Report of Geotechnical Exploration and Slope Stability Evaluation

Ash Disposal Areas 2 and 3 (Active Ash Disposal Area)
Johnsonville Fossil Plant
New Johnsonville, Tennessee

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Prepared for:
Tennessee Valley Authority
Chattanooga, Tennessee

April 13, 2010

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April 13, 2010

O.1.1.175559008R01

Mr. Michael S. Turnbow
Tennessee Valley Authority
1101 Market Street
LP 2N
Chattanooga, Tennessee

Re: Report of Geotechnical Exploration and Slope Stability Evaluation
Ash Disposal Areas 2 and 3 (Active Ash Disposal Area)
Johnsonville Fossil Plant
New Johnsonville, Tennessee

Dear Mr. Turnbow:

As requested, Stantec Consulting Services Inc. (Stantec) has completed our Geotechnical Exploration and Slope Stability Evaluation for Ash Disposal Areas 2 and 3 (Active Ash Disposal Area) at the Johnsonville Fossil Plant. The report documents the subsurface conditions, results of laboratory testing, findings from the historical document reviews, results of our analyses and evaluation, and recommendations for the facility. These services were performed under Engineering Service Request ESR/TAO 700 in accordance with the terms and provisions established in our System-Wide Services Agreement dated December 22, 2008.

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

Sincerely,

STANTEC CONSULTING SERVICES INC.

Paul Cooper

Paul J. Cooper, PE
Project Engineer

Stephen H. Bickel

Stephen H. Bickel, PE
Senior Principal

/day

Enclosures:

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Executive Summary

Stantec Consulting Services Inc. (Stantec) has completed the Geotechnical Exploration and Slope Stability Evaluation at the Johnsonville Fossil Plant Ash Disposal Areas 2 and 3 (Active Ash Disposal Area). This study was performed to address geotechnical issues identified during the Phase 1 Facility Assessment.

Background Information

The ash disposal area is situated on a 125-acre island within Kentucky Lake near New Johnsonville, Tennessee. It has been in operation for 40 years, and has been at its current top-of-dike elevation since 1978. Within the dike perimeter it is approximately 87 acres in area. The perimeter dike is 25 to 30 feet in height and is approximately 2 miles in length. The south one-third of the area contains three wet impoundments (Ponds A, B, and C) that total about 24 acres in surface area and contain about 313 acre-feet in water storage volume. The northern two-thirds of the area contains the sluice channel, a temporary ash stacking area and several temporary dredge cells. It has been classified by TVA as a significant hazard structure due to environmental consequences in the event of a dike breach.

Geotechnical issues associated with this facility focus on stability and seepage. Along the northeast and southeast sides, exterior slopes are steep and stand as steep as 1.5H:1V to 1.7H:1V. In 2001 a shallow slough developed on the northeast dike following a period of rain. It was repaired using riprap and geotextile fabric and has remained stable. There are areas of seepage along the toes of these dike sections. The most visible seepage area on the southeast dike, designated by TVA as Seep 3A, was addressed in February, 2009 by constructing a perforated pipe/crushed stone collection system so that the seepage could be readily monitored.

Scope of Geotechnical Exploration and Stability Analyses

The geotechnical exploration for the slope stability analyses included:

- Protocols and guidelines established by the US Army Corps of Engineers.
- A review of TVA's design drawings for comparison with existing conditions.
- Twenty nine soil test borings advanced by Stantec at separate stability cross-sections. These borings provide primary information for the stability analyses. Borings averaged from 50 to 60 feet total depth. Continuous sampling was performed.
- Piezometers for monitoring water levels were installed in separate borings drilled at these locations, with the exception of STN-HM.
- Twenty soil test borings were advanced by Stantec for supplemental geotechnical data. These borings averaged about 30 feet in depth.
- Sixty eight boring logs, developed during previous geotechnical studies, were reviewed for supplementary data.
- A laboratory testing program which included fourteen sets of consolidated undrained triaxial compression tests.

- Laboratory test results from earlier geotechnical studies were reviewed. Where applicable, these results were also used in establishing parameters for the analyses.
- Stability cross sections were surveyed by TVA's survey crews.
- Nine cross sections were selected for formal slope stability analyses. Seepage and slope stability analyses were performed and the long term steady state seepage factors of safety for both analyses were calculated at each cross section.
- Slope inclinometers were installed at four of the cross-sections that exhibited slope stability factors of safety less than 1.5.

Results of Exploration and Stability Analyses

The results from the geotechnical exploration indicate that the Active Ash Disposal Area perimeter dike system is comprised of clay and silty clay. Unlike the Kingston dredge cell dikes, the Johnsonville Active Ash disposal area perimeter dike does not contain ash. The perimeter dike was raised once in 1978 using the upstream method of construction; therefore, the upper dike's interior slopes extend over sluiced ash.

The dike is underlain by fill materials that were placed hydraulically by dredging to raise the land above the level of Kentucky Lake. In general, this material is not compacted, and it contains zones of higher permeability which transmit seepage from the ash disposal area. Beneath these materials, alluvial clays and silts, which grade into sand/gravel river deposits were encountered in the borings.

The results from the stability analyses indicate that a majority of the perimeter dike system does not exhibit an acceptable safety factor of 1.5 for the long term steady state seepage condition. Safety factors ranging from a low value of 1.2 (at Stability Section C on the northeast dike) to a high value of 1.6 (at Stability Section I on the southwest dike) were calculated for the "as-found" condition. In general, the lower safety factors were calculated on the northeast and southeast sections of the perimeter dike. To address the lower safety factors TVA is implementing the measures described below.

Future Closure Plan Stages to Improve Slope Stability

Concurrent with the geotechnical study, TVA decided that it will permanently close the Active Ash Disposal Area. The feasibility phase (Phase 1) of the final closure plan is on-going. During the period leading up to final closure, it is TVA's intent to address slope stability and to achieve acceptable safety factors throughout. Accordingly, the following four closure plan stages have been, or will be, designed and constructed:

1. A new spillway system was completed on the southwest dike during November, 2009. This has lowered the operating pool level for Ponds A, B, and C by about 2 feet and has lowered phreatic levels in the dike at Sections D through I.
2. The sluice channel has been rerouted across the active ash disposal area and away from the northeast and west dikes. The old northeast dike sluice channel is dewatered by pumping. The abandoned sluice channel inside the west dike will be lowered by about 2 feet.
3. The northeast dike exterior slope will be flattened using compacted clay and a rock stability berm will be installed against the bank slopes along the toe of the lower bench.

4. The southeast dike exterior slope will be flattened and a rock stability berm will be installed against the steep bank slopes along the toe of the lower bench.

Report of Geotechnical Exploration and Slope Stability Evaluation

Ash Disposal Areas 2 and 3 (Active Ash Disposal Area)
Johnsonville Fossil Plant
New Johnsonville, Tennessee

1. Introduction

In January, 2009 the Tennessee Valley Authority (TVA) requested that Stantec Consulting Services Inc. (Stantec) conduct assessments of its coal combustion product (CCP) disposal facilities at each of its eleven active and one closed fossil plants. The plants are located in the states of Kentucky, Tennessee and Alabama. The assessments were performed for the purpose of determining whether conditions are present that would indicate an unstable condition that could possibly cause a release of CCP's into the environment. Stantec's scope of services was developed within the framework of current dam safety practice, and was performed in several phases that are described as follows:

- Phase 1 – Review available documentation for CCP Disposal Facilities, perform site reconnaissance, obtain photographs and measurements, develop recommendations for engineering studies, and provide recommendations for work plans or designs to correct or improve the condition of the disposal facilities as necessary.
- Phase 2 – This phase includes engineering studies to determine geotechnical stability and/or hydrologic and hydraulic conditions of the disposal facilities.
- Phase 3 – Engineering design and permitting services related to remedial actions at existing or new CCP disposal facilities.

The Report of Phase 1 Facility Assessment for Coal Combustion Product Impoundments and Disposal Facilities for Tennessee was completed on June 24, 2009. The conclusions and recommendations for Ash Disposal Areas 2 and 3 (Active Ash Disposal Area) at the Johnsonville Fossil Plant (JOF) are included in that report. In addition to issues that require maintenance-type remedial activities, the Phase 1 site reconnaissance team for JOF documented steep, hummocky, and uneven dike slopes and areas at toes of dikes that displayed seepage, most notably along the northeast and southeast dikes. Shallow depressions were documented on the west dike slope. The Phase 1 document review revealed that the ash disposal area had been partially constructed on hydraulic fill that was placed to elevate the disposal area above the level of Kentucky Lake. In addition, a sinkhole had developed and been repaired on the southwest slope directly above a spillway outlet pipe during the 1990s.

For these reasons Stantec recommended a geotechnical study for the purpose of addressing the following concerns: 1) unknown general condition of materials comprising and underlying the dikes, and 2) seepage and slope stability of the dikes. This study was authorized by TVA under Engineering Services Request 700 during February, 2009. This report documents the scope and results of the study and contains Stantec's conclusions and recommendations concerning Ash Disposal Areas 2 and 3 (Active Ash Disposal Area).

2. General Site Description and Geology

2.1. Location and Description

The Johnsonville Fossil Plant is located in New Johnsonville which is in west-central Tennessee approximately 65 miles west from Nashville. The plant is situated on the eastern shore of Kentucky Lake in Humphreys County, Tennessee. It is approximately 3000 feet north from the US Highway 70 bridge that crosses Kentucky Lake, and 2.5 miles south from the Tennessee River and Trace Creek confluence.

The Active Ash Disposal Area at JOF is situated on a 125-acre island centered approximately 2,000 feet west of the plant's powerhouse. The island is connected to the mainland by a 1,000 foot causeway that supports an asphalt access road and the discharge piping from the plant to the sluicing channel in the disposal area. The island is surrounded by Kentucky Lake to the west and two dredged channels for coal unloading/barge mooring (the boat harbor channel) and intake condenser water to the east. Figure 1 on the following page provides a plan view of the active ash disposal area.

The ash disposal area is approximately 87 acres in area, and it is enclosed by a dike approximately 10,000 feet in length. The top of dike supports a gravel access road and is at or near Elev. 390 feet, which is 30 to 35 feet above the Kentucky Lake pool level. The dike slopes average about 25 feet in height, and they vary in steepness from 1.5H:1V throughout the east side of the disposal area, to greater than 2H:1V on the Kentucky Lake side. The slopes are vegetated with grasses and briers. Stands of mature trees exist at various locations around the lower perimeter near the pool level for Kentucky Lake. There is also a 3,500 foot length of the lower dike on the western perimeter covered by a blanket of riprap.

The Active Ash Disposal Area has been referred to using various names or terms throughout its existence. These include: Ash Disposal Area No. 2, 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. For the remainder of this report, reference to any of these names shall mean the Active Ash area as described above.

2.2. Geology

The geologic description for the Active Ash Disposal area is taken from the John Kellberg's report "Geology of the New Johnsonville Steam Plant Site", 1948. Based on this information the area is underlain by: recent river alluvium, and Devonian-age Chattanooga Shale and Camden Formations, in order of descending lithology.

Foundation drilling for the railroad bridge to the south indicated that alluvial deposits ranged up to 67 feet in depth, and averaged 60 feet deep beneath the floodplain (now submerged by Kentucky Lake) of the Tennessee River. Near the surface the alluvium consisted of fine grained silt and silty clay that grade into sand and river gravel with increasing depth. A groundwater monitoring well drilled at the Active Ash Disposal Area in 1986 encountered bedrock at approximate Elevation 290 feet, or about 100 feet below the dike. The sand and gravel alluvium was logged as being about 40 feet thick.

The Chattanooga Shale is a fissile, bituminous, carbonaceous shale that overlies the Camden Formation. It is likely thin to nonexistent beneath the Active Ash Disposal Area. The Camden formation consists of thin (from one to three inches thick) beds of cherty limestone and contains hard, dense, brittle, white chert pieces, separated by softer gritty clay layers.

Figure 1. Ash Disposal Areas 2 and 3

3. Review of Available Information

3.1. General

During the Phase 1 Facility Assessment, Stantec's engineers reviewed documents provided by TVA pertaining to the Active Ash Disposal Area. The main objective of the document review was to gain historical information prior to beginning the field geotechnical exploration. The documents reviewed included record drawings, cross sections of dikes, old contour maps, and annual dike stability reports. A complete listing of the reviewed documents is included in the Phase 1 report.

Of particular interest and use in this study are the following reports and geotechnical documents:

- <u>Geology of the New Johnsonville Steam Plant Site</u>, John M. Kellburg, TVA Water Control Planning Department, Geologic Division, January 14, 1948.

- <u>Johnsonville Steam Plant – Ash Pond – Soil and Foundation Exploration</u>, J.C. McGraw, TVA Construction Services Branch, September 17, 1969

- <u>Johnsonville Steam Plant – Ash Disposal Area No. 2 Dike Raising – Soil Exploration and Testing</u>, G. Farmer, TVA Construction Services Branch, November 22, 1977.

- <u>Report of Geotechnical Evaluation, Ash Pond Dike, New Johnsonville Fossil Plant, New Johnsonville, Tennessee</u>, Law Engineering and Environmental Services, Inc., January 17, 1994.

- <u>Subsurface Exploration Data, TVA Borings at Johnsonville Fossil Plant, Johnsonville, Tennessee</u>, Law Engineering and Environmental Services, Inc., October 11, 1994.

- <u>Johnsonville Groundwater Assessment</u>, TVA Resource Group, Engineering Services, March 1995.

- <u>Results of Laboratory Testing – Grab Samples from Active Ash Pond</u>, Law Engineering and Environmental Services, Inc., July 1995.

- <u>Report of Subsurface Exploration and Stability Analysis – TVA – Johnsonville Fossil Plant Ash Disposal Area – New Johnsonville, Tennessee</u>, Law Engineering and Environmental Services, Inc., September 19, 1997.

- <u>Report of Ash Pond Investigation, Johnsonville Fossil Plant, New Johnsonville, Tennessee</u>, MACTEC Engineering and Consulting, August 28, 2003.

- <u>Report of Ash Pond Investigation, Johnsonville Fossil Plant, New Johnsonville, Tennessee</u>, MACTEC Engineering and Consulting, December 4, 2003.

These reports included boring plans, driller's logs and results from laboratory tests. The information gained from these reports was evaluated and used to supplement the information obtained during Stantec's geotechnical exploration.

3.2. Site History

In the mid-1940s Kentucky Dam on the Tennessee River was completed resulting in the impoundment of Kentucky Lake. Construction began at the Johnsonville Fossil Plant in 1949, and the plant was completed in 1952. As part of this construction, the Boat Harbor and Condenser Water Inlet Channels were dredged and the materials removed were sluiced and deposited along the east side of what would later become Ash Disposal Areas 2 and 3. Placement of the fill was to approximate Elevation 370 feet. This fill served as a breakwater to protect barges in the harbor area from waves. During the initial fifteen years of plant operation, fly ash and bottom ash were disposed of at Ash Disposal Area 1, which is immediately north of the plant's coal yard.

By the mid-1960s Ash Disposal Area 1 was reaching its capacity and TVA began its planning for JOF's second ash disposal facility. During 1968 to 1969 TVA completed the dike from the east breakwater and enclosed the entire area. This dike created Ash Disposal Area 2 (the current Active Ash Pond). Based on a concern that the dike could possibly be overtopped by waves on Kentucky Lake during periods of high water, it was raised to Elevation 378 feet in 1970. The material used was excavated from within the interior of the diked area and from a borrow area at the old construction camp ground, immediately east of the coal storage area. This material was reportedly placed in compacted lifts.

Throughout the 1970s Ash Disposal Area 2 served as the JOF's sole ash disposal area. By 1977 the ash level within the dikes was at approximate Elevation 374 feet. At this time the dike was raised 12 feet to Elevation 390 feet using compacted clay. When the remaining volume within the diked area filled, TVA employed several other measures for CCP disposal at JOF.

First, TVA permitted and constructed two other disposal management units on its Johnsonville reservation. These have been located in the south rail loop and east from the gas combustion turbine areas. While these were in operation during the 1980s and 1990s, ash sluiced to the Active Ash Disposal Area was dredged and pumped to these new facilities for permanent disposal. Both facilities have now been filled and closed.

Second, in recent years TVA has hired a private ash handling contractor to manage and dispose of its CCPs. Approximately 260,000 tons of fly ash and 30,000 tons of bottom ash are wet-sluiced to the Active Ash Disposal Area each year. As ash is pumped to the sluicing channel the majority is removed using long reach hydraulic excavators. The material dipped from the sluice channel is stacked at a higher level where it drains and dewaters. During the summer the accumulated ash is loaded into dump trucks and transported to a permitted landfill site. 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.

3.3. Historical Geotechnical Issues

As discussed in Section 1, the Phase 1 Facility Assessment Report noted several observations and concerns that are mainly geotechnical in nature. These are discussed in more detail below.

3.3.1. Seepage and Slope Stability

The exterior dike slopes, particularly throughout the northeast and southeast sides are relatively steep. TVA's drawings show that exterior dike slopes were designed to be 3H:1V below Elevation 378 feet and 2H:1V from Elevation 378 to 390 feet. However, slopes of 1.5H:1V to 1.7H:1V were measured by TVA's survey crews on the northeast and southeast dikes for the full height. In addition, hummocky and uneven slope surfaces were observed which could be evidence of shallow slope movement (creep). The fact that the dikes have been raised using the upstream method of construction over sluiced fly ash was listed in the Phase 1 Report as a slope stability concern, and the overall unknown composition of the dikes and foundation materials, considering the dike's steepness, height, and noted areas of seepage, was also listed as a concern.

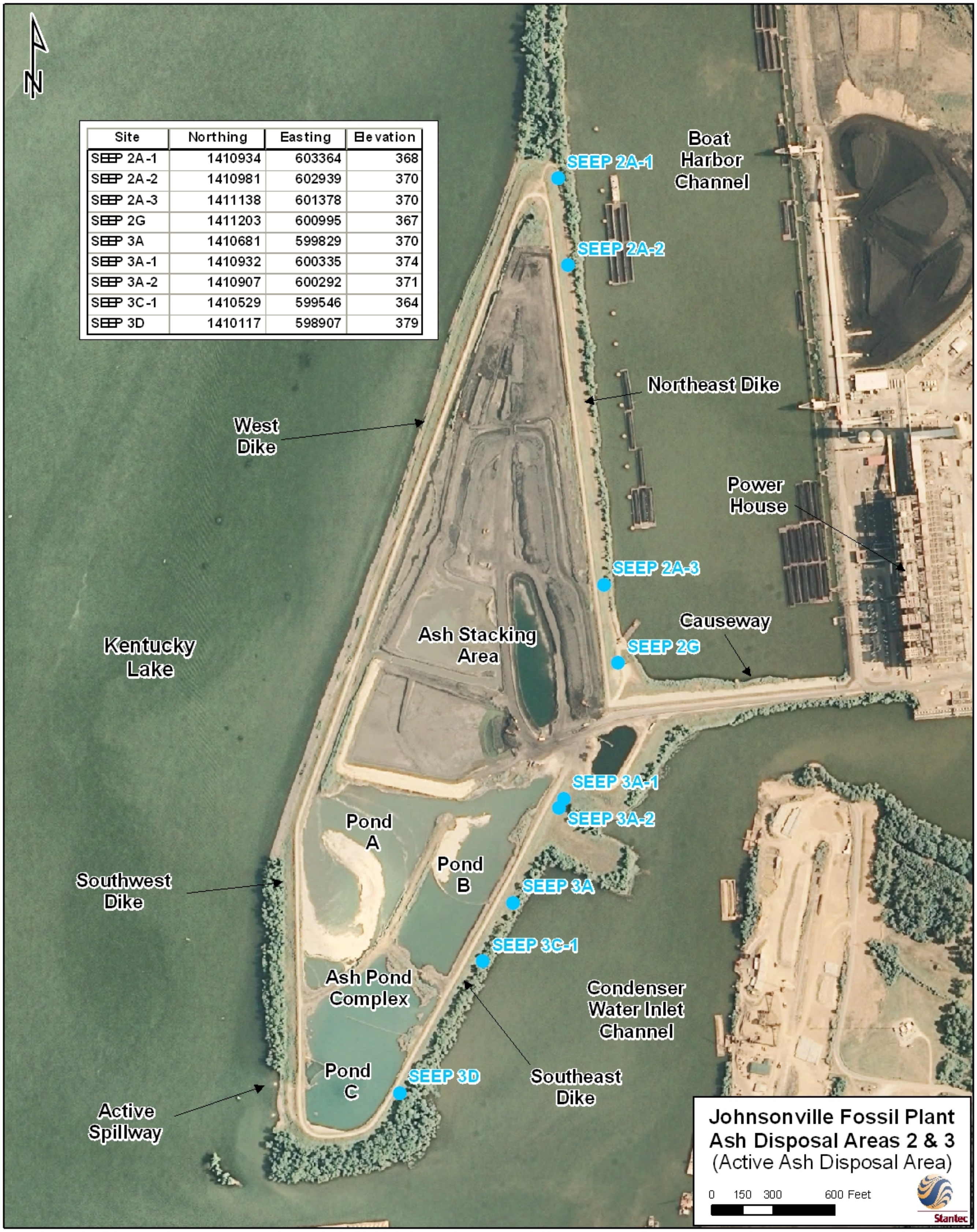
In December 2002 a localized (40-foot long) slide occurred on the east dike immediately north of the causeway. Earlier TVA Annual Dike Inspection Reports had noted a "bulge" in this area and attributed its formation to rock trucks hauling riprap for erosion repairs on the west side in the mid-1990's. Photographs taken by TVA staff indicate that the 2002 slide was probably shallow, and it was repaired using geotextile fabric, crushed stone, and riprap. This area has apparently been stable since being repaired.

Seepage has been documented by TVA on the northeast and southeast dikes beginning when the Ash Pond water level was raised above Elevation 370 feet in 1976. Since that time seepage has been noted at numerous points on the benches at Elevation 365 to 370 feet, and beneath the water level of Kentucky Lake. The 1995 TVA Annual Dike Inspection Report noted that seven seepage areas were present on the east dike north of the causeway and two areas existed on the southeast dike south of the causeway. One of the seepage areas, designated Seep 3A, is below the southeast dike approximately 1,000 feet southwest from the causeway. In 2008 this area was documented as having increased in size due to the raising of the water level within the Ash Pond by 2 feet. In February, 2009 TVA installed a system of perforated pipes enclosed in crushed stone and geotextile fabric. This collection system allows the seepage to be filtered and monitored for changes in flow rate and turbidity. During March, 2009 the average daily dry weather flow from Seep 3A was measured to be 3,500 GPD (2.4 GPM). This flow rate was relatively constant through the spring and summer of 2009.

Figure 2 is a plan of the active Ash Disposal Area that was prepared by TVA. It shows the locations of known seepage areas, their designations, and elevations.

3.3.2. Depressions on West Dike

Seven shallow depressions were observed during the Phase 1 site reconnaissance on the west dike exterior slope. The depressions begin approximately 1,500 feet south from the north end of the dike, and occur on intervals of about 200 to 400 feet for a distance of about 1,200 feet. They were mostly elongated (not circular) and measured up to 10 feet long and 18 inches deep. These depressions were generally located from about 12 feet in elevation below the crest of the slope. At the time of the Phase 1 site reconnaissance it was unknown as to their origin. Previous TVA inspection reports, beginning in 2003, indicated that depressions on the northwest dike were thought to be areas of subsidence resulting from the removal of trees on the slopes. Although the depressions did not appear to be sinkholes, the unknown nature of their origin caused them to be listed as a concern.



Site	Northing	Easting	Elevation
SEEP 2A-1	1410934	603364	368
SEEP 2A-2	1410981	602939	370
SEEP 2A-3	1411138	601378	370
SEEP 2G	1411203	600995	367
SEEP 3A	1410681	599829	370
SEEP 3A-1	1410932	600335	374
SEEP 3A-2	1410907	600292	371
SEEP 3C-1	1410529	599546	364
SEEP 3D	1410117	598907	379

Figure 2. Seepage Locations

3.3.3. Spillway Outlet

There are three sets of weir spillways (three risers and outlet pipes per set) at the Active Ash Disposal Area. They are located through the northwest dike, the southeast dike and at the southern end through the southwest dike. Until August, 2009 two of the southwest spillways were active and handling the 32 MGD flow that is directed into the ash disposal area.

According to TVA's drawings the structures were constructed using 36-inch diameter reinforced concrete pipe with bell and spigot joints, not restrained joints. The outlets are submerged beneath the Kentucky Lake pool level. This causes the pipes to air-surge on irregular intervals, with resulting pressure changes within the pipes. The concern is that joint separation and constant surging has resulted in soil piping and loss of soil material outside the pipe. The document reviews revealed that sinkholes have formed at two different times above the outlet pipes of the active spillways through the southwest dike. Repairs have consisted of grouting, sliplining, and backfilling with crushed rock. This concern exists for both active spillways, as well as spillways that have been taken out of service by raising the weir level above the normal pool elevation.

4. Scope of Exploration

The majority of the field geotechnical exploration was performed during the period from February 24 through April 8, 2009. These services were performed in general accordance with the Work Plan for Soil Borings and Laboratory Testing for Ash Disposal Areas 2 & 3 submitted to TVA dated February 20, 2009. The work plan describes the purpose for the different borings, the boring and sampling plan, and includes references to various Corp of Engineers procedures, guidelines, and standards that were followed. A copy of the work plan is included in Appendix A.

Thirteen stability sections (Sections A through M) were initially selected for drilling and analysis. At a later time Stantec added an additional stability section between Sections C and D. Designated Section C1, it was for the purpose of evaluating the effect of temporary ash stacking west of the sluice channel. The locations of the stability sections and borings are shown on the Boring Layout Plan in Appendix B. The sections and borings were initially staked by Stantec personnel prior to drilling. At completion of the drilling TVA's survey crew located the borings and profiled the groundlines at each of the slope stability sections.

In addition to the borings, Stantec also had excavated four inspection pits. These were dug on the west dike at four of the seven depressions referenced in Section 3. The pits were excavated by the ash handling contractor using a long reach trackhoe that operated from crest of the dike. Each pit was dug to a depth from 2.5 to 3 feet. The material removed was observed by a geotechnical engineer. The engineer also examined the bottom of the excavation and used a probe rod to check for voids within the dike material to a depth of about three or four feet below the excavation bottom. The locations of the test pits are shown on the Boring Layout Plan in Appendix B.

In August and September 2009, Stantec installed four slope inclinometers at Sections C, C1, E and K. The locations of the slope inclinometers and piezometers are shown on the Instrumentation Plan in Appendix B. Readings have been taken on monthly intervals.

In the laboratory, standard penetration test (SPT) samples were subjected to natural moisture content determination in accordance with ASTM D 2216. Selected SPT samples, representing the predominant soil layers were subjected to soil classification tests that included Atterberg limits testing (ASTM D 4318), specific gravity tests (ASTM D 854) and sieve and hydrometer analyses (ASTM D 422). Select bulk samples were also collected and subjected to standard moisture-density (Proctor) testing (ASTM D 698). Undisturbed samples were extruded and subjected to unit weight determination, unconfined compression testing (ASTM D 2166) and consolidated undrained triaxial compression testing with pore pressure measurements (ASTM D 4767).

5. Results of Geotechnical Exploration

5.1. Summary of Borings

The boring layout plan is contained in Appendix B and boring logs are presented in Appendix C. Detailed descriptions of the piezometer and slope inclinometer installations are provided in Appendices D and I, respectively. Results of laboratory tests are included in Appendix E. A summary of the boring elevation and depths is presented in Table 1 (all measurements are expressed in feet).

Table 5.1. Summary of Borings

Boring No.	Surface Elevation	Northing	Easting	Depth of Boring	Bottom of Boring Elev.
STN-AC	391.4	603148.82	1410894.84	61.5	329.9
STN-AT	368.4	603144.12	1410980.20	51.5	316.9
STN-BC	391.5	602313.93	1410981.18	61.5	330.0
STN-BT	369.8	602326.17	1411067.30	51.5	318.3
STN-CC	391.6	601437.52	1411070.75	61.5	330.1
STN-CCA	394.6	601382.49	1410633.59	49.0	345.6
STN-CT	368.9	601449.55	1411148.76	51.5	317.4
STN-C1C	391.5	601113.79	1411129.20	40.5	351.0
STN-C1CA	394.0	601054.92	1410641.77	51.5	342.5
STN-C1CB	398.4	601029.42	1410415.09	61.5	336.9
STN-C1T	365.5	601033.28	1411220.15	15.5	350.0
STN-DC	390.0	600191.17	1410774.31	61.5	328.5
STN-DT	365.3	600147.64	1410847.53	56.5	308.8
STN-EC	390.2	599528.35	1410416.19	61.5	328.7
STN-ECA	390.2	599528.35	1410416.19	33.0	357.2
STN-ET	363.8	599486.09	1410496.27	55.5	308.3
STN-FC	389.4	598898.88	1410062.79	61.5	327.9
STN-FT	362.9	598868.34	1410145.49	61.5	301.4
STN-GC	389.6	598719.43	1409736.38	61.5	328.1
STN-GT	360.8	598582.54	1409772.40	51.0	309.8
STN-HC	389.5	599345.93	1409646.07	61.5	328.0
STN-HM	377.9	599331.00	1409595.58	46.5	331.4
STN-HT	363.1	599308.41	1409545.23	51.5	311.6
STN-IC	389.8	600055.90	1409637.66	61.5	328.3

Table 5.1. Summary of Borings

Boring No.	Surface Elevation	Northing	Easting	Depth of Boring	Bottom of Boring Elev.
STN-IT	368.8	600103.14	1409560.28	51.5	317.3
STN-JC	389.6	600817.61	1409871.68	61.0	328.6
STN-JT	378.7	600838.26	1409820.33	51.5	327.2
STN-KC	389.8	601482.90	1410105.77	61.0	328.8
STN-KT	377.6	601488.26	1410056.92	51.5	326.1
STN-LC	389.9	602377.53	1410442.03	61.0	328.9
STN-LT	366.3	602392.94	1410352.26	51.5	314.8
STN-MC	390.6	603157.11	1410726.95	60.5	330.1
STN-MT	365.6	603187.15	1410653.44	51.5	314.1
STN-B-1	390.6*	---	---	31.5	359.1
STN-B-2	390.2*	---	---	31.5	358.7
STN-B-3	390.2*	---	---	31.5	358.7
STN-B-4	389.5*	---	---	31.5	358.0
STN-B-5	389.9*	---	---	31.5	358.4
STN-B-6	389.9*	---	---	31.5	358.4
STN-B-7	390.1*	---	---	31.5	358.6
STN-B-8	389.9*	---	---	31.5	358.4
STN-B-8A	389.9*	---	---	11.3	378.6
STN-B-9	389.7*	---	---	31.5	358.2
STN-B-10	389.1*	---	---	31.5	357.6
STN-B-11	389.6*	---	---	31.5	358.1
STN-B-12	Boring was initially planned to investigate divider dike. It was eliminated during the field work because the boring was not needed for the geotechnical evaluation.				
STN-B-13	390.1*	---	---	31.5	358.6
STN-B-14	367.3*	---	---	21.5	345.8
STN-B-15	378.9*	---	---	21.5	357.4
STN-B-16	389.6*	---	---	31.5	358.1
STN-B-17	389.1*	---	---	31.5	357.6
STN-B-18	391.0*	---	---	31.5	359.5
STN-B-19	388.3*	---	---	31.5	356.8
STN-B-20	388.9*	---	---	31.5	357.4
STN-B-21	389.2*	---	---	31.5	357.7
STN-SI-1	392.5	601441.71	1411095.09	115.0	277.5
STN-SI-1A	392.5	601436.97	1411096.68	80.0	312.5
STN-SI-2	391.7	601119.30	1411134.51	114.8	276.9
STN-SI-3	390.3	599513.18	1410433.95	120.0	270.3
STN-SI-4	390.4	601487.29	1410094.04	120.0	270.4

*Note: Boring elevations approximated from topographic information provided by TVA.

5.2. Subsurface Conditions

Using the boring logs and laboratory tests from this geotechnical exploration, the boring information contained in previous geotechnical studies at the facility, TVA design drawings, old contour maps and other historical information, Stantec developed a general profile for each of the stability sections at the Active Ash Disposal Area. The general profiles depict five horizons, or layers that are in the stratagraphic sequence of descending lithology as described below. The stability sections contained in Appendix H show these layers in graphical manner. In addition, the graphical logs shown on the stability sections also depict the material Unified Soil Classification System (USCS) classifications and results of natural moisture content, unconfined compression and penetration test blow counts.

The "Upper Clay Dike" extends from approximate Elevation 390 feet to Elevation 378 feet and consists of the final dike raising which occurred in 1978. The upper dike soils have a USCS classification of CL, with textural descriptions of lean clay, lean clay with sand, and lean clay with gravel. The soil was described as moist in moisture content and mostly brown in color. Based on SPT N-values and laboratory strength testing, the upper dike has strength consistencies ranging from medium stiff to very stiff.

The "Lower Clay Dike" extends from approximate Elevation 378 feet to Elevation 370 feet and consists of the dike raising which occurred in the early 1970s. The lower dike classified as CL and ML, with textural descriptions of lean clay, lean clay with sand, lean clay with gravel and silt according to the USCS classification system. The soil was described as mostly moist in moisture content with some isolated dry and wet zones encountered, and brown and gray in color. Based on SPT N-values and laboratory strength testing, the lower dike has strength consistencies ranging mostly from medium stiff to very stiff, with isolated zones of soft and very soft consistencies being encountered.

Below the lower dike, "Fill" material extending up to Elevation 370 feet was used to construct the initial pond prior to being brought into service in 1970. Based on a review of the historical information, some of the fill material was reportedly hydraulically placed during dredging of the nearby boat harbor while other portions were placed with equipment to construct the initial dike. The fill material classified as CL and ML, with textural descriptions of lean clay, sandy lean clay, lean clay with gravel, silt, and silt with sand according to the USCS classification system. The soil is described as moist to wet in moisture content, and brown and gray in color. In isolated samples, trace amounts of organic material were also encountered. A 10-foot layer of poorly graded gravel with sand was encountered at boring STN-CC within the fill horizon. Based on SPT N-values and laboratory strength testing, the fill material has strength consistencies ranging from very soft to medium stiff, with isolated zones of stiff and very stiff consistencies also being encountered. This broad range of strength consistencies and classifications correspond with the historical information which indicates that various borrow sources and methods were used to place this material.

Below the horizon of fill material, "Alluvial Clay and Silt" were encountered down to elevations ranging between about Elevation 320 feet and 334 feet. These materials have USCS classifications of CL and ML, with textural descriptions of lean clay, lean clay with sand, lean clay with gravel, sandy lean clay, silt, and silt with gravel. The soil was described as predominately wet in moisture content and brown and gray in color. Based on SPT N-values and laboratory strength testing, the alluvial clay and silt has strength consistencies ranging mostly from very soft to medium stiff, with isolated zones of stiff and very stiff consistencies also being encountered. A few thin lenses of sand were also encountered in this horizon.

Below the alluvial clay and silt horizon, "Alluvial Sand and Gravel" was encountered throughout the remainder of the borings. The alluvial sand and gravel had USCS classifications of SM, SP, SP-SM, SW, SW-SM, GP, GP-GM and textural descriptions of silty sand, silty sand with gravel, poorly graded sand with or without silt and gravel, well graded sand with or without silt and gravel, and poorly graded gravel with or without silt and sand. The sands and gravels were mostly described as wet in moisture content and brown in color. Based on SPT N-values, the alluvial sand and gravel has relative densities ranging mostly from medium dense to dense, with isolated thin zones of loose sands also being encountered in a few areas.

In addition to the soil horizons making up the dikes and foundations soils, ash materials were also encountered in 10 of the 14 (71%) crest borings advanced at the stability sections. The ash was normally encountered at a depth of about 15 feet, which indicated the bottom of the upper clay dike. Both bottom and fly ash were encountered at various locations. Classification testing performed on selected bottom ash samples resulted in USCS classifications of SM with a textural description of silty sand with gravel. The bottom ash was generally described as mostly black in color and moist to wet in moisture content. Classification testing performed on a selected fly ash sample resulted in a USCS classification of ML with a textural description of silt. Where encountered, the fly ash was generally described as being gray in color and wet in moisture content.

The subsurface logs presented in Appendix C include more detailed descriptions of the soils encountered at the specific boring locations.

5.3. Phreatic Conditions

Thirty two piezometers were installed at soil sample boring locations to measure pore water pressures. Refer to Appendix D for piezometer installation details and readings (up to most recent set of readings). Piezometer locations and tip elevations are summarized in Table 5.2. below.

Table 5.2. Summary of Piezometers

Boring No.	Concrete Pad Elevation (Feet)	Piezometer Tip Elevation (Feet)
STN-AC-PZ	391.6	366.6 (upper / lower clay dike)
STN-AC-PZ(2)	391.8	367.3 (upper / lower clay dike)
STN-AT-PZ	368.4	348.4 (fill)
STN-BC-PZ	392.4	367.4 (upper / lower clay dike)
STN-BT-PZ	369.8	337.8 (fill / alluvial clay and silt)
STN-CC-PZ	392.5	367.5 (upper / lower clay dike)
STN-CC-PZ(2)	392.4	368.4 (upper / lower clay dike)
STN-CT-PZ	368.9	343.7 (fill / alluvial clay and silt)
STN-C1C-PZ	391.5	361.5 (lower clay dike / fill)
STN-C1C-PZ(2)	392.4	368.4 (upper / lower clay dike)
STN-C1T-PZ	365.5	351.5 (fill)
STN-DC-PZ	391.2	366.2 (upper / lower clay dike)
STN-DC-PZ(2)	390.9	367.0 (upper / lower clay dike)

Table 5.2. Summary of Piezometers

Boring No.	Concrete Pad Elevation (Feet)	Piezometer Tip Elevation (Feet)
STN-DT-PZ	365.3	339.9 (alluvial clay and silt)
STN-EC-PZ	390.4	365.4 (upper / lower clay dike, ash)
STN-ET-PZ	363.8	329.8 (alluvial clay and sand / alluvial sand and gravel)
STN-FC-PZ	389.8	364.8 (upper / lower clay dike / ash)
STN-FT-PZ	362.9	327.6 (alluvial clay and sand / alluvial sand and gravel)
STN-GC-PZ	389.8	364.8 (ash / lower clay dike)
STN-GT-PZ	360.8	330.4 (fill)
STN-HC-PZ	390.0	365.8 (upper clay dike / ash)
STN-HT-PZ	363.1	316.1 (alluvial sand and gravel)
STN-IC-PZ	390.1	360.1 (ash / lower clay dike)
STN-IT-PZ	368.8	344.8 (fill / alluvial clay and silt)
STN-JC-PZ	390.0	365.0 (ash)
STN-JT-PZ	378.7	344.7 (fill)
STN-KC-PZ	390.5	365.5 (ash)
STN-KT-PZ	377.6	342.6 (fill / alluvial clay and silt)
STN-LC-PZ	390.5	365.5 (ash)
STN-LT-PZ	366.3	337.1 (alluvial clay and silt)
STN-MC-PZ	391.1	366.1 (upper clay dike / ash)
STN-MT-PZ	365.6	333.8 (fill / alluvial clay and silt)

*Note: Piezometer elevations shown for the crest borings are indicative of the top of the flush mount cover. Elevations shown for the toe borings are indicative of the existing ground surface adjacent to the piezometer.

5.4. Depressions on West Dike

As described in Section 4, four shallow inspection pits were excavated near the northern end of the northwest dike at locations where the ground displayed shallow (18 inch deep) depressions. The depressions were located about 40 feet down the slope, as measured from the exterior crest, and at approximate Elevation 376 feet. The depths of excavations were from 2.5 to 3 feet and the bottoms were probed for an additional depth of 3.5 feet. The material encountered at each of the excavations was clay. It was found to be medium to soft in consistency and very moist to saturated in natural moisture content. This material did not appear to have been placed in lifts or compacted in the same manner as the soils encountered in Stantec's borings through the dike crest. In addition, black, carbonaceous vegetative debris, that indicated remnants of cut trees, buried tree parts, and/or decayed root systems were encountered in each of the inspection pits.

Based on Stantec's observations and its reviews of old inspection reports, the depressions resulted from voids created when trees were removed from the dike, when buried tree parts rotted or when loosely placed clay was used to backfill and grade the bench at the former crest of the dike. The loose material to grade the bench may have been placed to repair a drill rig access route created for a groundwater monitoring well installed during 1986. It may also have been placed when riprap was installed to control erosion at the toe of the west dike in 1996. In any event, Stantec did not find evidence that indicated the depressions were sinkholes created by internal erosion through the dike.

6. Laboratory Testing

6.1. General

The results of laboratory testing performed are included within the appendices. ASTM testing specifications were observed. In particular, natural moisture content test results are shown on the boring logs in Appendix C and are also shown on the stability sections in Appendix H. The results of the classification testing performed on selected samples are included in Appendix E. The USCS classifications associated with each horizon are also discussed in Section 5.2 above. No further discussion relative to the results of moisture content and classification testing are provided in this section. The discussion that follows is limited to the laboratory testing associated with evaluation of the dike compaction characteristics and laboratory strength test results for the cohesive soil horizons.

6.2. Moisture - Density Relationships

Several bag samples were obtained of materials associated with the upper clay dike. The results of the standard moisture-density tests performed on these samples are summarized in Table 6.1.

Table 6.1. Standard Moisture-Density (Proctor) Test Results

Sample Location	Sample Depth Interval (feet)	Dike Location	Maximum Dry Density (pcf)	Optimum Moisture Content (%)
STN-BC	2.0 – 8.0	NE	112.5	15.6
STN-EC	5.0 – 10.0	SE	108.8	17.9
STN-HC	3.0 – 10.0	SW	109.1	16.0
STN-KC	3.0 – 10.0	NW	109.3	15.6

Following completion of the moisture-density testing, undisturbed samples were extruded and unit weight and moisture content determinations of the samples were performed. The results of the unit weight and moisture content determinations are shown in Table 6.2. A comparison between the moisture-density test results and the unit weight determinations obtained from the undisturbed samples are also included. The comparison was made by using the moisture density results that were nearest to the undisturbed sample location to estimate relative compaction.

Table 6.2. Comparison Between Undisturbed Tube Samples and Moisture-Density Test Results

Boring Location	Sample Depth Interval (feet)	Dike Location	Unit Weight Dry (pcf)	Moisture Content (%)	Maximum Dry Density (pcf)	Percent Maximum Dry Density (%)	Optimum Moisture Content (%)	Moisture Content Variation (%)
STN-AC-PZ	5.0-5.5	NE	111.2	17.7	112.5	98.8	15.6	2.1
STN-BC-PZ	5.0-5.5	NE	110.0	19.1	112.5	97.8	15.6	3.5
STN-BC-PZ	10.0-10.5	NE	108.2	18.4	112.5	96.2	15.6	2.8
STN-B-8	10.0-10.6	SW	106.1	17.6	109.1	97.3	16.0	1.6
STN-B-8	10.6-11.1	SW	105.8	20.3	109.1	97.0	16.0	4.3
STN-CC-PZ	10.3-10.8	NE	106.9	19.4	112.5	95.0	15.6	3.8
STN-DC-PZ	5.1-5.6	SE	109.8	17.6	108.8	100.9	17.9	(-0.3)
STN-DC-PZ	10.0-10.5	SE	109.3	18.7	108.8	100.5	17.9	0.8
STN-EC-PZ	5.0-5.5	SE	108.2	18.1	108.8	99.4	17.9	0.2
STN-EC-PZ	5.6-6.1	SE	108.7	19.8	108.8	99.9	17.9	1.9
STN-FC-PZ	5.1-5.6	SE	109.7	16.7	108.8	100.8	17.9	(-1.2)
STN-FC-PZ	5.7-6.2	SE	109.5	18.4	108.8	100.6	17.9	0.5
STN-FC-PZ	10.1-10.6	SE	105.5	18.3	108.8	97.0	17.9	0.4
STN-GC-PZ	5.1-5.6	SW	106.3	17.7	109.1	97.4	16.0	1.7
STN-GC-PZ	5.7-6.2	SW	108.5	18.8	109.1	99.5	16.0	2.8
STN-HC-PZ	5.1-5.6	SW	108.4	19.9	109.1	99.4	16.0	3.9
STN-IC-PZ	10.2-10.7	SW	108.0	19.7	109.1	99.0	16.0	3.7
STN-JC-PZ	5.1-5.6	NW	107.1	18.8	109.3	98.0	15.6	3.2
STN-JC-PZ	10.2-10.7	NW	107.3	20.2	109.3	98.2	15.6	4.6
STN-KC-PZ	5.1-5.6	NW	108.9	17.0	109.3	99.6	15.6	1.4
STN-KC-PZ	10.1-10.6	NW	105.8	20.3	109.3	96.8	15.6	4.7
STN-LC-PZ	5.1-5.6	NW	108.2	19.6	109.3	99.0	15.6	4.0
STN-LC-PZ	10.5-11.0	NW	106.4	22.4	109.3	97.3	15.6	6.8
STN-MC-PZ	5.1-5.6	NW	107.9	18.9	109.3	98.7	15.6	3.3
STN-MC-PZ	10.2-10.7	NW	116.9	12.3	109.3	107.0	15.6	(-3.3)

The existing in-situ dry densities of the upper dike were determined to range from about 95 percent to values of 100 percent or greater of the standard Proctor dry densities. This data indicates that the dike material appears to have been compacted in a controlled manner when compared to typically accepted target densities of 95 percent or greater for compacted clay soils in an earth dike. However, it should be noted that no construction documentation has been provided to date to confirm this comparison. The corresponding moisture values were typically about 2 to 4 percent above the optimum moisture value. This is likely attributed to the dike material being saturated by long term seepage through the dike.

6.3. Undisturbed (Shelby) Tube Samples

The borings drilled for Ash Disposal Area 2 and 3 included 58 undisturbed (Shelby) tube samples that were obtained within cohesive soil horizons. Stantec's soils laboratory extruded the tubes and trimmed 6-inch long specimens. Laboratory technicians determined visual classifications, unit weights (wet and dry), and natural moisture for each 6-inch specimen prior to submitting a summary of the extruded specimens to a geotechnical engineer for assignment of testing. Select 6-inch specimens extruded from Shelby tubes were then subjected to consolidated-undrained (CU) triaxial testing and unconfined compressive strength testing. The results of these tests are included in Appendix E and discussed below.

6.3.1. Consolidated Undrained (CU) Triaxial Testing

Stantec performed CU triaxial testing with pore pressure measurements on selected 6-inch long specimens extruded from 3-inch diameter Shelby tubes. CU testing provides indicators of effective-stress shear strength parameters for slope stability analyses. The results of the CU triaxial tests are presented on the stability sections in Appendix H, and are summarized in Table 6.3. The stress path envelopes derived from CU triaxial testing are also presented in Appendix E.

Table 6.3. Summary of Consolidated Undrained Triaxial Tests

Boring No.	Sample Depth Interval (feet)	Dike Location	Soil Horizon	Effective	
				c' (psf)	Ø' (deg.)
STN-AC-PZ	5.0 – 5.5	NE	Upper Dike	1,260	22.9
STN-BC-PZ	5.0 – 5.5				
	10.0 – 10.5				
STN-AT-PZ	19.0 – 19.5	NE	Fill	560	31.4
	19.6 – 20.1				
	20.2 – 20.7				
STN-AC	45.6 – 46.1	NE	Alluvial Clay and Silt	260	31.9
	46.2 – 46.7				
	46.8 – 47.3				
STN-FC-PZ	5.1 – 5.6	SE	Upper Dike	500	39.8
	5.7 – 6.2				
STN-GC-PZ	5.7 – 6.2				
STN-B-1	22.7 – 23.2	SE	Lower Dike	440	28.3
STN-BC	33.0 – 33.5				
STN-EC	31.7 – 32.2				
STN-B-3	20.5 – 21.0	SE	Lower Dike	860	28.6
	21.1 – 21.6				
STN-FC	35.1 – 35.6	SE	Alluvial Clay and Silt	340	31.4
	44.5 – 45.0				
	45.1 – 45.6				
STN-GC-PZ	5.1 – 5.6	SW	Upper Dike	900	25.6
STN-HC-PZ	5.1 – 5.6				
STN-IC-PZ	10.2 – 10.7				
STN-GC	22.6 – 23.1	SW	Lower Dike	320	30.9
STN-IC-PZ	30.1 – 30.6				
STN-IT-PZ	10.1 – 10.6	SW	Fill	840	28.5
	10.7 – 11.2				
STN-FC	19.6 – 20.1				
STN-GT	12.0 – 12.5	SW	Alluvial Clay and Silt	440	29.4
	12.5 – 13.0				
	13.0 – 13.5				

Table 6.3. Summary of Consolidated Undrained Triaxial Tests

<table>
<tr><th rowspan="2">Boring No.</th><th rowspan="2">Sample Depth Interval (feet)</th><th rowspan="2">Dike Location</th><th rowspan="2">Soil Horizon</th><th colspan="2">Effective</th></tr>
<tr><th>c’ (psf)</th><th>Ø’ (deg.)</th></tr>
<tr><td>STN-JC-PZ</td><td>5.1 – 5.6</td><td rowspan="3">NW</td><td rowspan="3">Upper Dike</td><td rowspan="3">240</td><td rowspan="3">33.6</td></tr>
<tr><td rowspan="2">STN-KC-PZ</td><td>5.1 – 5.6</td></tr>
<tr><td>10.1 – 10.6</td></tr>
<tr><td rowspan="2">STN-LC</td><td>28.8 – 29.3</td><td rowspan="3">NW</td><td rowspan="3">Lower Dike</td><td rowspan="3">400</td><td rowspan="3">30.7</td></tr>
<tr><td>29.4 – 29.9</td></tr>
<tr><td>STN-MC</td><td>28.9 – 29.4</td></tr>
<tr><td rowspan="3">STN-KT-PZ</td><td>12.0 – 12.5</td><td rowspan="3">NW</td><td rowspan="3">Fill</td><td rowspan="3">400</td><td rowspan="3">33.0</td></tr>
<tr><td>12.5 – 13.0</td></tr>
<tr><td>13.0 – 13.5</td></tr>
</table>

6.3.2. Unconfined Compressive Strength Testing

Stantec performed a limited number of unconfined compressive strength tests on specimens extruded from 3-inch diameter Shelby tubes. The tests provided quantitative strength comparisons for the numerous Standard Penetration Tests Stantec performed. The values obtained allowed Stantec to validate the soil consistency descriptions used in developing boring logs. The results of these tests are presented in Appendix E and in Table 6.4 below.

Table 6.4. Summary of Unconfined Compressive Strength Tests

Boring No.	Sample Depth Interval (feet)	Dike Location	Soil Horizon	Unconfined Compressive Strength (psf)
STN-B-8	10.0 – 10.6	SW	Upper Dike	6,020
STN-DT	18.5 – 19.0	SE	Alluvial Clay and Silt	3,000
STN-ET	6.0 – 6.5	SE	Fill	4,300
STN-ET	13.1 – 13.6	SE	Alluvial Clay and Silt	2,300
STN-HM	6.0 – 6.5	SW	Lower Dike	4,600
STN-JC	28.6 – 29.1	NW	Lower Dike	4,520

7. Engineering Analyses

7.1. General

Geotechnical engineering analyses included evaluations of strength and permeability parameters, seepage analyses, and slope stability analyses. Prior to beginning the analyses, the geotechnical data developed by Stantec and cross-sections provided by TVA were combined and the geometry of the existing dikes and soil horizons were approximated using current and historical information.

The following TVA drawings were also used to develop the internal geometry for the cross sections analyzed.

- Ash Disposal Area, West of Boat Harbor, Tennessee Valley Authority (TVA), Drawing Number 10W527 Revision 15, Revision Dated February 28, 1997
- Slope Stability Analysis for JOF Ash Pond New Cell, Purdue University, October 7, 2002
- Ash Disposal Area, Sections, TVA, Drawing Number 10W529 R6, Revision Dated October 4, 1995
- Seepage Through Ash Dike at Boat Harbor, TVA, J.L. Glover Sketch Number JLG92076, September 20, 1976
- Johnsonville Steam Plant – Ash Disposal Pond, TVA, J.R. Parrish, October 9, 1967
- Ash Disposal Studies Area #2, TVA, Initialed E.B.L., July 11, 1967
- Johnsonville Fossil Plant-Upgrade Ash Pond Dike Erosion Control Plan (PCN JOF118 – Lessons Learned, TVA, Jerry L. Glover, June 17, 1997

Once the geometry of the sections was approximated, each section was reviewed and evaluated to determine the critical cross-section for analyses. Selection of critical sections was based on the steepness of slopes, heights of dikes, geometry of the sections, phreatic surface, seepage conditions, and subsurface conditions. Based on this evaluation, nine representative cross-sections were selected for analyses (Sections A, B, C, C-1, E, F, I, K and M). The locations of the sections are shown on the layout drawing presented in Appendix B. Results of the analyses and evaluations are summarized in the following paragraphs, and are shown on drawings/computer output provided in Appendices G and H.

It should be noted that construction records indicating the methods used to construct dikes, as-built dike configurations, etc. were not available for review. As a result, assumptions and generalizations in soil parameters and dike geometry were needed to construct the seepage and stability models.

7.2. Soil Horizons

Based on the results of the drilling, laboratory testing, historical documentation, and drawings, the materials on site were divided into five different soil layers for seepage and stability analyses. Refer to the stability sections in Appendix H for locations of the soil horizons. The soil horizons are briefly described as follows (refer to Section 5.2 for further descriptions):

- Alluvial Sand and Gravel: This represents the layer of alluvial sand and gravel that directly overlies bedrock in the Tennessee River floodplain. It is on the order of 40 to 50 feet in thickness and extends upwardly from bedrock to an average Elevation 330 feet.
- Alluvial Clay and Silt: This layer overlies the alluvial sand and gravel. It is about 15 to 25 feet in thickness.

- Fill: This layer is the zone of heterogeneous materials that was initially placed to form a perimeter dike and elevate it above the level of Kentucky Lake. It consists of clay, silt, sand and gravel. The outslopes are variable, relatively flat, and they extend to Elevation 370 feet.

- Lower Clay Dike: This is the first dike placement that employed compacted clay. It was constructed to an approximate crest at Elevation 378 feet. Based on TVA's drawings, the interior slopes are assumed to be from 2H:1V to 3H:1V.

- Upper Clay Dike: This represents the material used for the 1978 construction of the raised dike to its current crest at approximate Elevation 390 feet. Based on TVA's drawings, the interior slopes are assumed to be 2H:1V.

- Hydraulically Placed (sluiced) Ash: This represents sluiced bottom ash/fly ash that is contained by the upper and lower dikes.

7.3 Seepage Analysis

7.3.1. SEEP/W Model

An analysis of steady state seepage through the Active Ash Area dikes was performed to estimate the magnitude of seepage gradients (for the evaluation of potential piping) and pore water pressures within the soils (for the evaluation of slope stability). The numerical seepage models were developed using SEEP/W 2007 (Version 7.15), a finite element code tailored for modeling groundwater seepage in soil and rock. SEEP/W is distributed by GEO-SLOPE International, Ltd, of Calgary, Alberta, Canada (www.geo-slope.com).

SEEP/W utilizes soil properties, site geometry, and boundary conditions provided by the user to compute the total hydraulic head at nodal points within the modeled cross-section. Among other features, SEEP/W includes a graphical user interface, semi-automated mesh generation routines, iterative algorithms for solving unconfined flow problems, specialized boundary conditions (seepage faces, etc.), capabilities for steady-state or transient analyses, and features for visualizing model predictions. The code also includes material models that allow tracking both saturated and unsaturated flow, including the transition in seepage characteristics for soils that become saturated or unsaturated during the problem simulation.

Nine (9) cross-sections through the Active Ash Area were modeled with SEEP/W, and then were subsequently evaluated for slope stability (Section 7.4). For the numerical analysis, each cross-section was subdivided into a mesh of elements, consisting of first-order quadrilateral and triangular finite elements. For seepage problems, where the primary unknown (hydraulic head) is a scalar quantity, first-order elements provide for efficient, effective modeling. Given appropriate hydraulic conductivity properties and applied boundary conditions, the finite element method (as implemented in the SEEP/W code) was then used to simulate steady seepage across the mesh. The total hydraulic head is computed at each nodal location, from which pore water pressures and seepage gradients can be determined.

7.3.2. Boundary Conditions

Steady-state seepage was assumed for the analysis, with the static pool level placed at the Ash Pond complex, or adjacent sluice channel water level, as appropriate for the section analyzed (based on TVA provided survey data).

Boundary conditions for the SEEP/W analysis were assumed as follows. Along the vertical, downstream edge of the model, the hydraulic head at each boundary node was constant with depth and assigned a value equal to the summer Kentucky Lake pool elevation The vertical, upstream edge of the model is located along the longitudinal center-line of the Active Ash Disposal Area island and modeled as a no-flow boundary (i.e., Q=0). The basis for this assumption is that the pond water would take the shortest path to the perimeter dike, and that the center-line is the dividing line for the direction of flow, hence a no flow boundary is considered appropriate. A total head value equal to the pool level was applied to all submerged nodes along the ground surface of the upstream side (submerged sluiced ash and interior upper dike). Other nodes along the ground surface were treated as potential seepage exits. At various steps in the computer analysis, if the software determines that water flows from the mesh at these nodes along the ground surface, SEEP/W assigned a head equal to the elevation of the node. This routine effectively models the seepage exit to the ground surface. The horizontal boundary at the base of the model (bedrock surface) was modeled as a seepage barrier, with no vertical flow across the boundary nodes.

7.3.3. Seepage Properties

For each modeled cross-section, a representative subsurface profile was compiled based on boring logs, available record drawings, and the known project history. Material properties were estimated based on available laboratory data, correlations with classification data, and on typical values for similar materials. Material properties used in the seepage analysis are summarized in Table 7.1.

Table 7.1. Summary of Soil Parameters for Seepage Analyses

Soil Horizon	k_v (cm/s) range	k_h / k_v range	G_s	Void Ratio e	Volumetric Water Content	
					Saturated (%)	Residual (%)
Upper Clay Dike	1.0e-06 to 3.0e-06	1 to 10	2.66	0.51	34	2
Lower Clay Dike	1.0e-06 to 2.0e-03	1 to 10	2.66	0.51	34	2
Fill	1.0e-05 to 1.0e-04	3 to 5	2.73	0.42	30	2
Alluvial Clay and Silt	1.0e-06 to 1.0e-05	20	2.64	0.65	39	2
Alluvial Sand and Gravel	1.0e-02	20	2.68	0.34	25	1
Ash	1.0e-04 to 2.0e-04	5 to 10	2.43	0.68	41	3
Riprap*	1.0	1	2.60	1.66	62	0

*Note: Riprap is present along the toe of the slope along the northwest portion of the pond as a result of dike repairs performed in 1994 and 1997.

Significant engineering judgment is needed to select appropriate hydraulic properties for earth/soil materials. Unlike other key properties, hydraulic conductivity can vary over several orders of magnitude for a range of soils, often with substantial anisotropy for seepage in horizontal versus vertical directions. Laboratory test samples often do not represent important variations within a larger soil deposit. An iterative process of parametric calibration (Section 7.3.4) was used to arrive at final estimates of the seepage properties. Results from trial simulations were compared to field data (measured piezometric levels and observed seepage) and the material parameters were then varied until the solutions reasonably matched the field data. The final set of parameters (Table 7.1) resulted in the comparisons presented in Section 7.3.4.

The ratio of horizontal hydraulic conductivity (kh) to vertical hydraulic conductivity (kv) was estimated based on placement, depositional characteristics, and origin of the materials. An isotropic material would have $k_h/k_v = 1$, while deposits of horizontally layered soils will have much higher values. In general, higher ratios were used for alluvial soil deposits, than for the compacted dike materials.

The governing equations in SEEP/W are formulated to consider seepage through unsaturated soils. In the SEEP/W simulations, this formulation is used to locate the phreatic surface for unconfined seepage through the dike cross-sections. To represent the change in hydraulic conductivity due to de-saturation of each soil, SEEP/W implements a model based on two curves, a hydraulic conductivity function and a volumetric water content function. Three parameters are needed to define this behavior: the saturated hydraulic conductivity, saturated water content, and residual water content (water content of air dried soil). Of these, only the residual water contents were not previously estimated for each material. Values were estimated based on typical values for similar soils. The simulation results are not sensitive to the selection of these values.

7.3.4. Comparison to Field Observations

After the initial seepage parameters were estimated, results from the SEEP/W model were compared to groundwater levels measured in piezometers installed within the Active Ash Disposal Area dikes. Data from the piezometers were used in this evaluation. Nodes were placed in the model at the same location as the piezometer tip was installed in the field so that the total head predicted at the node could be compared to the corresponding piezometer reading. The material properties in each modeled cross-section were then varied until a reasonable match was obtained between the seepage predictions and field data. Specifically, the saturated hydraulic conductivity and the k_h/k_v ratios were adjusted (while still maintaining the parameters within expected ranges) to give model predictions as consistent as possible with field measurements and observations.

The comparison between the field piezometer measurements and final SEEP/W predictions show the predicted groundwater table ranging from about 1 foot below to 7 feet above the readings obtained in the piezometers installed within the dike crest. For the dike toe areas, the seepage model consistently predicted the water table position to be from 3 feet below to 3 feet above actual toe piezometer readings.

The results from the seepage model can also be compared to field observations of seepage. For the Active Ash Disposal Area, historical seepage has been present along the majority of the northeast and southeast dikes. These observations correlate well with the seepage models for the cross-sections which generally show the shape of the predicted phreatic surface extending to the slope face.

In summary, the seepage models appear to give a reasonable prediction of the phreatic surface location when compared to field observations and piezometer measurements.

7.3.5. Critical Exit Gradients

Seepage forces, resulting from hydrodynamic drag on the soil particles, can destabilize earthen structures. Excessive hydraulic gradients near the ground surface can lead to the initiation of soil erosion and piping, which has caused numerous dam failures in the past. Hydraulic gradients (computed where seepage exits at the ground surface) can be evaluated to understand the potential severity of this problem.

Where upward seepage through a uniform soil exits the ground surface, the factor of safety with respect to soil piping (FSpiping) is as defined below.

$$FS_{piping} = \frac{i_{crit}}{i} \qquad \textbf{Eqn. 7.1}$$

Where “i” is the vertical gradient in the soil at the exit point. The critical gradient (icrit) is related to the submerged unit weight of the soil, and can be computed as:

$$i_{crit} = \frac{\gamma_{sub}}{\gamma_w} = \frac{G_s - 1}{1 + e} \qquad \textbf{Eqn. 7.2}$$

where γ_{sub} is the submerged unit weight of the soil, γ_w is the unit weight of water, G_s is the specific gravity of the soil particles, and e is the void ratio. For nearly all soils, the critical gradient is between about 0.6 and 1.4, with a typical value near 1.

When FS_{piping} = 1, the effective stress is zero and the near-surface soils are subject to piping or heaving, but only for vertical seepage that actually exits to the ground surface. If the phreatic surface is buried, then the FS_{piping} will be greater than 1 even when $i=i_{crit}$.

7.3.6. Results of Seepage Analysis

Plots from the SEEP/W analyses of the nine cross-sections through the Active Ash Area dikes are presented in Appendix G. The plots show the finite element mesh, material zones, and boundary conditions used in each analysis. The results are depicted in contour plots of total head, pore water pressure, and seepage gradients. For the slope stability analyses (Section 7.4), the pore water pressures along the considered slip surfaces were determined by interpolation between the nodal pore pressures predicted with the SEEP/W model. The seepage gradients were assessed for maximum exit gradients and the potential for soil piping.

On each modeled cross-section, examination of the graphical output (predicted phreatic surface and vertical gradients) can be used to determine where the potential for excessive vertical gradients might exist that could possibly initiate the erosion or piping of material. In general, areas of potential concern are where water seeps laterally out onto a sloping ground surface, or where vertical, upward seepage occurs at the near horizontal ground surface. The potential for piping was evaluated using the factor of safety equation as defined in Section 7.3.5. First, contour plots of vertical gradient (Appendix G) were examined to determine the general location of the maximum vertical exit gradient. On the modeled cross-sections, the maximum upward gradients occur near or beyond the toe of the lower dikes. For the factor of safety calculations, vertical gradients from these locations were then used along with the critical gradients determined from the soil properties.

The calculated factors of safety against piping are summarized in Table 7.2. Stantec recommends a target factor of safety against piping of 4, based on information contained in United States Army Corps of Engineers (USACE) manual EM 1110-2-1901. Hence, on four cross sections modeled (A,B,C and I), the Active Ash Disposal Area does not meet the recommended criteria for piping at the critical seepage exit points located at or beyond the dike toes.

Table 7.2. Summary of Computed Exit Gradients and Minimum Factors of Safety against Piping

Cross Section*	Vertical Gradient (i_y) at Critical Exit Point	Location of Critical Exit Point	Material	Critical Gradient (i_{crit})	FS_{piping}
A	0.46	Toe of first bench	Fill	1.22	2.7
B	0.49	Toe just below waterline	Fill	1.22	2.5
C	0.40	Toe at waterline	Fill	1.22	3.0
C1	0.215	Toe of first bench (just below Lower Dike)	Fill	1.22	5.7
E	0.17	Toe at waterline	Fill	1.22	7.1
F	0.12	Toe of dikes	Fill	1.22	10.4
I	0.28	Approximately 12 feet from shoreline	Alluvial Clay and Silt	1.00	3.6
K	0.11	Approximately 12 feet from shoreline	Alluvial Clay and Silt	1.00	8.8
M	0.04	Below toe of riprap	Fill	1.22	33.9

7.4. Slope Stability Analyses

7.4.1. SLOPE/W Model

The stability of the Active Ash Area dikes were evaluated using limit equilibrium methods as implemented in the SLOPE/W software, which is available from GEO-SLOPE International, Ltd., of Calgary, Alberta, Canada (www.geo-slope.com). Analyses were completed for static loading and long-term steady-state seepage conditions. SLOPE/W is a special-purpose computer code designed to analyze the stability of earth slopes using two-dimensional, limit equilibrium methods. With SLOPE/W, the distribution of pore water pressures within the earth mass can be mapped directly from a SEEP/W solution. In this study, steady-state pore pressures were obtained from the SEEP/W models described in Section 7.3.

7.4.2. Limit Equilibrium Methods in SLOPE/W

Limit equilibrium methods for evaluating slope stability consider the static equilibrium of a soil mass above a potential failure surface. For conventional, two-dimensional methods of analysis; the slide mass above an assumed failure surface is first divided into vertical slices, then stresses are evaluated along the sides and base of each slice. The factor of safety against a slope failure (FSslope) is defined as:

$$FS_{slope} = \frac{\text{shear strength of soil}}{\text{shear stress required for equilibrium}} \qquad \textbf{Eqn. 7.3}$$

where the strengths and stresses are computed along a defined failure surface located at the base of the vertical slices. The shearing resistance along the potential slip surface is computed, with appropriate Mohr-Coulomb strength parameters, as a function of the effective normal stress.

Spencer's solution procedure (Spencer 1967; USACE 2003; Duncan and Wright 2005), which satisfies all of the conditions of equilibrium for each slice, was used in this study. Spencer's procedure computes FS_{slope} for an assumed failure surface. A search must be made to find the critical slip surface corresponding to the lowest FS_{slope}. Both circular and noncircular potential failure surfaces can be evaluated.

7.4.3. Analysis Approach

The slope stability analyses were performed using SLOPE/W 2007 on the downstream (exterior) faces of the dikes. SLOPE/W incorporates various search routines to locate the critical slip surface. For the analyses presented here, "Entrance and Exit" method was employed. The distribution of pore water pressures obtained from the SEEP/W model was used in the analysis.

7.4.4. Selection of Shear Strength Parameters

The lower dikes for the Active Ash Disposal Area were originally constructed in 1970, and the upper dikes were constructed in 1978. These dikes have existed in their current cross sectional geometry (slopes and crest elevation) for at least 40 years. Hence, excess pore pressures generated in the underlying soil during construction have had sufficient time to dissipate and steady state seepage conditions have developed within the dikes. The stability analyses presented in this report focus only on static steady state seepage conditions (no earthquake or other dynamic loads). For these conditions, soil unit weights and drained strength parameters (c' and ϕ') are needed.

The drained shear strength parameters used for the clay dikes, fill, and alluvial foundation materials were derived using results of laboratory triaxial tests, along with consideration given to standard penetration test data and laboratory classification test data. In addition, the strength parameters selected were further refined or confirmed by comparisons with the strength parameters listed in the TVA-provided historical reports. Representative strengths for each horizon were selected using the methodology outlined in the US Army Corps of Engineers Engineer Manual EM 1110-2-1902 as a guide. Results of triaxial testing were evaluated and effective stress p' versus q scatter plots were prepared of all of the data points. The maximum effective principal stress ratio was used to determine failure criteria for selection of these values within Stantec's laboratory test results. Once the p' versus q plots were prepared, a failure envelope was then selected such that (minimum) two thirds of the plotted values were on or above the envelope. The p' versus q plots and selection of the failure envelope are shown for each horizon on the graphs presented in Appendix F. The strength parameters were rounded to the nearest degree with regards to ϕ'. The cohesion intercept point (c') was limited to a maximum of 200 pounds per square foot.

For non-cohesive alluvial sands and gravels, shear strength parameters were estimated using published relationships which correlate SPT N-values with relative density, specific soil types and angles of internal friction.

In addition to the dike and foundation soils, sluiced ash materials are present within the Active Disposal Area. Shear strength parameters for ash materials were estimated using historical data, typical values, and published correlations using SPT N-values.

The following table provides a summary of the effective stress shear strength parameters selected for use in the slope stability analyses.

Table 7.3. Shear Strength Parameters for Stability Analysis

Soil Horizon	Unit Weight (pcf)	Effective Stress Strength Parameters	
		c' (psf)	Ø' (degrees)
Ash	100	0	22
Upper Clay Dike	125	200	29
Lower Clay Dike	125	100	29
Fill	124	50	29
Alluvial Clay and Silt	124	100	30
Alluvial Sand and Gravel	120	0	30
Riprap	100	0	38

7.4.5. Results of Slope Stability Analysis

Using the strength parameters (c' and ϕ') listed in Table 7.3, in conjunction with the results of the seepage analyses and piezometer data, the existing dike configurations were analyzed at nine selected cross-sections. Geo-Slope's Slope/W computer program was used for the analyses with pore pressures imported from the seepage analyses. Long term (effective stress) steady state seepage conditions were analyzed using Spencer's method. For the Spencer's method analyses, circular failure surfaces with optimization were conducted.

The stability analyses focused on the potential for failure along the exterior dike face. SLOPE/W failure surfaces from these analyses are presented on the stability sections drawings in Appendix H. The results are summarized in Table 7.4 below.

Table 7.4. Summary of Slope Stability Analysis Results

Section	Failure Surface	Safety Factor
A	Deep Seated Failure Through Dike	1.5
B	Deep Seated Failure Through Dike Shallow Slough At Toe of Lower Bench	1.4 1.0
C	Deep Seated Failure Through Dike Shallow Slough At Toe of Lower Bench	1.2 1.1
C1	Deep Seated Failure Through Dike	1.3
E	Deep Seated Failure Through Dike	1.4
F	Deep Seated Failure Through Dike	1.4
I	Deep Seated Failure Through Dike	1.7
K	Deep Seated Failure Through Dike Shallow Slough At Toe of Lower Bench	1.5 1.4
M	Deep Seated Failure Through Dike	1.5

*Note: See Table 8.1 for safety factors during closure plan stages and following final closure.

Based on discussions with TVA, the guidelines presented in USACE Manual EM 1110-2-1902 “Slope Stability” and in accordance with current prevailing geotechnical practice, a minimum target factor of safety of 1.5 was established for long term conditions.

The results of the slope stability analyses demonstrate that the factors of safety against long-term, steady state seepage slope stability range from about 1.2 to greater than 1.5 for deep failures through the dikes. The results indicate that only four cross-sections (Sections A, I, K, and M) have safety factors that meet the target. In each case, the critical slip surfaces extend into the dike to affect the crest and represent a global failure surface. The critical slip surfaces are depicted in Appendix H.

There was no indication in the slope stability analyses that a noncircular failure surface would give a factor of safety lower than that obtained for circular surfaces. Overall, the geometry of the dike cross-sections and the foundation stratigraphy do not appear to be susceptible to sliding along a planar surface. The optimization scheme available within SLOPE/W was used to consider noncircular, curved slip surfaces. The results presented in Table 7.4 and in Appendix H represent factors of safety computed from the optimized, circular slip surface routine.

8. Conclusions and Recommendations

During the period of time that this geotechnical exploration and slope stability evaluation was performed, TVA has decided to close the Active Ash Pond. Stantec is currently in the first phase of the Final Closure Plan development for TVA. On December 15, 2009 a presentation titled “Johnsonville Active Ash Disposal Area Closure Plan” was provided by Stantec and TVA to the Tennessee Department of Environment and Conservation (TDEC). This plan described four interim closure plan stages that will be implemented by TVA prior to the final closure. Each of the interim stages will improve slope stability factors of safety, and when complete (estimated completion of interim stages will be during 2010), all stability sections will exhibit a minimum 1.5 static factor of safety. The closure stages are briefly described as follows:

- Closure Plan Stage 1 is construction of new spillways and lowering pool levels in the ash pond complex and sluice channel. The spillways were completed in November, 2009. The Ash Pond Complex now operates at elevation 384.6 feet, which is 2.4 feet lower as compared to its previous level.
- Closure Plan Stage 2 is relocating the sluice channel to flow in an east-west direction across the Active Ash Disposal Area. The abandoned sluice channel that ran inside the Northeast dike will be excavated to Elevation 378 feet and maintained in a dewatered condition by pumping. This stage was completed during the first quarter 2010.
- Closure Plan Stage 3 is improving the slope stability of the Northeast dike by installing an internal seepage filter and flattening the exterior slope. A rock buttress will also be constructed along the toe of the lower bench. This project is currently in construction and is scheduled for completion during the second quarter of 2010.
- Closure Plan Stage 4 is a slope stability improvement project on the Southeast dike, similar to Stage 3. It is scheduled for completion during the third quarter of 2010.

The conclusions and recommendations that follow are based on Stantec's understanding of the Active Ash Disposal Area, as discussed throughout this report, and on TVA's plan to close the facility using the interim closure plan stages described above.

8.1. Based on the results of the Phase 1 Facility Assessment (Document Review and Site Reconnaissance), and the Geotechnical Exploration, the following conclusions are developed regarding the perimeter dike at the Active Ash Disposal Area:

- The dikes from Elevation 370 to 390 feet (upper and lower clay dikes) have been built using clay and silty clay. These materials are primarily medium to stiff in consistency. While construction reports were not available for review, the results from borings indicate that the perimeter dike was compacted during construction.
- The interior portion of the upper clay dike (from Elevation 378 up to 380 feet) was placed above a zone of bottom ash that is underlain by sluiced fly ash.
- The materials immediately beneath the upper and lower clay dikes consist of fill materials. Historical records and the borings indicate much of this material was placed as hydraulic fill. Due to its reported placement method, the fill is heterogenous in textural properties, as well as consistency and strength. Seepage areas observed on the lower slopes and benches along the northeast and southeast dikes appear to be occurring though pervious zones within hydraulically-placed fill material.
- The dike was constructed using slopes that are steeper than indicated on the old construction drawings. Typical exterior slopes on the drawings reviewed by Stantec are 2H:1V or 3H:1V. Along the northeast and southeast dike sections slopes as steep as 1.5H:1V to 1.7H:1V were measured.
- The localized depressed areas along the northern end of the west dike do not appear to have been caused by internal erosion in the dike.

8.2. The seepage and slope stability models indicate that the perimeter dike currently exhibits safety factors against piping and slope stability that are less than the minimum targets of 4, and 1.5, respectively, for the long term steady state seepage condition. The final closure will ultimately result in the Active Ash Area being completely dewatered, graded and capped. Specific details of final closure will be determined during the design phase. In the interim, TVA will begin closing the area in stages, as described above. The following tables present the seepage and the slope stability factors of safety for each stage. It is recommended that piezometers and slope inclinometers be monitored monthly. The seepage and slope stability models should be updated as required to document actual factors of safety.

Table 8.1. Summary of Seepage Analyses Through Final Closure

Section	Seepage Location for Existing Condition	Factor of Safety					
		Existing	Closure Plan Stage				Proposed Final Closure*
			1	2	3	4	
A	Toe of first bench	2.7	2.8	4.2	10.6	10.6	------
B	Toe just below waterline	2.5	2.5	4.6	5.7	5.7	------
C	Toe at waterline	3.0	3.1	3.1	4.5	4.5	------
C1	Toe of first bench (just below Lower Dike)	5.7	5.7	7.1	13.6	13.6	------
E	Toe at waterline	7.1	7.1	7.1	7.1	>4	------
F	Toe of dikes	10.4	10.4	10.4	10.4	>4	------
I	Approximately 12 feet from shoreline	3.6	3.6	3.6	3.6	3.6	------
K	Approximately 12 feet from shoreline	8.8	8.8	8.8	8.8	8.8	------
M	Below toe of riprap	33.9	37.3	37.3	37.3	37.3	------

*Note: The factor of safety for piping not applicable for final closure since the facility will be dewatered and capped.

Table 8.2. Summary of Slope Stability Analyses Through Final Closure

Section	Failure Surface	Factor of Safety					
		Existing	Closure Plan Stage				Proposed Final Closure
			1	2	3	4	
A	Deep Seated Failure Through Dike	1.5	1.5	1.5	1.8	1.8	TBD*
B	Deep Seated Failure Through Dike Shallow Slough At Toe of Lower Bench	1.4 1.0	1.4 1.0	1.6 1.3	1.8 1.5	1.8 1.5	TBD*
C	Deep Seated Failure Through Dike Shallow Slough At Toe of Lower Bench	1.2 1.1	1.4 1.2	1.4 1.3	1.6 1.6	1.6 1.6	TBD*
C1	Deep Seated Failure Through Dike	1.3	1.3	1.5	1.7	1.7	TBD*
E	Deep Seated Failure Through Dike	1.4	1.5	1.5	1.5	>1.5	TBD*
F	Deep Seated Failure Through Dike	1.4	1.4	1.4	1.4	>1.5	TBD*
I	Deep Seated Failure Through Dike	1.7	1.8	1.8	1.8	1.8	TBD*
K	Deep Seated Failure Through Dike Shallow Slough At Toe of Lower Bench	1.5 1.4	1.5 1.4	1.5 1.4	1.5 1.4	1.5 1.4	TBD*
M	Deep Seated Failure Through Dike	1.5	1.5	1.5	1.5	1.5	TBD*

*Note: TBD – To be determined during final closure

8.3. Since its installation in February 2009, the seepage collection system at Seep 3A has been monitored for changes in quantity of flow and turbidity. With the exception of fluctuations caused by rainfall, there have not been noted increases in seepage flow or turbidity. Recent measurements of Seep 3A, taken following the pool reduction in the ash pond complex, indicate a dry weather flow of 2,300 GPD. This is a 34 percent reduction as compared to measurements prior to pool lowering.

Based on the measurements and observations made over the past 14 months, Stantec recommends that the effluent pipe be removed, and that a graded filter blanket drain be installed at Seep 3A during the construction phase of Closure Plan Stage 4.

8.4. The new spillway system for the Active Ash Area was designed and constructed during 2009. This has allowed the operating levels in Ash Pond Complex (Ponds A, B and C) to be reduced by 2.4 feet, thereby reducing seepage pressures and increasing freeboard. It is recommended that future monthly monitoring and analysis of Total Suspended Solids from the Pond C outfall be reviewed to determine if permit compliance could be achieved using a lower water level.

8.5. The localized depressions on the northern end of the west dike should be backfilled in the depressed areas with compacted clay soil and seeded. This will eliminate runoff from being intercepted and entering the exterior dike slope and will result in improved slope stability in these areas.

9. Closure and Limitations of Study

9.1. These conclusions and recommendations are based on data and subsurface conditions from the borings advanced during this investigation using that degree of care and skill ordinarily exercised under similar circumstances by competent members of the engineering profession. No warranties can be made regarding the continuity of conditions between borings.

9.2. The boring logs and related information presented in this report depict approximate subsurface conditions only at the specific boring locations noted and at the time of drilling. Conditions at other locations may differ from those occurring at the boring locations. Also, the passage of time may result in a change in the subsurface conditions at the boring locations.

10. References

The following is a list of historical documents that were used to evaluate Ash Disposal Areas 2 & 3 and prepare this report:

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