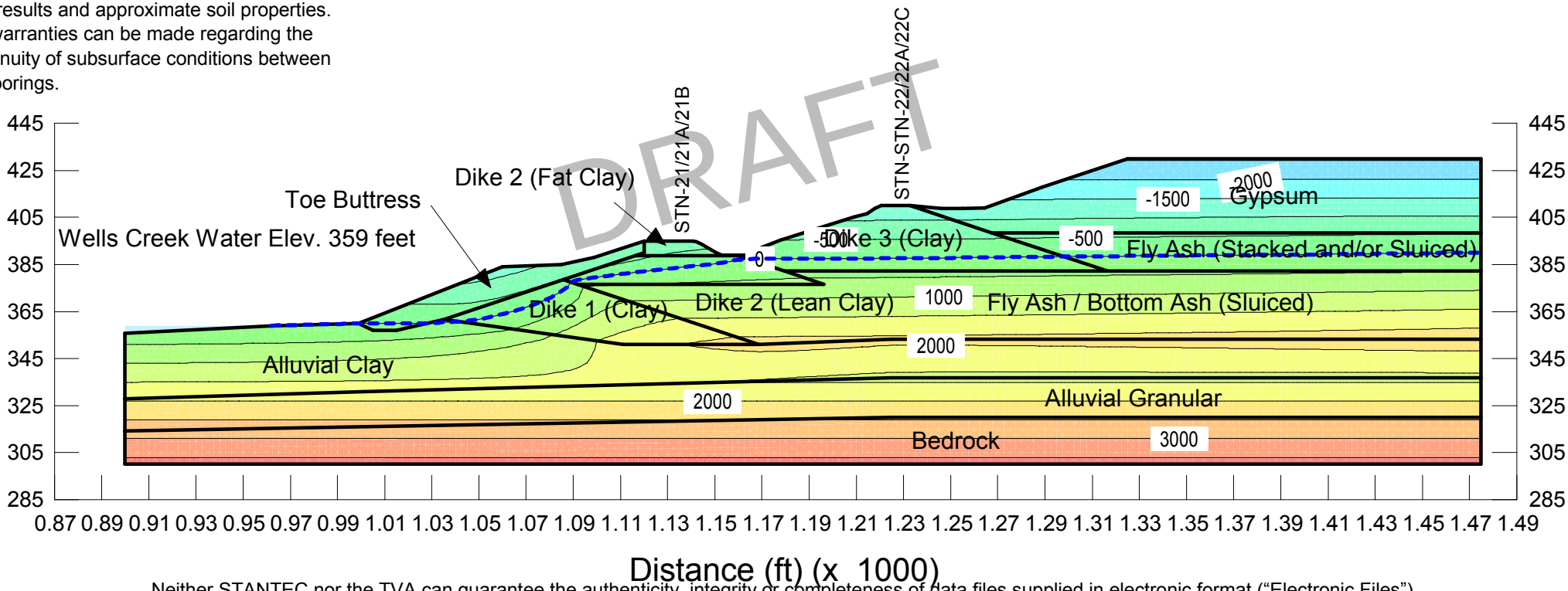
e:

e results of analysis shown here are based available subsurface information, laboratory results and approximate soil properties. warranties can be made regarding the tinuity of subsurface conditions between borings.

Pore Water Pressure (psf)

Material Type	Ksat	Kratio	Wsat
Dike 1 (Clay)	9.28e-009	1	0.399
Dike 2 (Lean Clay)	4.64e-008	0.2	0.355
Dike 2 (Fat Clay)	9.28e-008	0.1	0.444
Dike 3 (Clay)	1.37e-006	0.1	0.384
Gypsum	4.64e-006	0.02	0.516
Fly Ash (Stacked and/or Sluiced)	3.03e-006	0.02	0.543
Fly Ash / Bottom Ash (Sluiced)	3.03e-006	0.02	0.355
Alluvial (Clay)	2.82e-008	0.05	0.4
Alluvial (Granular)	0.00236	0.05	0.27
Toe Buttress (Rip Rap)	1.64	1	0.4
Bedrock	1.0e-12	0.1	0.05

Stantec



Seepage Analysis
Section H
Gypsum Disposal Complex

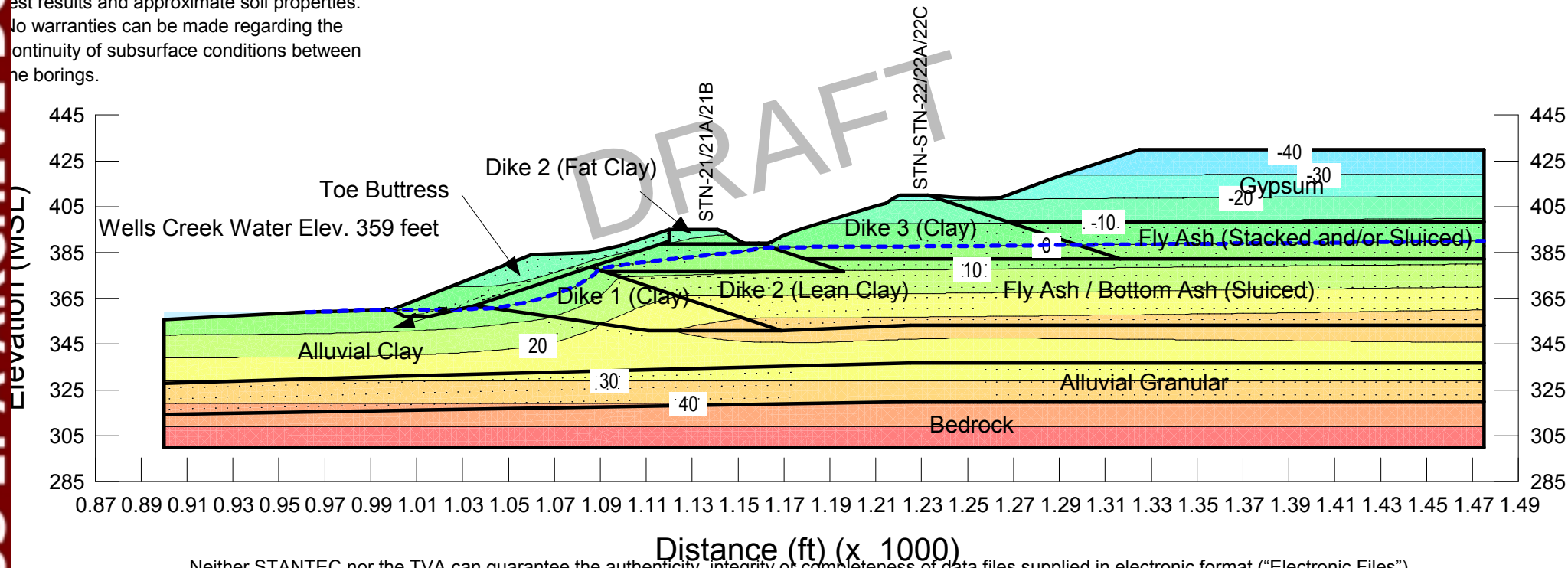
Cumberland Fossil Plant
Tennessee Valley Authority (TVA)

October 2012
Method: Steady-State
File Name: Section H_final.gsz
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.

Total Head with Flow Vectors

Material Type	Ksat	Kratio	Wsat
Dike 1 (Clay)	9.28e-009	1	0.399
Dike 2 (Lean Clay)	4.64e-008	0.2	0.355
Dike 2 (Fat Clay)	9.28e-008	0.1	0.444
Dike 3 (Clay)	1.37e-006	0.1	0.384
Gypsum	4.64e-006	0.02	0.516
Fly Ash (Stacked and/or Sluiced)	3.03e-006	0.02	0.543
Fly Ash / Bottom Ash (Sluiced)	3.03e-006	0.02	0.355
Alluvial (Clay)	2.82e-008	0.05	0.4
Alluvial (Granular)	0.00236	0.05	0.27
Toe Buttress (Rip Rap)	1.64	1	0.4
Bedrock	1.0e-12	0.1	0.05

Stantec



epage Analysis
ction H
psum Disposal Complex

mberland Fossil Plant
nnessee Valley Authority (TVA)

tober 2012
thod: Steady-State
e Name: Section H_final.gsz
e:
e results of analysis shown here are based available subsurface information, laboratory results and approximate soil properties. warranties can be made regarding the tinuity of subsurface conditions between borings.

Vertical Gradient

Piping Potential
Maximum occurs at (1020.0, 358.2)
Total Head = 360.0 ft
At (1020.9, 353.2)
Total Head = 361.3 ft
dH = 1.3 dL = 5.03
i=0.26 i(critical) = 0.97
FSpiping = 3.7

Stantec