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Coal Combustion Residue Impoundment

Round 11 - Dam Assessment Report

Johnsonville Fossil Plant

Tennessee Valley Authority
New Johnsonville, Tennessee

Prepared for:

United States Environmental Protection Agency
Office of Resource Conservation and Recovery

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INTRODUCTION, SUMMARY CONCLUSIONS AND RECOMMENDATIONS

The release of over five million cubic yards of coal combustion residue (CCR) from the Tennessee Valley Authority's 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 Johnsonville Fossil Plant active coal combustion residue (CCR) management unit is based on a review of available documents and on the site assessment conducted by Dewberry personnel on September 20, 2011. We found the supporting technical documentation to be generally adequate, although there is some deficiency (see Section 1.1.3). As described in Section 1.2.5, there is one recommendation based on field observations that may help to maintain a safe and trouble-free operation.

In summary, the Johnsonville Fossil Plant CCR management unit, Active Ash Disposal Area (Island Ash Area), is **FAIR** for continued safe and reliable operation. This rating is influenced by the lack of a critical engineering analysis of liquefaction potential under seismic loading of very loose ash under the upper dike raise embankment and of excessive deformation potential under seismic loading of very soft soils under the embankment and lower part of the embankment for the dike that impounds this CCR management unit. There are no other recognized existing or potential management unit safety deficiencies.

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 units) 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 Active Ash Disposal Area that was reviewed and used in preparation of this report.

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).

<u>LIMITATIONS</u>

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

The following conclusions pertain to the Active Ash Disposal Area (AADA) at the Johnsonville Fossil Plant. Conclusions are based on visual observations from a one-day site visit on September 20, 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)

Based on a review of the engineering data provided by TVA's technical staff and Dewberry engineers' observations during the site visit, the improved perimeter dike embankment and new outlet works of the Active Ash Disposal Area appear to be structurally sound under static loading conditions. Based on review of the furnished pseudo-static slope stability analysis completed by TVA's consultant, Stantec Consulting Services Inc., in February 2012, the perimeter dike embankment appears to be stable under relatively conservative seismic loading conditions, which were based on the 2,500-year return period event with a PGA = 0.254g (hard rock site). The design earthquake used in the analyses exceeds the historical New Madrid earthquakes (approximately 450 years return period). Although the rationale for using the hard rock PGA from the AMEC Geomatrix 2004 study as the design horizontal seismic coefficient is not entirely clear, the value (0.254g) appears conservative on the basis of values given in the old USACE reference "Recommended Guidelines for the Safety Inspection of Dams," dated September 1979.

Liquefaction potential and excessive deformation potential under seismic loading are unknown. Based on review of the available boring information, the embankment and foundation soils (clays and silts) do not appear to be of the type that would be highly susceptible to liquefaction. However, very loose ash may be susceptible to liquefaction, and it appears that there are some significant zones of very loose fly ash and bottom ash under the upper dike-raise embankment. Therefore, qualitatively, there could be potential for failure of the upper inside portions of these dikes due to liquefaction under earthquake loading. In addition, qualitatively, there could be some potential for excessive deformation to occur in the underlying very soft clays and silts under earthquake loading. TVA has indicated that liquefaction potential will be addressed as part of a comprehensive risk/consequences-based evaluation of seismic failure risks being conducted in closure design.

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

Furnished documentation shows that the AADA under current conditions should be able to pass the full 6-hour PMP event without overtopping the perimeter dike. Therefore, on the basis of furnished hydrologic/hydraulic documentation, the AADA appears to have satisfactory hydrologic/hydraulic safety.

1.1.3 Conclusions Regarding the Adequacy of Supporting Technical Documentation

The supporting technical documentation for the AADA is generally adequate. Engineering documentation reviewed is referenced in this report and selected parts of the documentation are included in Appendix A. There is some inadequacy in the technical documentation due to the current lack of analyses or assessments of the effects of the very soft foundation soils, as well as lack of assessment of liquefaction potential, particularly of very loose ash under the upper dike-raise embankment. As previously noted, TVA will address liquefaction potential as part of a comprehensive risk/consequences-based evaluation of seismic failure risks being conducted in closure design. The potential effects of the very soft foundation soils should be included in that evaluation.

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

The description of the management units provided by TVA is an accurate representation 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 units required to conduct thorough field observations. The visible parts of the dike embankments, spillway, and outlet structures were observed to have no signs of overstress, significant settlement, shear failure, or other signs of instability. The dike embankments appeared structurally sound. There are no apparent indications of unsafe conditions or conditions needing emergency 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 undocumented embankment repairs or prior releases observed during the field inspection.

1.1.7 Conclusions Regarding the Adequacy of the Surveillance and Monitoring Program

The surveillance program is adequate. The instrumentation monitoring program is adequate. In the absence of problem or suspect conditions, there is no need for additional performance monitoring instrumentation at this time.

1.1.8 Classification Regarding Suitability for Continued Safe and Reliable Operation

The Active Ash Disposal Area is <u>FAIR</u> for continued safe and reliable operation. No 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. Acceptable performance is also expected under applicable seismic loading conditions, provided that significant zones of the dike embankment and foundation soils are not susceptible to liquefaction and not subject to loss of shear strength as a result of strong seismic shaking. The rating is influenced by the lack of some critical engineering data for the dikes that impound this CCR management unit, namely analyses or assessments of liquefaction potential and excessive deformation potential. Implementation of recommendations as presented below would help improve the rating.

1.2 RECOMMENDATIONS

1.2.1 Recommendations Regarding the Structural Stability

No recommendations for physical or operational modifications to enhance structural stability appear warranted at this time. See Subsection 1.2.3 for recommendations regarding supporting technical documentation.

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

1) Perform a quantitative liquefaction analysis of embankment sections overlying very loose ash;

2) If liquefaction is indicated by the analysis, perform a post-earthquake analysis using static slope stability analysis using reduced shear strengths; and

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

1.2.4 Recommendations Regarding the Description of the Management Unit(s)

No recommendations appear warranted at this time.

1.2.5 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 poor grass growth, should be addressed by TVA's routine maintenance activities. However, it is recommended that the areas of the two small apparent seeps at either end of the gabion wall near the south end of the northeast dike be visually monitored in future inspections, to check for flowing seepage and movement of soil particles with any flowing seepage that may develop.

1.2.6 Recommendations Regarding the Maintenance and Methods of Operation

No recommendations appear warranted at this time.

1.2.7 Recommendations Regarding the Surveillance and Monitoring Program

No recommendations appear warranted at this time.

1.2.8 Recommendations Regarding Continued Safe and Reliable Operation

No recommendations appear warranted at this time.

1.3 PARTICIPANTS AND ACKNOWLEDGEMENT

1.3.1 List of Participants

*Stanley W. Notestine, Dewberry
*Fred Tucker, Dewberry
*John Dizer, TVA
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*Roberto Sanchez, TVA
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*R.J. Rodocker, TVA
*Griffin Lifsey, TVA
*Randy Roberts, Stantec
*Joshua Kopp, Stantec

*Participated in dike field observations

1.3.2 Acknowledgement and Signature

We acknowledge that the management unit referenced herein has been assessed on September 20, 2011.

_________________________ _________________________

Stanley W. Notestine, P.E.
Registered, TN 11369

Fred Tucker, P.E.

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

2.1 LOCATION AND GENERAL DESCRIPTION

The Johnsonville Fossil Plant (JOF) is located on the east bank of Kentucky Lake which is west of New Johnsonville, Tennessee, and lies immediately north of U.S. Highway 70. The plant draws cooling water from Kentucky Lake. The lake is the receiving body for discharge from the active CCR management unit at the JOF. See Appendix A Doc 01 for the location of the JOF site on an aerial map.

The JOF has one active CCR management unit, Active Ash Disposal Area (AADA), designed and permitted to contain fly ash, bottom ash, boiler slag, storm water, and plant process water. The Active Ash Disposal Area has been referred to variously as: Ash Disposal Area No. 2, Island Ash Area, Ash Disposal Area West of Boat Harbor, Trans Ash Cells 1, 2, 3A and 3B, Ash Disposal Areas 2 and 3, Main Ash Ponds A and B, and Stilling Pond C. In this report it will be referred to as the Active Ash Disposal Area or AADA.

The AADA is an island in Kentucky Lake immediately west of the plant generating facilities and immediately north of the U.S. Highway Bridge over Kentucky Lake. In plan view the island has a “stretched” diamond shape with the long dimension oriented generally north-south. The island (AADA) is accessed at its approximate midpoint by a causeway that forms the south side of the Boat Harbor, which lies between the north half of the island and the onshore plant generating facilities and Coal Yard. The plant intake channel is on the south side of the causeway. The AADA has two basic areas, including an ash stacking area in the northern majority of the island, and an ash-pond complex in the southern part of the island consisting of three ponds or cells separated by interior baffle dikes. The island also includes a small chemical treatment pond located on the south side where the access causeway connects to the island perimeter dike. The sluice lines from the plant discharge into the eastern part of the AADA and water flows west through a sluicing channel to the west side of the AADA, then south southwest to the ash pond complex. The water flows through a series of three ash ponds and ultimately discharges from the southernmost pond through a new spillway with six discharge pipes into Kentucky Lake. The normal water level in the ash ponds is currently maintained at an elevation to allow at least 5 feet of freeboard at the perimeter dikes. See Appendix A Doc 02 for an aerial view of the AADA showing dike locations, operation areas, and other features.

Table 2.1 shows a summary of the size and dimensions of the AADA perimeter dikes.

Table 2.1: Summary of Perimeter Dike Dimensions and Size	
	Active Ash Disposal Area
Maximum Dike Height (ft)	36
Crest Width (ft)	20 to 23
Approximate Length[1] (ft)	10,150
Side Slope (inside) H:V	2.1:1 to 3.5:1
Side Slope (outside) H:V	2.5:1 NE, 3:1 SE, 1.9:1 to 3:1 SW, 2:1 to 3.5:1 NW

[1]*Perimeter dike*

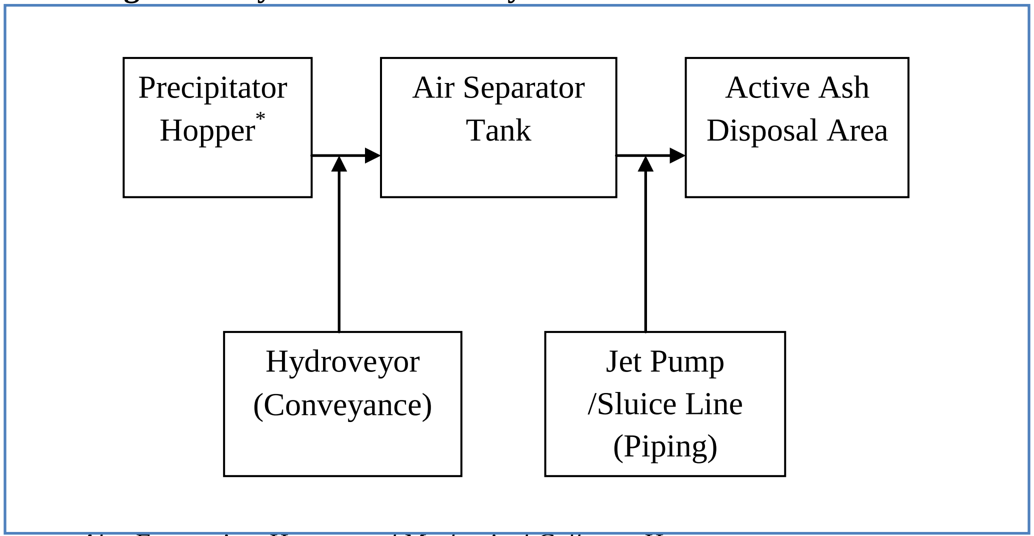
There are several other former ash disposal areas at the JOF including: South Railroad Loop Ash Disposal Area, Ash Dredge Pond East of Gas Turbines, and North Abandoned Ash Disposal (Areas A, B, and C). Their locations are outlined on the aerial view in Appendix A Doc 02. All of these former ash disposal areas, except Area C of the North Abandoned Ash Disposal Area, have been capped with soil and closed. It is understood from TVA personnel that Areas B and C of the North Abandoned Ash Disposal Area are within the fence of the adjacent DuPont Plant and that DuPont uses Area C for its plant discharges. The areas that could be accessed were briefly visited to confirm their status. None of these former ash disposal areas were assessed, since all except Area C are closed and cannot impound water; Area C is under DuPont’s control.

2.2 COAL COMBUSTION RESIDUE HANDLING

2.2.1 Fly Ash

Fly ash is collected and sluiced (pumped) to the Active Ash Disposal Area via a closed system process. Fly ash collected in precipitator hoppers is removed with hydroveyors to air separator tanks, where ash slurry is created. Ash from the economizer hoppers and mechanical collector hoppers is similarly combined in the slurry. A jet pump is used to convey the slurry through sluice lines (pipes) to a sluicing channel at the AADA. There is one fly-ash sluice line for each pair of the ten boilers at the JOF. Handling of the ash at the AADA is described in Section 8.1 of this report. See Image 2.1 for the general fly ash collection flow path.

Image 2.1: Fly Ash Collection System Flow Path



Also Economizer Hopper and Mechanical Collector Hopper

2.2.2 Bottom Ash

Bottom ash, along with boiler slag, is collected and sluiced (pumped) to the AADA via a closed system process. Ash collected in the bottom of the boiler is removed with the assistance of water jets, creating ash slurry. A jet pump is used to draw the slurry through a clinker grinder into a sluice line, which discharges to the sluicing channel at the AADA. (Although TVA did not specifically list process equipment such as water jets and clinker grinders, it is presumed that such equipment is used to help remove bottom ash from the boilers and grind it into suitable size for efficient sluicing.) There is one bottom-ash sluice line for each pair of the ten boilers at the JOF. As noted above, handling of the ash at the AADA is described in Section 8.1 of this report. See Image 2.2 for general bottom ash/boiler slag collection flow path.

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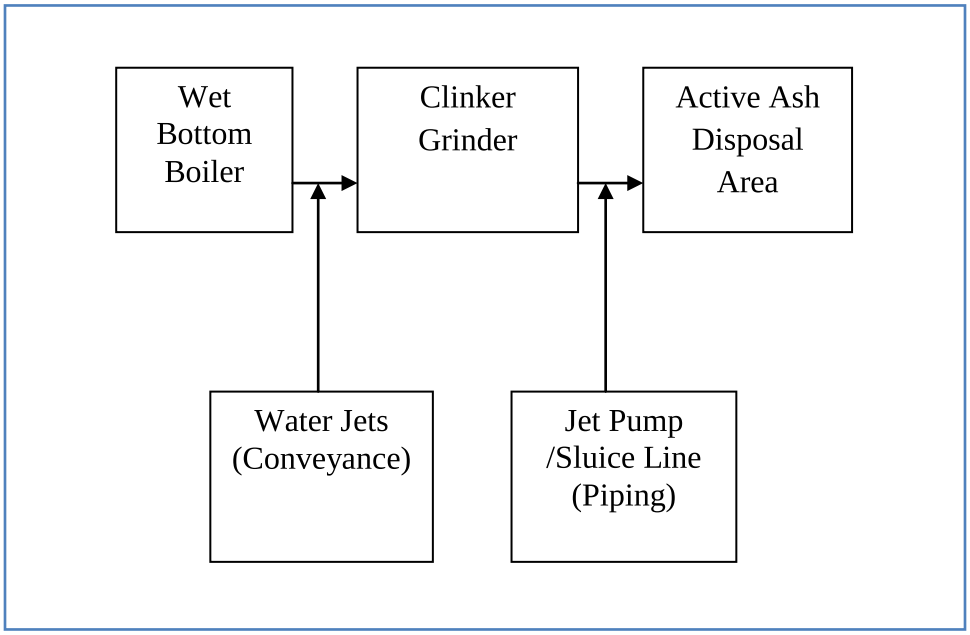
Image 2.2: Bottom Ash/Boiler Slag Collection System Flow Path



2.2.3 Boiler Slag

See Subsection 2.2.2 above. The boiler slag is collected with the bottom ash.

2.2.4 Flue Gas Desulfurization Sludge

JOF does not have equipment used for flue gas desulfurization sludge (FGD) collection, handling and disposition.

2.3 SIZE AND HAZARD CLASSIFICATION

Size classification is based on storage capacity (of water) and maximum dam height, see Table 2.2a. See Tables 2.1 and 2.3 for embankment height and estimated pond storage capacity.

The Active Ash Disposal Area currently has a Small Size Classification according to the USACE Size Classification criteria. However, it is noted that the capacity for water storage (to top of dike) would exceed 1,000 acre feet if a substantial volume of ash (on the order of 250 acre-feet or 403,333 cubic yards) were permanently removed; this would increase the size classification to Intermediate, based on available water storage capacity.

Table 2.2a: Size Classification (USACE ER 1110-2-106)		
	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 AADA embankments are not regulated for dam safety by a federal or state agency. Therefore, the AADA does not have a federal or state hazard classification. However, the TVA has assigned a Significant Hazard potential classification for the AADA. Dewberry concurs with this hazard potential classification on the basis of the hazard potential classification system adopted by USEPA; this classification system and the hazard potential determination are presented on the field observation checklist for the JOF AADA (identified as Active Ash Pond 2), included in Appendix B (also see Table 2.2b). The basis is that failure of the AADA perimeter dike embankment would discharge CCR into the adjacent Kentucky Lake and low-lying shoreline areas. Failure would not likely cause loss of life but would cause environmental damage and disruption of the plant operation.

Table 2.2b: Hazard Classification (FEMA Federal Guidelines for Dam Safety)		
	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)

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

Information on the amount of CCRs stored in the ash ponds was not provided. The amount of CCRs currently stored in the AADA was roughly estimated along with total volume capacity and remaining volume capacity, as summarized in Table 2.3 with other data.

Table 2.3: Estimated Capacity and other Data for the Unit	
Active Ash Disposal Area	
Surface Area[1] (acre)	87
Current Amount of Ash Stored (acre-feet)	2,090(v. approx.)
Current Remaining Volume Capacity[2] (level to top of dike) (acre-feet)	750 (v. approx.)
Total Volume Capacity (level to top of dike) (acre-feet)	2,840 (v. approx.)
Crest Elevation (feet)	390
Normal Pond Level (feet)	384.6

[1]Inside perimeter dike
[2]Includes Ash Pond Complex

The CCRs include fly ash and bottom ash/boiler slag. The current annual amount of ash sluiced to the AADA is 290,000 dry tons, including 260,000 dry tons of fly ash and 30,000 dry tons of bottom ash (including boiler slag). TVA's projected ash disposal amounts through the year 2015 are summarized in Appendix A Doc 03. TVA plans to close the AADA by 2016-20017 after converting from wet to dry operations; TVA plans to dispose of the dry ash in a permitted landfill. During closure of the AADA 1,129,000 cubic yards of dried ash will be removed and transported to a permitted landfill. TVA's Master Strategy for the JOF is summarized in Appendix A Doc 04.

2.5 PRINCIPAL PROJECT STRUCTURES

2.5.1 Earth Embankment

The Active Ash Disposal Area is encompassed by a perimeter dike, as illustrated in the aerial view of the AADA in Appendix A Doc 02. Segments of the embankment that comprise the perimeter dike are referred to as the northeast dike, southeast dike, southwest dike and northwest dike, according to their position in the diamond-shaped plan configuration of the perimeter dike. The perimeter dike embankment is constructed primarily of clay and silty clay. The perimeter dike embankment has been raised twice since original construction (see Section 4.1 for a summary of construction history). A summary of the perimeter dike dimensions is presented in Table 2.1. An aerial plan view of the AADA is shown on the Boring Plan and the Instrumentation Plan included in Appendix A Doc 05. Cross sectional views of the perimeter dike prior to recent improvements are illustrated by the analysis sections included in Appendix A Doc 06. Some design sections from drawings of remedial improvements for the southeast dike are included in Appendix A Doc 07 to illustrate the recent improvements made to both the southeast dike and the northeast dike.

2.5.2 Outlet Structures

The southernmost pond in the ash-pond complex in the south part of the AADA has a recently constructed primary spillway that discharges through the southwest dike to Kentucky Lake. This spillway consists of six 30-inch diameter high density polyethylene (HDPE) DR-17 pipes, each with a concrete overflow structure at the inlet end and concrete end wall at the outlet end. The overflow weir at each inlet consists of 5 removable fiberglass-reinforced stop logs that fit in slots formed in the sidewalls of the concrete structure. Each stop log is 6 inches high and 7 feet long. The stop logs are set to maintain a normal water elevation of 384.6 feet. The inlet area is protected from passing cenospheres and other floating debris with a galvanized metal skimmer wall. At the outlet end of the spillway conduits water discharges into a concrete apron with end sill before flowing onto a riprap-lined apron down to the lake. The concrete apron with end sill (energy dissipater) is cast integrally with the end wall.

There also are four 18-inch diameter DR-17 HDPE siphon pipes that were installed to provide dewatering of the ash ponds during construction of the new primary spillway. Each of the siphon pipes has a 34-foot long “torpedo” strainer at the inlet end consisting of the same pipe with 168, 4-inch diameter holes. The four siphons remain in place to serve as an emergency drawdown structure.

The former spillways included three sets of three decant towers with bottom discharge conduits including: one set located through the southeast dike, one set located through the southwest dike, and one set located through the northwest dike. These structures consist of reinforced concrete pipe (RCP). All the risers and conduits of the old spillways at the southeast and southwest dikes and all the conduits of the old spillways at the northwest dike were filled with grout and abandoned as part of the project to construct the new spillway, which was completed in November 2009. The risers of the spillways at the northwest dike reportedly had already been filled with concrete in 2003.

2.6 CRITICAL INFRASTRUCTURE WITHIN FIVE MILES DOWN GRADIENT

“Critical” infrastructure includes facilities such as schools, hospitals, fire stations, police stations, etc. There are 6 such facilities (schools and a fire station) that may be considered critical or potentially critical infrastructure located within a 5-mile radius of the plant. These facilities are noted on the 5-mile radius map and accompanying listing of the critical infrastructure included in Appendix A Doc 01. Most are located east and southeast in or near New Johnsonville on what appears to be higher ground and two are located on what appears to be higher ground on the other side of Kentucky Lake. None of these facilities would be threatened or directly impacted by failure of the AADA dike at the JOF. In general, the land use immediately around the JOF is industrial; a large DuPont plant is located on the north side of the JOF.

Flood and CCR released from postulated failure of the AADA perimeter dike would primarily impact Kentucky Lake and surrounding low-lying shore areas. A major failure and release of ash would likely disrupt plant operations and potentially block the water intake.

3.0 SUMMARY OF RELEVANT REPORTS, PERMITS, AND INCIDENTS

3.1 SUMMARY OF REPORTS ON THE SAFETY OF THE MANAGEMENT UNIT

Soon after the December 2008 failure of the CCR impoundment facility at the Kingston Fossil Plant, TVA engaged Stantec Consulting Services Inc. (Stantec) to visit and assess all of TVA's CCR impoundment facilities, including the Active Ash Disposal Area dike at the Johnsonville Fossil Plant. Stantec's initial field assessment was conducted on January 12 and February 23-25, 2009 and was subsequently documented in a Phase 1 report, which is included in Appendix A Doc 08 for reference. The Phase 1 report listed a number of notable observations and concerns and gave maintenance recommendations, as well as Phase 2 engineering and programmatic recommendations (see the Phase 1 report) that led to Phase 3 work. The Phase 3 work included design and construction of significant remedial improvements to correct a number of deficiencies. Aside from routine maintenance issues, some of the more significant concerns were:

- Stability of steep exterior slopes on the east (northeast) and southeast dikes;
- Raising the dikes by using upstream construction over sluiced ash;
- Significant seepage along the east (northeast) and southeast dikes;
- Use of pushed-together RCP stacked risers, surging of discharge from a couple of the old RCP spillway pipes, and history of sinkholes forming in the embankment over the active discharge pipes;
- High water level with only 2 feet or less of freeboard;
- Unknown composition of the perimeter dike and foundation material;
- Trend of not executing all maintenance recommendations from previous inspections; and
- Absence of Emergency Action Plan, Operation and Maintenance Plan, as-built drawings and construction testing records.

Stantec has performed additional engineering studies since the Phase 1 assessment. Furnished documentation reviewed includes Stantec's: "Report of Geotechnical Exploration and Evaluation of Slope Stability Ash Disposal Areas 2 and 3 (Active Ash Disposal Area)" dated April 13, 2010 (Appendix A, Doc 6), "Hydrologic and Hydraulic Calculations Summary" dated September 28, 2010 (Appendix A, Doc 9), and "Results of Pseudo-Static Slope Stability Analysis" dated February 15, 2012 (Appendix A, Doc 10. . Extensive remedial work has been performed at the AADA as a result of the engineering studies, as described in Subsection 4.1.3.

Stantec's 2011 annual inspection conducted on June 29-30, 2011 and presented in a report dated September 22, 2011, indicated no major structural or operational problems. Observations typically were of routine maintenance-type issues, such as eroded areas caused by surface runoff, bare spots lacking good vegetative growth (at exterior slope of southwest dike), animal burrows, small settled area (on new riprap buttress along the northeast dike), and a small localized slough 60 feet long near crest of the exterior slope of the northeast dike. Stantec provided recommendations for repair or monitoring of all these conditions. The small slough was repaired before the report was issued. The slough had occurred in the steep remnant of the original dike that still exists along the uppermost 5 vertical feet of both the northeast and southeast dikes. This steeper part was allowed to remain, since it has little impact on overall dike stability and because it will be removed during capping and final closure of the facility.

3.2 SUMMARY OF LOCAL, STATE, AND FEDERAL ENVIRONMENTAL PERMITS

Discharge from the AADA is regulated by the Tennessee Department of Environment and Conservation (TDEC). The JOF has been issued a National Pollutant Discharge Elimination System Permit No. TN0005355 with effective date of March 1, 2011 and expiration date of November 29, 2013.

3.3 SUMMARY OF SPILL/RELEASE INCIDENTS

TVA has indicated that there have been releases of cenospheres and of ash slurry from piping, associated with AADA operations.

4.0 SUMMARY OF HISTORY OF CONSTRUCTION AND OPERATION

4.1 SUMMARY OF CONSTRUCTION HISTORY

4.1.1 Original Construction

The Johnsonville Fossil Plant was built beginning in 1949 and completed in 1952. The first six units were completed in 1953, and the last four units were completed in 1959. The first ash disposal area was the North Abandoned Ash Disposal Area, which was built in the early 1950s and closed by 1976 and covered with soil, except Area C, which has continued to be actively used by the adjacent DuPont Plant. The original pond of the AADA was constructed to provide a second ash disposal area when the capacity of the first ash disposal area was nearing depletion. TVA historical information and reports from internet research indicate that, during 1968 and 1969, the original pond of the AADA was constructed by completing a hydraulic fill dike in Kentucky Lake, apparently along small islands, extending from the north end of the boat-harbor breakwater dike (original top elevation of 377 feet) and the south end of another dike (original top elevation of 363 feet) that extended southwest from the south end of the boat-harbor breakwater dike to apparently protect the intake channel. These original protective dikes had been constructed of hydraulic fill dredged from the boat-harbor channel and the intake channel. The dredge material consisted primarily of clay and silt, although a chert zone was encountered and mixed with the fine-grained dredge material to form a clayey gravel mixture in the south part of the boat-harbor dike. These older dikes bounded the northeast and southeast sides of the pond, and the new dike formed the southwest and northwest sides of the pond. The hydraulic fill construction brought the top elevation of the then new enclosing dike up to 368 feet to 370 feet, 9 feet to 11 feet above the Kentucky Lake summer pool elevation of 359 feet, except on the northeast side, which was already at 377 feet. Fill material for the dike construction came from dredging the interior area of the pond and consisted of primarily silty clay.

Soon after construction of the enclosing dike there was concern that waves from high water in Kentucky Lake/Tennessee River during flooding may overtop the dike at elevation 368 feet to 370 feet. Therefore, in 1970 the dike was raised to elevation 378 feet (first dike raise) using compacted clay from a borrow source located on the east side of the coal stockpile.

Apparently some ash had been sluiced into the pond before this first dike raise was constructed, but Stantec's 2010 test borings found no evidence of ash under this first dike raise. For purposes of this report the first dike raise is considered part of original construction, since it occurred soon after the initial hydraulic fill dike was completed. A spillway system was also constructed at that time, consisting of two sets of three spillway pipes. One set (South Spillways) was located near the south end of the southwest dike, and the other set (North Spillways) was located near the north end of the northwest dike. Each spillway reportedly consisted of a 48-inch diameter riser constructed of stacked RCP sections and a 36-inch diameter RCP outlet conduit through the dike embankment. The vertical riser and the near horizontal outlet conduit were connected via a precast concrete junction box at the inlet end of the conduit. No end walls were constructed at the outlet ends of the conduits.

4.1.2 Significant Changes/Modifications in Design since Original Construction

When the pond (AADA) began to reach capacity, the perimeter dike was again raised in 1978 in the upstream (inside) direction to the final (existing) crest elevation of 390 feet. Therefore, this second dike raise embankment was partly (more than 50 percent) founded on sluiced ash. After initially preparing a 4-foot thick base of compacted bottom ash under the upstream portion of the new dike, compacted clay was used to construct the new dike embankment. The clay was obtained from borrow areas located east of the 500kV switchyard and from the South Rail Loop Area. A third set of three spillways (East Spillways) was constructed at this time near the north end of the southeast dike. These spillways were similar to the original ones, except anti-seep collars were constructed around the conduits and rubber o-ring gasket seals were used in the RCP joints. The Chemical Treatment Pond was also constructed at this time.

4.1.3 Significant Repairs/Rehabilitation since Original Construction

Recent remedial improvements have been made at the AADA to address a number of concerns identified in Stantec's Phase 1 assessment (Appendix A, Doc 8) in early 2009 (see Section 3.1). The improvements have followed a four-stage approach of stability improvements or stability-related improvements forming the foundation or basis of final closure in 2017. These stages have included:

1. Completing construction of the new spillway system in November 2009 and lowering the normal operating water level in the ash pond complex to elevation 384.6 feet, 2.4 feet lower than the previous operating level. All the original spillway risers and conduits, which had not already been plugged with concrete, were fully grouted and abandoned.

2. Relocating the sluicing channel to flow from east to west across the AADA in the first quarter of 2010. The abandoned sluicing channel along the inside of the northeast dike was excavated to elevation 378 feet and maintained in a dewatered condition by pumping. This stage, as well as stage 1, has served to lower water levels in the AADA, which has led to lowering of the phreatic surface in the perimeter dike embankment, thus enhancing stability against a shear failure, as well as stability against a piping (internal erosion) failure.

3. Improving slope stability of the northeast dike by installing internal filtered drainage blankets over identified seepage areas, flattening the exterior slope using compacted clay, and constructing a rock buttress along the toe of the lower bench along the base of the northeast dike; these improvements were completed in August 2010.

4. Completing construction of similar (to 3. above) slope stability improvements of the exterior slope of the southeast dike in the third quarter of 2011.

Design cross sections shown on selected drawings of the remedial improvement plans for the southeast dike in Appendix A Doc 07 illustrate the typical stability improvements made at both the southeast and northeast dikes.

During construction of the rock (riprap) buttress along the toe of the lower berm of the northeast dike, two “slips” (i.e., sudden settlement forming a scarp) occurred. The first one was located approximately 300 feet north of the causeway and was about 50 feet long, with a 1.5-foot high vertical scarp aligned approximately along the original bank line. After several days of survey monitoring the slip was determined to have stabilized and additional rock was placed to grade. The second slip occurred approximately 1,500 feet north of the first one and was about 100 feet long. This slip stabilized over a period of several months, and the scarp was eliminated by grading back the existing in-place rock, rather than adding more rock. Both slips appeared to be localized bearing capacity failures, probably caused by placing the thick (15-foot) rock layer too quickly on the underlying clayey/silty hydraulic fill. Designers studied the slips and determined that neither was a serious threat to the perimeter dike.

As previously noted, remedial improvements at the southeast dike were similar to those at the northeast dike. One difference was that a seep area (identified as Seep 3A) was stabilized with biaxial geogrid before placing the graded filter. One localized area had to be stabilized by undercutting and replacing with riprap before placing the geogrid and graded filter. In addition, during tree clearing on the bank below Seep 3A, a 50-foot long area of increasing seepage issuing from the hydraulic fill of the lower bank was encountered. The area was first stabilized with riprap before placing reinforcing geogrid and a substantial graded filter with overlying rockfill buttress.

In February 2009 TVA had installed a toe-drain system along the outside toe of the southeast dike in the area identified as Seep 3A to collect and monitor the seepage. The toe drain consisted of perforated pipe enclosed in crushed stone and filter fabric. This collection system was removed during construction of the 2011 improvements that stabilized the area with biaxial geogrid before placing the new graded filter, which has no perforated pipe for collecting and monitoring the seepage.

An earlier improvement in 1996 included placing riprap on the exterior toe and lower slope below the toe access berm of the northwest dike, to control erosion by waves and currents from Kentucky Lake water level fluctuations.

4.2 SUMMARY OF OPERATIONAL PROCEDURES

4.2.1 Original Operational Procedures

Furnished documents do not include the original operational procedures. The AADA is a man-made basin designed and operated primarily to contain fly ash, boiler slag/bottom ash, ash sluice water, storm water, and plant process water. It is presumed that the original pond of the AADA was operated as a wet pond wherein CCR wastes were transported and placed by sluicing with water into the pond, where the suspended particles were allowed to settle out and the water detained temporarily in the pond for neutralization and equalization prior to discharge through the original six gravity-flow overflow structures. It is further presumed that interior ditches/swales were maintained to promote drainage.

4.2.2 Significant Changes in Operational Procedures since Original Startup

The manner of transporting and placing the fly ash and bottom ash/boiler slag into the AADA by the wet sluicing method has basically not changed since original startup. A significant change in operational procedures since original startup is the use of the AADA as a temporary storage area for ash received. The approximate date this practice started was probably in the early- to mid-1980s, when the first dredge cells were developed at the now closed South Railroad Loop Disposal Area. The practice of stacking in temporary stockpiles began in January 2010.

4.2.3 Current Operational Procedures

The AADA receives sluiced fly ash, bottom ash/boiler slag, sluice water, storm water, and plant process water. Currently, the ash is excavated from the sluicing channel, initially dewatered in a working area next to the sluicing channel, and then stacked in two temporary stockpiles. In the summer months the ash in the temporary stockpiles is loaded into dump trucks and hauled to a permitted landfill. Ash that bypasses the dredging operation in the sluicing channel settles in the ash pond complex at the southern end of the AADA. The settled ash in the ponds is removed by a suction dredge that discharges the ash to a dredge cell in the northern part of the AADA. This activity maintains proper function of the ponds and sufficient volume of water for treatment purposes in accordance with NPDES requirements. Current operational procedures are described in Section 8.1.

4.2.4 Other Notable Events since Original Startup

There appear to be no other notable events since original startup.

5.0 FIELD OBSERVATIONS

5.1 PROJECT OVERVIEW AND SIGNIFICANT FINDINGS

Dewberry personnel Stanley W. Notestine, P.E. and Fred Tucker, P.E. performed a site visit on Tuesday, September 20, 2011 in company with the participants listed in Subsection 1.3.1.

The site visit began at 09:00 AM. The weather conditions during the visit were cloudy with mild temperatures. Ground conditions were still wet from relatively heavy rainfall the previous day. Photographs were taken of conditions observed. Please refer to the "Coal Combustion Dam Assessment Checklist Form" in Appendix B Doc 14. Selected photographs are included here for ease of visual reference. Digital photographs were taken by Dewberry personnel during the site visit and provided to TVA.

The visual assessment of the perimeter dike and new spillway was that they were in satisfactory condition; no significant deficiencies were observed.

5.2 EARTH EMBANKMENT

5.2.1 Crest

The crest of the AADA perimeter dike was observed to be surfaced with crushed stone and accessible with rubber-tired vehicles. The crest along all the major segments, including the southeast, southwest, northwest, and northeast dikes, is shown in Photos 5.1 through 5.4. The perimeter dike crest was observed to be in good condition with only minor surface indentations and some minor rill erosion along the edges. No major depressions (caused by settlement), sags, tension cracks, or other signs of significant settlement or mass soil movement were observed.

Photo 5.1: Crest and inside slope along southeast dike, looking southwest. Note adjacent Pond B to the west (right in photo).

Photo 5.2: Crest and inside slope along northern part of southwest dike, looking north. Note adjacent Pond A.

5.2.2 Inside Slope and Interior (Disposal) Area

The visible parts of the inside slopes of the AADA perimeter dike were observed to be in satisfactory condition. Most of the inside slopes in the active disposal area is buried with ash. No areas of major erosion due to surface runoff or wave action and no obvious signs of slumps, slides, bulges, tension cracks, or animal holes were observed (see Photos 5.1 through 5.4). No woody vegetation was observed on the inside slopes, although some tall weeds were observed, particularly adjacent to the ponds in the southern part of the AADA (e.g., see Photo 5.1). Interior views of the AADA are shown in Photos 5.5 through 5.8, showing the west part of the sluicing channel (Photo 5.5), the south ash stockpile area (Photo 5.6), the north ash stockpile area (Photo 5.7), and the sluicing channel where the ash sluice lines discharge into it at the east end (Photo 5.8). No unstable stockpiles were observed. In fact, the north stockpile was observed to be mostly depleted from summer hauling operations to the landfill (see Photo 5.7). No unusual conditions (e.g., sinkholes) were obvious in the interior area. A view of the ash sluice lines and plant sump line extending across the causeway to the plant is shown in Photo 5.9, and a view of the typical route of the ash sluice lines extending from a pipe chase to the plant is shown in Photo 5.10. Some of the ash sluice lines (older ones) appeared to be flanged steel pipe and some (newer ones) were observed to be HDPE pipes. The exterior of the steel pipes was observed to be somewhat rusty but generally sound; near the discharge point a couple of the steel pipes were observed to have rust scale. No obvious leaks were observed. A coating of dry ash around the ends of the sluice lines suggests that the water level in the sluicing channel has been higher than observed at the time of the site visit.

Photo 5.3: Crest and inside slope/ interior area along northwest dike, looking northeast.

Photo 5.4: Crest and inside slope along northeast dike, looking southeast.

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Photo 5.5: Interior view toward south stockpile area, looking southeast.

Photo 5.6: Interior view of sluicing channel and long-reach excavators, looking east.

Photo 5.7: Interior view of north stockpile area, looking northeast. Note most of stockpile has been removed by summer hauling to permitted landfill; note marker poles to gauge stockpile height.

Photo 5.8: View of ash sluice lines discharge location at sluicing channel, looking west. Note plant sump line (largest) discharges through pipe to Ash Pond Complex.

Photo 5.9: View of identified ash sluice lines and plant sump line extending back to plant across causeway, looking east.

Photo 5.10: View of ash sluice lines extending from one of plant pipe races, looking west.

5.2.3 Outside Slope and Toe

The recently reconstructed outside slope and toe berm/buttress along the southeast dike is shown in Photos 5.11 and 5.12. Former seep areas are covered with filtered drainage blankets under the new berm and are not visible. It was observed that grass turf was used to establish permanent erosion protection of the new embankment slope. Some of the grass turf appeared to be "stressed." TVA personnel indicated that the non-viable turf will be replaced. Only minor surface erosion was observed on the new slope, such as shown in Photo 5.13. Some minor rill erosion was observed on the edges of the stone surfacing of the berm, particularly on the outer edge at the southwest end. A minor depression holding apparent surface runoff was observed at the base of the new slope and in the surface of the toe berm as shown in Photo 5.14.

The outside slope and toe area of the north part of the southwest dike is shown in Photo 5.15. The toe area had recently been cleared of trees. The re-graded slope was covered with grass turf, which had not yet become well-established, as shown by the yellowish color of the grass in Photo 5.15. Some minor bare areas were observed on this slope; the worst one is shown in Photo 5.16.

Photo 5.11: Outside slope and toe berm along southeast dike, looking southwest. Note remnant of original steep slope at top.

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Photo 5.12: Riprap buttress of toe berm along outside slope of southeast dike, looking southwest.

Photo 5.13: Small eroded area on outside slope of southeast dike.

Photo 5.14: Slight depression with some trapped water (apparent runoff) at base of slope and in surface of toe berm of southeast dike.

Photo 5.15: Outside slope and toe area along north part of southwest dike, looking south. The toe area had recently been cleared of trees. Discolored grass is recently placed sod.

Photo 5.16: Slightly eroded bare area on outside slope of southwest dike is above recently sodded area.

Minor bare spots were observed on the outside slope of the northwest dike; Photo 5.17 is representative. The riprap on the outside slope below the access berm along the lower part of the northwest dike was observed to have some relatively tall weeds and minor bushy vegetation, most of which appeared to have been treated with herbicide. During the flooding in May 2011, high water in Kentucky Lake rose above the riprap-protected lower part of the northwest dike outside slope and caused erosion of the embankment slope above the berm. Two separate areas were repaired with riprap. One of these is shown in Photo 5.18. The one shown is at a higher elevation than the other, and it appears that the damage at this location actually was caused by haul trucks on the narrow access berm down to the other repair site. The repairs were observed to be satisfactory. Some erosion was noted on a gravel-surfaced access ramp down the outside slope at the northern tip of the perimeter dike, as shown in Photo 5.19.

The recently reconstructed outside slope and toe berm/buttress along the northeast dike is shown in Photo 5.20. As at the southeast dike, former seep areas are covered with filtered drainage blankets under the new berm and are not visible. The area of repair of the shallow slough observed in the steep upper part of the slope during Stantec's inspection in June 2011 is shown in Photo 5.21. The repair simply involved removing the steep remnant of the old slope at the top where the slough occurred, which resulted in flattening the upper part to generally match the new slope below. Small apparent seeps with no discernable flow were observed along the toe of riprap at either end of a new gabion retaining wall that exists near the south end of the outside slope of the northeast dike (just north of causeway). One of these, shown in Photo 5.22, has a rust-colored growth or deposit, suggesting iron bacteria or possibly clay fines. The other seep was observed to have some green algae growth in it. Because of the presence of what appeared to be iron bacteria at one and green algae at the other, the wet areas appeared to be persistent and not just drainage of recent rainfall runoff from the riprap. Nevertheless, these apparent seeps appeared to be minor.

All the conditions observed along the outside slope and toe areas of the perimeter dike are minor maintenance-type concerns. No areas of major erosion and no obvious signs of slumps, slides, bulges, tension cracks, significant seepage, or animal holes were observed. No significant woody vegetation was observed on the outside slopes.

Photo 5.17: Outside slope of northwest dike above access berm showing bare strip, looking southwest.

Photo 5.18: Outside slope and toe area of northwest dike showing one of two areas repaired with riprap after being eroded by elevated Kentucky Lake level during flooding in May 2011, looking northeast.

Photo 5.19: Eroded access ramp on outside slope at northern tip of perimeter dike, looking south.

Photo 5.20: Outside slope and toe berm along northeast dike, looking northwest. Note remnant of original steep slope at top.

Photo 5.21: Area of repair of small slough observed earlier this year in steep part at top of the northeast dike outside slope.

Photo 5.22: Gabion retaining wall near south end of northeast dike outside slope. Small apparent seeps with no discernible flow observed along toe of riprap at each end of gabion wall.

5.2.4 Abutments and Groin Areas

Since the AADA is formed within a ring dike system, there are no natural abutments. However, no significant erosion or displacements were observed where the access causeway embankment intersects the perimeter-dike embankment on the east side or at the inside bends in the perimeter dike.

5.3 OUTLET STRUCTURES

5.3.1 Overflow Structures

The visible part of the new spillway overflow structures located at Pond C next to the southwest dike are shown in Photo 5.23. The water level in the pond appeared to be at the new normal operating water elevation (384.6 feet). The six abutting concrete overflow structures fitted with adjustable weirs of stop logs were observed to be in good condition. The concrete structures appeared sound. No significant corrosion was observed on the metal grating, skimmer wall, or other metal parts, although a "scum line" has formed on the skimmer wall at the normal water elevation.

Photo 5.23: View of six abutting concrete box overflow structures of new spillway fitted with stop logs, looking north. Note corrugated metal skimmer wall and adjacent Pond C.

A view of the tops of the old grout-filled overflow risers in Pond C to the south of the new overflow structures is shown in Photo 5.24. As previously noted, there are six other grout- or concrete-filled risers; three are located at the southeast dike and three are located at the northwest dike. No obvious problems with any of the abandoned overflow structures were observed, and no issues with any of them have been reported since their closure.

Photo 5.24: View of tops of old abandoned (grout-filled) overflow structures in Pond C next to southwest dike.

5.3.2 Outlet Conduits

Water that overflows the six new overflow structures discharges through the six new 30-inch diameter HDPE conduits that pass through the southwest dike. These conduits serve as the primary outlet for the AADA. The water discharges into an energy dissipater before flowing down a riprap-lined apron to Kentucky Lake. Water was discharging from these primary outlet conduits at the time of the site visit, as shown in Photo 5.25. The concrete endwall and energy dissipater appeared to be in good condition with no obvious undermining. The tops of air vents installed for each conduit to prevent surging flow from entrapped air are shown near the top of the southwest dike in Photo 5.26; they appeared to be functioning properly. No sinkholes or dropouts were observed in the embankment over the conduits. The riprap-lined apron appeared to be sound with no obvious areas of eroded and displaced riprap.

Photo 5.25: View of outlet ends of the six conduits of the new primary spillway discharging into energy dissipater, looking north. Note siphon pipes beyond.

Photo 5.26: View of outlet ends of the four new siphon pipes. Note air vents near top of southwest dike for the six new conduits of the primary spillway.

The outlet of the grout-filled abandoned outlet conduits associated with the previously described abandoned overflow risers were not visible in the field. However, no obvious indications of past sinkholes or dropouts were observed along the general alignments of the conduits through the perimeter dike at their respective locations, and no issues with any of the abandoned conduits have been reported since their closure.

Photo 5.27: View of flow to Kentucky Lake along riprap apron below the energy dissipater, looking northwest.

5.3.3 Low Level Outlet (Siphons)

There is no low level outlet. However, the four 18-inch diameter HDPE siphon pipes that were installed to lower the water level in the ash pond complex during construction of the new spillway will remain in place to provide a means of emergency drawdown of water in the ponds. The relative location of the siphons is shown in Photo 5.25, and the discharge ends of the siphon pipes with gate valves are shown in Photo 5.26. The siphons and associated gate valves and hardware were observed to be in satisfactory visual condition.

6.0 HYDROLOGIC/HYDRAULIC SAFETY

6.1 SUPPORTING TECHNICAL DOCUMENTATION

6.1.1 Flood of Record

No documentation has been provided about the maximum water surface elevations in Active Ash Disposal Area. The AADA is contained within a perimeter dike and does not receive off-site natural drainage. Therefore, it does not receive flood inflows from off-site. The source of water into the AADA, aside from sluicing water, plant drainage, and Coal Yard runoff, is precipitation that falls directly into the AADA. 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. At New Johnsonville 15.87 inches of rain were recorded in the 48-hour period, and at nearby Camden 19.41 inches were recorded. According to an “Average Recurrence Intervals Map for 48-Hour Duration,” prepared by the Hydrometeological Design Studies Center, Camden is in a location that experienced rainfall having an average recurrence interval exceeding 1000 years, and New Johnsonville is at a location at the upper end of a the 500- to 1000-year recurrence interval. In addition, significant flooding of Kentucky Lake occurred in early May 2011. The lake level rose above the access berm along the lower part of the northwest dike and caused some erosion of the outside slope, as previously described.

6.1.2 Inflow Design Flood

For the “small” size and “significant” hazard potential classification assigned to the AADA dike, 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 100-year frequency to 1/2 Probable Maximum Flood (1/2 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 has performed a hydrologic and hydraulic (H & H) analysis of the AADA. The analysis is summarized in their memo titled "Hydrologic and Hydraulic Calculations Summary" (H & H memo) dated September 28, 2010 (see Appendix A, Doc 09). Stantec's analysis evaluated the performance of the AADA ash pond complex for the 6-hour PMP. 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)	87	87
Dam Crest El (ft)	390	390
Normal Pool El (ft)	387.5	384.6
Normal Freeboard (ft)	2.5	5.4
Design Storm Max Pool El (ft)	Overtops	388.7
Min Freeboard During Design Storm (ft)	None	1.3

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

6.1.3 Spillway Rating

Stantec's H & H memo indicates that spillway rating curves were developed for the existing (old) spillways (for the pre-design analysis), but they are not included in the memo. The spillway rating for the new spillway, which has replaced the now grout-filled old spillways, appears to be represented by "paired data" that includes a storage-discharge relationship and an elevation-storage relationship in the HEC-HMS Input Files accompanying the memo (see Appendix A, Doc 09).

6.1.4 Downstream Flood Analysis

No downstream flood analysis has been provided for the AADA. A general qualitative analysis based on field observations and review of available data follows.

Failure of the AADA perimeter dike through either the northwest or southwest dikes would release water and ash carried with the water to impact primarily Kentucky Lake. For the ash from the northern part of the AADA to travel far through a breach it would have to be over-saturated by prolonged wet-weather conditions prior to a breach occurring by whatever cause (either geotechnical or hydrologic/hydraulic). Failure through the northeast dike would impact the boat harbor and potentially disrupt coal delivery and unloading systems. Failure of the southeast dike would impact the intake channel and potentially block it. A failure would not likely cause loss of life.

6.2 ADEQUACY OF SUPPORTING TECHNICAL DOCUMENTATION

Although the furnished information is not detailed, the hydrologic/hydraulic documentation available for the AADA appears to be adequate.

6.3 ASSESSMENT OF HYDROLOGIC/HYDRAULIC SAFETY

For assessment purposes the appropriate design storm for the AADA may be taken as 1/3 PMP, 6-hour duration (see Subsection 6.1.2). Stantec apparently selected the design storm on the basis of "Intermediate" size and "High" hazard potential classifications for the AADA, which is conservative.

Stantec's analysis shows that the ash pond complex under current conditions with new spillway and other recently constructed improvements should be able to pass the full 6-hour PMP event without overtopping the perimeter dike. Therefore, on the basis of furnished hydrologic/hydraulic documentation, the AADA appears to have satisfactory hydrologic/hydraulic safety.

7.0 STRUCTURAL STABILITY

7.1 SUPPORTING TECHNICAL DOCUMENTATION

7.1.1 Stability Analyses and Load Cases Analyzed

TVA's consultant, Stantec, has performed geotechnical explorations and analyses of the Active Ash Disposal Area perimeter dike. Stantec's stability assessment included analyses of static slope stability, seepage/piping potential, and simplified seismic slope stability using the pseudo-static method. Computer software programs commonly used in the geotechnical profession were used to aid in the analyses. The exploration results and/or analyses results are presented in the following Stantec reports:

1. "Report of Geotechnical Exploration and Evaluation of Slope Stability Ash Disposal Areas 2 and 3 (Active Ash Disposal Area)" dated April 13, 2010.

2. "Results of Pseudo-Static Slope Stability Analysis Active CCP Disposal Facilities - BRF, COF, GAF, JSF, JOF, KIF, PAF, and WCF" dated February 15, 2012.

The slope stability analyses focused on stability of the exterior slopes at nine different sections of the perimeter dike. The load cases analyzed included:

1. Static steady-state seepage, ash pond normal pool el 384.6 ft (Performed on the nine different sections)
2. Earthquake w/ horiz seismic coef = 0.254g, normal pool el 384.6 ft (Performed on one critical section that had the lowest factor of safety under static loading: Section K)

The load case initially analyzed for the perimeter dike before remedial improvements were made included only Case 1 above with the then normal pool elevation of 387.5 feet and up to 390 feet in sections at the sluicing channel. However, unsatisfactory factors of safety were obtained for the southeast dike and northeast dike in both the slope stability analysis and the seepage analysis. Consequently, remedial improvements were made in the four stages described in Subsection 4.1.3, to lower the normal water level in the ponds, lower the phreatic surface, and increase stability of the outside slopes of the southeast dike and northeast dike by physical modifications.

The various static stability analysis sections for the original dike sections are included in Appendix A Doc 06 for reference. (The analysis sections for the final slope geometries after improvements were not provided.) The pseudo-static analysis is summarized and illustrated in Stantec's February 15, 2012 report, applicable parts (Appendix A Doc 10).

7.1.2 Design Parameters and Dam Materials

The perimeter dike embankment soils consist of predominantly clay for the two dike raises and material identified as "fill" for the original dike embankment that was placed hydraulically; based on Stantec's test borings, it appears that the original dike consisted of predominantly clays and some silts. The upper (second) dike raise embankment was partly founded on ash. A relatively thick layer of alluvial clay and silt underlies the perimeter dike and extends down to a deeper layer of alluvial sand and gravel. 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 static stability analyses were as shown in the following Table 7.1:

Table 7.1: Design Properties and Parameters of Materials used in the Static Stability Analyses

Material	Unit Wt. (pcf)	Effective Stress Parameters	
		C´ (psf)	Ø´ (deg)
Ash	100	0	22
Upper Clay Dike	125	200	29
Lower Clay Dike	125	100	29
Fill	124	50	39
Alluvial Clay & Silt	124	100	30
Alluvial Sand & Gravel	120	0	30
Riprap	100	0	38

See analysis sections in Appendix A Doc 06 for source of information in this table.

Design properties and parameters used in the pseudo-static slope stability analyses were as shown in the following Table 7.2:

Table 7.2: Design Properties and Parameters of Materials used in the Pseudo Static Stability Analyses

Material	Unit Wt. (pcf)	Undrained Strength Parameters	
		C (psf)	Ø (deg)
Ash	100	0	10
Upper Clay Dike	125	521	16.2
Lower Clay Dike	125	533	20.1
Fill	124	630	17.8
Alluvial Clay & Silt	124	714	17.8
Alluvial Sand & Gravel	120	0	30
Riprap	100	0	38

See analysis section in Appendix A Doc 10 for source of information in this table.

7.1.3 Uplift and/or Phreatic Surface Assumptions

The phreatic surface in the embankment slope stability analysis sections was assumed to extend through the embankment section in a step fashion through the two dike raise embankments down from the pond water elevation to the lake elevation at the toe of the embankment section (see analysis sections in Appendix A, Doc 06 and Doc 10).

From visual observations in the field, the phreatic surface was not observed to crop out on the outside slope of the various segments of the perimeter dike. The small non-flowing seeps noted at the toe of riprap at either end of the gabion retaining wall near the south end of the northeast dike possibly are associated with the phreatic surface.

7.1.4 Factors of Safety and Base Stresses

The computed factors of safety for the load cases analyzed in the slope stability analyses of the perimeter dike are shown in the Table 7.3 for the most critical sections. Conventional minimum FS criteria are 1.5 for static long-term stability and 1.0 for earthquake stability (by pseudo-static method).

Seepage exit gradients were computed and compared with the critical gradient (1.0 to 1.22, depending on location) to calculate a factor of safety against piping ($FS_{piping} = i_{crit}/i$). The minimum computed $FS_{piping} > 4$ for the more critical analysis sections of the improved southeast and northeast dikes. This is an increase from the minimum $FS_{piping} = 2.5$ calculated for the most critical section (Section B-B' at the northeast dike) prior to improvements. The minimum computed $FS_{piping} = 3.6$ for the most critical section of the southwest dike (Section I-I') apparently remained unchanged, even though the normal pool elevation has been lowered. Stantec adopted a target minimum factor of safety criterion of 4.0 against piping for the improved dikes. 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.

Table 7.3: Slope Stability Factors of Safety (Outside Slope) – Most Critical Section[1, 2]

Load Case	Calculated Minimum Factor of Safety (FS)	
	Original	**Current (Stage 4)**
1. Static Steady State	1.2 Deep 1.1 Shallow	1.6 Deep 1.6 Shallow
2. Earthquake - 0.254g Horiz Seismic Coef (Conservatively taken as the PGA for the 2,500-yr Return Period Event)	Not Analyzed	1.00

[1]For static stability Section C-C' (Northeast Dike) is most critical. [2]With implementation of Stage 4 Section K-K' became the critical slope for seismic slope stability. *Sources: Stantec reports dated April 13, 2010 and October 3, 2011*

7.1.5 Liquefaction Potential

No liquefaction potential analyses have been provided. 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 "White Paper - Seismic Risk Assessment Closed CCP Storage Facilities" (White Paper) prepared by Stantec and included in Appendix A Doc 10 for reference. It is understood that Phase A of the seismic risk assessment study is currently in draft report form and is being actively peer-reviewed and that the Phase A report 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.

Stantec's test borings indicate that dike embankments are generally firm and compacted. However, many of the borings indicate the presence of very soft to soft clays and some very soft to soft silts in the lowest part of the embankment and in the foundation layer immediately underlying the dike embankment. In addition, the upper dike-raise embankment is partly founded on sluiced ash, some of which is very loose, according to a number of Stantec's test borings. Thus, it appears there could be a potential for liquefaction and/or some potential for excessive deformation to occur during strong seismic shaking. Evaluation of these conditions will require a quantitative analysis to determine the amount of potential deformation and its effects on the integrity and stability of the dike embankments. Based on currently available information, it is concluded that liquefaction potential and excessive deformation potential under seismic loading are unknown.

7.1.6 Critical Geological Conditions

The Active Ash Disposal Area is located on recent alluvium of the Tennessee River floodplain, which is largely inundated by Kentucky Lake. Based on geologic and subsurface information related in Stantec's report (April 13, 2010), the alluvium consists of fine-grained silt and silty clay that grade into sand and river gravel with increasing depth. Based on foundation drilling for the U.S. Highway 70 bridge the alluvium was found to range up to 67 feet in depth and to average 60 feet in depth beneath the former floodplain surface. In a groundwater monitoring well drilled at the AADA in 1986 bedrock was encountered at an elevation of 290 feet, which was indicated to be about 100 feet below the dike (presumably below the crest). This boring encountered a 40-foot thick layer of sand and gravel, presumably in the lower part of the alluvial soil profile. The alluvium is indicated to be underlain by Devonian-age Chattanooga Shale in turn underlain by the Camden Formations. The Chattanooga Shale is described as a fissile, bituminous, carbonaceous shale. It was noted to likely be thin to nonexistent beneath the AADA. The Camden formation was noted to consist of thin (from 1 to 3 inches thick) beds of cherty limestone and to contain hard, dense, brittle, white chert pieces, separated by softer gritty clay layers. Stantec's geologic information was referenced to a John Kellberg's report "Geology of the New Johnsonville Steam Plant Site," 1948.

The main hazard associated with the geology of the area is the potential for the presence of very soft soils that may behave unsatisfactorily under certain cases of loading, particularly seismic loading. As previously mentioned, many of Stantec's test borings penetrated very soft to soft alluvial soils immediately beneath the perimeter dike embankment and in the lower part of the embankment.

Seismicity – The Johnsonville Fossil Plant is located near the east edge of the New Madrid Seismic Zone. This zone is an area considered to have high seismic hazard, based on the historical record of strong earthquakes occurring in this area. Near the edge of this zone, where the plant and Active Ash Disposal Area (AADA) are located, the seismic hazard is considered to be moderate. From the USGS Interactive Deaggregation website, based on the USGS Seismic-Hazard Maps for Central and Eastern United States, dated 2008, the AADA is at a location anticipated to experience 0.270g peak (horizontal) ground acceleration (PGA) with a 2-percent probability of exceedance in 50 years (2,475-year exceedance return time, often rounded to 2,500 years), assuming uniform firm-rock site conditions, i.e., a site with average shear wave velocity of 2,500 feet per second (fps) in the upper 100 feet below the ground surface.

TVA uses seismic hazard results from the TVA Dam Safety Seismic Hazard Model developed by AMEC Geomatrix, 2004. Values of PGA from this model for the JOF are 0.254g for 2,500-year exceedance return time and 0.096g for the 500-year exceedance return time, which is approximately the expected return time for New Madrid earthquakes (450 years). The TVA values are based on "hard rock" rather than the "uniform firm-rock" site conditions assumed for the USGS Seismic-Hazard Maps. According to TVA's documentation, the hard rock to uniform firm rock amplification factor for PGA is 1.52. Therefore, the TVA PGA values would need to be multiplied by this amplification factor to compare with the USGS PGA values. Using this factor, the "uniform firm-rock" values estimated from TVA's "hard rock" values are higher than the values obtained for the JOF site from the USGS Interactive Deaggregation website.

7.2 ADEQUACY OF SUPPORTING TECHNICAL DOCUMENTATION

Structural stability documentation for the Active Ash Disposal Area perimeter dike is generally adequate with respect to static slope stability and seepage analysis. A flood surcharge case was not analyzed, but it is not expected that the clay embankment slope stability factor of safety would fall below an acceptable FS = 1.4 (criterion per USACE EM 1110-2-1902). A case of rapid drawdown of a maximum flood lake level on the outside slope also was not analyzed, but likewise, for the clay embankment it is not expected that the slope stability factor of safety would fall below an acceptable FS = 1.1 (criterion per USACE EM 1110-2-1902). In fact, the slope experienced a record flood elevation from Kentucky Lake in May 2010, and the drawdown of the subsiding water after reaching its peak apparently caused no drawdown failures on the outside slope, although some erosion damage was caused on the embankment slope above the riprap-protected lower part of the embankment, as previously mentioned.

The recently completed documentation for seismic stability of the AADA perimeter dike embankment is generally adequate. The peak (horizontal) ground acceleration (PGA) value of 0.254g for the 2,500-year exceedance return time from the TVA Dam Safety Seismic Hazard Model (2004) was adopted as the "design" horizontal seismic coefficient (k_h), but there appears to be no clear explanation of why the hard rock PGA was used as the k_h in the pseudo-static slope stability analysis. Use of the "hard rock" PGA as the seismic coefficient for analysis of a dike founded on thick alluvial soil deposits seems inappropriate. In addition, the seismic coefficient used in pseudo-static stability analyses is actually some fraction or percentage of the PGA, depending on earthquake magnitude and other factors. For example, one correlation in the literature shows k_h ranging from 0.2 x PGA for magnitude 6.5 earthquake to 0.5 x PGA for magnitude 8.25 earthquake. In the White Paper provided by TVA it is indicated that k_h = 0.1 x PGA would be used in pseudo-static stability analyses in the closure design, which seems unconservative in view of the above correlation. However, for analysis of the JOF AADA perimeter dike embankment, k_h = 1.0 x PGA was actually used. This on the one hand appears very conservative, but on the other hand use of the "hard rock" PGA seems unconservative. To help gage the conservativeness or unconservativeness of the seismic coefficient used in the analysis, it was compared with the value indicated on the old Seismic Zone Map shown in the USACE's "Recommended Guidelines for Safety Inspection of Dams," dated September 1979. The map gives tabulated values for the seismic coefficient associated with each seismic zone. The AADA site is located in Seismic Zone 2 near the boundary of Seismic Zone 3. For Seismic Zone 2 it is 0.05 (to be multiplied by the acceleration of gravity, g) and for Seismic Zone 3 it is 0.10. Thus, based on the old USACE reference, the selection of a design seismic coefficient of 0.254 is conservative. (Note: The seismic coefficient actually is the empirical factor that is multiplied times the acceleration of gravity, g.) As noted previously, the 500-year exceedance return time is approximately the expected return time for New Madrid earthquakes. Adopting the 2,500-year return time as the “design basis” for a “Significant” hazard potential facility is considered conservative and in accordance with EPA’s policy regarding stability of CCR management units.

Pseudo-static factors of safety that meet the minimum factor of safety criterion (FS = 1.0), imply that the associated deformations should be tolerable and acceptable. However, the pseudo-static method of analysis is valid only when the embankment and foundation soils will not experience significant loss of shear strength during earthquake shaking. Therefore, the presence of very soft to soft soils in the lowest part of embankment and in the alluvial foundation underlying the embankment at the AADA is a potential concern, particularly if these soils are sensitive (i.e., soils that lose significant shear strength upon experiencing even small shear strains, such as may be produced by the disturbing force of seismic shaking). In addition, the upper dike-raise embankment is founded on sluiced ash. Where very soft clays, silts, and ash exist below the water table, they could potentially develop excess pore water pressures during strong earthquake shaking and experience loss of shear strength, causing deformations in the very soft foundation materials, which could detrimentally impact the overlying embankment. Thus, there is some inadequacy in the technical documentation due to the current lack of analyses or assessments of the effects of the very soft foundation soils and ash, as well as lack of assessment of liquefaction potential. It is noted, however, that for relatively low dikes such as the AADA perimeter dike, the standard of practice, particularly at the time they were developed, usually did not include seismic stability analyses, deformation analyses, or liquefaction potential analyses. It is further noted that TVA has indicated that liquefaction potential will be addressed as part of a comprehensive risk/consequences-based evaluation of seismic failure risks being conducted in final design for closure of the AADA in 2017.

7.3 ASSESSMENT OF STRUCTURAL STABILITY

The structural stability of the Active Ash Disposal Area perimeter dike embankment at the Johnsonville Fossil Plant in its current improved condition appears to be satisfactory with respect to static stability based on the following:

- Documented static slope stability analyses showing satisfactory factors of safety against both deep and shallow potential circular arc shear failures under steady state seepage loading condition.
- Documented seepage analyses and evaluation of exit gradients showing satisfactory factors of safety against a piping failure.
- No indications of scarps, sloughs, major depressions or bulging anywhere along the slopes of the dike.

- No indications of boils, sinks, or uncontrolled seepage along the outside slope or toe of the dike.

- No major depressions and no significant vertical or horizontal alignment variations in the crest of the dike.

The furnished pseudo-static slope stability analysis results show that the AADA perimeter dike embankment has an acceptable factor of safety with respect to seismic stability under the design earthquake, i.e., the 2,500-year return period event. Using the "design" seismic coefficient $k_h = 0.256g$, which overall appears to be conservative, the perimeter dike embankment is shown to have FS = 1.0, which meets the acceptance criterion (FS = 1.0). New Madrid earthquakes have a return period on the order of 450 years. Thus, the AADA dike is indicated by the pseudostatic method of analysis to be safe (at its location) for known source earthquakes much greater than the New Madrid Earthquakes.

As previously concluded, liquefaction potential and excessive deformation potential under seismic loading are unknown. Based on review of the available boring information, the soils (clays and silts) encountered in the borings do not appear to be of the type that would be highly susceptible to liquefaction. However, very loose ash may be susceptible to liquefaction, and it appears from the review of the test borings that there are some significant zones of very loose fly ash and bottom ash under the upper dike raise embankment, particularly along the southwest dike and southern part of the northwest dike. Therefore, qualitatively, there could be potential for failure of the upper inside portions of these dikes due to liquefaction under earthquake loading. In addition, qualitatively, there could be some potential for excessive deformation to occur in the underlying very soft clays and silts under earthquake loading.

The overflow structures and outlet conduits of the new spillway appeared to be in sound and stable condition with no evidence of structural deterioration of the limited visible parts of the structures that could be seen. The concrete energy dissipater and riprap-lined channel appeared to be sound with no undermining or erosion. The metal parts and hardware at the overflow structure appeared to be sound and generally free of corrosion.

8.0 ADEQUACY OF MAINTENANCE AND METHODS OF OPERATION

8.1 OPERATING PROCEDURES

The Active Ash Disposal Area receives sluiced (pumped) ash slurry from both the bottom ash/boiler slag handling system and the fly ash handling system. Pairs of the 10 coal-fired boilers at the plant share two sluice pipes, one for fly ash and one for bottom ash/boiler slag, extending from the plant across the causeway to the east side of the AADA (total of 10 sluice lines), where they discharge into a sluicing channel (see Photo 5.8). There is an additional line that carries plant process water to a bypass inlet at the sluicing channel; the bypass drains directly to the ponds, to reduce the volume of water in the sluicing channel. Coal Yard runoff also is pumped to the AADA.

At the sluicing channel long-reach excavators scoop out most of the ash, and after initial draining of excess water in a working area next to the sluicing channel, the ash is stacked at a higher level on the northern part of the AADA to drain further while awaiting removal. Heavy equipment, such as bull dozers, scrapers, compactors, is used in the stacking operation. In the summer the dried ash is loaded into dump trucks and hauled to a permitted landfill in Camden, TN (former Bevins Quarry). In the winter when the ash loading and hauling is not feasible the ash is stacked in two separate stockpiles, north winter stockpile (north of sluicing channel) and south winter stockpile (south of sluicing channel), both in the northern portion of the AADA. The stacking is done within setbacks from the existing sluicing channel (130 feet to provide an area for initial dewatering before stacking) and the abandoned sluicing channel (40 feet). Boundary markers delineate the toe limits of the north stockpile. The top elevation of the ash stack is generally limited to 390 feet, except in the winter when the stacking may reach a maximum elevation of 405 feet. Side slopes of the stacked ash are maintained at 3H:1V, and a 20-foot wide bench is maintained at elevation 400 feet on the north and east sides of the north stockpile.

An ash-pond complex of three cells in the southern 40 percent of the AADA accumulates sluice water and storm water prior to discharge through the new spillway system. The ponds are dredged every two years with a suction dredge to remove ash that does not get removed by the removal operations at the sluicing channel. The dredged material is piped to a temporary dredge cell located on the north side of the ash disposal area. The cutter head at the end of the suction dredge pipe is restricted from getting closer than 100 feet from the centerline of the perimeter dike. Marker buoys are used as a visible reference to the set-back line for the dredge operator. Water trucks are used as necessary to control dust. The ash excavation, drying, and hauling operations are contracted. TVA's written operations procedures are included in Appendix A Doc 11.

The normal water level in the ash-pond complex in the southern part of the AADA is now maintained at elevation 384.6 feet, after spillway improvements were made, which allows for at least 5.0 feet of freeboard. Water discharges at the spillway outlet are monitored according to NPDES Permit requirements.

8.2 MAINTENANCE OF THE DAM AND PROJECT FACILITIES

Maintenance of the dike embankments and outlet works of the AADA, and essential operating equipment, such as the piping (ash sluice lines), pumps, and other equipment (e.g., gates, valves, etc.), are performed as needed, as determined by routine inspections performed by plant personnel. Vegetation on the embankment slopes is scheduled to be mowed at least three times during the growing season. Any woody vegetation is removed. Erosion repairs are made and animal holes filled as needed. TVA's written maintenance procedures are included in Appendix A Doc 11. TVA also follows written guidelines for repair of routine maintenance problems, such as gully and rill erosion repair, burrow repair, wave erosion repair, etc., as shown in Appendix A Doc 12.

8.3 ASSESSMENT OF MAINTENANCE AND METHODS OF OPERATIONS

8.3.1 Adequacy of Operating Procedures

Based on field observations and review of operations pertaining to CCR containment, operating procedures at the active AADA appear to be adequate.

8.3.2 Adequacy of Maintenance

Maintenance of the impounding embankments and outlet works of the AADA appears to be adequate. No major maintenance issues were noted from review of the latest annual dike inspection report.

9.0 ADEQUACY OF SURVEILLANCE AND MONITORING PROGRAM

9.1 SURVEILLANCE PROCEDURES

TVA has a program of conducting, daily, weekly, monthly, quarterly, and annual inspections of the Active Ash Disposal Area. The inspections are documented with checklist forms and written reports. Any deficiencies requiring correction or maintenance are reported and tracked. The Seepage Action Plan previously mentioned is used to track seeps and determine the level of repair necessary. In summary:

- Daily inspections are conducted by the on-site Contractor and documented in a Daily Field Report.
- The weekly inspections are carried out by the Field Supervisor and documented on a Weekly Facility Observation Form.
- The monthly inspections are conducted by the Construction Manager and documented on a Monthly/Quarterly/Special Facility Inspection Form.
- The quarterly inspections are performed by the Routine Handling Operations and Maintenance (RHOM) team led by the RHOM Manager and documented on the Monthly/Quarterly/Special Facility Inspection Form. Conditions requiring engineering recommendations are reported to Coal Combustion Products (CCP) Engineering or to a geotechnical engineer to provide recommendations for the repair.
- Unscheduled inspections are also performed after special events such as heavy rainfall and earthquake and documented on the Monthly/Quarterly/Special Facility Inspection Form.
- The annual inspections focus on structural integrity and are performed by a qualified geotechnical engineer (e.g., Stantec) under the responsibility of CCP Engineering. The inspection includes both active and inactive ash disposal areas, including closed disposal areas (i.e., South Railroad Loop Ash Disposal Area, Ash Dredge Pond East of Gas Turbines, and North Abandoned Ash Disposal Area A). The annual inspection is documented in a written report. Recommendations for any needed repairs or maintenance or needed studies are included in the annual report.

TVA's written inspection and reporting procedures are included in Appendix A Doc 11.

9.2 INSTRUMENTATION MONITORING

Dam performance monitoring instrumentation includes 32 piezometers in place along the crest and toe of the perimeter dike around the AADA and 4 slope inclinometers installed from the crest of the northeast (2), southeast, and northwest dikes. The piezometers were installed in many of the test borings made by Stantec during geotechnical explorations in February-April 2009 as part of the Phase 2 studies. The inclinometers were installed in August-September 2009, although it appears that a replacement inclinometer was installed in February 2010. The locations of the piezometers and inclinometers are shown on the Instrumentation Plan included in Appendix A, Doc 05. The piezometer water levels and inclinometers are typically measured monthly. The piezometer water-level readings and elevations for the approximately 2.5-year period of record from March 30, 2009 to August 1, 2011 are tabulated in Appendix A, Doc 13. The piezometer water levels appear to have gradually dropped to lower elevations after the normal water level in the ash-pond complex was lowered as a result of the spillway improvements. The piezometer water levels also appear to have fluctuated up and down at the lower elevations, depending on seasonal variations in rainfall and water level in Kentucky Lake, which is lowered 5 feet to approximately elevation 354 feet in winter. The furnished record of the inclinometer readings (November 2009 to March 2010 and March 2011 to July 2011), included in Appendix A Doc 13, indicates no notable magnitude or trend of movement in the axis transverse to the slope or the axis parallel to the crest. The very small recorded movements tend to fluctuate back and forth.

Visual monitoring for seep areas is performed and documented in a Seepage Log. Any needed actions are taken according to the Seepage Action Plan.

9.3 ASSESSMENT OF SURVEILLANCE AND MONITORING PROGRAM

9.3.1 Adequacy of Inspection Program

TVA's inspection program for the AADA dikes is appropriate and adequate. No major safety issues were noted in the last annual inspection report (see discussion in Section 3.1). Areas of concern noted in Stantec's Phase 1 assessment in early 2009 have been remediated through extensive improvements constructed since then.

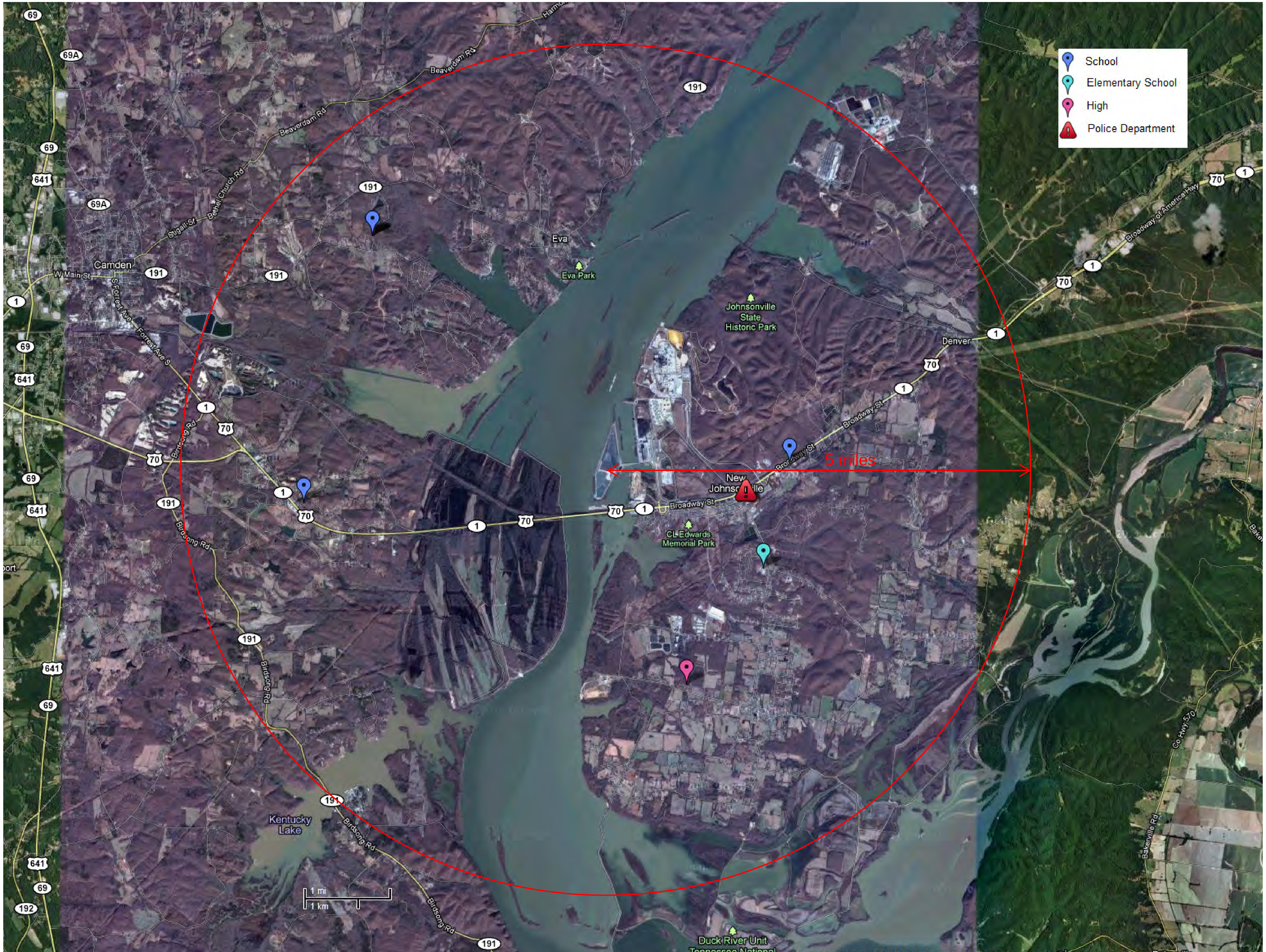
9.3.2 Adequacy of Instrumentation Monitoring Program

The instrumentation monitoring program is adequate. No problem or suspect condition, such as excessive settlement, major seepage, shear failure, or displacement was observed in the field that might be reason for installation of additional or different instrumentation. In the absence of stability problems or major seepage issues, there is no need for additional performance monitoring instrumentation at this time.

APPENDIX A

Document 1

Johnsonville Fossil Plant Aerial Vicinity Map and 5-Mile Radius


School
Elementary School
High
Police Department
5 miles
Camden
Eva
Eva Park
Johnsonville State Historic Park
Denver
New Johnsonville
Broadway St
CL Edwards Memorial Park
Kentucky Lake
Duck River Unit Tennessee National
Beaverdam Rd
Birdsong Rd
Forrest Ave S
W Main St
1 mi
1 km
©2011 Google - Imagery ©2011 DigitalGlobe, GeoEye, USDA Farm Service Agency, Map data ©2011 Google - Terms of Use

Happy Hollow Learning Center

Lakeview Elementary School

Tribble High School

Morris Chapel School

Pembroke School

New Johnsonville Police Department

APPENDIX A

Document 2

Active Ash Disposal Area Aerial View – Stantec Map

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

Stantec

Plan View - JOF

Tennessee Valley Authority
Johnsonville Fossil Plant
Johnsonville, Benton County, Tennessee

PROJECT NO.	171468118
DATE	MARCH 2009
DRAWN BY	AMG
CHECKED BY	DAG
CHECKED BY	
SCALE	AS SHOWN
REVISED	
1.	
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4.	
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SHEET	1 OF 1

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APPENDIX A

Document 3

Johnsonville Fossil Plant – Long Term Disposal Plan

JOHNSONVILLE FOSSIL PLANT - LONG TERM DISPOSAL PLAN
April 15, 2011

FISCAL YEAR ENDING	PROJECTED COAL BURN (TONS)	FLY ASH PRODUCTION (TONS)	BOTTOM ASH PRODUCTION (TONS)	TOTAL ASH PONDED (CY)	ASH UTILIZED / DISPOSED (CY)	DREDGED VOLUME (CY)	ASH POND CAPACITY (CY)	CAPITAL COSTS ($)	O & M COSTS ($)	TOTAL COSTS ($)	NET PRESENT COSTS ($)
2011	3,601,516	238,808	56,832	62,048	570,676	100,000	374,973	15,384,403	4,909,311	20,293,714	20,293,714
2012	3,620,486	240,066	57,131	62,375	570,676	0	312,598	8,789,000	5,032,044	13,821,044	13,821,044
2013	3,064,423	203,194	48,357	52,795	570,676	0	259,804	0	5,157,845	5,157,845	4,485,262
2014	2,297,959	152,372	36,262	39,590	570,676	0	220,214	0	5,286,791	5,286,791	3,997,343
2015	1,099,278	72,890	17,347	18,939	570,676	0	201,275	0	5,418,961	5,418,961	3,562,967
2016	0	0	0	0	0	0	201,275	26,292,000	0	26,292,000	15,033,766
2017	0	0	0	0	0	0	201,275	6,647,000	0	6,647,000	3,304,888
2018	0	0	0	0	0	0	201,275	0	0	0	0
2019	0	0	0	0	0	0	201,275	0	0	0	0
2020	0	0	0	0	0	0	201,275	0	0	0	0
2021	0	0	0	0	0	0	201,275	0	0	0	0
2022	0	0	0	0	0	0	201,275	0	0	0	0
2023	0	0	0	0	0	0	201,275	0	0	0	0
2024	0	0	0	0	0	0	201,275	0	0	0	0
2025	0	0	0	0	0	0	201,275	0	0	0	0
2026	0	0	0	0	0	0	201,275	0	0	0	0
							TOTAL =	**57,112,403**	**25,804,951**	**82,917,354**	**64,498,983**

Assumptions:
1) Coal burn is based on the January 2011 , Long Term Coal Burn Forecast.
2) Ash content is 7.89% plus 5.05% unburned carbon, based upon past 3 year average.
3) 80% of the total ash is fly ash, the remaining 20% is bottom ash.
4) 100% of the bottom ash and fly ash is sluiced to the main ash pond, approximately 83% the ash is retrieved for utilization.
5) Fly ash density (in pond) = 60 lb/c.f.; bottom ash density (stacked) = 75 lb/c.f.
6) The remaining capacity in the ash pond allows for the free water volume requirement of 266,389 c.y.
7) Inflation rate = 2.5%.
8) The net present values are based on a 15% discount rate.
9) Capital Cost are based on the FGD&C FY11 Budget 3/23/11.

APPENDIX A

Document 4

Johnsonville Fossil Plant – Master Strategy

TVA

Coal Combustion Products Master Strategy Review

April 2011

Coal Combustion Products Engineering

Programmatic Pyramid

Dry Storage

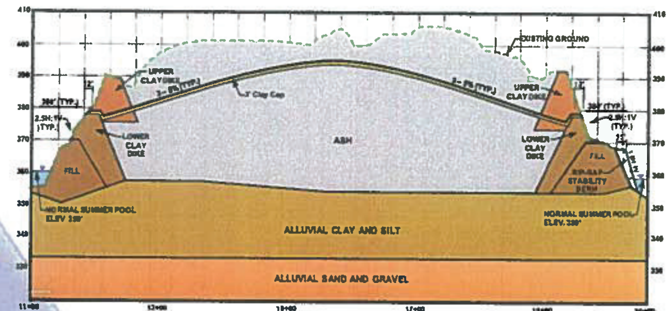
Close all Wet Storage Facilities

Develop a Water Management Strategy for Plant Process Water

Convert Wet CCP Processes to Dry Operation

(Fly Ash, Bottom Ash, and Gypsum)

Implement Long Term Stability Improvements and Remediation to Existing CCP Facilities

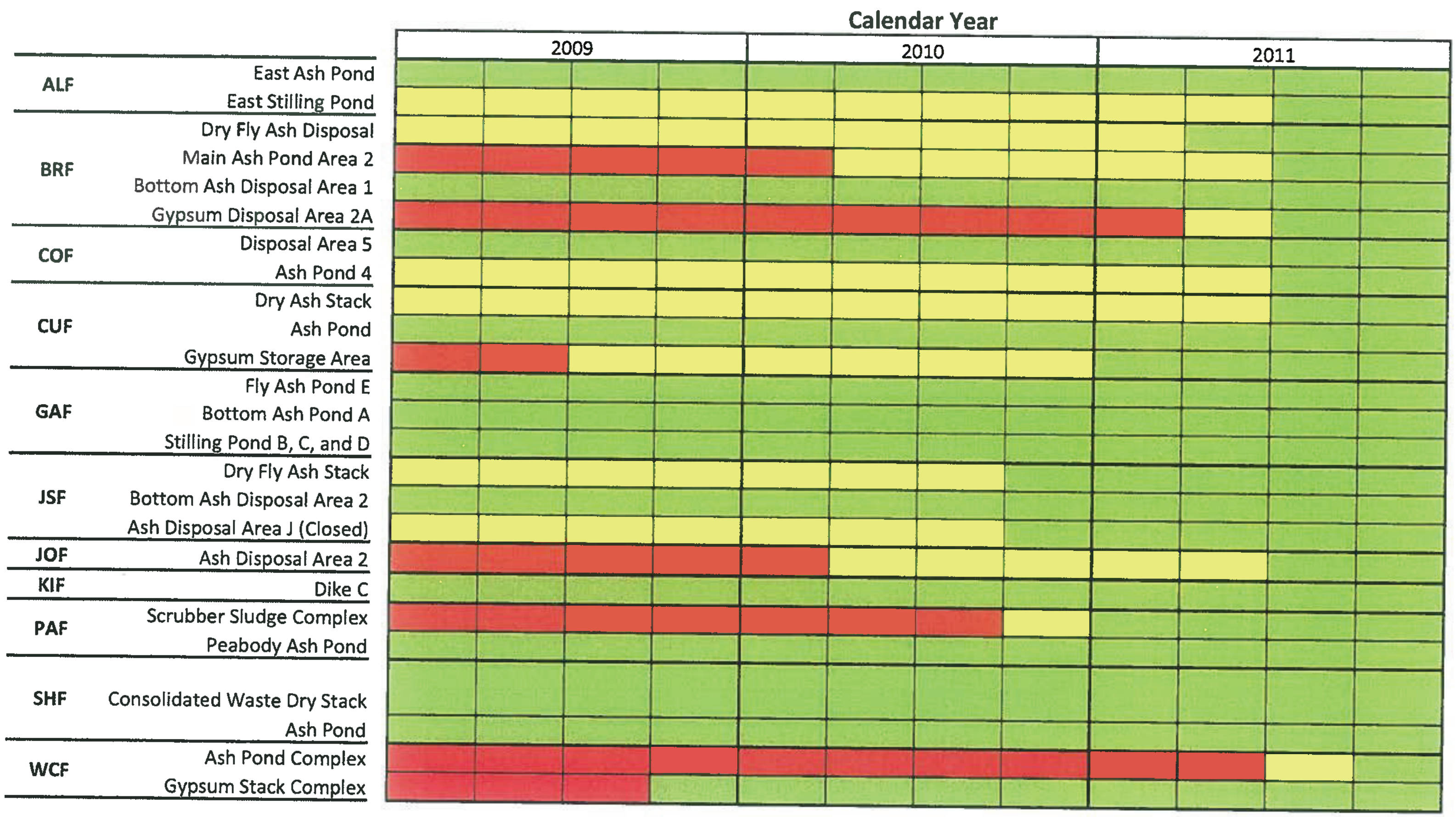


Typical Dike Section

04/04/2011

Coal Combustion Products – Projects and Engineering

Global Stability



Global Stability	As-Found	Current	Jun-2011
1.5 or Greater	12	17	25
1.3 to 1.5	6	6	0
Less than 1.3	7	2	0

04/04/2011

Coal Combustion Products – Projects and Engineering

CCP Score Card

TVA Fossil Plants	Facility	Global Stability	Target to Clear	Non-Global Stability	Target to Clear	Hazard Classification	Target to Clear	Spillways Priority	Schedule
Allen	Ash Disposal		Jun-11		Jun-18		Close		
Bull Run	Ash Pond		Jun-11		Jun-11		Close		
	Gypsum		Jun-11		Dec-11				
Colbert	Dry Stack				May-16	N/A	N/A	N/A	N/A
	Bottom Ash		Jun-11				Close		
Cumberland	Ash Pond						Aug-11	In-Progress	Aug-11
	Dry Stack		Jun-11		Jul-17	N/A	N/A	N/A	N/A
	Gypsum				Jul-17		Close		
Gallatin	Ash Disposal				Sep-17		Close		
John Sevier	Dry Stack				Sep-15	N/A	N/A	N/A	N/A
	Bottom Ash						Close		
Johnsonville	Ash Disposal		Mar-11				Close	In-Progress	Sep-11
Kingston	Dike C				Sep-11		Close		
Paradise	Ash Pond								
	Scrubber Complex				Oct-18				
Shawnee	Dry Stack					N/A	N/A	N/A	N/A
	Ash Pond				Sep-11		Close	In-Progress	May-11
Widows Creek	Ash pond		Sep-11		Sep-12		Close	In-Progress	Jul-12
	Gypsum				Sep-11		Close		

<1.3	Behind Sch	<1.3	Behind Sch	High	Behind Sch	Work Immediately	Behind Sch
1.3 - 1.5	At Risk	1.3 - 1.5	At Risk	Signficant	At Risk	Reduce Risk	At Risk
	On Sch		On Sch	Low	On Sch	Pond Closure	On Sch

04/04/2011

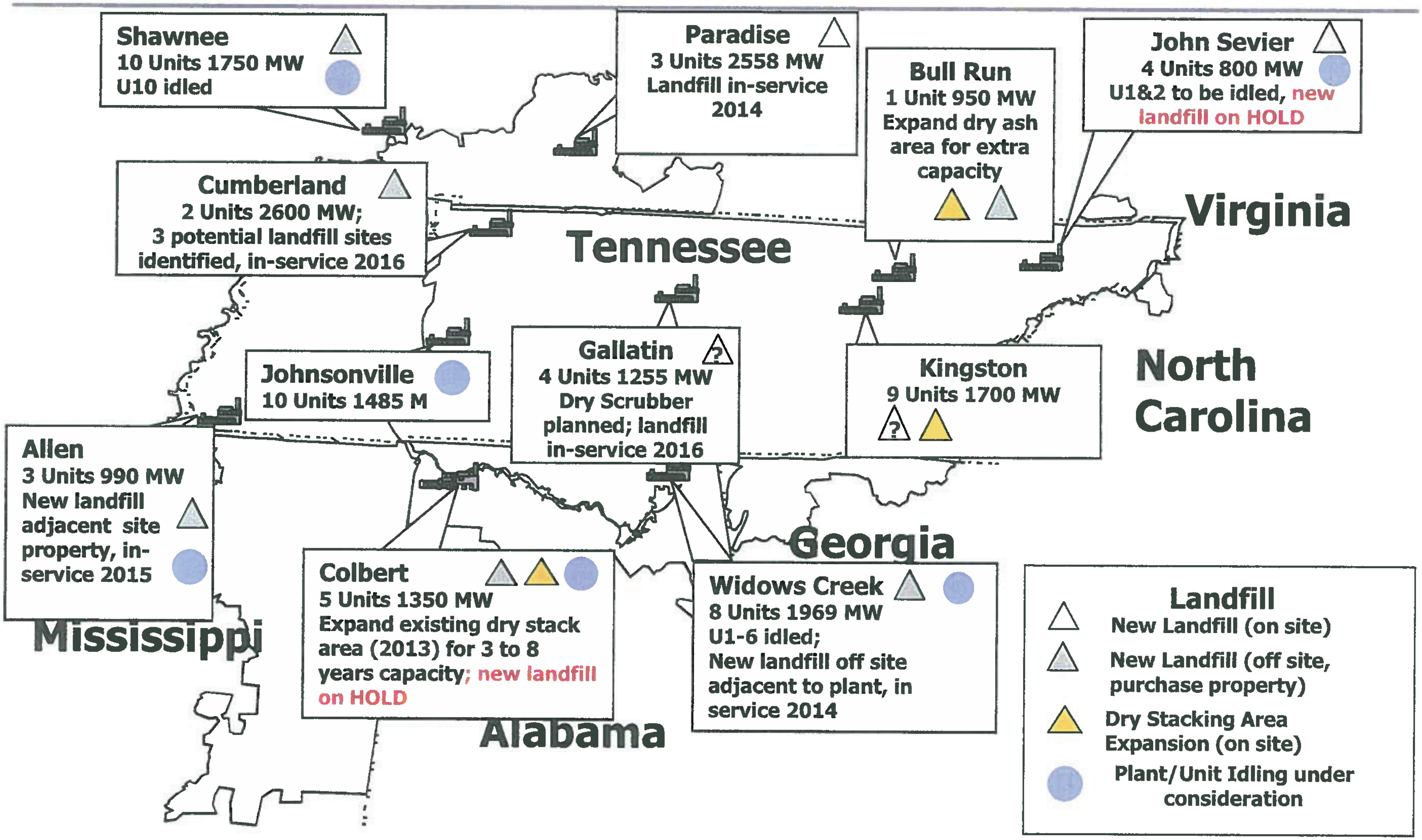
Annual Inspection Plan – 5 Year Plan

2011 Annual Inspection - 5 Year Plan

	2011	2012	2013	2014	2015
Allen	February	June	September	November	March
Bull Run	August	November	February	May	August
Colbert	July	October	January	April	July
Cumberland	June	September	December	March	June
Gallatin	November	February	May	August	November
Kingston	September	December	March	June	September
John Sevier	November	March	June	September	November
Johnsonville	June	September	December	March	June
Paradise	September	December	March	June	September
Shawnee	September	December	June	February	May
Widows Creek	June	September	December	May	February

Coal Combustion Products – Projects and Engineering

Projects Overview – Landfill Valley Map



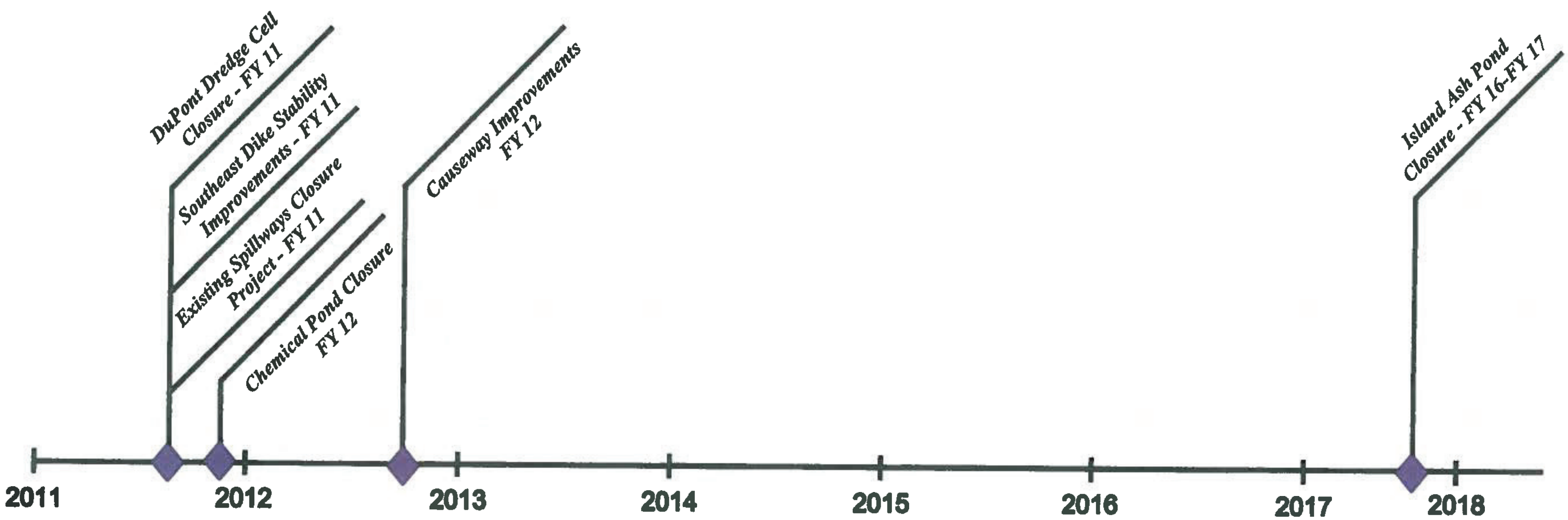
04/04/2011

Coal Combustion Products – Projects and Engineering

Johnsonville Fossil Plant

Northeast Dike Stability Improvements

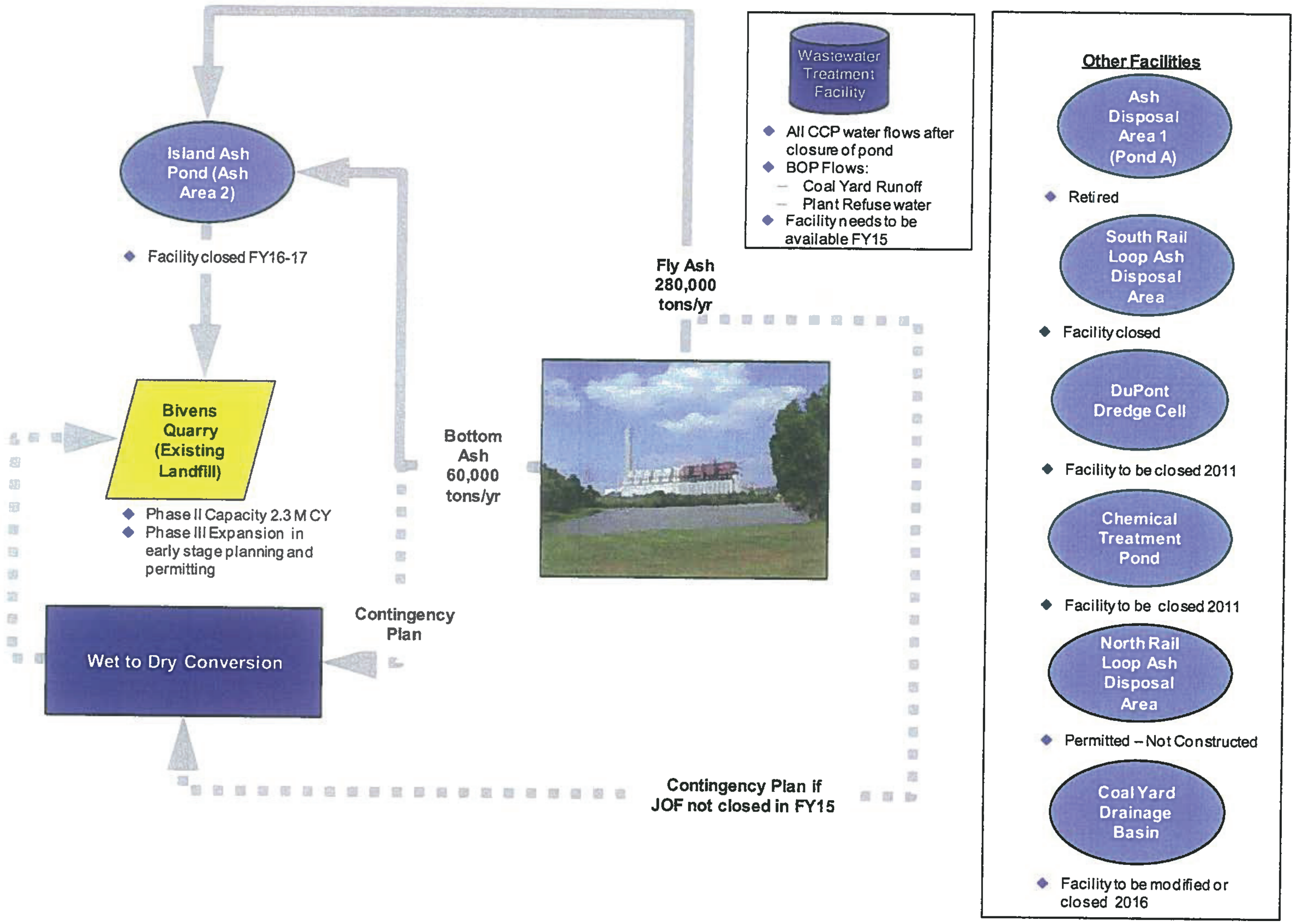
Spillway Replacement Project



04/04/2011

18

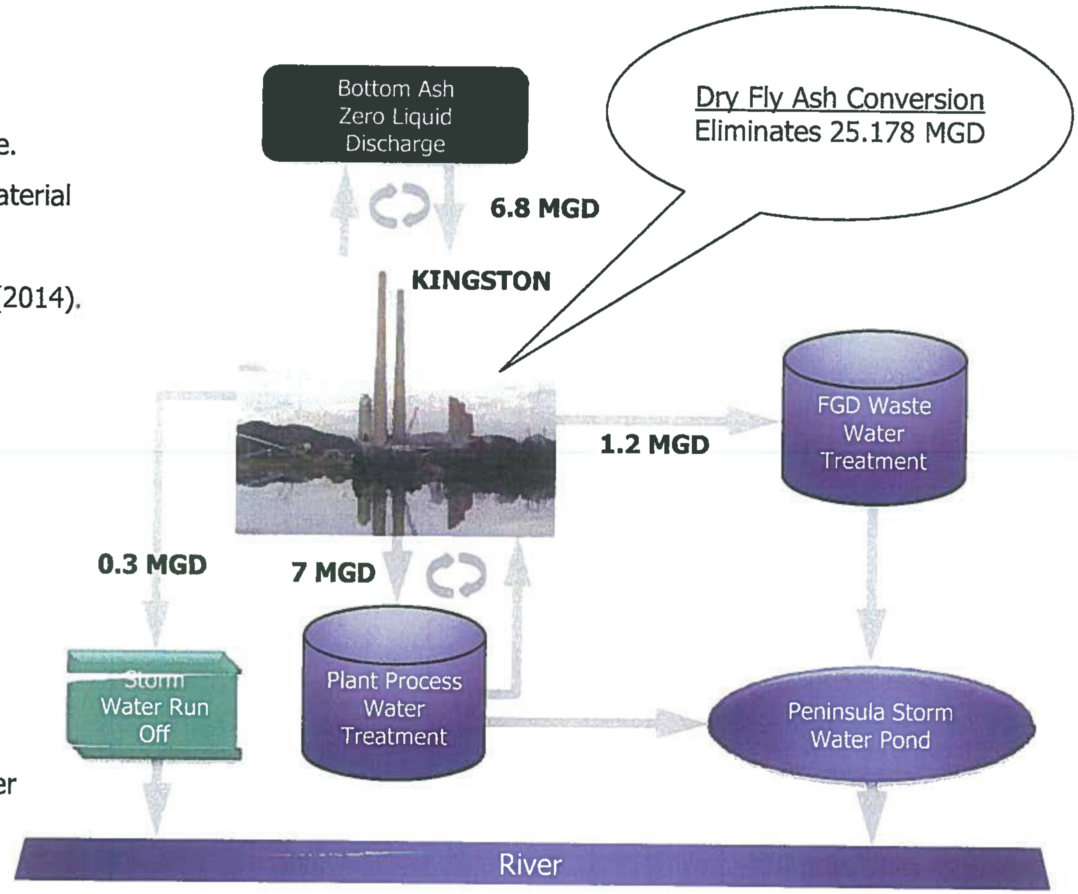
Coal Combustion Products – Projects and Engineering
Johnsonville Fossil Plant



Coal Combustion Products – Projects and Engineering
Water Management

Regulatory Drivers

- Closing all CCP Ponds/Impoundments.
- Regulator Changes will force Stilling Ponds to close.
- Dewatering Facilities will be built to handle CCP Material and discharge at current NPDES Discharge Limits.
- NEW Stringent EPA Effluent Limitation Guidelines (2014).
  - Eliminate Fly Ash Transport Water.
  - Selenium, Mercury, and Arsenic.
  - Expect EPA will require Bottom Ash ZLD.
  - Do not expect ZLD requirement on FGD.



TVA Team Approach

- Organized a team of FPG, FGDC (lead), E&T.
- Researched Industry and Manufactures of WWT.
- Performed Water Characterization Study of all water streams at BRF, KIF, and WCF.
- Developed Pilot Strategy for KIF.
- Will be the core JPT for all WWT Projects.
- Current Estimate of $80M-$160M per fossil facility.

04/04/2011

APPENDIX A

Document 5

Boring Plan and Instrumentation Plan

BORING LOCATION TABLE			
BORING	NORTHING	EASTING	ELEV. (FT.)
STN-AC	603,148.82	1,410,894.84	391.4
STN-AT	603,144.12	1,410,950.20	368.4
STN-BC	602,313.93	1,410,981.18	391.5
STN-BT	602,326.12	1,411,067.30	369.8
STN-C1C	601,113.79	1,411,129.70	391.5
STN-C1CA	601,054.92	1,410,641.77	394.0
STN-C1CB	601,029.42	1,410,415.09	398.4
STN-C1T	601,033.28	1,411,220.15	365.5
STN-CC	601,437.52	1,411,070.75	391.6
STN-CCA	601,382.49	1,410,633.59	394.6
STN-CT	601,449.55	1,411,148.76	368.9
STN-DC	600,191.17	1,410,774.31	390.0
STN-DT	600,147.64	1,410,847.53	365.3
STN-EC	599,528.35	1,410,416.19	390.2
STN-ECA	599,517.65	1,410,425.57	390.2
STN-ET	599,486.09	1,410,498.27	363.6
STN-FC	598,898.98	1,410,094.79	389.4
STN-FT	598,868.34	1,410,145.49	362.9
STN-GC	598,719.43	1,409,736.38	389.6
STN-GT	598,592.54	1,409,722.40	360.8
STN-HC	599,345.93	1,409,646.07	389.5
STN-HM	599,331.00	1,409,595.58	377.9
STN-HT	599,308.41	1,409,545.73	363.1
STN-IC	600,055.90	1,409,637.66	389.8
STN-IT	600,103.14	1,409,560.26	368.6
STN-JC	600,817.61	1,409,871.68	389.6
STN-JT	600,838.26	1,409,820.33	376.7
STN-KC	601,482.92	1,410,105.77	389.8
STN-KT	601,488.28	1,410,056.92	377.6
STN-LC	602,377.53	1,410,442.03	389.9
STN-LT	602,392.94	1,410,352.26	366.3
STN-MC	603,157.11	1,410,726.95	390.6
STN-MT	603,187.15	1,410,653.44	365.6
*STN-B-1			390.6
*STN-B-2			390.2
*STN-B-3			390.2
*STN-B-4			389.5
*STN-B-5			389.9
*STN-B-6			389.9
*STN-B-7			390.1
*STN-B-8			389.9
*STN-B-8A			389.9
*STN-B-9			389.7
*STN-B-10			389.1
*STN-B-11			389.6
*STN-B-12	Not Drilled		
*STN-B-13			390.1
*STN-B-14			387.3
*STN-B-15			378.9
*STN-B-16			389.6
*STN-B-17			389.1
*STN-B-18			391.0
*STN-B-19			388.5
*STN-B-20			388.9
*STN-B-21			389.2

*Borings not surveyed and elevations are approximate

INSPECTION TEST PIT LOCATION TABLE			
BORING	NORTHING	EASTING	ELEV. (FT.)
TP-1	601,957.34	1,410,239.85	–
TP-2	602,389.71	1,410,395.38	–
TP-3	602,721.20	1,410,510.10	–
TP-4	603,003.70	1,410,606.32	–

*Test Pit Locations are approximate

BORINGS / **REFERENCE**

BORINGS	REFERENCE
B-1 through B-9 and B-OW-1 through B-OW-6	Report of Geotechnical Exploration, Ash Pond Dike, New Johnsonville Plant, New Johnsonville, Tennessee, Law Engineering and Environmental Services, Inc., January 17, 1994.
94-5 through 94-14	Subsurface Exploration Data, TVA Borings at Johnsonville Fossil Plant, Johnsonville, Tennessee, Law Engineering and Environmental Services, Inc., October 11, 1994.
US-1B through US-6B	Johnsonville Steam Plant – Ash Pond – Soil and Foundation Exploration, J.C. McCrow, TVA Construction Services Branch, September 17, 1969.
SS-13, SS-15, SS-16	Johnsonville Groundwater Assessment, TVA Resource Group, Engineering Services, March 1995.
SS-1 through SS-11 and US-1, US-7 and US-8	Johnsonville Steam Plant – Ash Disposal Area No. 2 Dike Raising – Soil Exploration and Testing, G. Farmer, TVA Construction Services Branch, November 22, 1977.
C2-1 through C2-20	Report of Ash Pond Investigation, Johnsonville Fossil Plant, New Johnsonville, Tennessee, MACTEC Engineering and Consulting, December 4, 2003.

LEGEND

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

NOTES:

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

100 0 200 400 FEET
GRAPHIC SCALE

For Supporting Design Calculations see

Stantec Consulting Services Inc.

Stantec

SCALE: 1"=200' EXCEPT AS NOTED

YARD
ASH DISPOSAL AREAS 2 AND 3

GEOTECHNICAL EXPLORATION
BORING LAYOUT

JOHNSONVILLE FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

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

STANTEC | 0
TASK COMPLETED BY | REV NO.

PLOT FACTOR:XX
W_TVA

C.A.D. DRAWING
DO NOT ALTER MANUALLY

ID	NORTHING	EASTING	ELEV. (FT.)	INSTRUMENT TYPE
STN-AC-PZ	603,133.83	1,410,898.65	391.60	PZ
STN-AC-PZ(2)	603,085.01	1,410,909.64	391.80	PZ
STN-AT-PZ	603,144.12	1,410,980.20	368.40	PZ
STN-BC-PZ	602,314.45	1,410,988.89	392.40	PZ
STN-BT-PZ	602,326.17	1,411,067.30	369.80	PZ
STN-C1C-PZ	601,113.79	1,411,129.20	391.50	PZ
STN-C1C-PZ(2)	601,026.14	1,411,143.53	392.40	PZ
STN-C1T-PZ	601,033.28	1,411,220.15	365.50	PZ
STN-CC-PZ	601,439.64	1,411,087.11	392.50	PZ
STN-CC-PZ(2)	601,398.92	1,411,099.20	392.40	PZ
STN-CT-PZ	601,449.55	1,411,148.76	368.90	PZ
STN-DC-PZ	600,184.33	1,410,790.81	391.20	PZ
STN-DC-PZ(2)	600,140.29	1,410,778.22	390.90	PZ
STN-DT-PZ	600,147.64	1,410,847.53	365.30	PZ
STN-EC-PZ	599,517.65	1,410,428.37	390.40	PZ
STN-ET-PZ	599,486.09	1,410,496.27	363.80	PZ
STN-FC-PZ	598,892.27	1,410,076.24	389.80	PZ
STN-FT-PZ	598,868.34	1,410,145.49	362.90	PZ
STN-GC-PZ	598,696.58	1,409,758.99	389.80	PZ
STN-GT-PZ	598,582.54	1,409,772.40	360.80	PZ
STN-HC-PZ	599,314.96	1,409,635.77	390.00	PZ
STN-HT-PZ	599,308.41	1,409,545.23	363.10	PZ
STN-IC-PZ	600,086.63	1,409,629.83	390.10	PZ
STN-IT-PZ	600,103.14	1,409,560.28	368.80	PZ
STN-JC-PZ	600,825.25	1,409,856.37	390.00	PZ
STN-JT-PZ	600,838.28	1,409,820.33	378.70	PZ
STN-KC-PZ	601,483.81	1,410,099.66	390.50	PZ
STN-KT-PZ	601,488.26	1,410,056.92	377.60	PZ
STN-LC-PZ	602,374.84	1,410,429.97	390.50	PZ
STN-LT-PZ	602,392.94	1,410,352.28	366.30	PZ
STN-MC-PZ	603,157.20	1,410,719.51	391.10	PZ
STN-MT-PZ	603,187.15	1,410,653.44	365.60	PZ
STN-SI-1	601,441.71	1,411,095.09	392.50	SI
STN-SI-1A	601,436.97	1,411,086.68	392.50	SI
STN-SI-2	601,119.30	1,411,134.51	391.70	SI
STN-SI-3	599,513.18	1,410,433.95	390.20	SI
STN-SI-4	601,487.29	1,410,094.04	390.40	SI

Northing, Easting and Ground Surface Elevations were provided by TVA, Power Systems Operations, Surveying and Project Services.
Horizontal Datum: NAD 27 (Tennessee Lambert). Vertical Datum: NGVD29.
"PZ" denotes Piezometer
"SI" denotes Slope Inclinometer

LEGEND

- Piezometer (PZ)
- Slope Inclinometer (SI)
- TVA Survey Monument

NOTES:

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

100 0 200 400 FEET
GRAPHIC SCALE

For Supporting Design Calculations see

SCALE: 1"=200' EXCEPT AS NOTED

YARD
ASH DISPOSAL AREAS 2 AND 3

GEOTECHNICAL EXPLORATION
INSTRUMENTATION PLAN

Stantec Consulting Services Inc.

JOHNSONVILLE FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY

AUTOCAD R 2002 | 30 | C | XXWXXX-02 | R 0

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TASK COMPLETED BY | REV NO

PLOT FACTOR:XX

C.A.D. DRAWING
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APPENDIX A

Document 6

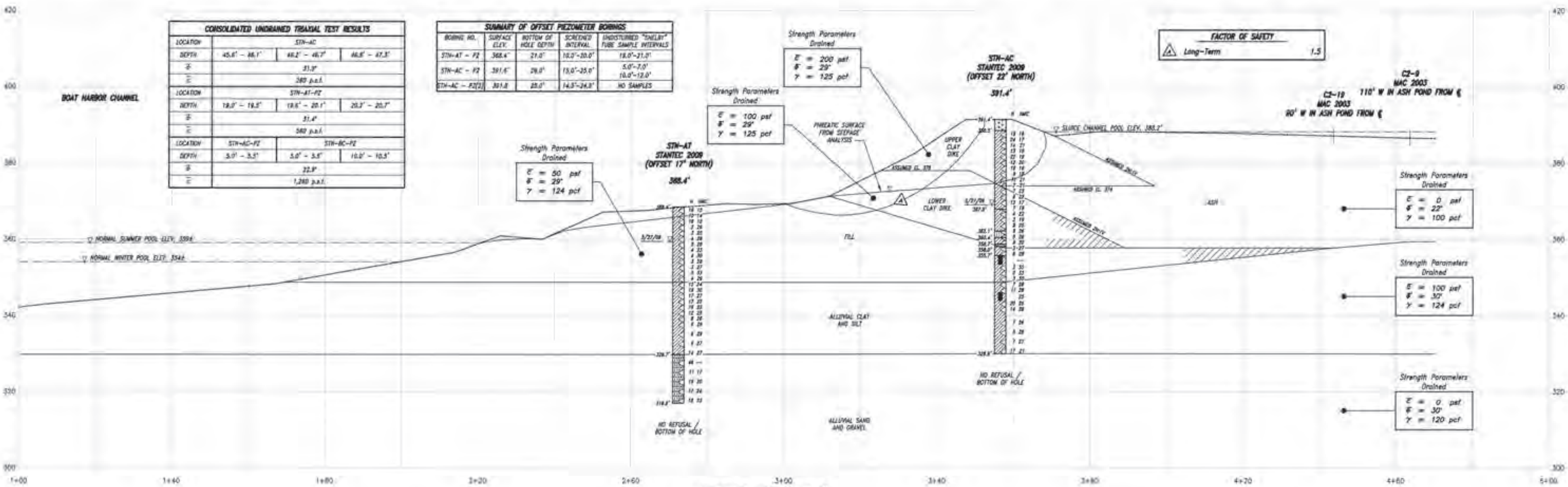
Analysis Sections – Original Conditions

Appendix H

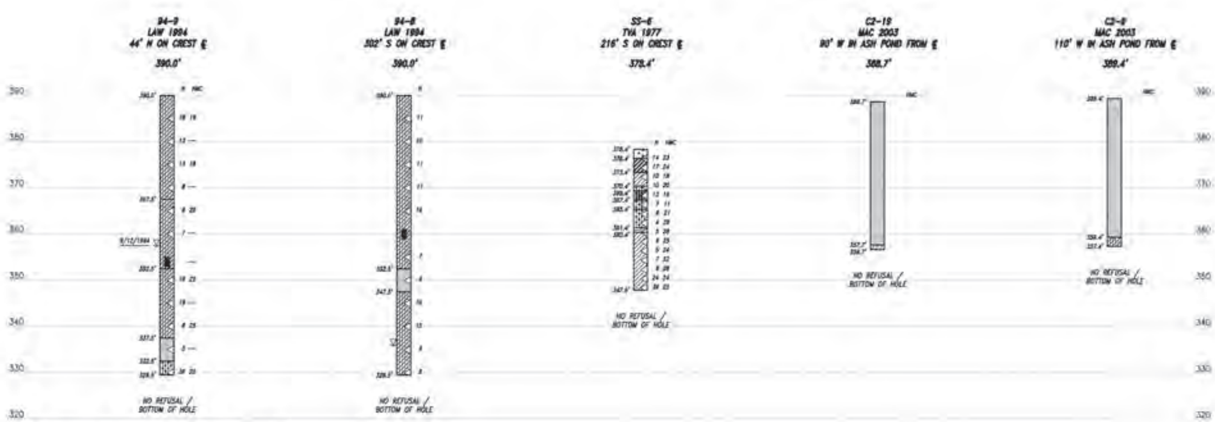
Slope Stability Sections

CONSOLIDATED UNDRAINED TRIAXIAL TEST RESULTS			
LOCATION	STN-AC		
DEPTH	45.6' - 46.1'	46.2' - 46.7'	46.8' - 47.3'
$\bar{\phi}$	31.9°		
$\bar{c}$	280 p.s.f.		
LOCATION	STN-AT-PZ		
DEPTH	19.0' - 19.5'	19.6' - 20.1'	20.2' - 20.7'
$\bar{\phi}$	31.4°		
$\bar{c}$	560 p.s.f.		
LOCATION	STN-AC-PZ	STN-BC-PZ	
DEPTH	3.0' - 3.5'	5.0' - 5.5'	10.0' - 10.5'
$\bar{\phi}$	22.9°		
$\bar{c}$	1,280 p.s.f.		

SUMMARY OF OFFSET PIEZOMETER BORINGS				
BORING NO.	SURFACE ELEV.	BOTTOM OF HOLE DEPTH	SCREENED INTERVAL	UNDISTURBED "SHELBY" TUBE SAMPLE INTERVALS
STN-AT - PZ	368.4'	21.0'	10.0'-20.0'	19.0'-21.0'
STN-AC - PZ	391.6'	26.0'	15.0'-25.0'	5.0'-7.0' 10.0'-12.0'
STN-AC - PZ(2)	391.8	25.0'	14.5'-24.5'	NO SAMPLES



STABILITY SECTION A–A'



LEGEND

- LEAN CLAY, SANDY LEAN CLAY, SANDY LEAN CLAY WITH GRAVEL, LEAN CLAY WITH GRAVEL, LEAN CLAY WITH SAND
- FAT CLAY
- CLAYEY SAND, CLAYEY SAND WITH GRAVEL
- CLAYEY GRAVEL
- SILT, SANDY SILT, SILT WITH GRAVEL, SILT WITH SAND
- ELASTIC SILT
- SILTY CLAY, SILTY CLAY WITH GRAVEL
- SILTY SAND, POORLY GRADED SAND WITH SILT AND GRAVEL, SILTY SAND WITH GRAVEL
- SILTY GRAVEL
- WELL GRADED SAND WITH SILT, WELL GRADED SAND WITH SILT AND GRAVEL
- POORLY GRADED SAND, POORLY GRADED SAND WITH GRAVEL, POORLY GRADED SAND WITH SILT
- POORLY GRADED GRAVEL, POORLY GRADED GRAVEL WITH SILT AND SAND, POORLY GRADED GRAVEL WITH SAND
- TOPSOIL
- FLY ASH
- BOTTOM ASH
- N STANDARD PENETRATION TEST BLOW COUNT (blows/ft)
- NMC NATURAL MOISTURE CONTENT (%)
- UC UNCONFINED COMPRESSIVE STRENGTH TEST (PSF)
- WATER LEVEL AND DATE RECORDED
- UNDISTURBED THIN-WALL (SHELBY) TUBE SAMPLE
- STANDARD PENETRATION TEST INTERVAL

NOTE: SURVEYED CROSS-SECTIONS AND HORIZONTAL AND VERTICAL LOCATIONS OF 2009 BORINGS PROVIDED BY T.V.A. LOCATION OF PREVIOUSLY DRILLED BORINGS FROM BORING LAYOUTS PROVIDED IN PREVIOUS REPORTS.

SCALE: 1" = 10' EXCEPT AS NOTED

YARD
ASH DISPOSAL AREAS 2 AND 3

GEOTECHNICAL EXPLORATION
STABILITY SECTION A–A'

Stantec Consulting Services Inc.

JOHNSONVILLE FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

AUTOCAD R. 2008 | 01/27/10 | 34 | C | XXWXXX-03 | R 0

PLOT FACTOR:XX

C.A.D. DRAWING DO NOT ALTER MANUALLY

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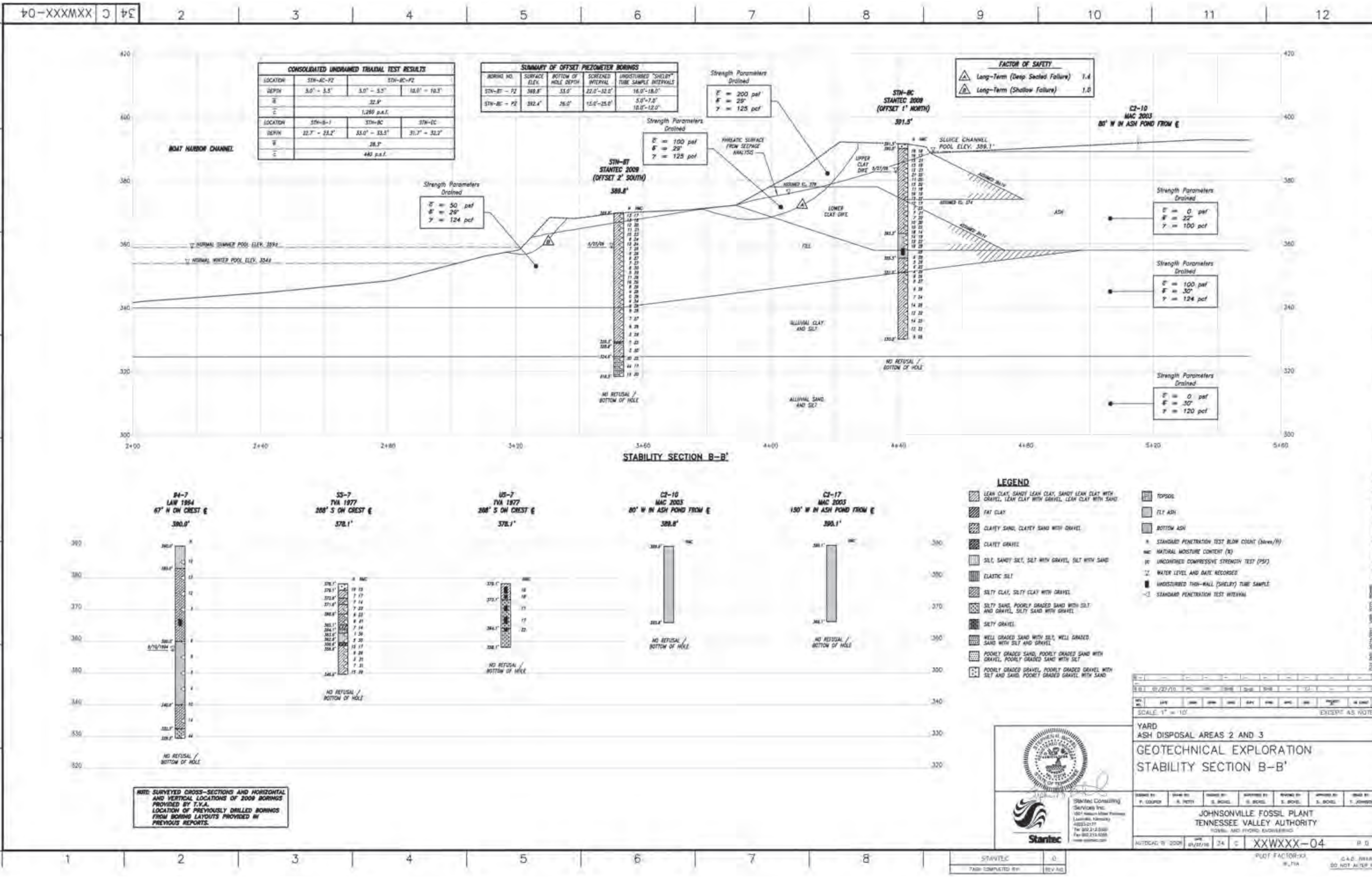
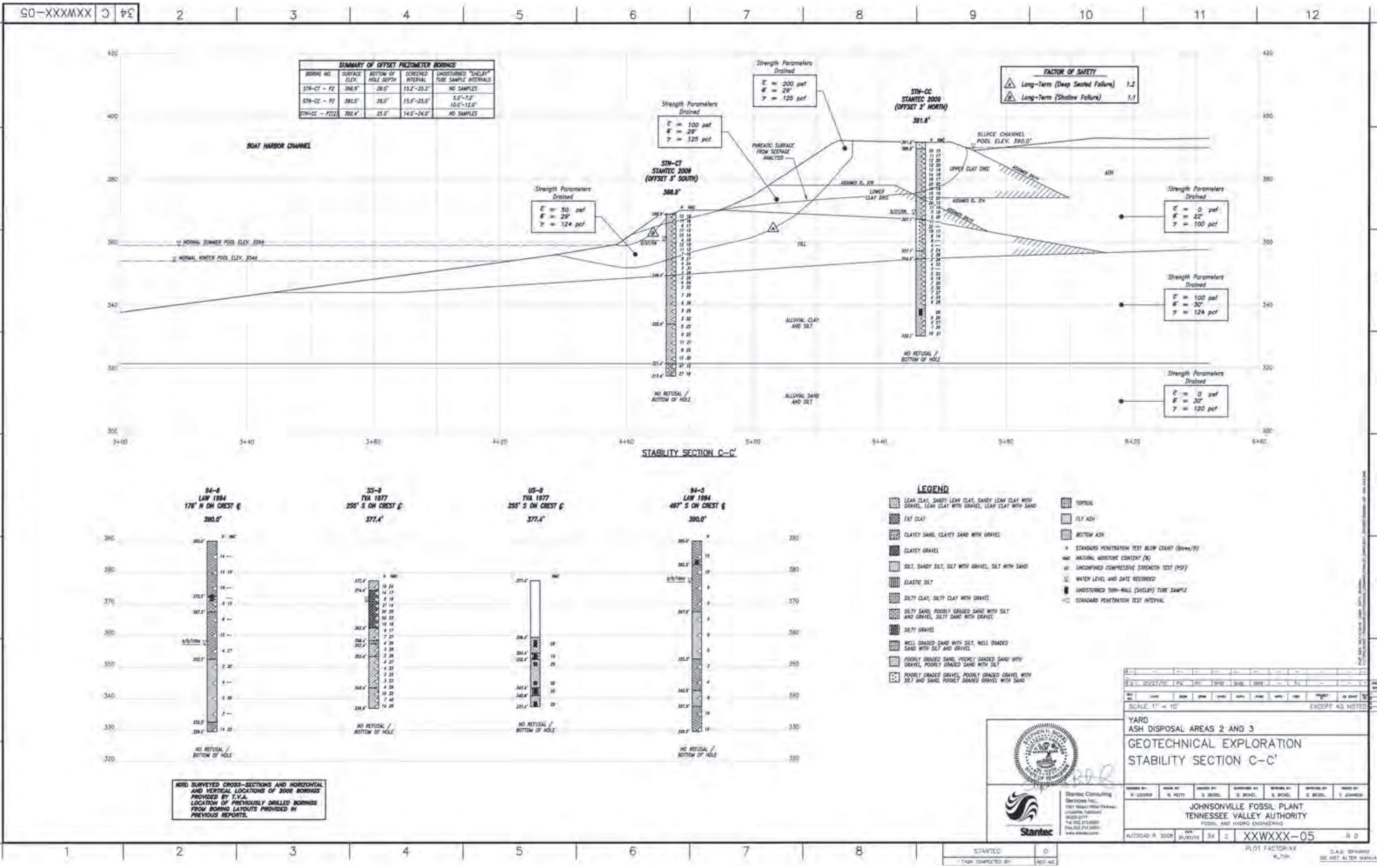
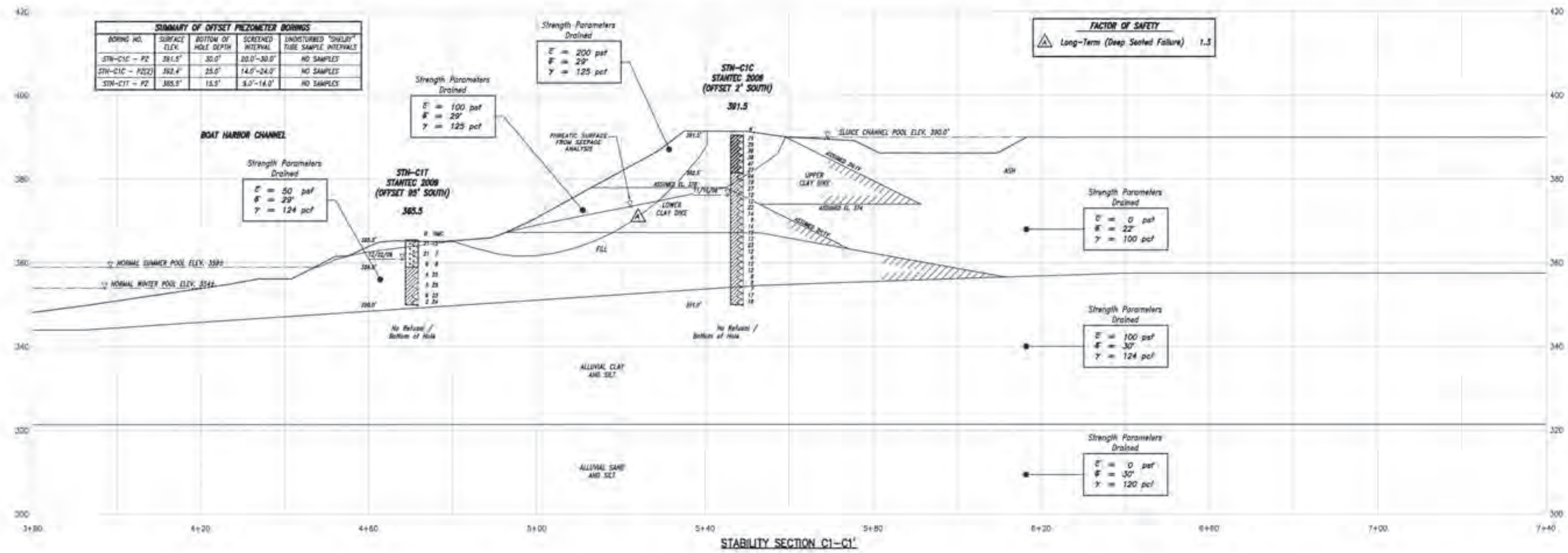
TASK COMPLETED BY: | REV NO.

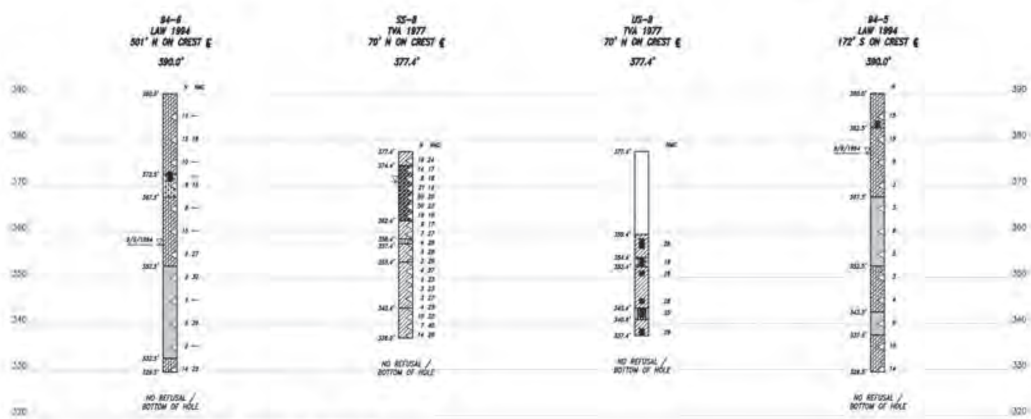

STABILITY SECTION B–B'
CONSOLIDATED UNDRAINED TRIAXIAL TEST RESULTS
SUMMARY OF OFFSET PIEZOMETER BORINGS
FACTOR OF SAFETY
Long-Term (Deep Seated Failure) 1.4
Long-Term (Shallow Failure) 1.0
BOAT HARBOR CHANNEL
NORMAL SUMMER POOL ELEV. 359±
NORMAL WINTER POOL ELEV. 354±
Strength Parameters Drained
STN-8T STANTEC 2009 (OFFSET 2' SOUTH) 388.8'
STN-8C STANTEC 2009 (OFFSET 1' NORTH) 391.5'
C2-10 MAC 2003 80' W IN ASH POND FROM ℄
SLUICE CHANNEL POOL ELEV. 389.1'
PHREATIC SURFACE FROM SEEPAGE ANALYSIS
UPPER CLAY DIKE
LOWER CLAY DIKE
FILL
ASH
ALLUVIAL CLAY AND SILT
ALLUVIAL SAND AND SILT
NO REFUSAL / BOTTOM OF HOLE
84-7 LAW 1984 67' N ON CREST ℄ 390.0'
SS-7 TVA 1977 288' S ON CREST ℄ 378.1'
US-7 TVA 1977 288' S ON CREST ℄ 378.1'
C2-10 MAC 2003 80' W IN ASH POND FROM ℄ 389.8'
C2-17 MAC 2003 150' W IN ASH POND FROM ℄ 390.1'
NOTE: SURVEYED CROSS-SECTIONS AND HORIZONTAL AND VERTICAL LOCATIONS OF 2009 BORINGS PROVIDED BY T.V.A. LOCATION OF PREVIOUSLY DRILLED BORINGS FROM BORING LAYOUTS PROVIDED IN PREVIOUS REPORTS.
LEGEND
YARD
ASH DISPOSAL AREAS 2 AND 3
GEOTECHNICAL EXPLORATION
STABILITY SECTION B–B'
JOHNSONVILLE FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
SCALE: 1" = 10'
XXWXXX–04
Stantec


XXWXXX-05
34 C
SUMMARY OF OFFSET PIEZOMETER BORINGS
BORING NO. | SURFACE ELEV. | BOTTOM OF HOLE DEPTH | SCREENED INTERVAL | UNDISTURBED "SHELBY" TUBE SAMPLE INTERVALS
STN-CT – PZ | 398.9' | 28.0' | 15.2'–25.2' | NO SAMPLES
STN-CC – PZ | 392.5' | 26.0' | 15.0'–25.0' | 5.0'–7.0' 10.0'–12.0'
STN-CC – PZ(2) | 392.4' | 25.0' | 14.0'–24.0' | NO SAMPLES
BOAT HARBOR CHANNEL
NORMAL SUMMER POOL ELEV. 359±
NORMAL WINTER POOL ELEV. 354±
Strength Parameters Drained
C = 50 psf
Φ = 29°
γ = 124 pcf
Strength Parameters Drained
C = 100 psf
Φ = 29°
γ = 125 pcf
Strength Parameters Drained
C = 200 psf
Φ = 29°
γ = 125 pcf
STN-CT
STANTEC 2009
(OFFSET 3' SOUTH)
388.9'
PHREATIC SURFACE FROM SEEPAGE ANALYSIS
STN-CC
STANTEC 2009
(OFFSET 2' NORTH)
381.8'
SLUICE CHANNEL POOL ELEV. 390.0'
UPPER CLAY DIKE
LOWER CLAY DIKE
ASH
FILL
ALLUVIAL CLAY AND SILT
ALLUVIAL SAND AND SILT
NO REFUSAL / BOTTOM OF HOLE
FACTOR OF SAFETY
Long-Term (Deep Seated Failure) 1.2
Long-Term (Shallow Failure) 1.1
Strength Parameters Drained
C = 0 psf
Φ = 22°
γ = 100 pcf
Strength Parameters Drained
C = 100 psf
Φ = 30°
γ = 124 pcf
Strength Parameters Drained
C = 0 psf
Φ = 30°
γ = 120 pcf
STABILITY SECTION C–C'
94-6
LAW 1994
176' N ON CREST ℄
390.0'
55-8
TVA 1977
255' S ON CREST ℄
377.4'
U5-8
TVA 1977
255' S ON CREST ℄
377.4'
94-5
LAW 1994
497' S ON CREST ℄
390.0'
LEGEND
LEAN CLAY, SANDY LEAN CLAY, SANDY LEAN CLAY WITH GRAVEL, LEAN CLAY WITH GRAVEL, LEAN CLAY WITH SAND
FAT CLAY
CLAYEY SAND, CLAYEY SAND WITH GRAVEL
CLAYEY GRAVEL
SILT, SANDY SILT, SILT WITH GRAVEL, SILT WITH SAND
ELASTIC SILT
SILTY CLAY, SILTY CLAY WITH GRAVEL
SILTY SAND, POORLY GRADED SAND WITH SILT AND GRAVEL, SILTY SAND WITH GRAVEL
SILTY GRAVEL
WELL GRADED SAND WITH SILT, WELL GRADED SAND WITH SILT AND GRAVEL
POORLY GRADED SAND, POORLY GRADED SAND WITH GRAVEL, POORLY GRADED SAND WITH SILT
POORLY GRADED GRAVEL, POORLY GRADED GRAVEL WITH SILT AND SAND, POORLY GRADED GRAVEL WITH SAND
TOPSOIL
FLY ASH
BOTTOM ASH
STANDARD PENETRATION TEST BLOW COUNT (blows/ft)
NATURAL MOISTURE CONTENT (%)
UNCONFINED COMPRESSIVE STRENGTH TEST (PSF)
WATER LEVEL AND DATE RECORDED
UNDISTURBED THIN-WALL (SHELBY) TUBE SAMPLE
STANDARD PENETRATION TEST INTERVAL
NOTE: SURVEYED CROSS–SECTIONS AND HORIZONTAL AND VERTICAL LOCATIONS OF 2009 BORINGS PROVIDED BY T.V.A. LOCATION OF PREVIOUSLY DRILLED BORINGS FROM BORING LAYOUTS PROVIDED IN PREVIOUS REPORTS.
SCALE: 1" = 10'
EXCEPT AS NOTED
YARD
ASH DISPOSAL AREAS 2 AND 3
GEOTECHNICAL EXPLORATION
STABILITY SECTION C–C'
JOHNSONVILLE FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING
Stantec
XXWXXX–05
R 0
PLOT FACTOR:XX
C.A.D. DRAWING DO NOT ALTER MANUALLY

SUMMARY OF OFFSET PIEZOMETER BORINGS				
BORING NO.	SURFACE ELEV.	BOTTOM OF HOLE DEPTH	SCREENED INTERVAL	UNDISTURBED "SHELBY" TUBE SAMPLE INTERVALS
STN-C1C – P2	391.5'	30.0'	20.0'–30.0'	NO SAMPLES
STN-C1C – P2(2)	392.4'	25.0'	14.0'–24.0'	NO SAMPLES
STN-C1T – P2	365.5'	15.5'	5.0'–14.0'	NO SAMPLES



STABILITY SECTION C1–C1'



NOTE: SURVEYED CROSS–SECTIONS AND HORIZONTAL AND VERTICAL LOCATIONS OF 2009 BORINGS PROVIDED BY T.V.A. LOCATION OF PREVIOUSLY DRILLED BORINGS FROM BORING LAYOUTS PROVIDED IN PREVIOUS REPORTS.

LEGEND

- LEAN CLAY, SANDY LEAN CLAY, SANDY LEAN CLAY WITH GRAVEL, LEAN CLAY WITH GRAVEL, LEAN CLAY WITH SAND
- FAT CLAY
- CLAYEY SAND, CLAYEY SAND WITH GRAVEL
- CLAYEY GRAVEL
- SILT, SANDY SILT, SILT WITH GRAVEL, SILT WITH SAND
- ELASTIC SILT
- SILTY CLAY, SILTY CLAY WITH GRAVEL
- SILTY SAND, POORLY GRADED SAND WITH SILT AND GRAVEL, SILTY SAND WITH GRAVEL
- SILTY GRAVEL
- WELL GRADED SAND WITH SILT, WELL GRADED SAND WITH SILT AND GRAVEL
- POORLY GRADED SAND, POORLY GRADED SAND WITH GRAVEL, POORLY GRADED SAND WITH SILT
- POORLY GRADED GRAVEL, POORLY GRADED GRAVEL WITH SILT AND SAND, POORLY GRADED GRAVEL WITH SAND
- TOPSOIL
- FLY ASH
- BOTTOM ASH
- N STANDARD PENETRATION TEST BLOW COUNT (blows/ft)
- NMC NATURAL MOISTURE CONTENT (%)
- UC UNCONFINED COMPRESSIVE STRENGTH TEST (PSF)
- WATER LEVEL AND DATE RECORDED
- UNDISTURBED THIN-WALL (SHELBY) TUBE SAMPLE
- STANDARD PENETRATION TEST INTERVAL

R 0 01/27/10

SCALE: 1" = 10' EXCEPT AS NOTED

YARD
ASH DISPOSAL AREAS 2 AND 3

GEOTECHNICAL EXPLORATION
STABILITY SECTION C1–C1'

Stantec Consulting Services Inc.

JOHNSONVILLE FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

AUTOCAD R 2006 | 34 | C | XXWXXX–06 | R 0

STANTEC — TASK COMPLETED BY — REV NO.

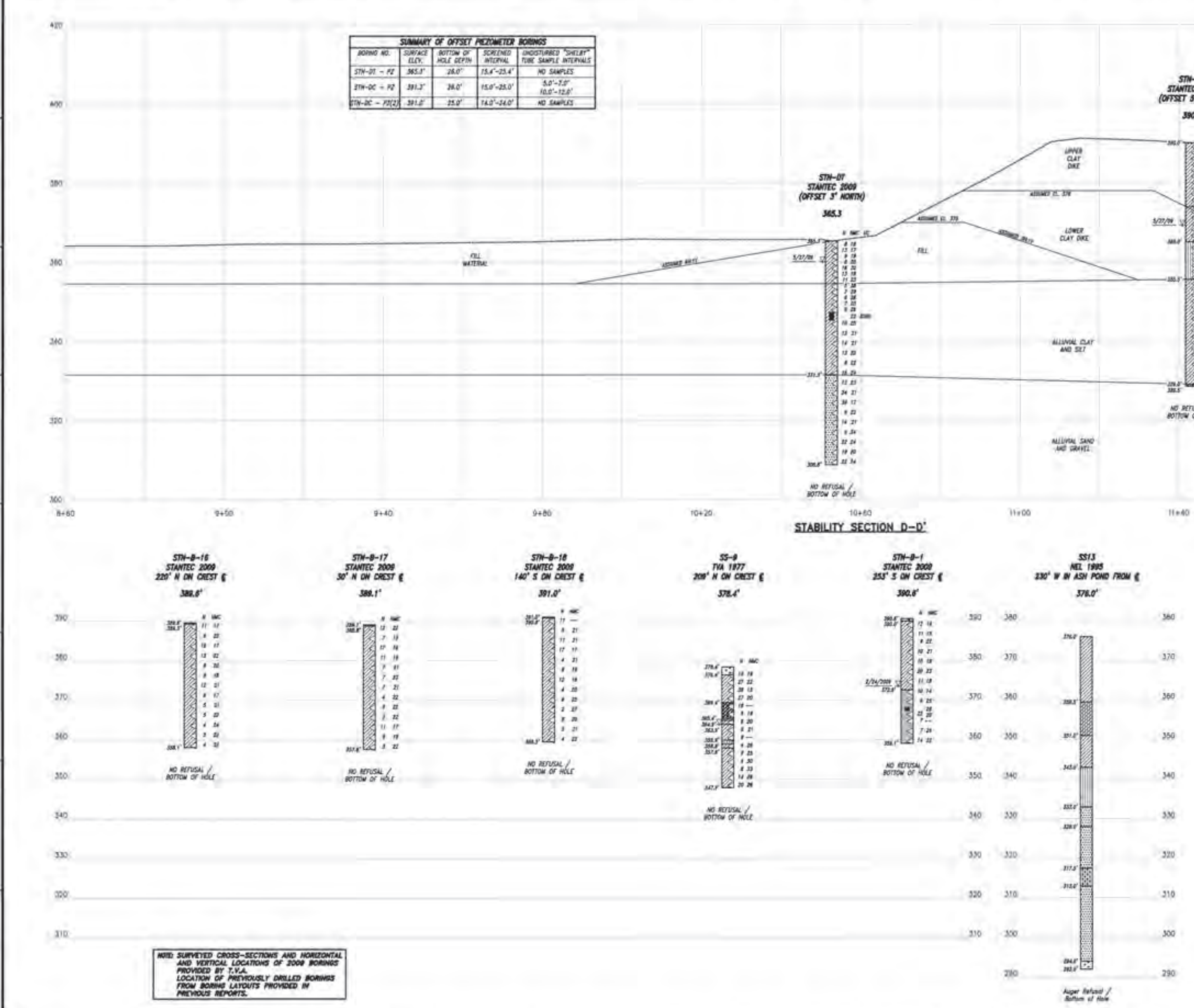
PLOT FACTOR: XX

C.A.D. DRAWING DO NOT ALTER MANUALLY

34 C XXWXXX–06

XXWXXX-07 | C | 34

SUMMARY OF OFFSET PIEZOMETER BORINGS				
BORING NO.	SURFACE ELEV.	BOTTOM OF HOLE DEPTH	SCREENED INTERVAL	UNDISTURBED "SHELBY" TUBE SAMPLE INTERVALS
STN-07 - PZ	365.3'	26.0'	15.4'-25.4'	NO SAMPLES
STN-0C - PZ	391.3'	26.0'	15.0'-25.0'	5.0'-7.0' 10.0'-12.0'
STN-0C - PZ(2)	391.0'	25.0'	14.0'-24.0'	NO SAMPLES



STABILITY SECTION D-D'

NOTE: SURVEYED CROSS-SECTIONS AND HORIZONTAL AND VERTICAL LOCATIONS OF 2009 BORINGS PROVIDED BY T.V.A. LOCATION OF PREVIOUSLY DRILLED BORINGS FROM BORING LAYOUTS PROVIDED IN PREVIOUS REPORTS.

LEGEND

- LEAN CLAY, SANDY LEAN CLAY, SANDY LEAN CLAY WITH GRAVEL, LEAN CLAY WITH GRAVEL, LEAN CLAY WITH SAND
- FAT CLAY
- CLAYEY SAND, CLAYEY SAND WITH GRAVEL
- CLAYEY GRAVEL
- SILT, SANDY SILT, SILT WITH GRAVEL, SILT WITH SAND
- ELASTIC SILT
- SILTY CLAY, SILTY CLAY WITH GRAVEL
- SILTY SAND, POORLY GRADED SAND WITH SILT AND GRAVEL, SILTY SAND WITH GRAVEL
- SILTY GRAVEL
- WELL GRADED SAND WITH SILT, WELL GRADED SAND WITH SILT AND GRAVEL
- POORLY GRADED SAND, POORLY GRADED SAND WITH GRAVEL, POORLY GRADED SAND WITH SILT
- POORLY GRADED GRAVEL, POORLY GRADED GRAVEL WITH SILT AND SAND, POORLY GRADED GRAVEL WITH SAND
- TOPSOIL
- FLY ASH
- BOTTOM ASH
- N STANDARD PENETRATION TEST BLOW COUNT (blows/ft)
- NMC NATURAL MOISTURE CONTENT (%)
- UC UNCONFINED COMPRESSIVE STRENGTH TEST (PSF)
- WATER LEVEL AND DATE RECORDED
- UNDISTURBED THIN-WALL (SHELBY) TUBE SAMPLE
- STANDARD PENETRATION TEST INTERVAL

SCALE: 1" = 10' EXCEPT AS NOTED

YARD
ASH DISPOSAL AREAS 2 AND 3
GEOTECHNICAL EXPLORATION
STABILITY SECTION D-D'

Stantec Consulting Services Inc.

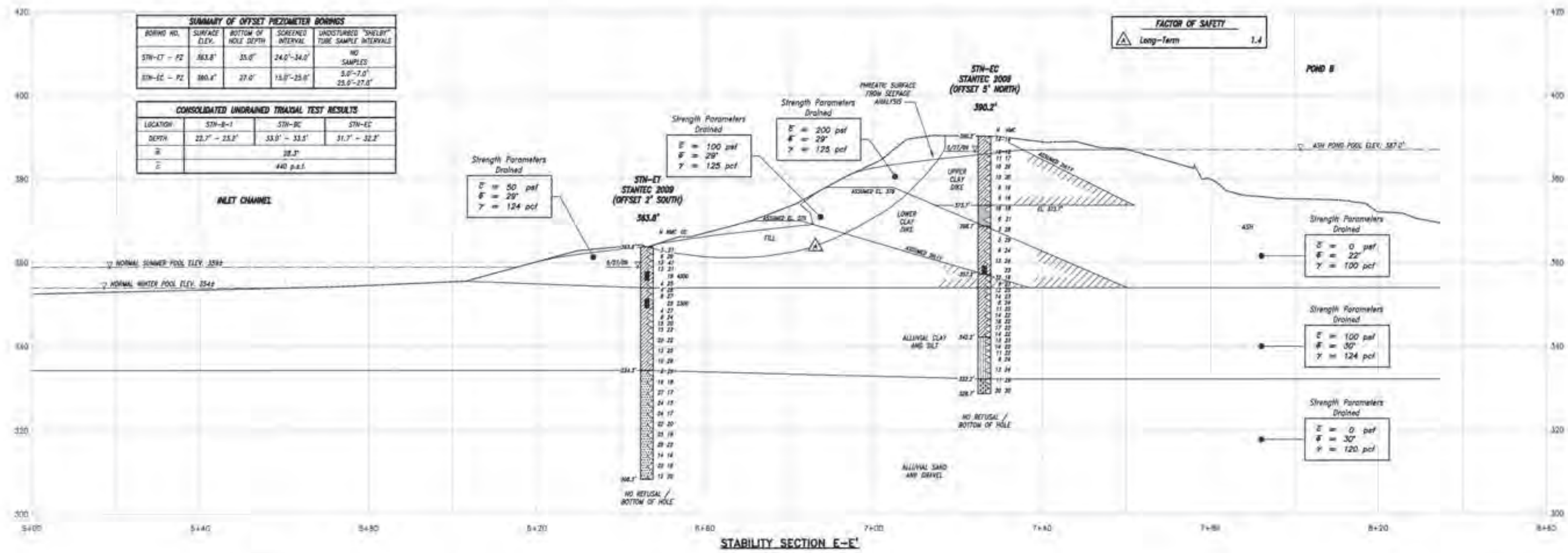
JOHNSONVILLE FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

AUTOCAD R 2009 | 34 | C | XXWXXX-07 | R 0

PLOT FACTOR: XX

C.A.D. DRAWING DO NOT ALTER MANUALLY

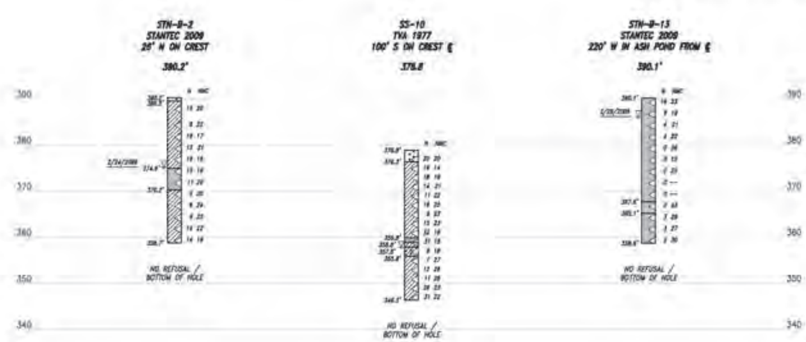
STANTEC | 0
TASK COMPLETED BY: | REV NO.



SUMMARY OF OFFSET PIEZOMETER BORINGS				
BORING NO.	SURFACE ELEV.	BOTTOM OF HOLE DEPTH	SCREENED INTERVAL	UNDISTURBED "SHELBY" TUBE SAMPLE INTERVALS
STN-ET – PZ	363.8'	35.0'	24.0'–34.0'	NO SAMPLES
STN-EC – PZ	380.4'	27.0'	15.0'–25.0'	5.0'–7.0' 25.0'–27.0'

CONSOLIDATED UNDRAINED TRIAXIAL TEST RESULTS			
LOCATION	STN-B-1	STN-BC	STN-EC
DEPTH	22.7' – 23.2'	33.0' – 33.5'	31.7' – 32.2'
φ̄	28.3°		
c̄	440 p.s.f.		

STABILITY SECTION E–E'



NOTE: SURVEYED CROSS-SECTIONS AND HORIZONTAL AND VERTICAL LOCATIONS OF 2009 BORINGS PROVIDED BY T.V.A. LOCATION OF PREVIOUSLY DRILLED BORINGS FROM BORING LAYOUTS PROVIDED IN PREVIOUS REPORTS.

LEGEND

- LEAN CLAY, SANDY LEAN CLAY, SANDY LEAN CLAY WITH GRAVEL, LEAN CLAY WITH GRAVEL, LEAN CLAY WITH SAND
- FAT CLAY
- CLAYEY SAND, CLAYEY SAND WITH GRAVEL
- CLAYEY GRAVEL
- SILT, SANDY SILT, SILT WITH GRAVEL, SILT WITH SAND
- ELASTIC SILT
- SILTY CLAY, SILTY CLAY WITH GRAVEL
- SILTY SAND, POORLY GRADED SAND WITH SILT AND GRAVEL, SILTY SAND WITH GRAVEL
- SILTY GRAVEL
- WELL GRADED SAND WITH SILT, WELL GRADED SAND WITH SILT AND GRAVEL
- POORLY GRADED SAND, POORLY GRADED SAND WITH GRAVEL, POORLY GRADED SAND WITH SILT
- POORLY GRADED GRAVEL, POORLY GRADED GRAVEL WITH SILT AND SAND, POORLY GRADED GRAVEL WITH SAND
- TOPSOIL
- FLY ASH
- BOTTOM ASH
- N STANDARD PENETRATION TEST BLOW COUNT (blows/ft)
- NMC NATURAL MOISTURE CONTENT (%)
- UC UNCONFINED COMPRESSIVE STRENGTH TEST (PSF)
- WATER LEVEL AND DATE RECORDED
- UNDISTURBED THIN-WALL (SHELBY) TUBE SAMPLE
- STANDARD PENETRATION TEST INTERVAL

NOTE
DATA FOR BORING STN-EC, SHOWN ABOVE, COMBINES DATA FROM BORINGS STN-EC AND STN-EC-A.

SCALE: 1" = 10' EXCEPT AS NOTED

YARD
ASH DISPOSAL AREAS 2 AND 3

GEOTECHNICAL EXPLORATION
STABILITY SECTION E–E'

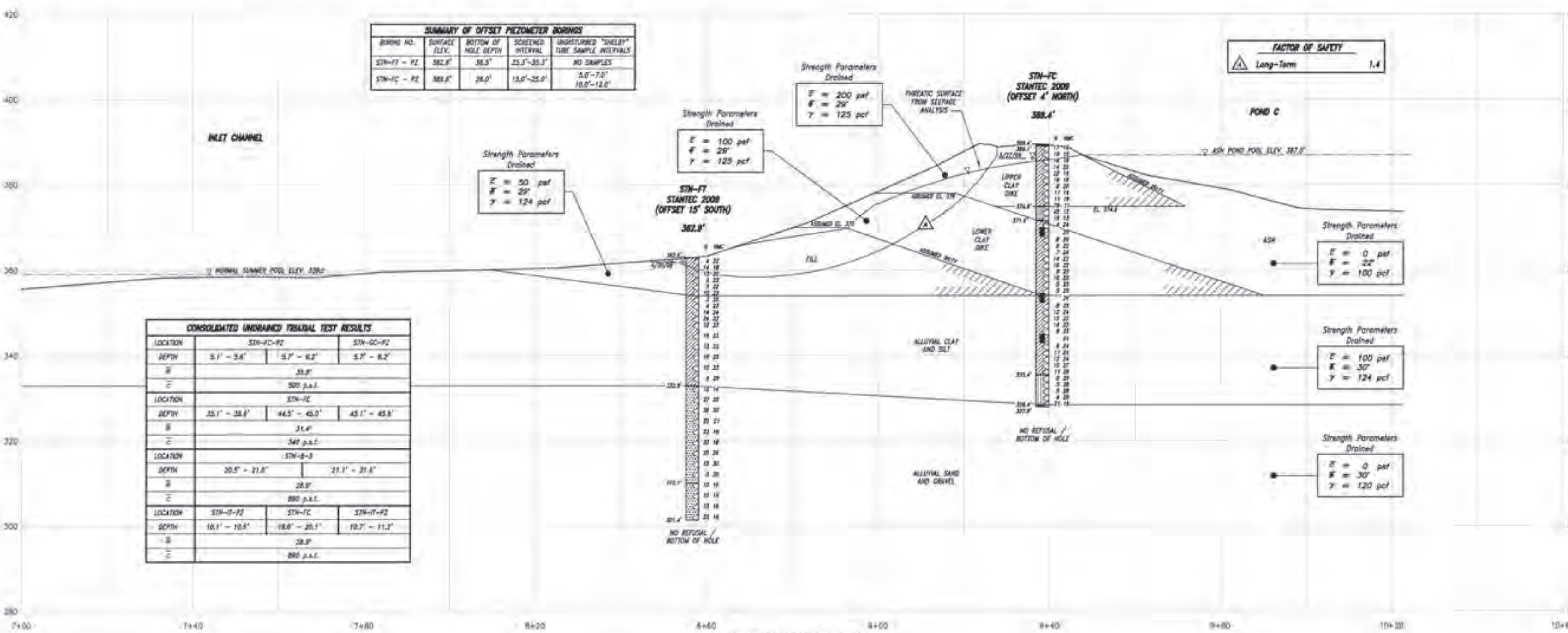
Stantec Consulting Services Inc.

JOHNSONVILLE FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

34 C XXWXXX–08 R 0

STANTEC 0
TASK COMPLETED BY REV. NO.

PLOT FACTOR
C.A.D. DRAWING DO NOT ALTER MANUALLY



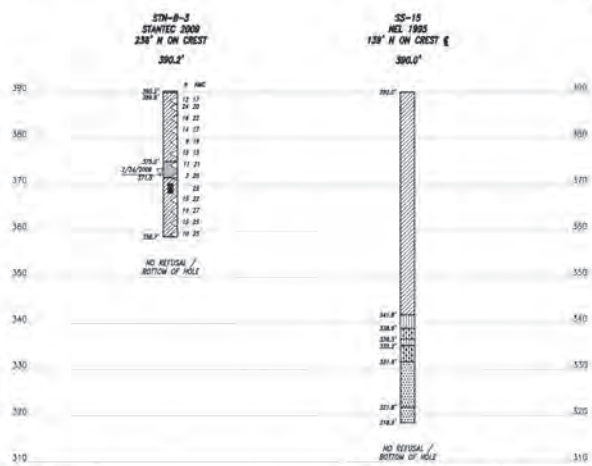
SUMMARY OF OFFSET PIEZOMETER BORINGS

BORING NO.	SURFACE ELEV.	BOTTOM OF HOLE DEPTH	SCREENED INTERVAL	UNDISTURBED "SHELBY" TUBE SAMPLE INTERVALS
STN-FT – PZ	382.9'	38.5'	25.3'–35.3'	NO SAMPLES
STN-FC – PZ	388.8'	28.0'	15.0'–25.0'	5.0'–7.0' 10.0'–12.0'

CONSOLIDATED UNDRAINED TRIAXIAL TEST RESULTS

LOCATION	STN-FC-PZ		STN-GC-PZ
DEPTH	5.1' – 5.6'	5.7' – 6.2'	5.7' – 6.2'
$\bar{\phi}$	33.8°		
$\bar{c}$	500 p.s.f.		
LOCATION	STN-FC		
DEPTH	35.1' – 35.6'	44.5' – 45.0'	45.1' – 45.6'
$\bar{\phi}$	31.4°		
$\bar{c}$	340 p.s.f.		
LOCATION	STN-B-3		
DEPTH	20.5' – 21.0'	21.1' – 21.6'	
$\bar{\phi}$	28.9°		
$\bar{c}$	880 p.s.f.		
LOCATION	STN-IT-PZ	STN-FC	STN-IT-PZ
DEPTH	10.1' – 10.6'	19.6' – 20.1'	10.7' – 11.2'
$\bar{\phi}$	28.5°		
$\bar{c}$	880 p.s.f.		

STABILITY SECTION F–F'



NOTE: SURVEYED CROSS-SECTIONS AND HORIZONTAL AND VERTICAL LOCATIONS OF 2009 BORINGS PROVIDED BY T.V.A. LOCATION OF PREVIOUSLY DRILLED BORINGS FROM BORING LAYOUTS PROVIDED IN PREVIOUS REPORTS.

LEGEND

- LEAN CLAY, SANDY LEAN CLAY, SANDY LEAN CLAY WITH GRAVEL, LEAN CLAY WITH GRAVEL, LEAN CLAY WITH SAND
- FAT CLAY
- CLAYEY SAND, CLAYEY SAND WITH GRAVEL
- CLAYEY GRAVEL
- SILT, SANDY SILT, SILT WITH GRAVEL, SILT WITH SAND
- ELASTIC SILT
- SILTY CLAY, SILTY CLAY WITH GRAVEL
- SILTY SAND, POORLY GRADED SAND WITH SILT AND GRAVEL, SILTY SAND WITH GRAVEL
- SILTY GRAVEL
- WELL GRADED SAND WITH SILT, WELL GRADED SAND WITH SILT AND GRAVEL
- POORLY GRADED SAND, POORLY GRADED SAND WITH GRAVEL, POORLY GRADED SAND WITH SILT
- POORLY GRADED GRAVEL, POORLY GRADED GRAVEL WITH SILT AND SAND, POORLY GRADED GRAVEL WITH SAND
- TOPSOIL
- FLY ASH
- BOTTOM ASH
- N STANDARD PENETRATION TEST BLOW COUNT (blows/ft)
- NMC NATURAL MOISTURE CONTENT (%)
- UC UNCONFINED COMPRESSIVE STRENGTH TEST (PSF)
- WATER LEVEL AND DATE RECORDED
- UNDISTURBED THIN-WALL (SHELBY) TUBE SAMPLE
- STANDARD PENETRATION TEST INTERVAL

SCALE: 1" = 10' EXCEPT AS NOTED

YARD
ASH DISPOSAL AREAS 2 AND 3

GEOTECHNICAL EXPLORATION
STABILITY SECTION F–F'

JOHNSONVILLE FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

Stantec Consulting Services Inc.

34 C XXWXXX–09 R 0

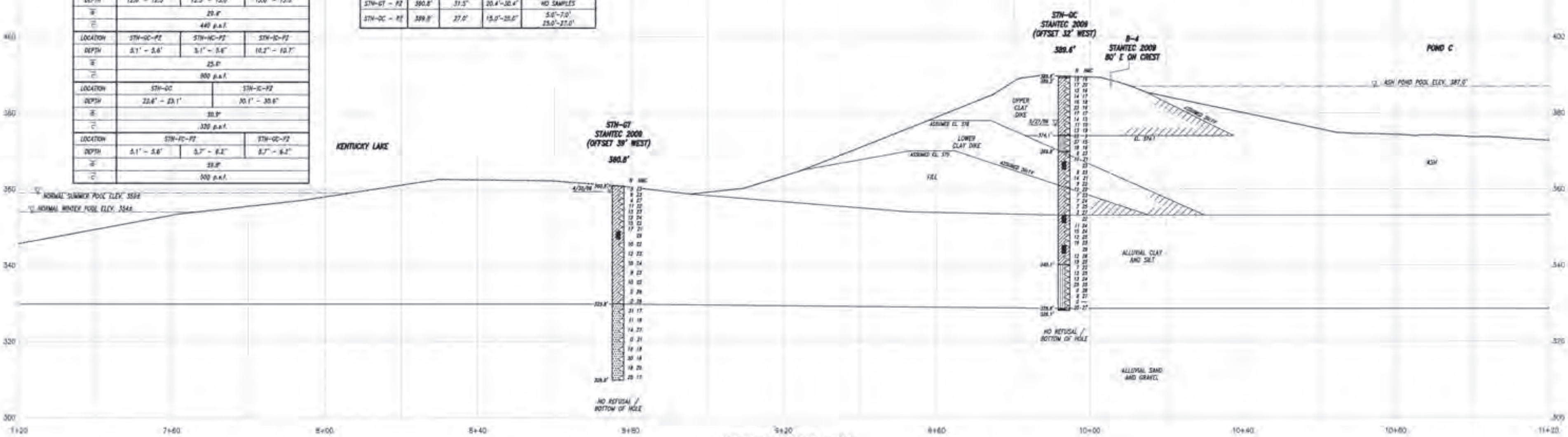
STANTEC | 0
TASK COMPLETED BY | REV NO.

PLOT FACTOR

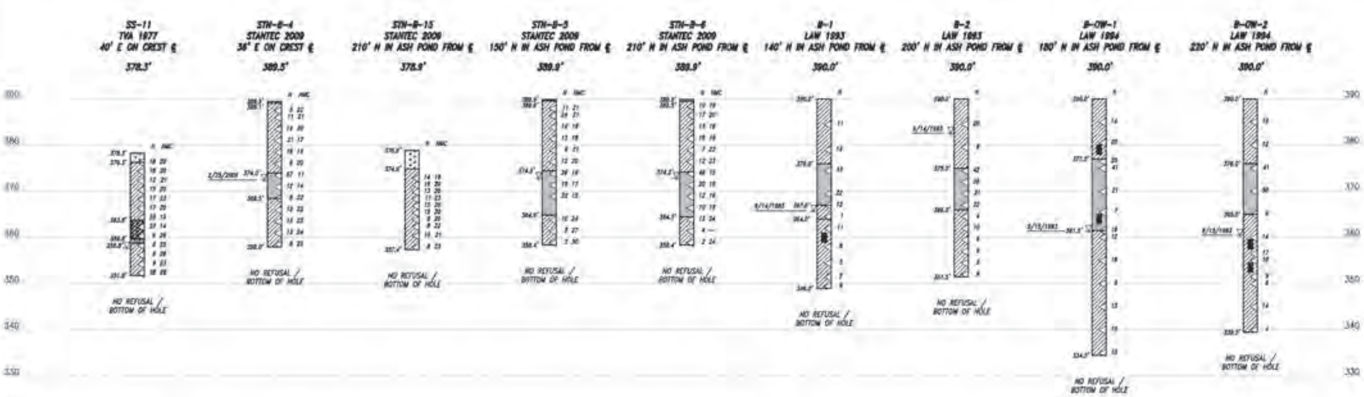
C.A.D. DRAWING DO NOT ALTER MANUALLY

CONSOLIDATED UNDRAINED TRIAXIAL TEST RESULTS			
LOCATION	STN-GT		
DEPTH	12.0' - 12.5'	12.5' - 13.0'	13.0' - 13.5'
$\bar{\phi}$	29.4°		
$\bar{c}$	440 p.s.f.		
LOCATION	STN-GC-PZ	STN-HC-PZ	STN-IC-PZ
DEPTH	5.1' - 5.6'	5.1' - 5.6'	10.2' - 10.7'
$\bar{\phi}$	25.0°		
$\bar{c}$	900 p.s.f.		
LOCATION	STN-GC	STN-IC-PZ	
DEPTH	22.6' - 23.1'	30.1' - 30.6'	
$\bar{\phi}$	30.9°		
$\bar{c}$	330 p.s.f.		
LOCATION	STN-FC-PZ		STN-GC-PZ
DEPTH	5.1' - 5.6'	5.7' - 6.2'	5.7' - 6.2'
$\bar{\phi}$	33.8°		
$\bar{c}$	300 p.s.f.		

SUMMARY OF OFFSET PIEZOMETER BORINGS				
BORING NO.	SURFACE ELEV.	BOTTOM OF HOLE DEPTH	SCREENED INTERVAL	UNDISTURBED "SHELBY" TUBE SAMPLE INTERVALS
STN-GT - PZ	380.8'	31.5'	20.4'-30.4'	NO SAMPLES
STN-GC - PZ	389.8'	27.0'	15.0'-25.0'	5.0'-7.0' 25.0'-27.0'



STABILITY SECTION G–G'



LEGEND

- LEAN CLAY, SANDY LEAN CLAY, SANDY LEAN CLAY WITH GRAVEL, LEAN CLAY WITH GRAVEL, LEAN CLAY WITH SAND
- FAT CLAY
- CLAYEY SAND, CLAYEY SAND WITH GRAVEL
- CLAYEY GRAVEL
- SILT, SANDY SILT, SILT WITH GRAVEL, SILT WITH SAND
- ELASTIC SILT
- SILTY CLAY, SILTY CLAY WITH GRAVEL
- SILTY SAND, POORLY GRADED SAND WITH SILT AND GRAVEL, SILTY SAND WITH GRAVEL
- SILTY GRAVEL
- WELL GRADED SAND WITH SILT, WELL GRADED SAND WITH SILT AND GRAVEL
- POORLY GRADED SAND, POORLY GRADED SAND WITH GRAVEL, POORLY GRADED SAND WITH SILT
- POORLY GRADED GRAVEL, POORLY GRADED GRAVEL WITH SILT AND SAND, POORLY GRADED GRAVEL WITH SAND
- TOPSOIL
- FLY ASH
- BOTTOM ASH
- N STANDARD PENETRATION TEST BLOW COUNT (blows/ft)
- NMC NATURAL MOISTURE CONTENT (%)
- qc UNCONFINED COMPRESSIVE STRENGTH TEST (PSF)
- WATER LEVEL AND DATE RECORDED
- UNDISTURBED THIN-WALL (SHELBY) TUBE SAMPLE
- STANDARD PENETRATION TEST INTERVAL

NOTE: SURVEYED CROSS-SECTIONS AND HORIZONTAL AND VERTICAL LOCATIONS OF 2009 BORINGS PROVIDED BY T.V.A. LOCATION OF PREVIOUSLY DRILLED BORINGS FROM BORING LAYOUTS PROVIDED IN PREVIOUS REPORTS.

NOTE
FOR CLARITY PURPOSES, BORINGS B-5, B-7, B-8 AND US-6B IN THE VICINITY OF STABILITY SECTION G FROM THE LAW ENGINEERING AND ENVIRONMENTAL SERVICES REPORT DATED JANUARY 17, 1994 AND TVA ASH POND EXPLORATION DATED SEPTEMBER 17, 1969 WERE NOT INCLUDED IN THE SECTION G PROFILE. LOGS OF THESE BORINGS CAN BE FOUND IN THEIR RESPECTIVE REPORTS.

SCALE: 1" = 10' EXCEPT AS NOTED

YARD
ASH DISPOSAL AREAS 2 AND 3
GEOTECHNICAL EXPLORATION
STABILITY SECTION G–G'

Stantec

JOHNSONVILLE FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

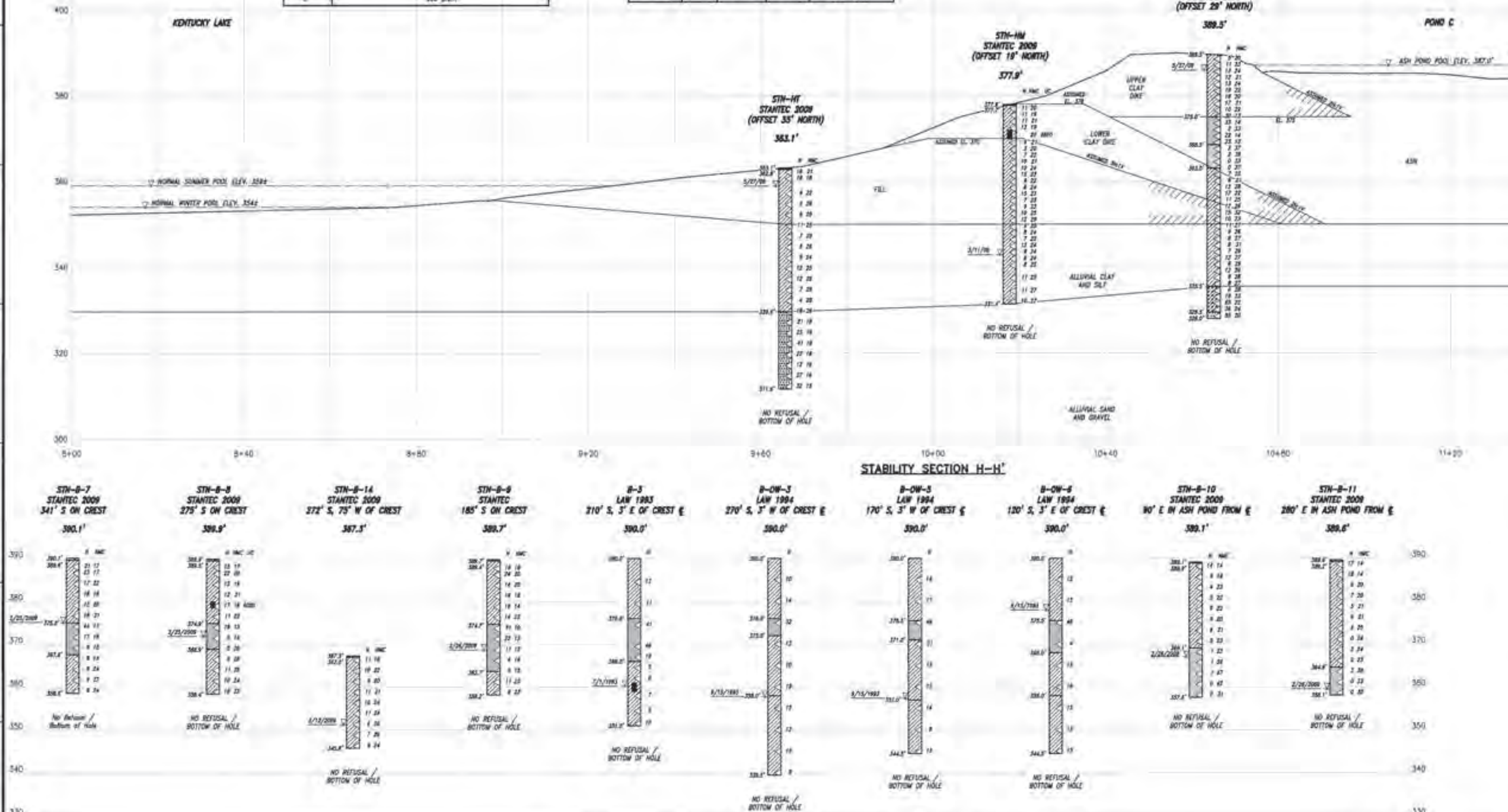
AUTOCAD R 2008 | 34 | C | XXWXXX-10 | R 0

PLOT FACTOR

E.A.D. DRAWING DO NOT ALTER MANUALLY

CONSOLIDATED UNDRAINED TRIAXIAL TEST RESULTS			
LOCATION	STN-GC-PZ	STN-HC-PZ	STN-IC-PZ
DEPTH	5.1' – 5.6'	5.1' – 5.6'	10.2' – 10.7'
$\bar{\phi}$	25.8°		
$\bar{c}$	900 p.s.f.		

SUMMARY OF OFFSET PIEZOMETER BORINGS				
BORING NO.	SURFACE ELEV.	BOTTOM OF HOLE DEPTH	SCREENED INTERVAL	UNDISTURBED "SHELBY" TUBE SAMPLE INTERVALS
STN-HT – PZ	383.1'	48.0'	27.0'–47.0'	NO SAMPLES
STN-HC – PZ	390.0'	37.0'	14.2'–24.2'	5.0'–7.0' 25.0'–27.0'



NOTE
FOR CLARITY PURPOSES, BORINGS B-6, B-9, AND B-OW-4 IN THE VICINITY OF STABILITY SECTION H FROM THE LAW ENGINEERING AND ENVIRONMENTAL SERVICES REPORT DATED JANUARY 17, 1994 WERE NOT INCLUDED IN THE SECTION H PROFILE. LOGS OF THESE BORINGS CAN BE FOUND IN THEIR RESPECTIVE REPORTS.

NOTE: SURVEYED CROSS-SECTIONS AND HORIZONTAL AND VERTICAL LOCATIONS OF 2009 BORINGS PROVIDED BY T.V.A.
LOCATION OF PREVIOUSLY DRILLED BORINGS FROM BORING LAYOUTS PROVIDED IN PREVIOUS REPORTS.

LEGEND

- LEAN CLAY, SANDY LEAN CLAY, SANDY LEAN CLAY WITH GRAVEL, LEAN CLAY WITH GRAVEL, LEAN CLAY WITH SAND
- FAT CLAY
- CLAYEY SAND, CLAYEY SAND WITH GRAVEL
- CLAYEY GRAVEL
- SILT, SANDY SILT, SILT WITH GRAVEL, SILT WITH SAND
- ELASTIC SILT
- SILTY CLAY, SILTY CLAY WITH GRAVEL
- SILTY SAND, POORLY GRADED SAND WITH SILT AND GRAVEL, SILTY SAND WITH GRAVEL
- SILTY GRAVEL
- WELL GRADED SAND WITH SILT, WELL GRADED SAND WITH SILT AND GRAVEL
- POORLY GRADED SAND, POORLY GRADED SAND WITH GRAVEL, POORLY GRADED SAND WITH SILT
- POORLY GRADED GRAVEL, POORLY GRADED GRAVEL WITH SILT AND SAND, POORLY GRADED GRAVEL WITH SAND
- TOPSOIL
- FLY ASH
- BOTTOM ASH
- STANDARD PENETRATION TEST BLOW COUNT (blows/ft)
- NMC NATURAL MOISTURE CONTENT (%)
- UC UNCONFINED COMPRESSIVE STRENGTH TEST (PSF)
- WATER LEVEL AND DATE RECORDED
- UNDISTURBED THIN-WALL (SHELBY) TUBE SAMPLE
- STANDARD PENETRATION TEST INTERVAL

SCALE: 1" = 10' EXCEPT AS NOTED

YARD
ASH DISPOSAL AREAS 2 AND 3

GEOTECHNICAL EXPLORATION
STABILITY SECTION H–H'

Stantec Consulting Services Inc.

JOHNSONVILLE FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

AUTOCAD R 2008 | 34 | C | XXWXXX–11

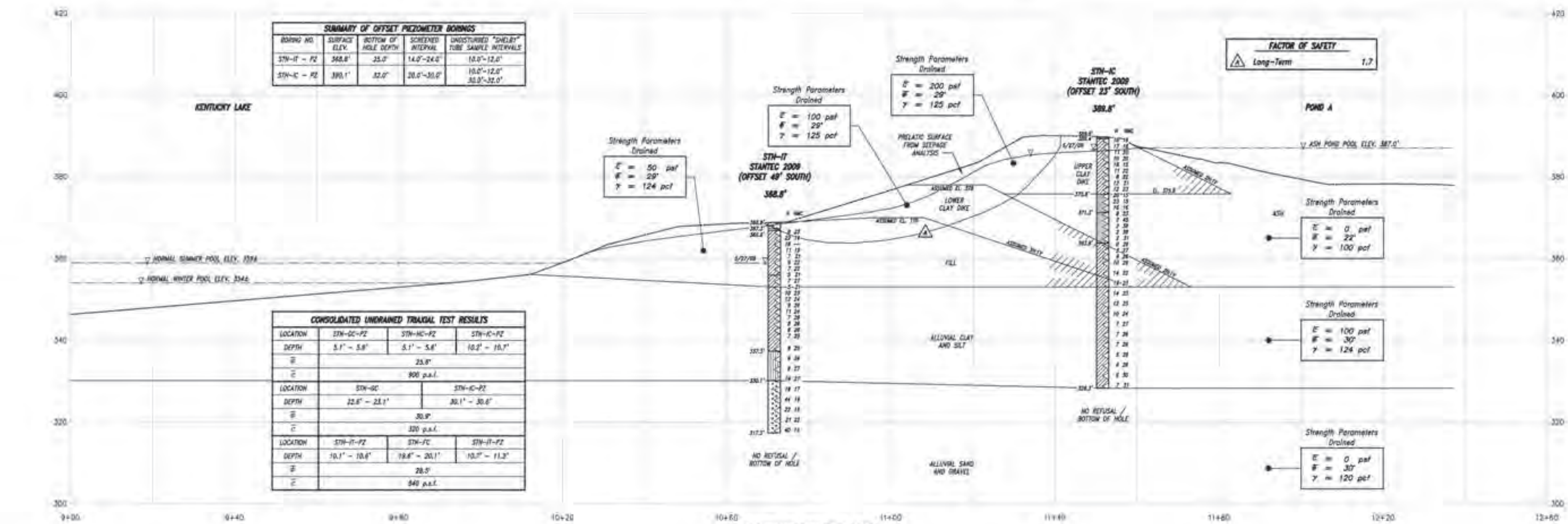
PLOT FACTOR-XX

C.A.D. DRAWING DO NOT ALTER MANUALLY

STANTEC | 0
TASK COMPLETED BY | REV NO

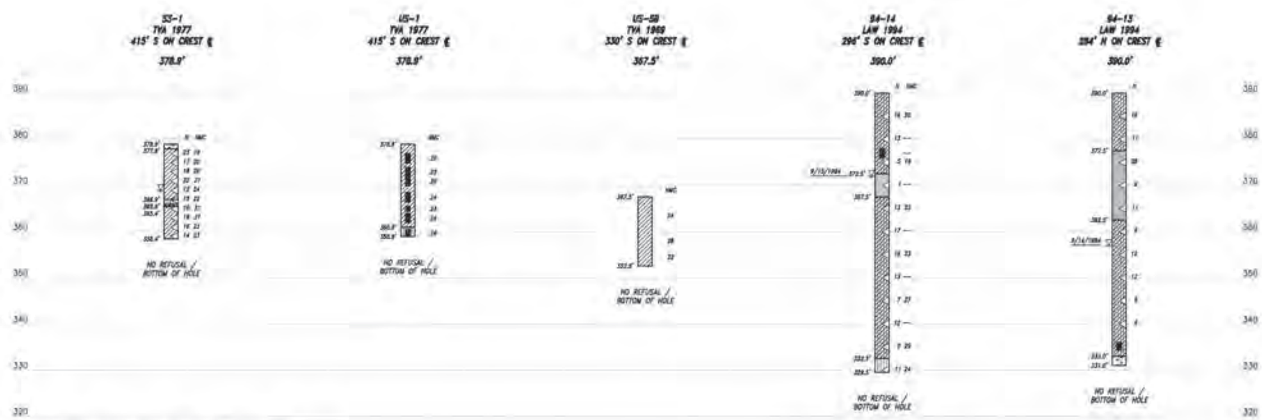
SUMMARY OF OFFSET PIEZOMETER BORINGS				
BORING NO.	SURFACE ELEV.	BOTTOM OF HOLE DEPTH	SCREENED INTERVAL	UNDISTURBED "SHELBY" TUBE SAMPLE INTERVALS
STN-IT – PZ	368.8'	25.0'	14.0'–24.0'	10.0'–12.0'
STN-IC – PZ	390.1'	32.0'	20.0'–30.0'	10.0'–12.0' 30.0'–32.0'

FACTOR OF SAFETY	
Long-Term	1.7



CONSOLIDATED UNDRAINED TRIAXIAL TEST RESULTS			
LOCATION	STN-GC-PZ	STN-HC-PZ	STN-IC-PZ
DEPTH	5.1' – 5.8'	5.1' – 5.6'	10.2' – 10.7'
$\bar{\phi}$	25.8°		
$\bar{c}$	900 p.s.f.		
LOCATION	STN-GC	STN-IC-PZ	
DEPTH	22.6' – 23.1'	30.1' – 30.6'	
$\bar{\phi}$	30.9°		
$\bar{c}$	320 p.s.f.		
LOCATION	STN-IT-PZ	STN-FC	STN-IT-PZ
DEPTH	10.1' – 10.6'	19.6' – 20.1'	10.7' – 11.2'
$\bar{\phi}$	28.5°		
$\bar{c}$	940 p.s.f.		

STABILITY SECTION I–I'



LEGEND

- LEAN CLAY, SANDY LEAN CLAY, SANDY LEAN CLAY WITH GRAVEL, LEAN CLAY WITH GRAVEL, LEAN CLAY WITH SAND
- FAT CLAY
- CLAYEY SAND, CLAYEY SAND WITH GRAVEL
- CLAYEY GRAVEL
- SILT, SANDY SILT, SILT WITH GRAVEL, SILT WITH SAND
- ELASTIC SILT
- SILTY CLAY, SILTY CLAY WITH GRAVEL
- SILTY SAND, POORLY GRADED SAND WITH SILT AND GRAVEL, SILTY SAND WITH GRAVEL
- SILTY GRAVEL
- WELL GRADED SAND WITH SILT, WELL GRADED SAND WITH SILT AND GRAVEL
- POORLY GRADED SAND, POORLY GRADED SAND WITH GRAVEL, POORLY GRADED SAND WITH SILT
- POORLY GRADED GRAVEL, POORLY GRADED GRAVEL WITH SILT AND SAND, POORLY GRADED GRAVEL WITH SAND
- TOPSOIL
- FLY ASH
- BOTTOM ASH
- N STANDARD PENETRATION TEST BLOW COUNT (blows/ft)
- NMC NATURAL MOISTURE CONTENT (%)
- UC UNCONFINED COMPRESSIVE STRENGTH TEST (PSF)
- WATER LEVEL AND DATE RECORDED
- UNDISTURBED THIN-WALL (SHELBY) TUBE SAMPLE
- STANDARD PENETRATION TEST INTERVAL

NOTE: SURVEYED CROSS-SECTIONS AND HORIZONTAL AND VERTICAL LOCATIONS OF 2009 BORINGS PROVIDED BY T.V.A. LOCATION OF PREVIOUSLY DRILLED BORINGS FROM BORING LAYOUTS PROVIDED IN PREVIOUS REPORTS.

SCALE: 1" = 10' EXCEPT AS NOTED

YARD
ASH DISPOSAL AREAS 2 AND 3
GEOTECHNICAL EXPLORATION
STABILITY SECTIONS I–I'

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JOHNSONVILLE FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

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PLOT FACTOR

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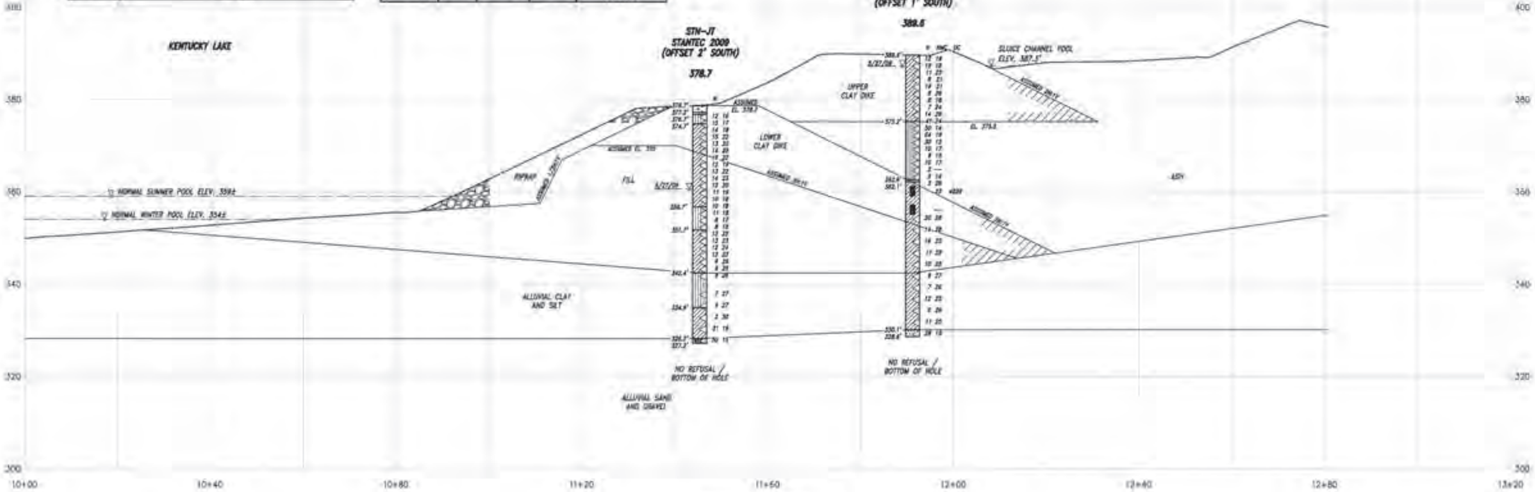
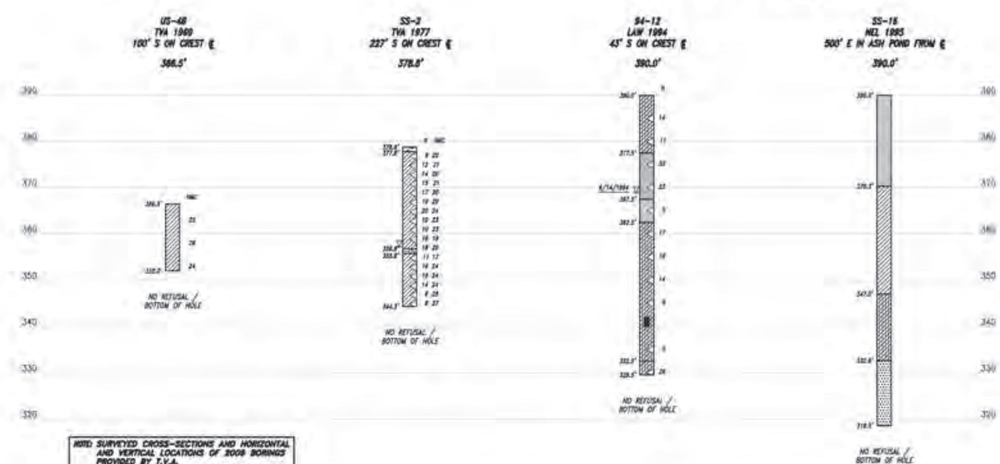
CONSOLIDATED UNDRAINED TRIAXIAL TEST RESULTS		
LOCATION	STN-JC-P2	STN-KD-P2	
DEPTH	5.1' – 5.4'	5.1' – 5.4'	10.1' – 10.6'
$\bar{\phi}$	33.8°		
$\bar{c}$	240 p.s.f.		

SUMMARY OF OFFSET PIEZOMETER BORINGS				
BORING NO.	SURFACE ELEV.	BOTTOM OF HOLE DEPTH	SCREENED INTERVAL	UNDISTURBED "SHELBY" TUBE SAMPLE INTERVALS
STN-JT – P2	378.7'	35.0'	24.0'-34.0'	10.0'-12.0' 22.0'-24.0'
STN-JC – P2	380.0'	25.0'	15.0'-25.0'	5.0'-7.0' 10.0'-12.0'



STABILITY SECTION J–J'



NOTE: SURVEYED CROSS-SECTIONS AND HORIZONTAL AND VERTICAL LOCATIONS OF 2009 BORINGS PROVIDED BY T.V.A. LOCATION OF PREVIOUSLY DRILLED BORINGS FROM BORING LAYOUTS PROVIDED IN PREVIOUS REPORTS.

LEGEND

- LEAN CLAY, SANDY LEAN CLAY, SANDY LEAN CLAY WITH GRAVEL, LEAN CLAY WITH GRAVEL, LEAN CLAY WITH SAND
- FAT CLAY
- CLAYEY SAND, CLAYEY SAND WITH GRAVEL
- CLAYEY GRAVEL
- SILT, SANDY SILT, SILT WITH GRAVEL, SILT WITH SAND
- ELASTIC SILT
- SILTY CLAY, SILTY CLAY WITH GRAVEL
- SILTY SAND, POORLY GRADED SAND WITH SILT AND GRAVEL, SILTY SAND WITH GRAVEL
- SILTY GRAVEL
- WELL GRADED SAND WITH SILT, WELL GRADED SAND WITH SILT AND GRAVEL
- POORLY GRADED SAND, POORLY GRADED SAND WITH GRAVEL, POORLY GRADED SAND WITH SILT
- POORLY GRADED GRAVEL, POORLY GRADED GRAVEL WITH SILT AND SAND, POORLY GRADED GRAVEL WITH SAND
- TOPSOIL
- FLY ASH
- BOTTOM ASH
- N STANDARD PENETRATION TEST BLOW COUNT (blows/ft)
- NMC NATURAL MOISTURE CONTENT (%)
- UC UNCONFINED COMPRESSIVE STRENGTH TEST (PSF)
- WATER LEVEL AND DATE RECORDED
- UNDISTURBED THIN-WALL (SHELBY) TUBE SAMPLE
- STANDARD PENETRATION TEST INTERVAL

SCALE: 1" = 10' EXCEPT AS NOTED

YARD
ASH DISPOSAL AREAS 2 AND 3

GEOTECHNICAL EXPLORATION
STABILITY SECTION J–J'

Stantec Consulting Services Inc.

JOHNSONVILLE FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

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PLOT FACTOR

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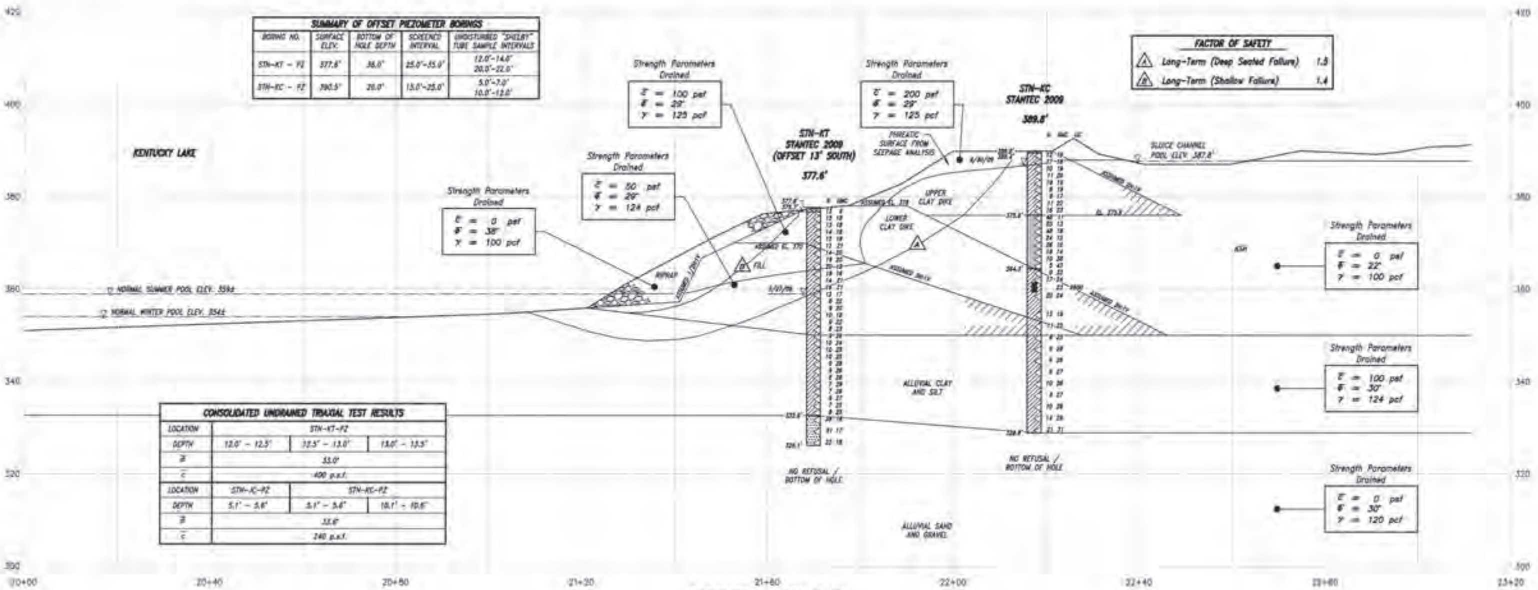
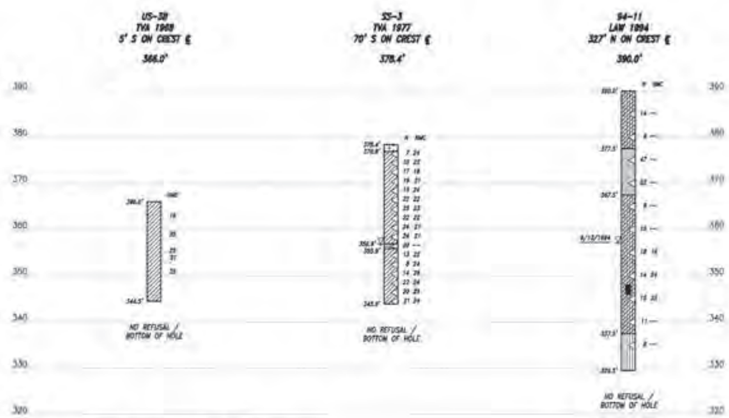


SUMMARY OF OFFSET PIEZOMETER BORINGS				
BORING NO.	SURFACE ELEV.	BOTTOM OF HOLE DEPTH	SCREENED INTERVAL	UNDISTURBED "SHELBY" TUBE SAMPLE INTERVALS
STN-KT – PZ	377.6'	36.0'	25.0'–35.0'	12.0'–14.0' 20.0'–22.0'
STN-KC – PZ	390.5'	20.0'	15.0'–25.0'	5.0'–7.0' 10.0'–12.0'

CONSOLIDATED UNDRAINED TRIAXIAL TEST RESULTS			
LOCATION	STN-KT-PZ		
DEPTH	12.0' – 12.5'	12.5' – 13.0'	13.0' – 13.5'
φ̄	33.0°		
c̄	400 p.s.f.		
LOCATION	STN-JC-PZ	STN-KC-PZ	
DEPTH	5.1' – 5.6'	5.1' – 5.6'	10.1' – 10.6'
φ̄	33.6°		
c̄	240 p.s.f.		

STABILITY SECTION K–K'



LEGEND

- LEAN CLAY, SANDY LEAN CLAY, SANDY LEAN CLAY WITH GRAVEL, LEAN CLAY WITH GRAVEL, LEAN CLAY WITH SAND
- FAT CLAY
- CLAYEY SAND, CLAYEY SAND WITH GRAVEL
- CLAYEY GRAVEL
- SILT, SANDY SILT, SILT WITH GRAVEL, SILT WITH SAND
- ELASTIC SILT
- SILTY CLAY, SILTY CLAY WITH GRAVEL
- SILTY SAND, POORLY GRADED SAND WITH SILT AND GRAVEL, SILTY SAND WITH GRAVEL
- SILTY GRAVEL
- WELL GRADED SAND WITH SILT, WELL GRADED SAND WITH SILT AND GRAVEL
- POORLY GRADED SAND, POORLY GRADED SAND WITH GRAVEL, POORLY GRADED SAND WITH SILT
- POORLY GRADED GRAVEL, POORLY GRADED GRAVEL WITH SILT AND SAND, POORLY GRADED GRAVEL WITH SAND
- TOPSOIL
- FLY ASH
- BOTTOM ASH
- N STANDARD PENETRATION TEST BLOW COUNT (blows/ft)
- NMC NATURAL MOISTURE CONTENT (%)
- UC UNCONFINED COMPRESSIVE STRENGTH TEST (PSF)
- WATER LEVEL AND DATE RECORDED
- UNDISTURBED THIN-WALL (SHELBY) TUBE SAMPLE
- STANDARD PENETRATION TEST INTERVAL

NOTE: SURVEYED CROSS-SECTIONS AND HORIZONTAL AND VERTICAL LOCATIONS OF 2008 BORINGS PROVIDED BY T.V.A. LOCATION OF PREVIOUSLY DRILLED BORINGS FROM BORING LAYOUTS PROVIDED IN PREVIOUS REPORTS.

SCALE: 1" = 10'   EXCEPT AS NOTED

YARD
ASH DISPOSAL AREAS 2 AND 3
GEOTECHNICAL EXPLORATION
STABILITY SECTION K–K'

JOHNSONVILLE FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

Stantec Consulting Services Inc.

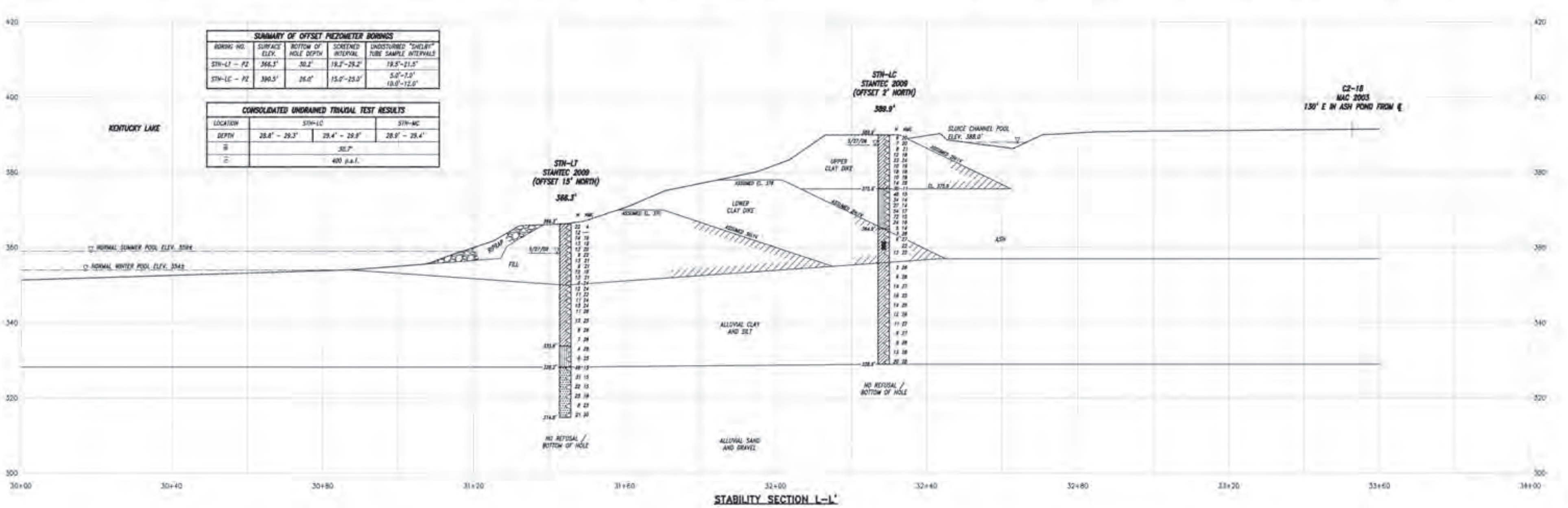
34 C XXWXXX–14

PLOT FACTOR:XX

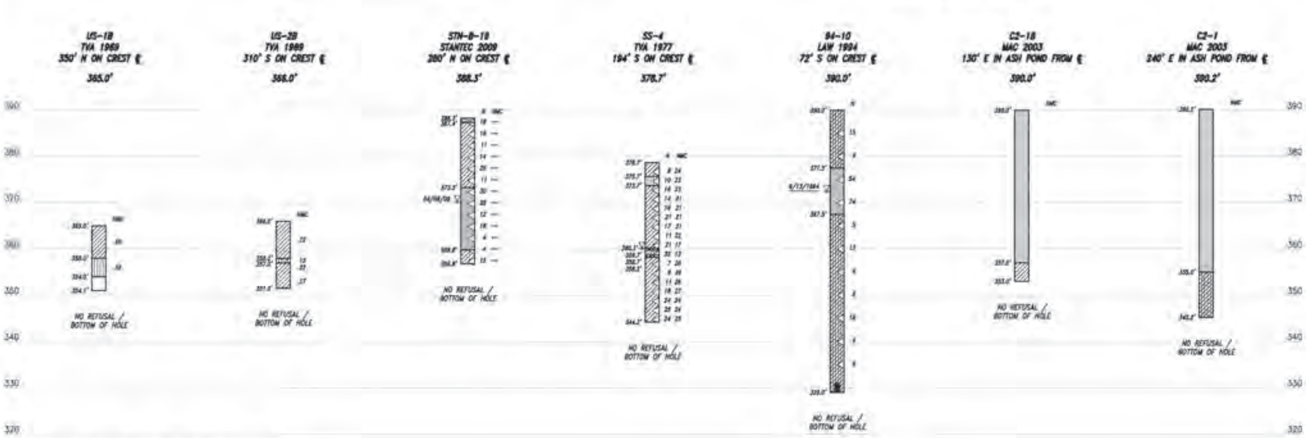
C.A.D. DRAWING DO NOT ALTER MANUALLY

SUMMARY OF OFFSET PIEZOMETER BORINGS				
BORING NO.	SURFACE ELEV.	BOTTOM OF HOLE DEPTH	SCREENED INTERVAL	UNDISTURBED "SHELBY" TUBE SAMPLE INTERVALS
STN-LT – PZ	366.5'	30.2'	19.2'–29.2'	19.5'–21.5'
STN-LC – PZ	390.5'	26.0'	15.0'–25.0'	5.0'–7.0' 10.0'–12.0'

CONSOLIDATED UNDRAINED TRIAXIAL TEST RESULTS			
LOCATION	STN-LC		STN-MC
DEPTH	28.8' – 29.3'	29.4' – 29.9'	28.9' – 29.4'
$\bar{\phi}$	30.7°		
$\bar{c}$	400 p.s.f.		



STABILITY SECTION L–L'



LEGEND

- LEAN CLAY, SANDY LEAN CLAY, SANDY LEAN CLAY WITH GRAVEL, LEAN CLAY WITH GRAVEL, LEAN CLAY WITH SAND
- FAT CLAY
- CLAYEY SAND, CLAYEY SAND WITH GRAVEL
- CLAYEY GRAVEL
- SILT, SANDY SILT, SILT WITH GRAVEL, SILT WITH SAND
- ELASTIC SILT
- SILTY CLAY, SILTY CLAY WITH GRAVEL
- SILTY SAND, POORLY GRADED SAND WITH SILT AND GRAVEL, SILTY SAND WITH GRAVEL
- SILTY GRAVEL
- WELL GRADED SAND WITH SILT, WELL GRADED SAND WITH SILT AND GRAVEL
- POORLY GRADED SAND, POORLY GRADED SAND WITH GRAVEL, POORLY GRADED SAND WITH SILT
- POORLY GRADED GRAVEL, POORLY GRADED GRAVEL WITH SILT AND SAND, POORLY GRADED GRAVEL WITH SAND
- TOPSOIL
- FLY ASH
- BOTTOM ASH
- N STANDARD PENETRATION TEST BLOW COUNT (blows/ft)
- NMC NATURAL MOISTURE CONTENT (%)
- UC UNCONFINED COMPRESSIVE STRENGTH TEST (PSF)
- WATER LEVEL AND DATE RECORDED
- UNDISTURBED THIN-WALL (SHELBY) TUBE SAMPLE
- STANDARD PENETRATION TEST INTERVAL

NOTE: SURVEYED CROSS-SECTIONS AND HORIZONTAL AND VERTICAL LOCATIONS OF 2009 BORINGS PROVIDED BY T.V.A. LOCATION OF PREVIOUSLY DRILLED BORINGS FROM BORING LAYOUTS PROVIDED IN PREVIOUS REPORTS.

SCALE: 1" = 10' EXCEPT AS NOTED

YARD
ASH DISPOSAL AREAS 2 AND 3
GEOTECHNICAL EXPLORATION
STABILITY SECTION L–L'

Stantec Consulting Services Inc.

JOHNSONVILLE FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

AUTOCAD R 2008 | 34 | C | XXWXXX–15 | R 0

PLOT FACTOR:XX

C.A.D. DRAWING DO NOT ALTER MANUALLY

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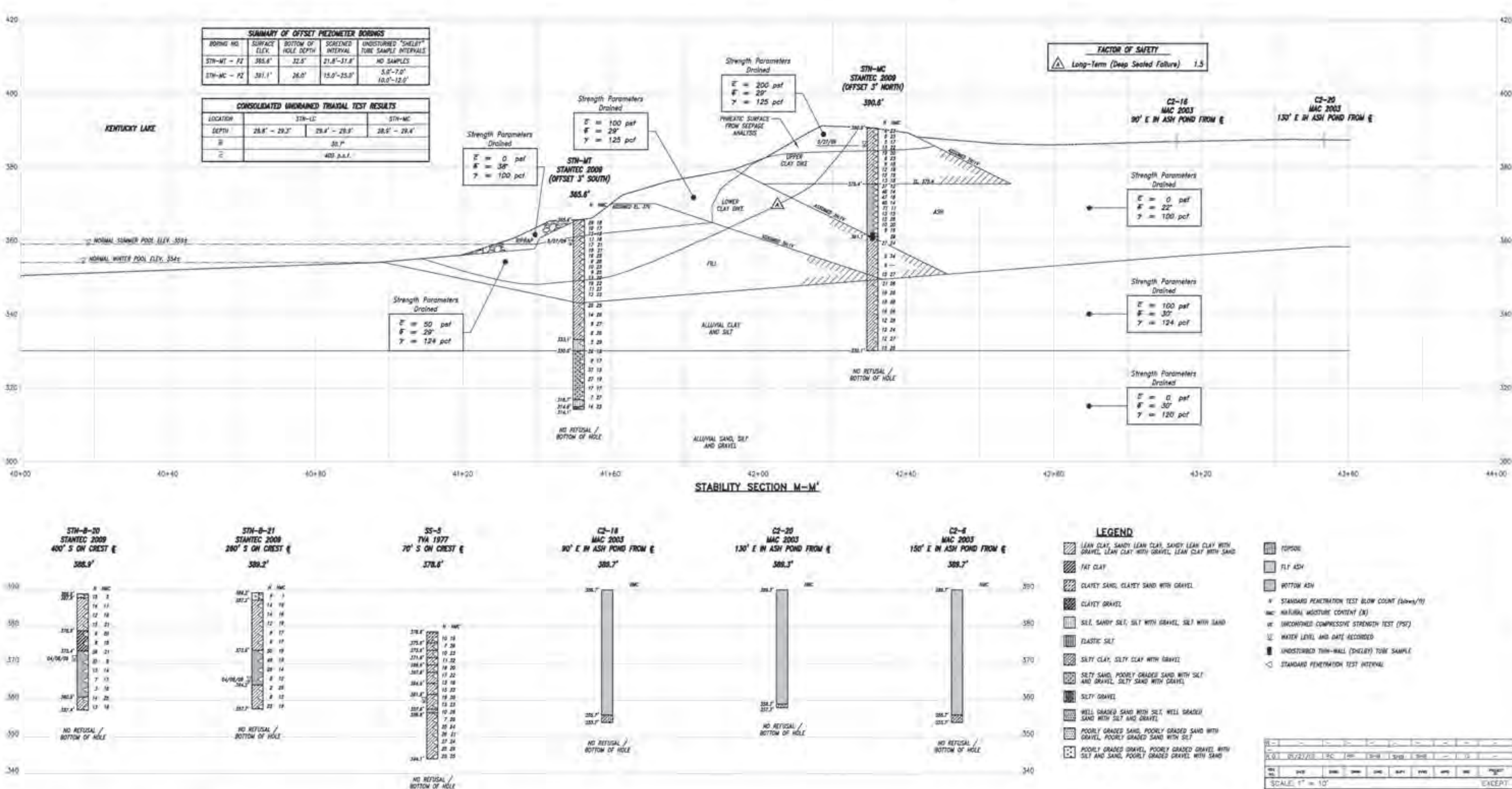

STABILITY SECTION M–M'

SUMMARY OF OFFSET PIEZOMETER BORINGS				
BORING NO.	SURFACE ELEV.	BOTTOM OF HOLE DEPTH	SCREENED INTERVAL	UNDISTURBED "SHELBY" TUBE SAMPLE INTERVALS
STN-MT – PZ	385.6'	32.5'	21.8'–31.8'	NO SAMPLES
STN-MC – PZ	391.1'	26.0'	15.0'–25.0'	5.0'–7.0' 10.0'–12.0'

CONSOLIDATED UNDRAINED TRIAXIAL TEST RESULTS			
LOCATION	STN-LC		STN-MC
DEPTH	28.8' – 29.2'	29.4' – 29.9'	28.5' – 29.4'
$\bar{\phi}$	30.7°		
$\bar{c}$	400 p.s.f.		

LEGEND

- LEAN CLAY, SANDY LEAN CLAY, SANDY LEAN CLAY WITH GRAVEL, LEAN CLAY WITH GRAVEL, LEAN CLAY WITH SAND
- FAT CLAY
- CLAYEY SAND, CLAYEY SAND WITH GRAVEL
- CLAYEY GRAVEL
- SILT, SANDY SILT, SILT WITH GRAVEL, SILT WITH SAND
- ELASTIC SILT
- SILTY CLAY, SILTY CLAY WITH GRAVEL
- SILTY SAND, POORLY GRADED SAND WITH SILT AND GRAVEL, SILTY SAND WITH GRAVEL
- SILTY GRAVEL
- WELL GRADED SAND WITH SILT, WELL GRADED SAND WITH SILT AND GRAVEL
- POORLY GRADED SAND, POORLY GRADED SAND WITH GRAVEL, POORLY GRADED SAND WITH SILT
- POORLY GRADED GRAVEL, POORLY GRADED GRAVEL WITH SILT AND SAND, POORLY GRADED GRAVEL WITH SAND
- TOPSOIL
- FLY ASH
- BOTTOM ASH
- N STANDARD PENETRATION TEST BLOW COUNT (blows/ft)
- NMC NATURAL MOISTURE CONTENT (%)
- UC UNCONFINED COMPRESSIVE STRENGTH TEST (PSF)
- WATER LEVEL AND DATE RECORDED
- UNDISTURBED THIN-WALL (SHELBY) TUBE SAMPLE
- STANDARD PENETRATION TEST INTERVAL

NOTE: SURVEYED CROSS-SECTIONS AND HORIZONTAL AND VERTICAL LOCATIONS OF 2009 BORINGS PROVIDED BY T.V.A. LOCATION OF PREVIOUSLY DRILLED BORINGS FROM BORING LAYOUTS PROVIDED IN PREVIOUS REPORTS.

SCALE: 1" = 10' EXCEPT AS NOTED

YARD
ASH DISPOSAL AREAS 2 AND 3
GEOTECHNICAL EXPLORATION
STABILITY SECTION M–M'

Stantec Consulting Services Inc.

JOHNSONVILLE FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

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PLOT FACTOR:XX

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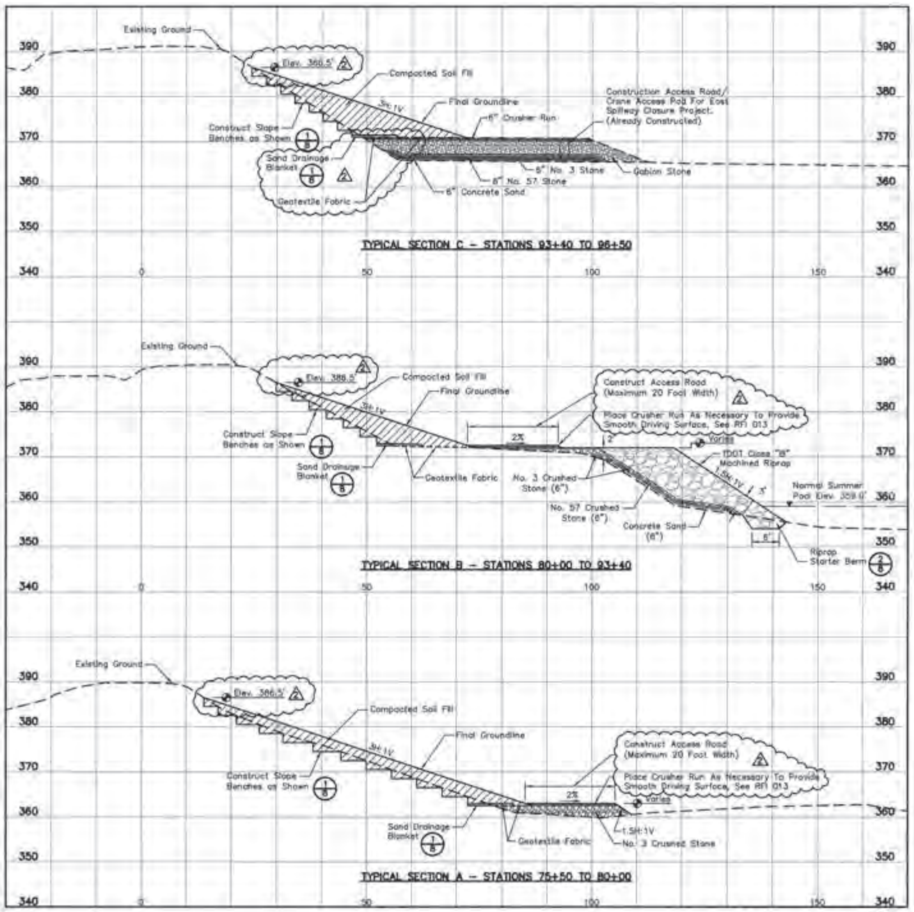
APPENDIX A

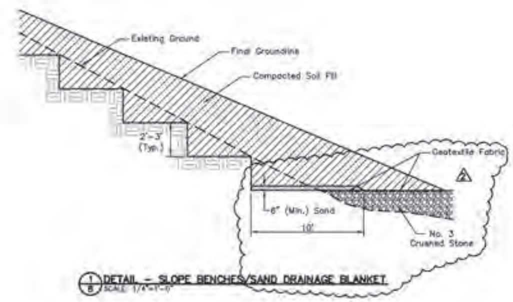
Document 7

Design Sections Southeast Dike – Selected Dwgs

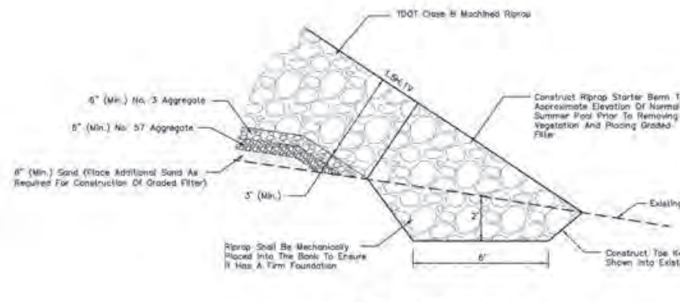


Notes:

1. Geotextile Fabric Shall Be Placed Between No. 3 Stone And The Sand Drainage Blanket Or Compacted Soil Fill.
2. Excavate Soft Zones And Replace With Compacted Soil, Sand, or Crushed Stone As Required To Stabilize For Proper Compaction Of Fill.



DETAIL – SLOPE BENCHES/SAND DRAINAGE BLANKET
SCALE: 1/4"=1'-0"



DETAIL – RIPRAP STARTER BERM
SCALE: 1/2"=1'-0"

ISSUED FOR CONSTRUCTION

Section or Detail No.
Sheet Where Shown
REFERENCE KEY

For Supporting Design Calculations See FPG.JOFFESCDX00000020100010

PROJECT REVISION HISTORY

SCALE: AS SHOWN — EXCEPT AS NOTED

YARD
ASH DISPOSAL AREA NO. 2
SOUTHEAST DIKE IMPROVEMENTS
TYPICAL SECTIONS
WORK PLAN 7 (JOF-100702-WP-7)

Stantec Consulting Services Inc.

JOHNSONVILLE FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

AUTOCAD R 2000 | 30 | C | 10W550-08 | R 2

PLOT FACTOR:XX

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APPENDIX A

Document 8

Stantec Phase 1 Report

TVA Disposal Facility Assessment
Phase 1 Plant Summary
Johnsonville Fossil Plant (JOF)

Location:	Johnsonville Fossil Plant (JOF) 535 Steam Plant Road New Johnsonville, Humphreys County, TN 37134 **Latitude:** 36.035 N **Longitude:** 87.984 W
Plant Contact:	Tony Dillion Program Administrator **Phone:** 931-535-8206 **Email:** ardillion@tva.gov
Facts and Figures:	The Johnsonville Fossil Plant has ten coal-fired generating units. Construction began in 1949 and was completed in 1952. The plant consumes approximately 9,600 tons of coal per day. It is located on the Tennessee River at Kentucky Lake, and is about 35 miles west of Dickson, TN.
Coal Combustion Byproduct Disposal:	Approximately 260,000 tons of fly ash is wet-sluiced to the Active Ash Disposal (Areas 2 & 3) each year. Roughly all of this fly ash is being hauled to an offsite structural fill project. In addition, previously deposited fly ash is being dredged to an internal cell, dewatered and hauled to the offsite structural fill site. Approximately 30,000 tons per year of bottom ash is wet-sluiced to the Active Ash Disposal. Dewatered bottom ash is reclaimed from the Active Ash Disposal and stacked within the pond footprint for later use in the offsite structural fill project.
Geology and Seismicity:	The Johnsonville Fossil Plant is located in west-central Tennessee along the eastern bank of the Tennessee River, just south (upstream) of the confluence of the river and Trace Creek. As such, much of the site is underlain by alluvium and terrace deposits varying in thickness from less than 20 feet along the tributary stream banks up to more than 100 feet within the floodplain of the Tennessee River. The underlying bedrock consists of the Lower Mississippian age Fort Payne Formation and Devonian age Chattanooga Shale and Camden Formations, in general order of descending lithology. The Fort Payne Formation varies from a sandy, cherty limestone in the upper portions of the unit to an interbedded shale and cherty limestone lower in the stratigraphic column. The Chattanooga Shale is a fissile, carbonaceous shale thought to act as an aquitard preventing the downward migration of groundwater, etc. into the underlying Camden formation, the principal aquifer in the region.

The Camden formation consists of thin beds of cherty limestone interbedded with softer clay layers. Previous drilling at the site, discussed in reports and other documentation provided by TVA, suggests the presence of several small faults and a larger fault in the bedrock underlying the plant, as inferred from borehole data in the Camden Formation.

Evaluations of seismic hazards affecting the western portion of middle Tennessee, and thus the plant site, are dominated by events emanating from the New Madrid Seismic Zone (NMSZ) of the central Mississippi Valley. The NMSZ is the most active seismic zone east of the Rocky Mountains and the continuing seismicity of the zone is thought to be associated with the reactivation of faults within the Reelfoot Rift System. Although the majority of the events emanating from this zone are too small to be felt at the surface, this zone produced a series of four earthquakes between December 1811 and early February 1812 each exhibiting estimated magnitudes on the order of 7.0 to 8.0. The “Geologic Hazards Map of Tennessee – Environmental Geology Series No. 5” developed and published by the Tennessee Department of Environment and Conservation (TDEC), Division of Geology and compiled by Robert Miller (1978) shows the plant to be located in Seismic Risk Zone 2.

Facilities Reviewed:

Active Ash Disposal Areas 2 & 3
South Railroad Loop Ash Disposal Area 4
Ash Dredge Pond East of Gas Turbines Area 5
North Abandoned Ash Disposal Area A

1. General Facility Information

Facility Status:	Active	**NID Identification:**	TN08512
Surface Area (inside dikes)	87 acres	**Maximum Height (toe to top of dike):**	36 feet
Free Water Volume:	Not provided by TVA	**Maximum Water Storage:**	Not provided by TVA
Estimated CCB Storage:	Not provided by TVA	**Dike Length:**	10,150 feet
Plant Discharge to Facility:	32 MGD	**Current Pool Elevation:**	387.5 feet

2. Site Visit Information

Stantec Assessment Team:	Stephen Bickel, PE, Nathan Bader, PE, Josh Kopp, EIT
TVA Staff Present:	Stuart Harris, Tony Dillon
Field Assessment Dates:	January 12, 2009 and February 23 - 25, 2009
Weather/Site Conditions:	Clear, moist ground during both assessments

3. History/Description of Usage

History and Operation:

Approximately 260,000 tons of fly ash is wet-sluiced to the Active Ash Disposal Areas 2 & 3 each year. Roughly all of this fly ash is being hauled to an offsite structural fill project. In addition, previously deposited fly ash is being dredged to an internal cell, dewatered and hauled to the offsite structural fill site. Approximately 30,000 tons per year of bottom ash is wet-sluiced to the Active Ash Disposal Area. Dewatered bottom ash is reclaimed from the Active Ash Disposal Area and stacked within the pond footprint for later use in the offsite structural fill project. Outlet is through the southern spillway which consists of two 48 inch RCP riser pipe/weirs that discharge through two 36 inch RCP sections into Kentucky Lake. The third spillway in this area has been raised and is not in use. Two other sets of spillways used in the past are also present; one to the northwest and one set to the southeast. The southeast set of spillways consist of three risers that have been raised and are no longer in use. The northwest set of spillways consists of three risers that were reportedly filled with concrete to abandon them. Ash Disposal Areas 2 & 3 was initially constructed in the late 1960s and was brought into service in 1970. The pond was constructed on an island with an initial 5 to 11 foot tall clay dike (Crest El. 370 feet). The dikes were reportedly raised in the early 1970s an additional 8 feet (Crest elevation 378 feet) using an upstream method with new clay dikes. Again in 1978, the dikes were raised another 12 feet (Crest elevation 390 feet) with clay using upstream methods. In both cases, the raised dikes were constructed over bottom ash placed within the pond as a base. A 4 foot cutoff trench was also excavated along the interior slope face and filled with clay to help tie the two dikes together and minimize seepage.

Past Failures/Releases: No failures or releases reported.

4. Owner's Operations, Maintenance and Inspection Information

Emergency Action Plan: No EAP has been prepared for this facility.

Operations Manual: A Byproducts Operations Manual is available for the Johnsonville Fossil Plant, covering all active facilities.

TVA Maintenance: Exterior slopes mowed twice annually.

TVA Inspections: TVA Engineering performs annual dike inspections and prepares reports for repair/maintenance activities. Plant personnel recently started making daily observations and performing weekly reviews of the disposal facilities at this plant.

Problems Previously Identified During Past TVA Inspections: Seepage along northeast and southeast slopes, animal burrows, heavy vegetation, isolated trees and depressions along exterior slopes at various areas around pond, pond freeboard is less than design, steep exterior slopes, sinkhole formed in the past above the south discharge pipes, abandoned weir structures, minimal storage capacity.

5. Documents Reviewed

See attached Document Log for complete list of documents provided by TVA for review. In particular, the following provided pertinent information for the assessment of this facility:

TVA Design Drawings:	Drawing numbers 10W527, 527-1, 527-2, 528, 529, 10N502, 503, 524, 528, 529, 531, 10E200-01, JOFNC01, 604B887R0, 604K861R1, 604K862R0, 604K881R0 through 886R0, KY Lake Safety Harbor 1 and 2, 461K509.
TVA As-Built Drawings:	Some previous dikes are shown on the drawings listed above, but are not documented as being as-built.
TVA Construction Testing Records:	None available.
TVA Annual Inspection Reports:	TVA Annual Inspection Reports 1970 to 2008.
Geotechnical Data:	"Johnsonville Steam Plant-Ash Disposal Area No. 2 Dike Raising, Soil Exploration and Testing", Memorandum from G. Farmer to G.L. Buchanan, November 22, 1977. "Report of Geotechnical Evaluation: Ash Pond Dike: New Johnsonville Plant", Law Engineering, January 1994. "Subsurface Exploration Data: TVA Borings at Johnsonville Fossil Plant", Law Engineering, October 11 1994. "Report of Subsurface Exploration and Stability Analysis, Johnsonville Fossil Plant Ash Disposal Area, New Johnsonville", Law Engineering and Environmental Services, Inc., September 19, 1997. "Report of Ash Pond Investigation: Johnsonville Fossil Plant, New Johnsonville, Tennessee", MACTEC Engineering and Consulting, August 28, 2003 "Results of Laboratory Testing-Grab Samples from Active Ash Pond", performed by Law Engineering, July 1995.

"Johnsonville Steam Plant-Ash Pond-Soil and Foundation Exploration", Memorandum from J.C. McGraw to F.P. Lacy, TVA, September 17, 1969.

"Johnsonville Groundwater Assessment", TVA Resource Group, Engineering Services, March 1995.

"Geology of the New Johnsonville Steam Plant Site", TVA Water Control Planning Dept., Geologic Division, January 14,1948.

6. Stantec Field Observations

See attached Concerns/Photo Log, Photos, and Site Plan Drawing.

6.1. Interior Slopes

Vegetation:	Tall grass, phragmites, dense coverage.	
Trees:	None observed.	
Wave Wash Protection:	Rip-rap slope protection present within portions of the pond (primarily within stilling pond and portions of the divider dikes).	
Erosion:	Few locations of wave erosion, size and length vary.	
Instabilities:	None observed.	
Animal Burrows:	None observed.	
Freeboard:	**Measured:**	2 feet. at Section 7
	Design:	4 feet
Encroachments:	Dewatering of fly ash and bottom ash is performed internally within the central portion of the pond.	
Slope:	**Measured:**	2.0H:1V (Estimated)
	Design:	2.0H:1V (from drawing 10W527)

6.2. Crest

Crest Cover and Slope:	Gravel access road, crest appeared relatively flat.
Erosion:	None observed.
Alignment:	Alignment appeared consistent with design drawings.
Settlement/Cracking:	None observed.
Bare Spots/Rutting:	None observed.
Width:	**Measured:** 23 feet at Section 7 20 feet at Section 10 **Design:** 16 feet for perimeter dike (from drawing 10W527)

6.3. Exterior Slopes

Vegetation:	Mostly grass with briars in various areas, adequate coverage. Briars have taken over slopes in the past and will continue to do so if not cleared regularly.
Trees:	Trees have been removed from majority of exterior slopes with the exception of those areas along the toe of the dike along the southern end of the pond.
Erosion:	Erosion rills, transverse depressions observed in various areas.
Instabilities:	Some minor shallow sloughing observed primarily along the eastern side of the pond.
Uniform Appearance:	Slopes appear fairly uniform.
Seepage:	Significant seepage along northeast and southeast dikes. Seepage collection system recently installed along southeast dike for better monitoring. Wet areas are present within the seepage areas observed. Standing water along the access road at the toe of the northeast dike was also observed.
Benches:	Benches observed along the northwestern portions of the dike. These benches appear to have been constructed for access by equipment to make repairs in the past.

Foundations, Drains, Relief Wells, Instrumentation:	No provisions for drainage/seepage control or instrumentation were observed with the exception of the recently installed seepage collection system on the southeast dike.	
Animal Burrows:	Numerous animal burrows observed throughout the majority of the dike on all sides.	
Slope:	**Measured:**	1.7H:1V at Sections 7, 8, 10, and 11 1.5H:1V at Section 9
	Design:	2.0H:1V with 3H:1V or flatter slopes below Elevation 378 feet (from drawing 10W527)
Height:	**Measured:**	Varies 20 to 30 feet
	Design:	Approximately 30 feet (from drawing 10W527)

6.4. Spillway Weirs/Riser Inlets

Number:	Three sets of 3 spillways; one set to the northwest (abandoned), one to the southeast (raised but not closed), and the current active set to the southwest.
Size, Type and Material:	48 inch RCP push-together riser sections with standard TVA steel skimmers.
Height of Riser Inlets:	Approximately 36 feet for the current active spillways.
Access:	Catwalk present to northernmost active spillway. No other access to current or abandoned spillways observed.
Joints:	Unable to observe joints or leakage below inlet level.
Mis-Alignment:	None observed or reported.
Closed/Abandoned Conduits:	The three spillways to the northwest were reportedly closed by filling them with concrete. Ash was covering these spillways at the time of this assessment and they could not be reviewed. The three spillways to the southeast were raised but no further efforts to close these structures were reported. The center spillway within the active set was raised and taken out of service due to what was believed to be joint separation in the discharge

pipe which caused a sinkhole to form along the exterior dike slope. Efforts were made to slip line this spillway but were unsuccessful.

6.5. Outlet Pipes

Number:	Three (3) abandoned to the northwest Three (3) currently out of service to the southeast Two (2) active and 1 out of service to the southwest
Size, Type and Material:	36 inch RCP
Headwall:	None observed or reported.
Joint Separations:	Separation in the central discharge pipe within the southwest set of spillways reported resulting in sinkhole on exterior slope. Slope was reportedly repaired. Efforts were made to slip line the pipe but were unsuccessful. The spillway was raised and taken out of service.
Mis-Alignment:	None observed.
Closed/Abandoned Conduits:	7 of 9 spillways have been taken out of service or closed as described above.

7. Notable Observations and Concerns

- The absence of an Emergency Action Plan, Operation and Maintenance Plan, as-built drawings and construction testing records is a concern.
- RCP push-together stacked riser structure spillways are a concern. A significant volume of water passes through the two open spillways with surging observed at the discharge into Kentucky Lake. The surging noted increases the potential for piping and internal erosion of the dike at joints in the discharge pipes. Document reviews indicate that in the late 1980s and early 1990s, sinkholes formed along the outslope below the current active spillways. It is believed that joint separation along the buried discharge pipes caused the subsidence. The area was repaired with rip-rap and the slope restored. No further documentation indicating that a detailed evaluation of the damaged structures was performed.
- Significant seepage present along the southeast and east dikes is a primary concern. A new seepage collection system has been installed along the toe of the southeast dikes with a single outlet for better monitoring. Continued evaluation of these seepage areas will be required.

- The perimeter dike outer slopes, particularly those along the east and southeast dikes are steep. Slopes of about 1.7H:1V were measured along the east and southeast sides of the perimeter dike. In addition, the hummocky and uneven surfaces that exist in several areas may be evidence of shallow slope movement (creep). Slope stability is a concern.
- Raising the dikes by using upstream construction over sluiced ash is a potential slope stability concern.
- The composition of the perimeter dikes and foundation materials are unknown. Considering the perimeter dike's steepness, height, and areas of seepage, it is a concern that the composition and engineering properties of the foundation and dike materials are largely unknown.
- The pond is operating at a high level with freeboard of about 2 feet or less. This is a primary concern when considering the seepage areas, slope stability issues, unknown composition of the dike and foundation materials, and potential for overtopping that is present.
- There are two sets of abandoned weir structures within the active pond. The first set is located to the northwest and the second is located along the southeast side of the pond. Each set has three structures. The freeboard at these abandoned structures is minimal, and the methods used for closing the northwest set of structures are relatively unknown. The southeast set of spillways have not been closed but have merely been raised to take them out of service.
- Animal burrows were noted along the perimeter dike faces in several areas. The animal burrows are abundant and have been reported for several years.
- Shallow depressions were observed in several areas on the perimeter dike outer slope along the west side. These depressions have been observed for several years and could be attributed to tree removal.
- There are several shallow transverse depressions and erosion rills on the southeast dike outer slope. The rills and depressions begin immediately below the crest and extend to the toe of slope in most cases. These are likely erosion rills even though there does not appear to be evidence of concentrated runoff from the dike road in these areas.
- Some rutting was observed along the toe of the east perimeter dike. The rutting was previously reported in annual inspections and is likely due to traffic within the seepage areas in this area.
- Phragmites are present on some of the divider dikes, the interior pond slopes and at exterior slope seepage areas where ground is soft.

- Trees are located along the toe of the perimeter dike slopes along the southwest and southeast portions of the pond. The trees are beginning to infringe upon the toe in these areas. In addition, briars are beginning to re-establish along the toe and outer slopes of the pond in these areas.
- Previous inspection reports appear adequate, but there is a trend of not all maintenance recommendations being executed.

8. Recommendations

8.1. Phase 2 Engineering and Programmatic Recommendations

- It is recommended that the perimeter dikes for Active Ash Disposal Areas 2 & 3 undergo further engineering study to evaluate slope stability and seepage. This slope stability program is currently underway at the Active Ash Disposal Areas 2 & 3.
- In addition to the slope stability evaluation being performed, it is recommended that a hydraulic and hydrologic study be performed to evaluate freeboard and pond outlet adequacy relative to process flow and stormwater. Currently, new spillways are being designed that should incorporate these analyses.
- It is recommended that the abandoned weir structures within Active Ash Disposal Areas 2 & 3 be evaluated and a plan prepared to properly close these structures.
- A plan is currently being prepared to lower the pool in Active Ash Disposal Areas 2 & 3 to allow for installation of a new spillway structure. Routine repairs and monitoring of the spillway systems should be continued until replacement is complete. Once the new spillway is in place, a plan should be prepared for proper closure of the old RCP stacked riser spillways.
- The seepage observed along the toe of the perimeter dike at the east side of Ash Disposal Area 2 & 3 should continue to be monitored. A seepage monitoring point should be installed similar to the collection system installed on the toe of the southeast dike.
- Because the active ash disposal pond is nearing capacity and there are significant concerns relative to the integrity of the structure, it is recommended that a new permitted disposal facility be identified and permitted as soon as possible.
- It is recommended that the existing Operations and Maintenance Manual be updated for this facility.
- It is recommended that a program to develop as-built drawings and construction records for future maintenance and construction activities be established.

8.2. Maintenance Recommendations

- Remove trees from noted locations at Ash Disposal Areas 2 & 3 and repair slopes as needed following tree removal. To minimize damage to the toe and slopes, rip rap should be placed along the slopes once tree removal is complete to protect against wave action.
- Cut and maintain heavy, tall phragmites growth on interior slopes of ponds to allow better observation.
- The plant should continue best management practice of repairing areas of erosion, animal burrows, depressions, etc. and covering and seeding exposed areas within the Active Ash Disposal Areas 2 & 3. The areas should continue to be monitored and repairs made as conditions warrant.
- Due to the history of heavy vegetation along the perimeter slopes of the Active Ash Disposal Area and the presence of some briars and heavier growth along the outslopes, the plant should consider mowing these areas more than twice a year.
- The seepage observed along the toe of the perimeter dike at the northeast side of Ash Disposal Area 2 should continue to be monitored. A seepage monitoring point consisting of a collection system and weir box or similar structure should be installed.
- Continue annual inspection program and execute recommendations.

APPENDIX A

Document 9

Stantec Hydrologic and Hydraulic Analysis Report

Memo

Stantec

To:	Scott Turnbow Chattanooga, TN	From:	Stephen Bickel, PE Louisville, KY
File:	175559008	Date:	September 28, 2010

Reference: **Hydrologic and Hydraulic Calculations Summary**
Spillway Replacement Project
Johnsonville Fossil Plant (JOF)
Ash Disposal Area No. 2

The purpose of this memorandum is to summarize the hydrologic and hydraulic calculations supporting design of the spillway replacement structures at the JOF Ash Disposal Area No. 2 (Main Ash Pond). Detailed design calculations and descriptions will be provided with the final spillway design report and calculation package. Construction of the spillway replacements structures was significantly complete in November 2009.

BACKGROUND

A hydrologic and hydraulic analysis was conducted for the Johnsonville Fossil Plant (JOF) Ash Pond Disposal Area No. 2 in support of the spillway replacement project. This pond complex consists of Ponds A, B, and C as well as the active ash stacking area. These ponds serve as settling basins for the ash slurry that is discharged from the plant as well as stormwater detention for runoff from the active ash stacking area.

WATERSHED & PROCESS FLOW

The area draining to the ash pond complex includes direct runoff from approximately 87 acres within the pond complex. The daily plant process flow from the slurry lines averages roughly 32 million gallons per day (MGD).

OUTLET DESCRIPTION

Flow discharges from the ash pond through six (6) stop-log inlet structures connected to 30-inch nominal diameter HDPE outlet pipes through the embankment. These structures were installed in 2009 to replace the previous tall, unsupported riser structures and provide additional freeboard. Abandonment of the former structures is currently in the construction phase. Aside from the primary spillways and siphon drawdown spillways, there are no defined emergency spillways or overflow paths. Discharge from the outlets flows directly into the Tennessee River.

Reference: Hydrologic and Hydraulic Calculations Summary
Spillway Replacement Project
Johnsonville Fossil Plant (JOF)
Ash Disposal Area No. 2

FREEBOARD

TVA's Master Programmatic Document requires 5 ft of operating freeboard for ash pond facilities. The perimeter dike crest elevation is 390 ft. The ash pond water surface elevation is currently maintained at 384.5 ft, resulting in an operating freeboard of 5.5 ft. This facility currently MEETS freeboard requirements. The water surface elevation prior to the 2009 spillway replacement project was 387.5 ft.

METHODOLOGY SUMMARY

The 6-hour PMP rainfall depth for Humphreys County, Tennessee was determined to be 35 inches 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 SCS curve number method was used to convert this rainfall into runoff. A composite curve number of 99 was assigned to the watershed based on the assumption that all runoff would flow directly to the pond. Stage-storage relationships for the main ash pond complex were developed using contour data and hydrographic survey data provided by TVA. This data was input into a Hydrologic Engineering Center Hydrologic Modeling System (HEC-HMS) 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

Results of storm routings are summarized in Table 1. Supporting documentation is provided as Attachment A and Attachment B to this memorandum.

Table 1 – 6-hr PMP Freeboard and Routing Summary

	Pre-design Conditions	Post-design Conditions
Drainage Area (ac)	87	87
Crest of Dam (ft)	390	390
Normal Pool Elevation (ft)	387.5	384.5
Normal Operating Freeboard (ft)	2.5	5.5
Normal Operation Flow (MGD)	32	32
Design Storm	6-hour PMP	6-hour PMP
Design Storm max. water surface elevation (ft)	Overtopping	388.7

**Reference: Hydrologic and Hydraulic Calculations Summary
Spillway Replacement Project
Johnsonville Fossil Plant (JOF)
Ash Disposal Area No. 2**

FUTURE MODIFICATIONS

Abandonment of former primary spillway structures (48-inch concrete risers with 36-inch concrete pipe outlets) is in construction phase.

REFERENCE DRAWINGS

Spillway replacement record drawings: Work Plan 3 – JOF-090515-WP-3 (10W502)
Spillway abandonment construction: Work Plan 4 – JOF-100407-WP-4 (10W505)

STANTEC CONSULTING SERVICES INC.

Stephen Bickel, PE
Senior Principal
Stephen.Bickel@stantec.com

Attachment A: Watershed Map

Attachment B: Hydrologic and Hydraulic Model Input / Output


Drainage Area = 86.6 Acres
Kentucky Lake
Pond A
Pond B
Pond C
New spillways
Former spillways
N
TVA Johnsonville Fossil Plant
Spillway Replacement
Attachment A - Active Ash Pond Drainage Area
Stantec
500 250 0 500 Feet

Permanent Spillway PMP Check

The Tennessee 1973 Safe Dam Acts (TCA, Section 69-11-101 through 125), last amended March 1996, requires that intermediate sized dams with a hazard potential classification of Category 1, must be able to pass a probable maximum precipitation (PMP) storm event without overtopping. To verify this, a Hydrologic Engineering Center – Hydrologic Modeling System (HEC-HMS) model was created.

This section of the calculation package provides all assumptions, references, and input data required to create the HEC-HMS model. Also attached are the output tables with results.

As can be seen in the output table, the maximum water surface elevation reached during a 6-hour PMP event is 388.7'. The lowest elevation of the top of the perimeter dike around the active ash pond is elevation 390.0'. Therefore, the new spillway system will pass a 6-hour PMP storm without overtopping the dike.

Assumptions, References, and Design Input

- Drainage Area - The contributing drainage area to the new spillway was determined by using the geographic information systems (GIS) program ArcMap and aerial imagery provided by TVA. It was assumed that the entire area within the access road around the top of the dike would be contributing to the spillway.

- Curve Number / Losses - It was assumed that the curve number for the active ash pond is 99 and that the entire area is impervious thus there are no losses. This will produce a conservative peak water surface.

- Baseflow - The baseflow was provided by TVA and is 32 MGD.

- Initial Water Surface Elevation - It was assumed that the initial water surface elevation will be the new steady state elevation (384.6') that will be obtained once the new spillway is complete and operational. This elevation was determined in the Permanent Spillway Sizing calculations.

- PMP Storm - The 6-hour PMP rainfall for Humphreys County, Tennessee is approximately 35 inches[1]. The rainfall distribution was based on a typical SCS 6-hour storm distribution.

- Active Ash Pond Storage - The active ash pond storage volumes listed in the tables *Storage-Discharge Function: Spillway 5* and *Elevation-Storage Function: Stilling Ponds 2* were compiled from a pond survey provided by TVA conducted on November 17th, 2008. The pond survey data sheets are attached for reference.

- Spillway Discharge Curve - The spillway discharge values listed in the table *Storage-Discharge Function: Spillway 5* are based on a rating curve for the

[1] Zurndorfer, E.A., Schwarz, F.K., and Hansen, E.M. "Probable Maximum and TVA Precipitation Estimates With Areal Distribution for Tennessee River Drainages Less than 3000 Mi^2 in Area" Hydrometeorological Report No. 56, Office of Hydrology, National Weather Service, October 1986.

spillway structures that was created using HEC-RAS as part of the Permanent Spillway Hydraulic Control calculations.

HEC-HMS Input Files

Basin Models

Basin: Active Ash Disposal Area
Description: Active ash pond at Johnsonville.
Last Modified Date: 28 October 2009
Last Modified Time: 13:04:41
Version: 3.2
Unit System: English
Missing Flow To Zero: No
Enable Flow Ratio: No
Allow Blending: No
Compute Local Flow At Junctions: No

Sediment Grade Scale: NONE
Enable Sediment Routing: No
Fall Velocity Method: UNSPECIFIED
End:

Subbasin: Active Ash Disposal Area
Canvas X: 901.0600706713776
Canvas Y: 1890.459363957597
Area: 0.1353
Downstream: Stilling Ponds

Canopy: None

Surface: None

LossRate: SCS
Percent Impervious Area: 100
Curve Number: 99
Initial Abstraction: 0

Transform: SCS
Lag: 10

Baseflow: Monthly Constant
Monthly rate: 49.5
Monthly rate: 49.5
Monthly rate: 49.5
Monthly rate: 49.5
Monthly rate: 49.5
Monthly rate: 49.5
Monthly rate: 49.5
Monthly rate: 49.5
Monthly rate: 49.5

Monthly rate: 49.5
Monthly rate: 49.5
Monthly rate: 49.5

Erosion: None
End:

Reservoir: Stilling Ponds
Description: Stilling Ponds A, B, and C
Canvas X: -141.34275618374522
Canvas Y: 689.0459363957598

Route: Modified Puls
Routing Curve: Storage-Elevation-Outflow
Initial Elevation: 384.6
Storage-Outflow Table: Spillway 5
Elevation-Storage Table: Stilling Ponds 2
Primary Table: Elevation-Storage
End:

Basin Schematic Properties:
Last View N: 5000.0
Last View S: -5000.0
Last View W: -5000.0
Last View E: 5000.0
Maximum View N: 5000.0
Maximum View S: -5000.0
Maximum View W: -5000.0
Maximum View E: 5000.0
Extent Method: Elements
Buffer: 0
Draw Icons: Yes
Draw Icon Labels: Yes
Draw Gridlines: No
Draw Flow Direction: No
Fix Element Locations: No
End:

Meteorologic Models

Meteorology: PMP
Description: 6-Hour PMP
Last Modified Date: 28 October 2009
Last Modified Time: 13:04:41
Version: 3.2
Unit System: English
Precipitation Method: Specified Average
Snowmelt Method: None
Use Basin Model: Active Ash Disposal Area
End:

Precip Method Parameters: Specified Average
Allow Depth Override: No
Set Missing Data to Zero: Yes
End:

Subbasin: Active Ash Disposal Area
Gage: 6-Hour PMP
End:

Control Specifications

Control: 24-Hour Run-time
Description: 6-Hour Event
Last Modified Date: 28 October 2009
Last Modified Time: 13:08:22
Start Date: 1 January 2000
Start Time: 00:00
End Date: 2 January 2000
End Time: 00:00
Time Interval: 15
End:

Time-Series Data

Precipitation Gages: 6-Hour PMP

Time (ddMMMYYYY, HH:mm)	Precipitation (IN)
01Jan2000, 00:00	0.00
01Jan2000, 06:00	0.00
01Jan2000, 06:15	0.50
01Jan2000, 06:30	1.12
01Jan2000, 06:45	1.85
01Jan2000, 07:00	2.70
01Jan2000, 07:15	3.72
01Jan2000, 07:30	4.90
01Jan2000, 07:45	6.21
01Jan2000, 08:00	9.05
01Jan2000, 08:15	14.61
01Jan2000, 08:30	20.68
01Jan2000, 08:45	23.02
01Jan2000, 09:00	24.50
01Jan2000, 09:15	25.98
01Jan2000, 09:30	27.20
01Jan2000, 09:45	28.37
01Jan2000, 10:00	29.43
01Jan2000, 10:15	30.28
01Jan2000, 10:30	31.15
01Jan2000, 10:45	31.73
01Jan2000, 11:00	32.41
01Jan2000, 11:15	33.18
01Jan2000, 11:30	33.86
01Jan2000, 11:45	34.46
01Jan2000, 12:00	35.00

Paired Data

Storage-Discharge Functions: Spillway 5

Storage (AC-FT)	Discharge (CFS)
0.0	0.0
6.9	13.6
13.8	40.4
20.6	74.1
27.5	113.1
34.4	158.0
41.3	198.0
48.1	207.9
55.0	208.1
61.9	215.0
68.8	221.3
75.6	226.2
82.5	230.1
89.4	233.6
96.3	237.1
103.1	241.2
110.0	245.4
116.9	249.4
123.8	253.3
130.6	257.0
137.5	260.6
144.4	264.1
151.3	267.4
158.1	270.6
165.0	273.7
232.0	299.7

Elevation-Storage Functions: Stilling Ponds 2

Elevation (FT)	Storage (AC-FT)
384	0.0
385	34.4
386	70.5
387	109.1
388	150.1
389	193.8
390	231.6

HEC-HMS Output Files

Active Ash Disposal Area

Project: Johnsonville-Spillway

Simulation Run: PMP: 6B 7'W

Subbasin: Active Disaposal Area

Start of Run 01Jan2000, 00:00

End of Run: 02Jan2000, 00:00

Basin Model: Active Ash Disposal Area

Meteorologic Model: PMP

Control Specifications: 24-Hour Run-time

Date	Time	Precip (IN)	Loss (IN)	Excess (IN)	Direct Flow (CFS)	Baseflow (CFS)	Total Flow (CFS)
1-Jan-00	0:00				0	49.5	49.5
1-Jan-00	0:15	0	0	0	0	49.5	49.5
1-Jan-00	0:30	0	0	0	0	49.5	49.5
1-Jan-00	0:45	0	0	0	0	49.5	49.5
1-Jan-00	1:00	0	0	0	0	49.5	49.5
1-Jan-00	1:15	0	0	0	0	49.5	49.5
1-Jan-00	1:30	0	0	0	0	49.5	49.5
1-Jan-00	1:45	0	0	0	0	49.5	49.5
1-Jan-00	2:00	0	0	0	0	49.5	49.5
1-Jan-00	2:15	0	0	0	0	49.5	49.5
1-Jan-00	2:30	0	0	0	0	49.5	49.5
1-Jan-00	2:45	0	0	0	0	49.5	49.5
1-Jan-00	3:00	0	0	0	0	49.5	49.5
1-Jan-00	3:15	0	0	0	0	49.5	49.5
1-Jan-00	3:30	0	0	0	0	49.5	49.5
1-Jan-00	3:45	0	0	0	0	49.5	49.5
1-Jan-00	4:00	0	0	0	0	49.5	49.5
1-Jan-00	4:15	0	0	0	0	49.5	49.5
1-Jan-00	4:30	0	0	0	0	49.5	49.5
1-Jan-00	4:45	0	0	0	0	49.5	49.5
1-Jan-00	5:00	0	0	0	0	49.5	49.5
1-Jan-00	5:15	0	0	0	0	49.5	49.5
1-Jan-00	5:30	0	0	0	0	49.5	49.5
1-Jan-00	5:45	0	0	0	0	49.5	49.5
1-Jan-00	6:00	0	0	0	0	49.5	49.5
1-Jan-00	6:15	0.5	0	0.5	107.4	49.5	156.9
1-Jan-00	6:30	0.62	0	0.62	183.7	49.5	233.2
1-Jan-00	6:45	0.74	0	0.74	233.5	49.5	283
1-Jan-00	7:00	0.85	0	0.85	276.2	49.5	325.7
1-Jan-00	7:15	1.01	0	1.01	326.7	49.5	376.2
1-Jan-00	7:30	1.18	0	1.18	384	49.5	433.5
1-Jan-00	7:45	1.31	0	1.31	434	49.5	483.5
1-Jan-00	8:00	2.83	0	2.83	780.4	49.5	829.9
1-Jan-00	8:15	5.57	0	5.57	1526.9	49.5	1576.4
1-Jan-00	8:30	6.07	0	6.07	1949.6	49.5	1999.1
1-Jan-00	8:45	2.34	0	2.34	1275	49.5	1324.5
1-Jan-00	9:00	1.48	0	1.48	746.5	49.5	796
1-Jan-00	9:15	1.48	0	1.48	573.7	49.5	623.2
1-Jan-00	9:30	1.22	0	1.22	473.2	49.5	522.7
1-Jan-00	9:45	1.18	0	1.18	426.3	49.5	475.8

Date	Time	Precip (IN)	Loss (IN)	Excess (IN)	Direct Flow (CFS)	Baseflow (CFS)	Total Flow (CFS)
1-Jan-00	10:00	1.05	0	1.05	387.9	49.5	437.4
1-Jan-00	10:15	0.85	0	0.85	329.6	49.5	379.1
1-Jan-00	10:30	0.87	0	0.87	309.5	49.5	359
1-Jan-00	10:45	0.58	0	0.58	242.9	49.5	292.4
1-Jan-00	11:00	0.68	0	0.68	234.3	49.5	283.8
1-Jan-00	11:15	0.76	0	0.76	254.7	49.5	304.2
1-Jan-00	11:30	0.68	0	0.68	246.1	49.5	295.6
1-Jan-00	11:45	0.6	0	0.6	222.6	49.5	272.1
1-Jan-00	12:00	0.54	0	0.54	200.2	49.5	249.7
1-Jan-00	12:15	0	0	0	75.1	49.5	124.6
1-Jan-00	12:30	0	0	0	18.5	49.5	68
1-Jan-00	12:45	0	0	0	4.4	49.5	53.9
1-Jan-00	13:00	0	0	0	0.9	49.5	50.4
1-Jan-00	13:15	0	0	0	0	49.5	49.5
1-Jan-00	13:30	0	0	0	0	49.5	49.5
1-Jan-00	13:45	0	0	0	0	49.5	49.5
1-Jan-00	14:00	0	0	0	0	49.5	49.5
1-Jan-00	14:15	0	0	0	0	49.5	49.5
1-Jan-00	14:30	0	0	0	0	49.5	49.5
1-Jan-00	14:45	0	0	0	0	49.5	49.5
1-Jan-00	15:00	0	0	0	0	49.5	49.5
1-Jan-00	15:15	0	0	0	0	49.5	49.5
1-Jan-00	15:30	0	0	0	0	49.5	49.5
1-Jan-00	15:45	0	0	0	0	49.5	49.5
1-Jan-00	16:00	0	0	0	0	49.5	49.5
1-Jan-00	16:15	0	0	0	0	49.5	49.5
1-Jan-00	16:30	0	0	0	0	49.5	49.5
1-Jan-00	16:45	0	0	0	0	49.5	49.5
1-Jan-00	17:00	0	0	0	0	49.5	49.5
1-Jan-00	17:15	0	0	0	0	49.5	49.5
1-Jan-00	17:30	0	0	0	0	49.5	49.5
1-Jan-00	17:45	0	0	0	0	49.5	49.5
1-Jan-00	18:00	0	0	0	0	49.5	49.5
1-Jan-00	18:15	0	0	0	0	49.5	49.5
1-Jan-00	18:30	0	0	0	0	49.5	49.5
1-Jan-00	18:45	0	0	0	0	49.5	49.5
1-Jan-00	19:00	0	0	0	0	49.5	49.5
1-Jan-00	19:15	0	0	0	0	49.5	49.5
1-Jan-00	19:30	0	0	0	0	49.5	49.5
1-Jan-00	19:45	0	0	0	0	49.5	49.5
1-Jan-00	20:00	0	0	0	0	49.5	49.5
1-Jan-00	20:15	0	0	0	0	49.5	49.5
1-Jan-00	20:30	0	0	0	0	49.5	49.5
1-Jan-00	20:45	0	0	0	0	49.5	49.5
1-Jan-00	21:00	0	0	0	0	49.5	49.5
1-Jan-00	21:15	0	0	0	0	49.5	49.5
1-Jan-00	21:30	0	0	0	0	49.5	49.5
1-Jan-00	21:45	0	0	0	0	49.5	49.5
1-Jan-00	22:00	0	0	0	0	49.5	49.5
1-Jan-00	22:15	0	0	0	0	49.5	49.5
1-Jan-00	22:30	0	0	0	0	49.5	49.5
1-Jan-00	22:45	0	0	0	0	49.5	49.5
1-Jan-00	23:00	0	0	0	0	49.5	49.5
1-Jan-00	23:15	0	0	0	0	49.5	49.5
1-Jan-00	23:30	0	0	0	0	49.5	49.5

Stilling Ponds

Project: Johnsonville-Spillway

Simulation Run: PMP: 6B 7'W *Subbasin: Active Disaposal Area*

Start of Run 01Jan2000, 00:00 *Basin Model: Active Ash Disposal Area*

End of Run: 02Jan2000, 00:00 *Meteorologic Model: PMP*

Control Specifications: 24-Hour Run-time

Date	Time	Inflow (CFS)	Storage (AC-FT)	Elevation (FT)	Outflow (CFS)
1-Jan-00	0:00	49.5	20.6	384.6	94.7
1-Jan-00	0:15	49.5	19.7	384.6	90.6
1-Jan-00	0:30	49.5	18.9	384.6	86.9
1-Jan-00	0:45	49.5	18.2	384.5	83.5
1-Jan-00	1:00	49.5	17.5	384.5	80.4
1-Jan-00	1:15	49.5	16.9	384.5	77.6
1-Jan-00	1:30	49.5	16.3	384.5	75.1
1-Jan-00	1:45	49.5	15.8	384.5	72.8
1-Jan-00	2:00	49.5	15.4	384.4	70.7
1-Jan-00	2:15	49.5	15	384.4	68.7
1-Jan-00	2:30	49.5	14.6	384.4	67
1-Jan-00	2:45	49.5	14.2	384.4	65.4
1-Jan-00	3:00	49.5	13.9	384.4	64
1-Jan-00	3:15	49.5	13.6	384.4	62.7
1-Jan-00	3:30	49.5	13.4	384.4	61.5
1-Jan-00	3:45	49.5	13.1	384.4	60.4
1-Jan-00	4:00	49.5	12.9	384.4	59.4
1-Jan-00	4:15	49.5	12.7	384.4	58.5
1-Jan-00	4:30	49.5	12.6	384.4	57.7
1-Jan-00	4:45	49.5	12.4	384.4	56.9
1-Jan-00	5:00	49.5	12.2	384.4	56.3
1-Jan-00	5:15	49.5	12.1	384.4	55.7
1-Jan-00	5:30	49.5	12	384.3	55.1
1-Jan-00	5:45	49.5	11.9	384.3	54.6
1-Jan-00	6:00	49.5	11.8	384.3	54.1
1-Jan-00	6:15	156.9	12.7	384.4	58.6
1-Jan-00	6:30	233.2	15.4	384.4	70.9
1-Jan-00	6:45	283	19.1	384.6	87.9
1-Jan-00	7:00	325.7	23.4	384.7	107.5
1-Jan-00	7:15	376.2	28.2	384.8	129.6
1-Jan-00	7:30	433.5	33.6	385	154.5
1-Jan-00	7:45	483.5	39.8	385.1	167.6
1-Jan-00	8:00	829.9	49.7	385.4	185.3
1-Jan-00	8:15	1576.4	70.4	386	222.3
1-Jan-00	8:30	1999.1	102.5	386.8	241
1-Jan-00	8:45	1324.5	131.7	387.6	257
1-Jan-00	9:00	796	148.2	388	265.9
1-Jan-00	9:15	623.2	157.3	388.2	269.9
1-Jan-00	9:30	522.7	163.6	388.3	272.4
1-Jan-00	9:45	475.8	168.2	388.4	274.3
1-Jan-00	10:00	437.4	172	388.5	275.9
1-Jan-00	10:15	379.1	174.7	388.6	277
1-Jan-00	10:30	359	176.6	388.6	277.8
1-Jan-00	10:45	292.4	177.6	388.6	278.2
1-Jan-00	11:00	283.8	177.8	388.6	278.3
1-Jan-00	11:15	304.2	178.1	388.6	278.4
1-Jan-00	11:30	295.6	178.5	388.7	278.6
1-Jan-00	11:45	272.1	178.7	388.7	278.6

Date	Time	Inflow (CFS)	Storage (AC FT)	Elevation (FT)	Outflow (CFS)
1-Jan-00	12:00	249.7	178.3	388.6	278.5
1-Jan-00	12:15	124.6	176.4	388.6	277.7
1-Jan-00	12:30	68	172.7	388.5	276.2
1-Jan-00	12:45	53.9	168.3	388.4	274.4
1-Jan-00	13:00	50.4	163.7	388.3	272.5
1-Jan-00	13:15	49.5	159.1	388.2	270.6
1-Jan-00	13:30	49.5	154.6	388.1	268.7
1-Jan-00	13:45	49.5	150	388	266.9
1-Jan-00	14:00	49.5	145.6	387.9	264.5
1-Jan-00	14:15	49.5	141.2	387.8	262.1
1-Jan-00	14:30	49.5	136.8	387.7	259.7
1-Jan-00	14:45	49.5	132.5	387.6	257.4
1-Jan-00	15:00	49.5	128.2	387.5	255.1
1-Jan-00	15:15	49.5	124	387.4	252.8
1-Jan-00	15:30	49.5	119.8	387.3	250.6
1-Jan-00	15:45	49.5	115.7	387.2	248.4
1-Jan-00	16:00	49.5	111.6	387.1	246.2
1-Jan-00	16:15	49.5	107.5	387	243.9
1-Jan-00	16:30	49.5	103.6	386.9	241.6
1-Jan-00	16:45	49.5	99.6	386.8	239.3
1-Jan-00	17:00	49.5	95.7	386.7	237.1
1-Jan-00	17:15	49.5	91.9	386.6	234.9
1-Jan-00	17:30	49.5	88	386.5	232.7
1-Jan-00	17:45	49.5	84.3	386.4	230.5
1-Jan-00	18:00	49.5	80.6	386.3	228.4
1-Jan-00	18:15	49.5	76.9	386.2	226.2
1-Jan-00	18:30	49.5	73.3	386.1	224.1
1-Jan-00	18:45	49.5	69.7	386	221.1
1-Jan-00	19:00	49.5	66.2	385.9	214.8
1-Jan-00	19:15	49.5	62.9	385.8	208.8
1-Jan-00	19:30	49.5	59.6	385.7	203.1
1-Jan-00	19:45	49.5	56.5	385.6	197.5
1-Jan-00	20:00	49.5	53.5	385.5	192.1
1-Jan-00	20:15	49.5	50.6	385.4	186.9
1-Jan-00	20:30	49.5	47.8	385.4	182
1-Jan-00	20:45	49.5	45.1	385.3	177.2
1-Jan-00	21:00	49.5	42.5	385.2	172.5
1-Jan-00	21:15	49.5	40	385.2	168.1
1-Jan-00	21:30	49.5	37.6	385.1	163.8
1-Jan-00	21:45	49.5	35.3	385	159.6
1-Jan-00	22:00	49.5	33.1	385	152.2
1-Jan-00	22:15	49.5	31.1	384.9	142.9
1-Jan-00	22:30	49.5	29.3	384.9	134.4
1-Jan-00	22:45	49.5	27.6	384.8	126.7
1-Jan-00	23:00	49.5	26.1	384.8	119.7
1-Jan-00	23:15	49.5	24.7	384.7	113.4
1-Jan-00	23:30	49.5	23.4	384.7	107.6
1-Jan-00	23:45	49.5	22.3	384.6	102.3
2-Jan-00	0:00	49.5	21.2	384.6	97.5

APPENDIX A

Document 10

Pseudo-Static Slope Stability Analysis Summary & Seismic Risk Assessment White Paper

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_002_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
BRF, COF, GAF, JSF, JOF, KIF, PAF, and WCF

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 Bull Run (BFR), Colbert (COF), Gallatin (GAF), John Sevier (JSF), Johnsonville (JOF), Kingston (KIF), Paradise (PAF), and Widows Creek (WCF) 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 simplicity and conservatism, the selected sections represent the facility's lowest current static (long-term) factor of safety. The SLOPE/W models were updated to reflect any significant mitigations or operational changes that have occurred since completion of Stantec's geotechnical studies.
- Undrained shear strength parameters were used.
- Ground motion levels 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 plant-

specific tables (Tables 13 through 23) 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, PE
Principal

Enclosures

/cdm

Pseudostatic Stability Analysis Summary - TVA Active CCP Disposal Facilities

BRF, COF, GAF, JSF, JOF, KIF, PAF, WCF

Plant	CCP Disposal Facility		Cross-Section	2,500 yr Return	
	Name	Type		PGA (g)	Factor of Safety
BRF	Gypsum Disposal Area 2A	Wet Stack	I	0.131	1.0
	Fly Ash Disposal Area 2	Impoundment	S		1.4
	Bottom Ash Disposal Area 1	Stack	D		1.1
COF	Disposal Area 5 Stack	Stack	I	0.138	1.0
	Disposal Area 5 Stilling Basin	Impoundment	J		1.2
	Ash Pond 4	Impoundment	D		1.0
GAF	Ash Pond A	Impoundment	K	0.108	1.0
	Ash Pond E	Impoundment	B		1.3
JSF	Bottom Ash Pond	Impoundment	I	0.115	2.2
JOF	Ash Disposal Area 2	Impoundment	K	0.254	1.0
KIF	Stilling Pond	Impoundment	132+37	0.115	1.0
PAF	Slag Ponds 2A and 2B	Impoundment	Typical	0.157	1.1
	Scrubber Sludge Complex	Impoundment	G		1.0
	Peabody Ash Pond	Impoundment	A		1.0
WCF	Gypsum Stack	Wet Stack	F	0.1	1.5
	Dredge Cell (Old Scrubber Sludge Pond)	Impoundment	D		1.1
	Main Ash Pond	Impoundment	J		1.4

Johnsonville Fossil Plant (JOF)

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

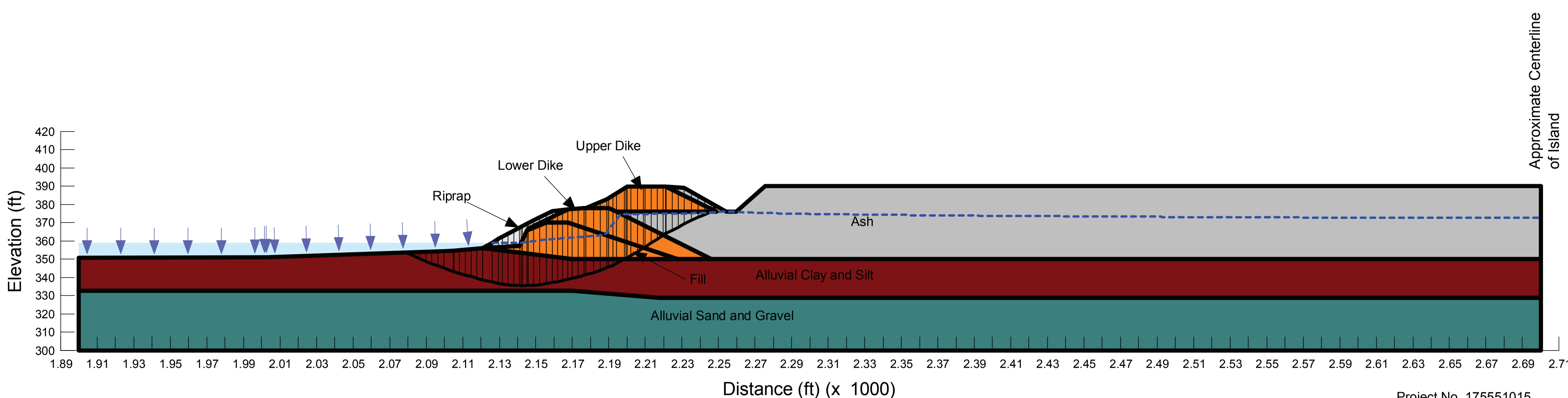
Section K - Ash Disposal Area No. 2
Johnsonville Fossil Plant
New Johnsonville, Tennessee

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 (pcf)	Cohesion (psf)	Friction Angle
Upper Dike	125	521	16.2 °
Lower Dike	125	533	20.1 °
Ash	100	0	10 °
Fill	124	630	17.8 °
Alluvial Clay and Silt	124	714	17.8 °
Alluvial Sand and Gravel	120	0	30 °
Riprap	100	0	38 °

Factor of Safety: 1.0

Horizontal Seismic Coefficient Kh = 0.254 g
2500-year Return Period Event



Date of Assessment - 11/22/11

Project No. 175551015

BORING LOCATION TABLE			
BORING	NORTHING	EASTING	ELEV. (FT.)
STN-AC	603,148.82	1,410,894.84	391.4
STN-AT	603,144.12	1,410,950.20	368.4
STN-BC	602,313.93	1,410,981.18	391.5
STN-BT	602,326.17	1,411,067.30	369.8
STN-C1C	601,113.79	1,411,129.20	391.5
STN-C1CA	601,054.92	1,410,641.77	394.0
STN-C1CB	601,079.42	1,410,415.09	398.4
STN-C1T	601,033.28	1,411,220.15	365.5
STN-CC	601,437.52	1,411,070.75	391.6
STN-CCA	601,382.49	1,410,633.59	394.6
STN-CT	601,449.55	1,411,148.76	358.9
STN-DC	600,191.17	1,410,724.31	390.0
STN-DT	600,147.64	1,410,847.53	365.3
STN-EC	599,528.35	1,410,416.15	390.2
STN-ECA	599,517.65	1,410,426.37	390.2
STN-ET	599,486.09	1,410,496.27	363.8
STN-FC	598,898.88	1,410,069.79	389.4
STN-FT	598,868.94	1,410,145.49	362.9
STN-GC	598,719.43	1,409,736.38	389.6
STN-GT	598,592.54	1,409,772.40	360.8
STN-HC	599,345.93	1,409,646.07	389.5
STN-HM	599,331.00	1,409,595.58	377.9
STN-HT	599,308.41	1,409,545.23	363.1
STN-IC	600,055.90	1,409,637.66	389.8
STN-IT	600,103.14	1,409,560.28	368.8
STN-JC	600,817.61	1,409,871.68	389.6
STN-JT	600,838.26	1,409,820.33	376.7
STN-KC	601,482.90	1,410,105.77	389.8
STN-KT	601,488.28	1,410,056.92	377.6
STN-LC	602,377.53	1,410,442.03	389.9
STN-LT	602,392.94	1,410,352.26	366.3
STN-MC	603,157.11	1,410,726.95	390.6
STN-MT	603,187.15	1,410,653.44	365.6
*STN-B-1			390.6
*STN-B-2			390.2
*STN-B-3			390.2
*STN-B-4			389.5
*STN-B-5			389.9
*STN-B-6			389.9
*STN-B-7			390.1
*STN-B-8			389.9
*STN-B-8A			389.9
*STN-B-9			389.7
*STN-B-10			389.1
*STN-B-11			389.6
*STN-B-12	Not Drilled		
*STN-B-13			390.1
*STN-B-14			367.3
*STN-B-15			378.9
*STN-B-16			389.6
*STN-B-17			389.1
*STN-B-18			391.0
*STN-B-19			388.3
*STN-B-20			388.9
*STN-B-21			389.2

*Borings not surveyed and elevations are approximate

INSPECTION TEST PIT LOCATION TABLE			
BORING	NORTHING	EASTING	ELEV. (FT.)
TP-1	601,957.34	1,410,239.85	–
TP-2	602,389.71	1,410,395.38	–
TP-3	602,721.20	1,410,510.10	–
TP-4	603,003.70	1,410,606.32	–

*Test Pit Locations are approximate

BORINGS / **REFERENCE**

BORINGS	REFERENCE
B-1 through B-9 and B-OW-1 through B-OW-6	Report of Geotechnical Exploration, Ash Pond Dike, New Johnsonville Plant, New Johnsonville, Tennessee, Law Engineering and Environmental Services, Inc., January 17, 1994.
94-5 through 94-14	Subsurface Exploration Data, TVA Borings at Johnsonville Fossil Plant, Johnsonville, Tennessee, Law Engineering and Environmental Services, Inc., October 11, 1994.
US-1B through US-6B	Johnsonville Steam Plant – Ash Pond – Soil and Foundation Exploration, J.C. McCraw, TVA Construction Services Branch, September 17, 1969.
SS-13, SS-15, SS-16	Johnsonville Groundwater Assessment, TVA Resource Group, Engineering Services, March 1995.
SS-1 through SS-11 and US-1, US-7 and US-8	Johnsonville Steam Plant – Ash Disposal Area No. 2 Dike Raising – Soil Exploration and Testing, G. Farmer, TVA Construction Services Branch, November 22, 1977.
C2-1 through C2-20	Report of Ash Pond Investigation, Johnsonville Fossil Plant, New Johnsonville, Tennessee, MACTEC Engineering and Consulting, December 4, 2003.

LEGEND

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

NOTES:

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

100 0 200 400 FEET
GRAPHIC SCALE

For Supporting Design Calculations see

SCALE: 1"=200' EXCEPT AS NOTED

YARD
ASH DISPOSAL AREAS 2 AND 3

GEOTECHNICAL EXPLORATION
BORING LAYOUT

Stantec Consulting Services Inc.

JOHNSONVILLE FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

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

PLOT FACTOR:XX
W_TVA

C.A.D. DRAWING
DO NOT ALTER MANUALLY

STANTEC | 0
TASK COMPLETED BY | REV NO.

30 C XXWXXX-01

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

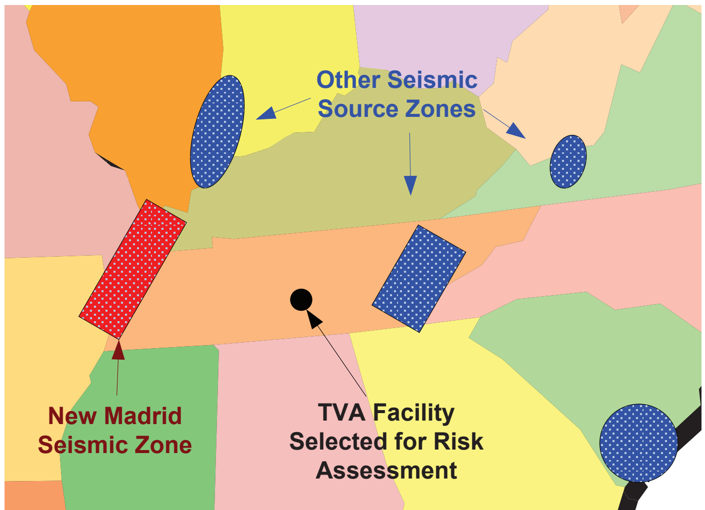
TVA Risk Event Probability Rating Scale		
Score	**Rating**	**Description**
5	Virtually Certain	95% probability that the event will occur in the next 3 years /10 years
4	Very Likely	75% probability that the event will occur in the next 3 years/10 years
3	Even Odds	50% probability that the event will occur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

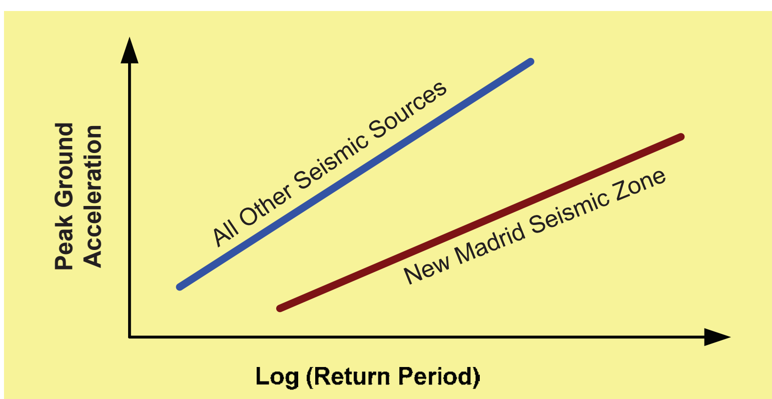


Figure 2. Typical Seismic Hazard Curves for Proposed Probabilistic Assessment of TVA Facilities

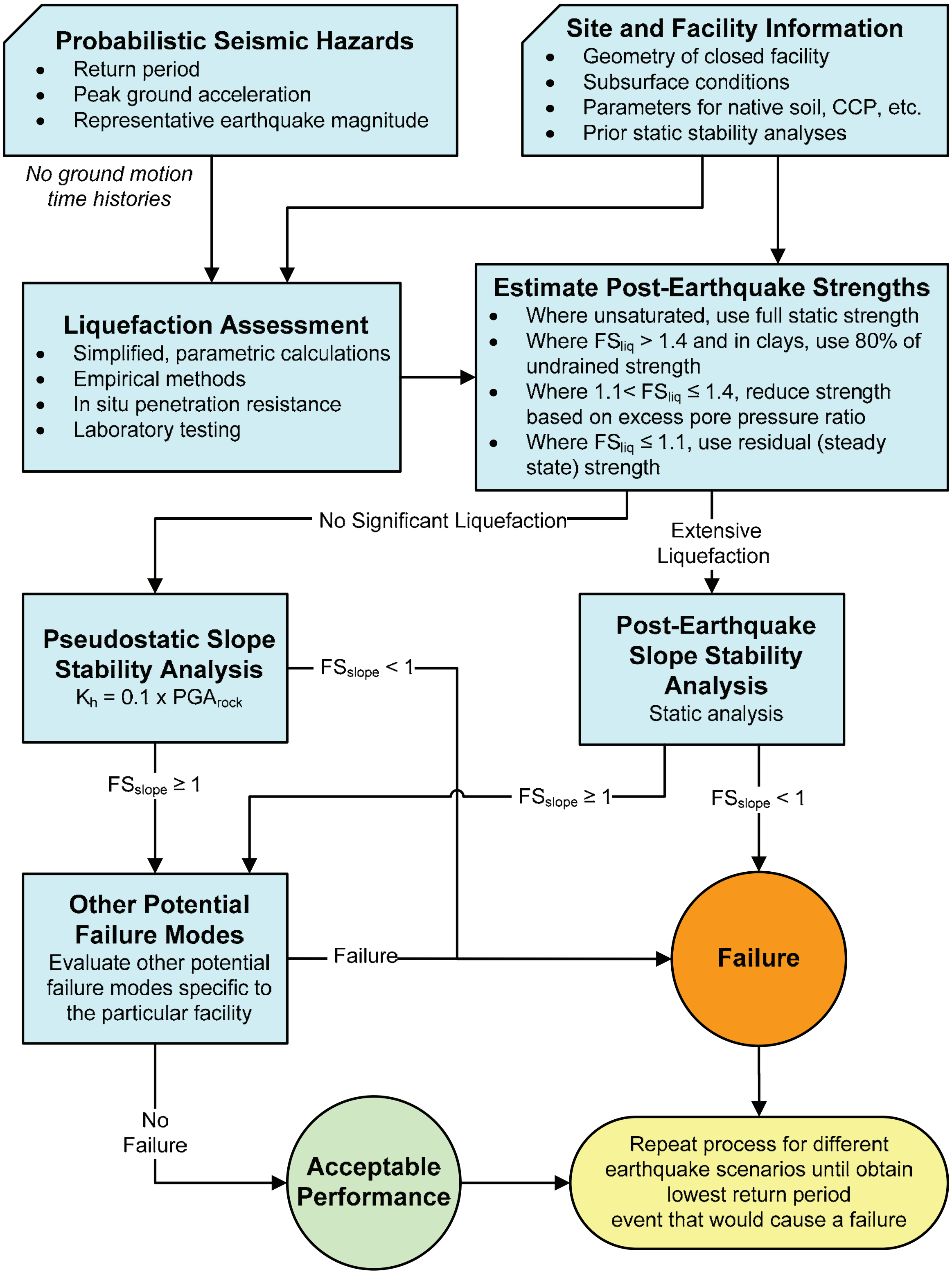


Figure 3. Simplified Flowchart for Assessing Facility Performance During a Probabilistic Seismic Event in Phase A

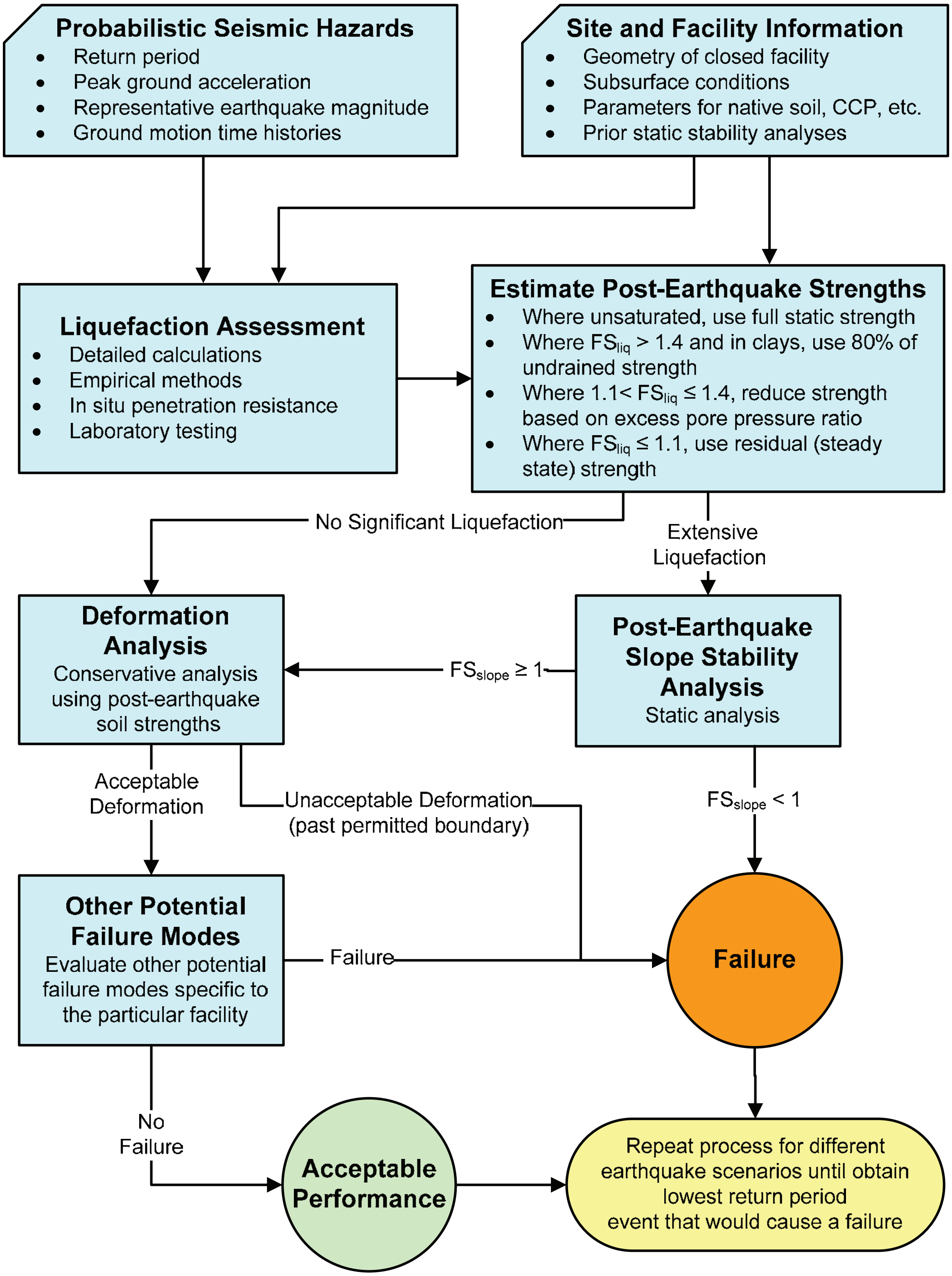


Figure 4. Simplified Flowchart for Assessing Facility Performance During a Probabilistic Seismic Event in Phase B

APPENDIX A

Document 11

Johnsonville Fossil Plant Procedures

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1.0 PURPOSE

The purpose of this document is to define the procedures for handling production ash at Johnsonville Fossil Plant (JOF). This document defines the roles and responsibilities of all parties, active permits, operational requirements, required documentation, and general procedures for the daily operations of the Ash Disposal Area No. 2 at JOF.

2.0 OPERATIONS

The operations at the Active Ash Disposal Area at JOF do not operate under a solid waste permit. The "island" facility is governed by the NPDES permit. Some of the operational requirements described in the following sections reference TDEC requirements for best-management practices.

The Active Ash Disposal Area has been referred to using various names or terms throughout its existence. These include: Ash Disposal Area No. 2, Island Ash Area, Ash Disposal Area West of Boat Harbor, Trans Ash Cells 1, 2, 3A and 3B, Ash Disposal Areas 2 and 3, Main Ash Ponds A and B, and Stilling Pond C. There is an inactive Chemical Treatment Pond on the east side of the Ash Disposal Area. This pond is scheduled for closure by 2012.

2.1 Ash Handling Operations

Approximately 350,000 tons of fly ash and bottom ash are wet-sluiced to the Active Ash Disposal Area each year. The process for handling ash is outlined below.

a. Ash is pumped to the sluicing channel and enters the channel on the east side of the disposal area.

b. The majority of ash is removed from the sluice channel using long reach hydraulic excavators.

c. The material removed from the sluice channel is stacked at a higher level where it drains and dewaters.

d. During the summer the accumulated ash is loaded into dump trucks and transported to a permitted landfill site.

e. Since a portion of the fly ash is not captured in the dipping process, it is necessary to periodically dredge the ponds and pump this material to an internal dredge cell for dewatering and hauling off site.

As recommended by TDEC the elevation of the ash stack shall not exceed 390 feet with exception to the winter ash stacking plan discussed below. Trans Ash is currently under contract for offsite disposal of the ash. The removal of ash shall continue until the quantities remaining in the Ash Disposal Area No. 2 can be stacked within the restrictions provided by TDEC.

As built surveys shall be performed to monitor the status of the ash removal compared to the predicted closure plan. Topographical surveys along with unit weight calculations shall be performed to estimate the amount of ash to be removed as well as the associated labor.

2.2 Winter Ash Stacking Plan

The winter ash stacking plan defines the location where material may be stacked during the winter months when ash is not transported to the permitted landfill site. This is a cyclical plan where ash is stockpiled during the winter then removed during the summer. The plan consists of a south winter stockpile and a north winter stockpile. The two sheet stacking plan is included under Section 700 – Construction Drawings. The main features of the plan are described below.

a. 190,000 cubic yards of storage (approximately 6 months of production).

b. 3H:1V side slopes.

c. Maximum stockpile elevation of 405 feet.

d. 20 foot bench at elevation 400 feet on the east and west sides of the north stockpile.

e. 9 boundary markers established to delineate the limits of the toe of the north stockpile.

f. 130 foot offset from existing sluice channel to provide area for ash to dewater prior to placement in the stockpiles.

g. 40 foot offset from the abandoned sluice channel on the east and west sides of the north stockpile.

2.3 Daily and Intermediate Cover

In reference to TDEC 1200-1-7-.04 (9) (c) 11, no daily or intermediate cover shall be required for the working areas of the facility. Ash is inert, physically stable, does not biodegrade, and does not attract animals, cover is not required. Intermediate cover should be placed on exterior side slopes excluding the winter stockpiles to reduce erosion.

2.4 Final Cover

In reference to TDEC 1200-1-7-.04 (9) (c) 11, final cover requirements will be determined during the closure design of the facility. Final cover will be constructed once disposal activities have been completed. Following the TVA Master Programmatic Documents, at a minimum, final cover should consist of a 24 inch compacted soil layer with a permeability equal to 1×10^{-7} cm/sec, and a vegetative layer of a minimum thickness of 12 inches. An alternate cover can be used if it can be demonstrated to provide equivalent or superior performance. Soils for the construction of the low permeability soil layer of the final cover system will be identified during a borrow soil evaluation for suitable cover material at nearby locations.

2.5 Operating Equipment

TVA or its designated contractor will utilize heavy equipment for the operation of the active ash disposal area. It is likely that the following pieces of equipment will be used:

- Long-reach track-hoes,
- Bulldozers,
- Compactors,
- Scrapers,
- Water Pumps,
- Water trucks, and
- Other conventional earthmoving equipment.

TVA or its designated contractor shall be able to provide additional equipment within 24 hours for construction or disposal operations in the event of a breakdown/emergency.

2.6 Storm Water Runoff

In reference to TDEC 1200-1-7-.04(2)(i), storm water runoff should be controlled in order to minimize erosion, minimize the conveyance of sediment laden storm water, and minimize the potential for water pollution. Best management practices for erosion control as noted in the Tennessee Erosion and Sediment Control Handbook should be followed to reduce erosion due to storm water runoff. These include silt fences, intermediate cover, temporary vegetation on slopes, rip rap protection, temporary sediment ponds, surface water ditches, etc.

2.7 Dust Control

In reference to TDEC 1200-1-7-.04(2)(j), dust shall be controlled by the operator as necessary to prevent dust from creating a nuisance or safety hazard to adjacent landowners or to persons engaged in supervising, operating, and using the site. Water trucks shall be used as necessary to maintain dust control.

2.8 Clay Dike Restrictions

The clay dike study report is due September 2, 2011. A summary of the results should be discussed here and included in Section 1113.

2.9 Dredging Guidelines

In order to maintain the required free water volume specified in the NPDES permit, it will be necessary to dredge ash from Pond A, Pond B, and Pond C periodically (every one to two years). Ash shall be removed using a hydraulic dredge, with a cutterhead located at the end of the dredge ladder to disturb the ash. The concentrated water-ash slurry is then removed via suction and pumped through a pipeline to a temporary dredge cell located on the north side of the ash disposal area. The following guidelines shall be followed for all dredging operations.

a. Dredging shall occur at least 100 feet from the centerline of the perimeter clay dikes as shown in Figure 1.

b. Buoys shall be established to mark the horizontal limits of dredging as to not affect the perimeter clay dikes.

c. The dredge ladder and cutterhead shall point towards the perimeter dikes during dredging operations, using the cable method to control forward progress of the dredge. In this method, a cable is attached to the stern of the dredge and a fixed point behind the dredge. The dredge then cuts along the arc set by the cable length, increasing the length of cable as needed to move forward.

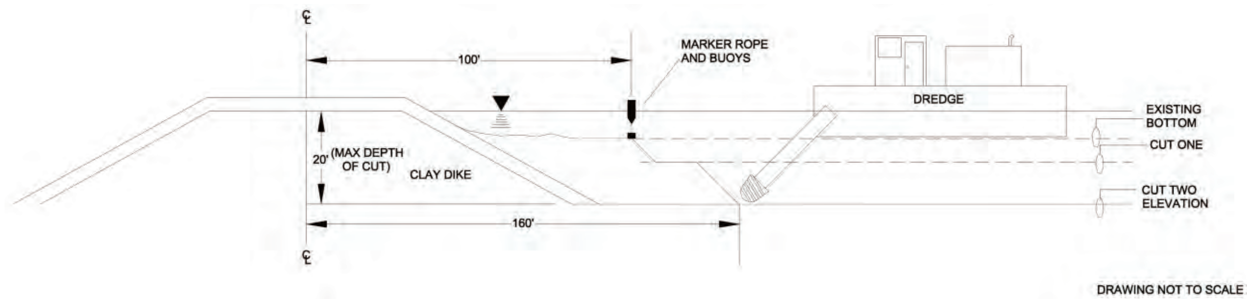


Figure 1. Dredging Requirements

3.0 GENERAL MAINTENANCE

3.1 Mowing and Vegetation Removal

The slopes shall be mowed to reduce the opportunity for tree growth and allow for visual inspection and observations. The slopes shall be mowed to a height of no less than 3-inches and no more than 12-inches tall with a minimum of three mowings per growing season. If woody growth is detected, it shall be removed.

3.2 Tree Removal

At the location of all trees which are greater than two (2) inches in base diameter, remove the tree and grub to the bottom of the root system at least twelve inches below grade. Backfill the excavations with a similar slope material and compact with a manual tamper. See the *General Guidelines for Tree Removal* in Section 1104.

3.3 Fertilize and Reseed Bare Areas

Prepare all regraded and exposed areas for seeding by disking the surface three (3) inches in depth. Apply fertilizer (600 lbs/acre), seed mixture (as directed by facility engineer), mulch (1.5 tons/acre), and netting (0.75 inch by 1.0 inch mesh openings) with pins to the prepared areas. Other application rates may be requested by the facility engineer. The seed mixture utilized depends on the seeding application period and location.

3.4 Erosion Rill and Gully Repair

The cause of erosion shall be identified before beginning repair. Causes of erosion include poor vegetative cover, breach of a hydraulic structure or ditch, long or steep slopes and concentrated flows. Gullies or rills shall be graded, re-seeded and covered with an erosion control blanket. If the problem is ongoing, then consider shaping the gully and forming a ditch lined with riprap. See the *General Guidelines for Rill and Gully Erosion Repair* in Section 1104.

3.5 Animal Burrow Repair

Animal burrows provide a potential location for seepage and piping to occur. In order to repair animal burrows, locate burrows, trap animals, and relocate or dispose of animals as directed. See the *General Guidelines for Animal Burrow Repair* in Section 1104.

3.6 Wave Wash Riprap Protection

Wave erosion shall 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. See the *General Guidelines for Wave Wash Erosion Repair & Rip-rap Protection* in Section 1104.

3.7 Rutting Repair

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 dike crest materials which have become weak over time because of moisture infiltration. Repeated passes of maintenance traffic/equipment over weakened materials can lead to rutting. The *General Guidelines for Rutting Repair* is provided in Section 1104. The attached 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.

3.8 Spillway and Siphon Systems

The spillway system located on the southwest side of the ash pond complex was installed in April, 2010. The new spillway configuration consists of six precast concrete inlet structures with stop logs and skimmers. Each inlet structure has a 30” diameter HDPE spillway outlet pipe. These pipes flow into a concrete basin with sills to diffuse the velocity of the flowing water. The slope is lined with rip-rap below the concrete basin to allow effluent water to flow into Kentucky Lake. At the location of the inlet structures, a single crane has been installed to aid in the maintenance of the inlet pipe structures. Also to the north of the spillway system is the siphon system that can be utilized when needed to assist the spillway system in lowering the water level of the ash pond complex. All parts of the spillway system need to be inspected and maintained to insure proper functionality of all parts of the system.

4.0 INSPECTIONS AND REPORTING

TVA conducts daily, weekly, quarterly, and annual inspections of the active ash disposal areas at JOF. Following these inspections, reports are completed and filed. Any deficiencies requiring corrective actions/maintenance are reported and tracked using Maximo. A seepage action plan has been developed to track seeps and determine the level of repair necessary. Signs have been placed at all known seeps and pipe penetrations to aid during inspections.

4.1 Daily Field Reports

The daily field reports are used to determine minor deficiencies in operations. These are compiled by the contractor into a weekly report. These two reports are described below.

a. **RHO&M Daily Field Report** - Contractor

The purpose of the **RHO&M Daily Field Report** is to list deficiencies found beyond routine maintenance issues such as seeps or boils, freeboard issues, sloughs, or spillway issues. Also, daily production and activities conducted shall be tracked. The RHO&M Daily Field Repot is included in Section 1105.

b. **RHO&M Weekly Field Report** - Contractor

The **RHO&M Weekly Field Report** summarizes the daily activities for the week based on the daily field report. The Weekly Field Report is included in Section 1106.

4.2 Weekly Inspections

The active ash disposal area shall be inspected weekly by the Field Supervisor. The inspection shall be recorded using the **Weekly Facility Observation Form** included in Section 1107. The dikes shall be inspected for cracks, rutting, settlement, erosion, sloughs, seepage, vegetation, animal burrows, sinkholes, and other deficiencies. Deficiencies noted in previous inspections shall be checked if repairs have not yet been implemented.

4.3 Monthly Inspections

The active ash disposal area shall be inspected monthly by the Construction Manager. The inspection shall be recorded using the **Monthly/Quarterly/Special Facility Inspection Form** included in Section 1007. The dikes shall be inspected for cracks, rutting, settlement, erosion, sloughs, seepage, vegetation, animal burrows, sinkholes, and other deficiencies. Deficiencies noted in previous inspections shall be checked if repairs have not yet been implemented.

4.4 Quarterly Inspections

Quarterly inspections shall be conducted once every three months. The inspection shall be recorded using the **Monthly/Quarterly/Special Facility Inspection Form** included as in Section 1007. The quarterly inspection shall be led by the RHOM Program Manager. The RHOM team including the construction manager and field supervisor shall walk the active ash disposal areas, looking for seeps, sloughs, animal burrows, and any other deficiency which could affect the integrity of the facility. All deficiencies shall be flagged, surveyed, and photographed. A report shall be compiled with all deficiencies, locations, photos, and recommendations for repairs. Areas requiring engineering recommendations shall be sent to CCP Engineering or a geotechnical engineer to provide recommendations for the repair.

4.5 Annual Inspection

Once a year, an annual inspection shall be performed under the ownership of CCP Engineering. This shall be performed by a qualified geotechnical engineer. The purpose of the annual inspection is to inspect both the active and inactive ash disposal areas for structural integrity and make recommendations for any deficiencies noted. Photos shall be taken to describe the existing conditions at the time of the inspection, as well as to show the deficiencies found.

4.6 Inspection Deficiencies

Each potential deficiency encountered as a result of an inspection should be recorded in accordance with the CPP RHO&M Work Control procedure (FGDC-SPP-07.007), Section 3.2 E. Deficiency Monitoring. Recorded deficiencies should be tracked in the Maximo system as “Other” work orders with a work type of “OTH.”

4.7 Seepage Monitoring

The Seepage Action Plan for Johnsonville dated June 25, 2010 shall be followed as planned to observe, document, and remediate potential seepage areas. The seepage action plan shall be routinely implemented and updated at Johnsonville. This requires stockpiles of aggregate, sandbags and culvert pipe and updates to the seepage log when evidence of seepage is observed. Signs shall be installed at any new seepage areas.

4.8 Spillway and Siphon Systems

The spillway system located on the southwest side of the ash pond complex was installed in April, 2010. The new spillway configuration consists of six precast concrete inlet structures with stop logs and skimmers. Each inlet structure has a 30” diameter HDPE spillway outlet pipe. These pipes flow into a concrete basin with sills to diffuse the velocity of the flowing water. The slope is lined with rip-rap below the concrete basin to allow effluent water to flow into Kentucky Lake. At the location of the inlet structures, a single crane has been installed to aid in the maintenance of the inlet pipe structures. Also to the north of the spillway system is the siphon system that can be utilized when needed to assist the spillway system in lowering the water level of the ash pond complex. All parts of the spillway system need to be inspected and maintained to insure proper functionality of all parts of the system.

5.0 PROJECT MANAGEMENT

The following forms are included to assist in project management requirements.

a. Project Startup Checklist

The purpose of the project startup checklist is to define the roles and responsibilities of the various groups within TVA and to insure that the required tasks are completed during the project planning stage. It also includes the required steps to be completed at project completion. The project startup checklist is included in Section 1109.

b. RHO&M Additional Work/ Change Order Form

The additional work/ change order form shall be used when the scope of work changes for the routine handling contractor. The form addresses the reason for the change, who initiated the change, who needs to be notified, and the financial impacts of the change. The additional work/ change order form is included in Section 1110.

c. Environmental Review (NEPA)

Procedures were developed to provide guidance for compliance with the National Environmental Policy Act (NEPA). These procedures for the environmental review of a project are included in Section 1111.

6.0 Work Control Process

The work control process was developed to provide guidance for implementing a work control process that maximizes safety, facility reliability, work productivity, and risk assessment and management. The procedures describe the process by which maintenance and modification work activities are identified, planned, scheduled, monitored, and completed. It describes the work order process using the Maximo system. The work control process is included in Section 1112.

7.0 Records

In accordance with the TVA Master Programmatic Documents, the Maximo database shall be used to track all inspection, monitoring, reporting, and maintenance recommendations. Final inspection reports and instrumentation data collection and analysis will be placed in the TVA BSL.

8.0 Subsections

Section 1104 – General Maintenance Guidelines

Section 1105 – RHO&M Daily Field Report

Section 1106 – RHO&M Weekly Field Report

Section 1107 – Weekly Facility Observation Form

Section 1108 – Monthly/Quarterly/Special Facility Inspection Form

Section 1109 – Project Startup Checklist

Section 1110 – RHO&M Additional Work/ Change Order Approval Form

Section 1111 – NEPA Process

Section 1112 – CCP RHO&M Work Control Procedures

Section 1113 – Clay Dike Restrictions

APPENDIX A

Document 12

Johnsonville Fossil Plant General Maintenance Guidelines

1.0 GENERAL MAINTENANCE GUIDELINES

1.1 GENERAL GUIDELINES FOR REPAIR OF ANIMAL BURROWS

1.1.1 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.

1.1.2 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.

1.2 GENERAL GUIDELINES FOR REPAIR OF RILL AND GULLY EROSION

1.2.1 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.

1.2.2 GUIDELINES FOR RILL AND GULLY EROSION REPAIR

<u>Shallow Rills and Gullies:</u>

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 –	Vegetation Specifications, Landfill Permit
Alabama Plants –	ALDOT Standard Specifications, Section 659

<u>Deep Rills and Gullies:</u>

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 –	Vegetation Specifications, Landfill Permit
Alabama Plants –	ALDOT Standard Specifications, Section 659

1.3 GENERAL GUIDELINES FOR REPAIR OF RUTTING

1.3.1 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.

<u>Guidelines for Rutting and Repair</u>

- 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

1.4 GENERAL GUIDELINES FOR TREE REMOVAL ON SLOPES

1.4.1 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.

1.4.2 GUIDELINES FOR TREE REMOVAL AND MAINTENANCE OF VEGETATION

<u>Tree Removal</u>

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.

<u>Maintenance of Vegetation</u>

- 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.

1.5 GENERAL GUIDELINES FOR REPAIR OF WAVE WASH EROSION REPAIR AND CONSTRUCTION OF RIPRAP PROTECTION

1.5.1 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.

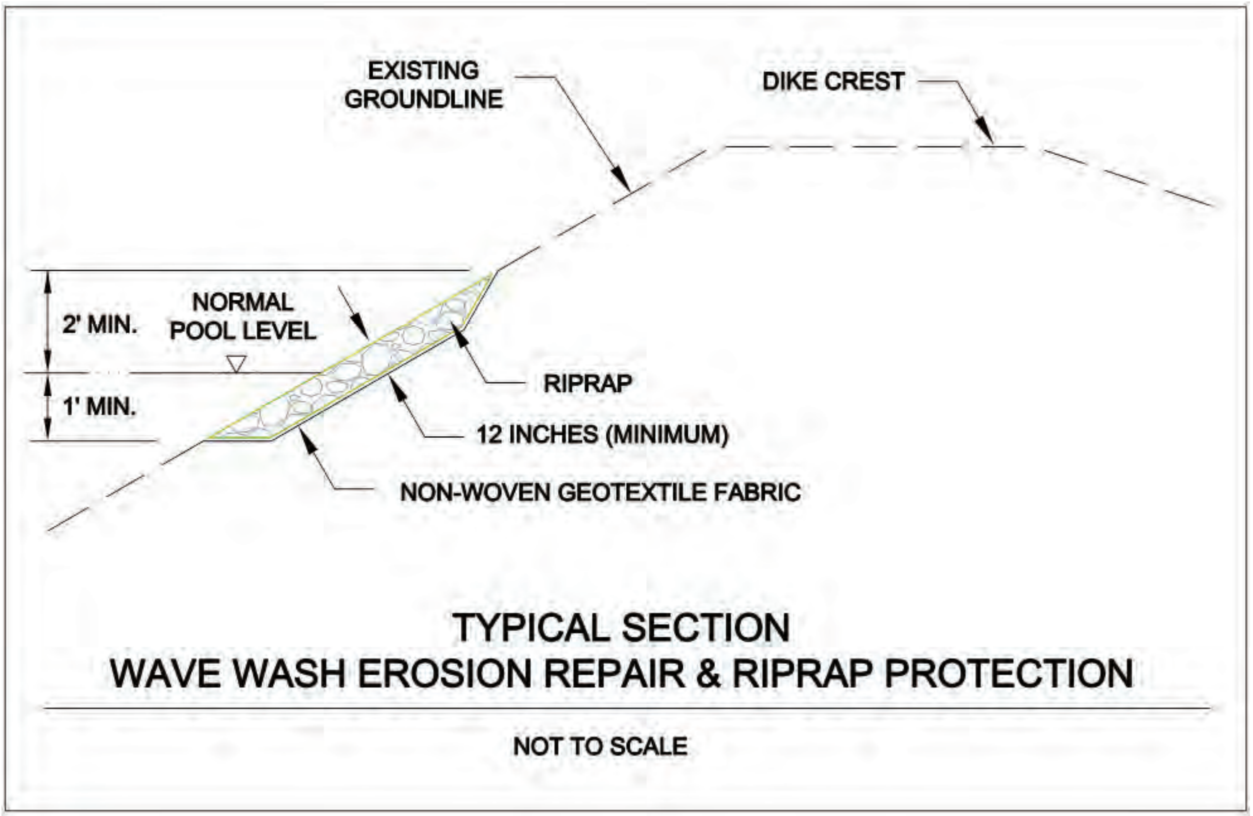
- Place non-woven geotextile fabric along the slope where vegetation and loose soil have been removed. Use fabric meeting or exceeding the following designations, depending on the state in which the work is being performed.

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.



1.6 GENERAL GUIDELINES FOR MOWING AND VEGETATION REMOVAL

Slopes shall be mowed to reduce the opportunity for tree growth and allow for visual inspection and observations. The slopes shall be mowed to a height of no less than 3 inches and no more than 12 inches tall with a minimum of three mowings per growing season. If woody growth is detected, it shall be removed.

1.7 GENERAL GUIDELINES FOR BARE AREA FERTILIZING AND RESEEDING

Prepare exposed or bare areas for seeding by discing the surface 3 inches in depth. Apply fertilizer (600 lbs./acre), seed mixture (as directed by facility engineer), mulch (1.5 tons/acre), and netting (0.75" x 1" mesh openings) with pins to the prepared areas. Other application rates may be requested by the facility engineer. The seed mixture utilized depends on the seeding application period and location.

APPENDIX A

Document 13

Instrumentation (Piezometer & Inclinometer) Readings

Stantec

Piezometer

Summary Report

Johnsonville Fossil Plan

535 Steam Plant Rd

175559008

	3/30/2009				4/9/2009				4/30/2009			
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)
STN-AT	368.4	3.1	9.3	362.2	368.4	3.1	8.7	362.8	368.4	3.1	8.9	362.6
STN-AC	391.6	0.0	0.0	391.6	391.6	0.0	22.7	368.9	391.6	0.0	23.2	368.4
STN-AC-2	Not Installed				Not Installed				Not Installed			
STN-BT	369.8	3.0	11.7	361.1	369.8	3.0	10.6	362.2	369.8	3.0	10.8	362.0
STN-BC	392.4	0.0	0.0	392.4	392.4	0.0	21.7	370.7	392.4	0.0	11.0	381.4
STN-CT	368.9	3.0	8.3	363.6	368.9	3.0	8.1	363.8	368.9	3.0	8.3	363.6
STN-CC	392.5	0.0	0.0	392.5	392.5	0.0	24.9	367.6	392.5	0.0	24.8	367.7
STN-CC-2	Not Installed				Not Installed				Not Installed			
STN-C1T	Not Installed				Not Installed				Not Installed			
STN-C1C	Not Installed				Not Installed				Not Installed			
STN-C1C-2	Not Installed				Not Installed				Not Installed			
STN-DT	365.3	3.2	5.8	362.7	365.3	3.2	4.9	363.6	365.3	3.2	5.2	363.3
STN-DC	391.2	0.0	0.0	391.2	391.2	0.0	23.0	368.2	391.2	0.0	22.2	369.0
STN-DC-2	Not Installed				Not Installed				Not Installed			
STN-ET	363.8	3.1	6.2	360.7	363.8	3.1	5.4	361.5	363.8	3.1	5.7	361.2
STN-EC	390.4	0.0	0.0	390.4	390.4	0.0	3.9	386.5	390.4	0.0	3.9	386.5
STN-FT	362.9	2.9	4.6	361.2	362.9	2.9	3.7	362.1	362.9	2.9	3.9	361.9
STN-FC	389.8	0.0	0.0	389.8	389.8	0.0	3.9	385.9	389.8	0.0	3.6	386.2
STN-GT	360.8	2.9	2.8	360.9	360.8	2.9	1.7	362.0	360.8	2.9	1.5	362.2
STN-GC	389.8	0.0	0.0	389.8	389.8	0.0	0.0	389.8	389.8	0.0	16.7	373.1
STN-HT	363.1	2.7	4.9	360.9	363.1	2.7	4.2	361.6	363.1	2.7	4.5	361.3
STN-HC	390.0	0.0	0.0	390.0	390.0	0.0	3.5	386.5	390.0	0.0	3.6	386.4
STN-IT	368.8	3.0	10.7	361.1	368.8	3.0	10.0	361.8	368.8	3.0	10.2	361.6
STN-IC	390.1	0.0	0.0	390.1	390.1	0.0	3.5	386.6	390.1	0.0	3.7	386.4
STN-JT	378.7	3.0	21.0	360.7	378.7	3.0	20.2	361.5	378.7	3.0	20.2	361.5
STN-JC	390.0	0.0	0.0	390.0	390.0	0.0	2.6	387.4	390.0	0.0	3.6	386.4
STN-KT	377.6	3.0	19.0	361.6	377.6	3.0	18.3	362.3	377.6	3.0	18.4	362.2
STN-KC	390.5	0.0	0.0	390.5	390.5	0.0	2.9	387.6	390.5	0.0	3.5	387.0
STN-LT	366.3	3.0	8.4	360.9	366.3	3.0	7.5	361.8	366.3	3.0	7.7	361.6
STN-LC	390.5	0.0	0.0	390.5	390.5	0.0	3.2	387.3	390.5	0.0	3.6	386.9
STN-MT	365.6	3.3	8.0	360.9	365.6	3.3	7.3	361.6	365.6	3.3	7.3	361.6
STN-MC	391.1	0.0	0.0	391.1	391.1	0.0	5.3	385.8	391.1	0.0	5.9	385.2

Stantec

Piezometer

Summary Report

Johnsonville Fossil Plan

535 Steam Plant Rd

175559008

	5/27/2009				9/2/2009				9/15/2009			
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)
STN-AT	368.4	3.1	9.0	362.5	368.4	3.1	10.7	360.8	368.4	3.1	14.4	357.1
STN-AC	391.6	0.0	22.6	369.0	391.6	0.0	23.4	368.2	391.6	0.0	23.3	368.3
STN-AC-2	Not Installed				Not Installed				Not Installed			
STN-BT	369.8	3.0	11.0	361.8	369.8	3.0	13.6	359.2	369.8	3.0	17.5	355.3
STN-BC	392.4	0.0	9.7	382.7	392.4	0.0	9.3	383.1	392.4	0.0	8.9	383.5
STN-CT	368.9	3.0	8.4	363.5	368.9	3.0	8.9	363.0	368.9	3.0	12.0	359.9
STN-CC	392.5	0.0	23.6	368.9	392.5	0.0	24.8	367.7	392.5	0.0	0.0	392.5
STN-CC-2	Not Installed				Not Installed				Not Installed			
STN-C1T	Not Installed				Not Installed				Not Installed			
STN-C1C	Not Installed				Not Installed				Not Installed			
STN-C1C-2	Not Installed				Not Installed				Not Installed			
STN-DT	365.3	3.2	5.2	363.3	365.3	3.2	0.0	368.5	365.3	3.2	0.0	368.5
STN-DC	391.2	0.0	22.0	369.2	391.2	0.0	21.5	369.7	391.2	0.0	21.2	370.0
STN-DC-2	Not Installed				Not Installed				Not Installed			
STN-ET	363.8	3.1	5.6	361.3	363.8	3.1	8.4	358.5	363.8	3.1	12.3	354.6
STN-EC	390.4	0.0	3.8	386.6	390.4	0.0	10.5	379.9	390.4	0.0	12.1	378.3
STN-FT	362.9	2.9	1.9	363.9	362.9	2.9	6.8	359.0	362.9	2.9	10.6	355.2
STN-FC	389.8	0.0	3.6	386.2	389.8	0.0	8.2	381.6	389.8	0.0	10.9	378.9
STN-GT	360.8	2.9	0.0	363.7	360.8	2.9	4.9	358.8	360.8	2.9	5.5	358.2
STN-GC	389.8	0.0	13.0	376.8	389.8	0.0	11.2	378.6	389.8	0.0	15.1	374.7
STN-HT	363.1	2.7	4.2	361.6	363.1	2.7	7.2	358.6	363.1	2.7	10.7	355.1
STN-HC	390.0	0.0	3.6	386.4	390.0	0.0	9.1	380.9	390.0	0.0	11.0	379.0
STN-IT	368.8	3.0	9.9	361.9	368.8	3.0	13.0	358.8	368.8	3.0	16.9	354.9
STN-IC	390.1	0.0	3.3	386.8	390.1	0.0	9.8	380.3	390.1	0.0	10.8	379.3
STN-JT	378.7	3.0	18.2	363.6	378.7	3.0	23.0	358.7	378.7	3.0	28.1	353.6
STN-JC	390.0	0.0	2.8	387.2	390.0	0.0	4.6	385.4	390.0	0.0	4.3	385.7
STN-KT	377.6	3.0	18.6	362.0	377.6	3.0	21.2	359.4	377.6	3.0	25.1	355.5
STN-KC	390.5	0.0	0.0	390.5	390.5	0.0	4.0	386.5	390.5	0.0	0.0	390.5
STN-LT	366.3	3.0	7.8	361.5	366.3	3.0	10.4	358.9	366.3	3.0	14.3	355.0
STN-LC	390.5	0.0	3.3	387.2	390.5	0.0	3.7	386.8	390.5	0.0	3.8	386.7
STN-MT	365.6	3.3	7.6	361.3	365.6	3.3	10.3	358.6	365.6	3.3	14.5	354.4
STN-MC	391.1	0.0	5.3	385.8	391.1	0.0	5.5	385.6	391.1	0.0	5.8	385.3

Stantec

Piezometer

Summary Report

Johnsonville Fossil Plan

535 Steam Plant Rd

175559008

Location	10/15/2009				11/16/2009				12/17/2009			
	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)
STN-AT	368.4	3.1	13.8	357.7	368.4	3.1	13.4	358.1	368.4	3.1	12.6	358.9
STN-AC	391.6	0.0	22.6	369.0	391.6	0.0	22.7	369.0	391.6	0.0	22.4	369.3
STN-AC-2	Not Installed				Not Installed				0.0	0.0	22.7	-22.7
STN-BT	369.8	3.0	16.6	356.2	369.8	3.0	15.7	357.1	369.8	3.0	14.6	358.2
STN-BC	392.4	0.0	9.0	383.4	392.4	0.0	9.1	383.3	392.4	0.0	9.8	382.6
STN-CT	368.9	3.0	11.7	360.2	368.9	3.0	11.9	360.0	368.9	3.0	11.6	360.4
STN-CC	392.5	0.0	4.6	388.0	392.5	0.0	4.5	388.0	392.5	0.0	4.5	388.0
STN-CC-2	Not Installed				Not Installed				0.0	0.0	24.0	-24.0
STN-C1T	Not Installed				Not Installed				0.0	0.0	4.4	-4.4
STN-C1C	391.5	0.0	14.2	377.3	391.5	0.0	14.3	377.2	391.5	0.0	12.9	378.6
STN-C1C-2	Not Installed				Not Installed				0.0	0.0	10.2	-10.2
STN-DT	365.3	3.2	0.0	368.5	365.3	2.6	10.0	357.9	365.3	2.6	8.7	359.2
STN-DC	391.2	0.0	21.2	370.0	391.2	0.0	21.1	370.1	391.2	0.0	20.8	370.4
STN-DC-2	Not Installed				Not Installed				0.0	0.0	24.4	-24.4
STN-ET	363.8	3.1	11.7	355.2	363.8	3.1	10.4	356.4	363.8	3.1	8.9	357.9
STN-EC	390.4	0.0	12.1	378.4	390.4	0.0	7.6	382.8	390.4	0.0	6.6	383.8
STN-FT	362.9	2.9	9.7	356.1	362.9	2.9	8.5	357.4	362.9	2.9	7.2	358.6
STN-FC	389.8	0.0	11.4	378.4	389.8	0.0	8.1	381.7	389.8	0.0	7.9	381.9
STN-GT	360.8	2.9	0.0	363.7	360.8	3.0	5.2	358.6	360.8	3.0	5.0	358.8
STN-GC	389.8	0.0	15.9	373.9	389.8	0.0	16.1	373.7	389.8	0.0	15.1	374.7
STN-HT	363.1	2.7	9.8	356.0	363.1	2.7	8.8	357.0	363.1	2.7	7.3	358.5
STN-HC	390.0	0.0	10.8	379.1	390.0	0.0	7.5	382.4	390.0	0.0	7.0	383.0
STN-IT	368.8	3.0	15.8	356.0	368.8	3.0	14.9	356.9	368.8	3.0	13.4	358.4
STN-IC	390.1	0.0	10.2	379.8	390.1	0.0	6.7	383.3	390.1	0.0	5.8	384.3
STN-JT	378.7	3.0	28.5	353.2	378.7	3.0	24.9	356.8	378.7	3.0	23.8	357.9
STN-JC	390.0	0.0	3.1	386.9	390.0	0.0	3.7	386.3	390.0	0.0	3.1	386.9
STN-KT	377.6	3.0	24.3	356.3	377.6	3.0	23.2	357.4	377.6	3.0	22.0	358.6
STN-KC	390.5	0.0	2.9	387.6	390.5	0.0	3.1	387.4	390.5	0.0	2.8	387.7
STN-LT	366.3	3.0	13.4	355.9	366.3	3.0	12.6	356.7	366.3	3.0	11.3	358.0
STN-LC	390.5	0.0	3.2	387.3	390.5	0.0	3.0	387.5	390.5	0.0	2.9	387.6
STN-MT	365.6	3.3	13.9	355.0	365.6	3.3	12.6	356.3	365.6	3.3	11.2	357.7
STN-MC	391.1	0.0	5.5	385.6	391.1	0.0	5.1	386.0	391.1	0.0	4.8	386.4

Stantec

Piezometer

Summary Report

Johnsonville Fossil Plan

535 Steam Plant Rd

175559008

	12/29/2009				1/25/2010				2/22/2010			
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)
STN-AT	368.4	3.1	14.6	356.9	368.4	3.1	12.8	358.7	368.4	3.1	15.4	356.1
STN-AC	391.6	0.0	22.8	368.8	391.6	0.0	22.5	369.1	391.6	0.0	23.2	368.4
STN-AC-2	0.0	0.0	23.1	-23.1	391.8	0.0	22.9	368.9	391.8	0.0	23.5	368.3
STN-BT	369.8	3.0	17.4	355.4	369.8	3.0	15.2	357.6	369.8	3.0	18.3	354.5
STN-BC	392.4	0.0	10.1	382.3	392.4	0.0	10.8	381.6	392.4	0.0	11.3	381.1
STN-CT	368.9	3.0	11.8	360.2	368.9	3.0	11.6	360.4	368.9	3.0	11.8	360.2
STN-CC	392.5	0.0	24.3	368.2	392.5	0.0	24.34*	368.2	392.5	0.0	24.3*	368.2
STN-CC-2	0.0	0.0	24.1	-24.1	392.4	0.0	24.1*	368.3	392.4	0.0	24.1	368.3
STN-C1T	0.0	0.0	5.1	-5.1	365.5	0.0	4.9	360.6	365.5	0.0	4.9	360.6
STN-C1C	391.5	0.0	10.6	380.9	391.5	0.0	12.6	378.9	391.5	0.0	11.5	380.0
STN-C1C-2	0.0	0.0	NA	NA	392.4	0.0	9.3	383.1	392.4	0.0	10.3	382.1
STN-DT	365.3	2.6	11.4	356.5	365.3	2.6	9.2	358.7	365.3	2.6	11.5	356.4
STN-DC	391.2	0.0	21.0	370.2	391.2	0.0	21.2	370.0	391.2	0.0	21.4	369.9
STN-DC-2	0.0	0.0	24.3	-24.3	391.0	0.0	24.38*	366.6	391.0	0.0	24.4*	366.6
STN-ET	363.8	3.1	11.7	355.1	363.8	3.1	9.5	357.3	363.8	3.1	12.6	354.2
STN-EC	390.4	0.0	6.3	384.1	390.4	0.0	6.0	384.4	390.4	0.0	6.2	384.2
STN-FT	362.9	2.9	8.8	357.0	362.9	2.9	7.5	358.3	362.9	2.9	8.8	357.1
STN-FC	389.8	0.0	5.9	383.9	389.8	0.0	5.7	384.1	389.8	0.0	5.7	384.1
STN-GT	360.8	3.0	7.0	356.8	360.8	3.0	5.1	358.7	360.8	3.0	6.0	357.9
STN-GC	389.8	0.0	14.7	375.1	389.8	0.0	13.1	376.7	389.8	0.0	14.2	375.6
STN-HT	363.1	2.7	10.1	355.7	363.1	2.7	7.9	357.9	363.1	2.7	11.4	354.4
STN-HC	390.0	0.0	3.0	387.1	390.0	0.0	7.0	383.0	390.0	0.0	5.7	384.3
STN-IT	368.8	3.0	16.1	355.7	368.8	3.0	13.9	358.0	368.8	3.0	17.2*	354.7
STN-IC	390.1	0.0	5.8	384.4	390.1	0.0	5.5	384.6	390.1	0.0	5.6	384.5
STN-JT	378.7	3.0	26.6	355.1	378.7	3.0	25.0	356.7	378.7	3.0	28.5	353.2
STN-JC	390.0	0.0	3.2	386.8	390.0	0.0	2.3	387.7	390.0	0.0	2.6	387.5
STN-KT	377.6	3.0	24.9	355.7	377.6	3.0	22.8	357.8	377.6	3.0	25.7	354.9
STN-KC	390.5	0.0	3.1	387.4	390.5	0.0	2.5	388.0	390.5	0.0	2.8	387.7
STN-LT	366.3	3.0	14.0	355.3	366.3	3.0	11.8	357.5	366.3	3.0	14.2	350.1
STN-LC	390.5	0.0	3.4	387.1	390.5	0.0	3.1	387.4	390.5	0.0	3.3	387.2
STN-MT	365.6	3.3	14.0	354.9	365.6	3.3	11.8	357.2	365.6	3.3	15.4	353.6
STN-MC	391.1	0.0	5.6	385.5	391.1	0.0	5.2	385.9	391.1	0.0	5.7	385.4

* Dry

Stantec

Piezometer

Summary Report

Johnsonville Fossil Plan

535 Steam Plant Rd

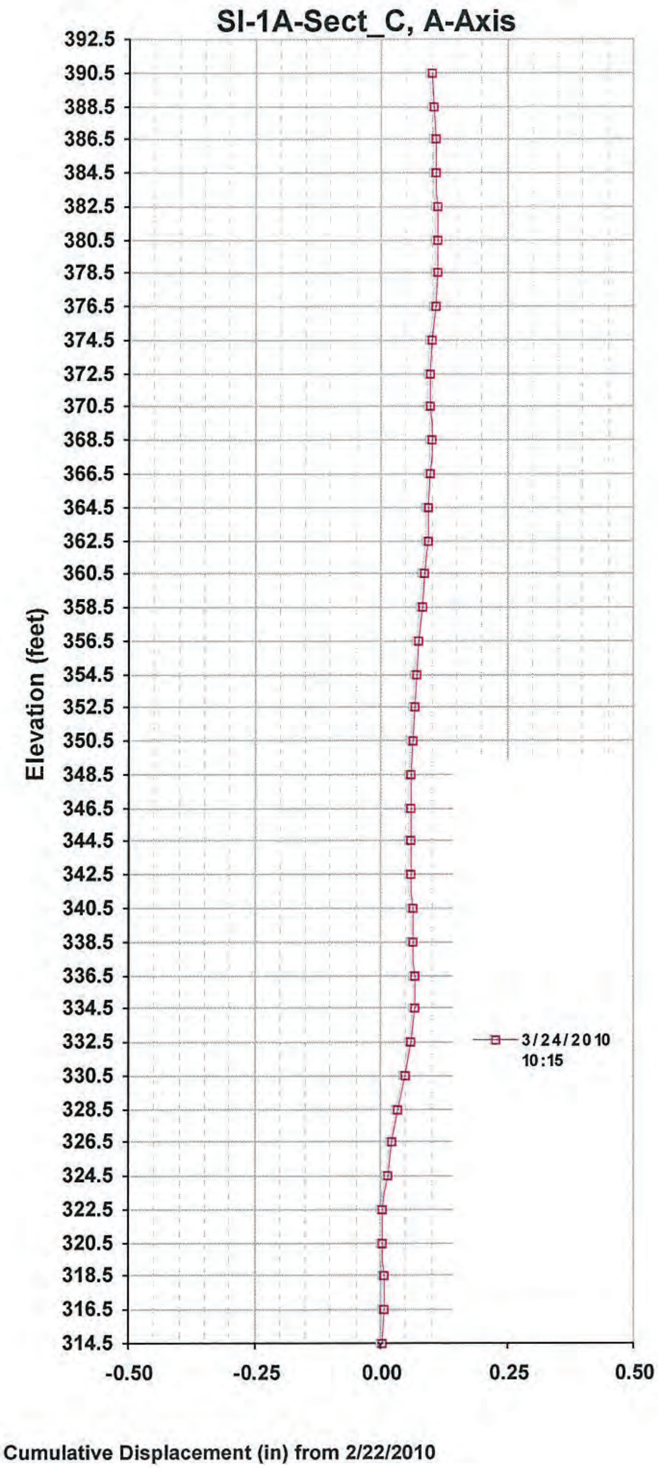
175559008

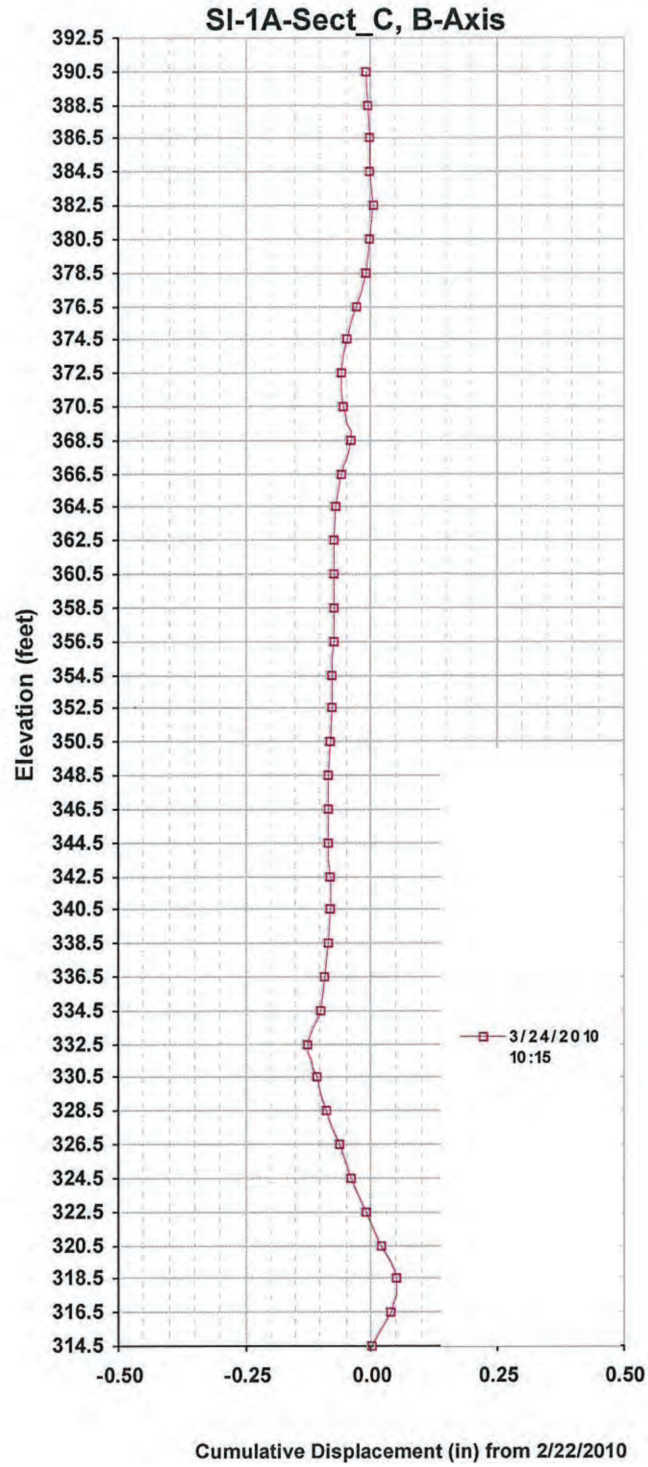
	3/24/2010			
Location	Surface Elevation (ft)	Stickup (ft)	Depth Measurement(ft)	Water Elevation (ft)
STN-AT	368.4	3.1	2.9**	368.6
STN-AC	391.6	0.0	23.8	367.8
STN-AC-2	391.8	0.0	24.2***	367.6
STN-BT	369.8	3.0	16.5	356.4
STN-BC	392.4	0.0	13.6	378.8
STN-CT	368.9	3.0	11.5	360.4
STN-CC	392.5	0.0	24.2	368.3
STN-CC-2	392.4	0.0	24.2*	368.3
STN-C1T	365.5	0.0	4.7	360.8
STN-C1C	391.5	0.0	13.1	378.5
STN-C1C-2	392.4	0.0	11.2	381.2
STN-DT	365.3	2.6	10.5	357.4
STN-DC	391.2	0.0	21.6	369.6
STN-DC-2	391.0	0.0	24.3*	366.7
STN-ET	363.8	3.1	11.1	355.7
STN-EC	390.4	0.0	6.3	384.2
STN-FT	362.9	2.9	9.0	356.9
STN-FC	389.8	0.0	5.9	383.9
STN-GT	360.8	3.0	6.1	357.7
STN-GC	389.8	0.0	14.8	375.0
STN-HT	363.1	2.7	9.6	356.2
STN-HC	390.0	0.0	5.8	384.2
STN-IT	368.8	3.0	15.5	356.4
STN-IC	390.1	0.0	5.7	384.4
STN-JT	378.7	3.0	26.2	355.5
STN-JC	390.0	0.0	2.5	387.5
STN-KT	377.6	3.0	24.2	356.4
STN-KC	390.5	0.0	3.0	387.5
STN-LT	366.3	3.0	13.0	356.3
STN-LC	390.5	0.0	3.6	386.9
STN-MT	365.6	3.3	13.5	355.4
STN-MC	391.1	0.0	6.5	384.6

* Dry

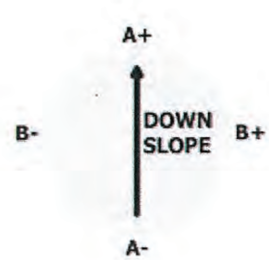
** Damaged by fill (rock)

*** Mud



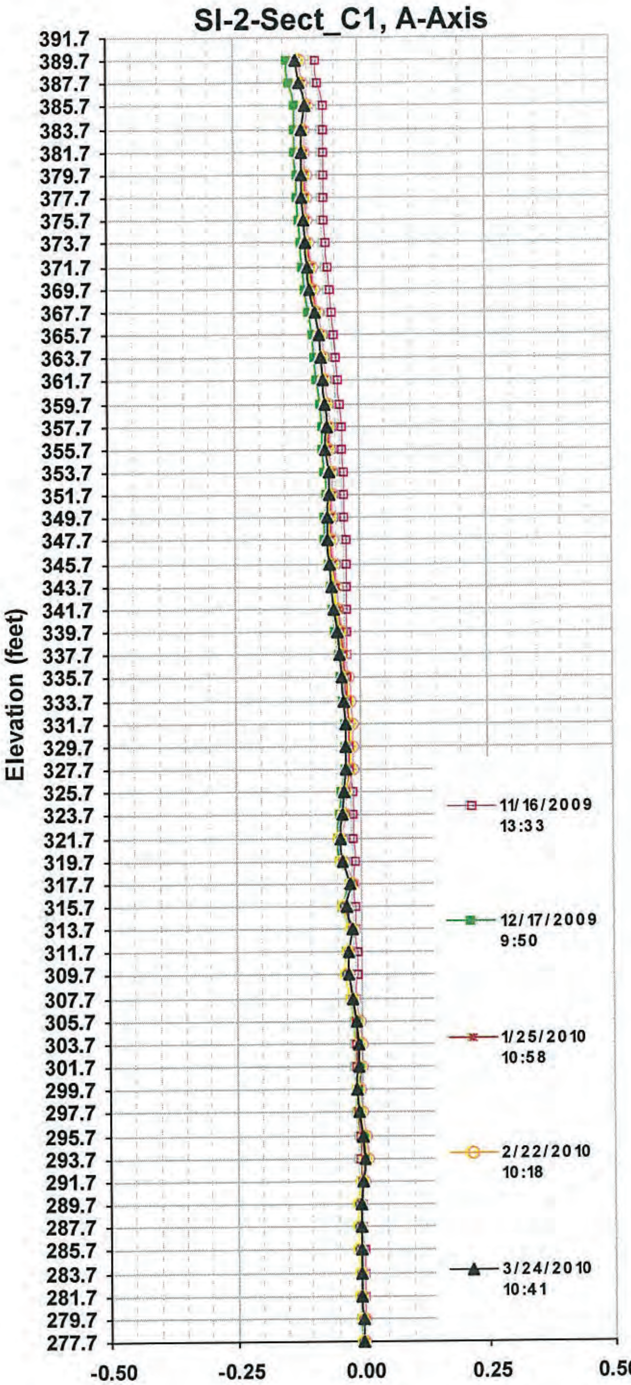


Stantec

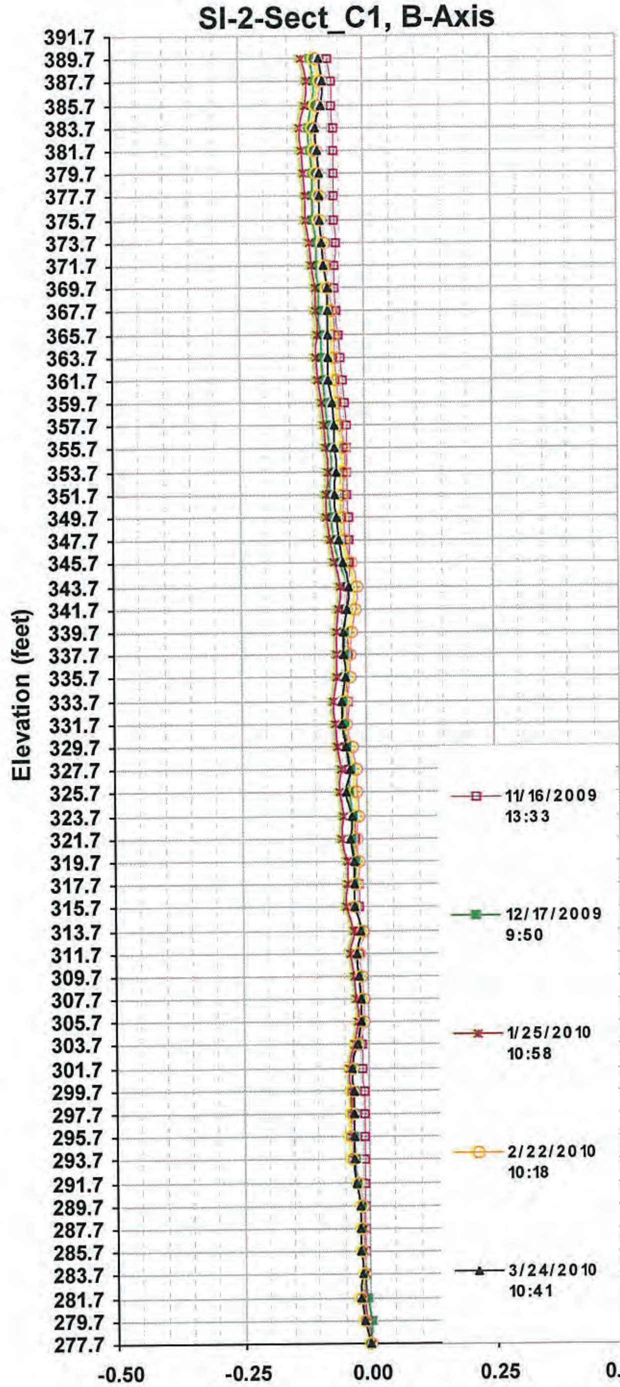


Johnsonville Fossil Plant

New Johnsonville, TN
175559008
4/7/2010



Cumulative Displacement (in) from 9/23/2009



Cumulative Displacement (in) from 9/23/2009

Stantec

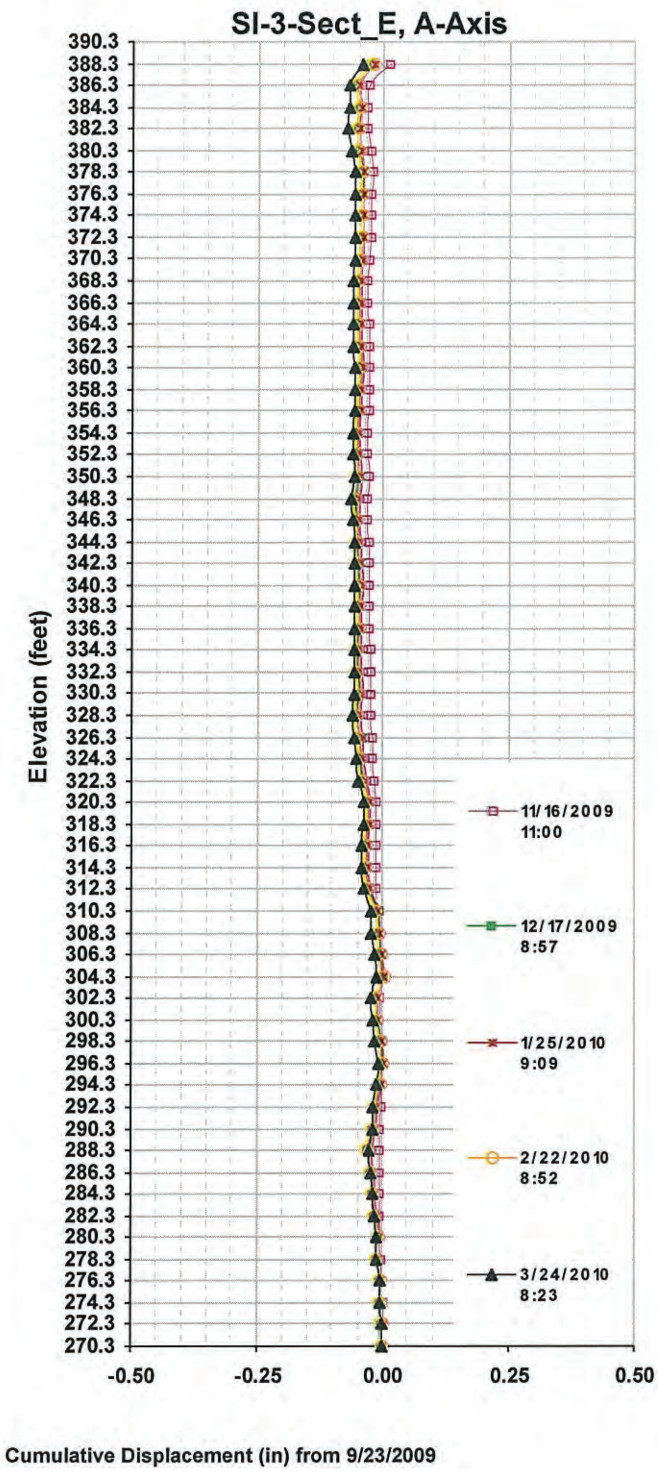
A+

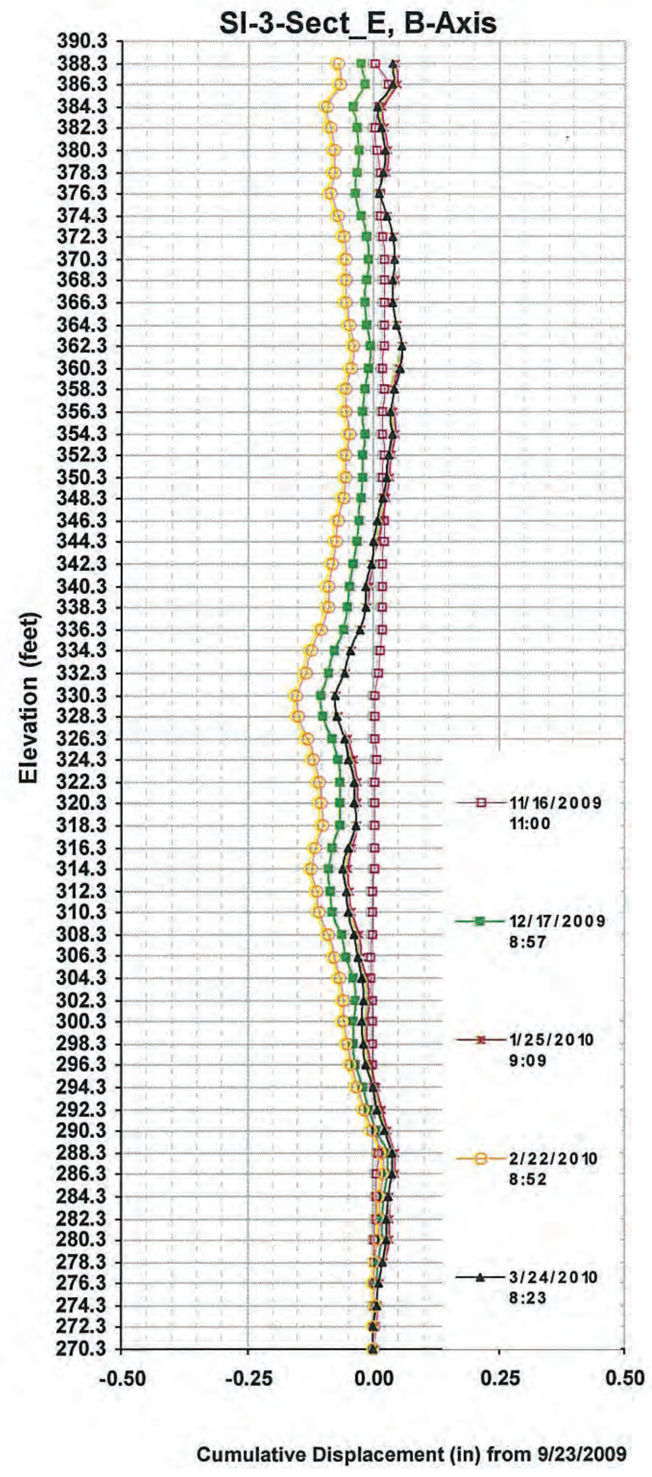
B- DOWN SLOPE B+

A-

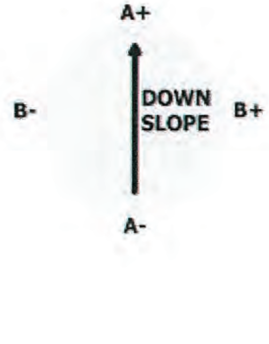
Johnsonville Fossil Plant

New Johnsonville, TN
175559008
3/26/2010





Stantec

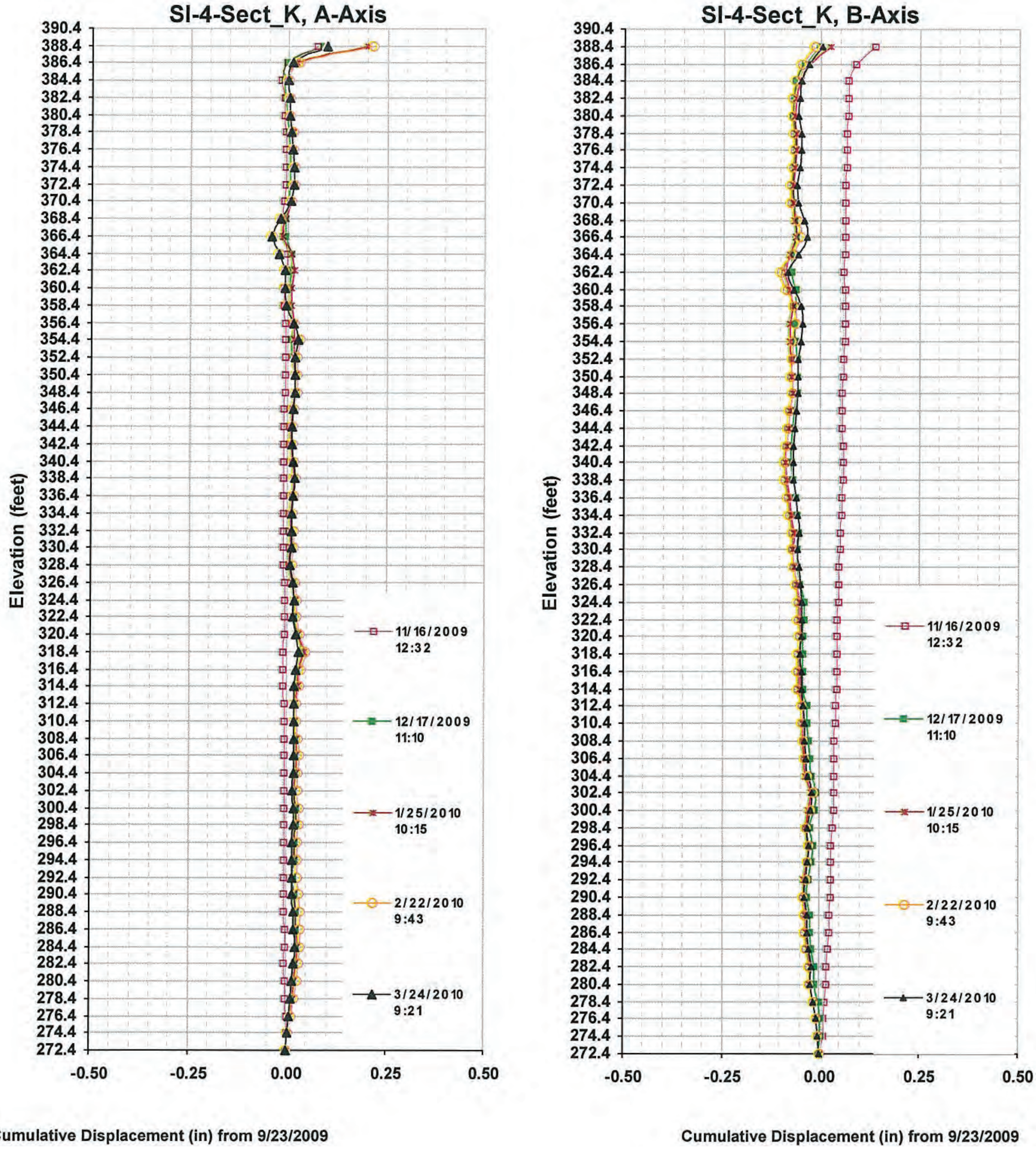


Johnsonville Fossil Plant

New Johnsonville, TN

175559008

3/26/2010



Stantec

Johnsonville Fossil Plant

New Johnsonville, TN

175559008

3/26/2010

APPENDIX B

Document 14

Dam Inspection Checklist Form

Site Name:	**Johnsonville Fossil Plant**	**Date:**	**20 September 2011**
Unit Name:	**Active Ash Pond 2**	**Operator's Name:**	**TVA**
Unit I.D.:	**NID: TN08512**	**Hazard Potential Classification:**	**High** ☐ **Significant** ☒[1] **Low** ☐
	Assessor's Name:	**Stanley W. Notestine, PE; Frederic C. Tucker, PE**	

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?	Annually[2]		18. Sloughing or bulging on slopes?		X
2. Pool elevation (operator records)?	384.5'		19. Major erosion or slope deterioration or animal holes?		X
3. Decant inlet elevation (operator records)?	384.0'		20. Decant Pipes:		
4. Open channel spillway elevation (operator records)? Notch bottom elevation.	N/A		Is water entering inlet, but not exiting outlet?		X
5. Lowest dam crest elevation (operator records)?	391' TBV		Is water exiting outlet, but not entering inlet?		X
6. If instrumentation is present, are readings recorded (operator records)?	X[3]		Is water exiting outlet flowing clear?	X	
7. Is the embankment currently under construction?		X[4]	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)?	UKN[5]		From underdrain?		X[7]
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?	N/A	
11. Is there significant settlement along the crest?		X	Over widespread areas?		X
12. Are decant trashracks clear and in place?	X[6]		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?	N/A	
16. Are outlets of decant or underdrains blocked?		X	23. Water against downstream toe?	X[8]	
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.

N/A = Not Applicable UKN = Unknown TBP = To Be Provided TBV = To Be Verified

Note #	Comments
1	Hazard potential classification was determined by TVA. The indicated "significant" hazard potential classification also is Dewberry's interpretation, based on EPA criteria shown on page 3.
2	TVA engineers conduct annual inspections. The inspections are documented in written reports, which include measures, as needed, for maintenance and repair. Plant personnel make observations throughout the year.
3	Four inclinometers and 32 piezometers are monitored monthly.

4	Extensive remedial work principally including a new spillway and slope flattening, inverted filters for seepage control and rock toe buttresses for increased slope stability along the northeast and southeast perimeter dikes has recently been completed.
5	The foundation of the northeast and southeast perimeter dikes is comprised of material dredged during original plant construction in the mid-1940s from the barge unloading area (boat harbor) and condenser water inlet (intake) channel and sluiced into place in the river (Kentucky Lake) to form protective “breakwaters” where the northeast and southeast perimeter dikes currently exist. During pond construction at a later time (1968-1970) the interior area of the ash pond was dredged and the material sluiced into place in the lake and on preexisting small islands to form the foundation of the embankment along the current northwest and southwest sides of the ash pond to enclose the pond area. Above the sluiced foundation materials the dike embankment was constructed of rolled earthfill (clay). Construction records are not available. It is not known how the original breakwater dikes were prepared prior to placing dike embankments during pond construction in 1968-1970, and it is not known how the original dike embankments were prepared when they were raised in 1978. Out of 48 test borings made in 2009, a couple encountered organic matter, one encountered peat, and one encountered a 6” diameter tree root at about elevation 377, just below the crest elevation of the original dike. Thus, there is some organic matter and deleterious material, but it does not appear to be extensive.
6	Skimmers are in place at the inlets.
7	Seepage was not observed. However, seepage presumably still exists, but it is covered and controlled with a new inverted filter, as well as a thick blanket of riprap, and is not visible. The inverted filter design did not include seepage collection and removal pipes. Therefore, there is no discreet discharge point for seepage from the inverted filter.
8	Kentucky Lake, the boat harbor, and the intake channel surround the ash pond.

Coal Combustion Residue (CCR)

Impoundment Assessment

Impoundment NPDES Permit TN0005444 **ASSESSOR** Stanley W. Notestine, PE; Frederic C. Tucker, PE

Effective Date 03/01/2011
Impoundment Name Active Ash Pond 2 (aka: Ash Disposal Areas 2 & 3)

Impoundment Company TVA
EPA Region 4

State Agency Tennessee Department of Environment and Conservation
Division of Water Pollution Control.
(Field Office) Address 401 Church street, 6th Floor, L & C Annex
Nashville, TN 37243-1534
Name of Impoundment Active Ash Pond 2

(Report each impoundment on a separate form under the same Impoundment NPDES Permit number)

New ☐ **Update** ☒

	Yes	**No**
Is impoundment currently under construction?	☐	☒
Is water or ccr currently being pumped into the impoundment?	☒	☐

IMPOUNDMENT FUNCTION: The impoundment currently serves as a transfer facility. The impoundment receives both fly ash and bottom ash, which are stored temporarily, dredged and stacked for dewatering; then loaded onto dump trucks and hauled to a landfill for permanent disposal near Camden, Tennessee, approximately 5 miles away.

Nearest Downstream Town Name: New Johnsonville, Tennessee

Distance from the impoundment: 0 miles (within city limits)

Location:

Latitude	36	Degrees	01	Minutes	37.3	Seconds	**N**
Longitude	87	Degrees	59	Minutes	36.9	Seconds	**W**

State Tennessee **County** Humphreys

	Yes	No
Does a state agency regulate this impoundment?	☒	☐

If So Which State Agency? Tennessee Department of Environment and Conservation. For water quality only.

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.

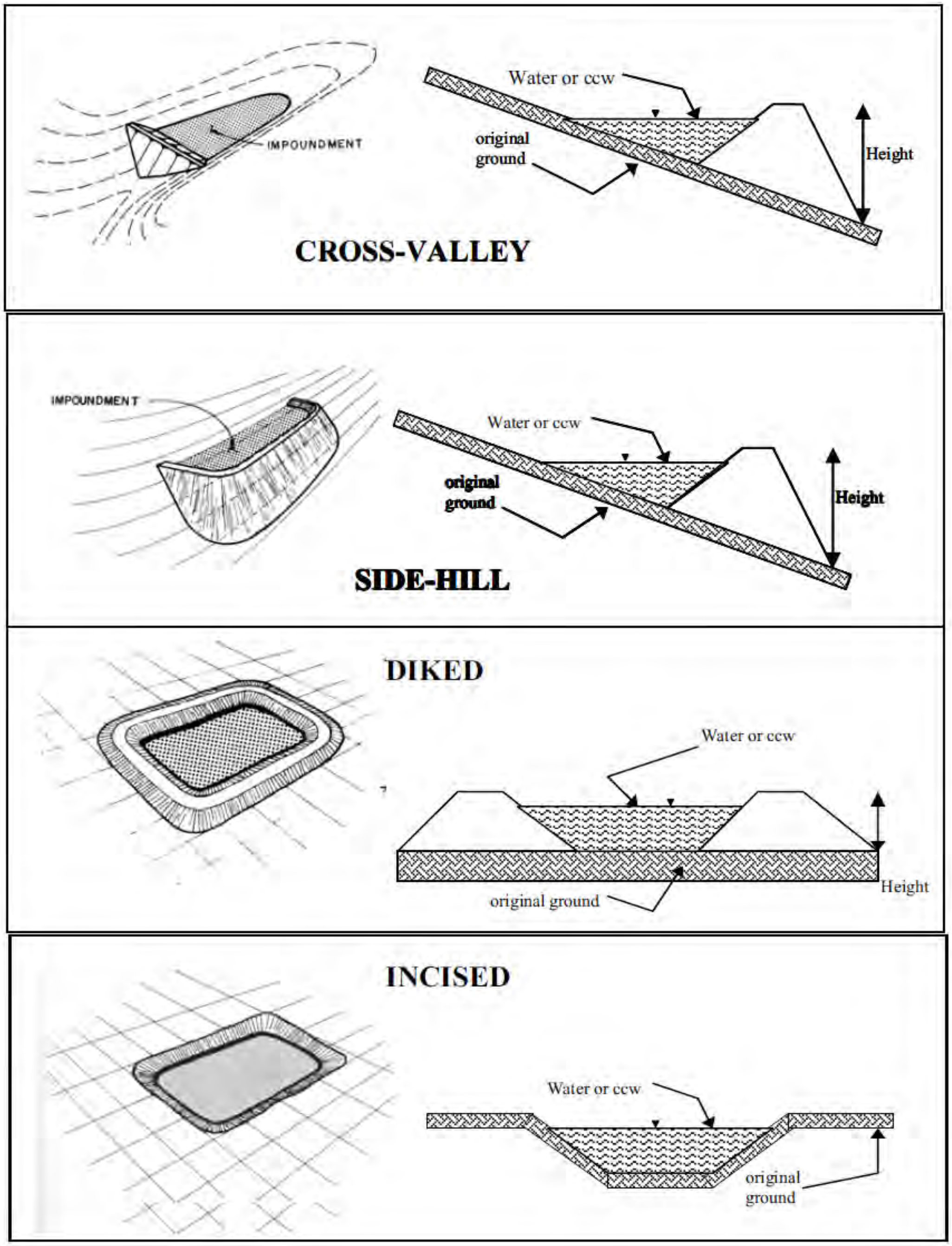
☒ **SIGNIFICANT HAZARD POTENTIAL:** Dams assigned the significant hazard potential classification are those dams where failure or misoperation results in no probable loss of human life but can cause economic loss, environmental damage, disruption of lifeline facilities, or can impact other concerns. 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:

Dike failure would discharge coal combustion residue directly into Kentucky Lake with significant environmental consequences and some potential impact on nearby lower-lying shore areas that are within the New Johnsonville City Limits.

CONFIGURATION:



- ☐ Cross-Valley
- ☐ Side-Hill
- ☐ Diked
- ☐ Incised (form completion optional)
- ☒ Combination Incised/Diked

Embankment Height (ft)	45 (max)	**Embankment Material**	Earth
Pond Area (ac)	87	**Liner**	No
Current Freeboard (ft)	6.5	**Liner Permeability**	N/A

TYPE OF OUTLET (Mark all that apply)

☐ **Open Channel Spillway**

☐ Trapezoidal

☐ Triangular

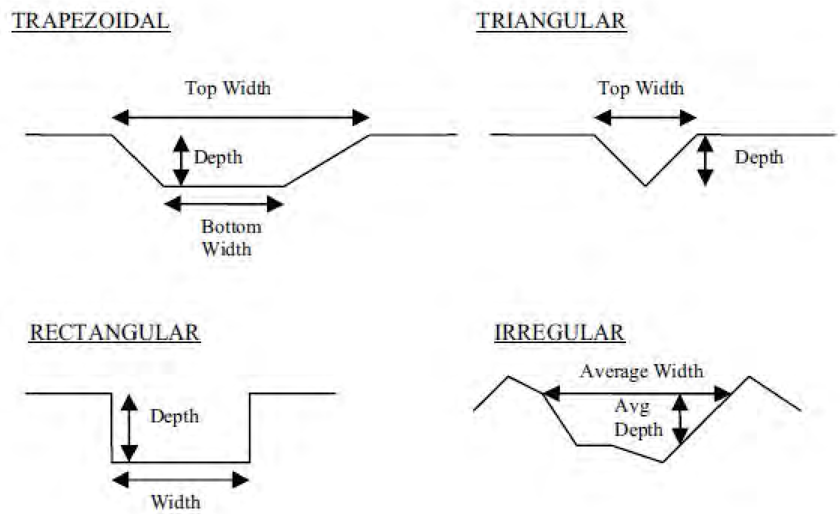
☐ Rectangular

☐ Irregular

depth (ft)

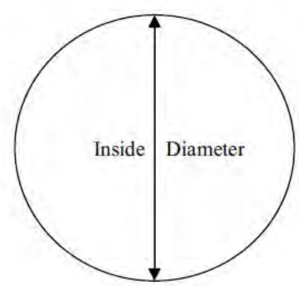
average bottom width (ft)

top width (ft)



Outlet

☒ New spillway consisting of 6-30" outside diameter (~26" ID) HDPE pipes each with a precast concrete inlet structure fitted with 6" high fiberglass stoplogs to control water level and a concrete end wall at the outlet end with a raised sill for energy dissipation.



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 TVA

	Yes	No
Has there ever been a failure at this site?	☐	☒

If So When?

If So Please Describe: There have been no failures that have caused releases. There was some minor erosion of the outside toe caused by high river flow in the mid-1990s, which was subsequently repaired. In 1994 a sinkhole developed over one of the old spillway pipes. That pipe was subsequently taken out of service. With the recent construction of the new spillway all the old spillway pipes through the dike embankment have been fully grouted.

	Yes	No
Has there ever been significant seepages at this site?	☒	☐

If So When? Reported in Phase I Report of assessments conducted by Stantec Consulting Services Inc. (Stantec) on January 12, 2009 and February 23-25, 2009 and in previous inspections performed by TVA.

If So Please Describe: Stantec reported "Significant seepage along the northeast and southeast dikes." It was noted that a seepage collection system had been installed along the southeast dike for better monitoring and that wet areas were present in the area of seepage and standing water was observed along the access road to the toe of the northeast dike. However, there was no mention of cloudy seepage or seepage flow velocities high enough to transport soil particles. All the seepage have subsequently been covered with inverted filters and are buried under the new rock toe buttresses along the outside toes of the northeast and southeast dikes.

	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,...)? Lowering operating water level in the pond 3.0’ by installing a new spillway system with lower inlet elevation than the previously existing spillway system.

If So Please Describe: The new spillway system was installed primarily to increase freeboard, to prevent overtopping during the selected design flood (Probable Maximum Flood), but a side benefit was lowering of the phreatic line, which decreased seepage pressures in the embankments and presumably decreased the quantity of seepage along the northeast and southeast dikes; the reduced seepage flow is filtered through a new drainage blanket.

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.

Yes. The perimeter dike was completed in 1970 and raised in 1978. The dike embankment raise was made in the upstream direction, which resulted in the raised section being partly founded on settled ash in the pond.

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?

There was no indication of prior releases, failures, or patchwork on the dikes. However, substantial improvements have recently been made to increase safety to U.S. Army Corps of Engineers standards.