Report of Phase 2 Geotechnical Exploration

Ash Pond Complex
Widows Creek Fossil Plant
Stevenson, Alabama

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Prepared for:
Tennessee Valley Authority
Chattanooga, Tennessee

February 4, 2010

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Mr. Barry Snider, PE
Tennessee Valley Authority
1101 Market Street
LP 5E-C
Chattanooga, Tennessee 37402

Re: Report of Phase 2 Geotechnical Exploration
Ash Pond Complex
Widows Creek Fossil Plant
Stevenson, Alabama

Dear Mr. Snider:

As requested, Stantec Consulting Service Inc. (Stantec) has completed our Geotechnical Exploration for the Ash Pond Complex at the Widows Creek Fossil Plant. This report documents the subsurface conditions encountered during the exploration, results of laboratory testing, findings from the historical document reviews, results of our analyses and evaluation, and recommendations for the facility.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 909 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.

Robert D. Fuller

Robert D. Fuller, PE
Project Manager

ALABAMA LICENSED No. 30931 PROFESSIONAL 02-04-2010 ENGINEER ROBERT DANIEL FULLER

/cmw/rdr

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Executive Summary

The purpose of this study was to evaluate the as found stability of the Ash Pond Complex at Tennessee Valley Authority's Widows Creek Fossil Plant (WCF). This document is the second report in a two part series covering the identified coal combustion products impoundments within the WCF property. At this time, TVA plans to operate the Active Stacking Area for another five years. The inactive Dredge Cell will be closed. The following assessment of the Ash Pond Complex and associated recommendations are described below.

The geotechnical engineering analyses focused on a total of ten representative cross sections which model the various slope conditions around the perimeter of the subject structures. Four of the cross sections (A, C, D and H) are located around the inactive Dredge Cell, two (J and L) around the Main Ash Pond A and Active Ash Stacking Area, two (M and O) are located within the Bottom Ash Stack area, one (S) through the Lower Stilling Pond dam and one (T) through the Pump Pond dam. Only Sections H and S listed above represent interior dike segments, the remaining sections are located on exterior/perimeter dikes. The cross sectional geometry, including the thickness and depth of various soil layers, were estimated using data from the site exploration program, the historical project drawings, and other project documentation. The exploration encountered native foundation soils consisting of clays and silty clays and confirmed that these soil types were utilized to construct the initial exterior/perimeter starter dikes. It was also confirmed that material impounded behind the perimeter dikes is a mixture of bottom ash, fly ash and gypsum.

Stability Analysis Results for As Found Conditions

The slope stability calculations produced factors of safety against sliding along various potential failure mechanisms. Current USACE criteria for the long-term stability require a factor of safety for slope stability of at least 1.5. The slope stability results indicate that the Ash Pond Complex meets this criterion for all the sections analyzed except for five (5) (Section A, D, J, L and S). In general, the lower safety factors were calculated along the north and west side of the Main Ash Pond A/Active Ash Stacking Area and the Lower Stilling Pond dam. Results of the stability analysis ranged from a factor of safety of 1.2 to a high of 3.3. The stability and seepage results for the Main Ash Pond A dam, represented by Section R, will be provided under a separate Design Report as part of a spillway replacement project which will begin construction in March, 2010.

Seepage Analysis Results for As Found Conditions

The factors of safety against piping were computed based on the exit gradient results from SEEP/W and the critical gradients determined from the soil properties. The United States Army Corps of Engineers (USACE) design criteria was used to select an acceptable factor of safety against piping ($FS_{piping} \geq 3$). Six of ten segments, represented by cross sections do not meet the design criteria for piping at the seepage exits. The factors of safety range from a 1.0 at Section O which is located along Scrubber Road on the southwest portion of the complex to an 8.1 at Section H which is along the interior dike between the Old Scrubber Sludge Pond and Main Ash Pond A.

Stability Analysis Results for Final Ash Stack Height

The results for the planned final ash stack height were also determined for long term steady state seepage conditions, assuming the future stack will be constructed in accordance with a proposed grading plan provided by Trans Ash in the Active Ash Stacking Area.

The factors of safety for the five-year build out conditions ranged from a low of 1.3 to a high of 2.1. The lowest factor of safety at the planned final stack height was found along the west perimeter of the Active Ash Stacking Area where a factor of safety of 1.3 was computed for Section L. Based on the geotechnical recommendations provided in this assessment report, the design engineer(s) will develop mitigation plans for construction. The objective of these mitigations plans will be to increase the factors of safety against sliding and soil piping to achieve the minimum target values as the stack height increases. The mitigation plans will incorporate specific interim risk reduction strategies and include both enhanced geotechnical instrumentation monitoring, inspections and construction of dike embankment buttress and storm water control systems.

Report of Phase 2 Geotechnical Exploration

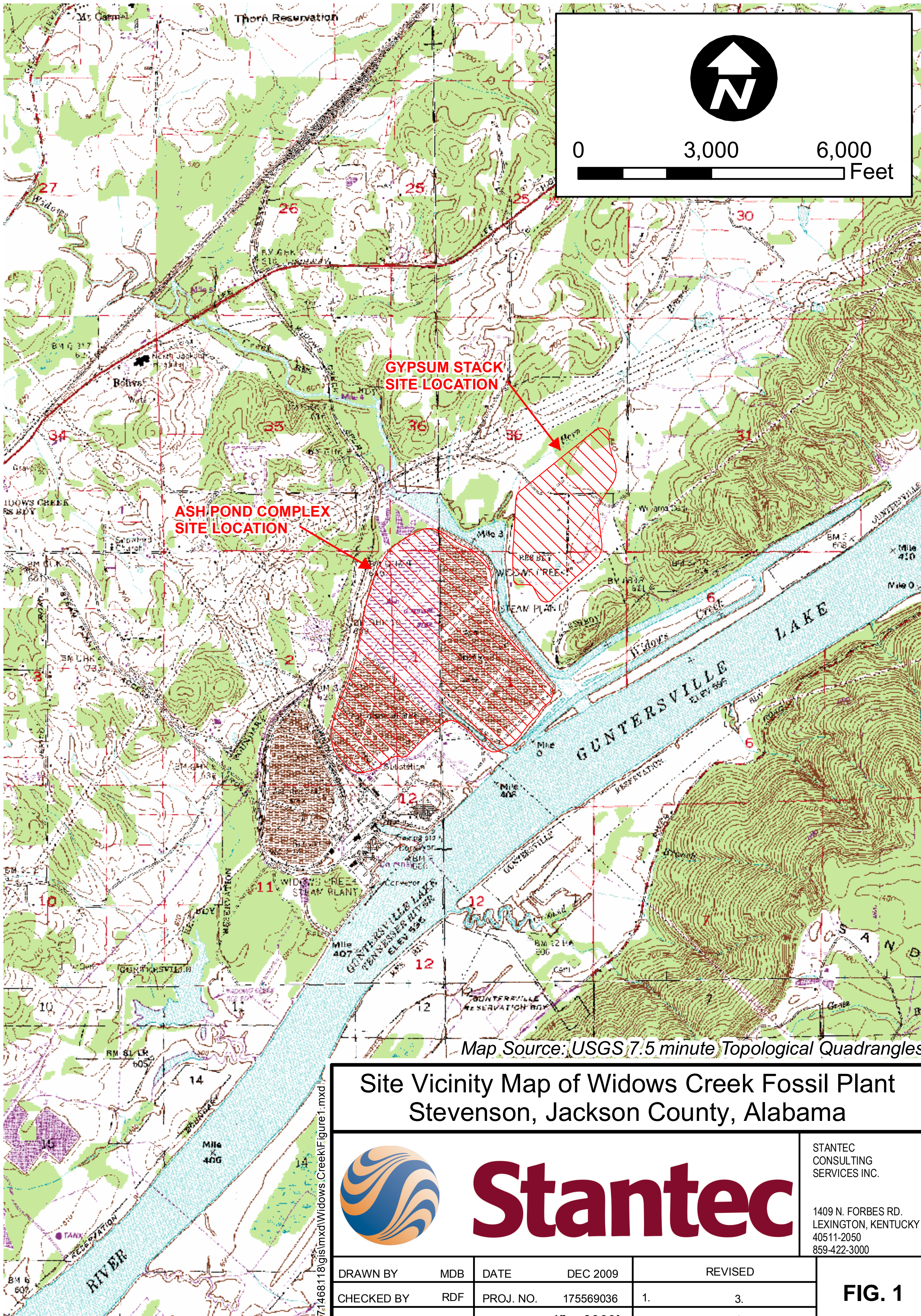
Ash Pond Complex
Widows Creek Fossil Plant
Stevenson, Alabama

1. Introduction

The Widows Creek Fossil Plant is located in northeastern Alabama along the west bank of the Tennessee River at the confluence of Widows Creek. More specifically, the plant is located at 2800 Steam Plant Road in Stevenson, Jackson County, Alabama approximately 40 miles southwest of Chattanooga, Tennessee. A site vicinity map showing the overall facility and the location of the Ash Pond Complex is depicted in Figure 1. This report constitutes the second report in a two-report series and addresses the active and inactive waste disposal areas located west of Widows Creek. The waste disposal areas east of Widows Creek was the subject of our report titled Phase 2 Gypsum Stack Geotechnical Report. Thus, for clarity, the common report sections from the first report (i.e., site location and site geology,) have not been repeated herein.

2. Ash Pond Complex

Construction at the TVA-Widows Creek Fossil Plant began in 1950 and was finished in 1965 with the completion of eight coal-fired generating units. Units 1 through 6 are the oldest units and Units 7 and 8 became operational in 1964. Based on published information, the winter net dependable generating capacity is 1,629 megawatts (MW) and the aggregate capacity of the eight units is 1,950 MW. The plant currently consumes 10,000 tons of coal per day resulting in approximately 280,000 dry tons of fly ash and approximately 110,000 dry tons of bottom ash being produced each year. The bottom ash and fly ash wastes are transported by wet-sluice methods to an on site stacking areas where they are deposited.



Site Vicinity Map of Widows Creek Fossil Plant
Stevenson, Jackson County, Alabama

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DRAWN BY	MDB	DATE	DEC 2009	REVISED		FIG. 1
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2.1. Structures

For the purposes of this report, eight (8) structures are defined within the Ash Pond Complex. The structures of the Ash Pond Complex are identified below in Table 1. The surface area limits of each structure are delineated on Figure 2. All elevations are expressed in feet.

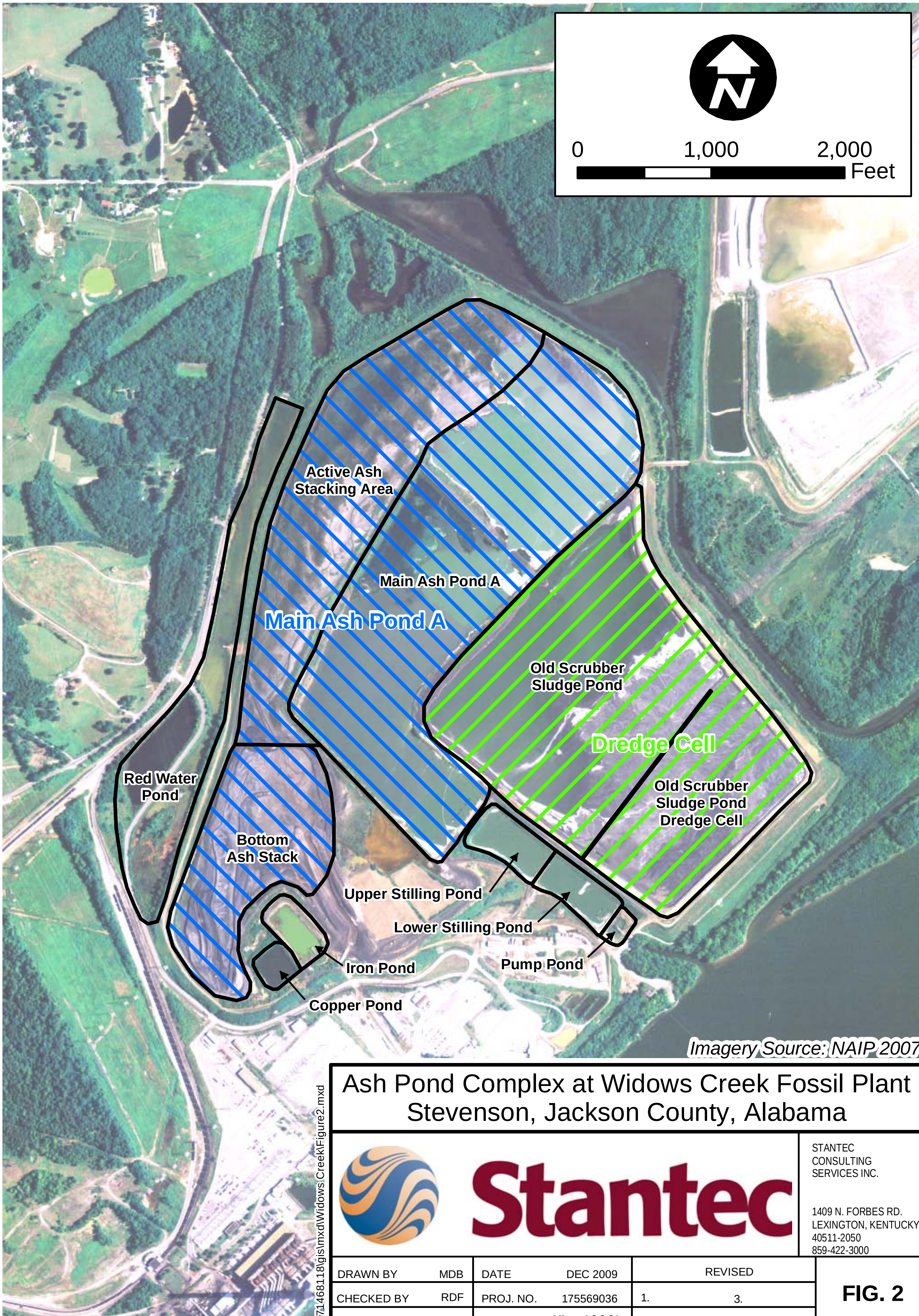
Table 1. Structures Within Ash Pond Complex

Pond	Surface Area (Acres)	Top Elevation (MSL)	Toe Elevation (MSL)
Dredge Cell	116	646	595
Main Ash Pond A	156	648	595
Upper/Lower Stilling Pond	4	614	595
Pump Pond	<1	636	595
Red Water Pond	32	625	609
Iron Pond	3	630	630
Copper Pond	3	630	630
Bottom Ash Stack	32	649	613

2.1.1. Dredge Cell

The elevated Dredge Cell is located to the south of the Main Ash Pond, and has a surface area of approximately 116 acres. The initial exterior dike was constructed of rolled earth fill to an elevation of 626 feet. A second rolled earth dike was constructed above the initial dike in 1983, bringing the top of dike elevation to 636 feet. According to a 2003 MACTEC report titled “Proposed Ash Pond Dike Raise,” the gypsum/scrubber sludge product was discharged into the southwest corner, south of the interior deflector dike running from west to east through the pond. Once TVA switched to a forced oxidation gypsum scrubber in the mid-1980’s, a new gypsum stack was created. After the Gypsum Stack was placed in-service beginning in 1986, the Scrubber Sludge Pond (referred herein as Dredge Cell) received dredged material from the Main Ash Pond A. In the mid 2000’s the dikes were raised to their current elevation of 646 feet using bottom ash fill. In 2007, plans were developed to again raise the exterior dikes around this area, as the remaining capacity would be exhausted by 2009. At this point, the exterior dikes have not been raised and the Dredge Cell is inactive and being allowed to dewater. A closure plan is currently in development.

Through the structure’s life, the Dredge Cell containment area has served several different purposes. Likewise, the area has been identified by a number of different names on TVA drawings which are listed in Table 2. Based on a review of aerial photographs dated over recent years, the Dredge Cell was subdivided by an interior deflector dike. The portion located south of the deflector dike was identified as the Old Scrubber Sludge Pond Dredge Cell. The portion located north of the deflector dike was identified Old Scrubber Sludge Pond. Since the interior dike no longer hydraulically separates the two areas, the combined area is now referred to as the Dredge Cell, as referenced herein.



Ash Pond Complex at Widows Creek Fossil Plant
Stevenson, Jackson County, Alabama

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DRAWN BY	MDB	DATE	DEC 2009	REVISED		FIG. 2
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Table 2. Former Names of the Dredge Cell

Name	Year	Source
Limestone Scrubber Pond	1959	10N7400-R1
Limestone and Ash Disposal Area	1973	10N7400-R5
SO_2 Scrubber Disposal Area	1983	10W7465-01
Gypsum Sludge Pond	1985	Memo B46-85-0723-001
Abandoned Gypsum Disposal Area	2003	MACTECT 2003 Report Figure 1B
Old Scrubber Sludge Pond	NA	Widows Creek FP General Information – TVA Surveying – Plan View with Pond Labels
Old Scrubber Sludge Pond Dredge Cell	NA	Widows Creek FP General Information – TVA Surveying – Plan View with Pond Labels
Dredge Cell	2007 to Current	TVA Scope of Work – WCF – Raise Dredge Cell

NA = Not Available

2.1.2. Red Water Pond

Commissioned in 2007, the Red Water Pond serves as a surface water runoff collection pond for the Main Ash Pond A. It has a surface area of approximately 32 acres, with the top of dike at elevation 625 feet and the toe of dike at elevation 609 feet. It is located to the west of the Bottom Ash Stack and Main Ash Pond A and has a total dike length of approximately 225 feet. It operates with a normal pool elevation of approximately 602.4 feet.

2.1.3. Upper/Lower Stilling Pond

The Upper/Lower Stilling Pond receives discharge from the Main Ash Pond A. Commissioned in 1986, the stilling pond receives effluent discharge through five morning-glory type spillways in the Main Ash Pond A. The pond has a surface area of approximately eight acres, with a top of dam elevation of approximately 614 feet and toe of the dike near elevation 596 feet. The Upper Stilling Pond and Lower Stilling Pond are hydraulically connected but partially separated by an internal deflector dike approximately one-hundred feet in length and fifteen to twenty feet in height. The pond operates at a normal pool elevation of approximately 611.5 feet and discharges effluent water to the Pump Pond via a series of five spillways.

2.1.4. Pump Pond

The Pump Pond is located southeast of the Lower Stilling Pond and south west of the Dredge Cell. It has a surface area of approximately 0.25 acres and receives effluent from the Lower Stilling Pond spillways. Commissioned in 1986, two recirculation pump stations located within the pond pump water to the Condenser Cooling Water Intake with a portion being pumped to the wet gypsum system. The pond has a normal pool elevation of approximately 602 feet. Three overflow-type spillway pipes are located in the dam and transport water directly to the discharge channel leading to the Tennessee River during large storm events or when the pump stations are not in operation. The top of dam elevation is approximately 614 feet.

2.1.5. Main Ash Pond A

The Main Ash Pond A is located to the northwest of the Dredge Cell and to the northeast of the Bottom Ash Stack. It has a surface area of approximately 156 acres. The area is first shown on TVA Drawing 10N7400-R5 dated 1959, labeled as New Ash Disposal Area. The initial starter dike was constructed of rolled earth fill built to elevation 626 feet. A second rolled earth dike was constructed above the initial dike sometime before 1973, bringing the top of dike elevation to 636 feet. The Main Ash Pond A receives both fly ash and bottom ash. The fly ash and bottom ash from Units 1-6 are sluiced to one slurry ditch, while the bottom ash from the ourfall from Units 7-8 are sluiced to a separate slurry ditch. The outfall from the sluice pipe lines for Units 1-6 are located west of the Copper Pond and Units 7-8 are positioned east of the Iron Pond. The ditches flow through the Bottom Ash Stack area separately and then converge into one ditch through the Active Ash Stacking Area which flows into the Main Ash Pond A. In the southwest corner of the Main Ash Pond A, five spillways decant water to the Upper Stilling Pond.

2.1.6. Bottom Ash Stack

The Bottom Ash Stack is located to the southwest of the Main Ash Pond A. It is formerly known as the Ash Disposal Area Units 7-8. CCP waste, which is predominantly bottom ash, is deposited by routine wet handling methods from two slurry ditches which traverse the Bottom Ash Stack area in a south to north direction. The Bottom Ash Stack has a surface area of approximately 32 acres. Currently, the stack has an approximate top elevation of 646 feet. The toe of the stack is approximately 610 feet in elevation. The available stacking plan indicates that the perimeter side slopes of 3H:1V are being used.

3. Review of Available Information

3.1. General

As a part of the Phase 1 site assessment Stantec engineers reviewed documents provided by TVA pertaining to the Gypsum Stack, Main Ash Pond A, Dredge Cell and associated water treatment ponds. The main objective of the document review was to develop a historical knowledge base prior to beginning the geotechnical exploration. The documents reviewed included record drawings, cross sections, aerial photographs, old contour maps, and annual inspection 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:

a. Ash Dike Raising, Plan of Soils Foundation Investigation Drawings and Boring Logs, August 7, 1980 and February 20, 1980.

b. Ash Dike Raising, Plan of Borrow Investigation from TVA, January 9, 1981.

c. Ash Pond Cell Stability Analysis (using PCSTABL5M), April 24, 1981.

d. Borrow Area For Scrubber Sludge Pond Dike Raising, Top of Rock Contour Map and Boring Logs, December 17, 1982.

e. TVA Memorandums, Widows Creek Fossil Plant Units 7 and 8, Ash Disposal Area – Bentonite Slurry Cutoff Wall Seepage Monitoring Program, dated November 7, 1984, February 6, 1985, April 10, 1985, June 3, 1985, and August 20, 1985.

f. Coal Combustion By-Product as Engineered Fill, Laboratory Test Results, July 1995 and various dates.

g. Final Report – Fly Ash, Bottom Ash, and Scrubber Gypsum Study, Law Engineering and Environmental Services, November 7, 1995.

h. Report of Geotechnical Exploration, Proposed Ash Pond Dike Raise, Widows Creek Fossil Plant, Stevenson, Alabama, MACTEC Engineering and Consulting, February 4, 2003.

i. Report of Cone Penetrometer Testing, Dredge Cell Dike, Widows Creek Fossil Plant, Stevenson, Alabama, MACTEC Engineering and Consulting, June 23, 2004.

j. Annual Ash Pond Dike Stability Inspection, Summary of Recommendations, 2008.

These studies included reports, recommendations, boring plans, driller's logs, and results from laboratory tests. The information gained from these historical documents were evaluated and used to supplement the information gathered from Stantec's geotechnical exploration.

3.2. Site History

A list of key events related to the planning, construction and operation of the Ash Pond Complex is provided in Table 3.

Table 3. Summary of Events

Date*	Event
June, 1952	Record Drawing (10N206-R1) of Ash Disposal Area No. 1 (Units 1-6) generated
March, 1959	Drawing (10N7400-R5) of Ash Disposal Area No. 2 (Units 7&8) generated with initial crest heights of 610.0 feet
October, 1973	Record Drawing (10N7421-R6) and Drawing (10N7422-R8) of Limestone and Ash Disposal Area generated showing initial crest of 626.0 feet and raised to 636.0 feet
January, 1978	Record Drawings (10W213-R3) of Chemical Treatment Ponds
Early 1980's	Review Drawings (10W7421-R9) of Raised Dredge Cell with crest elevation up to 661.0 feet with proposed underdrains to be implemented in 2012
1981	Foundation Inspection for Bridge across Widows Creek (10B421-01)
June, 1983	Initial exploration performed by Singleton Materials Engineering Laboratory (50 split-spoon borings and 8 undisturbed borings), topographic survey with top of rock contour map.
June 29, 1983	Eight additional split-spoon borings added near SS-19 due to soft soils
1984	Addition of Slurry Cutoff Wall around the west side of Ash Pond (10W7465-01,02), Designed and Constructed by CEB Geology 2nd Geotechnical Group
1987	Redwater Containment Dike/Spillway Improvements (10W7466-1)
2003	Mactec Geotechnical Exploration, Ash Pond Dike Raising Report
2004	Mactec CPT Drilling around Dredge Cell
March, 2007	Review Drawings (10W7463-1R1) of Ash Pond Cap at 2032
December, 2008	O&M management transferred from TVA-HED to Trans Ash
August, 2009	Trans Ash Begins Ash Pond Expansion/Ditch Realignment

*All dates listed are approximate based on Stantec's review of available documents.

Since 1967, TVA has performed yearly inspections of the waste disposal areas at the Widows Creek Fossil Plant and made subsequent repairs based on the observed conditions. As part of this year's facility assessment work, Stantec reviewed the information within these reports to gain an understanding of how the disposal areas were constructed.

3.3. 1952 Conceptual Design Recommendations

The initial Ash Disposal Area No. 1 (Units 1-6) is located within the railroad loop just north of the power plant. The basis of operation for the initial Ash Disposal Area No. 1 (Units 1-6) can be found on the referenced TVA Record Drawing No. 10N206R1 dated June, 1952 in Appendix A. Based on this document and a review of the available supporting information provided by TVA, the ash disposal area was managed by TVA O&E departments from the early 1950's to late 1960's. The final fill date for Area No. 1 was June 3, 1969 at a crest elevation of 625 feet. As an abandoned structure, the Ash Disposal Area No. 1 (Units 1-6) is not a subject of this report as directed by TVA.

Following the completion of Area No. 1, TVA designed a second ash disposal area, referred to as Area No. 2 and immediately adjacent to and west of the active stacking areas at that time, a Limestone Scrubber Pond (a.k.a., Dredge Cell), Chemical Treatment Pond, and a future ash disposal area (a.k.a., Main Ash Pond A area) to be used once Area No. 2 was filled to capacity. Area No. 2 was designed to receive two additional units (Units 7 & 8) with an initial crest elevation of 610 feet and an estimated fill date of December, 1967. A fifteen foot expansion was also later designed around the ash pond and filled by March, 1971. Then, based on the record drawings, the north interior dike of Area No. 2 was breached and the ash disposal area was allowed to flow into the designated "future" ash pond area. The limits of the disposal areas are outlined on the attached TVA Record Drawing No. 10N7400R5 dated March 16, 1959. This drawing also indicates a contractor landfill and asbestos landfill to be closed as referenced on TVA drawing No. 10W7464.

3.4. 1973 Record Modification

In 1973, TVA Drawing 10N7421-R6 and 10N7422-R8 indicate the initial dikes for the Limestone Scrubber Pond and Ash Disposal Area were first raised from 626 feet to 636 feet. The record drawings were dated February 15, 1973. However, no underdrain system, stability analyses, or seepage analysis was indicated on the available drawings or within the design documents provided by TVA.

3.5. 1978 Chemical Treatment Ponds

In 1978, TVA record Drawing 10W213-R3 indicates the initial layout/design of the chemical treatment pond, which is known as the Copper Pond today. Based on the drawing information, the pond was moved 31 feet north in 1976 and enlarged the following year. The addition of the Iron Pond was constructed in 1979 followed by rock revetment in 1982.

3.6. 1980 Dredge Cell IFR Drawings

In the early 1980s, Issued for Review (IFR) Drawings 10N7420, 10W7421, and 10W7422 includes a stacking plan which extended the crest elevation from 636 feet to 646 feet. Then in June 2008, these plans were revised to show an additional fifteen foot vertical expansion and a proposed (retrofit) underdrain system to be implemented in 2012. The proposed underdrain system was to consist of a series of 12" diameter perforated HDPE pipes connected to a 15" diameter solid HDPE outfall pipe.

3.7. 1981 and 1983 Geotechnical Exploration

In 1981 a geotechnical exploration for a new bridge across Widows Creek was conducted. The first documented exploration for the Ash Pond Complex was performed by Singleton Materials Engineering Laboratory in June, 1983. The bridge exploration consisted of nine percussion holes drilled at each bridge pier and extended a minimum of five feet into solid bedrock. The 1983 exploration initially included fifty (50) split-spoon borings and eight (8) undisturbed borings, however soft soils were encountered near SS-19 and eight (8) additional spilt-spoons were added at this location. The laboratory testing consisted of soil classification tests, triaxial shear testing, consolidation and permeability tests.

Based on the results of the above exploration, a slope stability analysis was performed by TVA, with Slope 2 software for a typical section of the Ash Pond. The typical section looked at raising the crest elevation from 626 feet to 636 feet and the analysis looked at three cases (Case I - End of Construction, Case II- Sudden Drawdown, Case III – Long Term Full Operation.) The results of the analysis all meet the minimum factors of safety listed in the summary of results provided in Appendix A.

3.8. 1984 Slurry Cutoff Wall Addition

In 1984 a slurry cutoff wall was designed and constructed by CEB Geology 2nd Geotechnical Group measuring three feet wide and approximately 6,090 feet in length. It was installed around Ash Disposal Area No. 2 to contain the excessive seepage being observed along the north ditch of the Scrubber Road. The crest of the Slurry Wall was at approximately 626 feet in elevation and the toe extended approximately three feet into the residual soil. As a result of the observed seepage and subsequent cutoff wall, TVA implemented a seepage monitoring program to observe the before and after seepage flows from the ash pond. The results of the observations are included in the historical documents in Appendix A.

3.9. 1987 Redwater Containment Dike/Spillway Improvements

In 1987, minor regrading and emergency spillway modifications were made to the Redwater Containment Dike as illustrated on TVA Drawing Nos. 10W7466 series.

3.10. 2001 Preliminary Engineering Scope

In 1995 and 1996 a study for long-term ash disposal at Widows Creek was conducted by TVA. Nine options were evaluated and ultimately two options were determined to be the most cost effective for raising the ash pond.

The first option selected to be implemented was to convert the original scrubber waste pond into an ash disposal area by breaching existing interior dikes, constructing an interior ash deflector dike, and relocating the scrubber makeup water pumps. The implementation of this option was completed in 1997. After the first option was implemented, additional long-term storage was to be obtained by raising the exterior dike on the now combined original ash pond and converted scrubber waste pond.

It was recommended that the exterior dike should be raised with earth to eliminate the need for an external drainage system to collect/treat ash seepage. In 2001 the runoff from the side of the existing earthen dikes did not require collection for treatment. Therefore, it was recommended that future dike expansions be designed with earth material.

3.11. 2003 and 2004 Mactec Geotechnical Exploration

In 2003 Mactec performed a geotechnical exploration around the Dredge Cell to further investigate the foundation soils under the dike. The 2003 exploration included three (3) soil borings, three (3) cone penetration borings (CPT), and two (2) vane shear borings. The 2004 exploration consisted of seven (7) CPT borings at the location shown on the attached historical drawings within the Mactec reports.

3.12. Ash Pond Complex Design Layouts

3.12.1. Ash Disposal Area No. 1

Figure 3 below presents a portion of the plan view from drawing 10N206-R1 titled "Ash Disposal Area". The drawing is dated June 3, 1952 and depicts the original ash disposal area, Area No. 1, designed to a maximum elevation of 625 feet.

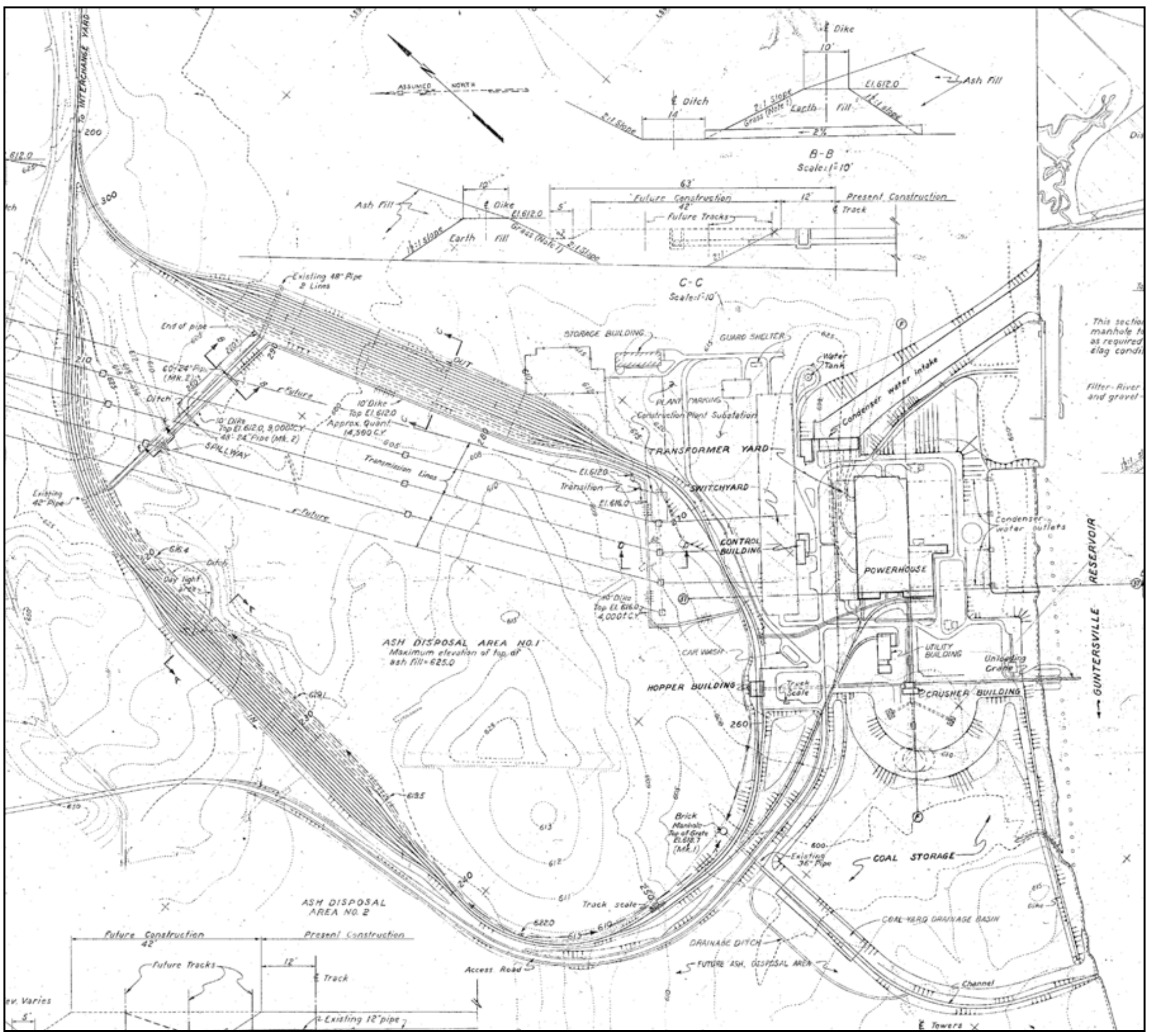

Figure 3. Ash Disposal Area, 1952

3.12.2. Ash Disposal Area No. 2

An excerpt taken from TVA drawing 10N7400-R5 titled “Ash Disposal Area” is presented below in Figure 4. This drawing from March 16, 1959 depicts the original ash disposal area for Units 1-6 and the ash disposal area for Units 7-8. It also depicts the New Ash Disposal Area above the Tennessee River, the Limestone Scrubber Pond, and Chemical Treatment Pond.

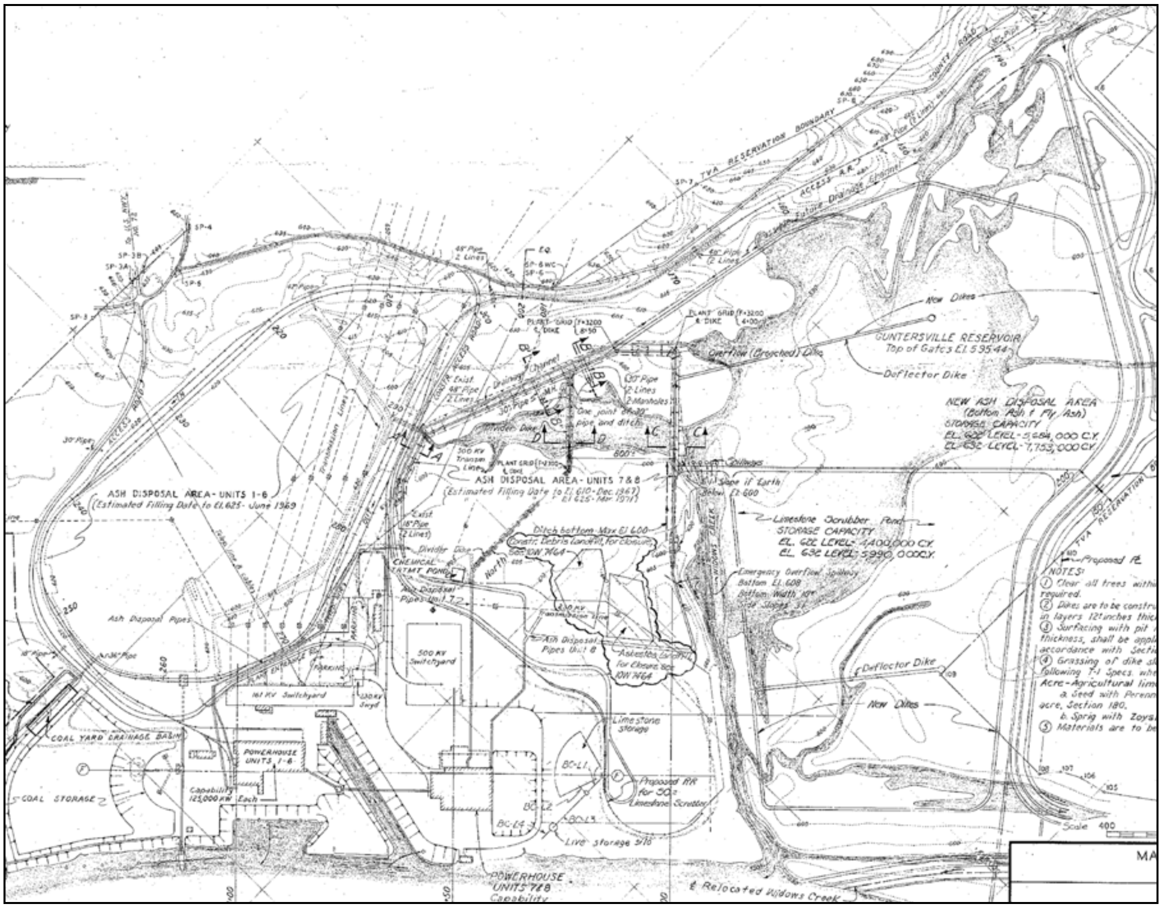

Figure 4. Ash Disposal Area, 1959

3.12.3. 1973 Dike Raising Typical Section

Figure 5 presents a typical section taken from TVA Design Drawing 10N7421-R6 "Limestone and Ash Disposal Area Plan – Sheet 2", dated February 15, 1973. The original exterior dikes had a top elevation of 626 feet. This section depicts new rolled earth fill placed on unclassified fill on top of the ash slurry, extending the top of the dike to 636 feet in elevation.

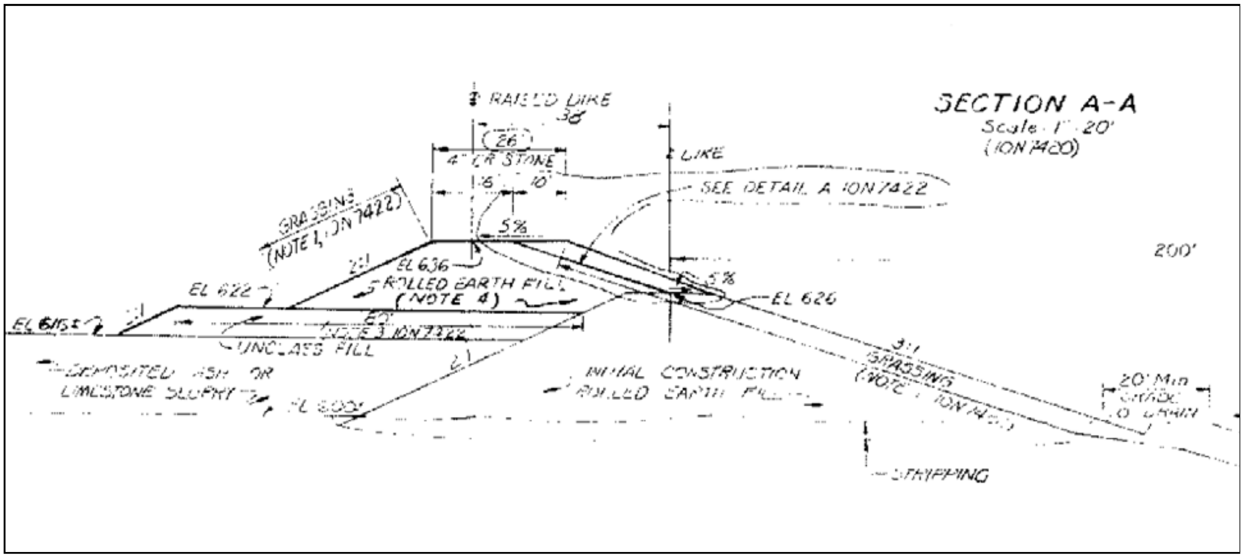

Figure 5. Typical Section 1973 Dike Raise

3.12.4. Slurry Cutoff Wall Layout

Figure 6 presents a plan view taken from TVA Design Drawing 10W7465-01 “Ash Disposal Area Units 7 & 8 - Plan”, dated July 14, 1983. This plan view shows the location of a slurry cutoff wall installed along the west and south embankment of Units 7 & 8 disposal area.

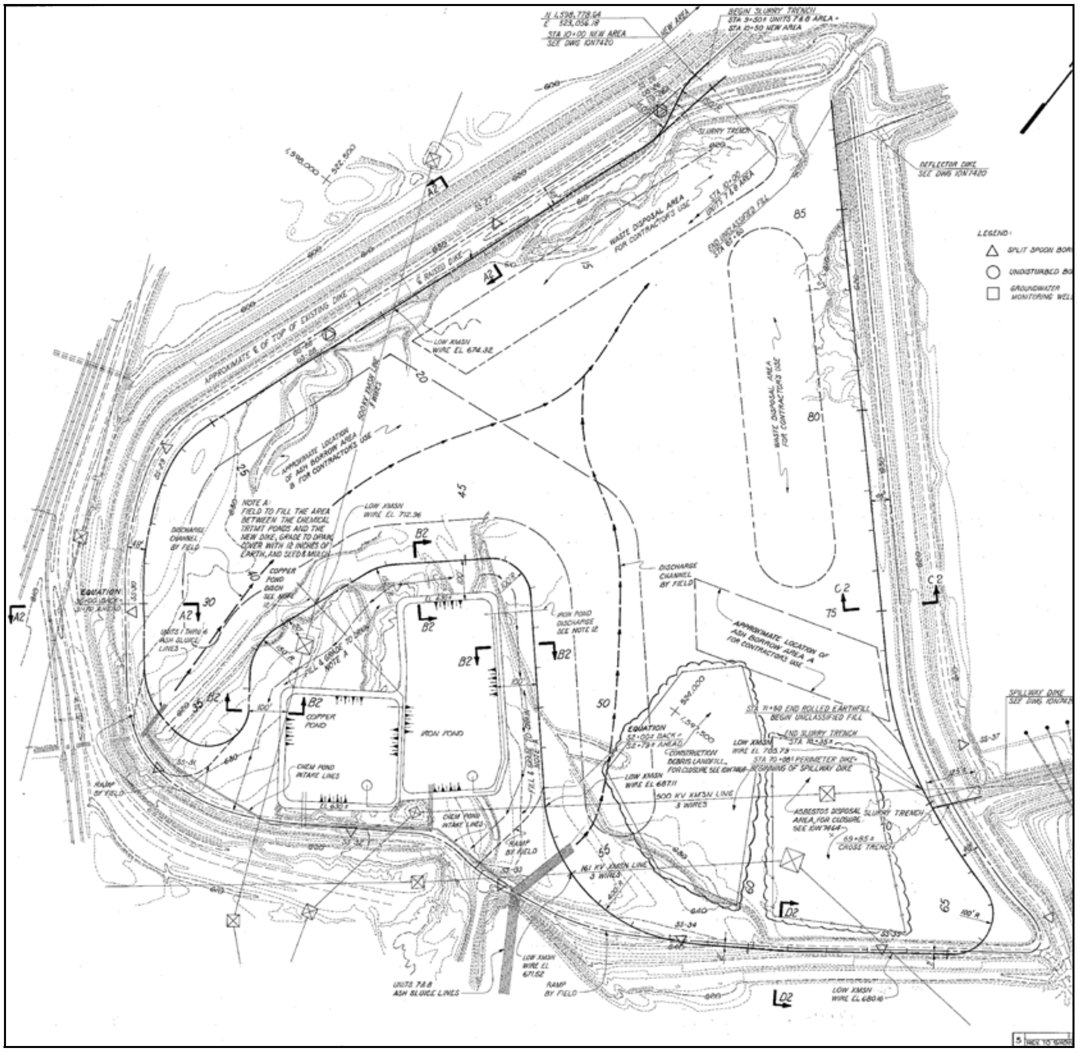

Figure 6. Slurry Cutoff Wall Plan View

3.12.5. Slurry Cutoff Wall Typical Section

Figure 7 depicts the detail for the slurry cutoff wall for Units 7 & 8 Ash Disposal Area. The detail was taken from TVA Design Drawing 10W7465-02 - “Ash Disposal Area Units 7 & 8 Section 8 Details”, dated July 14, 1983. It’s reported; the cutoff wall was embedded at least three feet into the in-situ clay residuum, was approximately three feet thick, and extended up to varying elevations based on the location. An impervious earth cap (minimum 4.5 feet thick) was placed on top of the cutoff wall. For a more detailed description see Figures 7 and 8 below.

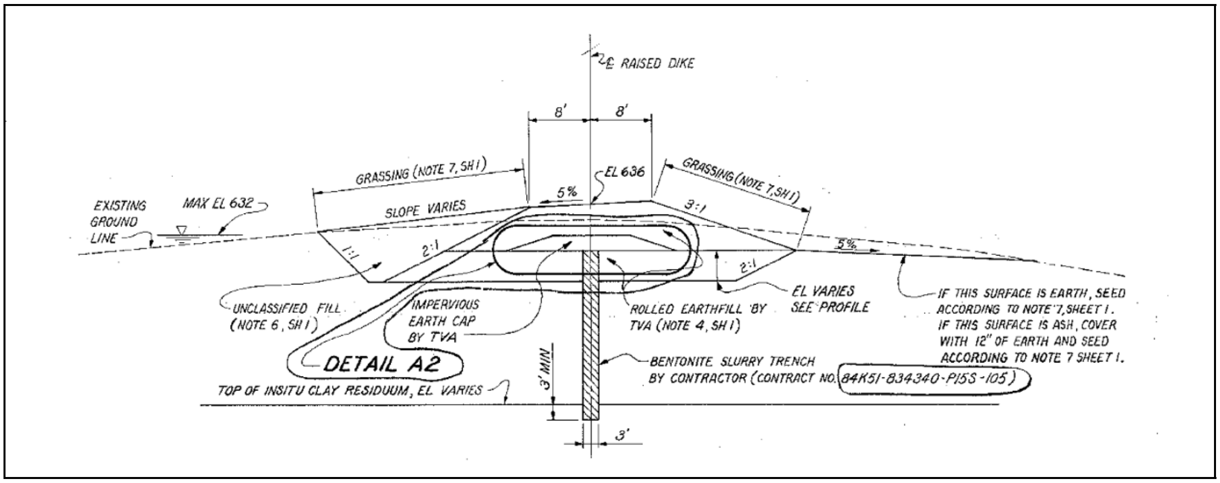

Figure 7. Slurry Cutoff Wall Typical Section

Figure 8 below shows the clay cap detail for the slurry cutoff wall. The detail was taken from TVA Design Drawing 10W7465-02 - "Ash Disposal Area Units 7 & 8 Section 8 Details", dated July 14, 1983.

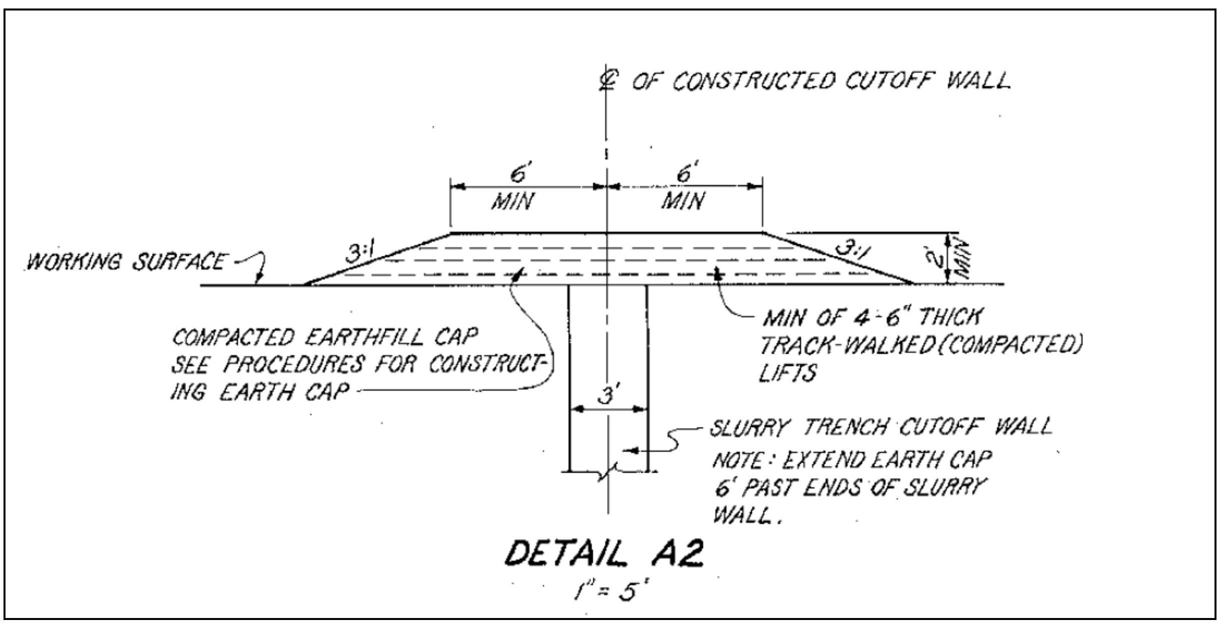

Figure 8. Slurry Cutoff Cap

3.12.6. Future Scrubber Sludge Pond Underdrain System

Figure 9 below shows the plan view for the proposed/future Scrubber Sludge Pond underdrain system. The plan view was taken from TVA Design Drawing 10W7420-3 - “Limestone and Ash Disposal Area Plan Sheet 2”, dated June 2, 2006.

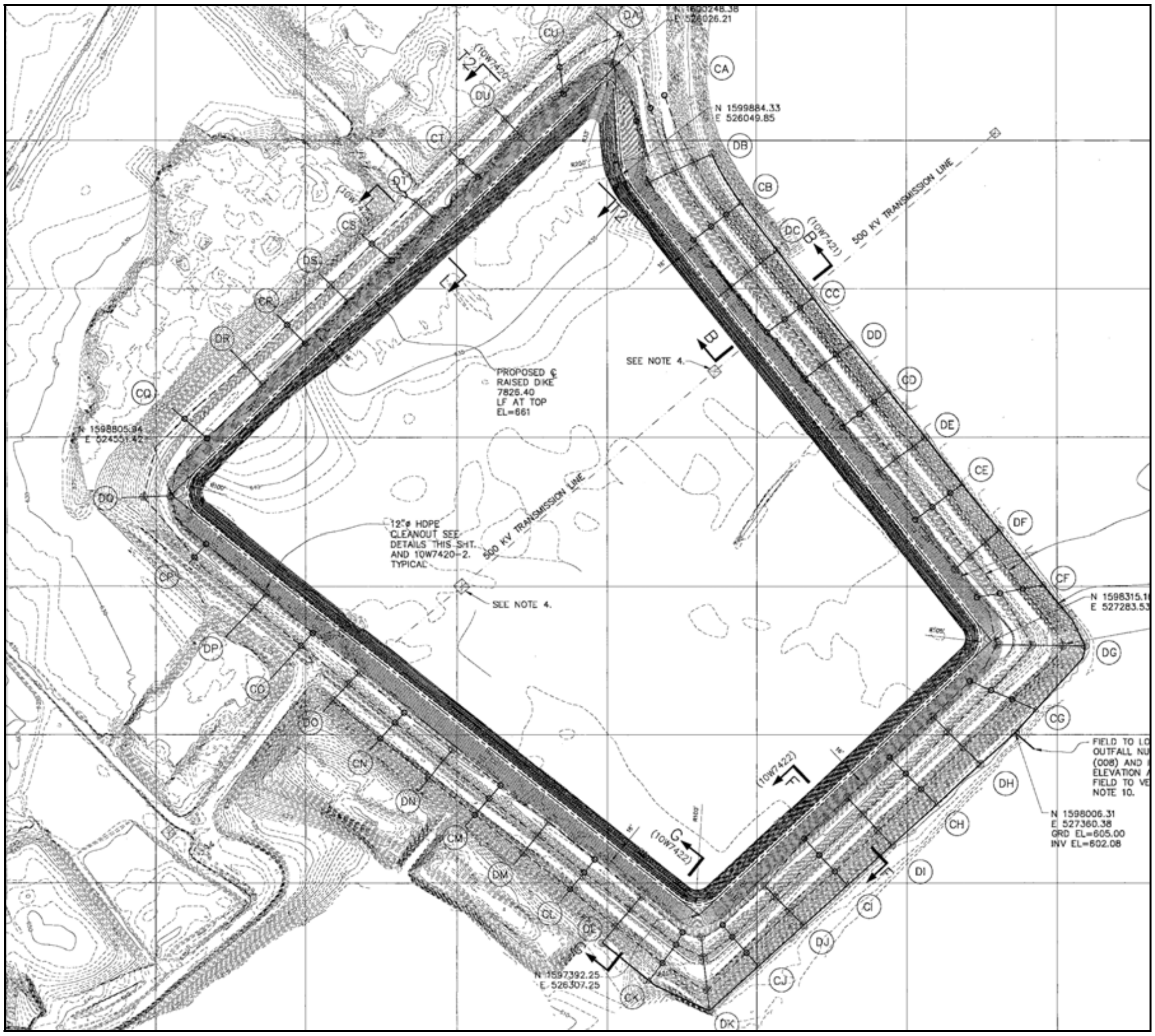

Figure 9. Scrubber Sludge Pond Underdrain Plan View

Figure 10 shows the underdrain system layout from the same TVA Design Drawing 10W7420-3 - "Limestone and Ash Disposal Area Plan Sheet 2", dated June 2, 2006.

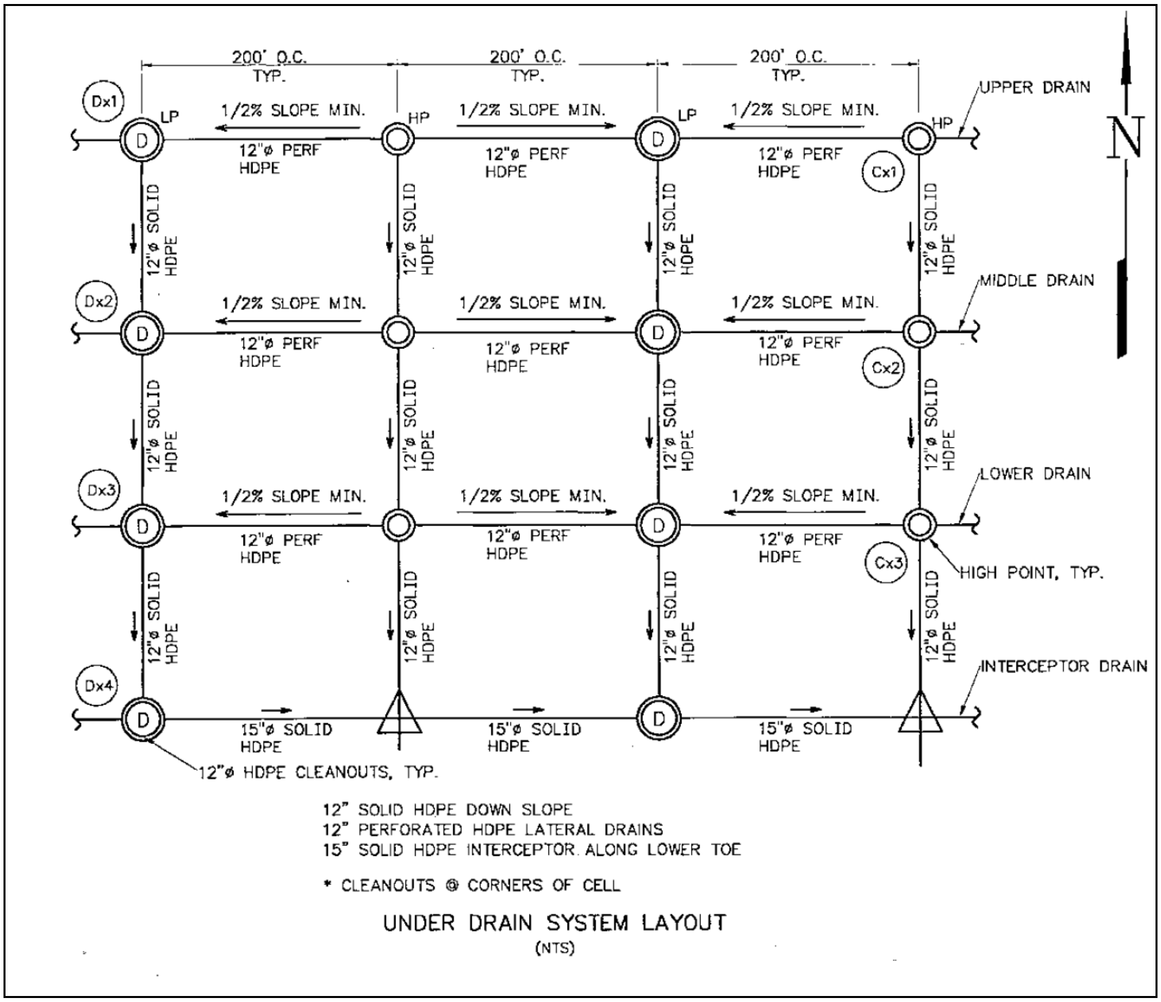

Figure 10. Scrubber Sludge Pond Underdrain Layout

3.12.7. Scrubber Sludge Pond Typical Section with Underdrains

Figure 11 below shows the typical section from 2006 with the proposed underdrain system and spillway structure. The section view was taken from TVA Design Drawing 10W7420-5 - "Ash Disposal Stack Sections and Details", dated June 2, 2006. The detail shows the initial rolled earth dike up to 626 feet in elevation followed by the 1973 rolled earth dike expansion to elevation 636 feet. Above this elevation, two additional expansions are shown with crest elevations at 646 feet and 661 feet. It should be noted that these future exterior dike raises appear to be created from Bottom Ash fill and a toe ditch is shown for each future expansion above 636 feet.

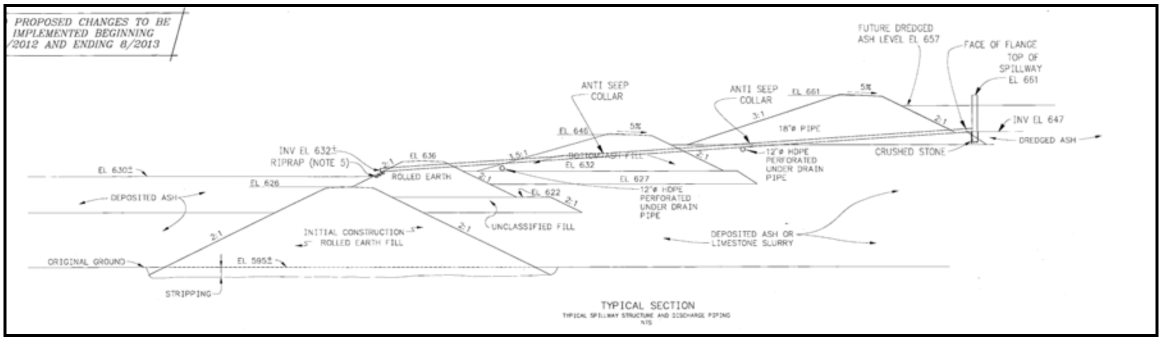

Figure 11. Typical Section, 2006

3.13. Surface Water Control

At this time, the only surface water control located around the ash pond complex is along the Scrubber Road. The perimeter side slopes show signs of erosion along the crest and perimeter dikes due little or no surface water control. The current dike conditions are depicted below in Figures 12 through 16.

Figure 12. Typical Rill Erosion, 2009

Figure 13. Typical Surface Ditch, 2009

Figure 14. Typical Roadway/Surface Rills, 2009

Figure 15. Erosion Along Crest of Dredge Cell, 2009

Figure 16. Erosion Along Crest of Dredge Cell, 2009

3.14. Record Drawings

During Stantec's review of the available documents, historical as-constructed drawings were reviewed. The reviewed drawings included a June, 1952 drawing of Ash Disposal Area No. 1, a March, 1959 drawing of Ash Disposal Area No. 2, an October, 1973 drawing of the Limestone and Ash Disposal Area, and a January, 1978 drawing of the Chemical Treatment Ponds.

3.15. O&M Manual

Stantec is unaware of any operations and maintenance manual for the Ash Pond Complex. Geotechnical studies of the ash pond complex conducted by Singleton Materials Engineering Laboratory in 1983 and TVA memorandum documents dated in 1995 and 1996 include operational recommendations, but it is understood a manual has not been published. TVA conducts annual site inspections of all CCP facilities at the Widows Creek Fossil Plant, including the Ash Pond Complex, and this information is used to identify and assess any potential problems. The annual reports are discussed in the following section.

3.16. Annual Reports

Since 1967, TVA has performed yearly inspections of the stacks at the Widows Creek Fossil Plant and made subsequent repairs based on the observed conditions. These inspections verify that the design plan is being followed, as well as identify any potential problem areas such as the observance of excessive seepage or surficial sloughing along the stack. The inspections also verify that the previous year's recommendations were carried out and present new recommendations based on the observed conditions. Stantec reviewed these annual reports to identify recurring issues observed at the Ash Pond Complex.

3.17. Underdrain System

At this time, no underdrain system is installed at the Dredge Cell or around the Main Ash Pond A. A conceptual plan has been developed for an underdrain system at the Dredge Cell as discussed in Section 3.12 above. This plan calls for 12" lateral perforated HDPE pipes which wrap around the Dredge Cell to collect seepage water and 12" solid HDPE outlet pipes. The 12" diameter outlet pipes discharge into a 15" solid HDPE collector pipe which traverses around the toe and discharges into approved outfalls.

3.18. Interim Risk Reduction Measure - Seepage Control

As discussed above, excessive seepage and high pore water pressures have been reported around the southern embankment of the Bottom Ash Stack near the Chemical Ponds. As a result of the excessive pore pressures, a slurry cutoff wall was installed in 1984 and ongoing seepage monitoring has been performed by TVA. This area continues to be a recurring problem, as noted in the annual inspections and as observed by Stantec during the 2009 field exploration. A mitigation work plan is currently under development to control the observed seepage using a graded media filter and rock toe buttress, which will increase the slope stability and control drainage in the area. The as-found site conditions for this area are shown in Figures 17 through 21.

Figure 17. Ditch/Seepage Conditions along Scrubber Road, 2009

Figure 18. Ditch/Seepage Conditions along Scrubber Road, 2009

Figure 19. Ditch/Seepage Conditions Scrubber Road, 2009

Figure 20. Discharge Pipe along Scrubber Road, 2009

Figure 21. Seepage/Excessive Pore Pressures along Scrubber Road, 2009

4. Scope of Work

Immediately following the Gypsum Stack uncontrolled dewatering event on January 9, 2009, TVA requested Stantec to mobilize to Widows Creek Fossil Plant and provide 24-hour emergency mitigation and engineering services for the Gypsum Stack. Following the geotechnical drilling for the Gypsum Stack, a general site inspection of the Ash Pond Complex was conducted and Stantec provided TVA with a written scope of work on May 12, 2009 to address TVA's Engineering Service Request (ESR) 909. As the project progressed and the scopes were better defined, Stantec submitted a second addendum for additional work requests on June 30, 2009. The revised addendum included Cone Penetration Test (CPT) borings and Vane Shear test borings, laboratory testing, and engineering analysis.

The fieldwork for the geotechnical exploration was completed in late June, 2009 with the exception of one boring (STN-113) which was drilled by the Limestone Runoff Surge Pond. The scope of work included advancing a total of fifty-three (53) auger sample borings, eight (8) vane shear borings, and nine (9) CPT borings across the site at the approximate locations shown on the attached boring layout in Appendix C. The borings were drilled using a truck-mounted drill rig equipped with 4¼-inch (ID) hollow stem augers and NQ size rock coring equipment. All of the boring locations were staked in the field by Stantec personnel prior to drilling. Continuous standard penetration testing (SPT) were performed in most of the borings advanced at the site, while undisturbed sampling, rock core sampling, cone penetration testing, and vane shear testing were performed in selected locations. Of the fifty-three (53) sample borings a total of thirty-nine (39) were instrumented with the following equipment; two (2) were instrumented with slope inclinometer casing and thirty-seven (37) were instrumented with standpipe piezometers with five-foot slotted screens. The slope inclinometers were installed to monitor any possible slope movement along the south exterior slopes of the Bottom Ash Stack near the Chemical Ponds and the standpipe piezometers were installed to determine the pore water pressures/groundwater levels at selected locations within the embankments. The final surface elevations and as-drilled locations for each boring were determined by TVA's survey crew upon completion of the drilling. Detailed boring logs and piezometer installation records can be found in Appendix B.

An engineer/geologist was present with each drill crew throughout the drilling operations and was responsible for directing the drill crews, logging the subsurface soil/rock materials encountered in each boring, and collecting the soil samples for laboratory testing. The subsurface materials were logged by observing the continuous SPT samples and the auger cuttings as they were conveyed to the surface. Particular attention was given to the texture, color, natural moisture content and consistency of the encountered soils. The bedrock was logged with particular attention to the rock type, color, grain size, hardness, and bedding characteristics. Upon completion of drilling, the borings were checked for the presence of subsurface water and then backfilled with a cement-bentonite grout mix and/or instrumented as indicated above. The recovered soil and rock samples were transported to Stantec's laboratory in Lexington, Kentucky for analyses.

Once the samples arrived in Lexington, selected SPT samples were subjected to laboratory sieve and hydrometer analyses in accordance with the American Society of Testing and Materials (ASTM D 422), No. 200 wash gradation (ASTM D 1140, ASTM C 136), and natural moisture content determinations (ASTM D 2216). Selected undisturbed Shelby tube samples were subjected to laboratory unconsolidated undrained triaxial compression (ASTM D 2850), consolidated undrained triaxial compression with pore pressure measurements (ASTM D 4767), unit weight determination (ASTM D 2166), permeability testing (ASTM D 5084), and natural moisture content determinations (ASTM D 2216). The results of the laboratory testing are described in more detail in Section 7 and summary tables are provided in Appendix I.

The results of the field and laboratory testing services were used to develop critical stability sections along the Dredge Cell, Main Ash Pond A, and Bottom Ash Stack areas. Stantec initially reviewed a total of twenty (20) stability sections (Section A through T) with respect to soil types, observed phreatic elevation, slope geometry, and location to the stack. Based on the results of this initial review, Stantec selected ten (10) sections to analyze (Section A, C, D, H, J, L, M, O, S and T). Stantec then performed seepage and slope stability analyses based on the observed existing conditions for the ten (10) critical sections. The results are presented in Section 9.

5. Results of Geotechnical Exploration

5.1. Summary of Borings

A summary of the boring information is presented in Table 4, where all measurements are expressed in feet. Typed boring logs and piezometer installation records are presented in Appendix B and the CPT results are included in Appendix G.

Table 4. Summary of Borings

Boring Number	Surface Elevation (MSL)	Top of Rock Elevation (MSL)	*Refusal/ Begin Core Elevation (MSL)	Boring Termination Depth (feet)	Length of Rock Core (feet)	Bottom of Hole Elevation (MSL)
STN-62	635.8	598.3	597.9	37.9	-	597.9
STN-63	646.1	-	NR (587.1)	59.0	-	587.1
STN-64	638.1	581.1	579.6	58.5	-	579.6
STN-65	645.0	-	NR (586.0)	59.0	-	586.0
STN-66	606.3	578.9	578.2	28.1	-	578.2
STN-67	647.0	-	NR (600.0)	47.0	-	600.0
STN-68	611.4	585.4	584.4	27.0	-	584.4
STN-69	647.6	-	NR (594.1)	53.5	-	594.1
STN-70	613.3	591.8	590.8	22.5	-	590.8
STN-71	636.6	-	NR (593.6)	43.0	-	593.6
STN-72	636.6	-	NR (596.1)	40.5	-	596.1
STN-73	598.5	588.0	587.5	11.0	-	587.5
STN-74	635.7	584.7	584.2	51.5	-	584.2
STN-75	636.7	582.7	581.7	55.0	-	581.7
STN-76	624.5	575.5	575.0	49.5	-	575.0
STN-77	636.1	586.3	584.6	51.5	-	584.6
STN-78	636.8	-	NR (593.8)	43.0	-	593.8
STN-79	622.8	577.8	577.8	45.0	-	577.8
STN-80	637.2	-	NR (586.2)	51.0	-	586.2
STN-81	625.5	584.5	584.5	41.0	-	584.5
STN-82	636.9	-	NR (587.4)	49.5	-	587.4
STN-83	624.9	585.8	585.8	39.1	-	585.8
STN-84	639.3	-	NR (597.3)	42.0	-	597.3
STN-85	614.6	587.6	586.2	28.4	-	586.2
STN-86	614.6	586.5	586.5	28.1	9.0	577.5
STN-87	637.4	-	NR (595.4)	42.0	-	595.4
STN-88	615.7	590.7	590.4	33.3	8.0	582.4
STN-89	636.3	581.1	581.1	65.8	10.6	570.5
STN-90	613.2	577.7	577.7	47.4	11.9	565.8
STN-91	641.1	-	NR (622.1)	19.0	-	622.1
STN-92	640.1	-	NR (620.6)	19.5	-	620.6
STN-93	641.8	-	NR (587.8)	54.0	-	587.8
STN-94	639.0	583.5	583.3	55.7	-	583.3
STN-95	645.4	-	NR (589.9)	55.5	-	589.9

Table 4. Summary of Borings

Boring Number	Surface Elevation (MSL)	Top of Rock Elevation (MSL)	*Refusal/ Begin Core Elevation (MSL)	Boring Termination Depth (feet)	Length of Rock Core (feet)	Bottom of Hole Elevation (MSL)
STN-96	647.6	-	NR (593.1)	54.5	-	593.1
STN-97	638.6	-	NR (596.6)	42.0	-	596.6
STN-98	602.5	578.0	577.8	24.7	-	577.8
STN-99	638.4	-	NR (590.4)	48.0	-	590.4
STN-100	644.1	-	NR (589.6)	54.5	-	589.6
STN-101	638.8	574.3	574.3	64.5	-	574.3
STN-102	638.7	-	NR (596.7)	42.0	-	596.7
STN-103	638.9	-	NR (593.9)	45.0	-	593.9
STN-104	645.9	599.4	597.9	48.0	-	597.9
STN-105	637.6	594.1	593.8	43.8	-	593.8
STN-106	645.9	-	NR (590.4)	55.5	-	590.4
STN-107	636.7	582.7	582.2	54.5	-	582.2
STN-108	601.5	576.5	576.0	25.5	-	576.0
STN-109	603.3	577.0	577.0	26.3	-	577.0
STN-110	606.5	578.0	578.0	28.5	-	578.0
STN-111	604.9	577.9	577.4	27.5	-	577.4
STN-112	604.2	576.7	575.2	29.0	-	575.2
STN-113	615.0	563.0	563.0	52.0	-	563.0
**CPT-1	636.6	-	587.6	49.0	-	587.6
**CPT-2	636.7	-	584.8	51.9	-	584.8
**CPT-3	636.0	-	592.0	44.0	-	592.0
**CPT-4	636.4	-	592.3	44.1	-	592.3
**CPT-5	642.4	-	615.1	27.3	-	615.1
**CPT-6	647.3	-	616.7	30.6	-	616.7
**CPT-7	638.2	-	603.9	34.3	-	603.9
**CPT-8	646.5	-	602.7	43.8	-	602.7
**CPT-9	604.3	-	579.6	24.7	-	579.6
***V-1	636.7	-	NR (604.7)	32.0	-	604.7
***V-2	636.4	-	NR (609.4)	27.0	-	609.4
***V-3	647.3	-	NR (621.0)	26.3	-	621.0
***V-4	645.9	-	NR (593.9)	52.0	-	593.9
***V-5	647.6	-	NR (610.6)	37.0	-	610.6
***V-6	645.0	-	NR (598.0)	47.0	-	598.0
***V-7	601.5	-	NR (579.5)	22.0	-	579.5
***V-8	646.1	-	NR (594.1)	52.0	-	594.1

*	Refusal, as used herein, refers to rock-like resistance to the advancement of the augers using a carbide-tipped-tooth bit. This may indicate the beginning of weathered bedrock, boulders, or rock remnants. An exact determination cannot be made without performing rock coring.
**	Denotes cone penetration test (CPT) boring with assumed surface elevation.
***	Denotes vane shear (V) boring with assumed surface elevation.
NR	No Refusal

5.2. Subsurface Soil Conditions

Based on the results of the drilling program, the Ash Pond Complex is underlain by five predominant soil types: Soil 1 – Silty Sand with Gravel (Bottom Ash), Soil 2 – Silt with Sand (Bottom Ash), Soil 3 – Fill Lean Clay with Sand, Soil 4 – Residual Lean to Fat Clay with Sand and Gravel, and Soil 5 - Silty Sand (Fly Ash).

The first soil type, Soil 1, was classified as Silty Sand with Gravel (Bottom Ash), gray to black in color, moist in natural moisture content, with medium to coarse grain sizes, loose to medium dense in density, with zones silty and a mix of ash and coal cinders. Soil 1 was encountered in thirty-one (31) of the borings. The average thickness of Soil 1 was 12 feet and the maximum thickness observed was 38.5 feet at STN-93.

The second soil, Soil 2, was classified as Silt with Sand (Bottom Ash), light gray to black in color, moist to damp in natural moisture content, fine to medium in grain size, loose to very dense in density, with occasional coal cinders. Soil 2 was encountered in fifteen (15) of the borings and had an average thickness of 14 feet.

The third predominant soil type, Soil 3, was classified as Fill: Lean to Fat Clay with Sand, tan to red in color, moist in natural moisture content, medium stiff to stiff in consistency, with numerous coal fragments, ash lenses, and chert fragments. Soil 3 was encountered in thirty-six (36) of the borings and an average thickness of 17 feet and a maximum thickness of 41.5 feet.

The fourth soil, Soil 4, was classified as Residual Lean to Fat Clay with Sand and Gravel, tan to red in color, moist in natural moisture content, medium stiff to very stiff in consistency, with chert fragments and occasional manganese concretions. Soil 4 was encountered in forty-eight (48) of the borings and had an average thickness of 14 feet and maximum recorded thickness of 33 feet. However, all of the borings were not extended to the top of rock and therefore the residual clay layer may be thicker then what was recorded during this exploration.

The fifth soil, Soil 5, was classified as Silty Sand (Fly Ash), light gray to black in color, moist to wet in natural moisture content, fine to medium grained, loose to medium dense in density, with occasion coal cinders/fragments. Soil 5 was encountered in ten (10) of the borings and averaged 21 feet in thickness.

Six additional soil types were encountered within the borings drilled during this exploration; however these soil types were noted in only a few borings and occurred at sporadic location throughout the ash pond complex. The remaining soil types encountered were visually described as follow: Soil 6 - Crushed Limestone Fill; Soil 7 - Lean Clay, tan to red, moist, medium stiff to very stiff, with chert fragments and occasional manganese concretions; Soil 8 - Residual Sandy Lean Clay, tan to red, moist, medium stiff to very stiff, with chert fragments and occasional manganese concretions; Soil 9 - Silty Sand, brown to gray, wet, fine grained; Soil 10 - Sand with Gravel. Soil 11 - Silt, light gray to black, moist to wet, loose to medium dense.

5.3. Standard Penetration Tests

A total of fifty-one (51) borings were sampled with standard penetration tests (SPT) at the approximate locations shown on the boring layout in Appendix C and at the depths indicated on the boring logs in Appendix B. The SPT sampling was performed in accordance with the procedures outlined in ASTM D 1586, "Penetration Test and Split Barrel Sampling of Soils". This method is typically used to obtain soil samples, estimate the consistency or relative density of the soil, and also to estimate the vertical limits of the subsurface soil horizons. A summary of the average blow counts for the four predominant soil horizons encountered in each of the borings is presented in Appendix I where all measurements are expressed in feet. The N values have also been corrected due to overburden and hammer efficiency and estimates of unit wet weights along with friction angle estimates are included in the SPT correlation tables in Appendix D.

Based on the average N-values presented in Appendix I, the two Bottom Ash soils appear to be dense, with the Silty Sand with Gravel (Bottom Ash) having a higher average N-value than the Silt with Sand (Bottom Ash) (24 and 16 respectively). Both the Fill Clay and Residual Clay have similar N-values of 16 and 19, respectively. The Fly Ash appears to be the weakest soil horizon, with an average N-value of 8. It should be noted that in numerous borings, the Fly Ash soil horizon had an N-value of 0. Based on the drilling information, the Silt with Sand (Bottom Ash) and the Fly Ash zone are most likely the historical pond layers which have subsequently been developed over as the stack height has increased throughout the years.

5.4. Undisturbed Sampling

A total of one-hundred twelve (112) undisturbed Shelby tube samples were obtained from select borings drilled during the exploration. The undisturbed samples were retrieved from the borings via a 30-inch long thin walled tube piston sampler, which measured 2 7/8-inches inside diameter. The undisturbed soil samples were performed in general accordance with the procedures outlined in ASTM D-1587, “Standard Practice for Thin-walled Tube Sampling of Soils for Geotechnical Purposes.” All of the Shelby tube samples were sealed in the field by Stantec representatives and transported to Stantec’s Lexington, Kentucky office. A summary of the undisturbed samples retrieved from the site is presented in Table 5, where all measurements are expressed in feet. For a more detailed description, see the boring logs in Appendix B and the laboratory test results in Appendix F.

Table 5. Summary of Undisturbed Shelby Tube Samples

Boring Location	Boring Number	Sample Depth, ft	Recovery Length, ft
Main Ash Pond A	STN-72	18.0-20.0	1.4
	STN-74	9.0-11.0	2
		24.5-26.5	2
		40.0-42.0	2
	STN-75	25.5-27.5	1.7
		35.0-37.0	1.8
	STN-77	21.0-23.0	2
		38.0-40.0	2
	STN-78	10.5-12.5	1.7
		25.5-27.5	0
		36.5-38.5	1.4
	STN-79	15.0-17.0	1.8
	V-1	10.0-12.0	2
		20.0-22.0	2
		30.0-32.0	2
	V-2	5.0-7.0	2
		15.0-17.0	2
		25.0-27.0	2
Old Scrubber Sludge Pond	STN-69	38.0-40.0	0
	STN-71	30.0-40.0	1.4
		39.5-41.5	1.7
	STN-93	30.0-32.0	2
		39.5-41.5	0
		49.0-51.0	2
	STN-94	15.0-17.0	2
		24.5-26.5	2
		49.0-51.0	2
	STN-103	19.5-21.5	1.5
		29.0-31.0	1.2
		40.0-42.0	2
	STN-104	24.0-26.0	0
		35.0-37.0	0
		44.5-46.5	0.6
	STN-105	30.0-32.0	1.5
		39.5-41.5	2
	STN-106	24.0-26.0	1.7
		35.0-37.0	0
		50.5-52.5	0.6
	STN-107	30.0-32.0	2
		39.5-41.5	1.7
		49.0-51.0	0.9
Upper Stilling Pond	STN-89	10.0-12.0	0
		15.0-17.0	2
		20.0-22.0	2
		40.0-42.0	1.8
Lower Stilling Pond	STN-90	5.0-7.0	1
		10.0-12.0	2
		20.0-22.0	0.8
Old Scrubber Sludge Pond Dredge Cell	STN-65	10.0-12.0	0.8
		20.0-22.0	0
		32.0-34.0	1.2
		45.0-47.0	2
	STN-67	9.0-11.0	1.5
		20.0-22.0	1
		40.5-42.5	1.7
	STN-95	25.5-27.5	1.9
		30.5-32.5	1.3
		35.5-37.5	0.6
		40.5-42.5	
		45.5-47.5	0.9
		50.5-52.5	2
	STN-96	25.5-27.5	2
		35.0-37.0	2
		44.5-46.5	0
		52.5-54.5	2
	STN-97	19.5-21.5	1.9
		29.0-31.0	1.9
		40.0-42.0	2
	STN-98	18.0-20.0	2
	STN-99	19.5-21.5	1.5
		29.0-31.0	2
		40.0-42.0	2
	STN-100	30.0-32.0	2
		33.5-35.5	2
		44.5-46.5	1.5
		49.5-51.5	2
	STN-101	19.5-21.5	1.5
		29.0-31.0	1.8
		40.0-42.0	2
	STN-102	19.5-21.5	1.8
		29.0-31.0	1.5
		40.0-42.0	0.5

Table 5. Summary of Undisturbed Shelby Tube Samples

Boring Location	Boring Number	Sample Depth, ft	Recovery Length, ft
	V-4	20.0-22.0	1
		50.0-52.0	2
	V-5	20.0-22.0	2
		30.0-32.0	1
Bottom Ash Stack	STN-80	10.5-12.5	1.5
		20.0-22.0	0.9
		28.0-30.0	1.5
		30.0-32.0	2
		35.0-37.0	1.8
		47.5-49.5	1.8
	STN-81	25.5-27.5	0
	STN-82	7.5-9.5	1.4
		20.0-22.0	1.5
		40.0-42.0	2
	STN-83	10.0-12.0	1.1
	STN-84	10.0-12.0	1.5
		20.0-22.0	1.5
		40.0-42.0	2
	STN-86	15.0-17.0	2
		24.0-26.0	2
	STN-87	37.0-39.0	1
	V-3	5.0-7.0	2
		15.0-17.0	2
		25.0-27.0	0.8

Boring Location	Boring Number	Sample Depth, ft	Recovery Length, ft
Old Scrubber Sludge Pond Dredge Cell	STN-108	10.5-12.5	2
		15.5-17.5	2
	V-6	25.0-27.0	0.5
	V-7	20.0-22.0	2
	V-8	45.0-47.0	1.1
		50.0-52.0	2

5.5. Vane Shear Testing

In June 2009, eight vane shear borings were advanced around the Ash Pond Complex (see Boring Layout in Appendix C). Three vane shear tests were conducted within each boring and undisturbed Shelby tube samples were performed at selected depths for comparison testing with Unconsolidated Undrained Triaxial Test Results in Section 7.3.3. The field testing was conducted in an attempt to better define the in-situ undrained shear strengths of the Bottom Ash and Fly Ash materials. A summary of the vane shear testing results are provided in Table 6 below.

Table 6. Summary of Vane Sheer Testing

Boring No.	Soil Horizon	Test Elevation (feet)	Maximum Measured Torque (lbs)	Vane Diameter Size (in)	Undrained Shear Strength (psf)	Residual Shear Strength (psf)	Sensitivity
V-1	Silt with Sand (Bottom Ash)	626.0	270	2.492	698	168	4.15
	Silt with Sand (Bottom Ash)	614.8	210	2.492	543	271	2.00
	Silt with Sand (Bottom Ash)	605.3	180	2.492	465	129	3.60
V-2	Silt with Sand (Bottom Ash)	631.4	445	2.031	2,209	496	4.45
	Silt with Sand (Bottom Ash)	621.4	140	2.031	695	298	2.33
	Silt with Sand (Bottom Ash)	611.4	160	2.031	794	248	3.20
V-3	Silt with Sand (Bottom Ash)	631.0	450	2.031	2,234	496	4.50
	Silt with Sand (Bottom Ash)	620.1	145	2.031	720	273	2.64
	Silt with Sand (Bottom Ash)	611.0	295	2.031	1,464	347	4.21
V-4	Silty Sand (Fly Ash)	625.9	270	2.031	1,340	496	2.70
	Silty Sand (Fly Ash)	615.9	220	2.492	569	207	2.75
	Silty Sand (Fly Ash)	605.7	320	2.492	827	259	3.20
	Silty Sand (Fly Ash)	595.8	460	2.031	2,283	745	3.07
V-5	Silty Sand (Fly Ash)	627.6	370	2.492	957	297	3.22
	Silty Sand (Fly Ash)	617.6	320	2.492	827	194	4.27
	Silty Sand (Fly Ash)	612.6	320	2.492	827	129	6.40
V-6	Silty Sand (Fly Ash)	620.0	190	2.492	491	103	4.75
	Silty Sand (Fly Ash)	610.0	370	2.492	957	233	4.11
	Silty Sand (Fly Ash)	600.0	350	2.492	905	259	3.50
V-8	Silty Sand (Fly Ash)	621.0	570	2.492	1,474	465	3.17
	Silty Sand (Fly Ash)	611.1	220	2.031	1,092	745	1.47
	Silty Sand (Fly Ash)	601.1	360	2.492	931	310	3.00

5.6. CPT Testing

Nine Cone Penetration Test (CPT) borings were performed at the Ash Pond Complex. Four (4) were performed along the north side of the Main Ash Pond A and two (2) along the north side of the Bottom Ash Stack. Three additional CPT's were performed around the Old Scrubber Sludge Pond at the approximate locations shown on the boring layout in Appendix C. The CPT testing was performed in general accordance with ASTM Standard D 5778 "Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils." The test involved advancing an integrated electronic seismic piezocone within the overburden materials to measure tip resistance, sleeve friction and dynamic pore pressure at roughly one-inch intervals. In addition, pore pressure dissipation testing and seismic testing was performed at selected intervals.

As noted on the boring layout, CPT borings were performed near previously drilled geotechnical sample borings. The previous sample logs were used to predict the depths for each soil horizon as the cone penetration testing was being performed. Due to the stiff soils located directly beneath the access roads, four of the CPT borings (CPT-5, CPT-7, CPT-8, and CPT-9) required pre-augering before the cone could be pushed through the softer ash materials. The required pre-auger depths ranged from 10 to 25 feet.

The CPT borings also provided a continuous readout of the undrained shear strength parameters and effective phi angles for each soil horizon. The strength values were plotted versus depth along with the Vane Shear test results to determine an empirical N_{kt} factor for the given soil profile (see results in Appendix G). It should be noted that significant engineering judgment is required in interpreting the shear strength data as the material exhibits a dilatency during the application of shearing strain. The dilatency results in the development of negative pore pressures at the tip of the vane thus affecting the undrained shear strength by some unknown amount. A more in depth discussion explaining the methods/procedures used for the CPT testing have been attached to this report along with the CPT logs in Appendix G.

5.7. Rock Core Samples

A total of thirty (30) borings were extended to auger refusal and four (4) of the borings (STN-86, STN-88, STN-89 and STN-90) were extended approximately ten feet into the underlying bedrock. The apparent top of rock elevation ranged from 563.0 feet in boring STN-113 to 599.4 feet in boring STN-104.

The rock core samples collected from the geotechnical borings show the underlying bedrock to consist of limestone with dolomitic zones. The limestone was described as gray in color, fine to micro-crystalline grained, thin to medium bedded, hard, with shale stringers and occasional argillaceous zones. The bedrock encountered at the site correlates well with the Sequatchie Formation, Nashville Group, and Stones River Group previously described in the Gypsum Stack report dated November 6, 2009. A detailed description of the rock core samples, including the base of weathered rock, is presented on the geotechnical logs in Appendix B.

6. Field Instrumentation

A total of thirty-nine (39) borings were instrumented with slope inclinometers and/or piezometers to monitor possible slope movement and determine the pore water pressures/groundwater levels at selected locations within the embankments of the Ash Pond Complex. The data collected is summarized below.

6.1. Piezometers

A total of thirty-seven (37) borings were instrumented with casagrande-type piezometers to monitor pore pressures at the specific depths shown on the piezometer installation records in Appendix B. Plan locations of the piezometers can be found on the boring layout in Appendix C.

In general, the slotted PVC piezometer screen (5 feet in length) was surrounded by a sand filter pack which extended approximately two feet above the upper most opening and two feet below the bottom of the screen. After, placing the sand filter pack, a two-foot thick layer of bentonite was placed on top of the sand to seal the filter zone. Next, the annulus of the borehole was grouted up to the surface with a bentonite and Portland cement mix. Finally, at the ground surface the piezometer was protected with a steel flush mount or riser type protective cover and a concrete pad was installed around the piezometer well. It should be noted that one of the borings (STN-83) was instrumented with double piezometers to monitor two separate sensing zones. The sensing zones monitored during this exploration are listed below in Table 7 and the piezometer installation records have been included in Appendix B. The graphical results of the piezometer readings have been included in Appendix H.

Table 7. Summary of Piezometers

Location	Soil Horizon	Boring Number	Sensing Depth Interval (feet)	Sensing Elevation Interval (feet)
Main Ash Pond A	Residual Clay	STN-72	35.5-40.5	601.1 - 596.1
	Residual Clay	STN-73	5.2-10.2	593.3 - 588.3
	Residual Clay	STN-75	49.8-54.8	586.9 - 581.9
	Residual Clay	STN-76	44.0-49.0	580.5 - 575.5
	Residual Clay	STN-78	32.0-37.0	604.8 - 599.8
	Residual Clay	STN-79	40.0-45.0	583.8 - 577.8
Old Scrubber Sludge Pond	Silty Sand (Fly Ash)	STN-69	25.5-30.5	622.1 - 617.1
	Residual Clay	STN-70	19.5-21.5	593.7 - 591.7
	Silt with Sand (Bottom Ash)	STN-93	20.0-25.0	621.8 - 616.8
	Residual Clay	STN-94	26.0-31.0	613.0 - 608.0
	Fill Clay	STN-103	35.0-40.0	603.9 - 598.9
	Silt with Sand (Bottom Ash)	STN-104	22.0-27.0	623.9 - 618.9
	Residual Clay	STN-105	27.5-32.5	610.1 - 605.1
	Fly Ash (Silt)	STN-106	22.5-27.5	623.4 - 618.4
	Fill Clay	STN-107	25.0-30.0	611.7 - 606.7
Bottom Ash Stack	Silt with Sand (Bottom Ash)	STN-62	18.0-23.0	617.8 - 612.8
	Silt with Sand (Bottom Ash)	STN-80	27.5-32.5	609.7 - 604.7
	Residual Clay	STN-81	28.5-33.5	597.0 - 592.0
	Silt with Sand (Bottom Ash)	STN-82	33.0-38.0	603.9 - 598.9
	Silt with Sand (Bottom Ash)	STN-83	13.0-18.0	611.9 - 606.9
	Silt with Sand (Bottom Ash)	STN-83	22.0-27.0	602.9 - 597.9
	Silt with Sand (Bottom Ash)	STN-84	23.0-28.0	616.3 - 611.3
	Residual Clay	STN-85	22.7-27.7	591.9 - 586.9
	Silt with Sand (Bottom Ash)	STN-87	25.0-30.0	612.4 - 607.4
Old Scrubber Sludge Pond Dredge Cell	Silty Sand (Fly Ash)	STN-63	31.0-36.0	615.1 - 610.1
	Residual Clay	STN-64	30.0-35.0	608.1 - 603.1
	Silty Sand (Fly Ash)	STN-65	25.5 - 30.5	619.5 - 614.5
	Residual Clay	STN-66	22.5-27.5	583.9 - 578.9
	Silty Sand (Fly Ash)	STN-67	30.0-35.0	617.0 - 612.0
	Residual Clay	STN-68	22.2-25.2	589.2 - 586.2

Table 7. Summary of Piezometers

Location	Soil Horizon	Boring Number	Sensing Depth Interval (feet)	Sensing Elevation Interval (feet)
Old Scrubber Sludge Pond Dredge Cell	Silt with Sand (Bottom Ash)	STN-96	33.5-38.5	614.1 - 609.1
	Fill Clay	STN-97	17.0-22.0	621.6 - 616.6
	Residual Clay	STN-98	15.5-20.5	587.0 - 582.0
	Fill Clay	STN-99	20.0-25.0	618.4 - 613.4
	Silty Sand (Fly Ash)	STN-100	31.0-36.0	613.1 - 608.1
	Fill Clay	STN-101	20.0-25.0	618.8 - 613.8
	Fill Clay	STN-102	20.0-25.0	618.7 - 613.7
	Residual Clay	STN-108	20.5-25.5	581.0 - 576.0

A total of six monitoring trips have been performed over the period from May 27, 2009 through October 22, 2009. A summary table of the piezometer readings is presented in Appendix H.

The piezometers were also tested in the field to determine the horizontal in-situ hydraulic conductivity of the underlying soil horizons. The tests were performed in general accordance with ASTM D 4044 “Test Method for (Field Procedure) for Instantaneous Change in Head (Slug) Tests for Determining Hydraulic Properties of Aquifers” and the results are shown in Table 8 and presented in Appendix G.

Table 8. Summary of Instantaneous Change in Head Test Results

Soil Horizon	Location	Boring Number	*K_h (cm/sec)
Silty Sand with Gravel (Bottom Ash)	Bottom Ash Stack	STN-62	4.72E-04
		STN-84	1.42E-04
		STN-83 L	2.35E-04
		STN-83 U	5.87E-04
	Old Scrubber Sludge Pond	STN-93	2.83E-04
Silt with Sand (Bottom Ash)	Bottom Ash Stack	STN-87	5.76E-03
		STN-80	1.89E-04
		STN-82	3.67E-06
	Old Scrubber Sludge Pond Dredge Cell	STN-96	4.97E-05
	Old Scrubber Sludge Pond	STN-104	2.30E-04
Fill Lean to Fat Clay with Sand	Main Ash Pond A	STN-78	1.75E-05
	Old Scrubber Sludge Pond Dredge Cell	STN-97	3.43E-05
		STN-99	NA
		STN-101	4.97E-07
		STN-102	3.72E-05
	Old Scrubber Sludge Pond	STN-103	2.69E-05
		STN-107	1.84E-06

Table 8. Summary of Instantaneous Change in Head Test Results

Soil Horizon	Location	Boring Number	*K_h (cm/sec)
Residual Lean to Fat Clay with Sand and Gravel	Bottom Ash Stack	STN-81	3.68E-04
		STN-85	1.34E-04
	Old Scrubber Sludge Pond Dredge Cell	STN-108	3.31E-04
		STN-66	3.58E-04
		STN-64	8.63E-08
		STN-68	3.36E-04
		STN-98	1.29E-04
	Main Ash Pond A	STN-72	1.32E-06
		STN-73	8.12E-07
		STN-75	3.49E-04
		STN-76	2.38E-06
		STN-79	2.88E-04
	Old Scrubber Sludge Pond	STN-70	1.59E-06
		STN-94	NA
		STN-105	9.72E-04
Silty Sand (Fly Ash)	Old Scrubber Sludge Pond Dredge Cell	STN-63	7.43E-05
		STN-65	5.94E-05
		STN-67	6.01E-05
		STN-100	1.36E-04
	Old Scrubber Sludge Pond	STN-69	NA
		STN-106	2.30E-05

* The k_h values presented above were based on an assumed k_h/k_v value of 1. The reported in-situ slug test values were then compared to laboratory Falling Head Permeability testing presented below in Section 7.3.4. The results of the comparison were then used to estimate an average k_h/k_v value for the given soil horizon. The result of the analysis is presented below in Table 18 (Seepage Parameters).
NA = Not Applicable

6.2. Slope Inclinometers

Two of the borings (STN-86 and STN-88) were instrumented with 2.75-inch OD slope inclinometer casing which was extended approximately ten feet into bedrock to monitor the southern slope between the Copper Pond and the Old Scrubber Road. A total of five readings have currently been performed at the site and were conducted from June 15, 2009 to October 22, 2009.

As of October 22, 2009, SI-88 shows only initial settling/shifting of the casing on the order of ½ inch. After settling, little to no movement has been observed for four months (first reading 6-15-09).

Likewise, SI-86 also showed initial movement due to the installation/construction process. However, small cumulative displacements have continued in the upper five feet of the casing between 614.6 feet (ground surface) and 610.0 feet in elevation which may indicate a shear plane. At this time, ¼ inch of movement (after the initial settling) has been recorded at the top of the casing. The displacement curves for the slope inclinometers are presented in Appendix E and the maximum displacement observed for each of the slope inclinometers is plotted on the respective cross section in Appendix C.

6.3. Measured Water Levels

A total of thirty-seven (37) piezometers were installed during the geotechnical exploration between April 27, 2009 and June 24, 2009. Since installation, each piezometer has been read monthly and the results have been input into a spread sheet to monitor the pore pressures readings and establish a baseline reading for the phreatic water level. A summary of the average water elevation in each piezometer is presented below in Table 9.

Table 9. Average Piezometer Water Elevations

Location	Boring Number	Number of Readings	Ground Elevation (MSL)	Average Water Elevation (MSL)	Average Depth to Water (feet)
Main Ash Pond A	STN-72	7	636.6	619.1	17.5
	STN-73	7	598.5	595.0	3.5
	STN-75	7	636.7	607.8	28.9
	STN-76	7	624.5	605.5	19.0
	STN-78	7	636.8	631.4	5.4
	STN-79	7	622.8	604.2	18.6
Old Scrubber Sludge Pond Dredge Cell	STN-69	8	647.6	636.0	11.6
	STN-70	8	613.2	600.1	13.1
	STN-93	5	641.8	634.0	7.8
	STN-94	1	639.0	632.6	6.4
	STN-103	7	638.9	615.8	23.1
	STN-104	7	645.9	634.8	11.1
	STN-105	5	637.6	633.0	4.6
	STN-106	7	645.9	636.7	9.2
	STN-107	5	636.7	632.3	4.4
Bottom Ash Stack	STN-62	10	635.8	623.6	12.2
	STN-80	7	637.2	632.1	5.1
	STN-81	7	625.5	613.3	12.2
	STN-82	7	636.9	612.0	24.9
	*STN-83U	10	624.9	612.4	12.5
	*STN-83L	10	624.9	612.4	12.5
	STN-84	10	639.3	618.1	21.2
	**STN-85	8	614.6	616.5	-1.9
	STN-87	10	637.4	621.9	15.5
Old Scrubber Sludge Pond Dredge Cell	STN-63	9	646.1	635.9	10.2
	STN-64	8	638.1	635.1	3.0
	STN-65	8	645.0	636.2	8.8
	STN-66	8	606.3	600.9	5.4
	STN-67	8	647.0	632.3	14.7
	STN-68	8	611.4	600.1	11.3

Table 9. Average Piezometer Water Elevations

Location	Boring Number	Number of Readings	Ground Elevation (MSL)	Average Water Elevation (MSL)	Average Depth to Water (feet)
Old Scrubber Sludge Pond Dredge Cell	STN-96	5	647.6	635.8	11.8
	STN-97	5	638.6	628.8	9.8
	STN-98	7	602.5	597.1	5.4
	STN-99	4	638.4	628.9	9.5
	STN-100	7	644.1	634.0	10.1
	STN-101	7	638.8	629.6	9.2
	STN-102	7	638.7	630.9	7.8
	STN-108	3	601.5	597.3	4.2

Notes: *Same Soil Horizon, Silty Sand with Gravel (Bottom Ash)
**Artesian flow

The phreatic levels measured in the piezometers were also compared to the estimated pool elevations for the surrounding ponds in an attempt to develop a hydraulic pattern between the pool elevation and the piezometer levels. However, since measured pool elevations were not available at the time of this report, the comparisons are for illustrative purposes, only.

7. Laboratory Testing and Analysis

7.1. Introduction

Laboratory testing was performed on selected disturbed split spoon samples, bulk bag samples, and undisturbed Shelby tube samples to gain a better understanding of the soil properties and strength parameters for the identified soil horizons at the Ash Pond Complex. The results of the lab testing were also compared to historical data to aid in selecting representative strength parameters. The laboratory data sheets for the tests performed are provided in Appendix F.

7.2. Testing of Standard Penetration Test (SPT) Samples

Recovered soil specimens from SPT samples were subjected to natural moisture content determinations and select samples were combined for sieve and hydrometer analyses. The results of the classification testing were then used in conjunction with the N-values from the SPT's to estimate soil strengths based on published correlations tables. The results of the moisture content tests are included on the boring logs in Appendix B and the SPT correlation tables are provided in Appendix D.

7.2.1. Natural Moisture Content

The natural moisture content of the split-spoon samples was determined in accordance with ASTM D 2216. This information was used in the SPT correlation spreadsheet to determine in-situ unit weights, corrected blow counts, and effective angle of internal friction. The results from the natural moisture content testing are presented in Appendix F.

7.2.2. Particle Size Distribution and Fines Content

Particle size distribution tests were conducted on forty-six (46) composite SPT samples and seventeen (17) bulk bag samples. In general, three to four SPT samples of similar soil type were combined and the particle size distribution test was performed on the composite sample. The test was conducted in accordance with ASTM D 422, "Particle Size Analysis of Soils." The gradation tests were performed on the predominant soil types to supplement the visual classifications made by the engineer/geologist in the field. The gradation curves from the particle size distribution tests are presented in Appendix F.

For each particle size distribution test, the specific gravity of the soil was also reported. The specific gravity testing was conducted in accordance with ASTM D 854 "Specific Gravity of Soils". The average specific gravity of each soil was then used to calculate the void ratio and soil porosity for each sample tested. The average specific gravity for each soil type is presented in Table 10 below.

Table 10. Average Specific Gravity per Soil Type

Soil Type	Specific Gravity
Silty Sand with Gravel (Bottom Ash)	2.48
Silt with Sand (Bottom Ash)	2.32
Fill Lean to Fat Clay with Sand	2.73
Residual Lean to Fat Clay with Sand and Gravel	2.72
Silty Sand (Fly Ash)	2.53

7.2.3. Classification Testing

Soil classification testing consisting of Atterberg Limits (ASTM D 4318), particle-size analysis (ASTM D 422), and specific gravity (ASTM D 854) were performed on selected disturbed standard penetration test samples and disturbed bulk bag samples. These tests are used specifically for classifying the different soil strata. The results can be found in Appendix I

7.3. Testing of Cohesive Soils/Undisturbed (Shelby) Tube Testing

Thirty-three (33) of the fifty-two (52) borings drilled for the Ash Pond Complex included undisturbed (Shelby) tube soil sampling with a 3-inch diameter fixed-head piston sampler. The undisturbed samples were obtained within all five of the predominant soil horizons and transported back to Stantec's laboratory in Lexington, Kentucky. Here the samples were extruded and trimmed into six-inch long specimens. Lab personnel performed visual inspections of the soil samples, unit weights (wet and dry), and natural moisture for each six-inch specimen prior to submitting a summary report of the extruded specimens to the geotechnical engineer for assigning lab testing.

Based on the extrusion logs and careful examinations, an engineer selected which undisturbed specimens would be subjected to consolidated-undrained (CU) triaxial testing, unconsolidated-undrained (UU) triaxial testing, and permeability testing. The results of these tests are included in Appendix F and discussed below.

7.3.1. Dry and Wet Unit Weights

A total of one-hundred and seven (107) unit weights were determined from approximately eighty-two (82) Shelby tube samples. Both the wet unit weight and dry unit weight were determined for each sample, as well as the moisture content of the sample. The unit weights of the different soil types can vary significantly depending on the method of deposition and location around the ponds. Because of this, average unit weights per location were calculated and are presented in Table 11 below.

Table 11. Average Unit Weights per Location

Location	Average Center Elevation (feet)	Natural Moisture Content (%) @ 40°C	Average Wet Unit Weight (pcf)	Average Dry Density (pcf)	Average Void Ratio (e)	Average Porosity (n)
Main Ash Pond A	608.9	20.0	127.3	106.5	0.6	0.6
Bottom Ash Stack	606.2	27.8	116.4	91.6	0.8	0.6
Old Scrubber Sludge Pond	605.6	24.2	124.1	100.4	0.7	0.6
Old Scrubber Sludge Pond Dredge Cell	606.6	34.9	117.1	89.8	0.9	0.6
Lower Stilling Pond	601.2	28.7	123.8	100.9	0.7	0.6
Upper Stilling Pond	606.8	22.4	127.7	104.6	0.6	0.7

Stantec also reviewed the unit weights per soil type encountered at the site, the results of this analysis are presented in Table 12. For a complete list of all the unit weights tested at the Ash Pond Complex, please see Appendix I.

Table 12. Average Unit Weights per Soil Type

Soil Horizon	Horizon Center Elevation (feet)	Natural Moisture Content (%) @ 40°C	Average Wet Unit Weight (pcf)	Average Dry Density (pcf)	Average Void Ratio, (e)	Average Porosity, (n)
Silty Sand with Gravel (Bottom Ash)	616.6	28.4	118.6	92.6	0.9	0.5
Silt with Sand (Bottom Ash)	608.3	29.8	112.2	87.4	0.8	0.6
Fill Lean to Fat Clay with Sand	614.4	23.0	125.4	101.9	0.7	0.6
Residual Lean to Fat Clay with Sand and Gravel	596.3	24.7	124.9	100.5	0.7	0.6
Silty Sand (Fly Ash)	609.0	47.1	112.5	82.9	1.1	0.5

7.3.2. Consolidated-Undrained (CU) Triaxial Testing

Consolidated-Undrained triaxial compression tests were performed on thirty-eight (38) selected undisturbed Shelby tube samples. Each sample was tested in accordance with ASTM Standard D 4767. During the shear test, the axial load, vertical strain, cell pressure, and pore pressures were continuously monitored and electronically recorded.

Table 13 summarizes the results of the triaxial compression tests based on soil types. The average effective angle of internal friction (ϕ') and average effective cohesion (c') was determined for each type of soil. A detailed table showing each sample tested has been included in Appendix I.

Table 13. Consolidated Undrained Triaxial Test Results per Soil Type

Soil Type	Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	c' (psf)	ϕ' (deg.)
Silty Sand with Gravel (Bottom Ash)	NA	NA	NA	NA
Silt with Sand (Bottom Ash)	106.4	78.8	388	33.4
Fill Lean to Fat Clay with Sand	124.6	100.0	388	29.6
Residual Lean to Fat Clay with Sand and Gravel	125.0	104.2	420	29.3
Silty Sand (Fly Ash)	110.3	78.3	149	37.0

NA = Not Applicable

7.3.3. Unconsolidated-Undrained (UU) Triaxial Testing

Unconsolidated-Undrained triaxial compression tests were performed on selected undisturbed Shelby tube samples obtained during the vane shear testing. The average results per soil type are presented in Table 14 below. See Appendix I for a detailed table with each test result.

Table 14. Unconsolidated Undrained Triaxial Test Results per Soil Type

Soil Type	Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Shear Stress (psf)	Corrected Deviator Stress (psf)	Axial Strain, (%)
Silty Sand with Gravel (Bottom Ash)	107.3	83.3	5,678	3,011	10.9
Silt with Sand (Bottom Ash)	103.4	72.8	2,313	4,594	13.3
Fill Lean to Fat Clay with Sand	111.9	83.0	2,660	5,288	11.9
Residual Lean to Fat Clay with Sand and Gravel	123.5	99.0	1,520	3,011	14.9
Silty Sand (Fly Ash)	104.7	75.8	2,903	5,777	14.3

The results obtained from the vane shear testing were compared with the laboratory results from the unconsolidated undrained triaxial tests. In general, the unconsolidated undrained triaxial tests yielded higher shear strength values than the vane shear tests conducted in the field.

7.3.4. Falling Head Permeability Testing

Falling head permeability tests were performed on twelve undisturbed Shelby tube samples. After extrusion, each sample was mounted in a triaxial-type permeameter and tested in accordance with ASTM Standard D 5084, using distilled water. The results of the permeability tests can be found in Appendix F and are summarized below in Table 15.

Table 15. Summary of Falling Head Permeability Test Results

Boring Number	Soil Horizon	Test Interval (feet)	Sample Center Elevation (MSL)	Initial Conditions				Coefficient of Permeability K_v (cm/sec)
				Dry Density (pcf)	Moisture Content (%)	Void Ratio, (e)	Degree of Saturation (%)	
STN-67	Silty Sand (Fly Ash)	40.5-42.5	605.5	76.6	47.9	1.314	103.6	1.79×10^{-6}
STN-72	Silt with Sand (Bottom Ash)	18.0-18.5	618.4	105.0	17.1	0.587	77.7	1.70×10^{-6}
STN-78	Fill Clay	10.5-11.0	626.1	102.1	22.1	0.669	90.2	2.79×10^{-8}
STN-78	Residual Clay	36.5-37.0	600.1	104.0	22.1	0.614	96.9	1.94×10^{-8}
STN-80	Silt with Sand (Bottom Ash)	28.0-28.5	609.0	68.8	43	0.968	96.4	4.84×10^{-5}
STN-89	Residual Clay	40.0-42.0	595.3	116.2	16.7	0.462	98.4	1.78×10^{-7}
	Silt with Sand (Bottom Ash)		595.3	105.8	18.6	0.564	87.4	1.83×10^{-5}
STN-90	Fill Clay	10.0-10.5	603.0	98.0	26.0	0.727	96.9	1.29×10^{-8}
STN-96	Silt with Sand (Bottom Ash)	35.6-36.1	611.8	97.1	27.5	0.75	99.8	1.78×10^{-5}
STN-100	Residual Clay	49.5-50.0	594.4	86.4	33.9	0.937	97	1.06×10^{-5}
STN-101	Fill Clay	40.6-41.1	598.0	95.8	27.5	0.792	95.5	1.80×10^{-8}
STN-106	Silty Sand (Fly Ash)	50.5-51.0	595.2	73.3	41.8	0.943	101	7.05×10^{-6}

The average coefficient of permeability determined from the falling head permeability test for the Silt with Sand (Bottom Ash) is 2.16×10^{-5} cm/sec and for the Silty Sand (Fly Ash) is 4.42×10^{-6} cm/sec. For the Fill Clay and Residual Clay the average permeability is 1.96×10^{-8} cm/sec and 3.60×10^{-6} cm/sec, respectively. Due to the difficulty of obtaining an intact/undisturbed sample, no falling head permeability tests were conducted on the Shelby tube samples obtained from the Silty Sand with Gravel (Bottom Ash) soil horizon.

8. Engineering Analyses

8.1. General

The following geotechnical engineering analyses includes an evaluation of the soil strengths and permeability parameters for the five (5) predominant soil horizons, as well as, a seepage analyses and slope stability analyses for the as found slope conditions and a proposed final stack height in the active ash stacking area. Prior to beginning the seepage and slope stability analyses, twenty (20) initial cross sections were selected and the geometry of the existing embankment slopes and soil horizons were approximated using current and historical information. Once the geometry of the sections was determined, the geotechnical data for each section was reviewed and evaluated to determine the most critical reaches around the Ash Pond Complex. The criteria for selecting the critical sections were based on the following: the geometry/steepness of the section cut, height/location of the phreatic surface, subsurface soil conditions/horizon thickness, and historical information/existing geotechnical data. Based on this evaluation, ten (10) critical cross sections (Section A, C, D, H, J, L, M, O, S and T) were selected for seepage and slope stability analyses. Results of the analyses and evaluations are summarized in the following paragraphs. The results of the seepage and slope stability analyses are included in Appendix G and the plan locations for each cross section is identified on the geotechnical drawings included in Appendix C.

8.2. Soil Horizons

Based on the results of the drilling, laboratory testing, historical documentation, and drawings, the materials encountered during this exploration were divided into five primary soil layers. Please refer to the stability sections in Appendix G which depict the approximate soil breaks/horizons for each section. The soil layers identified on the cross sections are as follows:

- *Fill: Lean to Fat Clay with Sand*: This material represents the clay soils which were encountered within the clay dikes. Based on historical information the interior slopes for the clay starter dike were to be constructed at 2:1 and the exterior slopes were to be constructed at 3:1. However, all divider dikes were constructed at 2:1 side slopes.

- *Residual Lean to Fat Clay with Sand and Gravel*: This material represents the residual clay soils encountered below the clay starter dikes/sluiced ash materials. The residual clay ranged from sandy/gravelly lean clay to fat clay depending on the location of the soil sample with respect to the old Tennessee River and the Tennessee River.

- *Silty Sand with Gravel (Bottom Ash)*: This represents material encountered during the field exploration above the original starter dike which appears to have been used for interim/upstream raising of the dike or used for repairs when required. The material consists of a mixture of coarser bottom ash which has been mechanically excavated from the rim ditch and/or dredge cell and placed but not compacted. The material was then allowed to dry and consolidate under its own weight. Based on historical information the interior slopes were to be constructed at 2:1 and the exterior slopes were constructed at 3:1. It should be noted the stability sections constructed in Appendix G have been updated based on the boring information and topographic survey. Therefore, the interior/exterior outslopes and soil horizons depicted on the sections may differ from TVA's original design template.

- *Silt with Sand (Bottom Ash)*: This represents the hydraulically placed bottom ash that is contained by the original starter dike and coarse bottom ash lifts. It was primarily encountered upstream of the starter dike and below/upstream of the raised bottom ash dike.

- *Silty Sand (Fly Ash):* This horizon represents the lower/original sluiced materials which seem to be concentrated only within the old scrubber sludge pond and dredge cell area. The results of the exploration indicate this material is generally very loose and exhibits relatively low shear strength.

8.3. Seepage Analyses

8.3.1. SEEP/W Model

An analysis of steady state seepage through the perimeter embankment slopes was conducted by Stantec to evaluate the magnitude of potential seepage/piping of the fine grained soils within the embankment and to evaluate the potential build-up of pore water pressures which could trigger in-stability within the embankment slopes. As discussed above, eleven critical cross sections representing the most critical segments around the ash pond complex were modeled for the seepage analysis, then subsequently evaluated for slope stability (see Section 9.2.). The numerical seepage model for the Ash Pond Complex was developed using SEEP/W 2007 (Version 7.14), a finite element code tailored for modeling groundwater seepage problems in soil and rock. SEEP/W is distributed by GEO-SLOPE International, Ltd, of Calgary, Alberta, Canada (www.geo-slope.com).

For the numerical analysis, each cross section was subdivided into a five-foot minimum 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.

8.3.2. SEEP/W Boundary Conditions

Steady-state seepage was assumed for the analysis, with static water levels on both the upstream and downstream side of the embankments, where applicable. The SEEP/W boundary conditions (water/pool elevations) modeled for each section are presented below in Table 16.

Table 16. SEEP/W Boundary Conditions

Stability Section	Upper Source	Upper Water/Pool Elevation (feet)	Lower Source	Lower Water/Pool Elevation (feet)
Section A	Piezometer Reading (STN-63)	634.0	Lower Stilling Pond	611.5
Section C	Piezometer Reading (STN-65)	635.0	Tennessee River	594.5
Section D	Piezometer Reading (STN-100)	633.0	Widows Creek	594.5
Section H	Piezometer Reading (STN-106)	635.6	Main Ash Pond A	631.9
Section J	Main Ash Pond A	631.9	Widows Creek	594.5

Table 16. SEEP/W Boundary Conditions

Stability Section	Upper Source	Upper Water/Pool Elevation (feet)	Lower Source	Lower Water/Pool Elevation (feet)
Section L	Fly Ash Ditch	634.7	Red Water Pond	602.4
	Bottom Ash Ditch	633.3		
	Main Ash Pond A	631.9		
Section M	Fly Ash Ditch	635.0	Red Water Pond	602.4
	Bottom Ash Ditch	635.0		
Section O	Fly Ash Ditch	634.6	Tennessee River	594.5
Section S	Stilling Pond	611.5	Pump Pond	601.7
Section T	Pump Pond	601.7	Tennessee River	594.5

The phreatic water levels used in the SEEP/W analysis for the ponds described above were modeled as total head equal to the given pool elevation and/or piezometer reading. For sections where the pool limits were just beyond the cross section or the groundline was above recorded pond elevation, a total head vertical boundary line equal to the pond elevation was input into the model. For this scenario, the hydraulic head at each node was constant with depth and equal to the pool elevation on that side of the embankment. At other locations along the ground surface where potential seepage might occur, a total flux condition was modeled and potential seepage reviewed. The slurry wall was modeled as none permeable interface. Therefore, no flow was allowed to cross the structure. The horizontal boundary at the base of the model (located within the limestone bedrock) was modeled as a seepage barrier, with no vertical flow across the boundary nodes.

8.3.3. Seepage Properties

For each cross section analyzed, 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 and typical values for similar soils. Material properties used in the seepage analysis are summarized in Table 17.

Table 17. Seepage Parameters

Soil Horizon	Saturated K_V (cm/s)	Ratio k_h/k_v	Volumetric Water Content	
			Saturated (%)	Residual (%)
Silty Sand with Gravel (Bottom Ash)	2.16×10^{-5}	50	15.0	1.0
Silt with Sand (Bottom Ash)	2.16×10^{-5}	50	40.0	1.0
Fill Lean to Fat Clay	1.96×10^{-8}	25	25.0	3.0
Residual Lean to Fat Clay	3.60×10^{-6}	15	25.0	3.0
Silty Sand (Fly Ash)	4.42×10^{-6}	50	40.0	2.0

Significant engineering judgment is needed to select appropriate hydraulic properties for earth material. 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. For the Ash Pond Complex, an iterative parametric calibration was used to arrive at final seepage design parameters. The results from trial SEEP/W simulations were compared to field data (measured piezometric levels). The material parameters were then varied until the solutions reasonably matched the field data for the representative cross sections. The final set of parameters identified in Table 18 was a result of the piezometer calibration process presented in Section 8.3.4.

The ratio of horizontal hydraulic conductivity (k_h) to vertical hydraulic conductivity (k_v) was estimated based on the known depositional environment of the given material and slug test results within the various soil horizons. An isotropic material (sands and gravels) would have $k_h/k_v = 1$, while deposits of horizontally layered soils (fly ash, silt, bottom ash) might have values as high as $k_h/k_v = 100$. For the Ash Pond Complex, a ratio of 50 was assumed for the ash materials and a ratio of 25 was assumed for the fill clay. A more modest value of k_h/k_v = 15 was assumed for the residual lean to fat clay deposits.

8.3.4. Comparison to Field Observations

After the initial seepage parameters were estimated, results from the SEEP/W models were compared to the pore water pressures measured in the field piezometers installed along the corresponding cross section. Data from seventeen (17) piezometers were used in this evaluation. Nodes were placed in the model at the same location as the piezometer tip and then the total predicted head at the node was compared to the corresponding piezometer reading.

After reviewing/comparing the results for seventeen (17) piezometers, the material properties in each modeled cross section was varied (if necessary) until a reasonable match was obtained between the predicted SEEP/W phreatic elevation and the actual field piezometer readings. The comparison between the field piezometer readings and the predicted SEEP/W values at each piezometer are plotted in Figure 22. The maximum difference observed between the SEEP/W model and the field measured is 20.5 feet in piezometer STN-79 (Section L), 12.3 feet in piezometer STN-72 (Section J), and 7.4 feet in piezometer STN-66 (Section C). The remaining wells modeled for the analysis are within 5 feet of the predicted values. A difference of less than ten feet is typically acceptable in a seepage model, given the global differences between the homogenous cross section modeled and the actual heterogeneous soil conditions.

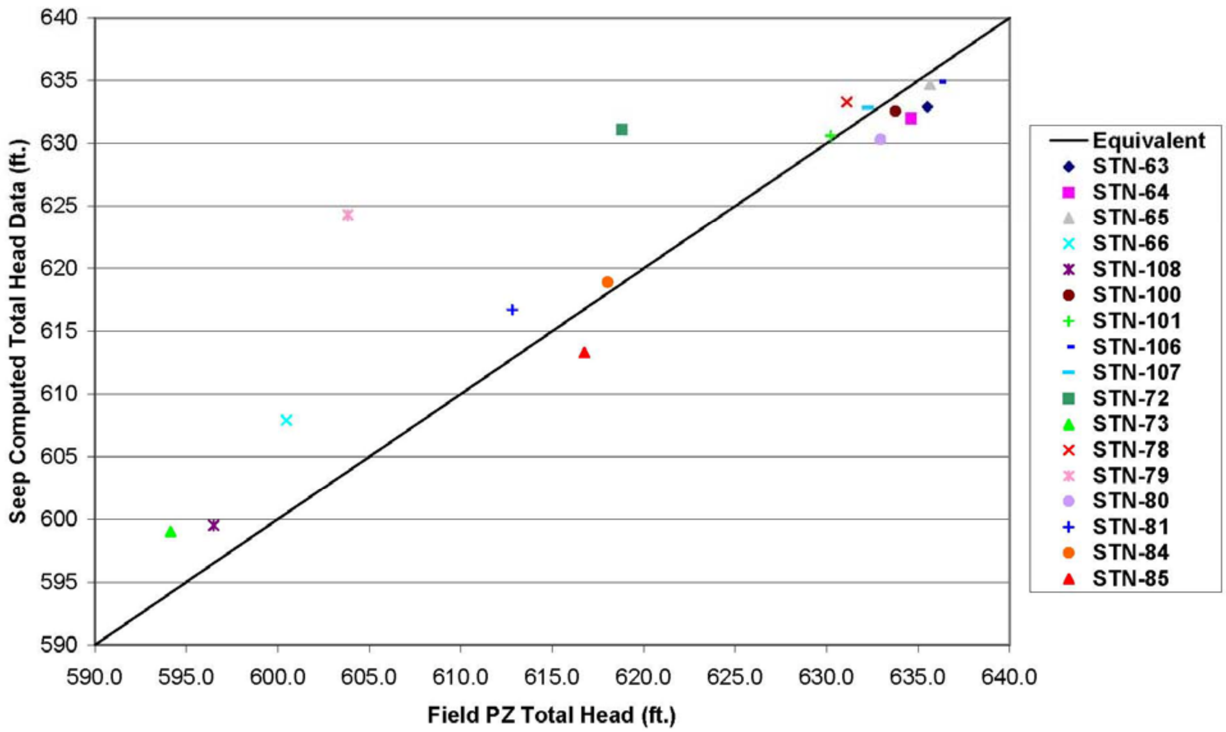

Figure 22. Comparison between the Field Piezometer Readings and Pore Water Pressures predicted in the SEEP/W Model

8.4. Strength Parameter Selection

The static stability of the Ash Pond Complex at the Widows Creek Fossil Plant was evaluated using the limit equilibrium slope stability methods. The soil parameters used in these stability analyses were established as follows. Please refer to Table 18 for a summary of the derived soil parameters.

The first ash disposal area (Area No. 1) was first constructed in 1952 with an estimated fill date of 1969. In 1959 a second ash disposal area (Area No. 2) was constructed and the initial crest elevation of 610 feet was reached in 1967. Then a 15-foot expansion was developed and Area No. 2 reached capacity in 1971. The northern dike was then breached and ash was allowed to be deposited into the "future disposal area", which is the current ash pond. Given this timeline and the soil types encountered at the site, it is assumed that excess pore pressures have dissipated and steady state seepage conditions have developed within the dike. In addition, the current analyses will focus only on static conditions (no earthquake or other dynamic loads). For these conditions, only soil unit weights and drained strength parameters (c' and ϕ') are needed. If stabilizing berms or other surcharging load modifications to the dike cross section are built, then undrained, total stress stability analyses will be needed to assess stability during construction of site improvements.

As described in Section 5.2 and 8.2 of this report, two (2) clay soil horizons were encountered in borings drilled at this site, the dike material and the foundation residual clay. According to historical information, it is believed that residual clay excavated from the interior of the pond is the source of the dike material (fill). Therefore, the properties of these two soils should be similar. According to classification testing performed on representative samples, the Plasticity Index was determined to be 31 and 23 for the clay/fill dike material and residual clay, respectively. Furthermore, based on in-situ testing (average SPT N-value of 18), both soil horizons have a stiff to very stiff consistency.

The cohesive soils sampled during the field exploration were also subjected to CU triaxial tests. The results of triaxial testing were evaluated and effective stress p' versus q scatter plots were prepared using all of the data points. Failure was assumed to occur at the point of the maximum effective principal stress ratio (σ_1'/σ_3'). Once the p' versus q plots were prepared; a failure envelope was then selected such that about two-thirds of the plotted values were above the envelope. The p' versus q plots and selection of the failure envelope are shown for each soil horizon on the graphs presented in Appendix I. The measured cohesion intercept was neglected (assumed c'=0) in evaluating the dike stability. The unit wet weight was determined by taking the average unit weight of the samples that are included within the soil horizon. These results were then compared to the historical laboratory test results.

A relationship between the plasticity index and peak friction angles for normally consolidated clays is shown in Table 18 (from Duncan and Wright, 2005). The results of the testing can be found in Appendix F of this report.

Table 18. Typical Values of Peak Friction Angle (ϕ') for Normally Consolidated Clays

Plasticity Index	ϕ' (deg)
10	33 ± 5
20	31 ± 5
30	29 ± 5
40	27 ± 5
60	24 ± 5
80	22 ± 5

When surficial soils have c' = 0, shallow sliding parallel to the ground surface will be the critical failure mechanism (lowest factor of safety) found in a slope stability analysis. However, apparent cohesion in unsaturated soils and/or weak cementation is often sufficient to prevent shallow sliding. This mode of failure, which might require periodic maintenance, is considered to be less critical in a stability analysis. For deep seated failures, the assumption of c' = 0 is routinely used for all soils.

The two Bottom Ash soils were primarily encountered above the residual clay soils and fill clay soil horizons. The type of bottom ash encountered, whether Silty Sand with Gravel or Silt with Sand was dependent on the location of sample. The coarser material was closer to the sluice line outfalls and internal dike expansions and the Silt with Sand (Bottom Ash) was encountered at lower depths within the ponds. Stantec performed CU triaxial tests on selected samples and the results were plotted on a scatter plot as described above. Again the unit wet weight was determined by taking the average unit weight of the samples included within the soil horizon. These values were compared to the values obtained from the SPT correlation tables. However, since no CU triaxial tests were conducted on the Silty Sand with Gravel (Bottom Ash), the phi angle determined for this soil horizon was determined from the SPT correlation tables.

The Silty Sand (Fly Ash) was located within the interior clay starter dikes and below the bottom ash materials described above. Again, a scatter plot was created based on the CU triaxial testing of samples from this soil horizon. The scatter plot yielded a phi angle of 35°. However, since this soil horizon had weak zones with blow counts of 0, the more conservative SPT correlation tables were used to determine the phi angle. The wet unit weight was determined from laboratory unit weight tests. The strength parameters selected for soil types described above are presented in Table 19.

Table 19. Selected Strength Parameters for Stability Analysis

Soil Horizon	Unit Weight (pcf)	Effective Stress Strength Parameters	
		c' (psf)	ϕ' (degrees)
Silty Sand with Gravel (Bottom Ash)	119	0	33.0
Silt with Sand (Bottom Ash)	112	0	33.0
Fill Lean to Fat Clay	125	0	32.0
Residual Lean to Fat Clay	125	0	32.0
Silty Sand (Fly Ash)	112	0	27.5

8.5. Slope Stability Analyses

Ten (10) stability sections were evaluated using SLOPE/W. All of the stability analyses presented in this report were analyzed for static, long-term conditions with steady-state flow parameters. In this study, steady-state pore pressures were obtained from the SEEP/W. The long-term analyses were evaluated using effective-stress, internal angle of friction, and zero cohesion parameters to simulate the condition which will exist long after the excess pore pressures have dissipated from dike raising/surcharging. The unit weight and shear strength parameters used in the stability analyses were selected based on the geotechnical information and laboratory testing results described herein.

9. Results

9.1. Seepage Exit Gradients for As Found Conditions

Contour plots of the hydraulic gradients computed from the SEEP/W solutions are shown for each modeled cross section in Appendix G. Large gradients and significant seepage can be seen at various locations within the cross sections, but the concern is for areas where these gradients can initiate the erosion or piping of material. In general, areas of potential concern are where water seeps laterally out onto a sloping ground surface, or where vertical, upward seepage occurs at the ground surface. Away from the ground surface, the potential movement of material due to seepage forces is arrested by the adjacent soil. Hence, the evaluation of seepage gradients at the Ash Pond Complex is focused on areas near the ground surface on the downstream side of the dike.

The potential for piping due to vertical seepage to the ground surface was evaluated using the same target factors of safety defined in the Phase 2 Gypsum Stack Geotechnical Report which was submitted to TVA on January 14, 2010. Stantec first reviewed the contour plots with respect to the vertical exit gradients (see Appendix G) to determine the general location of the maximum vertical exit gradient. Then for the critical seepage areas an average vertical gradient was determined over a depth of 3 to 5 feet just below the ground surface. This way, when the model geometry converged to a sharp point (as it normally does), the high exit gradients within this small zone (which is not reflective of the actual conditions in the field) was ignored. For the eleven critical cross sections analyzed, the maximum upward gradient occurs at or near the toe of the embankment.

The factors of safety against piping were computed based on the exit gradient results from SEEP/W and the critical gradients determined from the soil properties. The results of the computed exit gradients and factors of safety against piping are summarized in Table 19. The lowest computed factor of safety is 1.0 at Section O. As with the Gypsum Stack report, the United States Army Corps of Engineers (USACE) design criteria (EM 1110-2-1901) was used to select an acceptable factor of safety against piping ($FS_{piping} \geq 3$). Six of ten cross sections evaluated do not meet the design criteria for piping at the seepage exits.

Table 20. Summary of Computed Exit Gradients and Factors of Safety against Piping

Section	Vertical Gradient (i_v) at Critical Exit Point	Location of Critical Exit Point	Soil Horizon	Critical Gradient (i_{crit})	FS_{piping}
Section A	0.24-0.35	Toe of Embankment and Stilling Pond	Residual Clay	1.00	3.0
Section C	0.14-0.23	Lower Dyke Toe - Perimeter Ditch	Residual Clay	1.00	4.4
Section D	0.38-0.60	Toe of Dredge Cell Pond	Residual Clay	1.00	1.7
Section H	0.06-0.10	Toe of Embankment at Main Ash Pond A	Silty Sand with Gravel (Bottom Ash)	0.81	8.1
Section J	0.42-0.76	Lower Dyke Toe & Foundation Soil Interface	Fill Clay	1.00	1.3
Section L	0.46-0.54	Toe of Embankment at Red Water Pond	Fill Clay	1.00	1.8
Section M	0.85-1.04	Toe of Embankment at Red Water Pond	Fill Clay	1.02	1.0
Section O	0.21-1.05	Perimeter/ Roadway Ditch	Fill Clay	1.02	1.0
Section S	0.23-0.33	Toe of Embankment in Pump Pond	Fill Clay	1.02	3.0
Section T	0.60-0.67	Toe of Embankment in TN River	Residual Clay	1.00	1.5

9.2. Slope Stability Results for As Found Conditions

Using the strength parameters selected (c’ and ϕ’), in conjunction with the results of the seepage analyses, the existing Ash Pond Complex configuration was analyzed at Section A, C, D, H, J, L, M, O, S and T for a rotational failure which was optimized by SLOPE/W software. The pore pressure from the SEEP/W analysis was imported into the SLOPE/W files. Factors of safety computed by this program for rotational failures are based on the Spencer’s method of slices. The rotational failure with the lowest factor of safety is then optimized by the software to determine if slight variations in the failure surface result in a lower factor of safety. Each section was analyzed for a deep seated global failure using the grid and radius method which was allowed to fail through any of the above mentioned soil horizons. The grid and radius method allows the user to specify a grid to analyze for the center of the failure planes along with a range of tangent lines for the radius.

There was no indication in the slope stability analyses that a noncircular failure surface would give a factor of safety lower than obtained for the optimized circular surfaces. Overall, the geometry of the dike cross section 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 segments along the curved slip surfaces. The results in Table 21 and Appendix G represent factors of safety computed from the optimized, circular slip surface routine.

The Alabama Department of Environmental Management (ADEM) does not specifically address target factors of safety for slope stability for this type of structure. Based on discussions with TVA and to be in accordance with current prevailing practice a minimum factor of safety of 1.5 was established for long term conditions using the guidelines presented in the US Army Corps of Engineers Engineering Manual EM 1110-2-1902, "Slope Stability".

Table 21. Stability Analysis Results for As Found Conditions

Stability Section[1]	Station	Computed Factor of Safety (Global Failure)
A	8+98	1.4[3]
C	24+35	1.6[3]
D	31+01	1.3[3]
H	104+44	3.3[3]
J	67+36	1.2[3]
L	106+22	1.3[3]
M	116+72	1.7[3]
O	139+00	2.1[3]
S	181+23	1.2[2]
T	181+22	2.3[3]

(1) Refer to Appendix C for plan view of site with project baseline.

(2) Grid and Radius, Auto Search (DMIN = 20')

(3) Grid and Radius, Auto Search (DMIN = 25')

Based on the results of the analysis, Sections A, D, J, L and S failed to meet the minimum required factor of safety of 1.5 for the grid and radius method. However, all of the sections have a factor of safety greater than one. For these sections which did not meet the minimum required factor of safety, slope mitigation (e.g., rock toe buttress) will be required to improve the factor of safety against a slope failure. The critical slip surfaces are depicted in Appendix G. The stability and seepage results for the Main Ash Pond A dam (Section R) will be provided with the spillway design package which is currently being developed.

It should be noted that past seepage and shallow sliding (infinite slope failures) and/or creep movement has been observed along the perimeter slopes of the Bottom Ash Stack near the Chemical Ponds. Reportedly, this area has had historical problems due to seepage, as noted by The slurry cutoff wall installed in 1984 and the Scrubber Road ditch regrade performed by TVA in 2008 and 2009. Since the above repairs, this area continues to show signs of seepage and relatively shallow, non-global slope movement on the order of five feet or less in depth as indicated by the slope inclinometer data traces. A work plan (Work Plan

No. 7) is currently under development to control the observed seepage using a graded media filter and rock toe buttress, which will increase the stability of slope and control drainage of the area.

9.3. Slope Stability Results for Planned Final Stack Conditions

As directed by TVA, the northern perimeter dikes of Main Ash Pond A and the perimeter dikes of the Bottom Ash Stack areas were evaluated for slope stability based on assumed projected five-year build out site conditions. The final height of the stack was extrapolated based on production rates and slope geometries provided on a three-year stacking plan supplied by Trans Ash dated January 20, 2009. A production rate of 450,000 cubic yards per year was assumed along with 3:1 design side slopes above the 636 perimeter road elevation. Based on the production rate stated above and the provided design configuration, the projected five-year stack level would reach elevation 658 feet. The above design concept was then modeled in AutoCAD Civil 3D along with the original Main Ash Pond A basemap dated February 4, 2008 provided by TVA. Three critical cross sections (L, M, and O) were then selected and analyzed with SEEP/W and SLOPE/W. The results of the five year build out analysis are presented in Table 22 below.

Table 22. Stability Analysis Results for Planned Final Stack Conditions

Stability Section	Station[1]	Target Factor of Safety	Computed Factor of Safety (Global Failure[2])	Upper Pond Modeled	Upper Pond Elevation
Section L	106+22	1.5	1.3	Fly Ash Pond	631.9
Section M	116+72	1.5	1.7	Fly Ash Ditch	635.0
Section O	139+00	1.5	2.1	Fly Ash Ditch	634.6

(1) Refer to Appendix C for plan view of site with project baseline.

(2) Grid and Radius, Auto Search (D_{MIN} = 25)

Based on the planned stack configuration, Section L does not meet the minimum required factor of safety for the five-year build out. However, Section L did not meet the required factor of safety for the existing condition, either. Based on a preliminary evaluation of possible mitigation measures, it was determined that the same remediation measures used to treat the existing slope geometry (see Appendix J) will also improve the factor of safety for the five-year build out to meet the design criteria of 1.5. Therefore, only one interim risk reduction design will be recommended for this area. The possible mitigation measures are discussed in detail in the last section of this report.

10. Limitations of Study

The scope of this evaluation was limited to considering only the potential risks at the Ash Pond Complex due to excessive seepage and slope instability. This assessment did not consider potential failure modes related to spillway capacity and overtopping, seepage along penetrations through the embankment (including the buried spillway pipes), erosion due to wave action or flood stage flows, vegetation on the dike face, performance of the internal divider dikes, or other possible mechanisms. The risks associated with these potential failure modes will be addressed in a final comprehensive facility assessment that is currently underway.

The stability of the Ash Pond Complex during a potential earthquake was not specifically analyzed. Data from the site explorations indicate low penetration resistance (low density) in the saturated Silty Sand (Fly Ash) material. In a strong earthquake, these soils will be prone to liquefaction, which would undermine the stability of the ponds. However, the seismic risk at this site (likelihood of experiencing a large magnitude earthquake) is quite low for the remaining life.

Stability analyses were not performed for rapid drawdown conditions within the Stilling Ponds, Pump Pond, Chemical Ponds, or Redwater Pond. A slope failure due to rapid drawdown could result in an upslope failure of an adjoining pond due to a breach in the dike or failure of the spillway. While the upstream dike slope may be vulnerable to sliding due to rapid drawdown, this mechanism would not likely result in a breach or global failure of the pond. Therefore, this failure scenario was not evaluated.

11. Conclusions

The conclusions and recommendations that follow are based on the available historical documents provided by TVA, discussions with TVA personnel throughout the course of this work, and the results obtained from our geotechnical exploration and stability analysis.

A general topographic review of the available 1973 drawings (TVA 10N7421 series drawings) and the updated 2008 mapping indicate the exterior slopes below the 636 level at the Main Ash Pond A, the 639 level at the Bottom Ash Stack, and the 646 level at the Dredge Cell are on the order of 3:1 and above the bench levels they are approximately 3:1 or flatter, where applicable. Therefore, based on Stantec's review, the slope conditions appear to follow the intended design slopes.

The results from the seepage analyses were examined to identify conditions where piping and erosion of soil might develop due to seepage forces. The model results indicated a shallow phreatic surface (ground water table) along the out slope of the dike which is concentrated at the toe of the slope. The results of the analysis are confirmed by the seepage problems observed along the Old Scrubber Road (Section O). Out of the ten (10) cross sections modeled, six (6) indicate high exit gradients at the toe of the embankment slopes. This condition creates the potential for the initiation of soil piping, as seepage water will tend to erode material from within the dike. Upward, vertical exit gradients in the area of the toe were also found to be excessive. Factors of safety against piping, computed for the surficial 3 to 5 feet of soil in these areas, ranged from 1.0 to 8.1. Based on USACE design criteria (EM 1110-2-1901), factors of safety against piping should be no less than three. The results from the seepage model thus demonstrate that the majority of the Ash Pond Complex does not meet the required factor of safety for preventing soil piping due to seepage.

Current criteria for the long-term stability require a factor of safety for slope stability of at least 1.5. The slope stability results for the as found stack height show the Ash Pond Complex does not meet the minimum value required for five of the ten sections modeled. Sections A, D, J, L and S have an existing factor of safety ranging from 1.2 to 1.4. As a result, interim risk reduction measures will be recommended within the areas represented by these cross sections.

Based on the stability analysis results for the final dry ash stack height, the calculated minimum factor of safety is 1.3, with factors of safety ranging from 1.3 to 2.1. Section L does not meet the required factor of safety of 1.5 and mitigation measures will be required to improve the stability within the north segment of Main Ash Pond A. Therefore, TVA should undertake specific efforts to improve the safety of the stack within these areas and reduce the interim risks during the remaining period of operation. The following actions are recommended.

12. Recommendations

12.1. Seepage and Stability Improvements

It is recommended that the slope stability/seepage mitigation plans address each geotechnical and hydraulic deficiency identified in Section 9. Based on the stability analysis results of the as-found conditions, five of the ten stability sections and one of the three stability sections for the final stack height does not meet the minimum factor of safety of 1.5. This is primarily due to a relatively high phreatic surface. In light of not having an underdrain system, TVA should initiate mitigation design and construction plans to improve the long-term stability of the stack. The mitigation project should involve the placement of stabilizing berms on the downstream slope of the clay starter dikes.

Consistent with USACE design criteria, the berm dimensions should be selected to obtain factors of safety greater than 1.5 for sliding under long-term, drained conditions. For the period during and immediately after such construction, undrained stability analyses will be needed to demonstrate a factor of safety of at least 1.3 for short-term conditions.

The berm should also be designed to provide protection against seepage and soil piping failures, and increase the factor of safety against piping to exceed the value of 3. Where the berm is built over areas subject high exit seepage gradients (i.e., at the toe of stacks and downstream slope and toe of the clay dikes), the gradation of the berm (graded media filter) should be selected to prevent the loss of embankment materials.

Based on the results from the slope inclinometer data, it is recommended that any slope repair (due to shallow surface creep) be extended to a minimum depth of five feet and the slope inclinometers continue to be monitored until a interim risk reduction measure can be implemented in this area.

Sketches of the conceptual design modifications for each segment are presented in Appendix J. The improvements shown on the typical cross sections result in a minimum factor of safety against sliding $FS_{sliding} \geq 1.5$ and $FS_{piping} \geq 3.0$. The cross section templates will require design modifications as it is applied around the perimeter in the detailed design phase of work.

12.2. Dredge Cell

Stantec recommends closing the inactive Dredge Cell. The pond is no longer in use, presents an unacceptable risk of dike failure to TVA, periodically has fugitive dust issues and the remaining storage capacity is limited. Reportedly, the height of the inactive Dredge Cell is close to reaching the minimum clearance for operations near the existing high-voltage (500kV) power lines.

A closure plan should be designed and constructed with storm water management elements to readily control and direct surface water runoff from the cap to the Main Ash Pond A. Design features should include:

- Cap grading improvements that prohibits infiltration and conveys surface runoff to armored ditches;
- Improvements to the roadside perimeter ditches which have existing slopes near zero percent along the east, west and south sides;
- Protected piezometer installations at selected locations for long term monitoring use.
- Vegetative cover that prevents fugitive dust with side slopes that can accessed by inspectors and maintained by readily-available mowing equipment; and
- Vehicle access ramps and roads of sufficient width to the top and perimeter.
- Signage in accordance with TVA standards.
- Best management practices for erosion control.

As these improvements are implemented, the phreatic surface should lower resulting in improved slope stability and improved seepage conditions. It is recommended that TVA start the detailed closure design process immediately, continue to monitor instrumentation, and continue to routinely inspect for evidence of seepage and sand boils using trained and qualified personnel.

12.3. Bottom Ash Stack

Based on Stantec's Phase 1 field observations, evidence of excessive seepage is present and causing shallow slope instability issues in the Bottom Ash Stack area along the Scrubber Road (Section O). Mitigation plans currently under design will address the specific seepage issues observed along the southern perimeter of the Bottom Ash Stack. TVA should implement these plans to improve the safety of the stack and reduce maintenance costs.

12.4. Active Ash Stacking Areas

The active ash stacking areas within the Main Ash Pond A and Bottom Ash Stack should be operated in accordance with an updated Operations and Maintenance (O&M) manual and updated Stacking Plans. Stacking Plans should be designed using current site condition using new topographic and hydrographic mapping and based on a planned fill date of January, 2015 at which time the waste disposal operations will cease at the existing facilities. The plans should also be developed in coordination with ongoing stability mitigation plans, inactive Dredge Cell closure, NPDES environmental compliance and spillway replacement projects.

12.5. Spillways

It is recommended that existing spillways with tall, unsupported (a.k.a., bell-mouthed or morning glory) risers be replaced in accordance with Phase 1 Facility Assessment recommendations. Capital improvement projects at each of the three existing spillways located in the Ash Pond Complex are in the planning phase of work. The design reports for these projects will be issued under a separate cover. The stability and seepage results for Section R will be submitted with the Main Ash Pond A spillway replacement design report.

12.6. Additional Geotechnical Drilling and Instrumentation

In an effort to characterize soil conditions of the Main Ash Pond A dam, Lower Stilling Pond dam and Pump Pond dam, additional sample borings and piezometers will be advanced as part of spillway replacement projects. The information gained from this additional drilling will be incorporated into the ongoing spillway foundation design efforts for Work Plan Nos. 11 and 15.

The original piezometers identified as STN-99, STN-69, and STN-94 have been damaged and will be replaced. Therefore, it is recommended that three additional piezometers be installed at these locations such that the instrumentation system remains intact and can continue to be monitored.

Based on the results of the geotechnical drilling, testing and slope stability analysis, it is recommended that four (4) additional driven piezometers be installed at the mid slope along critical section L, M, J and C. The additional borings will help to better define the actual phreatic surface at these mid-slope locations. Upon TVA management approval, the above mentioned drilling is planned as part of the current field work and will be completed in the first quarter of 2010. After additional drilling has been completed, Stantec will reevaluate the as found conditions for Sections M, L, J and C. The results will be provided as an addendum to the geotechnical report.

13. Closure

The scope of Stantec's services did not include an environmental assessment or investigation for the presence or absence of wetlands and hazardous or toxic materials in the soil, surface water, groundwater or air, on below or around the site. Any statements in this report or on the boring logs regarding odors noted or unusual or suspicious items or conditions observed are strictly for the information of the client.

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. 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.

It should be noted that construction records indicating the methods used to construct the Ash Pond Complex, as-built configurations, etc. were not available for review. In addition, the variable nature of the historical and current data shows some signs of inconsistencies in the construction of the dikes. As a result, consideration should be given to some of the generalizations made in this report with regards to dike construction and geometry prior to using this data in future evaluations.

The analyses presented herein represent a general evaluation of the subject structures. The computed factors of safety against slope stability failure are applicable only for the particular geometries and conditions evaluated and presented herein. Variations in the height of the as-found and predicted final perimeter dikes, natural ground elevation, elevation of underdrains relative to the perimeter dike can significantly alter the results of these analyses and change the conclusions presented in this report. A detailed design of the mitigation plans, operational stacking plans, closure and post-closure plans is still required. The various elements of the graded media filter and rock toe berm should also be tested to confirm mechanical and chemical compatibility with Widows Creek gypsum-fly ash and process waters.

The borings drilled for this geotechnical exploration did not encounter any signs of karst activity (voids, vugs, significant changes in the bedrock surface elevation, soft foundation soils) which may indicate the presence of karst features. The available geological mapping did point out the majority of the site to be underlain by the Sequatchie, Nashville, and Stones River Group Formation which does contain fossiliferous limestone. Therefore, TVA needs to be aware that karst features (voids, sinkholes, solution channels, etc) could develop at the project site.

The bedrock conditions observed during the exploration are not unlike the majority of Jackson County, Alabama where developments have been successfully constructed on similar conditions. Construction in limestone areas is accompanied by some risk that internal soil erosion and ground subsidence could affect existing/new structures in the future. Furthermore, it is impossible to completely investigate a site to eliminate all possibilities of future karst related problems. However, Stantec believes that compliance with good construction practices and guidance from a professional engineer experienced with karst development can reduce these risks to acceptable levels when developing structures (building, earthen dams, etc.) in this type of bedrock lithology.

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Appendix A

Historic Documents

Appendix B

Boring Logs and Piezometer Installation Records

Appendix C

Geotechnical Drawings

Appendix D

SPT Correlation Tables

Appendix E

Slope Inclinometer Results

Appendix F

Results of Laboratory Testing

Appendix G

Results of Stability Analysis for As Found Conditions

Appendix H

Piezometer Readings

Appendix I

Laboratory Summary Tables

Appendix J

Recommendations – Plan and Cross Sections

Appendix K

Results of Stability Analysis for Planned Final Stack Conditions